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Seismic Design of Bolted Steel Moment-Resisting Frames

by

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This report discusses some issues related to seismic behavior of various types of steel moment-resisting frames used in building structures. However, the emphasis of the report is on the seismic behavior and design of steel moment-resisting frames with bolted beam-to-column connections. A summary of relevant research and applicable code provisions is provided followed by design procedures that can be used to design steel moment-resisting frames. The appendices to the report provide typical details of bolted moment connections, a numerical example and photographs of structures designed and constructed recently using bolted steel moment-resisting frames.

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This work is dedicated to the memory of Professor Frank Baron (1914-1994) of the University of California, Berkeley who was one of the pioneer teachers and researchers in comparative studies of rivets and high-strength bolts subjected to fatigue.

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The opinions expressed in this report are solely those of the author and do not necessarily reflect the views of the University of California, Berkeley where the author is a professor of civil engineering, the Structural Steel Educational Council or other agencies and individuals whose names appear in this report.

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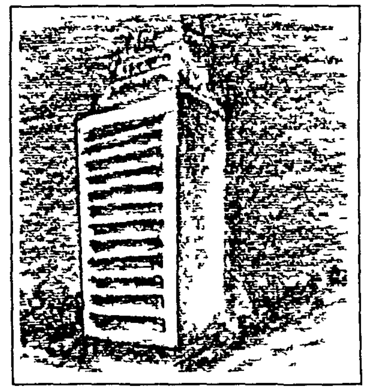
CONTENTS

1. INTRODUCTION



1.1. Introduction

Moment-resisting frames (MRFs) are structures that resist applied forces primarily by bending of their members and connections. MRFs can provide large open spaces without the obstruction usually caused by braces or shear walls. In addition, because of their flexibility and relatively long period of vibration, MRFs usually attract smaller seismic forces than the comparable braced or shear wall systems.

Since the early days of riveting, steel MRFs have been very popular in building construction. Many structures including the monumental high-rises of the late nineteen and early twentieth centuries have been built using riveted steel MRFs. On the west coast, many turn-of-the-century tall buildings in San Francisco have riveted steel MRFs. Since the 1960's, with the advent of high-strength bolting as well as welding technologies, bolted steel moment-resisting frames (BMRFs) and welded steel moment-resisting frames (WMRFs) have been one of the main structural systems used in office and residential buildings.

In recent years because of ease of fabrication and design and for economical reasons, most of the steel moment-resisting frames used in seismic areas such as California have had welded moment connections. However, welded steel moment-resisting frames are only one of the many possibilities of steel moment frames.

The main purpose of this report is to present information on the seismic design of steel rigid moment-resisting frames with *bolted* or *bolted/welded* connections. Today, there is sufficient information and experience that bolted and bolted/welded steel moment-resisting frames can be designed and fabricated to provide safe and economical structural systems for seismic regions.

1.2. Types of Steel Moment-Resisting Frames

Steel moment-resisting frames can be divided into several categories on the basis of (a) configuration of the moment frame, (b) the type of connectors used, (c) the ductility of the connection, (d) the relative rotational stiffness of the connection and the members, and (e) the relative moment capacity of the connections and the members. The common categories of steel moment frames are shown in Figure 1.1. The chart in Figure 1.1 can be used to select a desirable combination of frame attributes. The emphasis of this report is on bolted, special, rigid frames the design of which is based on the strong-column, weak-beam concept. The frames are highlighted in Figure 1.1.

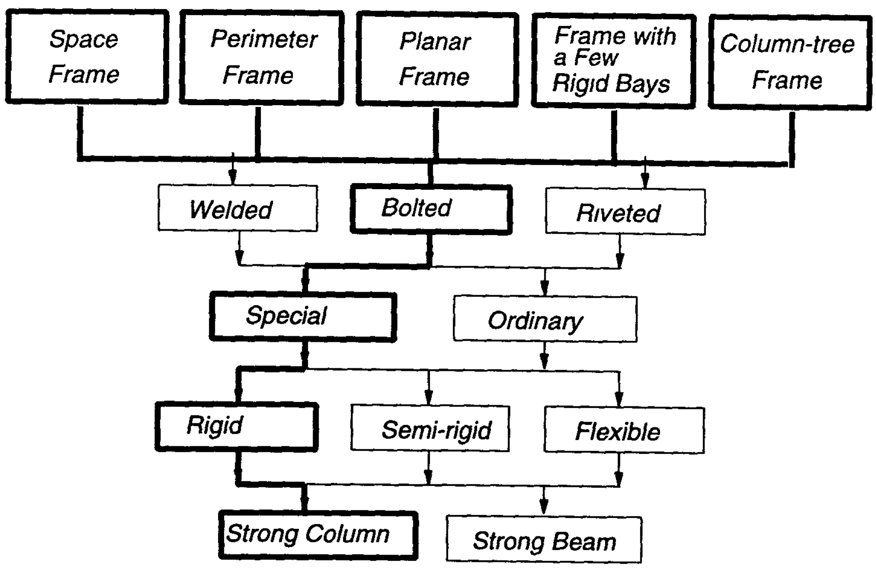


Figure 1.1. Selection Chart for Steel Moment-resisting Frames

1.3. Categories of Moment-Resisting Frames Based on Configuration

Common categories of MRFs are:

- Space moment-resisting frame
- Full perimeter moment-resisting frame
- Planar moment-resisting frame in one direction
- Moment-resisting frame in only a few bays
- Column-tree moment-resisting frame
- Moment-resisting frame with truss girders
- Moment-resisting frame with Vierendeel girders
- Tube-in-tube moment-resisting frame
- Bundled tube moment-resisting frame

The above configurations are discussed in the following sections.

1.3.a. Space, Perimeter and Moment-Resisting Frames in Only a Few Bays

A typical *space* MRF is shown in Figure 1.2(a) where a three-directional structural system composed of columns, girders and connections resist the applied load primarily by the flexural stiffness, strength and ductility of its members and connections, with or without the aid of the horizontal diaphragms or floor bracing systems (ICBO, 1994). In today's welded space frames, usually all girder-to-column connections are designed and fabricated as rigid.

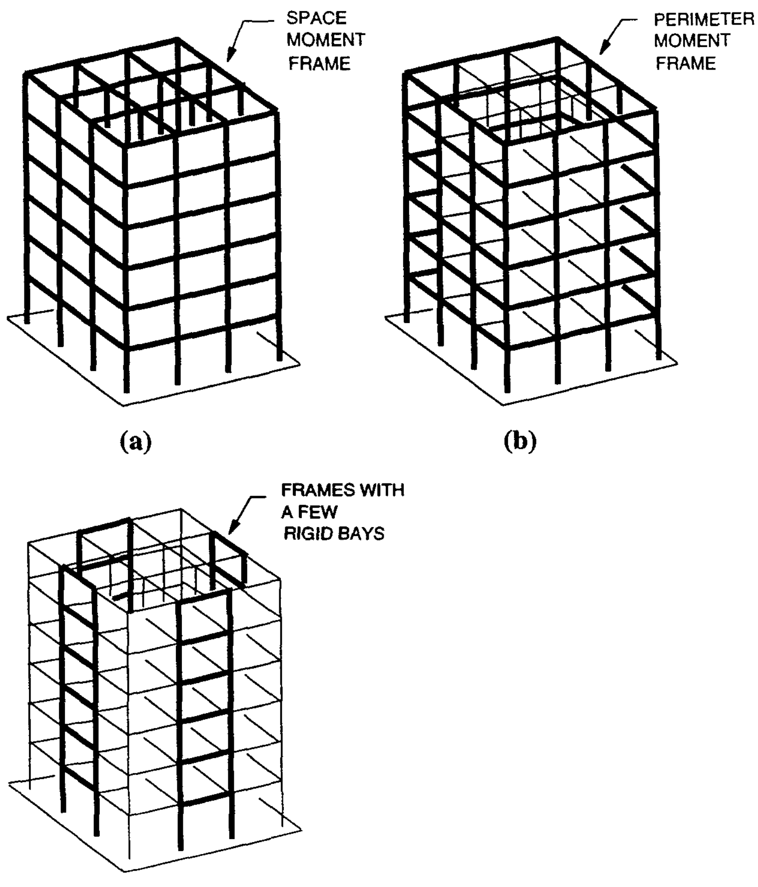


Figure 1.2. Space Frame, Perimeter Frame and a Structure with Only a Few Rigid Bays

The cost of fabrication and erection of rigid moment connections, particularly field-welded connections, is usually higher than the cost of fabrication of shear connections. As a result, to achieve more economical designs, there has been a trend in the United States in recent years to use a smaller number of moment connections in a given structure. This trend may have been the reason for the design and construction of many steel structures in recent years with only a few bays designed as moment-resisting frames.

In a perimeter MRF system, as shown in Figure 1.2(b), only the exterior frames are moment-resisting frames providing a moment-resisting frame box to resist the lateral load of the entire building. The interior columns and girders that are not part of the perimeter moment-resisting frame are all connected by shear (simple) connections to carry only their tributary gravity loads.

The columns inside a perimeter moment-resisting frame are often called "leaner" or "gravity" columns. In current design practice, it is often assumed that gravity columns do not participate in resisting the lateral loads. However, during an earthquake, the gravity columns, girders and their connections that were assumed not to participate in lateral-load resisting will, in fact, do so to some extent. In addition, the floor diaphragms and some non-structural elements also provide unknown amounts of stiffness, strength and damping. This is due to the fact that during earthquakes, the entire building is shaken and all members and connections undergo deformations and rotations. This issue has been recognized by the codes. For example, the Uniform Building Code (ICBO, 1994) requires that shear connections of leaning columns be designed to accommodate deformations (rotations) imposed on them by lateral displacement of the moment frames.

By using steel perimeter MRFs instead of space MRFs, the number of rigid moment connections is reduced, in many cases, to less than one half of the number of connections in the comparable space frame. As a result, significant cost saving is achieved. However, in doing so the redundancy of the lateral-load resisting system is also reduced.

The importance of the redundancy and the secondary load path in improving seismic performance of structures is intuitively accepted by structural engineers. However, no systematic study has been published yet to show the effect redundancy on performance of moment-resisting frames quantitatively. Until such studies are done, probably the effects of redundancy on seismic behavior will correctly remain in the domain of the intuitive feeling and professional judgment of the structural engineer in charge of the seismic design.

According to data collected by Youssef et al, (1995), in the aftermath of the Northridge earthquake, damage to space MRFs was apparently less than damage to perimeter MRFs. At this time, however, there is not sufficient data to discard the less redundant steel perimeter moment-resisting frame system. One of the advantages of the perimeter moment-resisting frame system is that the girder spans of the perimeter frames can be made quite small. The close spacing of the columns in perimeter moment-resisting frames can compensate to some degree for the loss of some redundancy as well as enable the perimeter moment-resisting frame to act as a tube structural system.

Another type of steel MRF system that has been used frequently in recent years in southern California is frame with only a few moment-resisting bays as shown in Figure 1.2(c). In this system only a few bays of the entire planar frame have rigid connections while all other connections are shear connections. The columns that are not part of the moment-resisting frame, are leaner (gravity) columns and are not considered in design to participate in resisting lateral load.

Information on the actual behavior and design of frames with only a few rigid bays was very limited and almost non-existent prior to the 1994 Northridge earthquake. Egelkirk (1993) provides some information on seismic design of steel MRFs with a few rigid bays.

A large percentage of the steel structures damaged during the 1994 Northridge earthquake had this structural system. At this time (May 1995), the exact cause(s) of the damage to welded steel moment-resisting frames during the Northridge earthquake has not been established. Therefore, it is not clear if the use of moment-resisting frames with only a few rigid bays was a major parameter contributing to the damage.

In MRFs with only a few rigid bays to resist lateral forces, the members and connections of the rigid bays become extraordinarily large. As a result, it is possible that the large members (jumbo shapes) connected by very large size welds could not behave in a ductile manner. However, adding to the complexity of the Northridge damage is the fact that many of the buildings that developed weld cracks had small and medium-weight sections and not very heavy Jumbo shapes.

1.3.b. Significance of Gravity Load Acting on Lateral-Load Resisting Frames

One of the important issues in seismic behavior and design of steel MRFs is how significant are the gravity load effects compared to the seismic effects. This can be measured by a "mass ratio" parameter, γ, defined here as:

$$\gamma = \frac{W}{Mg} \tag{1.1}$$

where W is the weight tributary to the moment-resisting frame, M is the horizontal mass tributary to the moment frame under consideration and g is the acceleration of the gravity.

The "mass ratio" as defined by Equation 1.1 can be a useful tool in identifying how much of the gravity-load carrying system is also responsible for carrying seismic loads. In space moment-resisting frames, almost all elements of the frame are responsible for carrying their own tributary gravity and seismic load, whereas, in perimeter moment-resisting frames and in moment frames with a few rigid bays, only a portion of the gravity-load carrying system is involved in carrying lateral loads.

For space MRFs, the mass ratio, γ, is about 1.0 meaning that members and connections of space MRFs are responsible for carrying only their own share of the gravity and seismic forces. In other words, the entire gravity-load carrying system of the space moment-resisting frame participates in resisting the lateral loads. For comparison, in the common perimeter moment-resisting frame the mass ratio is about 1/2 to 1/3. For MRFs with only a few rigid moment-resisting bays, in some of the existing structures in Los Angeles the mass ratio is as low as 1/6 meaning that only 1/6 of the gravity load carrying members are participating in carrying seismic lateral loads.

Since the gravity-load carrying system is needed after an earthquake to carry the service gravity load and to prevent collapse, by using the above definition of mass ratio, two interesting questions arise:

1. Is it better to use only a portion of the gravity-load carrying system to carry the seismic load, as in frames with a few rigid bays and perimeter moment-resisting frames? or is it better to use all members of the structure to carry the seismic load, as is the case for space moment-resisting frames?

2. Considering the fact that in the aftermath of a very strong earthquake, the lateral-load resisting systems of many structures can be damaged, is it a sound design philosophy to construct space MRFs and end up with the entire gravity-load carrying system damaged during the earthquake? Or is it better to have a few bays as rigid moment-resisting bays to resist the lateral load? If these few rigid bays are damaged, at least the remaining gravity load carrying elements are not affected and can carry their gravity load safely. In addition, such gravity-load carrying columns and girders usually act as a semi-rigid frame and a secondary load path for lateral-load resistance.

Without comprehensive technical and cost-efficiency studies, at this time there are no definite answers to the above questions. In addition, since there is no solid research data on comparative seismic performance of space MRFs, perimeter MRFs and frames with a few rigid bays, none of the three systems can be condemned as not suitable for seismic applications. The decision to use any of the above systems (or other systems not mentioned above) is left properly by the profession to the judgment of the structural engineers. After the decision is made about what system to use, the system has to be designed to have sufficient stiffness, strength and ductility to perform safely and according to the governing performance criteria. In all of the design steps, inevitably, economical considerations play a major role.

1.3.c. Column-tree Moment Frames

An example of a "column tree" moment-resisting frame system is shown in Figure 1.3. In a column-tree system short segments of the girders, usually one to two feet long, are welded to the columns in the shop. Then, after the column-trees are erected in the field, the middle segment of the girder is usually bolted to the ends of short girder stubs. Therefore, the system is a shop-welded, field-bolted steel structure. The shop welding provides for high quality and economical welding as well as easy inspection. The field bolting results in the economy and ease of field erection as well as the possibility of year-round construction almost independent of weather conditions.

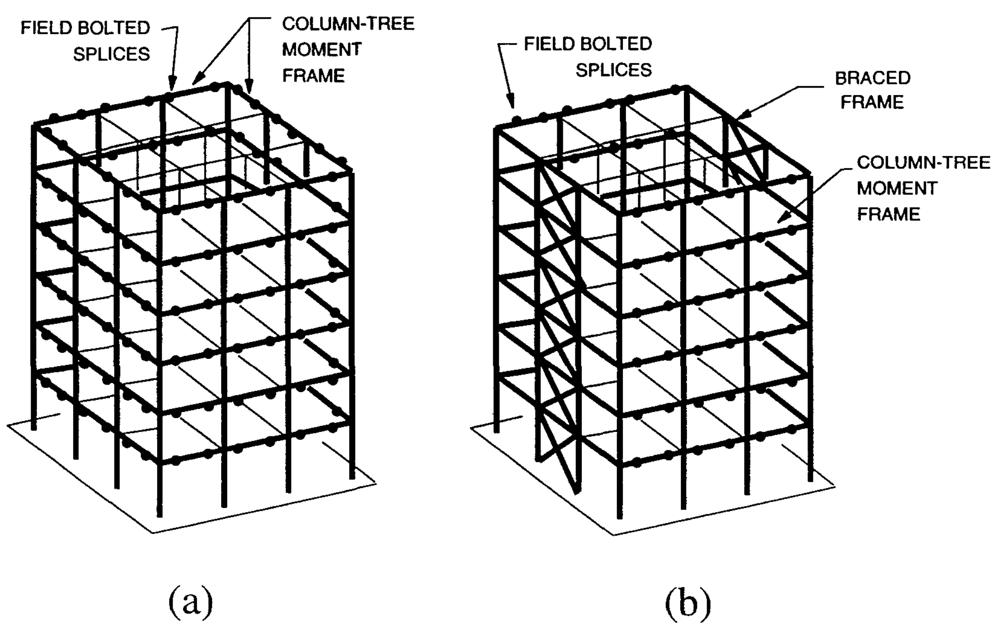


Figure 1.3. Example of the Column-Tree System used in
(a) Perimeter Moment-resisting Frame; and
(b) Planar Moment-resisting Frame

Various configurations of the rigid column-tree system have been used in the past in the United States. The shop-welded, field-bolted column-tree system is still popular for construction during cold weather. Also in projects that field welding and field inspection are too costly or cannot be done easily, the use of column-tree system can be more economical than the other systems with field welding. In Japan perhaps because of the high cost of labor, and the fact that shop welding is mostly automated, column-tree systems are currently very popular. The performance of structures during the 1995 Great Hanshin Earthquake indicates that modern steel column tree systems in the affected areas performed well and much better than field welded MRFs. However, there were a number of column-tree structures that developed cracks through the weld connecting beam stubs to steel tube columns (AIJ, 1995b).

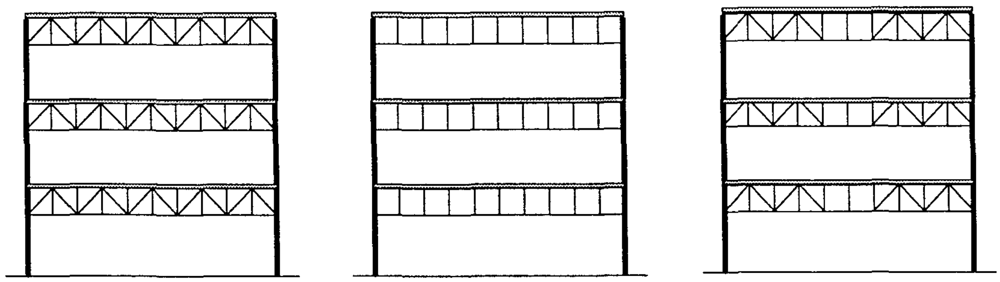
In the standard column-tree system the bolted splice connection of the beam is designed to be stronger than the connected beams. As a result, after erection, the bolted splice does not play a major role in seismic performance of the frame. To utilize the bolted splice to control and improve seismic performance, a semi-rigid version of the column-tree moment resisting frame system was proposed by A. Astaneh-Asl (1988, 1991). In the proposed semi-rigid column-tree the bolted connection of the girder, located away from the column, is made semi-rigid. By using semi-rigid connections, stiffness, strength, ductility and energy dissipation capacity can be easily manipulated to reduce seismic forces, displacements and damage and to improve seismic performance.

Recently, a study of standard rigid and the proposed semi-rigid column-tree systems was conducted at the Department of Civil Engineering of the University of California, Berkeley (McMullin et al, 1993). In the study, the semi-rigid column-tree system was shown to be a potentially reliable and economical seismic resisting structural system. One of the main advantages of semi-rigid column-tree system over the standard rigid system is that the bolted semi-rigid connection, located at the girder splice, acts as a fuse and protects the welded connections at the face of columns from being subjected to large moments. In addition, the use of semi-rigid connections can increase damping, elongate period of vibration, reduce stiffness to a desirable level and can result in reduction of seismic forces and displacements.

1.3.d. Moment-Resisting Frames with Truss Girders

Moment-resisting frames with truss girders usually consist of rolled wide flange columns and welded steel truss girders. Figure 1.4 shows examples of moment frames with truss girders. Currently, information on the seismic behavior and ductility of moment frames with truss girders is relatively limited.

During the 1985 Mexico earthquake, two 10 and 23-story steel structures in a complex of high-rise structures collapsed and a third 23-story structure developed more than 2% permanent roof drift (Astaneh-Asl, 1986a). The structural systems of these buildings were truss girder moment-resisting frame and braced frames. Even though the cause of failures was related primarily to local buckling of the bases of columns, nevertheless, welds in many truss-to-column connections had cracked.



(a) Truss Girder (b) Veirendeel Girder (c) Ductile Truss Girder

(After Basha & Goel, 1994)

Figure 1.4. Examples of Moment Frames with Truss Girders

Another version of the steel MRFs with truss girders is the system where Vierendeel trusses are used as horizontal members, Figure 1.4(b). Recently, a seismic study was conducted of an existing 6-story structure, which has Vierendeel truss girders and is located near the Hayward fault (Tipping, 1995). The inelastic time history analyses showed very good seismic behavior and well distributed yielding of the members of the truss girders.

Recent experimental and analytical studies (Basha and Goel, 1994) provides information on the seismic behavior and design of a special ductile version of moment-resisting frames with truss girders. In the proposed system, the diagonal members of a few panels at mid span of the truss girders are removed. In a way, this system is a good combination of regular truss and Vierendeel truss systems. Tests and analysis of the resulting system reported in above references have indicated good seismic behavior and potential for use in seismic areas.

