A. Detailed Procedures for Performance Evaluation

A.1 Scope

This appendix provides detailed procedures for evaluating the performance capability of steel moment-frame buildings. These detailed procedures are provided as a supplement to the simplified performance evaluation procedures in Chapter 3. They may be used to demonstrate enhanced levels of confidence with regard to the ability of a particular building to meet desired performance objectives, relative to the confidence levels that may be derived using the more simplified procedures, and they must be used instead of the procedures of Chapter 3 for irregular structures and for structures with connections that have not been prequalified. This appendix also provides criteria for performance evaluation for deterministically defined hazards.

Commentary: Chapter 3 provides procedures for a simplified method of performance evaluation, using factored-demand-to-capacity ratios to determine a level of confidence with regard to a building's ability to provide a desired performance objective. The tabular values of demand and resistance factors and confidence indices contained in Chapter 3 were derived using the procedures presented in this appendix, applied to the performance evaluation of a suite of regularly configured model buildings. Since this suite of model buildings is not completely representative of any individual structure, the use of the tabular values inherently entails some uncertainty, and thus reduced levels of confidence, with regard to performance prediction. The detailed procedures in this appendix permit reduction in these uncertainties, and therefore enhanced confidence, with regard to prediction of building performance. These more detailed procedures must be used for those irregular building configurations not well represented by the model buildings used as the basis for the values contained in Chapter 3.

A.2 Performance Evaluation Approach

A.2.1 Performance Objectives and Confidence

As defined in Section 3.2 of these *Recommended Criteria*, performance is defined in terms of probabilistic performance objectives. A performance objective consists of the specification of a performance level and an acceptable low probability that poorer performance could occur within a specific period of time, typically taken as 50 years. Alternatively, deterministic performance objectives can also be evaluated. Deterministic performance objectives consist of the specification of a performance level and a specific earthquake, that is, fault location and magnitude, for which this performance is to be attained.

Two performance levels are defined: the Immediate Occupancy performance level and the Collapse Prevention performance level. Detailed descriptions of these performance levels may be found in Chapter 3. The evaluation procedures contained in this appendix permit estimation of a level of confidence associated with achievement of a performance objective. For example, a design may be determined to provide a 95% level of confidence that there is less than a 2% probability in 50 years of more severe damage than represented by the Collapse Prevention level.

For another example, a design may be determined to provide a 50% level of confidence that the structure will provide Immediate Occupancy performance, or a better performance, for a Richter magnitude 6 earthquake along a defined fault.

Commentary: The probability that a building may experience damage more severe than that defined for a given performance level is a function of two principal factors. The first of these is the structure's vulnerability, that is, the probability that it will experience certain levels of damage given that it experiences ground motion of certain intensity. The second of these factors is the site hazard, that is, the probability that ground shaking of varying intensities may occur in a given time period. The probability that damage exceeding a given performance level may occur in a period of time is calculated as the integral over a year's time of the probability that damage will exceed that permitted within a performance level. Mathematically, this may be expressed as:

$$P(D > PL) = \int P_{D>PL}(x)h(x)dx \qquad \text{(A-1)}$$

where:

P(D>PL) = *Probability of damage exceeding a performance level in a period of t years*

$P_{D>PL}(x)$ = *Probability of damage exceeding a performance level given that the ground motion intensity is level x, as a function of x,*

h(x)dx = *probability of experiencing a ground motion intensity of level (x) to (x + dx) in a period of t years*

Vulnerability may be thought of as the capacity of the structure to resist greater damage than that defining a performance level. Structural response parameters that may be used to measure capacity include the structure's ability to undergo global building drift, maximum tolerable member forces, and maximum tolerable inelastic deformations. Ground accelerations associated with the seismic hazard, and the resulting enforced global building drift, member forces and inelastic deformations produced by the hazard may be thought of as demands. If both the demand that a structure will experience over a period of time and the structure's capacity to resist this demand could be perfectly defined, then performance objectives, the probability that damage may exceed a performance level within a period of time, could be ascertained with 100% confidence. However, the process of predicting the capacity of a structure to resist ground shaking demands as well as the process of predicting the severity of demands that will actually be experienced entail significant uncertainties. Confidence level is a measure of the extent of uncertainty inherent in this process. A level of 100% confidence may be described as perfect confidence. In reality, it is never possible to attain such *confidence. Confidence levels on the order of 90 or 95% are considered high, while confidence levels less than 50% are considered low.*

