2. GENERAL REQUIREMENTS

2.1 Scope

These *Recommended Criteria* apply to the seismic design of Special Moment Frames and Ordinary Moment Frames designed using the R, C_d, and W_0 values given in Table 5.2.2, pages 45-50, of *FEMA-302*. They do not apply to structures designed in accordance with the applicable Provisions of *FEMA-302* for "Structural Steel Systems Not Specifically Detailed for Seismic Resistance". These *Recommended Criteria* replace and supercede all design guidelines contained in *FEMA-267*, *FEMA-267A*, and *FEMA-267B*.

This chapter presents overall criteria for the seismic design of steel moment frames for new buildings and structures. Included herein are general criteria on applicable references including codes, provisions and standards, recommended performance objectives, system selection, system analysis, frame design, connection design, specifications, quality control and quality assurance.

2.2 Applicable Codes, Standards, and References

Steel moment-frame systems should, as a minimum, be designed in accordance with the applicable provisions of the prevailing building code as supplemented by these *Recommended Criteria*. These *Recommended Criteria* are specifically written to be compatible with the requirements of *FEMA-302 – NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Where these *Recommended Criteria* are different from those of the prevailing code, it is intended that these *Recommended Criteria* should take precedence. The following are the major codes, standards and references referred to herein:

FEMA-302	*NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition, Part 1 – Provisions (BSSC, 1997a)*
FEMA-303	*NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition, Part 2 – Commentary (BSSC, 1997b)*
AWS D1.1	*Structural Welding Code, 1998 Edition (AWS, 1998)*
AISC Seismic	*Seismic Provisions for Structural Steel Buildings, April 15, 1997, (AISC, 1997) including Supplement No. 1, February 15, 1999 (AISC, 1999)*
AISC-LRFD	*Load and Resistance Factor Design Specifications for Structural Steel Buildings (AISC, 1993)*
AISC-Manual	*LRFD Manual of Steel Construction, Second Edition, 1998 (AISC, 1998b)*
FEMA-353	*Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications* (SAC, 2000d)
FEMA-273	*NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (ATC, 1997a)

Commentary: The 1997 AISC Seismic Provisions (AISC, 1997) provide design requirements for steel moment-frame structures. FEMA-302 adopts the AISC Seismic Provisions by reference as the design provisions for seismic-force-

resisting systems of structural steel. The International Building Code is based generally on the FEMA-302 Provisions, and incorporates design requirements for steel structures primarily based on the AISC Provisions. These Recommended Criteria are written to be compatible with the 1997 AISC Seismic Provisions and FEMA-302 Provisions and reference is made to sections of those documents where appropriate herein.

2.3 Basic Design Approach

The recommended design approach consists of the following basic steps:

Step 1: Select a structural system type and frame configuration in accordance with Section 2.5 of these *Recommended Criteria*.

Step 2: Select preliminary frame member sizes and perform a structural analysis for earthquake loading and frame adequacy using the applicable R, C_d and W_0 values, strength criteria, drift limits, and redundancy requirements of *FEMA-302*, as supplemented by Section 2.9 of these *Recommended Criteria*.

Step 3: Select an appropriate connection type, in accordance with Section 2.5.3 of these *Recommended Criteria*. Connections may be prequalified, project qualified, or proprietary, as indicated in Chapter 3 of these *Recommended Criteria*.

Step 4: Perform an analysis in accordance with Sections 2.7 and 2.8 of these *Recommended Criteria*, considering the effects (if any) of the selected connection type on frame stiffness and behavior, to confirm the adequacy of member sizing to meet the applicable strength, drift, and stability limitations.

Step 5: Confirm or revise the member sizing based on the connection type selected and following Sections 2.9 and 3.2 of these *Recommended Criteria*. Return to Step 4, if necessary.

Step 6: Complete the design of the connections, in accordance with Chapter 3 of these *Recommended Criteria*.

As an option, when it is desired to design for specific performance, rather than simply achieving code compliance, a Performance Evaluation following the guidelines of Chapter 4 may be performed.

Commentary: This section outlines the basic steps recommended for design intended to meet the minimum criteria of the building code. Since the 1994 Northridge earthquake, the 1997 AISC Seismic Provisions have required that laboratory test data be submitted to demonstrate that connection detailing will be capable of adequate service. With the publication of these Recommended Criteria, and the establishment of a series of prequalified connection details, it is intended that substantiation of connection detailing by reference to laboratory test data will not be required for most design applications. However, design *procedures for some types of prequalified connections entail significant calculation.*

The optional Performance Evaluation procedures contained in Chapter 4 and Appendix A of these Recommended Criteria need not be applied to designs intended only to meet the requirements of the building code. Regular, well-configured Special Moment Frame and Ordinary Moment Frame structures designed and constructed in accordance with FEMA-302, and building code requirements as supplemented by these Recommended Criteria, are expected to provide a high level of confidence of being able to resist collapse under Maximum Considered Earthquake demands. Section 2.4 of these Recommended Criteria and FEMA-303 provide additional information on this performance goal. Structures with significant irregularity, low levels of redundancy, or poor configuration may not be capable of such performance. The Performance Evaluation procedures of Chapter 4 and Appendix A may be used to confirm the capability of such structures to meet the performance intended by the building code, or may be used to implement performance-based designs intended to meet higher performance objectives.

2.4 Design Performance Objectives

Under *FEMA-302*, each building and structure must be assigned to one of three Seismic Use Groups (SUGs). Buildings are assigned to the SUGs based on their intended occupancy and use. Most commercial, residential and industrial structures are assigned to SUG I. Buildings occupied by large numbers of persons or by persons with limited mobility, or that house large quantities of potentially hazardous materials are assigned to SUG II. Buildings that are essential to postearthquake disaster response and recovery operations are assigned to SUG III. Buildings in each of SUG II and III are intended to provide better performance, as a group, than buildings in SUG I. As indicated in *FEMA-303,* buildings designed in accordance with the provisions for each SUG are intended, as a minimum, to be capable of providing the performance indicated in Figure 2-1.

The *FEMA-302* provision attempts to obtain these various performance characteristics through regulation of system selection, detailing requirements, design force levels, and permissible drift. This regulation is based on the SUG, the seismicity of the region containing the building site, and the effect of site-specific geologic conditions. All structures should, as a minimum, be assigned to an appropriate SUG, in accordance with the building code, and be designed in accordance with the applicable requirements for that SUG.

Although the *FEMA-303 Commentary* to *FEMA-302* implies that buildings designed in accordance with the requirements for the various SUGs should be capable of providing the performance capabilities indicated in Figure 2-1, *FEMA-302* does not contain direct methods to evaluate and verify the actual performance capability of structures, nor does it provide a direct means to design for performance characteristics other than those implied in Figure 2-1, should it be desired to do so. It is believed, based on observation of the performance of modern, code-conforming construction in recent earthquakes, that *FEMA-302* provides reasonable reliability with regard to attaining Life Safe performance for SUG-I structures subjected to design events, as indicated in Figure 2-1. However, the reliability of *FEMA-302* with regard to the attainment of other performance objectives for SUG-I structures, or for reliably attaining any of the performance objectives for the other SUGs seems less certain and has never been quantified or verified.