1.3.e. Tube-in-Tube and Bundled-Tube Moment-Resisting Frames

Two other steel MRFs are the tube-in-tube and the bundled-tube systems. The tube-in-tube system consists of a perimeter moment-resisting frame inside a larger perimeter moment-resisting frame. The bundled-tube system is a collection of perimeter MRFs bundled together to form a single system. The Sears Tower in Chicago, currently the world's tallest building, has a steel bundled-tube MRF system. Seismic behavior of these systems is expected to be somewhere between the behavior of space MRFs and perimeter MRFs.

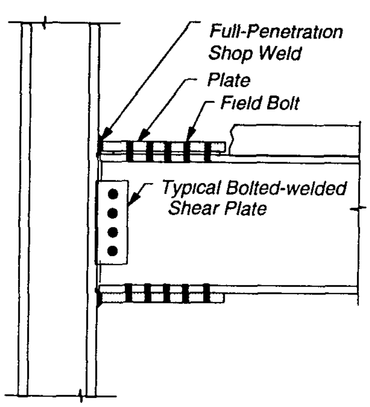
1.4. Categories of Moment-Resisting Frames Based on Type of Connections

Steel MRFs can be categorized on how flanges of a girder are connected to the columns. The categories are:

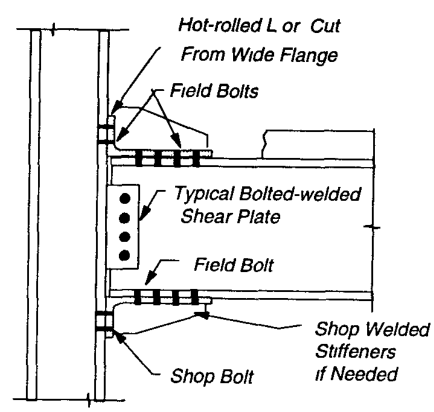
- Field-Welded
- Field-Bolted
- Riveted (used until mid 50's in the field and until 70's in the shop)

In this report the welded moment-resisting frames (WMRFs) are defined as those that have girder flanges welded to the columns in the field directly or through connection elements such as plates or angles. The bolted moment-resisting frames (BMRFs) are defined as frames having only bolting done in the field with no field welding. These latter frames can have some welding in which case the welding should be done in the shop. In both welded and bolted moment frames, the transfer of shear force from the web of the girder to the column can be by welded or bolted connections.

Examples of field-bolted and field-welded MRF connections are shown in Figures 1.5 and 1.6, respectively. Figure 1.6 (a) shows the details of the typical welded connection used almost exclusively in recent years in special moment-resisting frames in California. A number of these welded connections cracked in a brittle manner through the welds, columns, girders or panel zones during the 1994 Northridge earthquake. Other possible details of bolted and bolted-welded MRF connections are provided in Appendix A of this report.



Welded-Bolted Plates
(a)



Bolted Angles
(b)

Figure 1.5. Examples of Field-Bolted Steel Moment Frame Connections

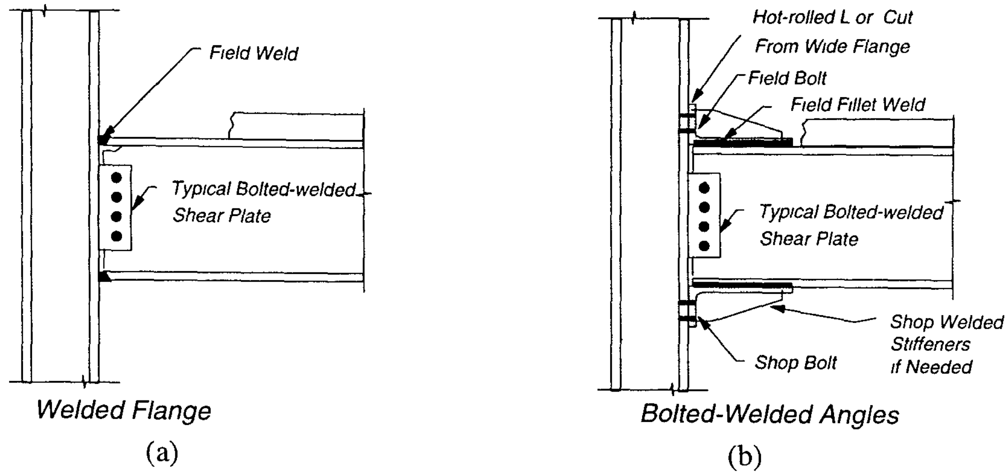


Figure 1.6. Examples of Field-Welded Steel Moment Frame Connections

1.5. Categories of Moment-Resisting Frames Based on Ductility

Steel MRFs are divided into two categories on the basis of:

- Special Ductile Moment-Resisting Frames; and
- Ordinary Moment-Resisting Frames

Figure 1.7 shows the lateral-load lateral-displacement behavior of the typical ordinary (Line OB) and special ductile moment-resisting frames (Line OA). Line OE in Figure 1.7 shows the response of a completely elastic system.

It is well known that, depending on the extent of the inelasticity (damage) in a structure, the magnitude of the seismic forces developed in the structure will vary. The inelasticity reduces stiffness, causes energy dissipation, increases damping and elongates the period of vibrations. These changes in most common structures result in a reduction in the seismic forces developed in the structure. The current seismic design approach and code procedures are based on the concept of using inelasticity (permitting some damage) to reduce the seismic design forces.

Inelasticity in steel structures, in general, can result from yielding, slippage, buckling and the fracture of the structural members or the connection elements. Yielding of the steel is the most desirable source of inelasticity and energy dissipation. This is due to the fact that currently used structural steels are very flexible and ductile materials. For example, typical A36 steel yields at a tensile strain of about 0.0015 and can deform inelastically up to strains of about 0.18. These strains indicate a ductility of about 120 for material of A36 steel. This very high ductility has been the main source of excellent performance of well designed steel structures in the past. In some cases, because of the occurrence of local or overall buckling, the fracture of net areas of metal or the fracture of connectors such as the weld fractures during the Northridge earthquake of 1994, the structure has not been able to utilize the high ductility of the steel.

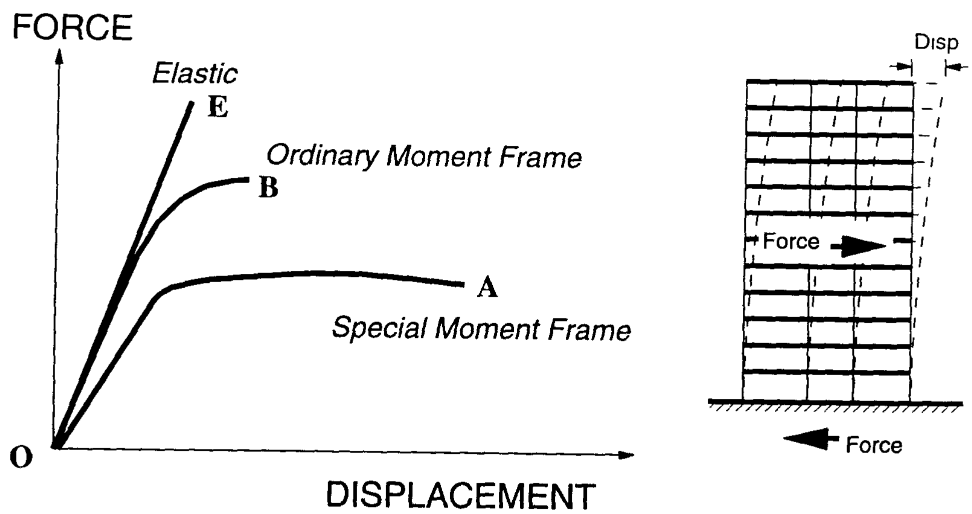


Fig. 1.7. Behavior of Special and Ordinary Moment-Resisting Steel Frames

A source of inelasticity in steel structures is slippage. If slippage occurs under service load, it may create problems with serviceability of the structure and cause cracking of the brittle non-structural elements. However, if slippage occurs under controlled conditions during earthquakes, in many cases, the slippage can improve seismic performance. The improvement can occur in three ways:

1. If slippage occurs by overcoming friction forces, such as in bolted connections, a considerable amount of energy can be dissipated in the process increasing the damping and energy dissipation capacity of the structure.

2. The slippage acts as a stiffness fuse and releases the stiffness, thus changing the dynamic character of the structure by changing its stiffness during the shaking.

3. Due to slippage in bolted moment connections, the rotational ductility of the connection is increased. Currently, one of the major deficiencies of welded connections is relatively low rotational ductility.

It can be concluded that if friction slip occurs under loads that exceed the service load by a reasonable margin of safety, and the slip strength can be maintained under cyclic action, such slippage can be used very efficiently to control and improve seismic response of steel structures and reduce the damage.

The issue of local and overall buckling of steel components needs special attention. In many cases, it is not possible to force steel to undergo only yielding. Because of slenderness of the steel components, during large cyclic deformations, overall or local buckling can occur. However, minor local buckling that does not result in cyclic fracture can be useful in improving cyclic behavior of steel structures during large earthquakes. The locally buckled areas act as fuses and limit the amount of force that can exist in these locally buckled areas. By limiting the force to local buckling capacity, other brittle elements of the connection such as welds can be protected. Current codes indirectly accept minor local buckling by limiting b/t ratios to about six to eight.

In general, buckling is less desirable than yielding since, because of cyclic buckling the capacity and stiffness of the steel component deteriorates to some extent. The deterioration of critical components can result in serious reduction of strength and stiffness of the system to carry the gravity load after an earthquake. In addition, the deformed shape of a globally or locally buckled member can be of concern to the user and in most situations the member will need to be repaired or replaced. In past earthquakes, buckling of the structural members has occasionally resulted in costly damage to nonstructural elements, such as breaking the water pipes and other lifelines causing serious collateral damage to the building contents. Therefore, it makes sense to check the consequences of member buckling and deformations.

The most undesirable source of inelasticity in structures is fracture. In the context of seismic design, fracture in general is non-ductile and unacceptable for steel, particularly, if there is no other parallel load path for the fractured member to redistribute its load. Because of fracture, the gravity load-carrying capacity of the structure can be seriously impaired resulting in partial or full instability and collapse. Such behavior is non-ductile and unacceptable. Current design codes discourage such non-ductile behavior by specifying larger design forces to be used in the design of non-ductile MRFs compared to those for the design of ductile MRFs. This is done by specifying a reduction factor, Rw, of 12 for Special Ductile Moment-Resisting Frames and 6 for Ordinary and less ductile frames. However, the decision to use structures with multiple load paths to facilitate redistribution of the seismic forces is properly left to the judgment, ingenuity and intuition of the structural engineer.

On the basis of the source of inelasticity and the ability of the inelastic elements to deform while maintaining their strength, the steel moment frames are divided into two categories of Special and Ordinary MRFs as discussed in the following. The force-displacement plots of these frames are shown in Figure 1.7.

1.5.a. Special Moment-resisting Frames

The connections and the members of Special Moment-resisting Frames (SMRFs) are designed such that fracture and premature buckling of the structural members and the connections are prevented. As a result, the special MRFs behave in a ductile manner. In special MRFs, the damage should be in the form of slippage, yielding of steel, delayed and limited local buckling within the girder connections or plastic hinges. Fracture in any part that can impair the gravity-load carrying system should be avoided. This type of behavior categorizes the system as a ductile system.

Currently, there is debate in the profession on how much ductility supply is necessary for a given steel MRF to be categorized as a Special Ductile Moment-Resisting Frame? Some researchers (Popov et al, 1994) have suggested values of 0.015 and 0.02 radian to be the desirable rotation capacity of moment connections. However, the Northridge damage has cast serious doubt on these limits. On the basis of studies of rigid and semi-rigid MRFs, Nader and Astaneh (1992) have suggested a rotational ductility of 0.03 radian. In addition, it is suggested herein that the cumulative inelastic cyclic rotation capacity of a ductile moment connection should be at least 0.15 radian.

1.5.b. Ordinary Moment-Resisting Frames

If a steel moment-resisting frame does not meet the requirements of the Special moment frame, then the frame is not expected to behave in a ductile manner and it is categorized in the seismic design codes as an Ordinary MRF. Ordinary MRFs still need to have sufficient rotational ductility to make them eligible to be designed using a reduction factor of Rw equal to 6. Again there is no well-established value of required ductility supply for Ordinary MRF. It is suggested here that, in the absence of more reliable value, the connections of Ordinary MRFs should have a rotational ductility of at least 0.02 radian. The cumulative cyclic rotational capacity is suggested to be at least 0.10 radian.

1.6. Categories of Moment-Resisting Frames Based on Stiffness

The following discussion applies to moment-resisting frames with strong columns and weak beams. In these systems, the behavior of girder and connection dominates the global behavior.

The behavior of a steel MRF strongly depends on the rotational behavior of its connections and the bending stiffness of its beams and columns. Traditionally, steel MRFs are divided into the three categories of Rigid (Fully Restrained, FR), Semi-rigid (Partially Restrained, PR) and Flexible (Simple) (AISC, 1994). Flexible Moment Frames can be found in some existing structures or are used as a back-up system for braced frame systems. The above division is primarily based on the bending stiffness and the strength of the beam-to-column connections.

The parameter that has been frequently used in the past to define the relative rotational stiffness of a girder and its connections is the stiffness parameter m defined as:

$$m = \frac{K_{con}}{\left(\frac{EI}{L}\right)_g} \tag{1.2}$$

where K_{con} is the rotational stiffness of the beam-to-column connection, and $(EI/L)g$ is the bending stiffness of the girder. Depending on the value of m, the girder span is categorized as:

Rigid span if	$m \geq 18$	
Semi-rigid span if	$18 > m \geq 0.5$	and
Flexible span if	$m < 0.5$	

Figure 1.8 shows the above three regions of the moment-rotation behavior based on the relative rotational stiffness of the connection and the girder.

The above categorization is solely based on the elastic rotational stiffness of the connections and the girders in a single span. Such categorization has been used in the past in the elastic design of girders under gravity load.

In seismic design, however, the plastic moment capacity of the connections and the girders should also be considered in categorizing the span. For example if in a rigid span, i.e. $m > 18$, the plastic moment capacity of the connections is less than the plastic moment capacity of the girder, the span will behave as semi-rigid after the connections reach their plastic moment capacity and develop plastic hinges. To define the behavior of a span as rigid, semi-rigid or flexible, in addition to the stiffness parameter m, a strength parameter α is introduced which is defined as:

$$\alpha = \frac{(Mp)_{con}}{(Mp)_g} \tag{1.3}$$

where, $(Mp)_{con}$ and $(Mp)_g$ are plastic moment capacities of connection and girder, respectively.

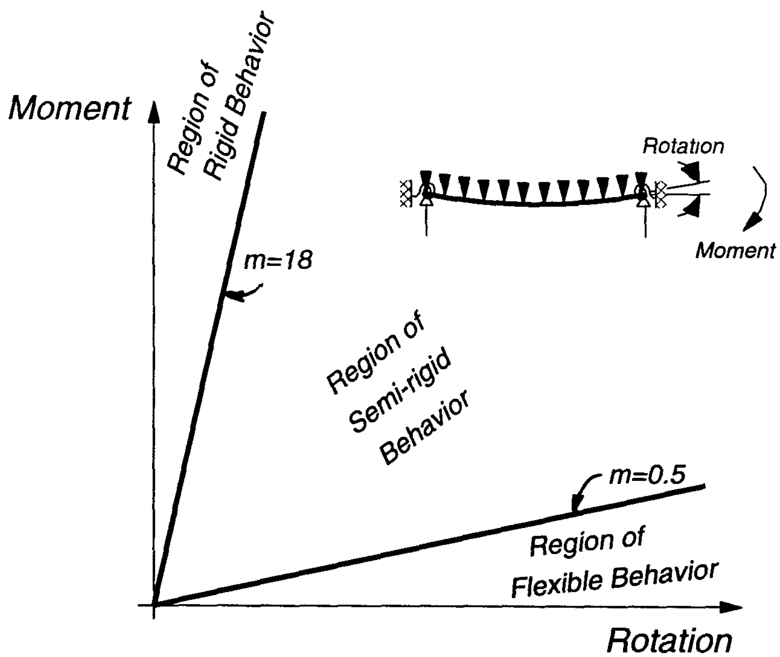


Figure 1.8. Regions of Rigid, Semi-rigid and Flexible Behavior of Elastic Beams

Incorporating the effects of inelasticity of the girder and the connections, the definitions of rigid, semi-rigid and flexible spans are enhanced and given as follows:

For Rigid Spans: $m \geq 18.0$ and $\alpha \geq 1.0$ (1.4a)

For Semi-rigid Spans: either [$m > 18$ and $0.2 < \alpha < 1.0$] (1.4b)
or [$18.0 \geq m > 0.5$ and $\alpha > 0.2$]

For Flexible (Simple) Spans: either $m \leq 0.5$ (1.4c)
or $\alpha < 0.2$

The above definitions are shown in Figure 1.8.

In order to categorize a moment-resisting frame as rigid, semi-rigid or flexible, the above definitions for girder spans are extended to moment-resisting frames and the following definitions are suggested:

1.6.a. Rigid Moment-Resisting Frame

A rigid MRF is a moment frame in which all spans satisfy the condition that

$$m \geq 18.0 \quad \text{and} \quad \alpha \geq 1.0 \tag{1.5a}$$

Where m and α are defined as the ratio of the stiffness and strength of the connections to the stiffness and strength of the girders, respectively, see Equations 1.2 and 1.3.

In establishing m and α for moment frames to be used in Equations 1.5, the average value of m and α for the spans of the mid-height story of the moment frame can be used.

1.6.b. Semi-rigid Moment-Resisting Frame

A semi-rigid moment frame is a moment frame in which at least 80% of the spans satisfy the condition that

$$\begin{array}{ll} \text{either} & m > 18 \quad \text{and} \quad 0.2 < \alpha < 1.0 \\ \text{or} & 18.0 \geq m > 0.5 \quad \text{and} \quad \alpha > 0.2 \end{array} \tag{1.5b}$$

1.6.c. Flexible Moment-Resisting Frame

A flexible moment frame is a moment frame in which at least 80% of the spans satisfy the condition that

$$\begin{array}{ll} \text{either} & m \leq 0.5 \\ \text{or} & \alpha < 0.2 \end{array} \tag{1.5c}$$

The above equations are shown in Figure 1.9.

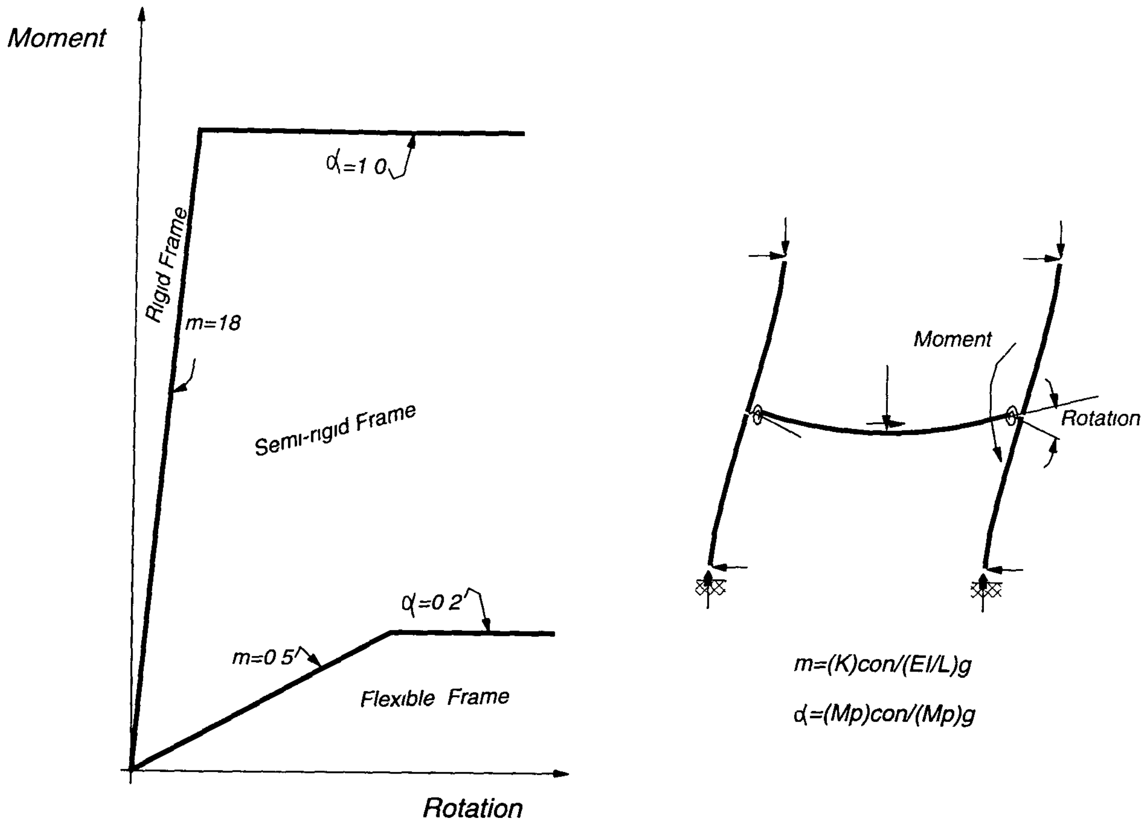


Figure 1.9. Definition of Rigid, Semi-rigid and Flexible Moment-Resisting Frames

1.7. Categories Based on the Moment Capacity of the Connected Members

Depending on relative bending capacities of columns and girders in the joints of a moment-resisting frame, the frame is categorized as one of the following:

- Strong Column - Weak Beam
- Strong Beam- Weak Column

The strong column-weak beam frames are used very frequently and many structural engineers believe that these systems have superior seismic behavior to that of the weak column-strong beam frames.