Generally, uncertainty can be reduced, and confidence increased, by obtaining better knowledge or using better procedures. For example, enhanced understanding and reduced uncertainty with regard to the prediction of the effects of ground shaking on a structure can be obtained by using a more accurate analytical procedure to predict the structure's response. Enhanced understanding of the capacity of a structure to resist ground shaking demands can be obtained by obtaining specific laboratory data on the physical properties of the materials of construction and on the damageability of individual beam-column connection assemblies.

The simplified performance evaluation procedures of Chapter 3 are based on the typical characteristics of standard buildings. Consequently, they incorporate significant uncertainty in the performance prediction process. As a result of this significant uncertainty, it is anticipated that the actual ability of a structure to achieve a given performance objective may be significantly better than would be indicated by those simple procedures. The more detailed procedures of this appendix may be used to improve the definition of the actual uncertainties incorporated in the prediction of performance for a specific structure and thereby to obtain better confidence with regard to the prediction of performance for an individual structure.

As an example, using the simplified procedures of Chapter 3, it may be found that for a specific structure, there is only a 50% level of confidence that there is less than a 10% chance in 50 years of poorer performance than the Collapse Prevention level. This rather low level of confidence may be more a function of the uncertainty inherent in the simplified procedures than the actual inadequate capacity of the building to provide Collapse Prevention performance. In such a case, it may be possible to use the procedures contained in this appendix to reduce the uncertainty inherent in the performance estimation and find that instead, there may be as much as a 95% level of confidence in obtaining such performance.

In both the procedures of this appendix and Chapter 3, the uncertainties associated with estimation of the intensity of ground motion have been neglected. These uncertainties can be quite high, on the order of those associated with structural performance or even higher. Thus, the confidence estimated using these procedures is really a confidence with regard to structural performance, given the seismicity as portrayed by the USGS hazard maps that accompany FEMA-273 and FEMA-302.

A.2.2 Basic Procedure

As indicated in Chapter 3, a demand and resistance factor design (DRFD) format is used to associate a level of confidence with the probability that a building will have less than a specified probability of exceedance of a desired performance level. The basic approach is to determine a confidence parameter, l, which may then be used, with reference to Table A-1, to determine the confidence level that exists with regard to performance estimation. The confidence parameter, l, is determined from the factored-demand-to-capacity equation:

$$l = \frac{g g_a D}{f C} \tag{A-2}$$

where:

C = median estimate of the capacity of the structure. This estimate may be obtained either by reference to default values contained in Chapters 3 and 6, or by more rigorous direct calculation of capacity using the procedures of this appendix,

D = calculated demand on the structure, obtained from a 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,

g_a = an analysis uncertainty factor that accounts for the bias and uncertainty associated with the specific analytical procedure used to estimate structural demand as a function of ground shaking intensity,

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,

l = a confidence index parameter from which a level of confidence can be obtained by reference to Table A-1.

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 demand, or drift angle demand, on individual beam-column connection assemblies, and therefore a good predictor of the performance of beams, columns and connections. Other parameters used in these guidelines include column axial compression and column axial tension. In order to determine a level of confidence with regard to the probability that a building has less than a specified probability of exceeding a performance level over a period of time, the following steps are followed:

1. **The performance objective to be evaluated is selected**. This requires selection of a performance level of interest, for example, Collapse Prevention or Immediate Occupancy,

and a desired probability that damage in a period of time will be worse than this performance level. Representative performance objectives may include:

- 2% probability of poorer performance than Collapse Prevention level in 50 years
- 50% probability of poorer performance than Immediate Occupancy level in 50 years.

It is also possible to express performance objectives in a deterministic manner, where attainment of the performance is conditioned on the occurrence of a specific magnitude earthquake on an identified fault.

