

## CHAPTER 1 INTRODUCTION

### 1.1 MOMENT END-PLATE CONNECTIONS

The typical moment end-plate connection shown in Fig. 1-1 consists of a plate that is shop-welded to the end of a beam which is then bolted to the supporting member in the field. The supporting member is typically a column flange or another beam in the case of a splice connection. The extended portion of the end-plate (the part above the top flange) does not extend below the bottom flange, as the usual design procedure allows for such connections under gravity and/or wind loading. However, common practice is to extend the plate below the bottom flange symmetrically for end-plates to be used in seismic regions so that the connection is capable of carrying a full reversal of load during an earthquake. Commonly used end-plate configurations are shown in Fig. 1-2 with the bottom portion of the end-plate extended. Advantages of this type of connection include a low relative cost, ease of erection, and the absence of field welding (which is advantageous for winter construction). The main disadvantage is that the beams must be cut to exact length to ensure that the building remains plumb during erection. On the other hand, modern technology has all but eliminated this problem as more and more fabricators have the ability to cut the beam with high precision. Tests with finger shims have been performed and the results from these tests indicate that their use does not effect connection response.

### 1.2 SEISMIC RESPONSE

During an earthquake, energy dissipation occurs via inelastic deformations of the components making up a steel moment frame. Plasticity can occur in the form of beam or column hinging, shear yielding in the panel zone, or yielding of the connecting elements themselves. Within tolerable limits, all forms of inelastic deformations are acceptable. Since the primary concern of the design engineer is to avoid any type of brittle fracture, the selection of the design location for inelastic behavior of the system must be considered carefully. Also, more stringent requirements are typically made for yielding in the column panel zone or connection, and designing for beam yielding might be a more feasible alternative.

What is meant by inelastic rotation can be generally defined as the sum of all inelastic deformations in the proximity of a connection that can be measured as a change

in some angle at the connection. As will be shown later, the precise definition and determination of this value depends upon the particular design philosophy and source document at hand. It should be noted that throughout this chapter, the terms inelastic rotation and plastic rotation are used interchangeably and denote the same rotation. This is done to be consistent with document-specific terminology. Also, the rotation of a connection does not necessarily refer only to the actual rotation of the connecting elements, but may include beam, column, and panel zone deformations at or near the connection. Regardless, all deformations are measured with respect to the connection's location and are referred to as connection rotations.

Both fully restrained (FR) and partially restrained (PR) connections are considered. As detailed in Section A2 of the *AISC Load and Resistance Factor Design Specification for Structural Steel Buildings* (1993), when considering FR connections it is assumed that angles made by member intersections are maintained throughout the loading. On the contrary, PR connections have insufficient rigidity to maintain these angles. The type of connection chosen for design must be specified on all design documents, and all connections in the structure must be designed with this selection.

Provisions to be mentioned in the following sections of this chapter pertain only to the design of steel moment frame connections in buildings for which design earthquake forces cause the system to respond inelastically in the absence of any base isolation or energy dissipation system.

### 1.3 RECOMMENDATIONS, CODES, AND THE DEVELOPMENT OF CURRENT SEISMIC DESIGN REQUIREMENTS

The main problems discovered after the Northridge earthquake deal primarily with welded connections. In numerous cases, the so called "pre-Northridge" connection proved unable to provide adequate strength at the bottom beam flange-to-column flange weld. The flanges of the connecting beam were directly welded to the column flange via a full penetration groove weld. The main reason for the failure was that the typical welding practice at the time proved inadequate, as the actual specimens could not provide the inelastic rotation that similar experimentally tested connections had shown. For a complete description of this connection and a discussion on its failure theories, see Miller (1998). As a result, the *SAC Interim Guidelines* (FEMA [1995]) revamped design

requirements and system response philosophies regarding the means by which a steel moment frame should provide plastic rotation to dissipate energy during an earthquake.

### 1.3.1 SAC INTERIM GUIDELINES

Although the *SAC Interim Guidelines* contains only recommendations for the design of moment connections, it should be examined first, given its influential role in the development of the *AISC Seismic Provisions for Structural Steel Buildings* (1997). According to the *SAC Interim Guidelines*, the connection should be designed with enough strength to force a plastic hinge to occur in the beam at some predetermined distance away from the connection. This can be accomplished by reinforcing the connection itself or reducing the beam section locally. In addition to any gravity loads that may incur, the connection should be designed to resist all forces redistributed as a result of the plastic hinge formation.