In the strong column-weak beam frame, the moment capacity of the beams in a joint is less than the moment capacity of the columns. Therefore under combinations of gravity and lateral loads, plastic hinges are expected to form in the beams. In the strong beam-weak column design, plastic hinges are expected to form in the columns.

The design philosophy of the strong column-weak beam has been used very frequently in seismic design. This is primarily due to the importance of the columns in carrying the gravity load after an earthquake as well as the P-Δ effects on the column buckling and the overall stability of the structure. Most current codes (ICBO, 1994) also promote the use of the strong column-weak beam philosophy. Recent studies have shown that the steel MRFs that develop hinges in the girders (strong column-weak beam design) can be more stable than the frames that have column hinges (strong beam-weak column).

The philosophy of the strong column and weak beam design is a rational and well accepted seismic design approach. However, occasionally, especially in low-rise buildings and long spans, it is difficult and costly to implement this philosophy. One way to implement the strong column and weak beam design properly is by use of semi-rigid beam-to-column connections (Nader and Astaneh-Asl, 1992). In this case, even though the beam can be very strong and stiff, the moments transferred to the columns will be limited to the moment capacity of the semi-rigid connections and not the moment capacity of the girder. The moment capacity of the semi-rigid connections can be selected such that the plastic hinges are forced to form in the connections and not in the columns resulting in a new version of the strong column-weak girder system.

In recent years, a new trend in seismic design of steel moment frames has emerged which is to permit some inelasticity in the panel zone of the columns. The 1994 Uniform Building Code has provisions to implement this concept by requiring that the panel zone shear capacity need not exceed the shear required to develop 0.8 of the moment capacity of the connected beams. It should be mentioned that the main benefit of permitting limited yielding of the panel zone is to reduce, and in most cases to eliminate, the need for doubler plates. However, on account of the fracture of some panel zones and columns adjacent to panel zones during the 1994 Northridge earthquake, it appears that there is a need for re-examination of the effects of panel zone yielding on the overall seismic behavior and stability of steel moment frames. Until such studies are concluded and also until the cause of fracture of some panel zones during the 1994 Northridge earthquake is established, it is suggested here that widespread yielding and distortion of the panel zones be avoided in Seismic Zones 3 and 4.

It is interesting to note that an economical and reliable way to reduce or eliminate the need for doubler plates in the panel zones is by the use of semi-rigid girder-to-column connections. The use of semi-rigid connections with a pre-designed moment capacity will result in control and reduction of the moment transferred to the column panel zones, thus reducing the need for doubler plates. In addition, the semi-rigid connection can act as a fuse and prevent large moments from being transferred to the column and the restrained panel zones (Nader and Astaneh-Asl, 1992).

1.8. Selection of a Suitable Moment-Resisting Structural System

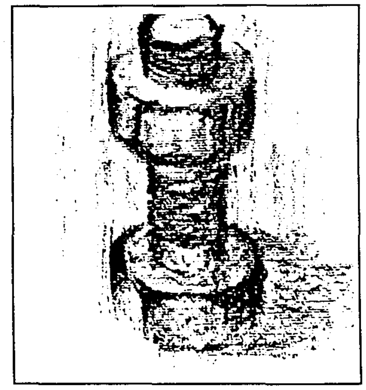
Selection of a suitable structural system for a given building depends on many parameters such as economy, architectural and mechanical constraints, soil conditions, geometry, site condition, ease of fabrication, speed of construction and preference of owner, architect and the structural engineer. Whenever steel moment-resisting frames are selected as the structural system, there is a variety of configurations that can be used. Various categories of steel MRFs were discussed earlier in this chapter. Figure 1.1 shows a flow chart of the possibilities for steel MRFs.

A number of connections in welded MRFs were damaged during the 1994 Northridge earthquake. As Figure 1.1 indicates, the welded special moment frame system is only one of the possible types of steel MRF systems. Other systems, such as bolted steel special moment frame systems, have been used in the past with great success and currently are being used in a number of structures as a replacement for the welded special moment frames.

Appendix C of this report shows examples of bolted steel special moment frames that were designed and constructed after the 1994 Northridge earthquake. The structures were originally designed as pre-Northridge types of welded special moment-resisting frames. However, in the aftermath of the 1994 Northridge damage, the connections were redesigned and the frames were converted to bolted special MRFs. The structures are currently completed and occupied. According to the structural engineers in charge of these designs, (Hettum, 1994), design and construction of these bolted moment frames have been very cost efficient and had very few problems.

During the last ten to twenty years, for a variety of reasons, the fully-welded rigid steel moment frame had become almost the *only* steel MRF system used in California. All of the steel moment frames damaged in Los Angeles during the 1994 Northridge earthquake have this one system. It is not surprising that when Northridge caused damage, many modern structures using this system were affected. It is hoped that information provided in this report will be useful to structural engineers, code officials, permit agencies and others in diversifying and utilizing other structural steel systems such as bolted special rigid moment frames (subject of this report), bolted semi-rigid steel frames (Nader and Astaneh-Asl, 1992) and column-tree systems (Astaneh-Asl, 1988; McMullin et al., 1993).

2. SEISMIC BEHAVIOR OF BOLTED STEEL MOMENT CONNECTIONS



2.1. Introduction

Actual seismic behavior of structures can be studied by: (a) investigation of the damage due to earthquakes and (b) by realistic laboratory testing of the structures and their components. With the exception of the 1994 Northridge and the 1995 Great Hanshin earthquakes, there are very few reports of consequential damage to modern steel moment frames. Perhaps the Mexico-City earthquake of 1985 was the first earthquake to cause the collapse of a 23-story high-rise welded steel structure. The cause of the collapse of that structure was related to low quality and low strength of the welds as well as to local buckling of the built-up box columns (Astaneh-Asl, 1986a; Martinez-Romero, 1988).

Seismic performance of bolted steel moment frames during past earthquakes is briefly summarized in the following Section 2.2. A brief summary of research projects on laboratory behavior of steel moment frames and their components is provided later in this Chapter.

2.2. Performance of Bolted Steel Moment-Resisting Frames in the Past

There are many existing riveted, bolted and welded steel structures that have been shaken by earthquakes in the past. No report of damage of any consequence or collapse of major riveted MRFs could be found in the literature. One of the early tests of seismic performance of riveted steel structures was the 1906 San Francisco earthquake. In the post earthquake reports and photographs taken in the aftermath of the 1906 quake, it appears that there was no collapse or structural damage to riveted steel structures in downtown San Francisco. All tall buildings of the time (all riveted steel structures) appear in photographs and reports to be undamaged. Alas, the later photographs, taken only few days after the quake, show a few of the same buildings engulfed by the fire that swept through most of downtown San Francisco after the quake.

In the photographs taken after the fire in San Francisco, there are several instances of steel column buckling and structural failures that appear to have been due to the intense heat of the fire reducing the strength of the members below their service load level, thus causing partial or total collapse of a number of steel structures. Today, with higher fire-proofing standards and practices in steel structures, such fire hazard is reasonably mitigated.

In the aftermath of the 1906 earthquake, the California State Board of Trade stated in 1906:

> ".. The earthquake damage was inconsiderable. Every building on both side of Market street stood against the earthquake. The modern steel-frame buildings were unhurt, and that style of structure stands vindicated. The city has to rise from the ashes of conflagration, and not from the ruins of an earthquake. .." (Saul and Denevi, 1981).

Since the 1906 earthquake, there has been no published report of serious and consequential damage to bolted steel MRFs during earthquakes. Of course, the lack of damage reports, can in part be attributed to the fact that prior to 1994 Northridge earthquake, very limited reconnaissance effort was expended on inspecting the damage to steel structures. However, if there was any damage to bolted steel structures, it must have been minor and not of consequence.

According to Martinez-Romero (1988) performance of bolted steel structures during the 1985 Mexico earthquake was outstanding. The type of connections used in these structures were generally top- and bottom-plate or flange tee connections.

Studies of performance of steel structures during the 1994 Northridge and the 1995 Great Hanshin earthquake in Japan also indicates very good performance of bolted steel structures. However, a number of welded connections of low and mid-rise steel moment frames fractured during both earthquake (Astaneh-Asl et al., 1994 and 1995).

It should be emphasized that most of the existing riveted and bolted MRFs were not designed and detailed as Special Ductile MRFs and can be categorized as Ordinary MRFs. Therefore it is expected that some of them could experience damage during future major earthquakes. However, because of the relatively higher quality control for bolted steel structures than for field-welded structures, more redundancy in bolted connections, and less three-dimensional stress field than for the welded joints, the likelihood of brittle damage is low.

In addition, because of slippage of the bolts and gap opening and closing in the connections, bolted steel structures demonstrate a certain amount of semi-rigidity during earthquakes. The author believes that the main reason for the very good performance of bolted steel structures during past earthquakes is the semi-rigidity of bolted connections. In many cases, such semi-rigidity increases damping, releases and reduces stiffness, dissipates seismic energy, isolates the mass from the ground motions and elongates the period, all of which cause reduction in the seismic response of the structure. More information on performance and seismic design of steel semi-rigid moment frames can be found in Astaneh-Asl (1994), Nader and Astaneh-Asl (1992) and other publications, some of which are listed in the References.

2.3. Behavior of Bolted Steel Moment-Resisting Frame Connections in Laboratory Tests

The systematic study of the cyclic behavior of steel moment connections started in the 1950's with the pioneering work of Egor Popov at the University of California, Berkeley and Ben Kato of the University of Tokyo. Since then a number of important research projects have been conducted in this field worldwide. The following sections provide a summary of selected projects that directly relate to the subject of this report.

2.3.a. Tests by Popov et al.

From the late 1950's through the late 1980's a series of cyclic tests and studies of the cyclic behavior of steel welded moment connections were conducted at the University of California at Berkeley (Popov et al., 1957, 1965, 1973, 1988). The majority of connections tested were welded specimens with the exception of one project where bolted top- and bottom- plate connection specimens were also tested and studied. A summary of studies of welded moment connections can be found in Bertero et al., (1994) and only the performance of bolted specimens (Pinkney and Popov, 1967) is summarized here

The specimens in the above tests consisted of a cantilever beam connected to a supporting column by top and bottom bolted plate connections. The specimens were subjected to cyclic moment by applying a cyclic load to the end of the cantilever beam. The failure modes observed in these specimens were local buckling of the beam and fracture of the net area of the beam or plate. In these specimens, in general, the top and bottom plates were stronger than the girder flange forcing the failure mode to be fracture of the girder flange. As the tests presented in the next section indicated, by following the current design procedures in the AISC Manual (AISC, 1994) for top and bottom plate connections, a more balanced design results. Such a balanced design results in the strengths of the connection and member being close and the damage being spread into the connection rather than concentrated along the net section of the girder.

2.3.b. Tests of Bolted Top-and-Bottom Plate Moment Connections

In 1989, Harriott and Astaneh-Asl (Astaneh-Asl et al., 1991) conducted experimental and analytical studies of the cyclic behavior of bolted top-and-bottom plate moment connections. The objective was to investigate the cyclic behavior of three types of steel bolted beam-to-column connections under severe seismic loads. By using the information collected during the experiments, seismic design procedures for these connections were developed and proposed. A refined version of these procedures is proposed in Chapter 4 of this report.

Sketches of the beam-to-column connections that were tested are shown in Figure 2.1. Each specimen consisted of a 7-feet long W18x50 beam connected to a 3-feet long column by top and bottom bolted flange plates and a shear connection. In all specimens the top and bottom plates were the same and were welded to the column by full penetration welds. The only difference among the specimens was the mechanism of shear transfer.

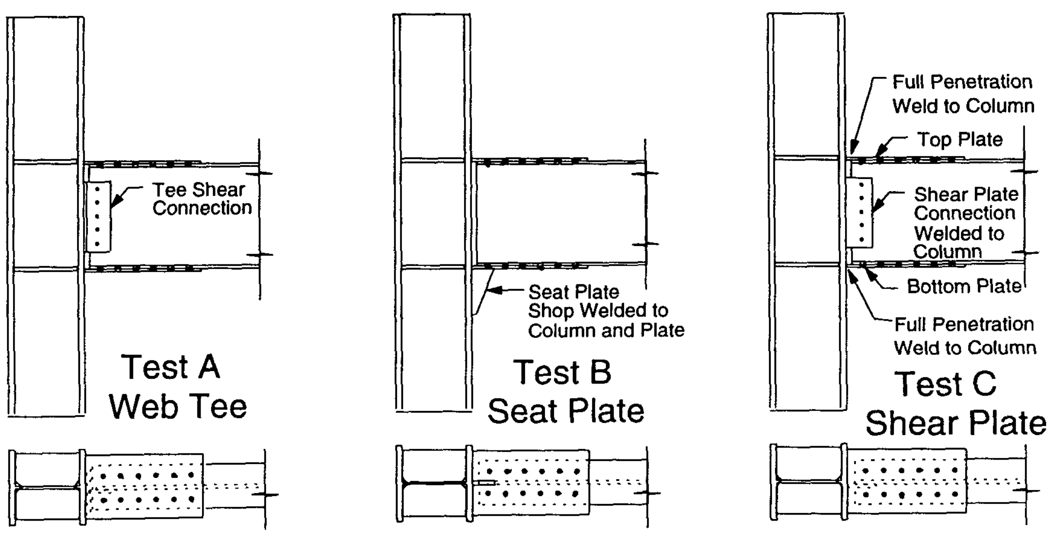


Figure 2.1. Test Specimens for Bolted Top- and Bottom- Plate Connections (Astaneh-Asl et al., 1991)

In Specimen A, the web connection was a structural tee. Specimen B did not have a web connection. To transfer shear from beam to column, in this connection, a vertical stiffener was used under the bottom flange. The stiffener was welded to the column flange as well as to the bottom flange plate of the girder. Specimen C had a single-plate shear connection. The shear plate was welded to the column flange and bolted to the beam web by five bolts.

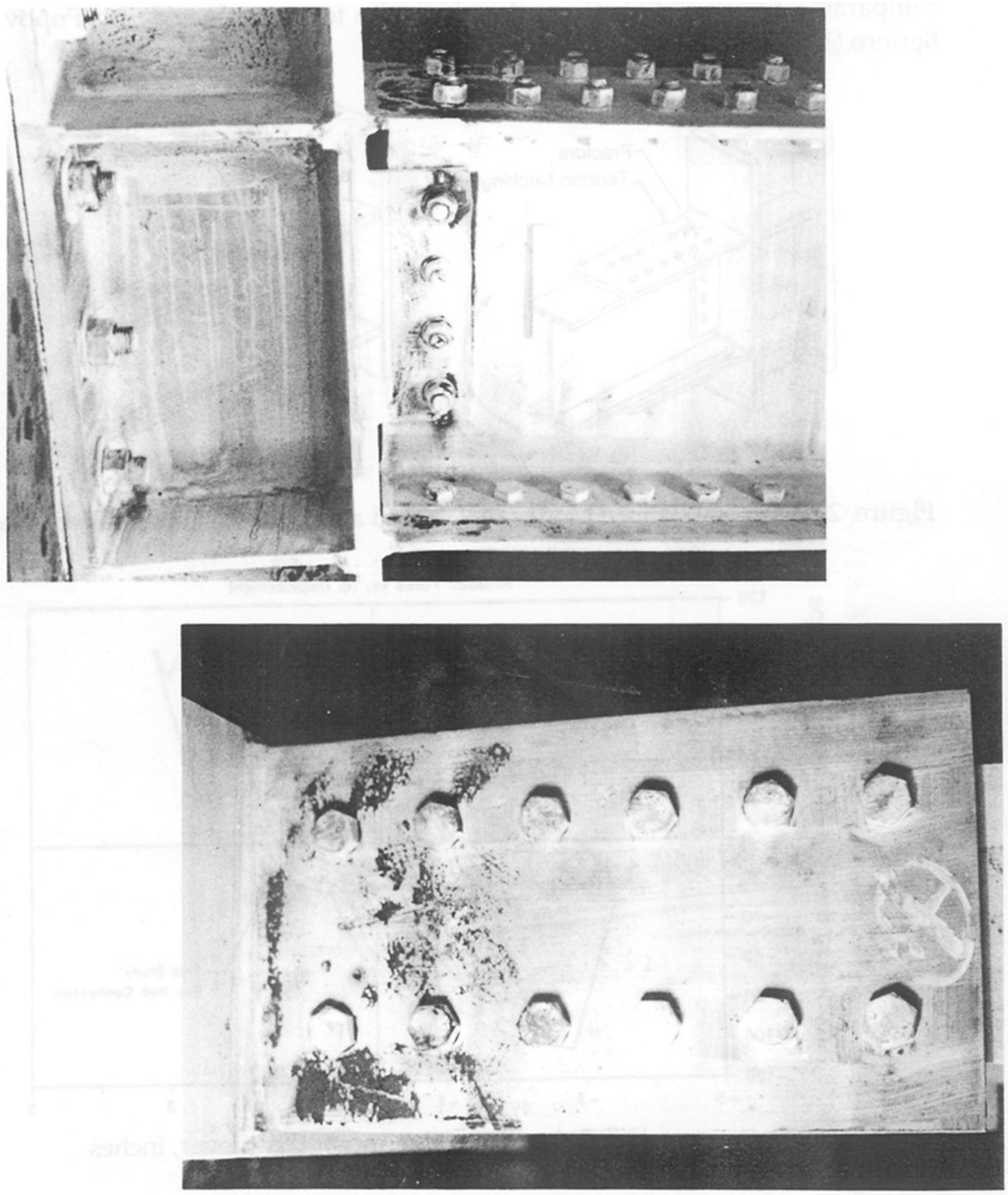


Figure 2.2. Side and Top Views of Specimen with Web Shear Plate at the End of the Tests (Astaneh-Asl et al., 1991)

2.4. Summary of Behavior of Top-and-Bottom Plate Bolted Moment Connections

Figure 2.3 shows typical failure modes of welded and bolted rigid moment connections while Figure 2.4. shows a comparison of the moment-rotation behavior of a bolted connection (Astaneh-Asl et al., 1991) and a comparable fully welded connection from the tests conducted by Popov and Bertero (1973).

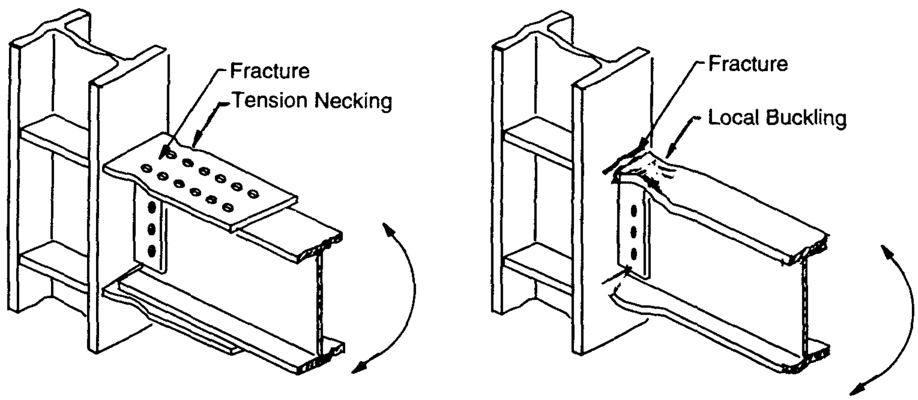


Figure 2.3. Typical Failure Modes of Welded and Bolted Moment Connections

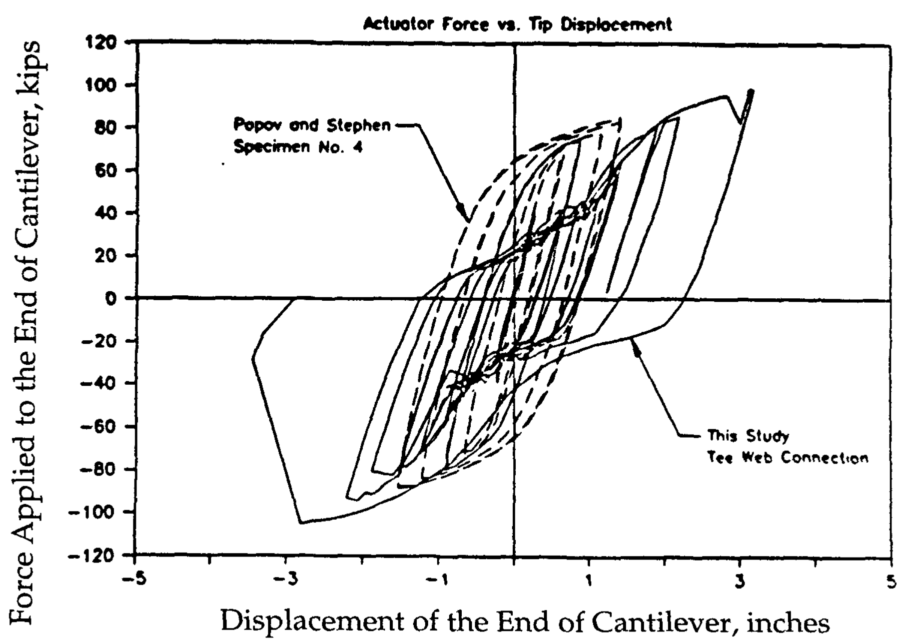


Figure 2.4. Comparison of Moment-Rotation Curves for Welded and Bolted Connections (Astaneh-Asl et al., 1991)

The following observations are based on the results of the cyclic tests of bolted and welded connections summarized above.

1. The initial elastic stiffnesses of bolted and welded specimens are almost the same. After several cycles of slippage, the elastic stiffness of the bolted specimen is slightly less than that of the comparable welded specimen. (Notice the unloading slopes during late cycles).

2. As cyclic loading continued, both the welded and bolted specimens continued to develop larger moment capacity (notice no deterioration of strength in Figure 2.4.)