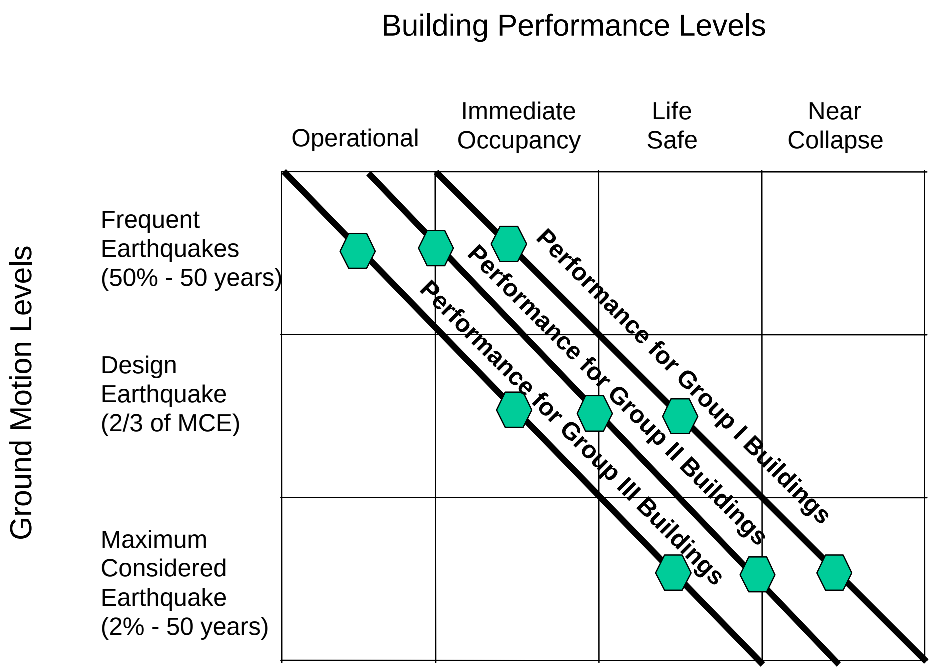

Figure 2-1 NEHRP Seismic Use Groups (SUG) and Performance

Chapters 2 and 3 of these *Recommended Criteria* present code-based design recommendations for steel moment-frame buildings. All buildings should, as a minimum, be designed in accordance with these recommendations. For buildings in which it is desired to attain other performance than implied by the code, or for which it is desired to have greater confidence that the building will actually be capable of attaining the desired performance, the procedures in Chapter 4 and Appendix A may be applied.

Commentary: FEMA-302 includes three types of steel moment frames , two of which are incorporated in these Recommended Criteria. The three types are: Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF). Building code provisions for SMF systems strictly regulate building configuration, proportioning of members and connection detailing in order to produce structures with superior inelastic response capability. Provisions for OMF systems have less control on these design features and therefore, as a class, OMF structures are expected to have poorer inelastic response capability than SMF systems. Following the 1994 Northridge earthquake, the building code was amended to include substantial *additional requirements for SMF system design and construction, resulting in an increase in the development cost for such structures. In 1997, the IMF system was added to FEMA-302 and the AISC Seismic Provisions to provide an economical alternative to SMF construction for regions of moderate seismicity. Studies conducted under this project have indicated that the inelastic response demands on IMF systems are similar to those for SMF systems and that, therefore, the reduction in design criteria associated with the IMF system was not justified. Consequently, only Special Moment Frame and Ordinary Moment Frame systems are included herein. These systems are described in more detail in Section 2.5. In FEMA-302, a unique R value and C_d factor are assigned to each of these systems, as are height limitations and other restrictions on use. Regardless of the system selected, FEMA-302 implies that structures designed to meet the requirements therein will be capable of meeting the Collapse Prevention performance level for a Maximum Considered Earthquake (MCE) ground motion level and will provide Life Safe performance for the Design Basis Earthquake (DBE) ground motion that has a severity 2/3 that of MCE ground motion. This 2/3 factor is based on an assumption that the Life Safety performance on which earlier editions of the NEHRP Recommended Provisions were based inherently provided a minimum margin of 1.5 against collapse. Except for sites located within a few kilometers of known active faults, the MCE ground motion is represented by ground shaking response spectra that have a 2% probability of exceedance in 50 years (approximately 2500-year mean return period). For sites that are close to known active faults, the MCE ground motion is taken either as this 2%/50-year spectrum, or as a spectrum that is 150% of that determined from a median estimate of the ground motion resulting from a characteristic event on the nearby active fault, whichever is less.*

The FEMA-302 Provisions define classes of structures for which performance superior to that described above is mandated. Additionally, individual building owners may desire a higher level of performance. The FEMA-302 Provisions attempt to improve performance for SUG-II and SUG-III structures, (1) through use of an occupancy importance factor that increases design force levels, and therefore reduces the amount of ductility a structure must exhibit to withstand strong ground shaking, and (2) through specification of more restrictive drift limits than those applied to SUG-I structures. This combination of increased design forces and more restrictive drift limitations leads to substantially greater strength in systems such as SMFs, the design of which is governed by drift.

The FEMA-302 R factors, drift limits, and height limitations, as well as the inelastic rotation capability requirements corresponding to the R value for each moment-frame type (SMF, IMF, or OMF), are based more on historical precedent and judgment than they are on analytical demonstration of adequacy. In the research program on which these Criteria are based, an extensive series of nonlinear analytical investigations has been conducted to determine the drift *demands on structures designed in accordance with the current code when subjected to different ground motions, and for a variety of assumed hysteretic behaviors for connections. The results of these investigations have led to the conclusion that some of the FEMA-302 Provisions and 1997 AISC Seismic Provisions were not capable of reliably providing the intended performance. These Recommended Criteria directly modify those Provisions so as to increase the expected reliability of performance to an acceptable level. On the basis of these analytical studies, it is believed that regular, well-configured structures designed in accordance with these Recommended Criteria and constructed in accordance with FEMA-353, provide in excess of 90% confidence of being able to withstand Maximum Considered Earthquake demands without global collapse and provide mean confidence of being able to withstand such ground motion without local structural failure.*

It should be recognized that application of the modifications suggested in these Recommended Criteria, while considered necessary to provide this level of confidence with regard to achieving the indicated performance for moment-resisting frames, may result in such systems having superior performance capabilities relative to some other systems, the design provisions for which do not have a comparable analytical basis. In other words, the design provisions contained in FEMA-302 for some other structural systems, both of steel and of other construction materials, may inherently provide a lower level of assurance that the resulting structures will be able to provide the intended performance.

The three classes of steel moment-frame systems contained in FEMA-302 are themselves not capable of providing uniform performance. OMFs will typically be stronger than either IMFs or SMFs, but can have much poorer inelastic response characteristics. The result of this is that OMFs should be able to resist the onset of damage at somewhat stronger levels of ground shaking than is the case for either IMFs or SMFs. However, as ground motion intensity increases beyond the damage threshold for each of these structural types, it would be anticipated that OMFs would present a much greater risk of collapse than would IMFs, which in turn, would present a more significant risk of collapse then SMFs. For these reasons, FEMA-302 places limitations on the applicability of these various structural systems depending on a structure's height and the seismic hazard at the site.

Refer to Chapter 4 for more detailed discussion of recommended performance objectives and their implications.

2.5 System Selection

2.5.1 Configuration and Load Path

Every structure should be provided with a complete lateral and vertical seismic-force-resisting system, capable of transmitting inertial forces from the locations of mass throughout the structure to the foundations. For steel moment-frame structures, the load path includes the floor and roof diaphragms, the moment-resisting frames, the foundations, and the various collector elements that interconnect these system components.

To the extent possible, the structural system should have a regular configuration without significant discontinuities in stiffness or strength and with the rigidity of the structural system distributed uniformly around the center of mass.

Commentary: The importance of maintaining regularity in structural systems can not be overemphasized. The analytical investigations of structural performance conducted as part of this project were limited to regular structural systems. Irregularities in structural systems can result in concentration of deformation demands on localized portions of a structure, and early development of P-D instabilities. FEMA-302 includes significant limitations on structural irregularity, particularly for structures in Seismic Design Categories D, E and F. However, it was not possible, within the scope of this project, to determine if these limitations are sufficient to ensure that the intended performance capability is achieved and this should be the subject of future investigations.

