5. LEVEL 2 DETAILED POSTEARTHQUAKE EVALUATIONS

5.1 Introduction

Detailed evaluation is the second step of the postearthquake evaluation process. It should be performed for all buildings that are estimated to have experienced potentially damaging ground motions, using the screening procedures of Section 3.2 of these *Recommended Criteria*. As detailed evaluation can be a time-consuming process, it is recommended that a preliminary evaluation, in accordance with the procedures of Chapter 3, be conducted prior to detailed evaluation, to permit rapid identification of those buildings that may have been so severely damaged that they pose an immediate threat to life safety.

Many steel moment-frame buildings damaged in past earthquakes have displayed few outward signs of structural or nonstructural damage. Consequently, except for those structures which have been damaged so severely that they are obviously near collapse, brief evaluation procedures, such as those of Chapter 3, are unlikely to provide a good indication of the extent of damage or its consequences. In order to make such determination, it is necessary to perform detailed inspections of the condition of critical structural components and connections. If structural damage is found in the course of such inspections, it is then necessary to make a determination as to the effect of discovered damage on the structure's ability to resist additional loading. Ultimately, decisions as to the significance of damage, whether occupancy should be permitted in a building and whether specific types of damage should be repaired must be made on the basis of quantitative evaluation and engineering judgement.

Chapter 4 provides a series of recommended criteria for a detailed evaluation method in which occupancy and repair decisions are made based on the calculation of damage indices based on the observed distribution of damage in the structure. The distribution of damage is determined on the basis of detailed inspections of fracture-susceptible connections. Although it is preferred that all fracture-susceptible connections be inspected, the procedures of Chapter 4 permit inspections to be limited to a representative sample. This chapter provides procedures for a detailed evaluation processes based on structural analysis of the damaged structure's ability to resist additional strong ground shaking. In order to perform such an analysis it is necessary to inspect all fracture-susceptible connections in the building in order to understand their condition.

> *Commentary: The Level 1 evaluation approach of Chapter 4 is based on the methodology developed immediately after the 1994 Northridge earthquake and first presented in FEMA-267. The Level 2 evaluation approach described in this chapter is a more comprehensive analytical approach that is compatible with the analytical methodology that forms the basis for design and performance evaluation criteria contained in the suite of FEMA/SAC publications on steel moment frames.*

5.2 Data Collection

Prior to performing a detailed evaluation, the original construction drawings should be reviewed (if available) to identify the primary lateral and gravity load-resisting systems, typical detailing, presence of irregularities, and other features pertinent to structural performance. Pertinent available engineering and geotechnical reports, including any previous damage survey reports and current estimates of ground motion intensity for the damage causing event, should also be reviewed. Specifications (including the original Welding Procedure Specifications) shop drawings, erection drawings, and construction records should be reviewed when available.

When structural framing information is not available, a comprehensive field study must be undertaken to determine the location and configuration of all vertical frames, and the details of their construction including member sizes, material properties, and connection configurations. A companion publication, *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings,* provides procedures for obtaining as-built information and determining material properties for steel moment-frame buildings.

Commentary: It is important to collect data on all framing, whether or not it was originally intended as part of the design to participate in the lateral force resistance of the structure. Studies have shown that vertical frames provided only for gravity load resistance can provide substantial supplemental stiffness and strength in steel moment-frame structures and the analytical procedures of this chapter include direct consideration of such framing. Data collection should obtain sufficient information on this framing, as well as that intended to provide the structure's lateral-force resistance to permit an accurate analytical model of the structure to be developed.

In addition to reviewing available documentation, a complete inspection of all critical framing and connections in the building should be undertaken, to determine their condition. Connections to be inspected include all fracture-susceptible moment-resisting framing connections and column splices. The following connections are considered to be fracture-susceptible:

- Moment-resisting beam-column connections in which the beams are connected to columns using full penetration welds between the beam flanges and column, and in which yield behavior is dominated by the formation of a plastic hinge within the beam at the face of the column, or within the column panel zone.
- Splices in the exterior columns of steel moment frames when the splices consist of (1) partial penetration groove welds between the upper and lower sections of the column, or (2) bolted connections that are incapable of developing the full strength of the upper column in tension.

Section 4.4.1.1 provides procedures for conducting connection inspections, and for classifying and recording any damage found.

Commentary: Most welded, moment-resisting beam-column connections constructed prior to 1994 will be of the fracture susceptible type described here. Following the 1994 Northridge earthquake, guidelines for improved connection designs and details were developed and were rapidly adopted throughout the western United States, particularly in zones of high seismicity, including California, Washington, Utah and Alaska. However, fracture-susceptible connections may exist in some post-1994 buildings, particularly those constructed in zones of lower seismicity.

5.3 Evaluation Approach

In a Level 2 evaluation, inspections are conducted of all critical structural elements and connections. An analytical model is then developed for the building representing its strength and stiffness in the damaged state and an analysis is performed to provide information on the residual capacity of the building to resist additional earthquake loading. The results of the analysis are used together with engineering judgement and evaluation of other important factors including the nature of the building's occupancy, the economic and other impacts of loss of building use and/or building failure, in order to form an opinion as to appropriate postearthquake disposition for the building. Alternative actions that may be appropriate in different situations include:

- Accept the damage as being stable and not detrimental to future building performance, in which case no repair action will be required.
- Determine that repairs of some or all of the damage must be undertaken to provide an acceptable level of risk for long-term occupancy, but that the building remains an acceptable risk for occupancy until such time as the repairs are completed.
- Determine that the building is an unacceptable risk for occupancy until such time as temporary stabilization or permanent repair can be undertaken.
- Determine that the building is an unacceptable risk for occupancy until such time as repair and structural upgrade can be undertaken.
- Determine that the building is an unacceptable risk for occupancy and impractical to repair and upgrade, in which case the building should be demolished.

A number of alternative analytical approaches may be used in support of the formation of recommendations for postearthquake building disposition. Individual engineers and building officials may choose to use any or perhaps several of these approaches, in support of the postearthquake decision making process:

- **Determine the capacity of the damaged building relative to current code requirements**. In this approach the ability of the damaged building to meet the strength and drift criteria specified by the building code for new construction is evaluated. Decisions relative to repair and occupancy are triggered based on the extent of compliance of the damaged building with new building requirements. For example, if a damaged building provides 90% or more of the strength and stiffness required of new buildings, and the damage is stable, i.e., not subject to further degradation, then it may be appropriate to accept the damage and conduct no repairs.

If the degraded strength or stiffness of the building fall below 50% of that required for a new building it may be appropriate to restrict occupancy. It should be noted that the engineer, and building official, may select any appropriate “trigger” rules when using this approach.

- **Determine the capacity of the damaged building relative to pre-earthquake conditions.** In this approach, the amount that the strength and stiffness of the building has degraded as a result of the damage incurred relative to the pre-earthquake condition is used as an index to guide decisions. For example, if a building retains 90% of the strength and stiffness that existed prior to the earthquake, and the damage is stable, than it may be appropriate to accept the damage and conduct no repairs. If the degraded strength or stiffness of the building fall below 50% of the pre-earthquake values, then it may be appropriate to restrict occupancy. As noted above, the engineer and building official may select any appropriate “trigger” rules when using this approach.
- **Determine the probability of earthquake-induced collapse of the damaged building**. In this approach, a direct evaluation of the building’s ability to resist collapse for a defined level of ground shaking (or at defined hazard probability) is determined and used as a basis for making decisions. For example, if analyses permit a high level of confidence to be developed that a damaged building can provide Collapse Prevention performance for ground shaking demands with a 10% chance of exceedance in 50 years, and the damage is stable, it may be appropriate to accept the damage, without repair. Similarly, if a high degree of confidence can not be developed that the damaged building could survive ground shaking demands with 50% chance of exceedance in 50 years, it may be decided to restrict building occupancy. Again, the specific “trigger” rules may be selected based upon the judgement of the engineer and building official.

The recommended criteria of this chapter adopt the last approach indicated above. Specifically, a methodology is provided whereby the engineer can determine a level of confidence with regard to the ability of the damaged building to resist a repeat of the same ground shaking that caused the initial damage, without collapse. If a high degree of confidence is obtained that the building could survive such ground shaking without significant risk to life safety then the building can remain occupied. If there is low confidence that the building can protect life safety in a repeat of the same event, then occupancy restrictions are recommended.

The basic tool used to implement any of the evaluation approaches described above is a structural analysis of the damaged building. In addition to presenting detailed criteria for the probabilistic evaluation process, this chapter also provides guidance on modeling of damaged structures that can be useful with any analytical approach selected by the engineer in assessing appropriate postearthquake actions.

Commentary: As noted, a number of different criteria have historically been used to determine whether a building has sustained so much damage that it should not continue to remain occupied. In all of these, the decision to post a building against occupancy is based on a finding that the building is likely to endanger life safety if subjected to additional strong ground shaking. Approaches that have most commonly been used in the past include:

- *comparison of the building's residual lateral-force-resisting capacity with that specified by the building code for design of new structures,*
- *comparison of the building's residual lateral-force-resisting capacity with that which existed prior to the onset of damage, and*
- *application of the engineer's judgment as to the extent which the building poses an imminent or extreme hazard.*

Each of these approaches has drawbacks. If a comparison of the building's residual lateral-force-resisting capacity with that specified by the building code is used, it will often be found that a building that has not been damaged or has only minimal damage falls below the trigger level that indicates a "dangerous" condition, just due to the fact that the building was designed to earlier editions of the code that had less stringent design criteria. This results in a paradox, in that engineers typically do not post buildings as "unsafe", even if they have low calculated lateral-force-resisting capacity, unless they have been severely damaged.

The second approach, in which the computed degradation of a building's lateral-force-resisting capacity is used as the measure of whether or not a building should be occupied is somewhat more attractive in that it provides a direct measure of the effect of the damage sustained on the safety of the building and thereby differentiates low-strength conditions that are a result of original design characteristics, as opposed to those resulting from damage. However, this approach is also somewhat flawed in that some buildings have significant over-strength and reserve capacity and can sustain substantial reduction in initial capacity without becoming hazardous.

Approaches limited to application of the engineers judgment are attractive to many engineers, but inherently arbitrary. Further, different engineers will form different judgments as to the hazard that damage has caused in a building and will recommend different posting actions.

Review of statistics of past earthquakes indicates that within the relatively brief period of a year or so following a major earthquake in a region, the most likely events that the region will experience are of a similar or reduced magnitude to the original shock. Therefore, these procedures recommend evaluation of damaged structures for their ability to resist collapse (ability to provide Collapse Prevention performance) for such an event. For the purposes of accounting for variability in the likely locations and magnitudes of major aftershocks, and also to permit development of confidence levels for ability of the building to provide Collapse Prevention performance, a one-year return period is assumed for an arbitrary aftershock, comparable in intensity at the building site to the initial shock. Variability in ground motion is somewhat arbitrarily accounted for by *assuming a distribution of likely ground shaking at the building site due to such an aftershock that has a mean value equal to that which caused the original damage and having a coefficient of variation of 0.5.*

The safety evaluation approach presented in this section is intended only for use in assessing whether a building should remain occupied while it is repaired, based on the probability of collapse during the period immediately following the earthquake. It is not intended as a tool for evaluating the adequacy of building performance over the longer term of the building's remaining life. For guidelines on such performance evaluations refer to the companion publication, FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.

5.4 Field Inspection

Prior to performing an analytical evaluation of building safety, a thorough inspection of the building should be conducted to determine its condition. This inspection should include visual inspection of all critical connections including moment-resisting beam-column connections and column splices, supplemented by nondestructive testing where visual inspection reveals the fracture-susceptible potential damage that cannot be quantified by visual means alone. Beam-column connections should be inspected, and the damage recorded, as indicated in Section 4.4.1.