3. The slippage behavior of the bolted connections was very stable. The slope of the slip plateau was considerable indicating gradual slippage At the end of the slip plateau, the bolted specimens were able to recover almost all of their initial elastic stiffness.

4. Because of slippage and ductile yielding of the top- and bottom-plates and the shear connections, rotational ductility of bolted specimens was nearly twice as much as that of comparable welded specimens.

5. In bolted specimens, there was almost no local buckling. Only very minor buckling was observed after at least ten inelastic cycles. In welded specimens, severe local buckling has been observed. In many cases, in welded specimens, the severity of local buckling was such that the locally buckled girder would need to be replaced after the earthquake in a real building.

6. In bolted specimens when a flange plate is subjected to compression, it yields in the area between the column weld and the first row of bolts. The same plate subjected to tension in the bolted connection, yields between the first and second rows of the bolt under a 45° degree angle as shown in Figure 2.2. In fully welded connections, both tension and compression yielding occur in the heat-affected zone of the welded flange adjacent to the weld line connecting the flange to the column as shown in Figure 2.3.

7. The separation of compression and tension yield areas in bolted specimens and the bracing provided by the plate and the beam flange for each other are the main reasons for the very ductile behavior of bolted connections. In other words, because of separation of the compression and tension zones of the steel in bolted connections, deterioration of stiffness due to the Bauschinger effect is almost non-existent.

8. The cyclic behavior of the above bolted specimens was very ductile. All specimens could tolerate more than 15 inelastic cycles being able to reach cyclic rotations exceeding 0.03 radian.

9. As expected, the rotational stiffness of the connections was less than that predicted by the theoretical assumption of infinite rigidity. The elastic stiffness of the specimen with the web shear tab was almost the same as that of welded specimens tested by Popov and Bertero (1973) while the stiffness of specimens with web tee connection and seat connection was slightly less than that for the welded connections. All three bolted specimens could be categorized as rigid, ductile, moment connections.

10. Slippage in bolted connections was small and about 1/8 inch after ten inelastic cycles.

11. In bolted connections, bending moment causing slippage could be predicted well by using a coefficient of friction of 0.33 given in the literature for unpainted clean mill scale (Class A) surfaces.

Finally, It should be added that the semi-rigidity observed in the bolted specimens does not necessarily reflect an inferior characteristics for the seismic behavior of frames using these connection. As shown in the following section, shaking table tests (Nader and Astaneh-Asl, 1991) as well as analytical studies (Nader and Astaneh-Asl, 1992) have demonstrated that the semi-rigidity of ductile steel connections can improve and reduce the seismic response of steel frames.

2.5. Seismic Behavior of Bolted End-Plate Connections

End plate moment connections are more common in Europe than the U.S. One of the difficulties often mentioned by engineers and fabricators in using end plate connections is the lack of fabrication tolerances. In addition, until recently, (Ghobarah et al., 1990 and 1992) there was almost no seismic design procedures for end plate moment connections.

Early cyclic tests of end plate moment connections were conducted in Europe by a number of researchers. The results of some of these studies can be found in Balio et al. (1990). In North America during the 1980's and 1990's a number of cyclic tests of bolted end plate connections were conducted by Astaneh-Asl (1986c), Tsai and Popov (1990), and Ghobarah et al (1990 and 1992). The most extensive work in this field is the extensive studies done by Ghobarah and his research associates in Canada. The reader is referred to above references for more information on cyclic behavior and seismic design of moment-resisting frames with bolted end plate connections. In the following, a summary of the results of cyclic tests of end plate connections conducted by the author in 1986 is provided.

2.6. Cyclic Tests of a Typical End Plate and an Innovative Pre-stressed End Plate Connection (Astaneh-Asl, 1986c)

In 1986, using the test set-up developed by Tsai and Popov (1990) at the University of California, Berkeley, A. Astaneh-Asl (1986c) conducted two cyclic tests of extended end plate connections. The test set-up and connections are shown in Figures 2.5 and 2.6 respectively. The data from the tests were processed (Astaneh-Asl and Nisar, 1988) and the results were presented at professional gatherings including (SAC, 1994). In the following a summary of the results is presented.

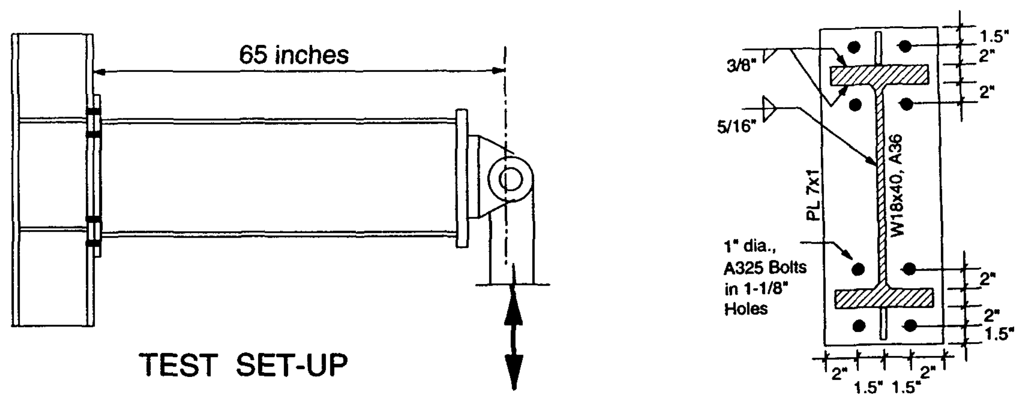


Figure 2.5. Test Set-up and Connection Detail Used in Cyclic Tests of End Plate Moment Connections (Astaneh-Asl, 1986c)

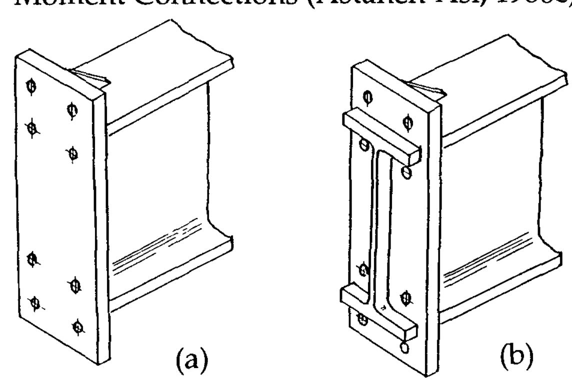


Figure 2.6. Standard and Innovative Pre-stressed End Plate Connections (Astaneh-Asl, 1986)

2.6.a. Cyclic Behavior of Standard End-Plate Connection

The standard end plate connection that was tested (Astaneh-Asl, 1986c) is shown in Figure 2.6(a). The connection was designed to develop moment capacity of a W 18x40 A36 beam. The design procedure in the AISC Manual was followed. It should be mentioned that the procedure in the AISC Manual is not specifically for seismic design. For that reason, one of the objectives of the test was to investigate how an end plate designed according to the AISC Manual procedures will perform under severe inelastic cyclic loading.

As shown in Figure 2.5(a), welds connecting the beam to the end plate were E70xx fillet welds and not full penetration welds usually thought to be used for this application. The reason for using fillet welds was to investigate if fillet welds that are more ductile and less costly can be used in this application. The tests indicated that in both specimens, the fillet welds performed well and were able to develop cyclic moment capacity of the beam section.

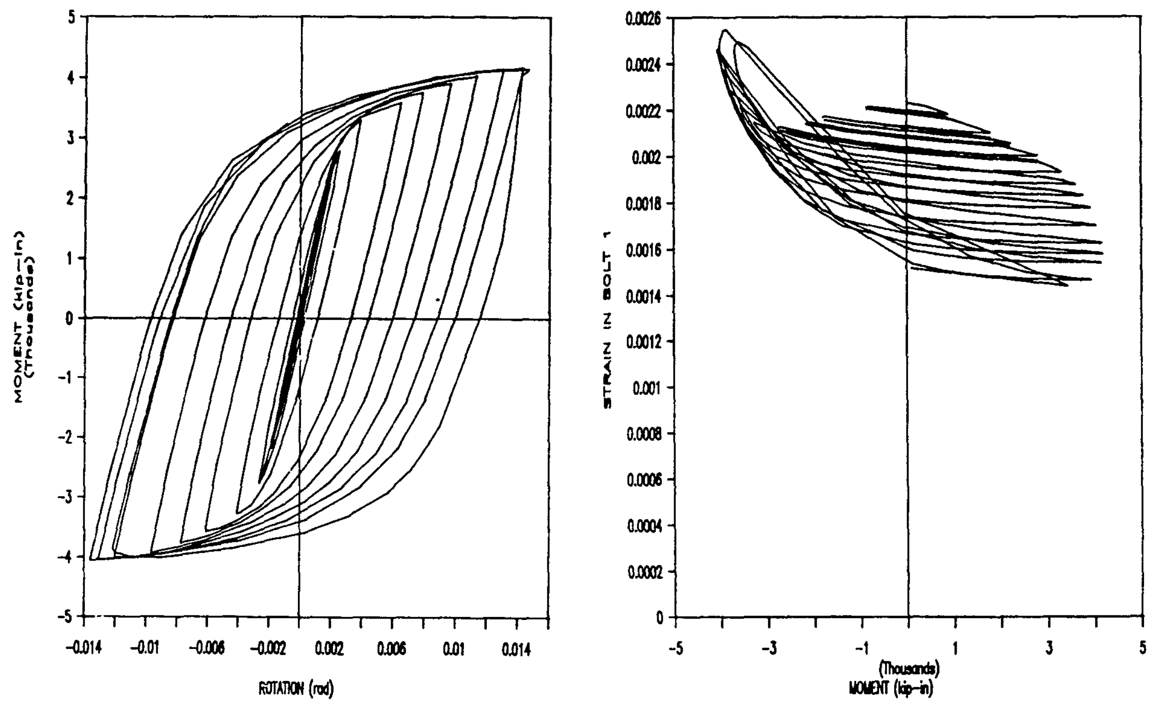


Figure 2.7. (a) Moment-Rotation Curves and (b) Bolt Strains in Standard End Plate Specimen (Astaneh-Asl, 1986c)

Figure 2.7(a) shows moment-rotation behavior of the standard end plate connection. The connection performed well under cyclic loading and a well-defined and stable plastic hinge formed outside the connection and in the beam. Figure 2.7(b) shows the variation of strain in the bolt outside the beam. The bolt continued to lose its pretensioning force but retained about 60% of its initial pre-tensioning force.

The failure mode of this specimen was cyclic local buckling of flanges of the beam. Local buckling started after seven inelastic cycles when the rotation reached 0.014 radian. At the time of initiation of local buckling the compressive strain in the locally buckled area of flange was measured at 0.035. Cyclic loading stopped at a maximum rotation of about 0.02 without any observed fracture. Figure 2.8 shows the specimen at the end of the cyclic tests.

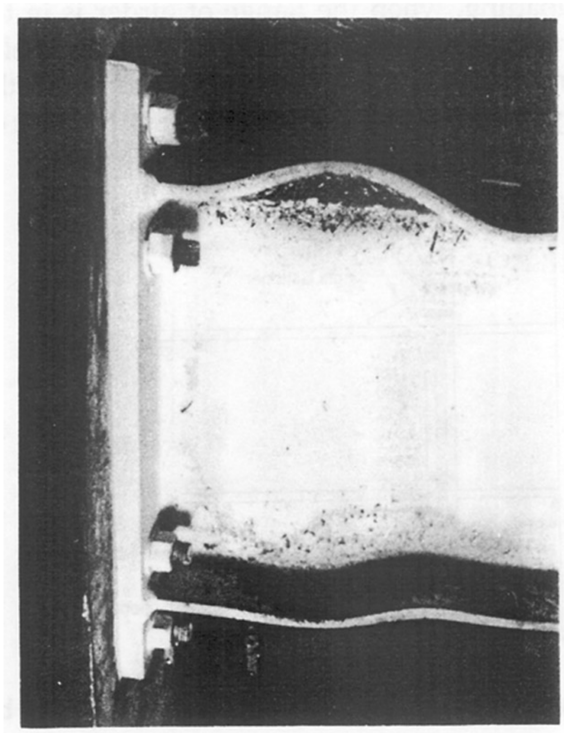


Figure 2.8. Standard End Plate Specimen at the End of the Tests (Astaneh-Asl, 1986c)

2.6.b. Cyclic Behavior of Pre-stressed End Plate Proposed by A. Astaneh-Asl (1986c)

According to some fabricators, one of the obstacles that prevents widespread use of end plate connections is the lack of erection tolerances. In girders with end plates the total back-to-back length of the girder should match the face-to-face distance of the supporting columns. Quite often, to facilitate erection the girder with end plates is fabricated slightly shorter and the gap between the end plate and the column face is filled with shims. The prestressed end plate connection proposed by the author was one solution to the problem.

In the proposed pre-stressed connection (Astaneh-Asl, 1986c), the girder with end plates is fabricated 1/2 inch to 3/4 inch shorter and the gap between the end plate and the column face is filled with a 1/2 inch to 3/4 inch length of the beam as shown in Figure 2.9.

When the short I shape element (actually cut from the beam) is placed between the end plate and the column and the bolts are tightened, the I-shape element develops compression force almost equal to the tension in the bolts. During cyclic loading, when the flange of girder is in tension, the tension force causes relief in the compression force in the I-shape element. When the beam flange is in compression, the compression is added to the I-shape element. As a result, in this system, the bolts do not feel the full extent of cyclic loading.

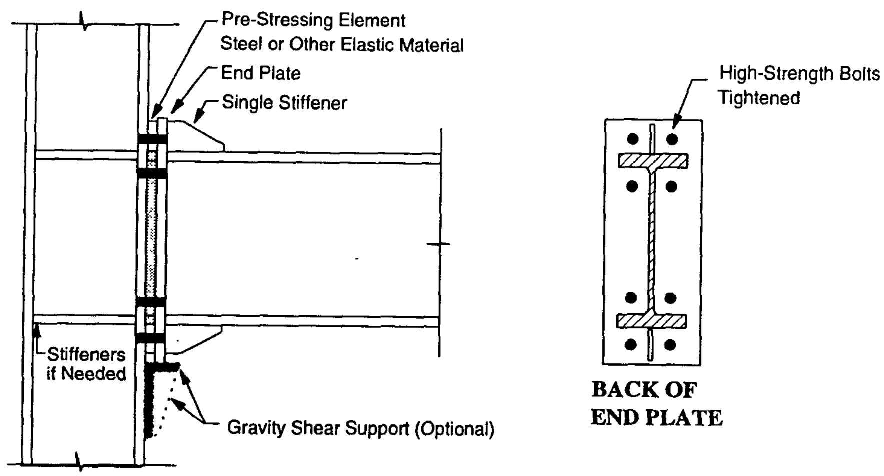


Figure 2.9. Prestressed End Plate Connections Proposed by Astaneh-Asl (1986c)

The specimen that was tested is shown in Figure 2.6(b). Figure 2.10(a) shows moment-rotation curves for this specimen. Figure 2.10(b) shows the strain in bolts outside the beam.

Several observations on the behavior of this specimen could be made:

a. The connection performed as rigid elastic during initial cycles and was able to develop plastic moment capacity of the beam.

b. After few cycles of compression, the I-shaped element placed between the end plate and the column yielded in compression. the compression yielding caused the loss of pre-tensioning load in the bolts and resulted in the bolts becoming the active elements.

c. From the performance of this one specimen it was concluded that if the I-shaped element placed between the end plate and the column had remained elastic, the connection would have performed extremely well and better than the standard end plate connections. One way of achieving such an elastic behavior, which is the key to maintaining prestressing forces, is to use higher strength I-shape elements with larger cross section than the flange of the beam. Further development of the proposed concept is currently under consideration by the author.

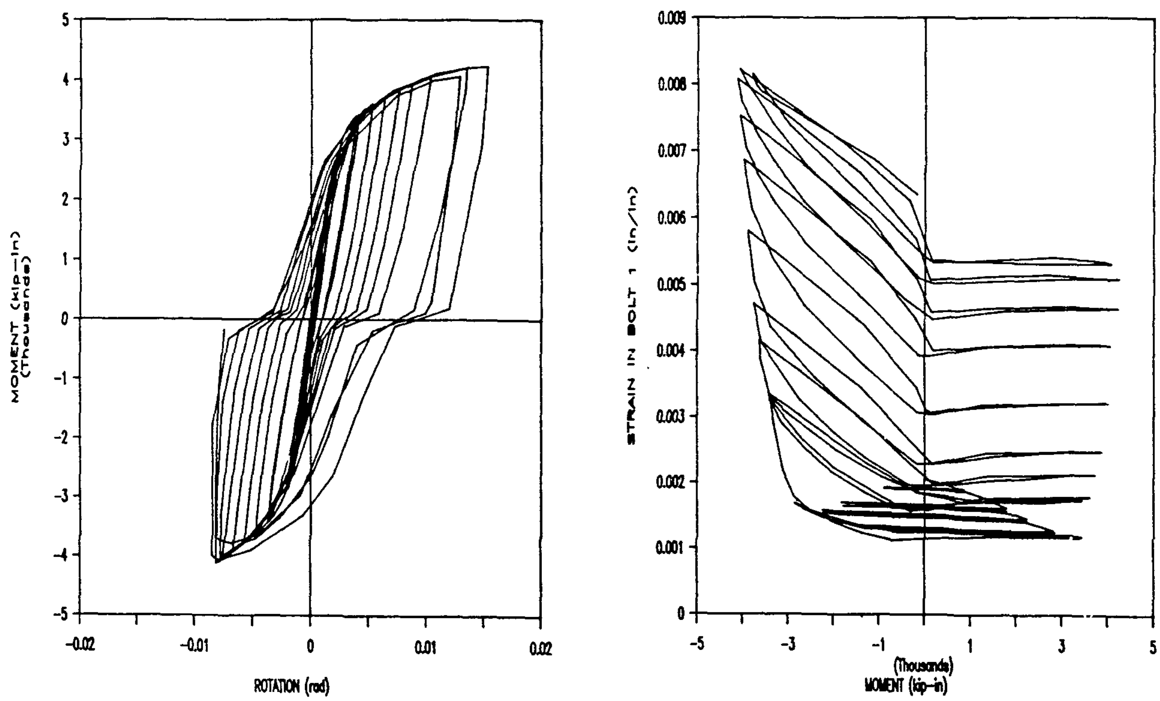


Figure 2.10. (a) Moment-Rotation Curves and (b) Bolt Strains in the prestressed End Plate Specimen Proposed by A. Astaneh-Asl (1986c)

In general, behavior of the proposed prestressed end plate connection was ductile. The failure mode was local buckling of the beam flanges. The local buckling occurred when the rotation reached about 0.01 radian. At this point the strain in the locally buckled flange was about 0.06. Figure 2.11 shows the specimen at the end of the tests.

The available data on cyclic behavior of end plate connections indicate that it is possible to design sufficiently strong yet economical end plate connections and force the plastic hinges to form in the connected girders. The plastic hinges in the girders can be made ductile by using girders with relatively low b/t ratios. However, in developing plastic hinge in the girder, significant local buckling damage occurs as shown in Figure 2.8. Such severe local bucklings will require repairs after a major earthquake. In addition, it is not clear if a girder with severe local buckling can carry its gravity load after a major earthquake. If the objective of design is for the structure to survive a major earthquake and then the locally buckled areas be repaired, then formation of plastic hinge and severe local buckling in the girder can be justified. However, such design philosophy can result in closure of the building after a major earthquake and can result in high repair costs.

The above issue of damageability of a structure is not limited to steel moment frames. Most other structures including the reinforced concrete structures will sustain severe damage after a major earthquake and will require repairs. However, notice that by using the top-and-bottom plate connections,as discussed earlier,severe local buckling can easily be avoided. Figure 2.2 shows a typical top-and-bottom plate connection at the end of the test with almost no visible damage. The only damage to the structure is yielding of connection elements.

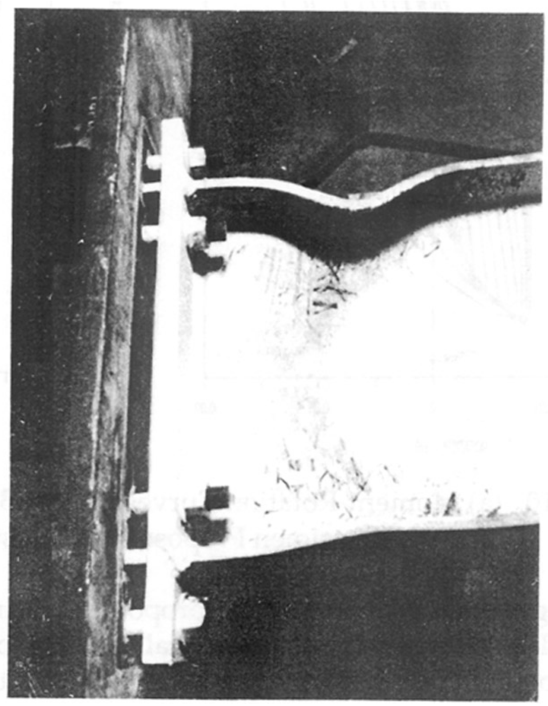


Figure 2.11. Prestressed End Plate Specimen Proposed by A. Astaneh-Asl in 1986 at the End of the Tests (Astaneh-Asl, 1986c)

2.7. Shaking Table Tests of Rigid , Semi-rigid and Flexible Frames

In 1988 a series of 51 shaking table tests were conducted to study the behavior of welded and bolted, rigid, semi-rigid and flexible (simple) steel frames (Nader and Astaneh-Asl, 1991). A one-story one-bay steel frame, shown in Figure 2.12, was constructed such that the beam-to-column connections could be replaced. Three types of connections, flexible, semi-rigid and rigid, were used resulting in flexible, semi-rigid and rigid frames, Figure 2.12.