Structures categorized as regular under FEMA-302 may not actually behave in a regular manner. FEMA-302 categorizes a multistory buildings as being regular if the vertical distribution of lateral stiffness and strength is uniform. Thus, a structure with equal lateral stiffness and strength in every story would be categorized as regular. However, such structures would not actually behave as regular structures when responding to strong ground motion. Instead such structures would develop large concentrations of inelastic behavior and deformation at the lower stories of the structure. To provide true strength and stiffness regularity in multistory structures, it is necessary to maintain uniform ratios of (1), lateral strength to tributary mass, and (2), lateral stiffness to tributary mass, for each story of the structure, where tributary mass may be considered as that portion of the structure's mass supported at and above the story.

2.5.2 Structural System Selection

The moment frame may be designed either as an SMF or OMF. The selection of moment-frame type should be governed by the prevailing code and by the project conditions. Consideration should be given to using Special Moment Frames whenever conditions permit.

Commentary: FEMA-302 defines three types of steel moment frames: Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF). Detailing and configuration requirements are specified for each of these three systems to provide for different levels of ductility and global inelastic response capability, varying from highest in SMFs to lowest in OMFs. IMF systems have intentionally been omitted from these Recommended Criteria because nonlinear analyses of buildings designed to the criteria for IMF systems contained in FEMA-302 have indicated that the inelastic demands for these structures are nearly as large as those for SMF structures. Therefore, it is not possible to justify on technical grounds the use of the relaxed detailing criteria provided for IMFs in FEMA-302 unless more restrictive design force levels and drift criteria are also specified in order to limit the amount of inelastic demand these structures may experience. Rather than developing such criteria, it was decided to omit this system, which had only recently been introduced into the building codes, from further consideration.

Ordinary Moment Frames are relatively strong (compared to SMFs) but have much less ductility. As a result, Ordinary Moment Frame structures, as a class, would be anticipated to have less damage than SMFs for moderate levels of ground shaking and significantly more damage than SMFs for severe levels of ground shaking. In recognition of this, FEMA-302 places limitations on the height, occupancy and ground motion severity for which Ordinary Moment Frame systems can be used. In recognition of the superior performance characteristics of SMF systems when subjected to high-intensity ground shaking, it is recommended that designers consider their use, even when IMF or OMF systems are permitted under the building code.

2.5.3 Connection Type

Moment-resisting connections in SMFs and OMFs, except connections in OMFs designed to remain elastic under design level earthquake ground shaking, should be demonstrated by test and by analysis to be capable of providing the minimum levels of interstory drift angle capacity specified in Section 3.9 of these *Recommended Criteria*. Interstory drift angle is that portion of the interstory drift ratio in a frame resulting from flexural deformation of the frame elements, as opposed to axial deformation of the columns, as indicated in Figure 2-2. Sections 3.5, 3.6 and 3.7 present details and design procedures for a series of connections that are recommended as prequalified to meet the criteria of Section 3.9 without further analysis or testing, when used within the indicated limits applicable to each connection type.

Commentary: FEMA-302 and the 1997 AISC Seismic Provisions set minimum strength criteria for connections. In addition, except for connections in OMFs that are designed to remain elastic, the 1997 AISC Seismic Provisions require that connections be demonstrated capable of providing minimum levels of rotational capacity. The 1997 AISC Seismic Provisions uses plastic rotation angle as the performance parameter by which connections are qualified. In these

Recommended Criteria, interstory drift angle is used instead. This is because this parameter, (1) seems to be stable with regard to prediction of frame performance, (2) is closely related to plastic rotation angle, (3) is less ambiguous with regard to definition, and (4) is a quantity that is easily determined from the results of standard frame analyses using either linear or nonlinear methods.

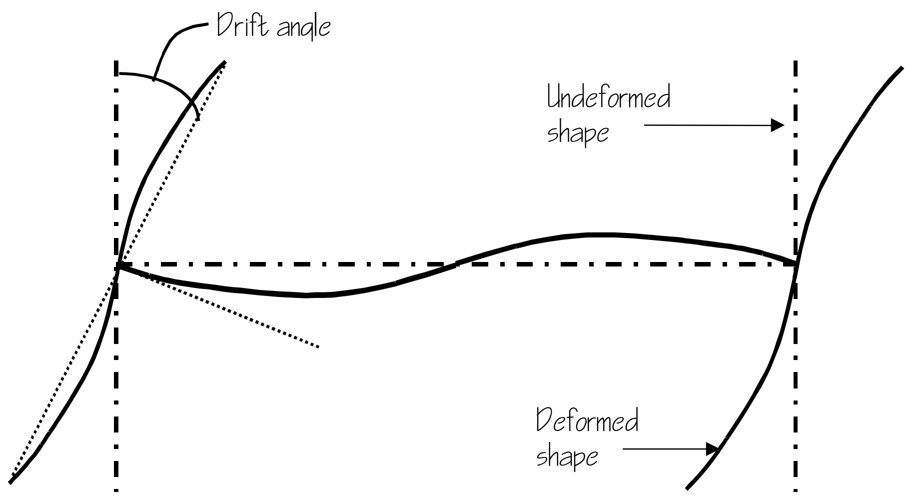

Figure 2-2 Interstory Drift Angle

Figure 2-2 illustrates the interstory drift angle, for a frame with fully restrained (FR) connections and rigid panel zones. Prior to lateral deformation, the beam and column are joined at right angles to each other. Under elastic deformation, the column and beam will remain joined at right angles and the beam will rotate in double curvature between the two columns. The interstory drift angle is measured as the angle between the undeformed vertical axis of the column and the deformed axis of the column at the center of the beam-column joint. For the idealized FR frame with rigid panel zones, shown in the figure, this same angle will exist between the undeformed horizontal axis of the beam and the deformed axis of the beam, at the beam-column connection. In FEMA-273, this angle is termed the chord angle and is used as the parameter for determining beam-column connection performance. However, for frames with panel zones that are not rigid, frames with partially restrained connections, or frames that exhibit plasticity at the connection, the chord angle of the beam will not be identical to the interstory drift angle. For such frames, the interstory drift angle, reduced for the effects of axial column elongation, is a better measure of the total imposed rotation on all elements of the connection, including panel zones and connection elements, and is used as the basis of these Recommended Criteria.

2.5.4 Redundancy

Structures assigned to Seismic Design Categories D, E, and F of *FEMA-302* shall be provided with sufficient bays of moment-resisting framing to satisfy the redundancy requirements of those *Provisions*. In addition, the strength of members of the seismic-force-resisting system shall be evaluated for adequacy to resist horizontal earthquake forces that are factored by the redundancy factor r in accordance with the load combinations of *FEMA-302*.

Commentary: There are several reasons why structures with some redundancy in their structural systems should perform better than structures without such redundancy. The basic philosophy underlying the design provisions of FEMA-302 is to permit substantial inelastic behavior in frames under ground shaking of the severity of the design earthquake or more severe events. Under such conditions, occasional failures of elements may occur. Structures that have nonredundant seismic-force-resisting systems could potentially develop instability in the event of failure of one or more elements of the system. Redundant structures, on the other hand, would still retain some significant amount of lateral resistance following failure of a few elements.

Another significant advantage of providing redundant framing systems is that the use of a larger number of frames to resist lateral forces often permits the size of the framing elements to be reduced. Laboratory research has shown that connection ductile capacity generally increases as the size of the framing elements decreases.

FEMA-302 includes a redundancy factor r with values between 1.0 and 1.5, which is applied as a load factor on calculated earthquake forces for structures categorized as Seismic Design Category D, E, or F. Less redundant systems (frames with fewer participating beams and columns) are assigned higher values of the redundancy factor and therefore must be designed to resist higher design forces to compensate for their lack of redundancy. Minimum permissible levels of redundancy are set, through lower-bound values specified for the redundancy factor, for structures located in regions of high seismic risk.

The maximum permitted r values given in FEMA-302 were based only on the judgment of the writers of that document. They should not be construed as ideal or optimum values. Designers are encouraged to incorporate as much redundancy as is practical into steel moment-frame buildings.