2. **Characteristic motion for the performance objective is determined.** For probabilistic performance objectives, an average estimate of the ground shaking intensity at the probability of exceedance identified in the performance objective definition (step 1) is determined. For example, if the performance objective is a 2% probability of poorer performance than the Collapse Prevention level in 50 years, then an average estimate of ground shaking demands with a 2% probability of exceedance in 50 years would be determined. Ground shaking intensity is characterized by the parameter S_{aT1}, the 5% damped spectral response acceleration at the site for the fundamental period of response of the structure. *FEMA-273* provides procedures for determining this parameter for any probability of exceedance in a 50-year period.

   For deterministic performance objectives, an average estimate of the ground motion at the building site for the specific earthquake magnitude and fault location must be made. As with probabilistic estimates, the motion is characterized by S_{aT1}.

3. **Structural demands for the characteristic earthquake ground motion are determined.** A mathematical structural model is developed to represent the building structure. This model is then subjected to a structural analysis, using any of the methods contained in Chapter 3. This analysis provides estimates of maximum interstory drift demand, maximum column compressive demand, and maximum column-splice tensile demand, for the ground motion determined in step 2.

4. **Median estimates of structural capacity are determined**. Median estimates of the interstory drift capacity of the moment-resisting connections and the building frame as a whole are determined, as are median estimates of column compressive capacity and column-splice tensile capacity. Interstory drift capacity for the building frame, as a whole, may be estimated using the default values of Chapter 3 for regular structures, or alternatively, the detailed procedures of Section A.6 may be used. These detailed procedures are mandatory for irregular structures. Interstory drift capacity for moment-resisting connections that are prequalified in Chapters 3 and 6 of these *Recommended Criteria* may be estimated using the default values of Chapters 3 and 6, or alternatively, direct laboratory data on beam-column connection assembly performance capability and the procedures of Section A.5 of this appendix may be used. Median estimates of column compressive capacity and column-splice tensile capacity are made using the procedures of Chapter 3.

5. **A factored-demand-to-capacity ratio, l is determined**. For each of the performance parameters, i.e., interstory drift as related to global building frame performance, interstory drift as related to connection performance, column compression, and column splice tension,

Equation A-2 is independently applied to determine the value of the confidence parameter l. In each case, the calculated estimates of demand *D* and capacity *C* are determined using steps 3 and 4, respectively. If the procedures of Chapter 3 are used to determine either demand or median capacity estimates, than the corresponding values of the demand factors g and resistance factors f should also be determined in accordance with the procedures of Chapter 3. If the procedures of this appendix are used to determine median demand, or capacity, then the corresponding demand and resistance factors should be determined in accordance with the applicable procedures of this appendix.

6. **Evaluate confidence**. The confidence obtained with regard to the ability of the structure to meet the performance objective should be the lowest value determined using the values of l determined in accordance with step 5 above, back-calculated from the equation:

$$l = e^{-b b_{UT}\left(K_X - k b_{UT}/2\right)} \tag{A-3}$$

where:

b = a coefficient relating the incremental change in demand (drift, force, or deformation) to an incremental change in ground shaking intensity, at the hazard level of interest, typically taken as having a value of 1.0,

b_{UT} = an uncertainty measure equal to the vector sum of the logarithmic standard deviation of the variations in demand and capacity resulting from uncertainty,

k = the slope of the hazard curve, in ln-ln coordinates, at the hazard level of interest, i.e., the ratio of incremental change in S_{aT1} to incremental change in annual probability of exceedance (refer to Section A.3.2),

K_X = standard Gaussian variate associated with probability *x* of not being exceeded as a function of number of standard deviations above or below the mean found in standard probability tables.

Table A-1 provides a solution for this equation, for various values of the parameters, k, l, and b_{UT}.

The values of the parameter b_{UT} used in Equation A-3 and Table A-1 are used to account for the uncertainties inherent in the estimation of demands and capacities. Uncertainty enters the process through a variety of assumptions that are made in the performance evaluation process, including, for example, assumed values of damping, structural period, properties used in structural modeling, and strengths of materials. Assuming that the amount of uncertainty introduced by each of the assumptions can be characterized, the parameter b_{UT} can be calculated using the equation:

$$b_{UT} = \sqrt{\sum_i b_{ui}^2} \tag{A-4}$$

where: b_{ui} are the standard deviations of the natural logarithms of the variation in demand or capacity resulting from each of these various sources of uncertainty. Sections A.4, A.5 and A.6 indicate how to determine b_{ui} values associated with demand estimation, beam-column connection assembly behavior, and building global stability capacity prediction, respectively.