Geologic site hazards such as fault rupture, landslide, rock fall, and liquefaction may influence the damage in a building and also its future performance. A detailed discussion of these hazards is provided in *FEMA 273* and should be considered as part of a postearthquake evaluation. The structure should be inspected to detect whether differential settlement has occurred as differential movement between columns in a frame has the potential to place severe demands on the moment connections.

Commentary: Foundation inspection is typically difficult to accomplish since most foundations are buried. In most cases, inspection of foundation condition can be performed by observing floors for indications of settlement. Where significant settlements are indicated, local excavation to expose the foundation condition for inspection should be considered.

5.5 Material Properties and Condition Assessment

In order to perform a meaningful evaluation, it is necessary to understand the structure's basic configuration, its condition, and certain basic material properties. Original construction documents, including the drawings and specifications, supplemented by damage survey reports, prepared in accordance with Chapter 4 of these *Recommended Criteria*, will provide sufficient data for the evaluation of most damaged steel moment-frame buildings, so long as the building was actually constructed in accordance with these documents. If no construction documents are available, then extensive field surveys may be required to define the structure's configuration, including the locations of frames, the sizes of framing elements and connection details, as well as the materials of construction.

5.5.1 Material Properties

The primary material properties required to perform analytical evaluations of a steel moment-frame building include the following:

- yield strength, ultimate tensile strength and modulus of elasticity of steel for the columns in the moment frames,
- yield strength, ultimate tensile strength and modulus of elasticity of steel for the beams in the moment frames,
- ultimate tensile strength and notch toughness of the weld metal in the moment-resisting connections, and
- yield and ultimate tensile strength of bolts in the moment-resisting connections.

Although structural steel is an engineered material, there can be significant variability in the properties of the steel in a building, even if all of the members and connection elements conform to the same specifications and grades of material. Exhaustive programs of material testing to quantify the physical and chemical properties of individual beams, columns, bolts, and welds are not justified and should typically not be performed. It is only necessary to characterize the properties of material in a structure on the basis of the likely statistical distributions of the properties noted above, with characteristic mean values and coefficients of variation. Knowledge of the material specification and grade that a structural element conforms to, and its approximate age will be sufficient to define these properties for nearly all evaluations.

In general, analytical evaluations of global building behavior are performed using expected or mean values of the material properties based on the likely distribution of these properties for the different grades of material present in the structure. Expected values are denoted in these procedures with the subscript “e”. Thus, the expected yield and ultimate tensile strength of steel are denoted, respectively, F_{ye} and F_{ue}. Some calculations of individual connection capacities are performed using lower-bound values of strength. Where lower-bound strength values are required, the yield and tensile strength are denoted as F_y and F_u, respectively. Lower-bound strengths are defined as the mean minus two standard deviations, based on statistical data for the particular specification and grade.

If original construction documents, including drawings and specifications are available, and indicate in an unambiguous manner the materials of construction to be employed, it will typically not be necessary to perform materials testing in a steel moment-frame building. When material properties are not clearly indicated on the drawings and specifications, or the drawings and specifications are not available, the material grades indicated in Table 5-1 may be presumed. Alternatively, a limited program of material sample removal and testing may be conducted to confirm the likely grades of these materials.

Table 5-1 Default Material Specifications for Steel Moment-Frame Buildings

Element Type	Age of Construction	Default Specification
Beams and Columns	1950-1960	ASTM A7, A373
	1961-1990	ASTM A36
	1990-1998	ASTM A572, Grade 50
	1999 and later	ASTM A992
Bolts	1950-1964	ASTM A307
	1964-1999	ASTM A325
Weld Filler Metal	1950-1964	E6012 or E7024[1]
	1964-1994	E70T4 or E70T7[2]
	1994-1999	See note 3

Note 1 Prior to about 1964, field structural welding was typically performed with the Shielded Metal Arc Welding (SMAW) process using either E6012 or E7024 filler metal. Neither of these electrode classifications are rated for specific notch toughness, though some material placed using these consumables may provide as much as 40 ft-lbs or greater notch toughness at typical service temperatures. It should be noted that due to other inherent characteristics of the moment resisting connection detailing prevalent prior to the 1994 Northridge earthquake, the presence of tough filler metal does not necessarily provide for reliable ductile connection behavior.

Note 2 During the period 1964-1994, the Flux Cored Arc Welding (FCAW) process rapidly replaced the SMAW process for field welding in building structures. Weld filler metals typically employed for this application conformed either to the E70T4 or E70T7 designations. Neither of these weld filler metals are rated for specific notch toughness.

Note 3 Following the 1994 Northridge earthquake, a wide range of weld filler metals were incorporated in steel moment-frame construction. Most of these filler metals had minimum ultimate tensile strengths of 70ksi and minimum rated toughness of 20 ft-lbs at -20^{o}F. However, due to the variability of practice, particularly in the period 1994-1996, a limited sampling of weld metal in buildings constructed in this period is recommended to confirm these properties.

If sampling is performed, it should take place in regions of reduced stress, such as flange tips at ends of simply supported beams, flange edges in the mid-span region of members of steel moment frames, and external plate edges, to minimize the effects of the reduced area. If a bolt is removed for testing, a comparable bolt should be reinstalled in its place. Removal of a welded connection sample must be followed by repair of the connection. When sampling is performed to confirm the grades of material present in a structure, mechanical properties should be determined in the laboratory using industry standard procedures in accordance with ASTM A-370.

For the purpose of analytical evaluation of steel moment-frame buildings, the expected and lower bound strength of structural materials shall be taken from Table 5-2, based on the age, material specification, and grade of material.

Commentary: In general, great accuracy in the determination of the material properties of structural steel elements in steel moment-frame buildings is neither justified nor necessary in order to perform reasonably reliable evaluations of building performance. The two most important parameters are the yield strengths of the beams and columns and the toughness of the weld metal.

Table 5-2 Lower Bound and Expected Material Properties for Structural Steel Shapes of Various Grades[2]

		Yield Strength (ksi)		Tensile Strength (ksi)	
Material Specification	**Year of Construction**	**Lower Bound**	**Expected**	**Lower Bound**	**Expected**
ASTM A7, A373	Pre - 1960	30	35	60	70
ASTM A36	1961-1990				
Group 1		41	51	60	70
Group 2		39	47	58	67
Group 3		36	46	58	68
Group 4		34	44	60	71
Group 5		39	47	68	80
ASTM A242, A440, A441	1960-1970				
Group 1		45	54	70	80
Group 2		41	50	67	78
Group 3		38	45	63	75
Group 4		38	45	63	75
Group 5		38	45	63	75
ASTM A572	1970 – 1997				
Group 1		47	58	62	75
Group 2		48	58	64	75
Group 3		50	57	67	77
Group 4		49	57	70	81
Group 5		50	55	79	84
A36 and Dual Grade 50	1990 – 1997				
Group 1		48	55	66	73
Group 2		48	58	67	75
Group 3		52	57	72	76
Group 4		50	54	71	76

Notes:

1 Lower bound values for material are mean minus two standard deviations from statistical data. Expected values for material are mean values from statistical data.

2. For wide-flange shapes produced prior to 1997, indicated values are representative of material extracted from the web of the section.

3. For material conforming to ASTM A992, the values for ASTM A572, Grade 50 may be used. No adjustment in values, per note 2, should be taken.

4. For structural plate, expected strength may be taken as 125% of the minimum specified value. Lower-bound strength should be taken as the minimum specified value.

Commentary: In general, great accuracy in the determination of the material properties of structural steel elements in steel moment-frame buildings is neither justified nor necessary in order to perform reasonably reliable evaluations of building performance. The two most important parameters are the yield strengths of the beams and columns and the toughness of the weld metal.

Weld Filler Metal

Welds in most steel moment-frame buildings constructed in the period 1964-1994 were made with the Flux Cored Arc Welding (FCAW) process, employing either E70T4 or E70T7 weld filler metal. This material generally has low notch toughness. Precise determination of the notch toughness of individual welds is not required in order to predict the probable poor performance of moment-resisting connections made with these materials and the detailing prevalent until 1994. However, if weld metal with significant notch toughness (40 ft-lbs at service temperature) has been used in a building, even connections of the type typically constructed prior to the 1994 Northridge earthquake can provide some limited ductility. It is rarely possible to determine the type of weld filler metal used in a building without extraction and testing of samples. Construction drawings and specifications typically do not specify the type of weld filler metal to be employed and even when they do, contractors may make substitutions for specified materials. Welding Procedure Specifications (WPS) for a project, if available, would define the type of weld filler metal employed, but these documents are rarely available for an existing building. Given the near universal use of the FCAW process with low toughness weld filler metal during the period 1964-1994, sampling of weld metal for buildings constructed in this period is not recommended. For buildings constructed prior to 1964, sampling and testing of weld filler metal may indicate the presence of weld metal with superior notch toughness, which would provide a higher level of confidence that the building would be capable of meeting desired performance objectives. Buildings constructed prior to 1964 may conservatively be assumed to be constructed using weld filler metal with low notch toughness, or samples may be extracted.

Most buildings constructed after 1996 employ weld filler metals with adequate notch toughness to provide ductile connection behavior. Sampling and testing of welds for buildings constructed in this period are not, therefore, deemed necessary. During the period 1994-96, many different types of weld filler metal were employed in buildings. Sampling and testing of weld filler metal in buildings of this period may be advisable.

When it is deemed advisable to verify the strength and notch toughness of weld filler metals, it is recommended that at least one weld metal sample be obtained and tested for each construction type (e.g., column-splice joint, beam-flange-to-column-flange joint). Samples should consist of both local base and weld metal, such that the composite strength of the connection can be assessed. If ductility is required at or near the weld, the design professional may conservatively assume, in lieu of testing, that no ductility is available.

Beams and Columns

The actual strength of beam and column elements in a steel moment-frame structure is only moderately important for the performance evaluation of such structures. The primary parameter used in these Recommended Criteria to evaluate building performance, is the interstory drift induced in the building by earthquake ground shaking. Building drift is relatively insensitive to the actual yield strength of the beams and columns. However, building interstory drift can be sensitive to the relative yield strengths of beams and columns. In particular, large interstory drifts can occur in buildings with weak columns and strong beams, as such conditions permit the development of a single story mechanism in which most of the building deformation is accommodated within the single story. During the 1970s and 1980s, it was common practice in some regions for engineers to specify beams of A36 material and columns of A572, Grade 50 material in order to develop economical designs with a strong-column-weak-beam configuration. If the properties of materials employed in a steel moment-frame building are unknown, it may be conservatively assumed that the beams and columns are of the same specification and grade of material, in accordance with the default values indicated in Tables 5-1 and 5-2. However, if it can be determined that different grades of material were actually used for beams and columns, it may be possible to determine a higher level of confidence with regard to the ability of a building to meet desired performance objectives. In such cases, it may be appropriate to perform a materials sampling and testing program to confirm the material specifications for beams and columns.

When it is decided to conduct a materials testing program to confirm the specification and grade of material used in beams and columns, it is suggested that at least two tensile strength coupons should be removed from each element type for every four floors. If it is determined from testing that more than one material grade exists, additional testing should be performed until the extent for each grade has been established.

Bolts

Bolt specifications may be determined by reference to markings on the heads of the bolts. Where head markings are obscured, or not present, the default specifications indicated in Table 5-1 may be assumed. If a more accurate determination of bolt material is desired, a representative sample of bolts should be extracted from the building and subjected to laboratory testing to confirm the material grade.

5.6 Structural Performance Confidence Evaluation

The basic process of postearthquake evaluation, as contained in these procedures, is to develop a mathematical model of the damaged structure, and by performing structural analysis, to determine the likelihood that the building will resist ground shaking demands that can be anticipated to occur during the immediate postearthquake period, without collapse. The structural analysis is used to predict the value of various structural response parameters. These include:

- interstory drift, and
- axial forces on columns and column splices.