2.5.5 Frame Beam Spans

The connection prequalification data provided for each prequalified connection in Chapter 3 includes specification of the minimum beam-span-to-depth ratio for which the connection is prequalified. Span-to-depth ratios for beams in moment frames should equal or exceed the minimum span-to-depth ratio applicable to the connection type being used, unless project-specific qualification testing is performed as described in Section 3.9, or other rational analysis is employed to demonstrate that hinge rotations or bending strains will not exceed those for which the connection is prequalified.

Where the effective span for a frame beam (distance between points of plastic hinging of the beam) is such that shear yielding of the beam will occur before flexural yielding, the web of the beam shall be detailed and braced as required by the 1997 *AISC Seismic Provisions* for long links in eccentric braced frames.

Commentary: In determining the layout of moment frames, it should be recognized that excessively short spans can affect both frame and connection behavior. Possible effects include the following:

1. *For connection types that move the hinge significantly away from the column face, the plastic rotation demand at the hinge will be significantly larger than the frame interstory drift angle, due to geometric effects.*

2. *The steeper moment gradient resulting from the shorter spans will decrease the length of the beam hinge, requiring that the beam develop greater bending strains to accommodate the same interstory drift angle.*

3. *If the effective span length becomes too short, shear yielding of the beam, rather than flexural yielding, will control inelastic behavior.*

Most testing of prequalified connections performed under this project used configurations with beam spans of about 25 feet. Most tested beams were either W30 or W36, so that span-to-depth ratios were typically in the range of 8 to 10. Refer to FEMA-355D, State of the Art Report on Connection Performance for more information on the effects of short spans.

2.6 Structural Materials

2.6.1 Material Specifications

Structural steel should conform to the specifications and grades permitted by the 1997 *AISC Seismic Provisions,* as modified by *FEMA-353,* and as indicated in the specific connection prequalifications, unless a project-specific qualification testing program is performed to demonstrate acceptable performance of alternative materials.

Commentary: Under the 1997 AISC Seismic Provisions, rolled shapes used in OMF or SMF applications may conform to the ASTM A36, A572 or A913 specifications. In the 1980s, it was common practice in some regions to design moment frames with columns conforming to the ASTM A572 Grade 50 specification and with beams conforming to the ASTM A36 specification, in order to obtain frames economically with strong columns and weak beams. During the 1990s, however, the steel production industry in the United States has undergone a significant evolution, with many of the older mills being replaced by newer mills that use scrap-based production processes. These newer mills routinely produce higher strength steel than did the older mills. Since the A36 and A572 specifications do not place an upper bound on material strength, much of the steel *shipped by these mills, particularly for material ordered as conforming to the A36 specification, is much stronger than the minimum strength controlled by the specification, and use of the combination of A36 and A572 materials to provide for strong-column-weak-beam conditions will not reliably achieve this goal. In 1997, ASTM introduced a new A992 specification to address this problem. The A992 specification is similar to the ASTM A572, Grade 50 specification, except that maximum as well as minimum yield strengths are specified to provide for more controlled design conditions. In addition, the A992 specification includes increased control on trace elements and can be more weldable than some A572 steels. It is recommended that either A992 or A913 steel be used in SMF applications.*

2.6.2 Material Strength Properties

The strength of materials shall be taken as indicated in the *AISC Seismic Provisions* and as modified by these *Recommended Criteria*. Where these *Recommended Criteria* require the use of “expected strength,” this shall be the quantity $R_y F_y$ as indicated in the *AISC Seismic Provisions*. The value of R_y for material conforming to *ASTM A992* shall be the same as for material conforming to *ASTM A572 Grade 50*. Where these *Recommended Criteria* require the use of lower-bound strength, or specified strength, the minimum specified value of the yield strength F_y as indicated in the applicable ASTM specification shall be used.

Commentary: The AISC Seismic Provisions specify values of R_y for various materials as indicated in Table 2-1. The quantity R_yF_y is intended to approximate the mean value of the yield strength of material produced to a given specification and grade. The AISC Seismic Provisions permit other values of R_y to be used, if the value of the expected mean yield strength F_{ye} is determined by appropriate testing.

Table 2-1 Values of R_y for Various Material Grades

Material Specification	R_y
ASTM A36	1.5
ASTM A572 Gr. 42	1.3
Other Specifications	1.1

As part of the program of investigations conducted in support of the development of these Recommended Criteria, studies of the statistical variation in strength properties of rolled sections of Grade 50 steel were conducted. These studies indicate that the 1.1 value for R_y is a good representation of the mean value of yield strength when applied to the webs of cross sections. The flexural properties of structural steel, however, are more closely related to the yield strength of the flanges of rolled shapes, which tend to have somewhat lower strength than do the webs. When applied to calculations of the flexural strength *of beams, the use of an R_y value of 1.1 actually approximates a mean-plus-one-standard-deviation value. Since values of expected strength are used to estimate the amount of force that can be delivered to adjacent connected elements, the use of this conservative value is appropriate. More information on the statistical variation of steel strength may be found in FEMA-355A, State of the Art Report on Base Metals and Fracture.*

2.7 Structural Analysis

An analysis should be performed for each structure to determine the distribution of forces and deformations under code-specified ground motion and loading criteria. The analysis should conform, as a minimum, to the code-specified criteria for the equivalent lateral force method or the modal response spectrum method, as applicable.

Chapter 4 provides guidance on analysis methods that can be used as part of the Performance Evaluation approach for steel moment-frame structures.

Commentary: Seismic design forces for low-rise and mid-rise buildings without major irregularities have traditionally been determined by using the simple "equivalent lateral force" method prescribed by the codes. Such methods are incorporated in FEMA-302 and are permitted to be used for structures designated as regular, and up to 240 feet in height. Buildings that are over 5 stories or 65 feet in height and have certain vertical irregularities, and all buildings over 240 feet in height, require use of dynamic (modal or response history) analysis. The use of inelastic response history or nonlinear static analysis is also permitted by some codes though few guidelines are provided in codes on how to perform or apply such an analysis. Projects incorporating nonlinear response-history analysis should be conducted in accordance with the Performance Evaluation provisions of Chapter 4. For such applications, structures should be demonstrated as capable, with 90% confidence, of providing Collapse Prevention performance for MCE hazards based on considerations of global behavior and column adequacy. A 50% confidence level should be demonstrated for connection behavior.

2.8 Mathematical Modeling

2.8.1 Basic Assumptions

In general, a steel moment-frame building should be modeled, analyzed and designed as a three-dimensional assembly of elements and components. Although two-dimensional models may provide adequate design information for regular, symmetric structures and structures with flexible diaphragms, three-dimensional mathematical models should be used for analysis and design of buildings with plan irregularity as defined by *FEMA-302*. The two-dimensional modeling, analysis, and design of buildings with stiff or rigid diaphragms is acceptable, if torsional effects are either sufficiently small to be ignored, or are captured indirectly.

Vertical lines of framing in buildings with flexible diaphragms may be individually modeled, analyzed and designed as two-dimensional assemblies of components and elements, or a three-dimensional model may be used, with the diaphragms modeled as flexible elements.

Explicit modeling of connections is required only for nonlinear procedures and only if (1) the connection is weaker than the connected components, or (2) the flexibility of the connection results in a significant increase in the relative deformation between connected components. Additional guidance in using these methods is found in Chapter 4.

Commentary: A finite-element model will provide information on forces and deformations only at places in the structure where a modeling element is inserted. When nonlinear deformations are expected in a structure, the designer must anticipate the location of the plastic hinges and insert nonlinear finite elements at these locations if the inelastic behavior is to be captured by the model. Additional information is found in Chapter 4.