Table A-1 Confidence Parameter λ, as a Function of Confidence Level, Hazard Parameter *k*, and Uncertainty β_{UT}

Confidence Level	2%	5%	10%	20%	30%	40%	50%	60%	70%	80%	90%	95%	99%
$\beta_{UT} = 0.1$													
k=1	1.24	1.19	1.14	1.09	1.06	1.03	1.0	0.98	0.95	0.92	0.88	0.85	0.80
k=2	1.24	1.19	1.15	1.10	1.07	1.04	1.01	0.99	0.96	0.93	0.90	0.85	0.80
k=3	1.25	1.20	1.15	1.11	1.07	1.04	1.02	0.99	0.96	0.93	0.90	0.86	0.80
k=4	1.26	1.20	1.16	1.11	1.08	1.05	1.02	1.0	0.97	0.94	0.90	0.87	0.81
$\beta_{UT} = 0.2$													
k=1	1.55	1.41	1.32	1.21	1.13	1.07	1.02	0.97	0.92	0.86	0.79	0.73	0.64
k=2	1.58	1.45	1.34	1.23	1.16	1.09	1.04	0.99	0.94	0.88	0.81	0.75	0.66
k=3	1.61	1.48	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
k=4	1.64	1.51	1.40	1.28	1.20	1.14	1.08	1.03	0.97	0.91	0.84	0.78	0.68
$\beta_{UT} = 0.3$													
k=1	1.95	1.71	1.54	1.35	1.23	1.13	1.05	0.97	0.89	0.81	0.71	0.64	0.52
k=2	2.04	1.79	1.61	1.41	1.28	1.18	1.09	1.01	0.93	0.85	0.75	0.66	0.55
k=3	2.14	1.88	1.68	1.48	1.34	1.23	1.13	1.06	0.98	0.89	0.78	0.70	0.57
k=4	2.23	1.96	1.76	1.54	1.40	1.29	1.20	1.11	1.02	0.93	0.82	0.73	0.60
$\beta_{UT} = 0.4$													
k=1	2.49	2.10	1.81	1.52	1.33	1.20	1.08	0.98	0.88	0.77	0.65	0.56	0.43
k=2	2.70	2.27	1.96	1.65	1.45	1.30	1.17	1.06	0.95	0.84	0.70	0.61	0.47
k=3	2.92	2.46	2.12	1.78	1.57	1.40	1.27	1.15	1.03	0.90	0.76	0.66	0.50
k=4	3.16	2.66	2.30	1.93	1.70	1.52	1.38	1.25	1.11	0.98	0.83	0.71	0.55
$\beta_{UT} = 0.5$													
k=1	3.21	2.59	2.15	1.73	1.48	1.28	1.13	1.0	0.87	0.74	0.60	0.50	0.36
k=2	3.63	2.93	2.44	1.96	1.67	1.45	1.28	1.13	0.99	0.84	0.68	0.56	0.40
k=3	4.11	3.32	2.76	2.22	1.90	1.65	1.45	1.28	1.12	0.95	0.77	0.64	0.46
k=4	4.66	3.76	3.13	2.52	2.14	1.87	1.65	1.45	1.26	1.08	0.87	0.72	0.52
$\beta_{UT} = 0.6$													
k=1	4.17	3.22	2.58	1.99	1.65	1.39	1.20	1.03	0.87	0.72	0.56	0.44	0.30
k=2	5.00	3.86	3.09	2.39	1.97	1.67	1.43	1.23	1.04	0.86	0.66	0.53	0.36
k=3	5.98	4.62	3.70	2.86	2.35	2.00	1.72	1.48	1.25	1.03	0.80	0.64	0.43
k=4	7.15	5.52	4.42	3.42	2.82	2.39	2.05	1.76	1.49	1.23	0.95	0.76	0.52

A.3 Determination of Hazard Parameters

Two basic hazard parameters are required by these performance evaluation procedures. The first of these, S_{aT1}, is the median, 5%-damped, linear spectral response acceleration, at the fundamental period of the building, at the desired hazard level (probability of exceedance in a 50-year period or specific earthquake magnitude and fault). Section A.3.1 provides guidelines for obtaining this parameter. The second parameter is the slope *k* of the hazard curve in logarithmic space, also evaluated at the desired hazard level. Section A.3.2 provides guidelines for obtaining this parameter.