For a tested connection of a typical frame configuration, a minimum plastic rotation capacity of 0.03 radians is recommended. It is important to understand the *SAC Interim Guidelines'* definition of plastic rotation as used in this section. The plastic rotation is defined as the plastic chord rotation angle,  $\theta_p$ , which is the plastic deflection  $\Delta_{CL}$  of the beam span centerline divided by the distance  $L_{CL}$  from the beam mid-span to the centerline of the panel zone of the beam-to-column connection. Obviously, this rotation is different than the rotation that would occur at a discrete plastic hinge at a known location in the beam span. The definition above is a result of tests performed with typical assemblies. Such tests have shown that it is very difficult to obtain discrete plastic hinge rotation values, and that discrete plastic hinges rarely occur without being coupled with plasticity in the panel zone.

For a given connection to be acceptable, several criteria must be satisfied. Testing must show that the connection can develop the minimum plastic rotation capacity discussed above for at least one loading cycle. The minimum moment at the column face is specified separately for strengthened connections and reduced beam sections.

For strengthened connections, the minimum moment at the connection should be equal to the plastic moment  $M_p$  of the beam found using the minimum yield strength  $F_y$  of the beam. However, if beam flange buckling is the controlling beam limit state, the

maximum of 80% of  $M_p$  or the nominal moment strength for beam flange local buckling is the lower bound.

For reduced beam sections, the minimum moment at the column face should be equal to  $M_p$  of the reduced section calculated using the minimum  $F_y$  of the beam. However, this moment should not be less than 80% of the unreduced section's  $M_p$ .

In addition to these strength criteria, all connections should exhibit ductility throughout the loading process. Any test which suggests a brittle limit state should be considered unsuccessful and the connection unacceptable for design. Until the minimum plastic rotation is obtained, the connection should be able to support dead and live loads as required by the building code. No minimum number of tests to qualify a connection is suggested by the *SAC Interim Guidelines*.

### 1.3.2 AISC SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

The *AISC Seismic Provisions for Structural Steel Buildings* (1997) was developed cooperatively with the *1997 NEHRP Provisions* (1997) and includes many suggestions made in the *SAC Interim Guidelines*. As a result, this document should form the basis for the seismic provisions of the *International Building Code* (2000) pertaining to steel frames and connections. For intermediate and special moment frames, the *AISC Seismic Provisions for Structural Steel Buildings* has adopted the position that for FR connections, yielding must take place in the members of the frame (plastic hinge in beam, panel zone, etc.) and not in the connections. However, since yielding in the column is the least desirable result, the design engineer should consider designing the system such that flexural yielding occurs in the beam. For FR connections that are part of ordinary moment frames, the connecting elements may yield as long as 0.01 radians of plastic rotation can be provided by the system. For PR connections that are part of any type of frame, connecting elements may yield. The requirements of FR connections that are part of ordinary moment frames are:

- Connection flexural strength must be designed to be at least the lesser of (i)  $1.1R_yM_p$  of the beam and (ii) the maximum moment that can possibly be applied to the system,

where  $R_y$  is the ratio of the steel's expected yield strength  $F_{ye}$  to its minimum specified yield strength  $F_y$ .

- In lieu of the above requirement, cyclic testing can be used to obtain the flexural strength of the connection as long as a minimum inelastic rotation of at least 0.01 radians is attained.

For PR connections, the following requirements must be satisfied:

- The minimum design strength of the connection shall be greater than the largest minimum design strength required to resist loading combinations A4-1 through A4-6 of the *AISC LRFD Design Specification for Structural Steel Buildings* (1994).
- The nominal flexural strength of the connection shall be at least  $0.5M_p$  of the beam or column, whichever is less.
- Cyclic testing must show that an adequate rotation capacity is attainable at rotations that result in the design story drift.
- In addition to second-order effects, the decreased strength and stiffness requirements of the connection must be considered in the design.

For connections that are part of intermediate moment frames, several criteria must be satisfied:

- Qualifying cyclic tests must show that an inelastic rotation capacity of at least 0.02 radians is achievable.
- For FR connections, the connection must have a flexural strength of at least  $M_p$  of the beam at the required inelastic rotation, unless local flange buckling is the beam's controlling limit state or a reduced beam section is used, in which case  $0.8M_p$  is the limiting design strength.
- The use of PR connections is acceptable only if the above required rotation is achievable and the design strength of the connection is greater than that required to resist loading combinations A4-1 through A4-6 of the *AISC LRFD Design Specification for Structural Steel Buildings* (1994). Second-order effects must be considered.