These structural response parameters are related to the amount of damage experienced by individual structural components as well as the structure as a whole. These procedures specify acceptance criteria (median estimates of capacity) for each of the design parameters indicated above. Acceptability of structural performance is evaluated considering both local (element level) and global performance. Acceptance criteria have been developed on a reliability basis, incorporating demand and resistance factors related to the uncertainty inherent in the evaluation process, and variation inherent in structural response and capacity, such that a confidence level can be established with regard to the ability of a structure to provide specific performance at selected probabilities of exceedance.

Once an analysis is performed, predicted demands are adjusted by two factors, an analysis uncertainty factor g_a that corrects the analytically predicted demands for bias and uncertainty inherent in the analysis technique, and a demand variability factor g that accounts for other sources of variability in structural response. These predicted demands are compared against acceptance criteria, which have also been factored, by resistance factors, f, to account for uncertainties and variation inherent in structural capacity prediction. If the factored demands are less than the factored acceptance criteria (capacities), then the structure is indicated to be capable of meeting the desired performance, with at least a mean level of confidence. If the factored demands exceed the factored acceptance criteria, then there is less than a mean level of confidence that the desired performance will be attained. Procedures are given to calculate the level of confidence, based on the ratio of factored demand to factored capacity. If the predicted level of confidence is inadequate, then the occupancy of the structure should be suspended until such time as the structure can be temporarily shored, and/or repaired, and a suitable level of confidence attained. In some cases it may be possible to improve the level of confidence with regard to the ability of a building to resist collapse by performing a more detailed analysis. More detailed and accurate analyses allow better understanding of the structure's probable behavior to be attained, resulting in modifications to the demand and capacity factors.

Table 5-3 summarizes the recommended posting condition for a building, as a function of the level of confidence determined with regard to the structure's ability to resist collapse for the level of ground shaking likely to be experienced in the immediate postearthquake period. Refer to Table 3-2 for information on the recommended actions related to each posting.

Table 5-3 Recommended Occupancy Actions, Based on Detailed Evaluation

Confidence Level of Attaining Collapse Prevention Performance	Recommended Occupancy Posting
50% or greater confidence of non-collapse	Green-1, Green-2, or Green-3, as appropriate
25% or greater confidence of non-collapse but less than 50%	Red-1
Less than 25% confidence of non-collapse	Red-2

Note: Refer to Table 3-2 for explanation of postings.

Four alternative analytical procedures are considered by these recommendations, for the prediction of building response parameters. These are the same basic procedures contained in *FEMA-273* and include the Linear Static Procedure (LSP), the Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP). Section 5.8 outlines these procedures in some detail. The reader is referred to *FEMA-273* for additional information and discussion.

Commentary: These Recommended Criteria adopt a Demand and Resistance Factor Design (DRFD) model for performance evaluation. This approach is similar to the Load and Resistance Factor Design (LRFD) approach adopted by the AISC design specifications except that the LRFD provisions are conducted on an element basis, rather than structural system basis, and the demands in these procedures can be drifts as well as forces and stresses. The purpose of this DRFD approach is to quantify the level of confidence associated with estimation of a damaged building's ability to provide Collapse Prevention performance given the probable ground shaking that may be experienced in the period immediately following a damaging earthquake, taken as one year.

First, it is necessary to presume a hazard relationship for the site, during the immediate postearthquake period. Most strong earthquakes are followed by a large number of aftershocks, that decrease in frequency over time. Aftershocks typically occur on the same fault on which the main shock occurred, though, occasionally, an earthquake on a nearby fault has been triggered by the redistribution in crustal strains produced by the main shock. Aftershocks typically have less magnitude than the main shock, though there are some instances when an aftershock has actually exceeded the first shock. This forces a change in the naming of the two shocks, to foreshock and main shock. Generally, aftershock activity decays to insignificant levels within a period of approximately a year following the main event.

The actual motion experienced at a site during aftershock activity is dependent on the size of the individual events, their location relative to the site and the faulting mechanism of the individual events. It is possible for aftershocks to produce stronger motion at a specific site than is experienced in the main earthquake. For the purposes of this guideline, it is assumed that the probable maximum intensity value for aftershock-induced ground shaking at the building site is the same as that experienced in the original damaging earthquake, that the variability in this intensity is normally distributed and that it has a coefficient of variation of 50%. While these assumptions may not be accurate for any specific earthquake, and will be conservative for most earthquakes, they present a reasonable planning scenario for postearthquake building safety assessments.

With the above assumptions in place, together with an estimate of the intensity of motion that actually occurred at the site during the damaging earthquake, it is possible to construct a hazard curve indicating the annual probability of exceeding ground motion of defined intensity at the site. For the purposes of evaluations conducted in accordance with these Recommended Criteria, the hazard curve is plotted as a function of the spectral response acceleration, S_a, at the fundamental period of the damaged building, and the annual probability of exceedance for these accelerations. Figure 5-1 presents such a hazard curve, with spectral response acceleration normalized to the value actually thought to have been experienced in the first damaging earthquake. The primary parameters of importance from this hazard curve are the slope of the curve evaluated at S_a and the value of S_a itself.

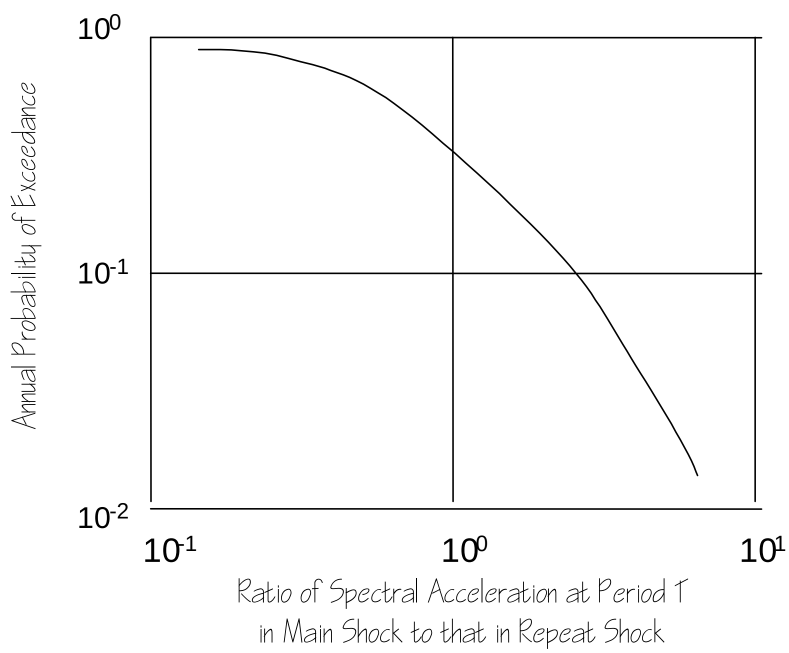

Figure 5-1 Presumed Postearthquake Hazard Curve

Using the S_a value estimated to have been experienced during the first damaging earthquake, a structural analysis is performed to determine the maximum interstory drift demand for the damaged structure under a repeat of that event, as well as the maximum axial forces on critical columns. These demands are factored by a demand variability factor γ to account for the variation associated with estimation of the character of the ground motion and its effect on structural response, and an analysis uncertainty factor γ_a to account for the uncertainty and bias inherent in the selected analytical approach.

The factored demand, $\gamma_a\gamma D$ calculated from the analysis represents a mean estimate of the probable maximum demand during the immediate postearthquake period, given the assumed distribution of ground shaking during this period, as represented by the assumed hazard curve.

These Recommended Criteria also specify median estimates of capacity for individual elements and the global structure. These capacities are dependent on frame and connection configuration. In addition to capacities, capacity reduction, or resistance, factors ϕ that adjust the estimated capacity of the structure to a mean value are also provided.

Once the factored demands and capacities are determined, a factored-demand-to capacity parameter λ is calculated from the equation:

$$\lambda = \frac{\gamma_a \gamma D}{\phi C} \qquad (5\text{-}1)$$

where D and C are respectively, the demand and capacity. The value of λ is then used directly to determine an associated confidence level for the desired performance, based on tabulated values related to the uncertainty inherent in the estimation of the building's demands and capacities. Values of λ less than 1.0 indicate greater than mean confidence of achieving the desired performance. Values greater than 1.0 indicate less than mean confidence.

5.7 Ground Motion Representation

The damaged structure should be analyzed for ground shaking demands representative of those that caused the initial damage. Ground shaking demands should be represented in the form of a 5% damped elastic response spectrum or with ground acceleration time-histories, compatible with this spectrum as required by the selected analytical procedure. Ground shaking demands may be determined by one of the following approaches.

5.7.1 Instrumental Recordings

When an actual recording of the ground shaking that caused the damage, obtained from the building site, or a nearby site with similar conditions is available, this may be used directly to perform analyses of the damaged structure. The ground acceleration time-history should be converted into a smoothed, 5% damped response spectrum, similar in form to the generalized response spectrum described in *FEMA-273*, and completely enveloping the actual response spectrum obtained for the acceleration record over the period range 0.5T to 2.0T, where T is the computed fundamental period of the damaged structure. If the selected analytical procedure is response history analysis, a suite of accelerograms constructed in accordance with the recommendations of *FEMA-273* and matched to the spectrum, should be used, one of which should be the actual site recording.

Commentary: The best possible estimate of ground shaking experienced at a site consists of actual ground motion recordings obtained from a free-field instrument located at the building site. Free field instruments are preferable to instruments located within the building or another structure as they will not be influenced by structural response effects.

Even in zones of high seismicity, few buildings have strong motion instrumentation, so it is highly unlikely that such records will be available for most buildings. Recordings of ground shaking obtained from other nearby sites may be used providing that the site of the instrument is at a comparable distance and azimuth to the fault rupture as the damaged building, and providing that site soil conditions are reasonably similar. Site soil conditions may be considered to be reasonably similar if they are of the same site class, as defined in FEMA-302, the NEHRP Recommended Provisions.

The intent of postearthquake analyses is not to evaluate the damaged building's response for the actual ground shaking that caused the original damage, but rather to evaluate this response for ground shaking likely to be experienced in the immediate postearthquake period. As previously discussed, this is assumed to be similar, though not identical to that which caused the original damage. For this reason, response spectra obtained from actual ground motion recordings are smoothed, to approximate a standard Newmark and Hall spectrum, as described in FEMA-273.

5.7.2 Estimated Ground Motion

When instrumental recordings of the damaging ground shaking, as described in Section 5.7.1 are not available, an estimated response spectrum for this ground shaking should be constructed. These spectra should be constructed as recommended by *FEMA 273* except that rather than using mapped values for the parameters S_S and S_1, these parameters should be calculated using standard attenuation relationships and appropriate estimates of the magnitude of the damage causing event, its distance from the building site, the site soil characteristics, faulting mechanism and other parameters required by the attenuation equation. Alternatively, these parameters may be estimated based on available recordings of ground shaking from the damage causing event.

Acceleration time histories, if required, should be constructed in accordance with the recommendations of *FEMA-273*.

5.8 Analytical Procedures

In order to evaluate the performance of a damaged steel moment-frame structure it is necessary to construct a mathematical model of the damaged structure that represents its strength and deformation characteristics and to conduct an analysis to predict the values of various design parameters when it is subjected to design ground motion. This section provides procedures for selecting an appropriate analysis procedure and for modeling. General requirements for the mathematical model are presented in Section 5.9.

Four alternative analytical procedures are available. The basic procedures are described in detail in *FEMA-273*. This section provides supplementary guidelines on the applicability of the *FEMA-273* procedures and also provides supplemental modeling recommendations. The four basic procedures are:

- Linear static procedure – an equivalent lateral force technique, similar, but not identical to that contained in the building code provisions
- Linear dynamic procedure – an elastic, modal response spectrum analysis
- Nonlinear static procedure – a simplified nonlinear analysis procedure in which the forces and deformations induced by monotonically increasing lateral loading is evaluated using a series of incremental elastic analyses of structural models that are sequentially degraded to represent the effects of structural nonlinearity.
- Nonlinear dynamic procedure – a nonlinear dynamic analysis procedure in which the response of a structure to a suite of ground motion time histories is determined through numerical integration of the equations of motion for the structure. Structural stiffness is altered during the analysis to conform to nonlinear hysteretic models of the structural components.