The structure with three types of connections, one type at a time, was subjected to various levels of ground motions simulating 1940-El Centro, 1952-Taft and 1987-Mexico-City earthquake records. A total of 51 shaking-table tests was conducted. The results of one series of tests, when rigid, semi-rigid and flexible structures were subjected to the Taft earthquake with maximum peak acceleration of 0.35g are summarized and discussed. More information on the shaking table tests can be found in the report (Nader and Astaneh-Asl, 1991).

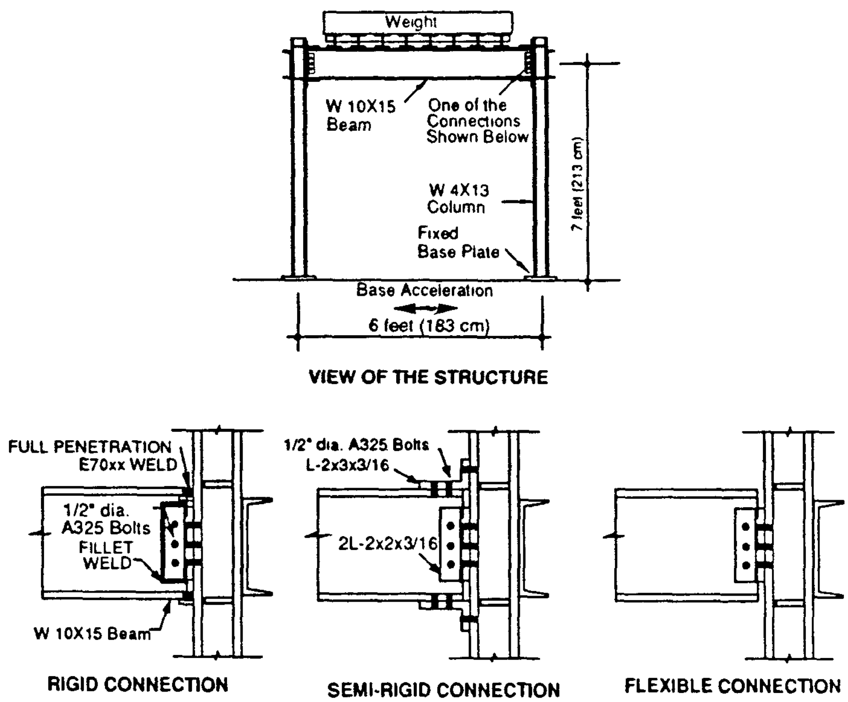


Figure 2.12. Shaking Table Test Frame and Three Types of Connections Used (Nader and Astaneh-Asl, 1991)

Figure 2.13 shows the base shear-lateral drift response of three frames. The frames showed almost an "equal displacement" response. The rigid frame behaved almost elastically. The semi-rigid frame behaved in very ductile manner, developed smaller base shear than the rigid frame but had slightly larger displacement. The behavior of the flexible frame was also stable and ductile with no traceable P-Δ effects. Figure 2.14 shows examples of moment-rotation response of connections in rigid, semi-rigid and flexible moment frames.

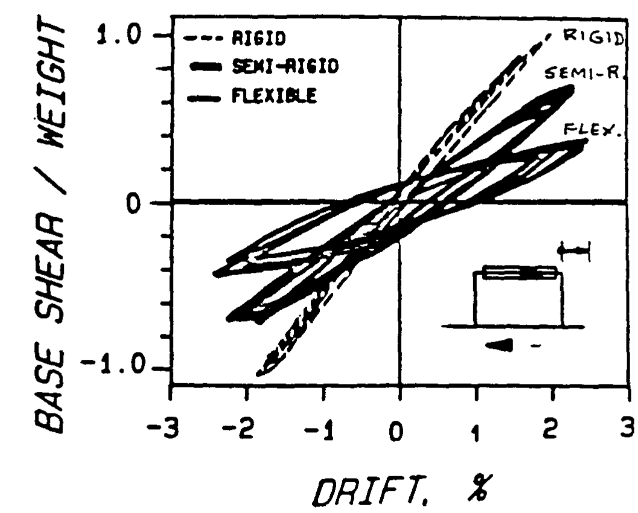


Figure 2.13. Base Shear versus Lateral Displacement Response
(Nader and Astaneh-Asl, 1992)

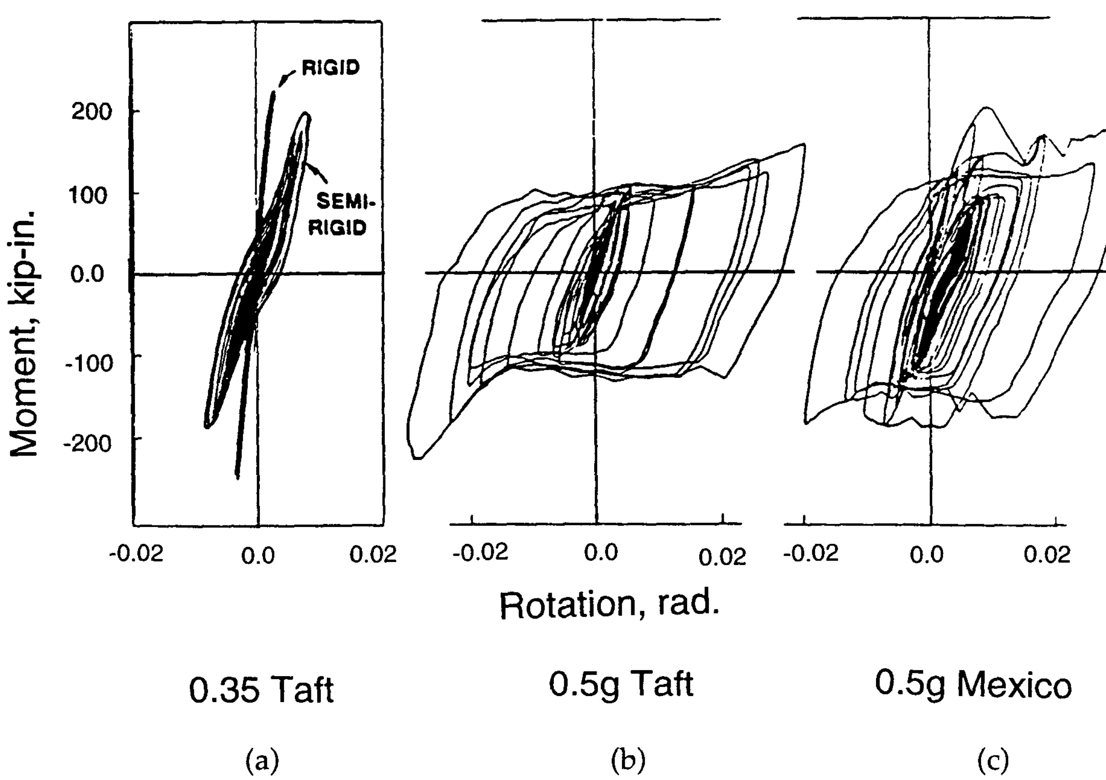
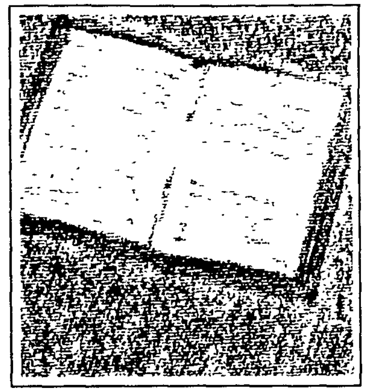


Figure 2.14. Moment-Rotation Behavior of Connections (Astaneh-Asl and Nader, 1991)
(a) Response of Rigid and Semi-rigid Connections to 0.35g Taft Earthquake
(b) Response of Semi-rigid Connections to 0.5g Taft Earthquake
(c) Response of Semi-rigid Connection to 0.5g Mexico-city Earthquake

3. CODE PROVISIONS ON BOLTED STEEL MOMENT-RESISTING FRAMES



3.1. Introduction

Seismic design codes have a number of provisions applicable to bolted moment frames. In this chapter, some of the provisions in the Uniform Building Code (ICBO, 1994) that directly relate to seismic design of bolted steel moment-resisting frames are discussed.

3.2. Special and Ordinary Moment-Resisting Frames According to the Uniform Building Code (1994)

The Uniform Building Code (ICBO, 1994) defines special and ordinary moment frames as follows:

"Special Moment-Resisting Frame is a moment-resisting frame specially detailed to provide ductile behavior and comply with the requirements given in Chapter 19 [reinforced concrete] or 22 [steel]

Ordinary Moment-Resisting Frame: is a moment-resisting frame not meeting special detailing requirement of ductile behavior. "

Chapter 22 of the Uniform Building Code (ICBO, 1994) provides more information on the design and detailing of the Special Moment-Resisting Frames in Seismic Zones 3 and 4 and Seismic Zones 1 and 2. Some of the important requirements affecting the design of connections in Seismic Zones 3 and 4 are discussed in the following. For a full text of the UBC-94 requirements, the reader is referred to the Uniform Building Code (ICBO, 1994) and its Emergency Changes implemented after the 1994 Northridge earthquake.

3.2. Provisions in UBC on Bolted Special Steel Moment Frames

The Uniform Building Code, UBC-94, has the following provision regarding strength of girder-to-column connections in special moment-resisting frames (SMRF), including bolted special moment-resisting frames.

> "**Sec. 2211.7.1.1 Required strength.** The girder-to-column connection shall be adequate to develop the lesser of the following:
>
> 1. The strength of the girder in flexure.
> 2. The moment corresponding to development of the panel zone shear strength as determined from Formula (11-1).
>
> **EXCEPTION:** Where a connection is not designed to contribute flexural resistance at the joint, it need not develop the required strength if it can be shown to meet the deformation compatibility requirements of Section 1631.2.4. "
>
>

The Formula (11-1) in Part 2 above is given as the following in UBC-94:

$$V = 0.55F_y d_c t\left[1+\frac{3b_c t_{cf}^2}{d_b d_c t}\right] \qquad \text{(Formula 11-1 of UBC-94)} \qquad (3.1)$$

The EXCEPTION in the above UBC provision is primarily for shear and semi-rigid connections that are not considered in design as part of the lateral-load resisting system. Section 1631.2.4 of the UBC-94 (ICBO, 1994) has the following provisions on the issue:

> **Sec. 1631.2.4 Deformation compatibility.** All framing elements not required by design to be part of the lateral-force-resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity when displaced $3(R_W/8)$ times the displacement resulting from the required lateral forces. $P\ \Delta$ effects on such elements shall be accounted for. "
>
>

The first and second printing of the Uniform Building Code (ICBO, 1994) in its Section 2211.7.1.3 has provisions permitting the use of "Alternate" connections which includes bolted special moment-resisting frame connections. In the aftermath of the 1994 Northridge earthquake and damage to welded special moment frame connections, the ICBO Board of Directors on September 14, 1994 approved the following emergency code change. The following text is from Reference (Building Standards, 1994):

1994 UNIFORM BUILDING CODE™, VOLUME 2

Sec. 2211.7.1.2, page 2-361. Delete the entire section.

Also:

Sec. 2211.7.1.3, page 2-361. Renumber and revise the section as follows:

Sec. 2211.7.1.~~3~~2 ~~Alternate~~ ~~c~~Connection strength. Connection configurations utilizing welds or high-strength bolts ~~not conforming with Section 2211.7.1.2. may be used if they are shown~~ shall demonstrate , by approved cyclic test results or calculation, the ability to sustain inelastic rotation and ~~to meet the~~ develop the strength criteria in Section 2211.7.1.1 considering the effects of steel overstrength and strain hardening. ~~Where conformance is shown by calculation, 125 percent of the strengths of the connecting elements may be used."~~

(Note: The strike-through texts are deleted and the underlined texts are added, both by the ICBO.)

Procedures for seismic design of the special bolted moment frames are presented in Chapter 4 of this report. The procedures are based on the results of cyclic tests of bolted moment frame connections. The test procedures and results are summarized in Chapter 2 of this report. The test specimens, satisfied the overstrength and strain hardening of the beam stipulated in the above changes. The beams for specimens were ordered to be A36. However, the coupon tension tests of the girder flange indicated a yield stress of 57 ksi. As a result, almost the entire rotational ductility of the bolted connections that were tested, came from the connection. The girders because of their high yield point did not yield and did not contribute to the ductility. Even with girders remaining almost elastic, the rotational ductility of the bolted moment connections that were tested was in excess of 0.03 radian.

As indicated above, the provisions regarding design of welded rigid moment connections in special moment frames have been revised significantly since the 1994 Northridge earthquake (ICBO, 1994). With the revisions of seismic design procedures for welded moment frame connections, the cost of fabrication, erection, field-welding, quality control and inspection of welded special moment frames has risen significantly. As a result, bolted special moment-resisting frames, the subject of this report, have become more economical. In particular, bolted special moment frames show great potential and economy for use in low- and medium-rise space moment frames and perimeter moment frames.

3.3. Lateral Forces for Seismic Design

The minimum forces and other requirements to be considered in seismic design of the steel bolted moment frames are those provided by the governing code for "Special Steel Moment-Resisting Frames". The Uniform Building Code (ICBO, 94) has provisions for establishing minimum equivalent *static* and more realistic *dynamic* seismic forces. The code also provides guidelines on when the two, static or dynamic force procedures, can or cannot be used. In general, in current practice, where the structure is not taller than 240 feet and is not irregular, the static force method is used to establish equivalent seismic lateral forces. For taller and irregular structures the UBC requires the use of dynamic force procedures.

In this section selective parts of the Static Load Procedure of the Uniform Building Code (ICBO, 1994) relevant to special bolted moment frames are discussed. The excerpts from the UBC are provided here only for discussion purposes. The actual seismic design should be done by proper use and interpretation of the Uniform Building Code itself by a competent professional engineer.

In UBC, the base shear is established as:

$$V = \frac{ZIC}{R_w} W \tag{3.2}$$

$$C = \frac{1.25S}{T^{2/3}} W \tag{3.3}$$

According to UBC-94, the value of C need not exceed 2.75 and may be used for any structure without regard to soil type or structural period. The minimum value of C/Rw is limited to 0.075 except for provisions, such as lateral drift check, where code forces are scaled-up by 3(Rw/8).

The Uniform Building Code (ICBO, 1994) permits calculation of T, the fundamental period, from one of the following methods A and B:

Method A: For all buildings, the value of T may be approximated from the following formula:

$$T = C_t(h_n)^{3/4} \tag{3.4}$$

where C_t is a constant for steel moment frames given as 0.035 in UBC-94 and h_n is the height of the building in feet.

Method B: In this method, the fundamental period T is calculated using the structural properties and deformational characteristics of the structural elements and using a more precise analysis

The reduction parameter R_W represents the performance and damageability of the structure. Depending on the seismic performance and ductility of the common structural systems, appropriate reduction factors have been established. For steel special moment frames, the Uniform Building Code specifies an R_W of 12.

Since the 1994 Northridge earthquake and damage to some of the welded special moment frames, some concern has been expressed whether R_W of 12 is appropriate for the welded moment frames. In Europe and Japan, smaller reduction factors are used in seismic design of all structures. At this writing, the profession is studying the damage to steel welded moment frames and the main cause of damage in steel moment frames has not been established.

The value of R_W for any structural system is directly related to the amount of inelasticity (damage) that will occur in the system. A higher value of R_W is an indicator of a higher amount of inelasticity (yielding damage). The current philosophy of seismic design codes is based on achieving life safety and preventing collapse. The current values of R_W have proven to be able to achieve the life safety criterion in steel buildings since there has been no partial or no full collapse of special steel moment frames during the 1994 Northridge earthquake. However, since there has not been a very strong earthquake in the United States to shake the modern steel or reinforced concrete structures, it is not clear whether all structures designed using an R_W of 12 will survive such a quake without collapse.

It is the opinion of the author that a systematic study of the Reduction Factor based design and of values of R_w for all structural systems in steel, reinforced concrete and composite construction needs to be conducted. The current R_w values in the codes have evolved primarily from experience of the performance of structures during past earthquakes and the intuition of engineers involved in developing the code procedures. The recent earthquakes, particularly the 1994 Northridge and the 1995 Great Hanshin earthquakes, have clearly indicated that there is a need to revisit some of the basic concepts in seismic design including Rw's.

A limited study of R_w as part of a larger study of the performance of steel moment frames (Nader and Astaneh-Asl, 1992) indicated that instead of an R_w of 12, a value of R_w of about 9 is more justified for use with currently designed and constructed special moment frames. It should be noticed that the implication of using a higher R_w is to have less initial cost of construction but, most likely, heavy damage and higher cost of repair after a severe earthquake. The impact of this trade-off needs to be systematically studied and optimum values of R_w need to be established. However, until the Uniform Building Code changes any values of R_w, the values given in the code need to be considered as the maximum R_w's to achieve minimum design loads.

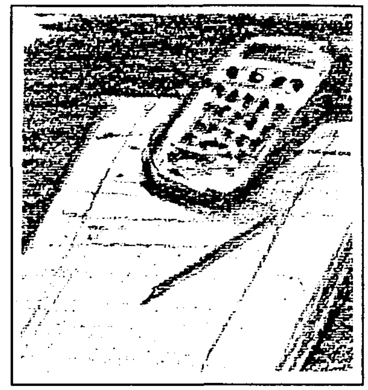
For bolted special steel moment-resisting frames, because of their high ductility, there is no reason not to use an R_W of 12 provided that the bolted connections be designed to have the high ductility observed in the test specimens presented in Chapter 2. The procedures to design the bolted connections of the bolted special steel moment-resisting frames are presented in Chapter 4.

Therefore, for bolted steel special moment frames:

$$R_w = 12 \tag{3.5}$$

After establishing base shear, the procedures given in Section 1628.4 of the Uniform Building Code (ICBO, 1994) can be used to distribute the base shear over the height of the building.

4. SEISMIC DESIGN OF BOLTED MOMENT-RESISTING FRAMES



4.1. Introduction

Seismic design of bolted MRFs is similar to seismic design of welded MRFs. First, seismic lateral loads need to be established. This was discussed in the previous chapter. Second, seismic forces in combination with gravity loads are applied to a realistic model of the structure and by analyzing the structure component forces and nodal displacements are calculated. Finally, the components (members) and connections are designed to ensure that they have sufficient strength, stiffness and ductility for the applied forces and that the displacements of the structure do not exceed the permissible limits.

In bolted moment connections, depending on the connection details, slight slippage and gap-opening can occur. Such minor displacements are not expected to change the seismic behavior of rigid moment connections in an adverse manner. In fact, the available data indicates that such minor movements and release of stiffness in the connection can be beneficial in improving overall seismic behavior. To satisfy serviceability requirements, it is suggested that slippage and gap-openings be avoided under the service loads.

4.2. Connection Design Philosophy in Special Moment-Resisting Frames

According to current codes, UBC-94 (ICBO, 1994) and AISC Specification (AISC, 1993), for special moment resisting frames, girder-to-column connections should be designed to develop at least the bending strength of the connected members, or to have sufficient ductility if it can be shown by laboratory tests. However, currently, there is no well established definition of "sufficient ductility".

Traditionally, ductility of a steel moment connection is measured by cyclic moment rotation tests. In the past, some researchers had proposed that if a connection can reach a rotation of 0.02 radian under cyclic loading, the connection is sufficiently ductile (Popov et al., 1993). Others, including the author, have established that for a connection to be considered sufficiently ductile, it should be able to reach at least 0.03 radian rotation under cyclic loading (Nader and Astaneh-Asl, 1992). In addition, based on experimental and analytical studies, it was suggested that the cumulative inelastic rotation under cyclic loading should be at least 0.1 radian (Nader and Astaneh-Asl, 1992). Three recent studies of the behavior of steel rigid moment frames (Englekirk, 1994; D'Amore and Astaneh-Asl, 1995; Astaneh-Asl, et al., 1995) confirm that the moment connections should have sufficient ductility to tolerate 0.03 radian rotation without fracture.

To satisfy the general equation of design: *Capacity* $\geq$ *Demand*, the rotational ductility of a moment connection should be greater than the rotational demand. However, establishing a realistic value for cyclic rotational demand has proven to be a complex matter. This is due to many uncertainties regarding the future ground motions, complexity of the inelastic seismic behavior of the structures and a lack of sufficient research data on cyclic behavior of many connections.

Traditionally, ductility of the moment connections is measured in terms of rotational ductility. However, it is not clear, at least to the author, if defining ductility of a moment connection in terms of its rotational capacity is the most rational way. It appears that a criterion based on the magnitude of local strain in the welds or steel would be more appropriate. After all, it is the local ductility of the weld or steel that, if exhausted, will result in the initiation and propagation of the fracture cracks. To clarify the point consider two moment connections which have beams with different depths. If both connections are subjected to the same rotations, the local strain in the welds in the deeper beam will be larger than the strain in the welds of the smaller beam.

In the absence of a well-defined, reliable and universally accepted criterion to establish ductility demand, one rational approach is to focus on increasing the ductility supply of the connection. With the significant uncertainties that currently exist with regard to the characteristics of future earthquakes and their effects on the structure, the increased supply of ductility, above and beyond any specified demand (such as 0.03 radian) can improve the seismic performance of the structure significantly.

To increase supply of ductility, the *ductile failure modes,* such as limited friction slip, yielding of steel and minor local buckling, should be made the governing failure modes. The occurrence of brittle failure modes, such as fracture of welds and bolts or fracture of the net section of the members should be delayed and if possible prevented altogether. In the following section, the seismic design philosophy of avoiding brittle fracture modes and its implementation in design of bolted steel momnt-resisting frames is discussed in more detail.

4.4. Proposed Design Criteria for Bolted Connections in Special Steel Moment-Resisting Frames

In design of connections in seismic areas, three issues need to be addressed: (a) stiffness, (b) strength and (c) cyclic and cumulative ductility.