A.3.1 Spectral Response Acceleration

Probabilistic, 5%-damped, linear spectral response acceleration, S_{aT1} at the fundamental period of the building, at the desired hazard level (probability of exceedance in a 50-year period), may be determined in several different ways. These include:

a. Site-specific seismological and geotechnical investigation. *FEMA-273* provides guidelines for this method.

b. Use of national hazard maps developed by the United States Geologic Survey. *FEMA-273* also provides guidelines for the use of these maps for this purpose.

Deterministic 5%-damped, linear spectral response acceleration S_{aT1} at the fundamental period of the building, shall be determined based on site-specific seismological and geologic study.

The spectral response acceleration S_{aT1} is used as a reference point, through which a response spectrum is plotted. This response spectrum may be used directly in the structural analysis, or alternatively, may be used as a basis for the development of ground motion accelerograms used in the structural analysis. Refer to Chapter 3 for guidelines on analysis.

A.3.2 Logarithmic Hazard Curve Slope

In these procedures, the logarithmic slope *k* of the hazard curve at the desired hazard level is used to determine the resistance factors, demand factors and also the confidence levels. The hazard curve is a plot of probability of exceedance of a spectral amplitude versus that spectral amplitude, for a given period, and is usually plotted on a log-log scale. In functional form it can be represented by the equation:

$$H_{Si}(S_i) = k_0 S_i^{-k} \tag{A-5}$$

where:

$H_{Si}(S_i)$ = the probability of ground shaking having a spectral response acceleration greater than S_i,
k_0 = a constant, dependent on the seismicity of the individual site,
k = the logarithmic slope of the hazard curve.

The slope of the hazard curve is a function of the hazard level, location and response period. USGS maps provide values of 5%-damped, spectral response accelerations at periods of 0.2 seconds, termed S_s, and 1 second, termed S_1, for ground motions having 2% and 10% probabilities of exceedance in 50 years, for all locations in the U.S. This information is also available on their web site and on a CD-ROM. Since most steel moment-frames have relatively long fundamental periods, the slope of the hazard curve may be determined for most such structures using the S_1 values published by the USGS for probabilities of exceedance of 2% and 10% in 50 years, and substitution of these values into the following equation:

$$k = \frac{\ln\left(\frac{H_{S1(10/50)}}{H_{S1(2/50)}}\right)}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} = \frac{1.65}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} \qquad \text{(A-6)}$$

where:

$S_{1(10/50)}$ = spectral amplitude for 10/50 hazard level
$S_{1(2/50)}$ = spectral amplitude for 2/50 hazard level
$H_{S_{1(10/50)}}$ = probability of exceedance for 10% in 50 years = 1/475 = 0.0021
$H_{S_{1(2/50)}}$ = probability of exceedance for 2% in 50 years = 1/2475 = 0.00040

The accompanying sidebar provides an example of how k may be determined using this procedure, for a representative site. As an alternative to using this detailed procedure, an approximate value of k may be obtained from Table A-2. When deterministic ground shaking demands (specific magnitude earthquake on a fault) are used as the basis for a performance objective, the value of k shall be taken as 4.0, regardless of the site seismicity.

Table A-2 Default Values of the Logarithmic Hazard Curve Slope *k* for Probabilisitc Ground Shaking Hazards

Region	*k*
Alaska, California and the Pacific Northwest	3
Intermountain Region, Basin & Range Tectonic Province	2
Other U.S. locations	1

Note: For deterministic ground shaking demands, use a value of k = 4.0

Example determination of the parameter, *k*, the logarithmic slope of the hazard curve using hazard data from the USGS.

Example site location: Los Angeles City Hall
Referencing USGS maps, web site, find $S_{1(10/50)}$ = 0.45g, $S_{1(2/50)}$ = 0.77g
Substituting into equation A-5, find:

$$k = \frac{1.65}{\ln\left(\frac{0.77g}{0.45g}\right)} = \frac{1.65}{0.537} = 3.07$$

A.4 Determination of Demand Factors

The demand variability factor g and analysis uncertainty factor g_a are used to adjust the calculated interstory drift, column axial load and column-splice tension demands to their mean values, considering the variability and uncertainty inherent in drift demand prediction.