Commentary: The purpose of structural analyses performed as part of the postearthquake assessment process is to predict the values of key response parameters, that are indicative of the structure's performance, when it is subjected to ground motion. Once the values of these response parameters are predicted, the structure is evaluated for adequacy (appropriate level of confidence of achieving desired performance) using the basic approach outlined in Section 5.6.

Analyses conducted in these procedures take a markedly different approach than those used in the standard design process under the building code requirements. Rather than evaluating the forces and deformations induced in the structure under arbitrarily reduced loading levels, these analysis procedures attempt to predict, within probabilistically defined bounds, the actual values of the important response parameters under the design ground motion.

The ability of these procedures to estimate reliably the probable performance of the structure is dependent on the ability of the analysis to predict the values of these response parameters within acceptable levels of confidence. The linear dynamic procedure is able to provide relatively reliable estimates of the response parameters for structures that exhibit elastic, or near elastic behavior. The linear static procedure inherently has more uncertainty associated with its estimates of the response parameters because it less accurately accounts for the dynamic characteristics of the structure. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for structures that exhibit significant nonlinear behavior, particularly if they are irregular. However, it does not accurately account for the effects of higher mode response. If appropriate modeling is performed, the nonlinear dynamic approach is most capable of capturing the probable behavior of the real structure in response to ground motion; however, there are considerable uncertainties associated even with the values of the response parameters predicted by this technique.

5.8.1 Procedure Selection

Table 5-4 indicates the recommended analysis procedures for various conditions of structural regularity and dynamic properties. Note that structural regularity in these procedures is as determined in *FEMA-273*, rather than as alternatively defined in the building codes. Both regularity and dynamic properties shall be as determined for the building in the damaged state.

5.8.2 Linear Static Procedure (LSP)

5.8.2.1 Basis of the Procedure

Linear static procedure analysis of damaged steel moment-frame structures shall be conducted in accordance with the *FEMA-273 Guidelines*, except as specifically noted herein. In this procedure, lateral forces are applied to the masses of the structure, and deflections and component forces under this applied loading is determined. Calculated internal forces typically will exceed those that the building can develop, because anticipated inelastic response of components and elements is not directly recognized by the procedure. The predicted interstory drifts and column axial forces are evaluated using the procedures of Section 5.10.

Commentary: The linear static procedure is a method of estimating the response of the structure to earthquake ground shaking by representing the effects of this response through the application of a series of static lateral forces applied to an elastic mathematical model of the building's stiffness. The forces are applied to the structure in a pattern that represents the typical distribution of inertial forces in a regular structure responding in a linear manner to the ground shaking excitation, factored to account, in an approximate manner, for the probable inelastic behavior of the structure. It is assumed that the structure's response is dominated by the fundamental mode and that the lateral drifts induced in the elastic structural model by these forces represent a reasonable estimate of the actual deformation of the structure when responding inelastically.

Table 5-4 Selection Criteria for Analysis Procedure to Achieve Collapse Prevention

Structural Characteristics			**Analytical Procedure**			
Fundamental Period, T	**Regularity**	**Ratio of Column to Beam Strength**	**Linear Static**	**Linear Dynamic**	**Nonlinear Static**	**Nonlinear Dynamic**
$T \leq 3.5T_s$ [1]	Regular[2]	Strong Column[3]	Permitted	Permitted	Permitted	Permitted
		Weak Column[3]	Not Permitted	Not Permitted	Permitted	Permitted
	Irregular[2]	Any Conditions	Not Permitted	Not Permitted	Permitted	Permitted
$T > 3.5T_s$	Regular	Strong Column[3]	Not Permitted	Permitted	Not Permitted	Permitted
		Weak Column[3]	Not Permitted	Not Permitted	Not Permitted	Permitted
	Irregular[2]	Any Conditions	Not Permitted	Not Permitted	Not Permitted	Permitted

Notes:

1. T_s is the period at which the response spectrum transitions from a domain of constant response acceleration (the plateau of the response spectrum curve) to one of constant spectral velocity. Refer to *FEMA-273* or *FEMA-302* for more information.
2. Conditions of regularity are as defined in *FEMA-273*. These conditions are significantly different than those defined in *FEMA-302*.
3. 2.A structure qualifies as having a strong column condition if, at every floor level, the quantity $\sum M_{pr_c} / \sum M_{pr_b}$ is greater than 1.0, where ΣM_{prc} and ΣM_{prb} are the sum of the expected plastic moment strengths of the columns and beams, respectively, that participate in the moment-resisting framing in a given direction of structural response.

In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Earthquake demands for the LSP are represented by the static lateral forces whose sum is equal to the pseudo lateral load. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in displacement amplitudes approximating maximum displacements that are expected during the ground shaking under evaluation. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during this ground shaking. If the building responds inelastically to the earthquake ground shaking, as will commonly be the case, when ground shaking is severe, the internal forces that would develop in the *yielding building will be less than the internal forces calculated on an elastic basis.*

In addition to global structural drift, the collapse of steel moment-frame structures is closely related to inelastic deformation demands on the various elements that comprise the structure, such as plastic rotation demands on beam-column assemblies and tensile demands on column splices. Linear analysis methods do not permit direct evaluation of such demands. However, through a series of analytical evaluations of typical buildings for a number of earthquake records, it has been possible to develop statistical correlation between the interstory drift demands predicted by a linear analysis and the actual inelastic deformation demands determined by more accurate nonlinear methods. These correlation relationships are reasonably valid for regular structures, using the definitions of regularity contained in FEMA-273. Thus, the performance evaluation process using Linear Static Procedures (LSP) consists of performing the LSP analysis to determine an estimate of interstory drift demands, adjustment of these demands with the demand factors, g *and* g_a*, and comparison with tabulated interstory drift capacities.*

Although performance of steel moment-frame structures is closely related to interstory drift demand, there are some failure mechanisms, notably, failure of column splices, that are more closely related to strength demand. However, since inelastic structural behavior affects the strength demand on such elements, linear analysis is not capable of directly predicting these demands, except when the structural response is essentially elastic. Therefore, when LSP analysis is performed for structures that respond in an inelastic manner, column axial demands should be estimated using a supplementary plastic analysis approach.

Two basic assumptions apply in this evaluation approach. First, that the distribution of deformations predicted by an elastic analysis is similar to that which will occur in actual nonlinear response; second, that the ratio of computed strength demands from an elastic analysis to yield capacities is a relative indication of the inelastic ductility demand on the element. These assumptions are never particularly accurate but become quite inaccurate for structures that are highly irregular and experience large inelastic demands.

Most damaged structures will behave in a more non-linear manner than will undamaged structures, even when subjected to relatively low levels of ground shaking. Beam-column connections with fractures at the bottom flange of the beam, for example, will behave much like undamaged, fully restrained joints when loaded such that the fractured flange is in compression, and will behave much like pinned joints when loading produces tension at the bottom flange. Such behavior can not be accurately reflected in elastic analysis. In order to minimize the potential for analysis inaccuracies to result in overly optimistic estimates of the actual response of a damaged structure, these Recommended Criteria suggest *what are believed to be conservative modeling assumptions for damaged framing elements. However, the uncertainties inherent in the use of linear methods to model highly damaged structures are so large that it is recommended they not be used for this purpose.*

5.8.2.2 Modeling and Analysis Considerations

When damage results in a structure having different stiffness and strength for loading applied positively along one of the principal axes than it does for loading applied negatively, a separate model shall be developed and analysis performed for each direction of loading.

5.8.2.2.1 Period Determination

A fundamental period shall be calculated for each of two orthogonal directions of building response, using standard methods of modal analysis. The model used for this purpose should account for the damage sustained. Where damage results in a significantly different stiffness in the positive direction of response relative to the negative direction, separate analyses shall be performed for each such response direction.

Commentary: Modal analysis of a model of the building that includes representation of the structural damage is required to determine the building's period. This is because approximate formulae, used, for example, in FEMA-302 for this purpose, may be inaccurate for damaged structures.

5.8.2.3 Determination of Actions and Deformations

5.8.2.3.1 Pseudo Lateral Load

A pseudo lateral load, given by Equation 5-2, shall be independently calculated for each of two orthogonal directions of building response, and applied to a mathematical model of the building structure. Where damage results in a significantly different stiffness or strength in the positive direction of loading than in the negative direction, separate analyses shall be performed for each such response direction.

$$V = C_1 C_2 C_3 S_a W \tag{5-2}$$

where:

C_1 = modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 may be calculated using the procedure indicated in Section 3.3.3.3 in *FEMA 273* with the elastic base shear capacity substituted for V_y. Alternatively, C_1 may be taken as having a value of 1.0 where the fundamental period of response of the structure, T, is greater than T_s and shall be taken as having a value of 2.0 where the fundamental period of the structure is equal to or less than T_0. Linear interpolation shall be used to calculate C_1 for intermediate values of T.

T_0 = period at which the acceleration response spectrum for the site reaches its peak value, as indicated in *FEMA-302*. It may be taken as $0.2T_s$.

T_S = characteristic period of the response spectrum, defined as the period associated with the transition from the constant spectral response acceleration segment of the spectrum to the constant spectral response velocity segment of the spectrum as defined in *FEMA-302*.

C_2 = modification factor to represent the effect of hysteretic pinching on maximum displacement response. For steel moment-frame structures the value of C_2 shall be taken as 1.0.

C_3 = modification factor to represent increased dynamic displacements due to P-Δ effects and stiffness degradation. C_3 may be taken from Table 5-5 or alternatively, shall be calculated from the equation:

$$C_3 = 1 + \frac{5(q_i - 0.1)}{T} \geq 1.0 \qquad (5\text{-}3)$$

where:

q_i = the coefficient determined in accordance with Section 3.2.5.1 of FEMA-273.

S_a = response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.

W = total dead load and anticipated live load as indicated below:

- in storage and warehouse occupancies, a minimum of 25% of the floor live load,
- the actual partition weight or minimum weight of 10 psf of floor area, whichever is greater,
- the applicable snow load – see *FEMA-302*, and
- the total weight of permanent equipment and furnishings.

Table 5-5 Modification Factors C_3 for Linear Static Procedure

	C_3
Ductile fully-restrained connections	1.2
Brittle fully-restrained connections	1.4

Notes:

- Ductile connections are those connections capable of sustaining at least 0.03 radians, median, plastic rotation capacity without fracturing or sustaining significant loss of strength.
- Brittle connections are those connections not qualifying as ductile. Typical unreinforced moment-resisting connections in which beam flanges are CJP welded to the column, using low notch toughness weld filler metal shall be considered brittle unless laboratory data are available to substantiate their capability of behaving as indicated for ductile connections.

Commentary: The pseudo lateral force, when distributed over the height of the linearly-elastic analysis model of the structure, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation 5-2 may be significantly larger than the actual strength of the structure to resist this force. The acceptance evaluation procedures in Section 5.10 are developed to take this into account.

The values of the C_3 coefficient contained in Table 5-5 are conservative for most structures, and will generally result in calculation of an unduly low level of confidence. Use of Equation 5-3 to calculate C_3 is one way to improve calculated confidence without extensive additional effort, and is recommended.

5.8.2.3.2 Vertical Distribution of Seismic Forces

The lateral load F_x applied at any floor level x shall be determined as given in Section 3.3.1.3B of *FEMA-273*.

5.8.2.3.3 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level.

5.8.2.3.4 Determination of Interstory Drift

Interstory drifts shall be calculated using lateral loads in accordance with this section. Factored interstory drift demands, $g_a g d_i$, at each story "*i*", shall be determined by applying the appropriate demand variability factor g and analytical procedure uncertainty factor g_a obtained from Section 5.10.