4.4.a. Stiffness of Bolted Moment Connections

The initial rotational stiffness of the connection relative to the girder should be large enough so that *the girder span* is categorized as rigid. With reference to Chapter 1, this requirement is satisfied if:

$$\frac{(K)_{con}}{(\frac{EI}{L})_g} \geq 18 \tag{4.1}$$

where $(K)_{con}$ and $(EI/L)_g$ are rotational stiffnesses of the connection and girder, respectively.

4.4.b. Strength of a Bolted Moment Connection

Shear Connection of the Web: Currently, shear connections on the girder webs of the moment connections are designed to resist the gravity load acting in pure shear. This is in accordance with the traditional division of forces in the connection that assigns shear to the web and bending moment to the flanges. Because of the high ductility of steel as a material, and from the application of the Upper Bound theorem of plasticity, such assignment of forces makes the design simple and has worked satisfactorily in the past. However, in seismic design, particularly in seismic Zones 3 and 4, the connections can be pushed to their limit during major earthquakes and can develop damage. In such situations some parts of the connection might fail and other parts might then have to bear the load of the failed part and prevent collapse under the gravity load.

To increase the ductility of connections and the chance of survival and to avoid catastrophic collapse, the following suggestions are made for seismic design of shear plate connections in moment-resisting frames:

1. Design the shear plate to develop shear yield capacity as well as plastic moment capacity of the girder web. The suggested criteria can be written as:

$$h_p t_p (0.6 F_{y_p}) \geq h_{gw} t_{gw} (0.6 F_{y_g}) \tag{4.2}$$

$$h_p^2 t_p (F_{y_p}) \geq h_{gw}^2 t_{gw} (F_{y_g}) \tag{4.3}$$

The dimensions in the above formulae are shown in Figure 4.1. The yield stresses to be used in the above balanced-strength equations should be realistic yield stresses and not the nominal specified. For example for dual-strength A36 steel girder the higher yield stress should be used.

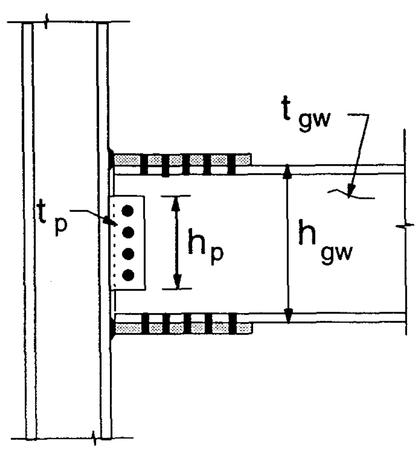


Figure 4.1. A Bolted Moment Connection

2. In seismic design ensure that the governing failure mode is yielding of the shear plate and not shear fracture of the bolts or fracture of the net area of the shear plate or girder web. The failure modes of shear plate connections have been studied in recent years (Astaneh-Asl, et al, 1989) and design procedures have been developed that are currently incorporated into the

AISC Specifications and the Manual (AISC, 1994). If one follows the procedures and tables in the AISC Manual (AISC, 1994 and 1992), the shear plate is expected to behave in a ductile manner and the failure mode is by design yielding of the steel. Caution should be exercised here since because of availability of high yield A36 steel, it is possible that in the actual structure, the desired yield failure mode may not occur. To ensure yielding of plate, the realistic yield stresses of material should be used in the design.

3. It is suggested here that the depth of shear plate be made almost equal to the clear depth of the web of the girder. In doing so it will be easier to satisfy the suggested criteria in Item 1 above. In addition, the full depth shear plate can result in increasing the participation of the girder web in developing its share of the plastic moment capacity.

4. In seismic Zones 3 and 4, it is suggested that the shear capacity of the bolt group connecting the shear plate and girder web be equal or greater than 1.25 times the shear yield capacity of the shear tab or the girder web, whichever is smaller.

5. During the 1994 Northridge earthquake, a number of shear plates partially fractured. Even though the fractures did not result in collapse of any span, it is suggested here that until further research is conducted, fillet welds should not be used to connect shear plates to the web of the girders. To increase participation of the girder web, it appears that the use of deeper shear plates (see Item 3 above) bolted to the girder web is better than fillet welds.

Design of Flange Connections: According to the AISC Manual (AISC, 1994) in the design of bolted moment connections, the applied moment is divided by the depth of the cross section and the connections of girder flanges are designed for the force M/h. Following this method, in some way flanges are expected to carry the entire applied moment without any help from the web. Again, as mentioned earlier, in reality the web and flange elements will share the load based on their stiffness and strength. This separation of moment and shear- resisting elements in design has worked well in the past. However, for seismic design a more rational approach that more closely relates to the actual ultimate behavior is needed.

In seismic design, and to ensure ductility of the connection, the governing failure mode of flange connections should be ductile failure modes such as friction slip, yielding of steel and *very* minor local buckling. Failure modes such as fracture of welds or fracture of net areas should be avoided.

To increase the ductility of connections in bending and to avoid costly damage to connections, the following suggestions are made for seismic design of flange connections in bolted special moment frames:

1. Design the flange connections to develop axial yield capacity of the girder flange. Do not use connections that have yield strength significantly greater than the girder. Doing so can result in flange connections staying elastic and all the ductility demand expected to be supplied by the girder flange. The resulting inelasticity can cause severe cyclic local buckling and premature fracture. The suggestion can be written as:

$$b_p t_p (F_{y_p}) \cong b_f t_f (F_{y_g}) \tag{4.4}$$

2. In seismic design, it must be ensured that the governing failure mode is yielding of the steel and not fracture of the net area of the flange connection elements or fracture of the net area of the girder. With uncertainties regarding variation of the yield point of the specified steel and what is actually delivered and used, it is suggested at this time that the capacity of the net section of the girder flange in tension be 1.25 times the yield capacity of the flange calculated using the specified yield point (i.e. 36 ksi or 50 ksi).

3. In seismic Zones 3 and 4, it is suggested that the capacity of the bolt group connecting the flange elements to the column and the girder be equal or greater than 1.25 times the axial yield capacity of the flange. With the current uncertainty regarding variation of the yield point for steel, the 1.25 factor is proposed to ensure that even if the girder has a higher yield point than specified, the bolt fracture will not precede the yielding of the girder.

The current seismic design codes, UBC-94 (ICBO, 1994) and AISC Specifications (AISC, 1994) permit limited yielding of the panel zone in shear by specifying that the shear strength of the panel zone need not exceed that required to develop 80% of the moments developed by the girders framing into the column flanges. In some cases during the 1994 Northridge earthquake, cracks that apparently initiated in the welds, propagated into the panel zones. At this time, the cause of cracks in welded connections is not well understood and the issue of permitting limited yielding in the panel zone of welded moment connections remains to be re-examined.

In the author's opinion, in bolted moment connections, it is relatively easy to design the connection to be able to supply all the ductility demand of the joint by yielding of connection elements outside the column while maintaining an almost elastic column. Therefore, until more information on the behavior of panel zones during the Northridge earthquake becomes available, it is prudent to design panel zones to remain elastic and confine almost all the yielding to the beam-to-column connection area and the girder flange outside the column panel zone.

4.4.c. Design and Detailing to Achieve Sufficient Ductility

To ensure ductility of a steel connection, all failure modes should be identified and divided into two categories: ductile and brittle. Then the seismic design of the connection should be done such that the ductile failure modes govern the design. A suggestion to achieve this is to design for the capacity of the brittle failure modes to be 1.25 times the capacity of the ductile failure modes.

4.5. Ductile and Brittle Limit States (Failure Modes) in Seismic Design of Connections

In seismic design of steel components, failure modes are divided into ductile and brittle failure modes as discussed below.

Ductile Failure Modes: When a component of a steel structure reaches a ductile limit state, the stiffness of the component is reduced significantly, but the strength of the component continues to be, more or less, maintained. An example of ductile limit state, or ductile failure mode, is yielding of steel.

In seismic design of steel components the following failure modes are considered ductile:

- Controlled and limited friction slippage
- Yielding of steel; and
- Minor local buckling

Brittle Failure Modes: When a component of a steel structure reaches a brittle limit state, both the stiffness and the strength of the component are almost entirely lost. An example of brittle limit state is fracture of the welds or shear failure of bolts.

In seismic design of steel components the following failure modes are considered brittle:

- Fracture of weld

- Fracture of bolt under shear, tension or combination of shear and tension.
- Fracture of steel
- Severe local buckling, that deteriorates the material in a locally buckled area and rapidly leads to premature fracture.

Slippage of the bolted components results in temporary loss of stiffness. Such temporary loss of stiffness can be used to work as a fuse during earthquakes. By designing the bolts to slip under a pre-determined level of force, the bolted connection can act as a fuse and limit the force that is transmitted through the bolts. In addition, the friction slippage results in significant energy dissipation and damping. Because of the relatively large number of connections in bolted moment-resisting frames, such slippage can occur in many locations dissipating the energy in a distributed and desirable manner without causing a single energy dissipating device to deteriorate.

For any bolted connection, before the bolts fail in shear, the connection needs to slip and engage the bolts and connected steel parts. Therefore, slippage of bolted connections subjected to shear is a natural phenomenon. The important question seems to be when is the best time to have friction slip. Of course slippage of bolts under service loads cannot be accepted. If slippage occurs under a force level close to the shear failure capacity of the bolts, because of high elastic stiffness up to the slippage, a large amount of strain energy is already in the structure. When slippage occurs under such large energy, from the resulting impact and the fact that the slippage force is too close to the fracture capacity, the bolts can fail in shear. To safeguard against such failures and to satisfy serviceability, the following criteria for bolt slippage under seismic loads are suggested:

$$1.25F_{Service} \le F_{Slippage} \le 0.80F_{Ultimate} \tag{4.5}$$

where:

$F_{Service}$= Applied shear force due to service (unfactored) code specified load combinations
$F_{Slippage}$ = Force that can cause friction slippage, calculated using AISC specified bolt pretension and the AISC specified friction coefficients, see LRFD Specification for Structural Joints Using ASTM A325 or A490 Bolts (AISC, 1994)
$F_{Ultimate}$ = Specified shear strength of the bolt (AISC, 1994)

The 1.25 and 0.80 factors in the above equation are introduced to provide a reasonable margin of safety against slippage under the service condition as well as to guard against slippage occurring too close to the ultimate capacity. Unfortunately test results on cyclic slippage behavior of steel structures are very limited. As a result, the reader is cautioned that the above limits of 1.25 and 0.8 are selected primarily based on the basis of engineering judgment and intuition, and are therefore, subject to the judgment and approval of the structural engineer in charge of the design. Figure 4.2. shows the slippage behavior of bolted connections and the suggested criteria.

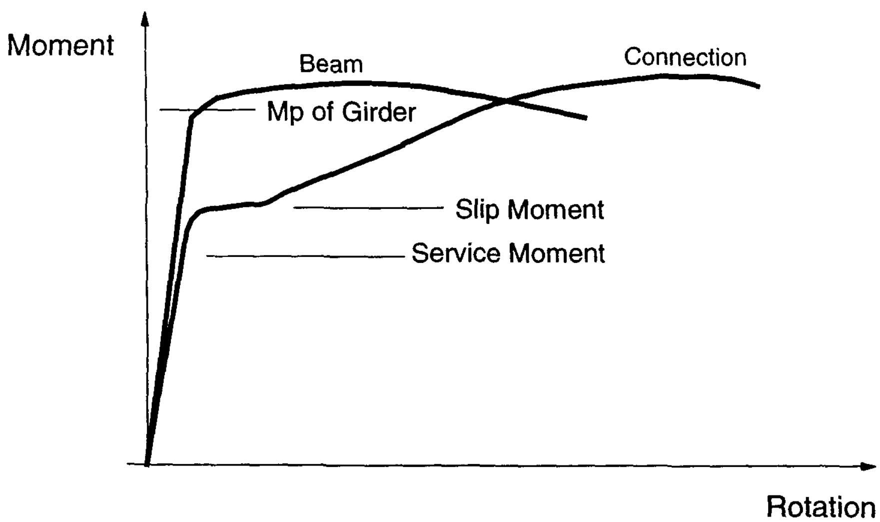


Figure 4.2 Slippage Behavior of Bolted Moment Connections

Local buckling cab be categorized as ductile or brittle depending on how rapidly the locally buckled area deteriorates during cyclic loading. Available cyclic test results indicate that steel members with high b/t ratios, say higher than λ_r given in the AISC Specifications (AISC, 1994), tend to form local buckling in a very sharp configuration, develop relatively large lateral displacements and fracture through the sharp tip of the locally buckled areas after a few inelastic cycles. Cyclic local buckling in this manner should be considered brittle. The value of λ_r suggested for the flanges of the girders in special moment-resisting frames is $95 / \sqrt{F_y}$. On the other hand, members with a b/t ratio less than those specified by the AISC Seismic Provisions (AISC, 1993) tend to develop local buckling after a relatively large number of inelastic cyclic deformations (usually more than 10 to 15 cycles of inelastic behavior before local buckling). The limit for the b/t ratio for the flanges of the girders currently given in the AISC Seismic Provisions (AISC, 1993), is $52 / \sqrt{F_y}$.

In addition, when the b/t ratio of the flange is less than $52 / \sqrt{F_y}$, the locally buckled area does not develop a sharp tip. These members can be considered sufficiently ductile.

For members with b/t ratios greater than $52/\sqrt{F_y}$ and less than $95/\sqrt{F_y}$ there is not sufficient data on their low-cycle fatigue behavior to result in a clear conclusion. In a conservative move and until more test data becomes available, cyclic local buckling of the members with b/t ratio between $52/\sqrt{F_y}$ and $95/\sqrt{F_y}$ can be considered nonductile (brittle) in seismic Zones 3 and 4 and sufficiently ductile for seismic Zones 1 and 2.

The following guidelines, which are based on the monotonic and cyclic local buckling behavior of steel members, are conservatively suggested by the author to be used to categorize local buckling failure modes as ductile or brittle in seismic Zones 3 and 4:

If b/t < 0.80 λ_p, behavior is ductile, otherwise behavior is considered to be nonductile (brittle)

where λ_p is the limit for the b/t ratio for plastic design of steel structures given in Table B5.1 of the AISC Specification (AISC, 1994). The table gives value of λ_p for flanges of rolled wide flange shape as $65/\sqrt{F_y}$.

In the following section, specific design procedures are proposed to achieve the above criterion.

4.6. Seismic Design Procedures for Bolted Top- and Bottom-Plate Moment Connections

Figure 4.3. shows a top- and bottom- plate bolted connection proposed for use in bolted special moment-resisting frames. The girder flange connection consists of two plates welded to the column in the *shop* with a full penetration weld. The web connection consists of a shear tab fillet welded to the column in the *shop* also. After planting the columns in the field, the girders are bolted to the flange and web plates using slip-critical high-strength A325 or A490 bolts (A325 is preferred in seismic Zones 3 and 4).

Failure modes of this connection have been identified (Harriott and Astaneh-Asl, 1990; Nader and Astaneh-Asl, 1992) as given in the following list. The list is in the order of desirability of the failure mode with most ductile and desirable failure mode being listed first and the most brittle and undesirable mode listed last. The list might appear long and give the impression that in order to design bolted connections many failure modes need to be checked. Although this might be true in some cases, the following list includes *all* possible failure modes of bolted connections and some of them are included for completeness.

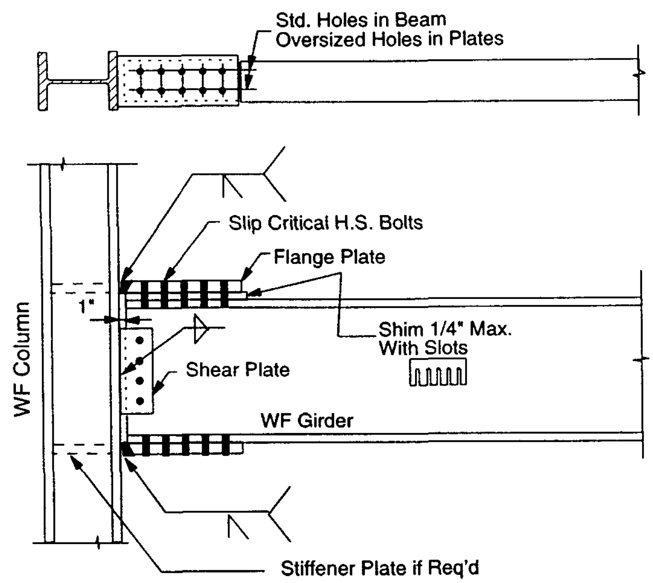


Figure 4.3. A Typical Top- and Bottom-Plate Moment Connection

The possible failure modes of a bolted top and bottom flange plate moment connection are:

Ductile Failure Modes for Flange Connections:

a. Slippage of the flange bolts
b. Yielding of the gross area of the top and bottom flange plates
c. Bearing yielding of the bolt holes in the girder flanges and the flange plates
d. Yielding of the gross area of the girder flange

Failure Modes with Limited Ductility for Flange Connections:

e. Local buckling of the top and bottom flange plates
f. Local buckling of the girder flanges
g. Shear yielding of the panel zone of the column

Brittle Failure Modes for Flange Connections:

h. Fracture of the edge distance or bolt spacing in the flange plate
i. Block shear failure of the top and bottom flange plates
j. Fracture of the net section of the flange plate
k. Fracture of the edge distance or bolt spacing in the girder flanges

l. Block shear failure of the girder flanges
m. Shear fracture of the flange bolts
n. Fracture of the welds connecting the top and bottom plates to the column
o. Net section fracture of the girder flanges

Ductile Failure Modes for Web Connections:

p. Various failure modes of the shear connection of the web

In the above list, failure modes (a) through (d) are considered ductile and desirable. Failure modes (e) and (f) are considered ductile provided that b/t ratios satisfy the limit given in Section 4.5 above. The panel yielding (g) is considered ductile if panel zone design satisfies the requirements of the Uniform Building Code (UBC, 1994). Failure modes listed as (h) through (o) are considered brittle and not acceptable to govern the strength of the bolted special moment-resisting frames. Figure 4.4 shows the above failure modes and their desirability as the governing failure mode.

Failure mode (p) in the above list presents failure of the shear connection which is responsible for carrying the gravity load after the quake. Because of the importance of shear connections in carrying the gravity load, brittle failure of the shear connection is considered catastrophic and listed as the most undesirable (unacceptable) failure mode. The reader is referred to References (Astaneh-Asl et al., 1989) for information on ductile design of shear connections.

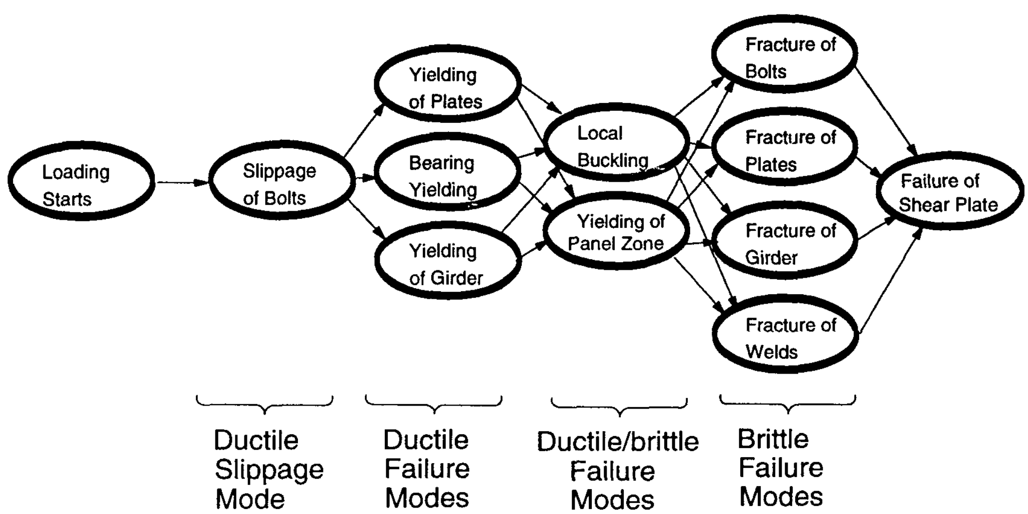


Figure 4.4. Failure Modes of Top and Bottom Flange Plate Connections

Also, the reader is reminded that because of the good performance of the shear plate connections there has been no published report of the collapse of any span during or after the 1994 Northridge earthquake. Even in structures with extensive cracking of the welds and other areas of the connections, and reports of some partial cracking of the shear plates or shear failure of some bolts, the shear plates were able to carry the service gravity load and prevent the collapse of the spans.

4.6.a. Slippage of Flange Bolts

Comprehensive information on the slip behavior of bolted connections has been given by Kulak et al. (1987) and in the AISC Manual, Volume II (1994). The important issue for bolted special moment connections, with regard to slippage, is should bolted connections in special moment frames be permitted to slip, and if slippage is permitted at what level of load should slippage be designed to occur?

From available test results on the cyclic slip behavior of bolts in shear, it is clear that controlled and limited slippage of high-strength bolts is a desirable phenomenon during severe earthquakes. As a result of slippage, the stiffness of the structure decreases, the period elongates and the energy dissipation and damping increase all of which, in general, result in a reduction of the dynamic response of the steel structure to ground motions. More important perhaps, even small slippage of the bolts in moment connections increases the rotational ductility significantly. This is shown in Figure 2.4, where because of slippage of the bolted moment connection, its ductility was increased significantly compared to welded moment connection. In addition, a literature survey of the issue did not reveal any report on adverse effects on seismic behavior of steel structures from slippage of bolted connections.

One of the concerns expressed by some structural engineers, regarding bolt slippage is that if bolted moment connections are permitted to slip, such a slip will make the structure more flexible and can result in development of larger drifts than for non-slip connections. Later in this chapter, some suggestions are made on how to incorporate stiffness of the connection into a computer model of the frame to calculate more realistic drift values. In general, the slippage of bolts in standard round holes is not expected to result in changes of any consequence in drift values.