Variability in drift demand prediction is primarily a result of the fact that due to relatively subtle differences in acceleration records, a structure will respond somewhat differently to different ground motion records, even if they are well characterized by the same response spectrum. Since it is not possible to predict the exact acceleration record that a structure may experience, it is necessary to account for the probable variation in demand produced by all possible different records. This is accomplished by developing a nonlinear mathematical model of the structure, and running nonlinear response history analyses of the structure for a suite of ground motion records, all of which are scaled to match the 5% damped linear spectral response acceleration, S_{aT1}, described in Section A.3.1. From these analyses, statistics are developed for the median value and standard deviation of the natural logarithm of the various demand parameters including maximum interstory drift, column axial load, and column splice tension. These standard deviations of the natural logarithms of these response parameters are denoted b_{D_R}.

Once the value of b_{D_R} has been determined, the demand variability factor, γ, is calculated from the equation:

$$g = e^{\frac{k}{2b} b_{D_R}^2} \tag{A-7}$$

where:

k is the logarithmic slope of the hazard curve, taken in accordance with Section A.3.1

b is a coefficient that represents the amount that demand increases as a function of hazard, and may normally be taken as having a value of 1.0

Uncertainty in the prediction of demands is due to an inability to define accurately the value of such parameters as the yield strength of the material, the viscous damping of the structure, the effect of nonstructural components, the effect of foundation flexibility on overall structural response, and similar modeling issues. Although it is not feasibly practical to do so, it is theoretically possible to measure each of these quantities for a building and to model their effects exactly. Since it is not practical to do this, instead we use likely values for each of these effects in the model, and account for the possible inaccuracies introduced by using these likely values, rather than real values. These inaccuracies are accounted for by developing a series of models to represent the structure, accounting for the likely distribution of these various parameters. Each of these models is used to run analyses with a single ground motion record, and statistics are developed for the effect of variation in these parameters on predicted demands. As with the variability due to ground motion, the standard deviation of the natural logarithms of the response parameters are calculated, and denoted by b_{D_U}. This parameter is used to calculate the analytical uncertainty factor, g_a.

In addition to uncertainty in demand prediction, the analytical uncertainty factor g_a also accounts for inherent bias, that is, systematic under- or over-prediction of demand, inherent in an analytical methodology. Bias is determined by using the analytical methodology, for example, elastic modal analysis, to predict demand for a suite of ground motions and then evaluating the ratio of the demand predicted by nonlinear time history analysis of the structure to that predicted by the methodology for the same ground motion. This may be represented mathematically as:

$$C_B = \frac{\textit{demand predicted by nonlinear time history analysis}}{\textit{demand predicted by analysis method}} \tag{A-8}$$

where C_B is the bias factor. The bias factor that is applicable to a specific structure is taken as the median value of C_B calculated from a suite of ground motions. The variation in the bias factors obtained from this suite of ground motions is used as one of the components in the calculation of b_{D_U}.

Once the median bias factor, C_B and logarithmic standard deviation in demand prediction b_{D_U} have been determined, the analysis uncertainty factor, g_a is calculated from the equation:

$$g_a = C_B e^{\frac{k}{2b} b^2_{D_U}} \tag{A-9}$$

The analysis uncertainty factors presented in Chapter 3 were calculated using this approach as applied to a suite of typical buildings. In addition to the uncertainties calculated using this procedure, it was assumed that even the most sophisticated methods of nonlinear time history analysis entail some uncertainty relative to the actual behavior of a real structure. Additional uncertainty was associated with other analysis methods to account for effects of structural irregularity, which were not adequately represented in the suite of model buildings used in the study. The value of the total logarithmic uncertainty b_{D_U} used as a basis for the analysis uncertainty factors presented in Chapter 3 are summarized in Table A-3. The bias factors C_B used in Chapter 3 are summarized in Table A-4. It is recommended that these default values for

C_B and b_{D_U} be used for all buildings. If it is desired to calculate building-specific b_{D_U} values, it is recommended that these values not be taken as less than those indicated in Table A-3 for nonlinear dynamic analysis, for the applicable building characteristics.