5.8.2.3.5 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces by the applicable analysis uncertainty factor g_a and demand variability factor g obtained in Section 5.10.3. Column forces shall be calculated either as:

1. the axial demands from the unreduced linear analysis, or

2. the axial demands computed from the equation:

$$P'_c = \pm\left[2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L - 2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R\right] \tag{5-4}$$

where:

$\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L$ = the summation of the expected plastic moment strength (ZF_{ye}) divided by the span length, L, of all moment-connected beams framing into the left hand side of the column, above the level under consideration, and

$\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R$ = the summation of the expected plastic moment strength (ZF_{ye}) divided by the span length, L, of all moment-connected beams framing into the right hand side of the column, above the level under consideration.

When a column is part of framing that resists lateral forces under multiple directions of loading, the Seismic Demand shall be taken as the most severe condition resulting from application of 100% of the Seismic Demand computed for any one direction of response with 30% of the Seismic Demand computed for an orthogonal direction of response.

Commentary: When determining axial demands on columns using Equation 5-4, the value of the quantity $2\frac{M_{pe}}{L}$ *may be reduced for beams with fractured connections, when the direction of response of the structure is such that loading tends to open the fracture in tension. For such loading, the* M_{pe} *value at the fracture may be reduced to 30% of the nominal value calculated for the beam. Thus, if a beam has a fracture at one end, rather than using the value* $2\frac{M_{pe}}{L}$ *for the axial load contribution from this beam, the quantity* $1.3\frac{M_{pe}}{L}$ *could be used, when loading tends to place this fracture in tension. If a beam has fractures at* *both ends that open in tension simultaneously, the contribution for this beam could be reduced to* $0.6\frac{M_{pe}}{L}$

5.8.3 Linear Dynamic Procedure (LDP)

5.8.3.1 Basis of the Procedure

Linear dynamic procedure analysis of damaged steel moment-frame structures should generally be conducted in accordance with the *FEMA-273 Guidelines*, except as specifically noted herein. Coefficients C_1, C_2, and C_3 should be taken as indicated in Section 5.8.2.3.1 of these *Recommended Criteria*.

Estimates of interstory drift and column axial demands shall be evaluated using the applicable procedures of Section 5.10. Calculated displacements and column axial demands are factored by the applicable analytical uncertainty factor g_a and demand variability factor g, obtained from Section 5.10, and compared with factored capacity values. Calculated internal forces typically will exceed those that the building can sustain because of inelastic response of components and elements, but are generally not used to evaluate performance.

Commentary: The linear dynamic procedure is similar in approach to the linear static procedure, described in the previous section. However, because it directly accounts for the stiffness and mass distribution of the structure in calculating the dynamic response characteristics, it is somewhat more accurate. Coefficients C_1*,* C_2*, and* C_3*, which account in an approximate manner for the differences between elastic predictions of drift response and inelastic behavior are the same as for the linear static method. Under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, dynamic analysis. Note that although the LDP is more accurate than the LSP for analysis purposes, it can still be quite inaccurate when applied to heavily damaged structures and should be used with caution.*

The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the Linear Static Procedure (LSP). The main exception is that the response calculations are carried out using modal spectral analysis. Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that are approximately correct, but will produce internal forces that exceed those that would be obtained in a yielding building.

5.8.3.2 Modeling and Analysis Considerations

5.8.3.2.1 General

The Linear Dynamic Procedure (LDP) should conform to the criteria of this section. The analysis should be based on appropriate characterization of the ground motion, as described in Section 5.7. The LDP should conform to the criteria in Section 3.3.2.2 of *FEMA 273*. The requirement that all significant modes be included in the response analysis may be satisfied by including sufficient modes to capture at least 90% of the participating mass of the building in each of the building's principal horizontal directions. Modal damping ratios should reflect the damping inherent in the building at deformation levels less than the yield deformation. Except for buildings incorporating passive or active energy dissipation devices, or base isolation technology, effective damping should be taken as 5% of critical.

The interstory drift, and other response parameters calculated for each mode, and required for evaluation in accordance with Section 5.8.3.3, should be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root of sum of squares) rule or the CQC (complete quadratic combination) rule is acceptable.

Multidirectional excitation effects may be accounted for by combining 100% of the response due to loading in direction A with 30% of the response due to loading in direction B; and by combining 30% of the response in direction A with 100% of the response in direction B, where A and B are orthogonal directions of response for the building. Where damage to the structure results in unsymmetrical response in either the A or B directions, then independent analyses should be performed with elements modeled to represent the behavior of the structure when pushed in the positive and negative senses along either the A or B directions.

5.8.3.3 Determination of Actions and Deformations

5.8.3.3.1 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by mulitplying the results of the response spectrum analysis by the product of the modification factors, C_1, C_2, and C_3 defined in Section 5.8.2.3 and by the analytical procedure uncertainty factor g_a and demand variability factor g obtained from Section 5.10.

5.8.3.3.2 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces, as given in Section 5.8.2.3.5, by the applicable analysis uncertainty factor γ_a and demand variability factor γ obtained from Section 5.10.3.

5.8.4 Nonlinear Static Procedure (NSP)

5.8.4.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating the inelastic material and geometric response of the damaged structure is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the damaged building are modeled directly. The mathematical model of the building is subjected to a pattern of monotonically increased lateral forces or displacements until either a target displacement is exceeded or mathematical instability occurs. The target displacement is intended to approximate the total maximum displacement likely to be experienced by the actual structure, in response to the ground shaking anticipated during the immediate postearthquake period. The target displacement shall be calculated by the procedure presented in Section 5.8.4.3.1. Because the mathematical model accounts directly for effects of material and geometric inelastic response, the calculated internal forces will be reasonable approximations of those expected during the anticipated ground shaking, presuming that an appropriate pattern of loading has been applied.

Results of the Nonlinear Static Procedure (NSP) are to be evaluated using the applicable acceptance criteria of Section 5.10. Calculated interstory drifts and column and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

5.8.4.2 Modeling and Analysis Considerations

5.8.4.2.1 General

In the context of these procedures, the Nonlinear Static Procedure (NSP) involves the monotonic application of lateral forces, or displacements, to a nonlinear mathematical model of a building, until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, such as often occurs in damaged buildings, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging to the target displacement d_t, given by Equation 3-11 of *FEMA 273*. Postearthquake assessment shall be based on those column forces and interstory drifts corresponding to minimum horizontal displacement of the control node equal to the target displacement d_t.

Gravity loads should be applied to appropriate components and elements of the mathematical model during the NSP. The loads and load combinations should be as follows:

1. 100% of computed dead loads and permanent live loads should be applied to the model.

2. 25% of transient floor live loads should be applied to the model, except in warehouse and storage occupancies, where the percentage of live load used in the analysis should be based on a realistic assessment of the average long term loading.

The analysis model should be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

The modeling and analysis considerations set forth in Section 5.9 should apply to the NDP unless the alternative considerations presented below are applied.

> *Commentary: As with any nonlinear model, the ability of the analyst to detect the presence of inelastic behavior requires the use of a nonlinear finite element at the assumed location of yielding. The model will fail to detect inelastic behavior when appropriately distributed finite elements are not used. However, as an alternative to the use of nonlinear elements, it is possible to use linear elements and reconfigure the model, for example, by adjusting member restraints, as nonlinearity is predicted to occur. For example, when a member is predicted to develop a plastic hinge, a linear model can be revised to place a hinge at this location. When this approach is used, the internal forces and stresses that caused the hinging must be reapplied, as a nonvarying static load.*

5.8.4.2.2 Control Node

The NSP requires definition of the control node in a building. These procedures consider the control node to be the center of mass at the roof of the building. The top of a penthouse should not be considered as the roof. The displacement of the control node is compared with the target displacement—a displacement that characterizes the effects of earthquake shaking.

5.8.4.2.3 Lateral Load Patterns

Lateral loads should be applied to the building in profiles given in Section 3.3.3.2C of *FEMA 273*.

5.8.4.2.4 Period Determination

The effective fundamental period T_e in the direction under consideration should be calculated using the force-displacement relationship of the NSP as described in Section 3.3.3.2D of *FEMA 273*.

5.8.4.2.5 Analysis of Three-Dimensional Models

Static lateral forces should be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level.

Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is required by Section 3.2.7 in *FEMA 273*. Refer also to Section 5.8.4.3 of these *Recommended Criteria*..

5.8.4.2.6 Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2, or the orthogonal A and B directions) of the building should be developed for two-dimensional analysis. The effects of horizontal torsion should be considered as required by Section 3.2.2.2 of FEMA-273.

5.8.4.3 Determination of Actions and Deformations

5.8.4.3.1 Target Displacement

The target displacement d_t for buildings with rigid diaphragms at each floor level shall be estimated using the procedures of Section 3.3.3.3 of FEMA-273. Actions and deformations corresponding to the control node displacement equal to the target displacement shall be used for evaluation in accordance with Section 5.10.

5.8.4.3.2 Diaphragms

The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

5.8.4.3.3 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum interstory drift calculated at the target displacement by the analytical uncertainty factor g_a and demand variability factor g obtained from Section 5.10.2.

5.8.4.3.4 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces at the target displacement by the analytical uncertainty factor g_a and demand variability factor g from Section 5.10.3.

5.8.5 Nonlinear Dynamic Procedure (NDP)

5.8.5.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis.

The basis, the modeling approaches, and the acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using Response-History Analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground motion time-histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during ground shaking.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 5.10. Calculated displacements and internal forces are factored, and compared directly with factored acceptable values.

5.8.5.2 Modeling and Analysis Assumptions

5.8.5.2.1 General

The NDP should conform to the criteria of given in Section 3.3.4.2A of *FEMA-273*.

5.8.5.2.2 Ground Motion Characterization

The earthquake shaking should be characterized by suites of ground motion acceleration histories, prepared in accordance with the recommendations of Section 2.6.2 of *FEMA-273* and corresponding to the ground motion described in Section 5.7 of these *Recommended Criteria*. A minimum of three pairs of ground motion records should be used. Each pair should consist of two orthogonal components of ground motion records.

Consideration of multidirectional excitation effects required by Section 3.2.7 of *FEMA-273* may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along the horizontal axes of the building.

The effects of torsion should be considered according to Section 3.2.2.2 of *FEMA-273*.

5.8.5.3 Determination of Actions and Deformations

5.8.5.3.1 Response Quantities

Response quantities should be computed as follows:

1. If less than seven pairs of ground motion records are used to perform the analyses, each response quantity (for example, interstory drift demand, or column axial demand) should be taken as the maximum value obtained from any of the analyses.

2. If seven or more pairs of ground motion records are used to perform the analyses, the median value of each of the response quantities computed from the suite of analyses may be used as the demand. The median value shall be that value exceeded by 50% of the analyses in the suite.

5.8.5.3.2 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum of the interstory drifts calculated in accordance with Section 5.8.5.3.1 by the analytical uncertainty factor g_a and demand variability factor g obtained from Section 5.10.2.

5.8.5.3.3 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the column forces calculated in accordance with Section 5.8.5.3.1 by the applicable analytical uncertainty factor g_a and demand variability factor g obtained from Section 5.10.3.

5.9 Mathematical Modeling

5.9.1 Modeling Approach

In general, a damaged steel 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*.

Two-dimensional modeling, analysis, and evaluation of buildings with stiff or rigid diaphragms is acceptable if torsional effects are either sufficiently small to be ignored or indirectly captured.

Vertical lines of moment frames with flexible diaphragms may be individually modeled, analyzed, and evaluated as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements.

If linear or static analysis methods are used, it may be necessary to build separate models to simulate the behavior of the structure to ground shaking demands in the positive and negative response directions, to account for the differing effects of damage in each direction of response.