Although requiring a greater inelastic rotation capacity than intermediate moment frame connections, special moment frame connections are treated similarly. Their requirements are:

- Qualifying cyclic tests must show that an inelastic rotation capacity of at least 0.03 radians is achievable.
- For FR connections, the connection must have a flexural strength of at least  $M_p$  of the beam at the required inelastic rotation, unless local flange buckling is the beam's controlling limit state or a reduced beam section is used, in which case  $0.8M_p$  is the limiting design strength.
- The use of PR connections is acceptable only if the above required rotation is achievable and the design strength of the connection is greater than that required to resist loading combinations A4-1 through A4-6 of the *AISC LRFD Design Specification for Structural Steel Buildings* (1994). Second-order effects must be considered.

In order that connection tests are performed adequately and interpolation of various testing configuration results is possible, the *AISC Seismic Provisions for Structural Steel Buildings* presents standard procedures and requirements as applied to qualifying tests. Independent of the type of moment frame used, two cyclic tests are required to insure an acceptable connection. These results can be obtained from tests specifically designed for the project at hand (representative member sizes, material properties, connection geometry, and connection processes), or from documented tests performed for other projects which have the same project conditions. Also, when deemed rational, varying member sizes may be acceptable when interpolation and extrapolation of test results warrants such a decision. The actual connection should mimic the materials, configurations, and processes of the tested system as closely as possible. Any test which reports a beam with a tested yield strength less than 85% of  $F_{ye}$  is not a qualifying test.

The *AISC Seismic Provisions for Structural Steel Buildings* defines inelastic rotation as the permanent plastic rotation between a beam and column, measured in

radians, and is based upon test specimen deformations. These deformations can include yielding of the connection or framing components, yielding of the connection and its components, or any slip that might occur at a connection. The measure of inelastic rotation shall be found at the line of intersection connecting the inflection point in the beam to the centerline of the beam at the column face.

Numerous testing details such as loading, connection details, and test reporting requirements are presented in Appendix S of the *AISC Seismic Provisions for Structural Steel Buildings* (1997).

The roles played by the *SAC Interim Guidelines* and the *1997 NEHRP Provisions* in the development of the *AISC Seismic Provisions for Structural Steel Buildings* are quite clear, which makes it seem only reasonable that it is this AISC document that will form the basis of the *International Building Code* (2000).

Although panel zone behavior and design requirements are barely mentioned here, extensive details can be found in Section 2.4 and Chapter 7 of this study.

#### 1.4 SUMMARY AND THE NEED FOR LARGER CONNECTIONS

The design requirements summarized in Section 1.3.2 are quite exhaustive and cover all types of moment connections. However, since this study is concerned primarily with moment end-plate connections, the requirements directly related to this type of connection are now summarized.

For ordinary moment frames, the end-plate connection strength can be designed to be stronger or weaker than the plastic moment capacity of the adjoining beam regardless of the type of construction (e.g., fully restrained, FR, or partially restrained, PR).

For intermediate and special moment frames, FR construction can be used only if the end-plate connection strength is designed stronger than the plastic moment capacity of the adjoining beam (e.g., a thick plate and large bolts are used). Otherwise, PR construction must be used for the entire structure. This requirement is in accordance with the *SAC Interim Guidelines* (FEMA [1995]) and the *1997 NEHRP Provisions* (FEMA [1997]) and results mainly from welding failures in the Northridge earthquake. Hence, it may be overly conservative for moment-end-plate connections, but no exception is allowed at this time. In addition to all this, the plastic behavior of the end-plate is beneficial during an earthquake, as it dissipates energy safely. This point is very

important, for it can be shown that the design procedures for the largest available end-plate configurations, four-bolt-wide unstiffened and eight-bolt stiffened configurations (as identified in the *AISC Load and Resistance Factor Design Manual of Steel Construction* [1994]), simply cannot develop all practical beam sizes. For special and intermediate moment frames, using FR construction results in a very thick end-plate and bolt forces which are unacceptable. Using PR construction results in an abnormally thin end-plate with very large (1 ½ in. diameter) high-strength bolts. Obviously, to meet the stringent requirements of the *AISC Seismic Provisions for Structural Steel Buildings*, larger connections containing numerous bolts must be designed. However, no design procedures are currently available to engineers. Simply put, new design procedures must be established for these connections so that they can be used effectively and economically in seismic regions.