Therefore, it is recommended that in the design of bolted steel moment connections, slippage of the bolts be permitted and incorporated into the design as a useful phenomenon to improve seismic behavior of the structure.

In incorporating slippage into seismic design, the question is when is the appropriate time for a moment connection to slip? In establishing appropriate slip moment capacity, M_{slip}, the following items need to be considered:

1. The bolted connection should not slip under the service loads. To be conservative, the slip moment greater than 1.25 times the moment in the connection due to service (not factored) loads is suggested. Therefore:

$$M_{slip} > 1.25M_{(service\ load)} \tag{4.6a}$$

2. The bolted connection should slip during moderate and strong earthquakes to reduce the stiffness, to increase ductility and to dissipate energy. On the basis of experience and intuition, it is suggested here that the slip moment be smaller than 0.8 times the plastic moment capacity of the girder.

$$M_{slip} < 0.80Mp_{(girder)} \tag{4.6b}$$

Without extensive data on this item, the structural engineer, knowing parameters of the design and the target performance, is the most qualified person to decide when bolted connections can be permitted to slip.

Combining the above two suggestions, the equation to establish slip moment is:

$$1.25M_{(service\ load)} \leq M_{slip} \leq 0.8Mp_{(girder)} \tag{4.7}$$

where

$M_{(service\ load)}$	= moment in the connection due to application of service loads
$Mp_{(girder)}$	= plastic moment capacity of the girder
M_{slip}	= moment that can cause slippage in the connection
	$= F_v A_b N d$
F_v	= nominal slip critical shear resistance (Table J3.6 of the AISC Spec., 1994)
A_b	= area of one bolt
N	= number of bolts in slip plane
d	= overall depth of girder

4.6.b. Yielding of Gross Area of Top and Bottom Plates

To increase ductility of the connection, yielding of top and bottom flange plates should be encouraged as the girder enters strain hardening. To achieve this, it is suggested that the plastic moment capacity of the connection should be close to or slightly greater than 1.25 times the plastic moment capacity of the girder, as expressed in:

$$Mp_{(plates)} \geq 1.25 Mp_{(girder)} \tag{4.8}$$

where

$Mp_{(girder)}$ = plastic moment capacity of the girder
$Mp_{(plates)}$ = moment causing yielding of the top and bottom plates
= $F_{yp}A_pd$
F_{vp} = minimum specified yield stress of the plates
A_p = gross area of one flange plate in the area between the first bolt line and the weld line.
d = back-to-back depth of girder

4.6.c. Bearing Yielding of Bolt Holes in Girder Flange and Plates

Bearing yielding of the bolt holes is beneficial in reducing seismic response during extreme events. It is suggested that in design the moment that can cause bearing yielding in the connection is equal to or slightly greater than 1.25 times the yield moment of the girder, as expressed in:

$$Mp_{(bearing)} \geq 1.25 Mp_{(girder)} \tag{4..9}$$

where:
$Mp_{(bearing)}$ = moment causing bearing yielding of the bolt holes
= $2.4F_{up}d_bNt$
F_{up} = minimum specified tensile strength of the plates
d_b = diameter of bolts
N = number of bolts
t = thickness of the plate or flange, whichever results in a smaller M_b.

4.6.d. Yielding of Gross Area of Girder

This failure mode occurs when a plastic hinge forms in the girder. This failure mode should be the target failure mode in the design of rigid connections. As indicated throughout this section, other failure modes are matched against this desirable failure mode.

The equation to establish plastic moment capacity of the girder is:

$$Mp_{(girder)} = F_yZ \tag{4.10}$$

where

$Mp_{(girder)}$ = plastic moment capacity of the girder
F_y = realistic minimum specified yield stress of the girder. For dual yield point A36, the higher yield value should be used in this context.
Z = plastic section modulus of the girder cross section

4.6.e. Local Buckling of the Top and Bottom Flange Plates

As discussed earlier in this document and by Astaneh-Asl and Harriott (1990), in bolted moment connections, the flanges of the girder and the plates brace each other to some extent delaying local buckling of the plate as well as the girder flange. The portion of the top and bottom flange plates between the first row of the bolts and the weld line is the most stressed region in compression and should be checked for buckling. This portion of a plate should be made as short as is practically possible. Considering clearances and the space needed around the bolts for tightening, the distance of the first row of bolts from the column face will be in the order of 4 to 5 inches in most practical situations. Longer spaces are not desirable since they can facilitate buckling of the plates during compression cycles and reduce the rotational rigidity of the connection. A shorter length for this portion can result in concentration of plasticity near or within the heat-affected zone resulting in premature fracture.

4.6.f. Local Buckling of Girder Flanges

As discussed earlier, if the b/t ratio of the girder flange is less than $52/\sqrt{F_y}$, local buckling of the girder flange will be sufficiently delayed during a cyclic event. When the cyclic local buckling occurs it will be relatively smooth and ductile without significant loss of strength.

4.6.g. Shear Yielding of Panel Zone

The Uniform Building Code permits limited yielding of the panel zones in special moment frames (UBC, 1994). The provisions of UBC state that the panel zone shear may be calculated by using 80 percent of moment capacity of the connected girders. Since some cracks have been observed in the panel zone in the aftermath of the 1994 Northridge earthquake, it is suggested that until the causes of these cracks are established the panel zone shear be calculated using 100 percent moment capacity of the connected girders.

The above suggestion is based on the fact that following these procedures, the bolted connections are designed to have a strength equal to 125 percent of the strength of the girder. Following the UBC provisions and designing the panel zone for a shear strength to develop 80 percent of girder capacity results in the panel zone having a shear strength of only 80/125= 64 percent of the connection strength. This will make the panel zone the weakest link in the system and cause its shear yielding to occur too early and to be too widespread. Such widespread yielding in the web of the columns cannot be desirable.

To protect the panel zone against extensive yielding, it is suggested that the panel zone shear capacity be at least equal to the shear that can be delivered to the panel zone by plastic moments of the girders:

$$V_n \geq \frac{\sum Mp_{girders}}{d_s} \tag{4.11}$$

where

V_n $= 0.55F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right]$

F_y = minimum specified yield stress of the plates
d_c = depth of the column
d_b = overall depth of the girder
t_p = total thickness of the panel zone
b_{cf} = width of the column flange
t_{cf} = thickness of the column flange
d_s = distance between the horizontal continuity plates (depth of panel zone).

As discussed in previous chapters, during the 1994 Northridge earthquake a number of panel zones fractured. These fractures have resulted in questions raised on the validity of the above equation in representing the actual behavior and capacity of the panel zones. Until the cause of panel zone fractures is established and a realistic design equation is developed (or the above equation is validated), the author suggests the use of equations that are given in the AISC-LRFD Specification (AISC, 1994). The equations are given for panel zone design when the effect of panel zone deformation on frame stability is not considered in the analysis. The equations from AISC-LRFD Specifications (AISC, 1994) are:

For $P_u \leq 0.4\, P y$

$V_n = \phi\, (0.60 F_y d_c t_p)$

For $P_u > 0.4\, P y$

$V_n = \phi\,(0.60 F_y d_c t_p)(1.4 - P u / P y)$

where

ϕ = reduction factor= 0.90
P u= axial tension or compression force in the column panel zone

4.6.h. Fracture of Edge Distance or Bolt Spacing in Plate

Fracture of edge distance by itself may not be catastrophic, but during cyclic loading a crack within the edge distance can jump the bolt hole and fracture the entire width of the plate. This behavior has been observed in past cyclic tests of bolted double-angle bracings (Astaneh et al, 1984)

On the basis of the limited information currently available on the cyclic behavior of bolt edge distances, it is suggested that in special moment frames bolt edge distances should not be less than 1.5 times the diameter of the bolt and preferably 2.0 times the diameter. In most bolted top and bottom connections, there is sufficient width of flange to accommodate easily an edge distance equal to two bolt diameters.

The bolt spacing, due to automation of drilling or punching is usually specified as 3 inches. In the absence of any report of failure of bolt spacing during earthquakes or in laboratory tests, it appears that 3 inch spacing is satisfactory.

4.6.i. Block Shear Failure of Top and Bottom Plates

Block shear failure is a fracture-yield type of failure where the boundary of a block of steel yields in some areas and fractures in the remaining areas. To ensure that this relatively brittle failure mode does not occur before the plates yield, the following condition is suggested:

$$\phi_n P_n \geq 1.25\,\phi\, M_p \;/\; d \tag{4.12}$$

where

ϕ_n = resistance reduction factor for fracture = 0.75
ϕ = resistance reduction factor for yielding = 0.90
d = depth of girder
P_n = nominal resistance of flange plate in block shear failure as given below:

(a) When $F_u A_{nt} \geq 0.6 F_u A_{nv}$

$$P_n = 0.6 F_y A_{gv} + F_u A_{nt} \tag{4.13}$$

(b) When $F_u A_{nt} < 0.6 F_u A_{nv}$

$$P_n = 0.6 F_u A_{nv} + F_y A_{gt} \tag{4.14}$$

A_{gv} = gross area subject to shear
A_{gt} = gross area subject to tension
A_{nv} = net area subject to shear
A_{nt} = net area subject to tension

4.6.j. Fracture of Net Section of Plate

The plates should be designed such that the fracture of plates does not occur before yielding and strain hardening of the girder. The following criterion is suggested:

$$\phi_n M_{pn} \geq 1.25 \phi M_p \tag{4.15}$$

where

M_{pn} = plastic moment capacity of the net section of the plates
= $F_y \, d \, A_{np}$
ϕ_n = resistance reduction factor for fracture =0.75
ϕ = resistance reduction factor for yielding =0.90
F_y = minimum specified yield stress of the plates
A_{np} = net area of one plate across the first bolt row
d = overall depth of girder

4.6.k. Fracture of Edge Distance or Bolt Spacing in Girder Flanges

Earlier in Section 4.6.h this issue was discussed for plates. The same discussion and recommendations apply to the girder flanges.

4.6.l. Block Shear Failure of Girder Flanges

Earlier in Section 4.6.i the issue of block shear failure of flange plates was discussed. For block shear failure of the flange itself the same discussion and equations as in Section 4.6.i apply.

4.6.m. Shear Fracture of Flange Bolts

This failure mode can occur when after slippage of the bolts and some bearing yielding, the applied moment is totally carried by the shear strength of the bolts. To encourage yielding of steel before bolt shear failure, the following criterion is suggested:

$$\phi_b F_b A_b N \; d \geq 1.25 \, \phi \, M_p \qquad (4.16)$$

where

ϕ_b = resistance reduction factor for fracture = 0.75
ϕ = resistance reduction factor for yielding = 0.90
F_b = shear strength of bolt
A_b = area of one bolt
d = overall depth of girder
N = number of bolts

4.6.n. Fracture of the Welds Connecting the Top and Bottom Plates to the Column

The welds connecting the top and bottom plates to the columns should be full penetration butt welds done in the shop following the provisions of the AWS-D1.1-94 Specifications (AWS, 1994) for design, quality control and inspection. A number of welds cracked during the 1994 Northridge earthquake. The exact cause of the cracks is still not known. However, there is no report of widespread damage to shop welds designed and fabricated following AWS requirements. Therefore, the shop welds connecting the flange plates to the column welds are expected to perform well and as a "matching" weld to develop the capacity of the plates.

4.6.o. Net Section Fracture of the Girder Flanges

If net sections of the flanges of the girder fracture, it is possible that the crack will propagate into the girder web. During or after the quake, the cracked web of the girder may not be able to carry the service gravity load and the crack could propagate across the entire section and result in the collapse of the span. Since such a scenario is not acceptable, fracture of the net section of the girder is considered very undesirable.

The Uniform Building Code (ICBO, 1994) in Section 2212 specifies that bolted flanges of girders in special moment frames satisfy the following requirement if Fu/Fy is less than 1.5.

$$\frac{A_e}{A_g} \geq \frac{1.2F_y}{F_u} \tag{4.17}$$

Currently , there is some uncertainty with regard to Fy and Fu for some A36 steel in the market. Therefore, it is suggested that the above requirement be applied to all cases regardless of the value of Fu/Fy.

To be consistent in providing an adequate margin of safety between yielding and fracture for all failure modes discussed here, it is suggested that the above equation be slightly modified as:

$$\frac{A_e}{A_g} \geq \frac{1.25F_y}{F_u} \tag{4.18}$$

4.6.p. Failure of Shear Connections

Failure modes of shear connections have been studied in recent years and reliable design procedures are available (AISC, 1994; Astaneh-Asl et al., 1989). The philosophy used in developing design procedure for shear plate connections has been to force yielding of steel to occur before fracture of the net area, bolts or welds (Astaneh-Asl et al., 1989). The concept is shown in Figure 4.5.

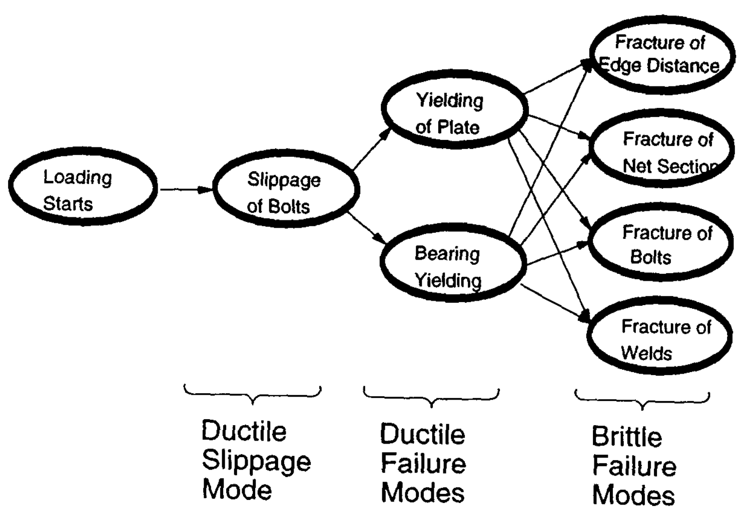


Figure 4.5. Failure Modes of Shear Plate Connections (Astaneh-Asl, 1989)

4.7. Establishing Stiffness of Top- and Bottom-Plate Bolted Moment Connections

4.7.a. Introduction

The difference between the rotational stiffnesses of a welded and a similar bolted connection is in the possibility of bolt slippage in the bolted connection. As discussed in previous chapters of this report, the slippage of the bolt is beneficial in providing damping, additional rotational ductility and redistributing the forces. If the design procedures outlined in previous sections are followed, the resulting bolted connection is expected to behave as a rigid connection, without bolt slippage, under the service load. However, during major earthquakes, it is expected that slippage will occur in bolted connections. The amount of slippage will be small and is expected to occur in a random manner among various bolted connections. However, if the structural engineer wishes to include the effects of bolt slip on the drift, the bolted connections can be modeled as rotational springs and be incorporated into the analytical model.

The bolted connections are small structures within the larger structure. In order to establish their stiffness one can model the connection elements, use powerful analytical methods such as Finite Element Methods and establish rotational stiffness. Or, in an approximate and more practical approach, the fundamental principles of mechanics of materials can be used to establish the rotational stiffness for use in design. If rotational springs are used in an elastic analysis of the frame, establishing the initial stiffness of the connection will be sufficient. If non-linear analysis programs are used, a bilinear moment-rotation curve will be necessary. It is suggested that for design purposes, the initial stiffness of the bilinear curve be the same as the elastic stiffness of the connection and the secondary stiffness be equal to 5% of the initial stiffness. The moment corresponding to yield point on the bilinear moment-rotation curve can be taken as equal to the Mp of the connection.

In the following a procedure is provided that can be used to establish initial elastic rotational stiffness of top- and bottom-plate moment connections.

4.7.b. Establishing Elastic Rotational Stiffness of Top- and Bottom-Plate Connections

Consider the top- and bottom-plate bolted moment connection. The moment rotation relationship for the connection is:

$$M_c = k_c \Theta_c \tag{4.19}$$

where M_c and Θ_c are the moment applied to the connection and the resulting rotation respectively. k_c is the elastic (initial) rotational stiffness of the connection.

Equation 4.19 can be rearranged and written in terms of axial displacement of the flanges:

$$k_c = \frac{M_c}{\Theta_c} = \frac{F_f h}{\Delta_f / (h/2)} = \frac{2F_f h^2}{\Delta_f} \tag{4.20}$$

In the above equation, the ratio F_f/Δ_f is the axial stiffness, k_f, felt by the girder flanges. The axial stiffness of the flange is provided by the flange plates and the friction slippage of the girder and plates. Assuming a shear slippage of about 1/16 inch the value of flange displacement will be:

$$\Delta_f = (\frac{F_f L_p}{A_p E}) + \frac{1}{16}(inch) \tag{4.21}$$

Using Equations 4.20 and 4.21, the rotational stiffness of the connection can be established.

In the above equations, A_p is the gross area of one flange plate, and E is the modulus of elasticity of steel, 29,000 ksi. The length L_p is the effective length of the bolted plate that can be considered fully loaded. It is suggested that the length be equal to 1/2 of the total length of the flange plate (Nader and Astaneh-Asl, 1992).

4.8. Seismic Design Procedures for Bolted Top- and Bottom-Angle Moment Connections

Figure 4.6 shows top and bottom bolted angle connections proposed for use in bolted special moment-resisting frames. The girder flange connection consists of two stiffened angles bolted to the column as well as to the girder.

Currently, the largest available rolled angle sizes are 14x14x1.4 inches rolled in Europe. Angle sizes of 10x10x1 inch and smaller are easier to obtain and to work with. In any event, if angles of large size are needed, such angles can be obtained by cutting WF or HP shapes. The web connection consists of a shear tab fillet welded to the column *in the shop* and bolted to the girder in the field. The bottom angles can be bolted to the column in the shop. After erecting the columns in the field, the girders are bolted to the shear tabs and bottom angles and then the top angle is bolted to the girder and the column.

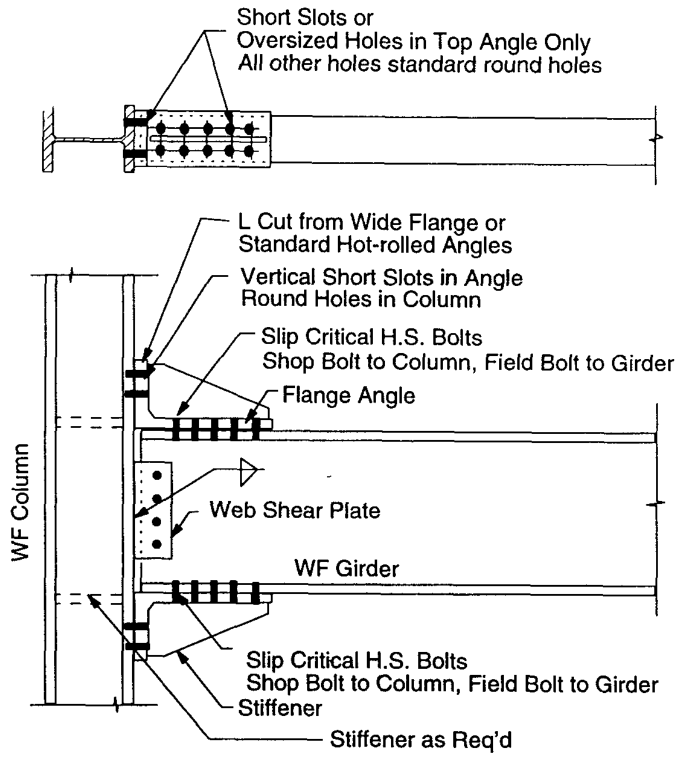


Figure 4.6. A Stiffened Bolted Top- and Bottom-Angle Moment Connection

The main failure modes of this connection are listed below. The list is in the order of desirability of the failure mode with the most ductile and desirable failure mode being listed first and the most brittle and undesirable mode listed last.

The main failure modes of a bolted top and bottom stiffened angle moment connection are:

Ductile Failure Modes for Flange Connections:

a. Slippage of the flange bolts
b. Yielding of the top and bottom angles
c. Bearing yielding of the bolt holes in the girder flanges and the angles
d. Yielding of the gross area of the girder flange

Failure Modes with Limited Ductility for Flange Connections:

e. Local buckling of the top and bottom angles
f. Local buckling of the girder flanges
g. Shear yielding of the panel zone of the column

Brittle Failure Modes for Flange Connections:

h. Fracture of the edge distance or bolt spacing in the angles
i. Block shear failure of the top and bottom angles
j. Fracture of the net section of the angles
k. Fracture of the edge distance or bolt spacing in the girder flanges
l. Block shear failure of the girder flanges
m. Shear fracture of the flange bolts
n. Tension fracture of the bolts connecting the angles to the column
o. Net section fracture of the girder flanges
p. Fracture of the welds connecting the angle stiffeners to the angles

Failure Modes for Web Shear Connection:

q. Various failure modes of the shear connection

In the above list, failure modes (a) through (d) are ductile. Failure modes (e) and (f) are considered ductile provided that b/t ratios satisfy the limit given in Section 4.5 above. Failure mode (g) is ductile if panel zone design satisfies the requirements of the Uniform Building Code (ICBO, 1994). Failure modes listed as (h) through (p) are considered brittle and not acceptable to govern the strength of the bolted special moment-resisting frames. Failure modes in Item (q) above are related to shear connections. These connections should be designed to survive earthquakes without failure since shear connections are needed to carry the gravity load after the quake.

Most of the above failure modes were discussed in the previous section and applicable design equations were provided. The same equations can be used for this connection. The only new failure mode for this connection is tension fracture of the bolts connecting the angles to columns, indicated as failure mode n in the above list. This failure mode is a brittle failure mode and needs to prevented until more ductile failure modes have occurred. To achieve this, as before, it is suggested that the strength of this brittle failure mode be made 1.25 times the strength of the beam in order to form a plastic hinge. Therefore:

$$\phi_b F_t A_b N h_b \geq 1.25 \phi M_{p(girder)} \tag{4.22}$$

where

F_t = tensile strength of bolts
ϕ_b = resistance reduction factor of fracture = 0.75
ϕ = resistance reduction factor of yielding = 0.90
A_b = area of one bolt
h_b = distance of C.G. of tension bolts from compression flange of the girder.
N = number of tension bolts.