2.8.2 Model Configuration

The analytical model should accurately account for the stiffness of frame elements and connections and other structural and nonstructural elements that may affect this stiffness. This section presents basic recommendations for analyses intended to meet the requirements of *FEMA-302*. More detailed modeling guidelines for the purposes of performance evaluation are presented in Chapter 4. Chapter 3 presents specific modeling guidelines for various prequalified connections, referred to by the guidelines of Section 2.8, and Chapter 4.

2.8.2.1 Regularity

Classification of a building as irregular, and analysis limitations based on regularity are discussed in *FEMA-302*. Such classification should be based on the plan and vertical configuration of the framing system, using a mathematical model that considers relevant structural members.

2.8.2.2 Elements Modeled

For the purpose of determining the adequacy of the structure to meet the strength and drift requirements of *FEMA-302*, only participating elements of the seismic-force-resisting system shall be included in the analytical model. When nonstructural or nonparticipating elements of the seismic-force-resisting system have significant influence on the stiffness or distribution of seismic forces within the elements of the seismic-force-resisting system, separate analyses should be performed to evaluate the effect of these elements on (1) the distribution of deformations and member forces, and (2) overall building performance.

Commentary: In order to comply with the requirements of FEMA-302, it is necessary that the seismic-force-resisting system be capable of resisting the design seismic forces without participation of other elements. However, steel moment-frame structures are inherently flexible. Rigid supported elements *including architectural wall systems, ramped floors, and large mechanical equipment items can affect both the stiffness of the structure and the distribution of forces within the structure. The best practice in the design and detailing of steel moment-frame structures is to detail elements that are not part of the seismic-force-resisting system such that they are isolated from participating in the resistance of earthquake-induced frame drifts. For those cases when such isolation is not possible, the effect of these elements on the behavior of the frame should be considered in the design.*

FEMA-302 does not permit consideration of elements that are not part of the primary lateral-force-resisting system as effective in meeting the strength and stiffness requirements of the provisions. However, in many steel moment-frame structures, framing provided only to resist gravity loads can provide substantial additional stiffness and strength. It is recommended that the effect of these nonparticipating structural elements be considered when performing analyses in support of performance evaluations, conducted in accordance with Chapter 4 of these Recommended Criteria.

2.8.2.3 Connection Stiffness

For frames with fully restrained connections, it shall be permissible to model the frame using centerline-to-centerline dimensions for the purpose of calculating stiffnesses of beams and columns. Alternatively, when justified by appropriate analytical or test data, more realistic assumptions that account for the stiffness of panel zones and connections may be used. In either case, calculation of beam moments and shears should be performed at the face of the column.

For linear analysis of structures with partially restrained connections, beams should be modeled with an equivalent *EI*, using the method shown in Chapter 5 of *FEMA-273*. Chapter 3 of these *Recommended Criteria* provides guidelines for estimating connection stiffness parameters for use in this procedure for the various prequalified partially restrained connections. For nonlinear analysis of frames with partially restrained connections, the nonlinear force-deformation characteristics of the connections should be directly modeled.

Commentary: In analytical studies of moment-resisting frame behavior (FEMA-355C) conducted in support of the development of these Recommended Criteria, it has been demonstrated that panel-zone deformations have little effect on analytical estimates of drift and need not be explicitly modeled, provided the panel zones are not excessively weak. Inelastic analyses of frames designed in accordance with these Recommended Criteria indicate that explicit modeling of panel zone shear strength and flexibility results in similar, albeit slightly smaller estimates of interstory drift than is obtained from models in which panel zones are not modeled and center-line-to-center-line framing dimensions are used. Therefore, this document recommends use of the simpler approach, in which panel zones are neglected in the model and center-line-to-center-line framing dimensions are used. It is permissible to use realistic assumptions for the *stiffness of panel zones, to modify the effective flexural span length of beams and columns, provided that such assumptions are based on appropriate data. Some connections, such as large haunches or slide plates, may significantly increase frame stiffness, meriting the inclusion of their effects in the analytical model. Additional discussion on modeling considerations, including methods to model connections and panel zones explicitly may be found in FEMA-355C, State of the Art Report on Systems Performance.*

2.8.3 Horizontal Torsion

The effects of horizontal torsion must be considered, as in *FEMA-302*. The total torsional moment at a given floor level includes the following two torsional moments:

a. Actual torsion: the moment resulting from the eccentricity between (1) the centers of mass at all floors above and including the given floor, and (2) the center of rigidity of the vertical seismic elements in the story below the given floor, and

b. Accidental torsion: an accidental torsional moment produced by an artificial horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

When the effects of torsion are investigated, the increased forces and displacements from horizontal torsion should be evaluated and considered for design. The effects of torsion cannot be used to reduce force and deformation demands on components and elements.

Commentary: Actual torsion that is not apparent in an evaluation of the center of rigidity and center of mass in an elastic stiffness evaluation can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner. For example, if the frames on the east and west sides of a structure have similar elastic stiffness the structure may not have significant torsion during elastic response. However, if the frames on the east side of the structure yield significantly sooner than the framing on the west side, then inelastic torsion will develop. Although the development of such inelastic torsion can be a serious problem, FEMA-302 does not address these phenomena. Designers can reduce the potential for severe inelastic torsion by providing framing layouts that have both stiffness and strength as symmetrical as possible about the center of mass.

2.8.4 Foundation Modeling

Foundations should generally be modeled as unyielding. Soil-structure interaction may be modeled as permitted by the building code. Assumptions for the extent of fixity against rotation provided at the base of columns should realistically account for the relative rigidities of the frame and foundation system, including soil compliance effects, and the detailing of the column base connections.

Commentary: Most steel moment frames can be adequately modeled by assuming that the foundation provides rigid support for vertical loads. However, the flexibility of foundation systems (and the attachment of columns to those systems) can significantly alter the flexural stiffness at the base of the frame. Where relevant, these factors should be considered in developing the analytical model.

2.8.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic framing system. Connections between diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for design and detailing of diaphragm components are given in *FEMA-302*.

Diaphragms should be classified as flexible, stiff, or rigid in accordance with *FEMA-302*. For buildings with steel moment-frame systems, most floor slabs with concrete fill over metal deck may be considered to be rigid diaphragms. Floors or roofs with plywood diaphragms should be considered flexible. The flexibility of unfilled metal deck, and concrete slab diaphragms with large openings should be considered in the analytical model.

Mathematical models of buildings with diaphragms that are not rigid should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

2.8.6 *P-D* Effects

The structure shall be investigated to ensure that lateral drifts induced by earthquake response do not result in a condition of instability under gravity loads. At each story, the quantity Y_i should be calculated for each direction of response, as follows:

$$Y_i = \frac{P_i R d_i}{V_{yi} h_i} \tag{2–1}$$

where:

P_i = portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on all of the columns within story level *i*, kips,

R = response modification coefficient obtained applicable to the structural system and used to determine the design seismic forces

δ_i = calculated lateral drift at the center of rigidity of story *i*, when the design seismic forces are applied in the direction under consideration, inches,

V_{yi} = total plastic lateral shear force in the direction under consideration at story *i*,

h_i = height of story *i*, which may be taken as (1) the distance between the centerline of floor framing at each of the levels above and below, (2) the distance between the top of floor slabs at each of the levels above and below, or (3) the distance between similar common points of reference.

Commentary: The quantity Y_i is the ratio of the effective story shear produced by first order P-D effects at the calculated story drift to the maximum restoring force in the structure. When this ratio has a value greater than 1.0, the structure does not have enough strength to resist the P-D induced shear forces and unless restrained, will collapse in a sidesway mechanism. If the ratio is less than 1, the restoring force in the structure exceeds the story shear due to P-D effects and unless additional displacement is induced or lateral forces applied, the structure should not collapse.