Commentary: An inherent assumption of linear seismic analysis is that the structure will exhibit the same stiffness and distribution of stresses regardless of whether loads are positively or negatively loaded. However, damage tends to create non-symmetrical conditions in structures. For example, fracture damage at the bottom flange of a beam will result in a substantial reduction in the connection's stiffness under one direction of loading, but will have negligible effect for the reverse direction of loading. In order to capture this behavior using linear analysis approaches, it is necessary to build two separate models, one in which the damage is effective and one in which the damage is not, to simulate the separate response in each direction of loading. A similar approach is required for nonlinear static analysis, in that the nonlinear behavior will be different, depending on the direction of loading. Only nonlinear dynamic analysis is capable of accurately simulating the effects of such damage with a single analytical model.

5.9.2 Model Configuration

The analytical model should include all frames capable of providing non-negligible stiffness for the structure, whether or not intended by the original design to participate in the structure's lateral force resistance. The model should accurately account for any damage sustained by the structure. Refer to Section 5.9.11 for procedures on modeling damaged connections.

Commentary: Gravity framing, in which beams are connected to columns with either clip angels or single clip plates can provide significant secondary stiffness *to a structure and should in general be modeled when performing postearthquake assessment analyses. The primary contributor to this added stiffness is the fact that the gravity load columns are constrained to bend to the same deflected shape as the columns of the moment-resisting frame, through their interconnection by the gravity beams which act as struts, and the diaphragms. As a secondary effect, the relatively small rigidity provided by the gravity connections provides some additional overall frame stiffness.*

5.9.3 Horizontal Torsion

The effects of actual horizontal torsion must be considered. In the building codes, the total torsional moment at a given floor level includes the following two torsional moments:

- the actual torsion, that is, the moment resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and
- the accidental torsion, that is, an accidental torsional moment produced by 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.

For the purposes of postearthquake evaluation, under these procedures, accidental torsion should not be considered. In buildings with diaphragms that are not flexible, the effect of actual torsion should be considered if the maximum lateral displacement d_{max} from this effect, at any point on any floor diaphragm, exceeds the average displacement d_{avg} by more than 10%.

Commentary: Accidental torsion is an artificial device used by the building codes to account for actual torsion that can occur, but is not apparent in an evaluation of the center of rigidity and center of mass in an elastic stiffness evaluation. Such torsion can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner in the structure. 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 frame on the east side of the structure yields significantly sooner than the framing on the west side, then inelastic torsion will develop. Rather than requiring that an accidental torsion be applied in the analysis, as do the building codes, these Recommended Criteria directly account for the uncertainty related to these torsional effects in the calculation of demand and resistance factors.

5.9.4 Foundation Modeling

In general, foundations may be modeled as unyielding. Assumptions with regard to 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. For purposes of determining building period and dynamic properties, soil-structure interaction may be modeled as permitted by the building code.

> *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.*

5.9.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic-force-resisting system. Development of the mathematical model should reflect the stiffness of the diaphragms. As a general rule, most floor slabs with concrete fill over metal deck may be considered to be rigid diaphragms and 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.

5.9.6 *P-D* Effects

P-D effects, caused by gravity loads acting on the displaced configuration of the structure, may be critical in the seismic performance of steel moment-frame structures, particularly for damaged structures that may have significant permanent lateral offset as part of the damage.

The structure should 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:

$$y_i = \frac{P_i d_i}{V_{yi} h_i} \qquad (5\text{–}5)$$

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*,

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 the distance between the centerline of floor framing at each of the levels above and below, the distance between the top of floor slabs at each of the levels above and below, or similar common points of reference, and

d_i = lateral drift in story *i*, including any permanent drift, from the analysis in the direction under consideration, at its center of rigidity, using the same units as for measuring h_i.

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. 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 procedures of Appendix A.

For nonlinear 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.

Commentary: The values of interstory drift capacity, provided in Section 5.10, and the corresponding resistance factors, were computed considering P-D effects (FEMA-355F). For a given structure, it is believed that if the value of y is less than 0.3 the effects of P-D have been adequately considered by these general procedures. For values of y greater than this limit the statistics on frame interstory drift capacities contained in Section 5.10 are inappropriate. For such frames explicit determination of interstory drift capacities, considering P-D effects using the detailed performance evaluation procedures outlined in Appendix A is required.

The plastic story shear quantity, V_{yi}, should be determined by methods of plastic analysis. In a story in which(1) all beam-column connections meet the strong column –weak beam criterion,(2) the same number of moment resisting bays is present at the top and bottom of the frame, and (3) the strength of moment-connected girders at the top and bottom of the frame is similar, V_{yi} may be approximately calculated from the equation:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pG\,j}}{h_i} \tag{5-6}$$

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. For girders with damaged connections, the quantity $2M_{pGi}$ should be taken as the sum of the plastic moment capacities at each end of the girder, accounting for the effect of damage on connection capacity as recommended in Section 5.9.11.*

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

In any story in which none of the columns meet the strong-column -weak-beam criterion, the plastic story shear quantity V_{yi} may be calculated from the equation:

$$V_{yi} = \frac{2\sum_{k=1}^{n} M_{pC_k}}{h_i} \tag{5-7}$$

where:

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.

5.9.7 Elastic Framing Properties

The complete axial area of rolled shapes should be used. For built-up sections, the effective area should be reduced if adequate load transfer mechanisms are not available. For elements fully encased in concrete, the axial stiffness may be calculated assuming full composite action if most of the concrete may be expected to remain after additional ground shaking. Composite action may not be assumed for strength unless adequate load transfer and ductility of the concrete can be assured.

The shear area of the elements should be based on standard engineering procedures. The comments above regarding built-up section, concrete encased elements, and composite floor beam and slab, apply.

The calculation of rotational stiffness of steel beams and columns in bare steel frames should follow standard engineering procedures. For components encased in concrete, the stiffness shall include composite action, but the width of the composite section should be taken as equal to the width of the flanges of the steel member and should not include parts of the adjoining floor slab, unless there is an adequate and identifiable shear transfer mechanism between the concrete and the steel.

5.9.8 Nonlinear Framing Properties

The elastic component properties, should be computed as outlined in Section 5.9.7. Appropriate nonlinear moment-curvature and interaction relationships should be used for beams and beam-columns to represent the effects of plastification.

5.9.9 Verification of Analysis Assumptions

Each component should be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length. Further, each component should be evaluated by rational analysis for adequate postearthquake residual gravity load capacity, considering reduction of stiffness caused by earthquake damage to the structure.

Where moments in horizontally-spanning primary components, due to the gravity loads, exceed 50% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than components ends should be specifically investigated by comparing flexural actions with expected component strengths. Modeling should account for formation of flexural plastic hinges away from component ends when this is likely to occur.

5.9.10 Undamaged Connection Modeling

Undamaged connections should be modeled in accordance with the following procedures.

5.9.10.1 Fully Restrained Connections

Framing connected with typical welded fully restrained moment-resisting connections, such as shown in Figure 5-2, should be modeled as indicated herein.

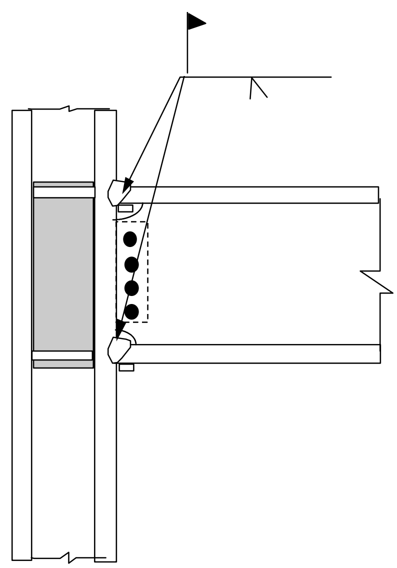

Figure 5-2 Welded Unreinforced Fully Restrained Connection (pre-1994)

5.9.10.1.1 Linear Modeling

Undamaged fully-restrained connections should be modeled using the gross cross section properties and assuming rigid attachment between the beams and columns. Modeling may use either center-line-to-center-line dimensions for beams and columns, or alternatively, rigid or flexible column panel zones may be modeled to offset the ends of the beams and columns from the intersection of the center lines of these members. Rigid offsets, used to represent the panel zone, should not exceed 80% of the dimension of the actual panel zone. Panel zone flexibility may be directly considered by adding a panel zone element to the model.

5.9.10.1.2 Nonlinear Modeling

Prior to developing a mathematical model for nonlinear analysis of beam-column assemblies with welded unreinforced fully restrained moment-resisting connections, an analysis should be conducted to determine the controlling yield mechanism for the assembly. This may consist of flexural yielding of the beam at the face of the column, flexural yielding of the column at the top and/or bottom of the panel zone; shear yielding of the panel zone itself, or a combination of these mechanisms. Elements capable of simulating the nonlinear behaviors indicated in these analyses should be implemented in the model. Regardless of whether or not panel zones are anticipated to yield, panel zones should be explicitly modeled. If calculations indicate that panel zones are unlikely to yield in shear, panel zones may be modeled as rigid links. If significant yielding is indicated to occur, a suitable element that models this behavior should be used. Expected yield strengths F_{ye} should be used for all nonlinear elements to indicate the expected onset of nonlinear behavior. Flexural strain hardening of beams and columns should be taken as 5% of the elastic stiffness, unless specific data indicates a more appropriate value. Panel zones may be assumed to strain harden at 20% of their elastic stiffness.

5.9.10.2 Simple Shear Tab Connections

This section presents modeling guidelines for the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design. Figure 5-3 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. A concrete floor slab, or slab on metal deck may be present at the top flange of the beam.

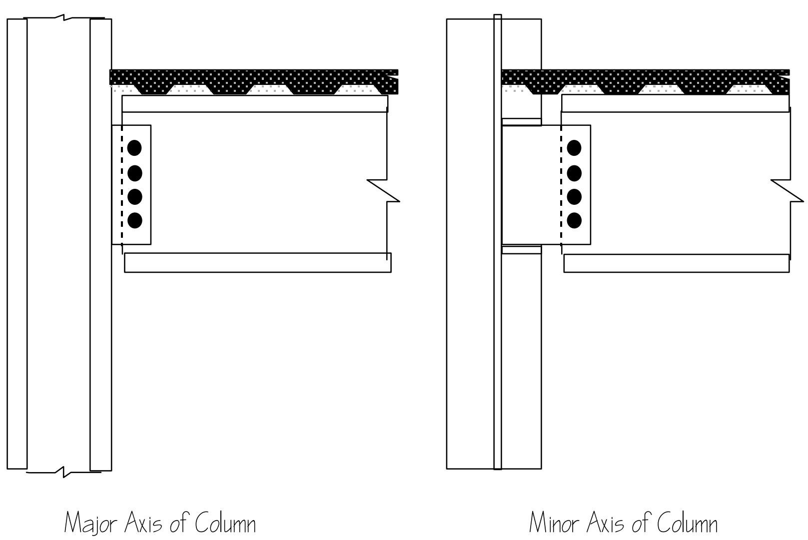

Figure 5-3 Typical Simple Shear Tab Connection with Slab

5.9.10.2.1 Modeling Guidelines - Linear Analysis

The connection stiffness should be explicitly modeled as a rotational spring that connects the beam to the column. The spring stiffness, K_q should be taken as:

$$K_q = 28000\left(d_{bg} - 5.6\right) \tag{5-8}$$

where d_{bg} is the depth of the bolt group, measured center-line-to-center-line of the outermost bolts, in inches and K_q is in units of k-inches per radian. In lieu of explicit modeling of the connection, beams that frame into columns with simple shear tab connections may be modeled with an equivalent rigidity, EI_{eq} taken as:

$$EI_{eq} = \frac{1}{\dfrac{6h}{l_b^2 K_q} + \dfrac{1}{EI_b}} \tag{5-9}$$

where:

E = the modulus of elasticity, kip/square inch
h = the average story height of the columns above and below the beam, inches
I_b = the moment of inertia of the beam, (inches)^4
l_b = the beam span center to center of columns, inches

Commentary: The presence of gravity framing, utilizing shear tab connectors, can provide substantial stiffening to the steel moment-frame system provided as the basic lateral force resisting system. The primary contributor to this added stiffness is the fact that the gravity load columns are constrained to bend to the same deflected shape as the columns of the moment-resisting frame, through their interconnection by the gravity beams, which act as struts, and the diaphragms. The flexural restraint on the columns represented by the spring stiffness given by Equations 5-8 and 5-9 is a secondary effect but can provide stability for frames at large displacements.