Table A-3 Default Logarithmic Uncertainty b_{DU} for Various Analysis Methods

	Analysis Procedure							
	Linear Static		Linear Dynamic		Nonlinear Static		Nonlinear Dynamic	
Performance Level	IO	CP	IO	CP	IO	CP	IO	CP
Type 1 Connections								
Low Rise (<4 stories)	0.17	0.22	0.15	0.16	0.14	0.17	0.10	0.15
Mid Rise (4 – 12 stories)	0.18	0.29	0.15	0.23	0.15	0.23	0.13	0.20
High Rise (> 12 stories)	0.31	0.25	0.19	0.29	0.17	0.27	0.17	0.25
Type 2 Connections								
Low Rise (<4 stories)	0.19	0.23	0.16	0.25	0.18	0.18	0.10	0.15
Mid Rise (4 – 12 stories)	0.20	0.30	0.17	0.33	0.14	0.21	0.13	0.20
High Rise (> 12 stories)	0.21	0.36	0.21	0.31	0.18	0.33	0.17	0.25

Table A-4 Default Bias Factors C_B

	Analysis Procedure							
	Linear Static		Linear Dynamic		Nonlinear Static		Nonlinear Dynamic	
Performance Level	IO	CP	IO	CP	IO	CP	IO	CP
Type 1 Connections								
Low Rise (<4 stories)	0.90	0.65	1.00	0.80	1.10	0.85	1.00	1.00
Mid Rise (4 – 12 stories)	1.10	0.85	1.10	1.15	1.40	0.95	1.00	1.00
High Rise (> 12 stories)	1.05	1.0	1.15	1.0	1.30	0.85	1.00	1.00
Type 2 Connections								
Low Rise (<4 stories)	0.75	0.90	1.00	1.20	0.90	1.25	1.00	1.00
Mid Rise (4 – 12 stories)	0.80	1.00	1.05	1.30	1.08	1.35	1.00	1.00
High Rise (> 12 stories)	0.75	0.70	1.30	1.20	1.30	1.30	1.00	1.00

Commentary: Although it may be possible, for certain structures, to increase the confidence associated with a prediction of probable earthquake demands on the structure, through calculation of structure-specific analysis uncertainty factors, in general this is a very laborious process. It is recommended that the default values of b_{DU} and C_B, contained in Tables A-3 and A-4, be used for most *structures. However, the procedures of this section can be used to adjust the analysis uncertainty and demand variability factors for the site seismicity k.*

A.5 Determination of Beam-Column Connection Assembly Capacities

The probable behavior of beam-column connection assemblies at various demand levels can best be determined by full-scale laboratory testing. Such testing can provide indications of the probable physical behavior of such assemblies in buildings. Depending on the characteristics of the assembly being tested, meaningful behaviors may include the following: onset of local buckling of flanges; initiation of fractures in welds, base metal or bolts; a drop in the moment developed by the connection beyond predetermined levels; or complete failure, at which point the connection is no longer able to maintain attachment between the beam and column under the influence of gravity loads. If sufficient laboratory data are available, it should be possible to obtain statistics, including a median value and standard deviation, on the demand levels at which these various behaviors occur.

In the past, most laboratories used plastic rotation as the demand parameter by which beam-column connection assembly behavior was judged. However, since plastic deformation may occur at a number of locations within a connection assembly, including within the beam itself, within the connection elements, and within the column panel zone or column, many laboratories have measured and reported plastic rotation angles from testing in an inconsistent manner. Therefore, in these *Recommended Criteria*, total interstory drift angle, as indicated in Section 3.6, is the preferred demand parameter for reporting laboratory data. This parameter is less subject to erroneous interpretation by testing laboratories and also has the advantage that it is a quantity directly predicted by linear structural analyses.

Median drift angle capacities, C, and resistance factors, f, for various prequalified connection types are presented in Chapters 3 and 6. These values were determined from cyclic tests of full-size connection assemblies using the testing protocols indicated in Section 6.9. The cyclic tests are used to determine the load-deformation hysteresis behavior of the system and the connection drift angle at which the following behaviors occur:

1. onset of local flange buckling of beams,
2. degradation of moment-resisting capacity of the assembly to a value below the nominal moment-resisting capacity,
3. initiation of fracture of bolts, welds, or base metal that results in significant strength degradation of the assembly, and
4. complete failure of the connection, characterized by an inability of the connection to maintain its integrity under gravity loading.