The plastic story shear quantity, V_{yi}, should be determined by methods of plastic analysis. In a story in which all beam-column connections meet the strong-column-weak-beam criterion, the same number of moment-resisting bays is present at the top and bottom of the frame and the strength of moment-connnected girders at the top and bottom of the frame is similar, V_{yi} may be approximately calculated from the equation:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pG\,j}}{h_i} \tag{2-2}$$

where:

M_{pGj} = *the plastic moment capacity of each girder "j" participating in the moment-resisting framing at the floor level on top of the story, and*

n = *the number of moment-resisting girders in the framing at the floor level on top of the story.*

In any story in which all columns do not meet the strong-column-weak-beam criterion, the plastic story shear quantity, V_{yi} may be calculated from the equation:

$$V_{yi} = \frac{2\sum_{k=1}^{m} M_{pC_k}}{h_i} \tag{2-3}$$

where:

m = the number of columns in moment-resisting framing in the story under consideration, and

M_{pCk} = the plastic moment capacity of each column "k", participating in the moment-resisting framing, considering the axial load present on the column.

For other conditions, the quantity V_{yi} must be calculated by plastic mechanism analysis, considering the vertical distribution of lateral forces on the structure.

In any story in which Y_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity Y_i in a story exceeds 0.1, the analysis of the structure should explicitly consider the geometric nonlinearity introduced by *P-D* effects. Most linear dynamic analysis software packages have the ability to consider *P-D* effects automatically. For nonlinear analysis procedures, second-order effects should be considered directly in the analysis; the geometric stiffness of all elements and components subjected to axial forces should be included in the mathematical model. When Y_i in a story exceeds 0.3, the structure shall be considered unstable, unless a detailed global stability capacity evaluation for the structure, considering *P-D* effects, is conducted in accordance with the guidelines of Appendix A.

Commentary: P-D effects can have very significant impact on the ability of structures to resist collapse when subjected to strong ground shaking. When the non-dimensional quantity, Y, calculated in accordance with Equation 2-3 significantly exceeds a value of about 0.1, the instantaneous stiffness of the structure can be significantly decreased, and can effectively become negative. If earthquake induced displacements are sufficiently large to create negative instantaneous stiffness, collapse is likely to occur.

Analyses reported in FEMA-355F, State of the Art Report on Performance Prediction and Evaluation, included direct consideration of P-D effects in determining the ability of regular, well configured frames designed to modern code provisions to resist P-D-induced instability and P-D-induced collapse. For regular, well configured structures, it is believed that if the value of Y is maintained within the limits indicated in this section, P-D-induced instability is unlikely to occur. Values of Y greater than this limit suggest that instability due to P-D effects is possible. In such cases, the frame should be reconfigured to provide greater resistance to P-D-induced instability unless explicit evaluation of these effects using the detailed Performance Evaluation methods outlined in Appendix A are performed.

The evaluation approach for P-D effects presented in this section appears similar to but differs substantially from that contained in FEMA-302, and in use in the building codes for many years. The approach contained in FEMA-302 and the building codes was an interim formulation. The research conducted in support of these Recommended Criteria indicates that this interim approach was not *meaningful. Some of the research performed in support of these Recommended Criteria included the explicit evaluation of P-D effects for buildings of varying heights, subjected to many different types of ground motion, and designed using different building code provisions . Using these and other parameters, several tens of thousands of nonlinear analyses were run to investigate P-D effects. A complete discussion of the analyses supporting these recommendations may be found in FEMA-355F. Extensive additional discussion on the issue of P-D effects and their importance in the response of structures at large interstory drifts is contained in FEMA-355C, State of the Art Report on Systems Performance.*

2.8.7 Multidirectional Excitation Effects

Buildings should be designed for seismic forces incident from any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of the building. For buildings with plan irregularity and buildings in which one or more components form part of two or more intersecting frames, multidirectional excitation effects should be considered. Multidirectional effects on components should include both torsional and translational effects.

The requirement that multidirectional (orthogonal) excitation effects be considered may be satisfied by designing frames for the forces and deformations associated with 100% of the seismic displacements in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction. Alternatively, it is acceptable to use the square root of the sum of the squares (SRSS) to combine multidirectional effects where appropriate.

2.8.8 Vertical Excitation

The effects of vertical excitation on horizontal cantilevers and prestressed elements should be considered by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum unless alternative vertical response spectra are developed using site-specific analysis. Vertical earthquake effects on other beams and column elements should be evaluated for adequacy to resist vertical earthquake forces, as specified in *FEMA-302*.

Commentary: There is no evidence that response to vertical components of ground shaking has had any significant effect on the performance of steel moment-frame structures. Consequently, the effect of this response is not recommended for consideration in the performance evaluation of these buildings, except as required by the building code.

Traditionally, vertical response spectra, when considered, have been taken as 2/3 of the horizontal spectra developed for the site. While this is a reasonable approximation for most sites, vertical response spectra at near-field sites, located within a few kilometers of the zone of fault rupture can have substantially *stronger vertical response spectra than indicated by this rule. Development of site-specific response spectra is recommended when vertical response must be considered for buildings on such sites.*

2.9 Frame Design

The following provisions supplement the parallel provisions contained in the 1997 *AISC Seismic Provisions*.

2.9.1 Strength of Beams and Columns

Multi-story frames should be designed with a strong-column-weak-beam configuration, to avoid the formation of single-story mechanisms. As a minimum, Equation 9-3 of the 1997 *AISC Seismic Provisions* should be satisfied. In the application of Equation 9-3, the quantity M_c as defined in Section 3.2.6 of these *Recommended Criteria* should be substituted for the quantity M^{*}_{pb}.

Commentary: When subjected to strong ground shaking, multi-story structures with columns that are weaker in flexure than the attached beams can form single-story mechanisms, in which plastic hinges form at the base and top of all columns in a story. Once such a mechanism forms in a structure, nearly all of the earthquake-induced lateral displacement will occur within the yielded story, which can lead to very large local drifts and the onset of P-D instability and collapse.

Although weak-column-strong-beam designs are not desirable, the 1997 AISC Seismic Provisions does permit their use under certain conditions, even for Special Moment Frames. Before utilizing weak-column-strong-beam configurations, designers should be aware that the prequalified connections for Special Moment Frames contained in these Recommended Criteria are based on tests using strong columns. When considering moment frames which include columns deployed in the weak direction, designers should be aware that only one connection type (RBS, Section 3.5.5) has been tested for use with weak-direction columns for application in Special Moment Frames and, although those tests were successful, insufficient data exists to prequalify such connections.

Nonlinear analyses of representative frames have clearly shown that the use of the provisions described above will not completely prevent plastic hinging of columns. This is because the point of inflection in the column may move away from the assumed location at the column mid-height once inelastic beam hinging occurs, and because of global bending induced by the deflected shape of the building, of which the column is a part.

Except for the case when a column hinge mechanism forms, column hinging is not a significant problem, provided that the columns are designed as compact sections, are properly braced and axial loads are not high. It is well understood *that a column hinge will form at the base of columns which are continuous into a basement, or which are rigidly attached to a stiff and strong foundation.*

2.9.2 Lateral Bracing of Column Flanges

Lateral bracing of column flanges, at beam-column connections should be provided whenever the following equation is not satisfied:

$$\frac{\sum M^*_{pc}}{\sum M_c} \geq 2.0 \qquad (2\text{-}4)$$

where:

M^*_{pc} is the quantity defined in Section 9.6 of the 1997 *AISC Seismic Provisions*

M_c is calculated as indicated in Section 3.2.6 of these *Recommended Criteria*.

Commentary: The relationship indicated in Equation 2-4 has been included in proposals for the 2000 NEHRP Recommended Provisions for New Buildings (now under consideration by the Building Seismic Safety Council) as a trigger for requirements for lateral bracing of column flanges. Large axial loads reduce the ductility of column hinges. Consideration should be given to applying larger factors for columns with axial loads exceeding 50% of the critical column load.