5.9.10.2.2 Modeling Guidelines - Nonlinear Analysis

The connection should be explicitly modeled as an elastic-perfectly-plastic rotational spring. The elastic stiffness of the spring should be taken as given by Equation 5-8. The plastic strength of the spring should be determined as the expected plastic moment capacity of the bolt group, calculated as the sum of the expected yield strength of the bolts and their distance from the neutral axis of the bolt group.

5.9.11 Damage Modeling

This section presents procedures for modeling various conditions of damage. In general, damage results in anisotropic frame behavior with affected framing exhibiting different hysteretic properties for loading in a positive direction, than it does for loading in the reverse direction. Except for nonlinear dynamic analyses, it is generally necessary to utilize multiple models to represent these different behaviors, with loading applied in an appropriate direction for each model.

5.9.11.1 Fully Restrained (FR) Connection Damage

Damaged type FR connections should be modeled in accordance with the guidelines of this section. Refer to Chapter 2 for detailed descriptions of the various damage conditions.

- Connections with any one of type G3, G4, G7, C2, C4, C5, W2, W3, W4, P5, or P6 damage at the bottom flange only or the top flange only may be modeled as undamaged for loading conditions in which lateral loading will tend to place the fractured surfaces into compression. For loading conditions in which the fracture is placed into tension, the connection should be modeled as an undamaged simple shear tab connection, per Section 5.9.10.2.
- Connections with any combination of type G3, G4, G7, C2, C4, C5, W2, W3, W4, P5, or P6 damage at the top and bottom flanges should be modeled as an undamaged simple shear tab connection, per Section 5.9.10.2 for loading in either direction.
- If any of the above conditions is present in combination with shear tab damage, types S1, S2, S3, S4, S5, or S6, then the connection should be modeled as a simple pin connection for both directions of loading.
- Connections with type P7 damage should be modeled as follows. The beam and column above the diagonal plane formed by the fracture should be assumed to be rigidly restrained to each other. The beam and column below the diagonal plane formed by the fracture should similarly be assumed to be rigidly restrained to each other. The two assemblies consisting of the rigidly restrained beam-column joint above and below the diagonal fracture should be assumed to be unconnected for loading that places the fracture into tension and should be assumed to be connected to each other with a “pin” for conditions of loading that place the fracture into compression.
- Connections with type P9 damage and oriented as indicated in Figure 5-4 should be modeled with the beams and columns below the fracture surface assumed to be rigidly connected. The column above the fracture surface should be assumed to be unconnected for loading that places the column into tension and should be assumed to be “pin” connected for loading that places the column into compression. If the orientation of type P9 damage is opposite that shown in Figure 5-4, then the instructions above for “top” and “bottom” columns should be reversed.

5.9.11.2 Column Damage

- If a column has type C1 or C3 damage in any flange, the column should be modeled as if having a pinned connection (unrestrained for rotation) at that location for loading conditions that induce tension across the fracture. The column may be modeled as undamaged for loading conditions that produce compression across the fracture surfaces.

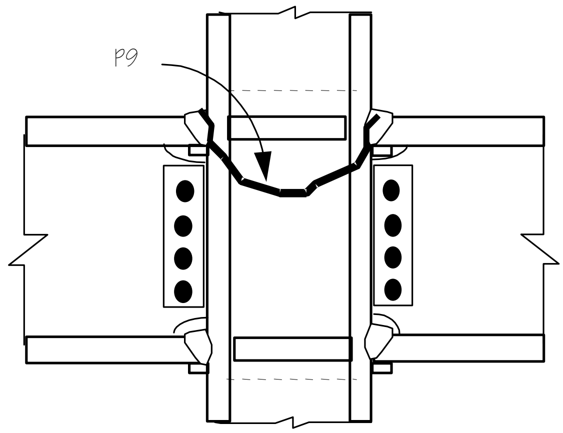

Figure 5-4 Type P9 Panel Zone Damage

- If a column has type C7, column splice fracture damage, it should be assumed to be unconnected across the splice for load conditions that place the column in tension and should be assumed to have a "pin" connection for load conditions that place the column in compression.
- If a column has type C6, buckling damage of a flange, the buckled length of the column should be modeled with a separate element with flexural properties calculated using only 30% of the section of the buckled element.

5.9.11.3 Beam Damage

- Beams that have lateral torsional buckling, type G8, should be modeled with a flexural pin at the center of the buckled region.
- Beams that have type G1, buckling damage of a flange should be modeled with the buckled length of the beam represented by a separate element with flexural properties calculated using only 30% of the section of the buckled flange.

5.9.11.4 Other Damage

Damage other than indicated in Sections 5.9.11.1, 5.9.11.2, or 5.9.11.3 need not be modeled unless in the judgment of the engineer, it results in significant alteration of the stiffness or load distribution at the connection. In such cases, the engineer should use judgment in developing the model such that it accurately reflects the behavior of the damaged elements.

5.10 Acceptance Criteria and Confidence Evaluation

A level of confidence with regard to the building's ability to provide Collapse Prevention performance for a repeat of the original damaging ground motion should be determined. Each of the parameters in Table 5-6 must be independently evaluated, using the procedures of Section

5.10.1 and the parameters and acceptance criteria of Sections 5.10.2, 5.10.3, and 5.10.4. The controlling parameter is that which results in the calculation of the lowest confidence for building performance.

Table 5-6 Performance Parameters Requiring Evaluation of Confidence

Parameter	Discussion
Interstory drift	The maximum interstory drift computed for any story of the structure shall be evaluated for global and local behaviors. Refer to Section 5.10.2
Column axial load	The adequacy of each column to withstand the calculated maximum compressive demand for that column shall be evaluated. Refer to Section 5.10.3
Column splice tension	The adequacy of column splices to withstand calculated maximum tensile demands for the column shall be evaluated. Refer to Section 5.10.4

5.10.1 Factored-Demand-to-Capacity Ratio

Confidence level is determined by first evaluating the factored-demand-to-capacity ratio l given by the equation:

$$l = \frac{g_a g D}{f C} \tag{5-10}$$

where:

C = capacity of the structure, as indicated in Sections 5.10.2, 5.10.3, and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

D = calculated demand for the structure, obtained from the structural analysis,

g = a demand variability factor that accounts for the variability inherent in the prediction of demand related to assumptions made in structural modeling and prediction of the character of ground shaking as indicated in Sections 5.10.2, 5.10.3, and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

g_a = an analytical uncertainty factor that accounts for bias and uncertainty inherent in the specific analytical procedure used to estimate demand as a function of ground shaking intensity as indicated in Section 5.10.2, 5.10.3 and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

f = a resistance factor that accounts for the uncertainty and variability inherent in the prediction of structural capacity as a function of ground shaking intensity, as indicated in Section 5.10.2, 5.10.3 and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively, and

l = a confidence index parameter from which a level of confidence can be obtained. See Table 5-7.

Factored-demand-to-capacity ratio l shall be calculated using Equation 5-10 for each of the performance parameters indicated in Table 5-6, which also references the appropriate section of this document where the various parameters, g_a, g, and f required to perform this evaluation may be found. These referenced Sections 5.10.2, 5.10.3, and 5.10.4 also define an uncertainty parameter b_{UT} associated with the evaluation of global and local interstory drift capacity, column compressive capacity, and column splice tensile capacity, respectively. These uncertainties are related to the building's configuration, the structural framing system (OMF or SMF), the type of analytical procedure employed, and the performance level being evaluated. Table 5-7 indicates the level of confidence associated with various values of the factored-demand-to-capacity ratio l calculated using Equation 5-10, for various values of the uncertainty parameter b_{UT}. Linear interpolation between the values given in Table 5-7 may be used for intermediate values of factored-demand-to-capacity ratio l and uncertainty b_{UT}.

Table 5-7 Factored-Demand-to-Capacity Ratios l and Uncertainty b_{UT}, for Specific Confidence Levels

Uncertainty Parameter b_{UT}	**Factored-Demand-to-Capacity Ratios l**										
0.2	1.43	1.31	1.23	1.16	1.11	1.05	0.99	0.93	0.86	0.79	0.70
0.3	1.84	1.62	1.47	1.35	1.25	1.16	1.07	0.97	0.85	0.76	0.63
0.4	2.49	2.10	1.84	1.65	1.49	1.35	1.21	1.06	0.89	0.77	0.59
0.5	3.54	2.86	2.44	2.12	1.87	1.65	1.43	1.22	0.99	0.82	0.59
0.6	5.30	4.10	3.38	2.86	2.46	2.12	1.79	1.48	1.14	0.91	0.62
Confidence Level	**10**	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**95**	**99**

Commentary: In order to predict structural performance, these procedures rely on the application of structural analysis and laboratory test data to predict the behavior of real structures. However, there are a number of sources of uncertainty inherent in the application of analysis and test data to performance prediction. For example, the actual strength of structural materials, the quality of individual welded joints, and the amount of viscous damping present is never precisely known, but can have impact on both the actual amount of demand produced on the structure and its elements, and on the capacity of the elements to *resist these demands. If the actual values of all parameters that affect structural performance were known, it would be possible to predict accurately both demand and capacity. However, this is never the case. In these procedures, confidence is used as a measure of the extent to which predicted behavior is likely to represent reality.*

The extent of confidence inherent in a performance prediction is related to the possible variation in the several factors that affect structural demand and capacity, such as stiffness, damping, connection quality, and the analytical procedures employed. In this project, evaluations were made of the potential distribution of each of these factors and the effect of variation in these factors on structural demand and capacity. Each of these sources of uncertainty in structural demand and capacity prediction were characterized as part of the supporting research for this project, by a coefficient of variation, b_U*. The coefficient* b_{UT} *is the total coefficient of variation, considering all sources of uncertainty. It is used, together with other factors to calculate the demand and resistance factors. It is assumed that demand and capacity are lognormally distributed relative to these uncertainty parameters. This allows confidence to be calculated as a function of the number of standard deviations that the factored-demand-to-capacity ratio,* l*, lies above or below a mean value. Table 5-7 provides a solution for this calculation, using a value of 5.0 for the hazard parameter, k, that is representative of the assumed regional seismicity during the year following a major earthquake. Further information on this method may be found in Appendix A.*

5.10.2 Performance Limited by Interstory Drift Angle

5.10.2.1 Factored Interstory Drift Angle Demand

Factored interstory drift demand should be computed as the quantity, gg_aD, where the demand D is the largest interstory drift in any story, computed from structural analysis, g_a is the coefficient obtained from Table 5-8, and γ is the coefficient obtained from Table 5-9.

Commentary: Several structural response parameters are used to evaluate structural performance. The primary parameter used for this purpose is interstory drift. Interstory drift is an excellent parameter for judging the ability of a structure to resist P-D instability and collapse. It is also closely related to plastic rotation, or drift angle, demand on individual beam-column connection assemblies, and therefore a good predictor of the performance of beams, columns and connections. For tall slender structures, a significant portion of interstory drift is a result of axial elongation (and shortening) of the columns. Although modeling of the structure should account for this frame flexibility, that portion of interstory drift resulting from axial column deformation in stories below the story under consideration should be neglected in determining local connection performance. Unfortunately, this portion of the interstory drift must be *determined manually as most computer programs do not separately calculate this quantity.*

Table 5-8 Interstory Drift Angle Analysis Demand Uncertainty Factors, g_a

Analysis Procedure / System Characteristic	LSP	LDP	NSP	NDP
Type 1 Connections				
Low Rise (<4 stories)	0.73	0.86	0.91	1.06
Mid Rise (4-12 stories)	1.05	1.32	1.02	1.19
High Rise (> 12 stories)	1.37	1.24	1.02	1.17
Type 2 Connections				
Low Rise (<4 stories)	1.03	1.40	1.35	1.06
Mid Rise (4-12 stories)	1.25	1.70	1.46	1.11
High Rise (> 12 stories)	0.96	1.51	1.71	1.17

Table 5-9 Interstory Drift Angle Demand Variability Factors, g, Type 1 and Type 2 Connections

Building Height	g
Type 1 Connections[1]	
Low Rise (< 4 stories)	1.6
Mid Rise (4 stories – 12 stories)	1.4
High Rise (>12 stories)	2.0
Type 2 Connections[2]	
Low Rise (< 4 stories)	1.7
Mid Rise (4 stories – 12 stories)	2.0
High Rise (>12 stories)	2.6

Notes:

1- Type 1 connections are capable of resisting median total drift angle demands of 0.04 radians without fracture or strength degradation.