If flange angles do not have stiffeners, the second row of bolts from the flange will not be as effective as the first row. Therefore, in calculating the number of tension bolts for unstiffened angles, 1/2 of the number of bolts in the second row should be considered.

4.9. Establishing Rotational Stiffness of Top- and Bottom-Angle Connections

Establishing the stiffness of top- and bottom-angle connections is much more complex than for top- and bottom-plate connections. The complexity arises from the two-dimensional plate bending of the vertical leg of the angle. However, by using stiffeners in the angles, it is expected that the vertical legs are very stiff and the bulk of connection flexibility is due to bolt slippage. As a rule of thumb, the angle-leg bending will be very small if the thickness of the angle leg is equal or greater than the diameter of the bolts. Therefore, for an approximation, the flexibility of the angle leg is ignored here and only bolt slippage is considered. As before, the moment-rotation relationship for the connection is given by Equations 4.19 and 4.20. The bolt slippage in Equation 4.20 is given as:

$$\Delta_f = \frac{1}{16} \mathit{inch}. \tag{4.23}$$

Using Equations 4.20 and 4.23, rotational stiffness of the connection can be established. If a more precise value of rotational stiffness is desired, three-dimensional finite-element analyses or, better yet, actual testing of connections can be done.

4.10. Wind Loads

Throughout this report the emphasis is placed on seismic loading. However, in many cases, wind loading governs the design. It is suggested that to obtain a desirable behavior under wind loading, bolted moment connections be designed such that they do not slip under combination of service wind and gravity load by using slip-critical bolts to resist service load.

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APPENDIX A
TYPICAL CONNECTION DETAILS

A.1. Introduction

In this Appendix a number of details of bolted moment frame connections are provided. The failure modes and design of these connections are similar to those discussed in Chapter 4 of the report.

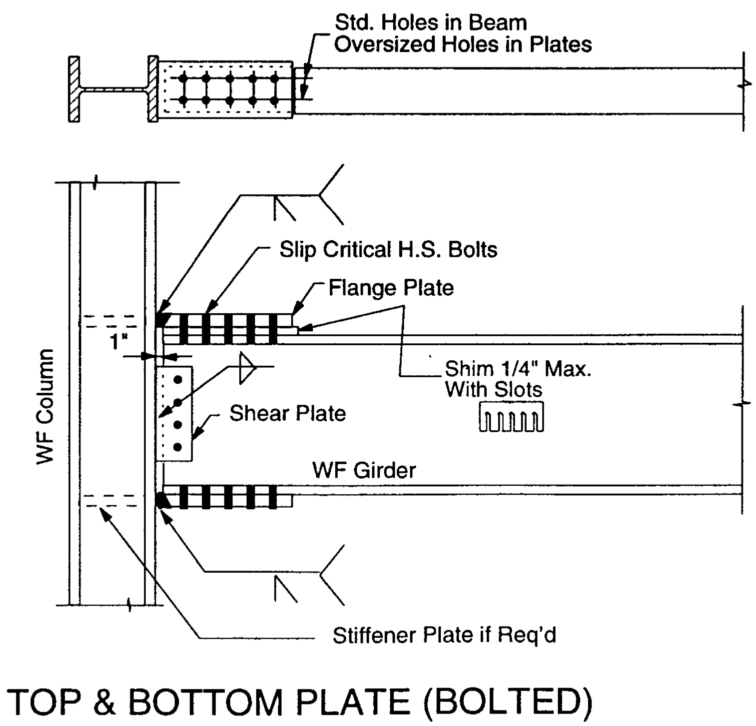
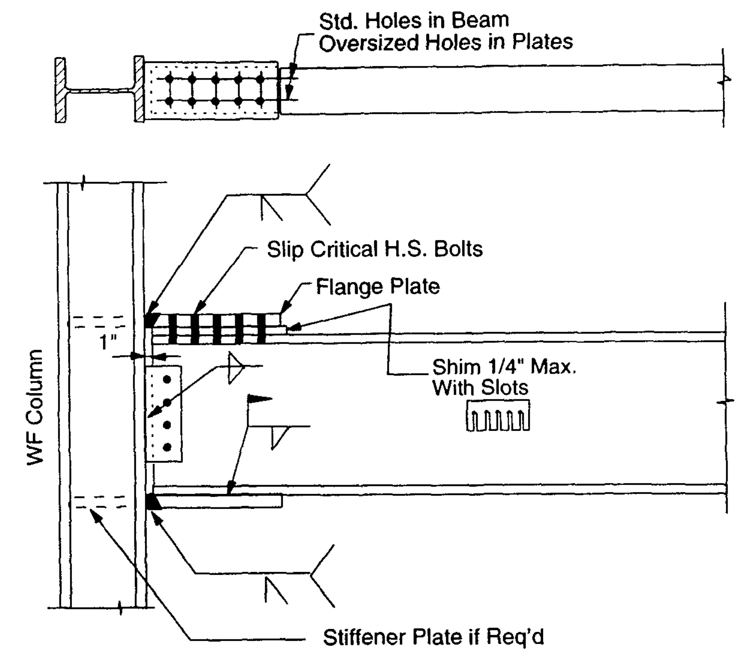
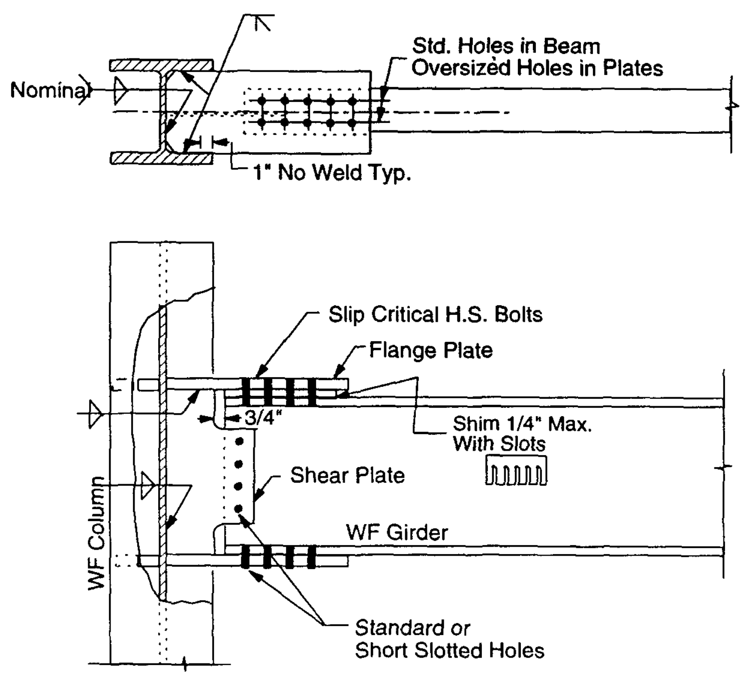


Figure A.1. A Typical Bolted Moment Connection

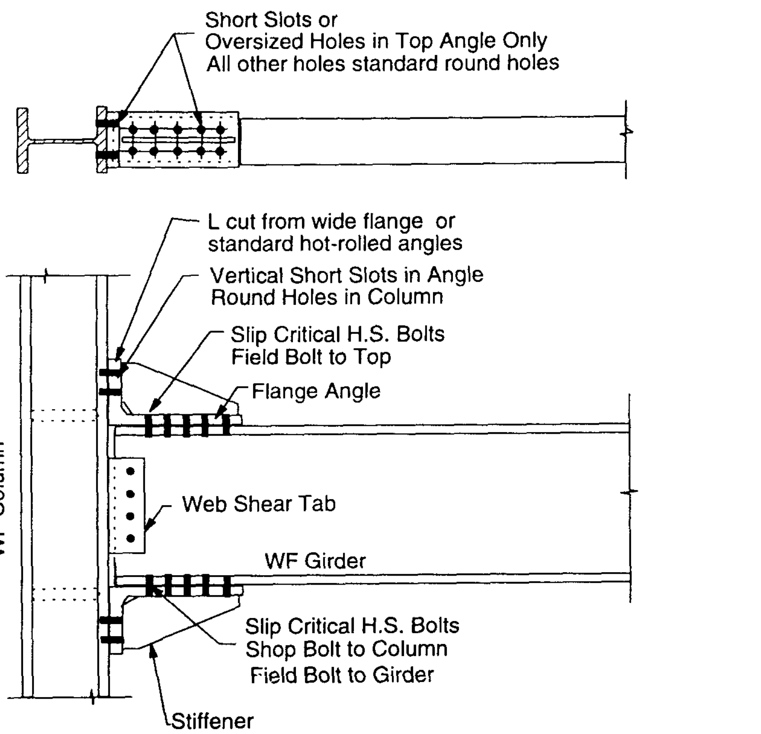


TOP & BOTTOM PLATE (BOLTED & WELDED)

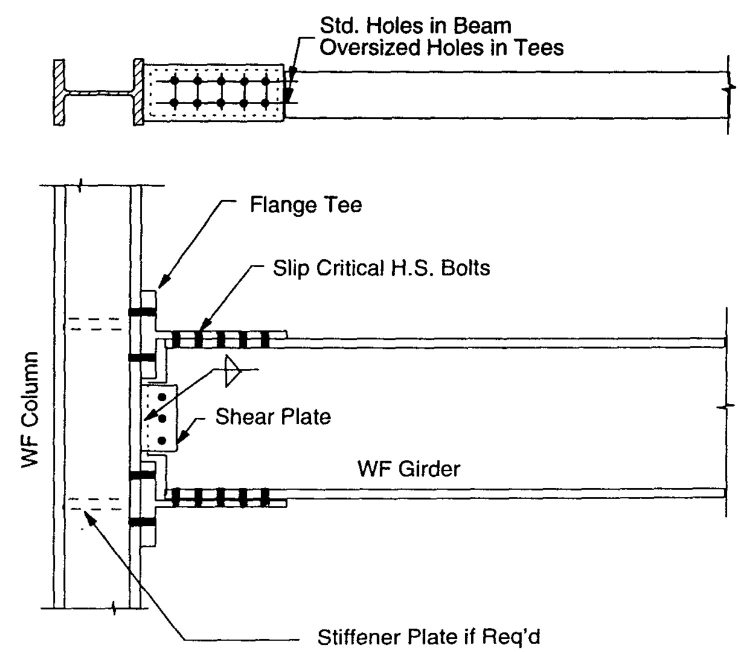


TOP & BOTTOM PLATE (TO COLUMN WEB)

Figure A.1. (Cont'd) Typical Bolted Moment Connections



TOP & BOTTOM STIFFENED ANGLES (BOLTED)



TOP & BOTTOM FLANGE TEE (BOLTED)

Figure A.1. (Cont'd) Typical Bolted Moment Connections

APPENDIX B
A NUMERICAL EXAMPLE

B.1. A Numerical Example

Design a bolted flange-plated Fully Restrained (rigid) moment connection for a W18x50 beam to W14x99 column-flange connection. For the column assume Fy=50 ksi and Fu=65 ksi; for the girder and connecting material assume Fy=36 ksi and Fu=58 ksi. Use 7/8 diameter ASTM A325-N bolts and 70 ksi electrodes. Notice that this example is almost the same as Example 10-1 in Chapter 10 of the 1994 AISC Manual, Volume II (AISC, 1994). The reason for choosing a similar example is to demonstrate the differences between the seismic ductile capacity design (proposed in this report) and the regular design (AISC Manual). The steel used in the girder is changed from grade 50 to A36 steel to be compatible with the current practice of strong column-weak beam design.

Given:
Connection factored forces obtained from analysis:

R_u= 45 kips
M_u= 250 ft-kips
R_u= 310 kips (Axial load in the panel zone)

The bending moment acting on the connection due to service loads (unfactored) obtained from analysis:

$M_{service}$= 145 ft-kips (due to governing combination of loads)

The above service moment will be used in the design of flange bolts to ensure that the connection does not slip under the service loads.

Properties of the girder and the column:

W18x50 (Fy=36, Fu=58 ksi), Span=20 ft.

d= 17.99 in., b_f= 7.495 in., Z_x= 101 in.^{3}, t_w= 0.355 in., t_f= 0.57 in.

W14x99 (Fy=50, Fu=65 ksi), Interior column.
d= 14.16 in., b_f= 14.564 in., k= 1-7/16 in., t_w= 0.485 in., t_f= 0.78 in., A=29.1 in2

Solution:

1. Establish plastic moment capacity of the girder:

$M_p = Z_x F_y$ = 101x36= 3,636 k-in.

2. Check net-section fracture of the girder:

Since F_u / F_y for the girder is not less than 1.5, there is no need to satisfy the UBC-94 (ICBO, 1994) requirement: $A_e/A_g \geq 1.2F_y/F_u$. If the girder material has actual F_y and F_u values other than 36 and 65 ksi, the A_e/A_g ratio needs to satisfy above equation.

3. Check local buckling of the girder flanges:

$$b/t = 7.495/(2x0.57) = 6.6 \leq \frac{52}{\sqrt{F_y}} = 8.6 \text{ O.K.}$$

4. Establish size of the flange plates:

$M_{plate} \geq 1.25\ M_p$
$M_{plate} \geq 1.25$ (3,636), $M_{plate} \geq$ 4,545 k-in.
$A_{plate} \geq (M_{plate})/(d)(F_y)$, or $A_{plate} \geq$ (4,545)/(17.99)(36)=7.0 in

Try: 8"x1" A36 flange plates

5. Check net section failure of the flange plates

$\phi_n M_{pn} \geq 1.25\phi M_p$ (4.15)
0.75 (8-2)(1)(58)(17.99)≥ 1.25 (0.9)(3,636)
4,695 > 4,090 O.K.

6. Establish number of the flange bolts:

Check number of bolts to satisfy
$\phi_b(F_b A_b N)(d) \geq 1.25\phi M_p$ (4.16)

$0.75(48)(0.601)(N)(17.99) \geq 1.25(0.9)(3{,}636)$

$N \geq 10.5$; Try: 12 7/8"dia A325N flange bolts

7. Check bearing capacity of the bolts:

$M_{bearing} \geq 1.25\, M_p$

$2.4(58ksi)(0.57")(7/8")(12)(17.99) \geq 1.25\,(3{,}636)$

14,980 k-in > 4,545 O.K.

8. Check to ensure that the bolts do not slip under the service loads:

The following condition needs to be satisfied:

$1.25 M_{service} \leq M_{slip} \leq 0.8 M_p$

$1.25\,(145x12) \leq (12)(10.2\text{ kips/bolt})(17.99) \leq 0.8(3{,}636)$

$2{,}175 \leq 2{,}202 \leq 2{,}908$ O.K.

It should be added that throughout this report the emphasis was placed on seismic design. However, the final design of connection will be governed by load combinations including the wind load. Following the design philosophy and concepts presented in this report, the designer should ensure that bolted connections are designed as slip-critical to resist the service loads without slip. Such approach will ensure that the connections will not slip during the service wind and small to moderate earthquakes.

9. Check edge distances:

Using a bolt gage of 4.5 inches c/c, provides sufficient edge distance for plate and girder to satisfy AISC(1994) requirements.

10. Check block shear failure:

Block shear failure does not govern.

11. Check panel zone yielding:

$$V_n \geq \frac{\sum Mp_{girders}}{d_s} \tag{4.11}$$

where

$$V_n = 0.55 F_y d_c t_p \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p} \right]$$

$$V_n = 0.55(50)(14.16)(0.485)\left[1+\frac{3(14.564)(0.78^2)}{17.99(14.16)(0.485)}\right] = 229\,kips$$

V_n = 229 kips < 2(3,636)/17.99= 404 kips. Therefore, doubler plates are needed.

$t_p \cong$ 0.485(404/229)-0.485 = 0.37" Use 3/8" doubler plate.

or change column size or column material if it results in more economical design

If instead of above UBC-94 equation, the equation given in the AISC-LRFD Specifications are used, the following will result:

$V_n = \phi\, 0.6 F_y d_c t_p$ =0.9(0.6)(50)(14.16)(0.485)=185 kips < 404 kips

Use 5/8" doubler plate

or change column size or column material if it results in more economical design

12. Establish rotational stiffness of the connection:

$$k_c = \frac{M_c}{\Theta_c} = \frac{F_f h}{\Delta_f/(h/2)} = \frac{2F_f h^2}{\Delta_f}$$

$$k_c = \frac{2(3,636/17.99)(17.99^2)}{\Delta_f} = \frac{130,820}{\Delta_f}$$

where;

$$\Delta_f = \left(\frac{F_f L_p}{A_p E}\right) + \frac{1"}{16} = \left[\frac{(3,636/17.99)(20"/2)}{(8"\text{x}1")(29,000)}\right] + 0.063 = 0.072\text{in.}$$

Therefore;

$$k_c = \frac{130,820}{0.072} = 1,817,000 \text{ kip-in/rad}$$

The value of m, the relative elastic rotational stiffness of the connection and the girder can be calculated as:

m=k_c/(EI/L)= 1,817,000/(29000x800/240)=18.8 > 18 (m for rigid).

The value of m equal to 18.8 for this connection indicates that it can be categorized as rigid moment connection.

APPENDIX C
RECENTLY DESIGNED BOLTED MOMENT-RESISTING FRAMES

C.1. Introduction

In the aftermath of the 1994 Northridge earthquake, a number of design firms has started replacing the welded moment frame design with bolted moment frames. Three of the recent buildings that have been converted to bolted moment frames (Hettum, 1994) are two 3-story and one 5-story building with approximately 240,000 sq. ft of total area. In this Appendix photographs of top and bottom plate moment connections of these buildings are shown.

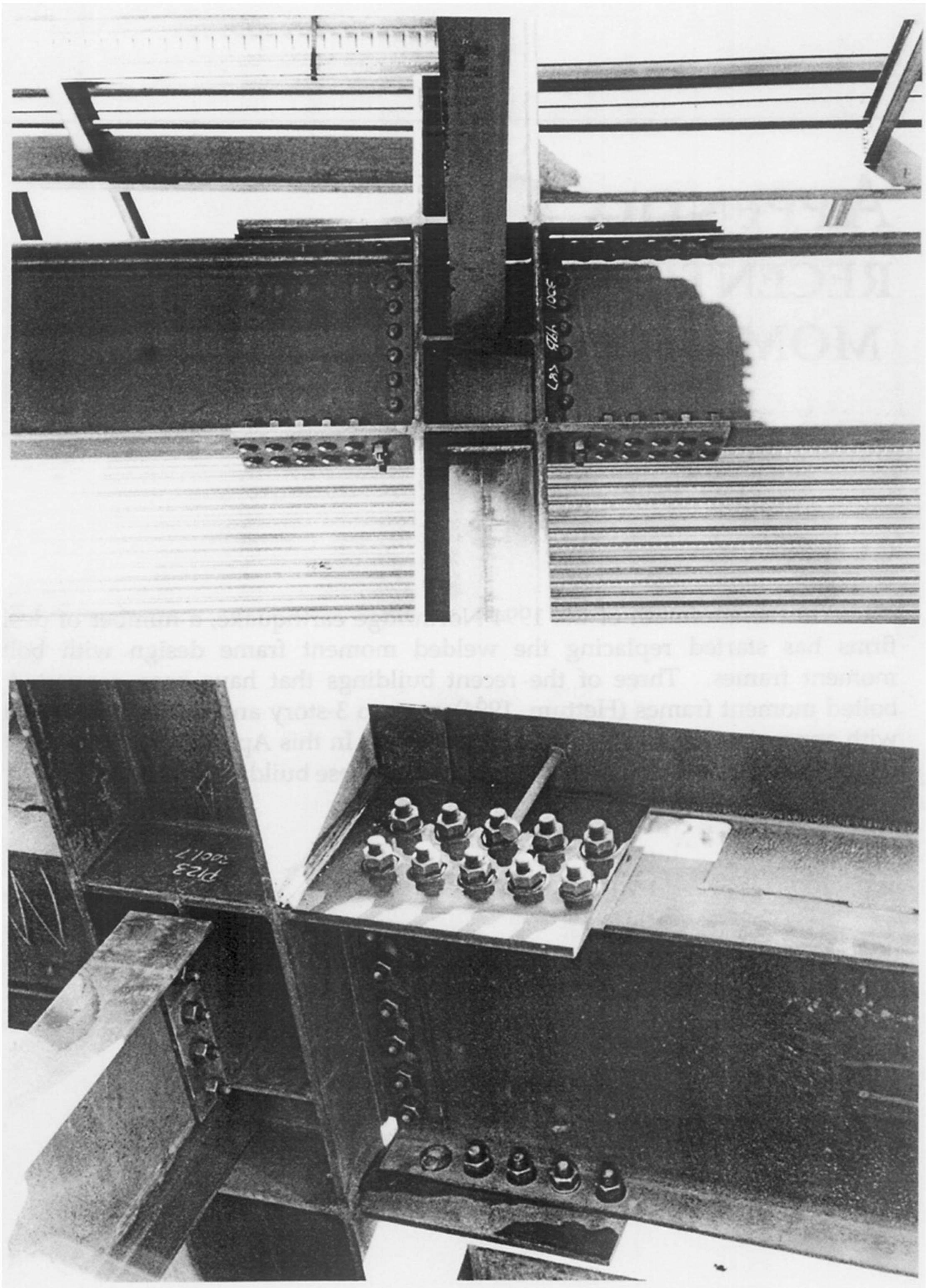


Figure C.1 Views of Bolted Connections in Recently Designed and Constructed Structures, Courtesy of Mackenzie Engineering Incorporated, (Hettum, 1994)

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