Bracing of the column at the location of the beam top flanges is normally supplied by the interconnection of the concrete slab, where such a slab occurs. At the location of the beam bottom flanges, sufficient lateral bracing can sometimes be shown to be provided by perpendicular beams and connected stiffeners for shallow column sections with wide flanges. Deeper beam-type sections, when used as columns, are typically less stable and normally will require direct lateral bracing of the flanges. See Section 2.9.6 for further guidelines on use of deep sections as columns.

2.9.3 Panel Zone Strength

Panel zones should conform to the strength requirements of Section 3.3.3.2 of these *Recommended Criteria* and the requirements of the individual prequalified connection design procedures.

Commentary: Connection performance can be affected either positively or negatively by panel zone strength. Some shear yielding of the panel zone can relieve the amount of plastic deformation that must be accommodated in other regions of the frame and many connections have been found to provide the largest inelastic deformation capacity when yielding is balanced between the panel zone and other connection elements. However, excessive panel zone deformation can induce large secondary stresses into the connection that can degrade connection *performance and increase fracture toughness demand on welded joints, and can also cause deformations which are undesirable for column performance. For this reason, the individual connection prequalifications include limitations on panel zone strength relative to beam strength.*

2.9.4 Section Compactness Requirements

Beams should conform to the section compactness requirements of *AISC Seismic Provisions*. Columns should also be compact, unless it can be shown by nonlinear analysis that the columns will not yield in response to the design earthquake.

Commentary: The 1997 AISC Seismic Provisions provide section compactness requirements for beams used in moment frames, and for columns which may be subjected to hinging. The effect of beam flange b/t as it relates to connection performance is discussed in Section 3.3.1.1 Beam Flange Stability. The effect of beam section compactness on overall frame performance is directly related to how local buckling affects strength degradation of individual beams and columns in the frame. Flange local buckling and lateral torsional buckling are sources of strength degradation.

It should be noted that for Reduced Beam Section (RBS) connections, the b/t in the area of beam hinging is reduced by the flange reduction, thereby reducing the propensity for flange local buckling. This may justify use of sections which are otherwise non-compact in frames employing these connections. See Section 3.3.1.1 for recommendations.

2.9.5 Beam Lateral Bracing

The 1997 *AISC Seismic Provisions* require bracing of flanges of beams for Special Moment Frame systems. The unbraced length between supports is not permitted to exceed the quantity $2500\, r_y / F_y$. In addition, lateral supports are required where analysis indicates that a plastic hinge will form during inelastic deformations of the Special Moment Frame. General bracing of Special Moment Frame beams should conform to the AISC requirements. For bracing of beams at plastic hinges, refer to Section 3.3.1.5.

2.9.6 Deep Columns

The prequalified connections included in Chapter 3 of these *Recommended Criteria* are not prequalified for use with deep (beam-type) sections used as columns. The prequalified connections should only be used with W12 and W14 column sections.

Commentary: Nearly all of the beam-column connection assemblies tested as part of this project, as well as by other researchers, utilized W14 column sections. In recognition of the fact that some designers prefer to use W24 or other deep section columns in order to increase frame stiffness economically, two tests of reduced beam section assemblies with W24 columns were conducted. These *assemblies performed poorly and one column failed through development of a fracture between the column web and flange. This fracture resulted from the combined effects of local torsional instability of the column and the presence of low-toughness material at the flange-to-web region, sometimes referred to as the k-area. The problem of low toughness material at the k-area of rolled structural shapes is a well documented phenomena related to the straightening practice used by some mills for certain ranges of shape. Additional information on this phenomena may be found in FEMA-355A, State of the Art Report on Base Materials and Fracture. However, there is relatively little data available on the instability of deep section columns in moment-resisting connections and this project was not able to develop adequate data on this effect to allow prequalification of connections with deep columns.*

2.9.7 Built-Up Sections

The prequalified connections included in Chapter 3 of these *Recommended Criteria* have not been tested with built-up sections used as beams or columns. The prequalified connections should only be used with such sections when it can be shown that the built-up section conforms with all of the requirements for rolled sections as specified for the connection to be used. Of particular concern should be the strength of the web-to flange connection of the built-up section.

2.10 Connection Design

Chapter 3 of these *Recommended Criteria* provides criteria for the design and detailing of several types of prequalified Fully Restrained (FR) and Partially Restrained (PR) connections. These prequalified connections are recommended as acceptable for use in steel moment-frame systems, within the limitations expressed in Chapter 3, without further qualification analyses or tests. Table 2-2 lists the prequalified connection details, and the systems for which they are prequalified. All of these prequalifications apply only to frames composed of wide flange beams connected to the major axis of wide flange columns.

In addition to the connections indicated in Table 2-2, Chapter 3 also provides information on several types of proprietary connections. Proprietary connections have not been prequalified by this project for service in specific systems. Engineers interested in the applicability of proprietary connections should obtain qualification information from the licensor.

For each connection in Table 2-2, a complete set of design criteria is presented in Chapter 3. Depending on the selected system type, the designer should select a suitable connection, then follow the design criteria to complete the design. Connections contained in Chapter 3 may be used in applications outside the indicated range of prequalification provided that a project-specific qualification program is followed, as indicated in Section 3.9.

Connection types not prequalified under the guidelines of Chapter 3 may also be used, subject to the project-specific qualification procedures.

Table 2-2 Prequalified Connection Details

Category	Connection Description	Acronym	Permissible Systems
Welded, fully restrained	Welded Unreinforced Flanges, Bolted Web	WUF-B	OMF
	Welded Unreinforced Flanges, Welded Web	WUF-W	OMF, SMF
	Free Flange	FF	OMF, SMF
	Welded Flange Plate	WFP	OMF, SMF
	Reduced Beam Section	RBS	OMF, SMF
Bolted, fully restrained	Bolted, Unstiffened End Plate	BUEP	OMF, SMF
	Bolted, Stiffened End Plate	BSEP	OMF, SMF
	Bolted Flange Plates	BFP	OMF, SMF
Bolted, partially restrained	Double Split Tee	DST	OMF, SMF

Commentary: For each of the prequalified connection types indicated in Table 2-2, sufficient laboratory testing, together with related analytical work, has been performed to provide an ability to predict with confidence the limiting modes of behavior for the connection when properly constructed and the probability that the connection will be able to sustain certain levels of inelastic deformation. This confidence only applies to application within certain limits, including material specifications, and member sizes. If a design falls outside the range of prequalification for a connection detail, it is necessary to extend the existing qualification for use in the specific application, by performing additional laboratory prototype testing. Chapter 3 indicates the extent of the additional testing recommended to extend connection qualification, on a project-specific basis, as well as more general recommendations for prequalifying connection details for broader application.

2.11 Specifications

FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications presents supplemental recommendations for fabrication and erection of steel moment-frame structures. These supplemental recommendations address welding and base materials, methods of fabrication and quality assurance. It is recommended that project specifications include the specific paragraphs of *FEMA-353* that are applicable to the design being used.

Commentary: FEMA-353 is written in the form of supplemental provisions to the existing provisions of the building codes, FEMA-302, and standard AISC, AWS and ASTM specifications. It is expected that eventually, these standard specifications and provisions will be amended to adopt the supplemental provisions recommended by FEMA-353. In the interim, the applicable sections and paragraphs can be reproduced in individual project specifications. When this *is done, it is recommended that the specific language taken from the reference be used without modification and attributed to the source, so that fabricators and erectors can readily recognize and become accustomed to the use of the FEMA-353 requirements.*

2.12 Quality Control and Quality Assurance

FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications provides complete guidelines and commentary for Quality Control and Quality Assurance. The designer should utilize those guidelines to ensure the proper selection and handling of materials and shop and field fabrication of moment-frame connections.

Commentary: FEMA-353 has a complete discussion of quality control recommendations and the reasons for them. Quality control and quality assurance are important for the achievement of the intended performance.