2- Type 2 connections are capable of resisting median total drift angle demands of 0.01 radians without fracture or strength degradation. Generally, welded unreinforced connections, employing weld metal with low notch toughness, typical of older steel moment-frame buildings should be considered to be of this type.

5.10.2.2 Factored Interstory Drift Angle Capacity

Interstory drift capacity may be limited either by the global response of the structure, or by the local behavior of beam-column connections. Section 5.10.2.2.1 provides values for global interstory drift capacity for regular, well-configured structures. Global interstory drift capacities for irregular structures must be determined using the detailed procedures of Appendix A. Section 5.10.2.2.2 provides procedures for evaluating local interstory drift angle capacity, as limited by connection behavior.

5.10.2.2.1 Global Interstory Drift Angle

Factored interstory drift capacity, fC, as limited by global response of the building, shall be based on the product of the resistance factor f and capacity C, which are obtained from Table 5-10, for connections with either Type 1 or Type 2 connections . Type 1 connections are capable of resisting median total interstory drift angle demands of 0.04 radians without fracturing or strength degradation. Type 2 connections are capable of resisting median total interstory drift angle demands of 0.01 radian without fracturing or strength degradation. Welded unreinforced moment-resisting connections with weld metal with low notch toughness should be considered Type 2. Table 5-11 provides values of the uncertainty coefficient b_{UT} to be used with global interstory drift evaluation.

Table 5-10 Global Interstory Drift Angle Capacity and Resistance Factors

Structure Type	Interstory Drift Capacity	Resistance factor f
Type 1 Connections		
Low Rise (< 4 stories)	0.10	0.85
Mid Rise (4 stories – 12 stories)	0.10	0.75
High Rise (>12 stories)	0.085	0.60
Type 2 Connections		
Low Rise (< 4 stories)	0.10	0.75
Mid Rise (4 stories – 12 stories)	0.079	0.60
High Rise (>12 stories)	0.057	0.60

Table 5-11 Uncertainty Coefficient b_{UT} for Global Interstory Drift Evaluation

Building Height	Connection Type	
	Type 1	**Type 2**
Low Rise (3 stories or less)	0.30	0.35
Mid Rise (4 – 12 stories)	0.40	0.45
High Rise (> 12 stories)	0.50	0.55

Notes: 1- Value of b_{UT} should be increased by 0.05 for the linear static procedure.
2- Value of b_{UT} may be reduced by 0.05 for the nonlinear dynamic procedure.

5.10.2.2.2 Local Interstory Drift Angle

Factored interstory drift angle capacity, fC, limited by local connection response, shall be based on the capacity of the connection, C, and resistance factor, f, obtained from Table 5-12, for the connection types present in the building. Table 5-13 provides values of the uncertainty coefficient b_{UT} to be used with local interstory drift evaluation

Table 5-12 Local Interstory Drift Angle Capacity and Resistance Factors

Connection Type	Interstory Drift Capacity	Resistance factor f
Pre-Northridge connection with low notch toughness weld metal	$0.053\text{-}0.0006d_b$	0.7
Pre-Northridge connection with notch tough weld metal (Note 1)	$0.060\text{-}0.0006d_b$	0.85
Shear tab connections	$0.16\text{-}0.0036d_b$	0.7
Post-Northridge connection intended for steel moment-frame Service (Note 2)	0.04	0.85

Notes:

1. Weld metal with a notch toughness 40 ft –lbs at anticipated service temperature
2. Many types of connections approved for steel moment-frame service in the post-Northridge period are capable of better performance than this. Refer to *FEMA-350*, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* for more detailed data.

Table 5-13 Uncertainty Coefficient b_{UT} for Local Interstory Drift Evaluation

Building Height	Connection Type	
	Type 1	Type 2
Low Rise (3 stories or less)	0.30	0.35
Mid Rise (4 – 12 stories)	0.35	0.40
High Rise (> 12 stories)	0.40	0.40

Notes: 1- Value of b_{UT} should be increased by 0.05 for linear static analyses.
2- Value of b_{UT} may be reduced by 0.05 for nonlinear dynamic analyses.

5.10.3 Performance Limited by Column Compressive Capacity

5.10.3.1 Column Compressive Demand

Factored column compressive demand shall be determined for each column as the quantity gg_aD, where:

D = the compressive axial load on the column determined as the sum of Dead Load, 25% of unreduced Live Load, and Seismic Demand. Seismic Demand shall be determined by either of the following four analysis methods:

Linear: The axial demands may be taken as those predicted by a linear static or linear dynamic analysis, conducted in accordance with Section 5.8.2 or 5.8.3 of these *Recommended Criteria*.

Plastic: The axial demands may be taken based on plastic analysis, as indicated by Equation 5-4 of Section 5.8.2.3.5 of these *Recommended Criteria*.

Nonlinear Static: The axial demands may be taken based on the computed forces from a nonlinear static analysis, at the target displacement, in accordance with Section 5.8.4 of these *Recommended Criteria*.

Nonlinear Dynamic: The axial demands may be taken based on the computed design forces from a nonlinear dynamic analysis, in accordance with Section 5.8.5 of these *Recommended Criteria*.

g_a = Analytical demand uncertainty factor, taken from Table 5-14.

g = Demand variability factor, taken as having a value of 1.1.

The uncertainty coefficient b_{UT} shall be taken as indicated in Table 5-14 based on the procedure used to calculate column compressive demand D.

Table 5-14 Analysis Uncertainty Factor g_a and Total Uncertainty Coefficient b_{UT} for Evaluation of Column Compressive Demands

Analytical Procedure	Analysis Uncertainty Factor g_a	Total Uncertainty Coefficient b_{UT}
Linear static or dynamic analysis	1.15	0.35
Plastic analysis (Section 4.4.3.3.6)	1.0	0.15
Nonlinear static analysis	1.05	0.20
Nonlinear dynamic analysis	$e^{1.4b^2}$	$\sqrt{0.0225+b^2}$

Note: β may be taken as the coefficient of variation of the axial load values determined from the suite of nonlinear analyses.

Commentary: The value of g has been computed assuming a coefficient of variation for axial load values resulting from material strength variation and uncertainty in dead and live loads of 15%. The values of g_a have been calculated assuming coefficients of variation of 30%, 0% and 15% related to uncertainty in the analysis procedures for linear, plastic and nonlinear static analyses, respectively. In reality, for structures that are stressed into the inelastic range, elastic analysis will typically overestimate axial column demands, in which case, a value of 1.0 could be used. However, for structures that are not loaded into the inelastic range, the indicated value is appropriate. Plastic analysis will also typically result in an upper bound estimate of column demand, and application of additional demand factors is not appropriate. For nonlinear dynamic analysis, using a suite of ground motions, direct calculation of the analysis demand factor is possible, using the equation shown. All of these demand factors are based on a hazard parameter k, having a value of 5.0, representative of the assumed seismicity for the immediate postearthquake period.

5.10.3.2 Column Compressive Capacity

Factored compressive capacity of each individual column to resist compressive axial loads shall be determined as the product of the resistance factor, f, and the nominal axial strength of the column, C, which shall be determined in accordance with the *AISC Load and Resistance Factor Design Specification*. Specifically, for the purposes of this evaluation, the effective length coefficient k shall be taken as having a value of 1.0 and the resistance factor f shall be assigned a value of 0.90.

5.10.4 Column Splice Capacity

The capacity of column tensile splices, other than splices consisting of complete joint penetration (CJP) butt welds of all elements of the column (flanges and webs) shall be evaluated in accordance with this section. Column splices consisting of CJP welds of all elements of the column, and in which the weld filler metal has a minimum notch toughness of 40 ft-lbs at the lowest anticipated service temperature, need not be evaluated.

5.10.4.1 Column Splice Tensile Demand

Factored column splice tensile demand shall be determined for each column as the quantity gg_aD, where D is the column splice tensile demand. Column splice tensile demand shall be determined as the computed Seismic Demand in the column, less 90% of the computed Dead Load demand. Seismic Demand shall be as determined for column compressive demand, in accordance with Section 5.10.3.1. The demand variability factor g shall be taken as having a value of 1.05 and the analysis uncertainty factor g_a shall be taken as indicated in Table 5-14. The total uncertainty coefficient b_{UT} shall also be taken as indicated in Table 5-14.

5.10.4.2 Column Splice Tensile Capacity

The capacity of individual column splices to resist tensile axial loads shall be determined as the product of the resistance factor, f, and the nominal tensile strength of the splice, C, as determined in accordance with the *AISC Load and Resistance Factor Design Specification*. Specifically, Chapter J shall be used to calculate the nominal tensile strength of the splice connection. For the purposes of this evaluation, f shall be assigned a value of 0.85.

5.11 Evaluation Report

Regardless of the level of evaluation performed, the responsible structural engineer should prepare a written evaluation report and submit it to the owner upon completion of the evaluation. When the building official has required evaluation of a steel moment-frame building, this report should also be submitted to the building official. This report should directly, or by attached references, document the inspection program that was performed, and provide an interpretation of the results of the inspection program and a general recommendation as to appropriate repair and occupancy strategies. The report should include but not be limited to the following material:

- Building address
- A narrative description of the building, indicating plan dimensions, number of stories, total square feet, occupancy, and the type and location of lateral-force-resisting elements. Include a description of the grade of steel specified for beams and columns and, if known, the type of welding (e.g., Shielded Metal Arc Welding, or Flux-Cored Arc Welding) present. Indicate if moment connections are provided with continuity plates. The narrative description should be supplemented with sketches (plans and evaluations) as necessary to provide a clear understanding of pertinent details of the building's construction. The description should include an indication of any structural irregularities as defined in the Building Code.

- A description of nonstructural damage observed in the building.
- An estimate of the ground shaking intensity experienced by the building, determined in accordance with Section 5.7.
- A description of the inspection and evaluation procedures used, including the signed inspection forms for each individual inspected connection.
- A description, including engineering sketches, of the observed damage to the structure as a whole (e.g., permanent drift) as well as at each connection, keyed to the damage types in Chapter 5; photographs should be included for all connections with significant visible damage.
- Calculations demonstrating the determination of a confidence level with regard to the building's ability to resist collapse in the immediate postearthquake period.
- A summary of the recommended actions (repair and modification measures and occupancy restrictions).

The report should include identification of any potentially hazardous conditions that were observed, including corrosion, deterioration, earthquake damage, pre-existing rejectable conditions, and evidence of poor workmanship or deviations from the approved drawings. In addition, the report should include an assessment of the potential impacts of observed conditions on future structural performance. The report should include the Field Inspection Reports of damaged connections (visual inspection and nondestructive testing records, data sheets, and reports), as an attachment, and should bear the seal of the structural engineer in charge of the evaluation.

The nature and scope of the evaluations performed should be clearly stated in the structural engineer's written evaluation report. If the scope of evaluation does not permit an informed judgment to be made as to the extent with which the building complies with the applicable building codes, or as to a statistical level of confidence that the damage has not exceeded an acceptable damage threshold, this should be stated .