2.13 Other Structural Connections

2.13.1 Column Splices

Column splices in moment frames should be designed to develop the full bending and shear strength of the column, unless an inelastic analysis is performed to determine the largest axial loads, moments and shears likely to occur at the location of the splice and the splice detail can be shown to be adequate to resist these axial loads, moments and shears, considering stress concentrations inherent in the types of joints being used.

Welded flange splices may be made either with full penetration groove welds, or with splice plates fillet welded to the column flanges. Weld metal with a minimum rated toughness as described in Section 3.3.2.4 should be used and weld tabs should be removed. Bolted column flange splices should be designed to preclude net section fracture, block shear failure, and bolt pull-through failure of the column flange or of the splice plates.

Column web splices may be either bolted or welded, or welded to one column piece and bolted to the other. Bolted splices using plates or channels on both sides of the column web are preferred because of the inherent extra safety afforded by “capturing” the web. Partial Joint penetration welded web splices are not recommended. Column web splices should be designed to resist the maximum shear force that the column is capable of producing.

Splices of columns that are not a part of the seismic-force-resisting system should be made in the center one-third of the column height, and should have sufficient shear capacity in both orthogonal directions to maintain the alignment of the column at the maximum shear force that the column is capable of producing.

Commentary: Section 8.3 of the 1997 AISC Seismic Provisions specifies requirements for design of column splices for columns that are part of the *seismic-force-resisting system. The requirements prohibit splices made with fillet welds or partial penetration groove welds located within four feet or within one-half the column clear height of the beam-to-column connections. This prohibition is because fillet welds in tensile applications and partial penetration butt welds are both details with relatively low tensile capacity and poor inelastic capability. For typical cases, the prohibition against such splices within four feet of a beam-column joint will control. The one-half column height requirement is intended to apply to those rare cases when the clear column height is less than eight feet. The 1997 AISC Seismic Provisions permit such splices in the mid-height zone of columns based on the belief that large flexural demands, and in particular inelastic demands are unlikely to occur in this region. Inelastic analyses of frames, however, clearly demonstrate that this presumption is incorrect for frames subjected to seismic loadings that exceed their elastic capacity. For this reason, as well as the severe potential consequences of column splice failure, the 1997 AISC Seismic Provisions are not considered to be sufficiently conservative in this area.*

Because bending and axial stresses at column splice welds may be high, it is recommended that weld filler metals with rated notch toughness be used for these splices and that runoff tabs be removed. Where CJP welds are used, removal of backing is not judged to be necessary because the configuration of backing for column-to-column flange welds is not conducive to crack formation, as it is for the right-angle condition of beam-to-column flange joints. Properly designed bolted flange splices may be shown to be adequate for some column splice applications.

Bolted web connections are preferred by many engineers and contractors because they have advantages for erection, and, when plates are placed on both sides of the web, they are expected to maintain alignment of the column in the event of a flange splice fracture. Partial joint penetration welded webs are not recommended, because fracture of a flange splice would likely lead to fracture of the web splice, considering the stress concentrations inherent in such welded joints.

Inelastic analyses have shown the importance of the columns that are not part of the seismic-force-resisting system in helping to distribute the seismic shears between the floors. Even columns that have beam connections that act as pinned connections may develop large bending moments and shears due to non-uniform drifts of adjacent levels. For this reason, it is considered to be important that splices of such columns be adequate to develop the shear forces corresponding to development of plastic hinges at the ends of the columns in both orthogonal directions.

2.13.2 Column Bases

Column bases can be of several different types, as follows:

1. The column may continue into a basement, crawl space, or grade beam, in such a way that the column's fixity is assured without the need for a rigid base plate connection.
2. Large columns may be provided at the bottom level to limit the drift, and a "pinned base" may be utilized.
3. A connection which provides partial fixity may be provided, so that the column base is fixed up to some column moment, but the base itself yields before the column hinges.
4. A heavy base plate assembly may be provided which is strong enough to force yielding in the column.

In all of these cases, the designer should consider the base connection as similar to a beam-to-column connection and apply similar principles of design and detailing.

Notes:

1. For the first case above, the designer should recognize that hinging will occur in the column, just above the first floor. The horizontal shear to be resisted at the ends of the column in the basement level should be calculated considering the probable overstrength of the framing.
2. For the "pinned base", the designer should ensure that the required shear capacity of the base can be maintained up to the maximum rotation that may occur.
3. In designing a base with partial fixity , the designer should consider the principles used in the design of partially-restrained connections. This type of base may rely on bending of the base plate (similar to an end plate connection), bending of angles or tees, or yielding of anchor bolts. In the latter case, it is necessary to provide bolts or rods with adequate elongation capacity to permit the required rotation and sufficient unrestrained length for the yielding to occur. Shear capacity of the base plate to foundation connection must be assured at the maximum rotation.
4. For the fully fixed base, the designer should employ the same guidelines as given for the rigid fully-restrained connections. Such connections may employ thick base plates, haunches, cover plates, or other strengthening as required to develop the column hinge. Where haunched type connections are used, it must be recognized that the hinging will occur above the haunch, and appropriate consideration should be given to the stability of the column section at the hinge.

Commentary: It is well recognized that achievement of a mechanism in a moment frame requires a hinge at, or near to, the base of the column. The column base detail must accommodate the required hinging rotations while maintaining the strength required to provide the mechanism envisioned by the designer. These conditions are similar to the requirements for beam-to-column connections, as described.

2.13.3 Welded Collectors and Chords

Connections of highly loaded collectors and chords are often made with welded or bolted flange details comparable to those employed in moment frames. Design of such connections should incorporate the principles applied to moment-frame connections, unless it can be shown that the connection will remain elastic under the combination of the axial load, calculated at the limit strength of the system, and the corresponding rotation due to building drift.

Commentary: The rotational demand on rigid connections made for other purposes are often comparable to those of moment-frame beams. When coupled with high axial loads, demands on welded or bolted joints can be high. The principles of design for moment-frame beam-to-column connections are applicable to such conditions.

2.13.4 Simple Beam-to-Column Gravity Connections

Simple welded shear tab connections of beams to columns in buildings employing moment frames and other relatively flexible lateral-force-resisting systems should utilize details that have been demonstrated to have sufficient rotational capacity to accommodate the rotations that occur at the anticipated drifts, while maintaining capacity for the required gravity forces. In the absence of a more detailed analysis, adequate rotation capacity can be considered to be that associated with the design story drift calculated using the methods of *FEMA-302* multiplied by 1.5. As described in the commentary below, calculations to justify the adequacy of this condition should not be necessary under normal conditions.

When deep beams with deep bolt groups are connected to small columns, the columns should be compact, or sufficient rotational capacity should be provided in the connections to preclude hinging of the column when subjected to the drift calculated as described above.

Commentary: Research conducted under this project has shown that the plastic rotational capacity of simple bolted shear tab type connections, designed using the methods of the AISC LRFD Specification, and with adequate clearance of beam flanges from the column flanges to prevent bearing, is dependent on the depth of the bolt group, d_{bg}, and can reasonably be calculated as:

$$q_p = 0.15 - 0.0036 d_{bg} \tag{2-5}$$

where d_{bg} is the vertical dimension of the bolt group in inches. The additional elastic rotational capacity of these connections is estimated as about 0.02 radians. This gives a total estimated drift capacity for such connections of:

$$q_p = 0.17 - 0.0036 d_{bg} \tag{2-6}$$

The use of Equation 2-6 above will result in a calculated rotational capacity of more than 0.09 radian for an 8-bolt group with bolts spaced at 3”, which will *be more than adequate for most conditions. Where the calculated rotational angle is not sufficient, slotted holes in the shear tab, or other means of accommodating larger rotations should be used. It should be noted that rotation capacities for connections made with clip angles bolted to the beam have not been found to be significantly higher than those for welded shear tabs. Refer to FEMA-355D for additional information.*