Based on this data, drift angle statistics, including a median value and logarithmic standard deviation are obtained for the Immediate Occupancy and Collapse Prevention damage states, as indicated in Table A-5. The quantity q_U, the ultimate capacity of the connection, is used to evaluate the acceptability of connection behavior for the Collapse Prevention performance level as limited by local behavior.

Table A-5 Behavior States for Performance Evaluation of Connection Assemblies

Symbol	Performance Level	Description
q_{IO}	Immediate Occupancy	The lowest drift angle at which any of behaviors 1, 2, or 3, occur (see Section A.5, above)
q_{U}	Ultimate	The drift angle at which behavior 4 occurs
q_{SD}	Strength Degradation	The lowest drift angle at which any of behaviors 2, 3, or 4 occur

A.5.1 Connection Test Protocols

Two connection test protocols have been developed under this project. The standard protocol is intended to represent the energy input and cyclic deformation characteristics experienced by connection assemblies in steel moment frames which are subjected to strong ground shaking from large magnitude earthquakes, but which are not located within a few kilometers of the fault rupture. This protocol presented in Section 6.9 is similar to that contained in *ATC-24* and consists of ramped cyclic loading, starting with initial cycles of low energy input within the elastic range of behavior of the assembly, and progressing to increasing deformation of the beam tip until assembly failure occurs. However, unlike *ATC-24*, the protocol incorporates fewer cycles of large-displacement testing to balance more closely the energy input to the assembly, with that likely experienced by framing in a real building. The second protocol is intended to represent the demands experienced by connection assemblies in typical steel moment-frame buildings responding to near-fault ground motion, dominated by large velocity pulses. This protocol (Krawinkler, 2000) consists of an initial single large displacement, representing the initial response of a structure to a velocity pulse, followed by repeated cycles of lesser displacement.

Performance characteristics of connection assemblies, for use in performance evaluation of buildings, should be selected based on the characteristics of earthquakes dominating the hazard for the building site, at the specific hazard level. Most buildings are not located on sites that are likely to be subjected to ground shaking with near-field pulse characteristics. Connection performance data for such buildings should be based on the standard protocols. Buildings on sites that are close to a major active fault are most likely to experience ground shaking with these strong pulse-like characteristics and connection performance for such buildings should be based on the near-fault protocol. However, qualification of connections for classification as either Type 1 or Type 2 connections should be based on the standard protocol.

A.5.2 Determination of Beam-Column Assembly Capacities and Resistance Factors

Median drift angle capacities for the quantities q_{IO} and q_U should be taken directly from available laboratory data. The median value should be taken as that value from all of the available tests that is not exceeded by 50% of the tests. The value of the quantity f, for each of the Immediate Occupancy and ultimate (Collapse Prevention) states should be determined by the following procedure.

1. Obtain the logarithmic standard deviation of the q_{IO} or q_U values available from the laboratory data. That is, take the standard deviation of the natural logarithms of the q_{IO} or q_U values respectively, obtained from each laboratory test. Logarithmic standard deviation may be determined from the formula:

$$b = \sqrt{\frac{\sum_{i=1}^{n}\left(\ln x_i - \overline{\ln x_i}\right)^2}{n-1}} \tag{A-10}$$

   where:
   b = the standard deviation of the natural logarithms of the test data
   x_i = individual test data value
   n = the number of tests from which data is available
   $\overline{\ln x_i}$ = the mean of the logarithms of the x_i values.

2. Calculate the connection resistance factor f_R due to randomness, the observed variation in connection behavior, from laboratory testing, using the equation:

$$f_R = \mathrm{e}^{-\frac{k}{2b}b^2} \tag{A-11}$$

   where:
   k = the slope of the hazard curve, determined in accordance with Section A.3.2
   b = a coefficient that relates the change in hazard to the change in demand, and which may be taken as having a value of 1.0
   b = the logarithmic standard deviation calculated in accordance with Equation A-10.

3. Determine the connection resistance factor accounting for random and uncertain behaviors from the equation:

$$f = f_R f_U = f_R \mathrm{e}^{-\frac{k}{2b}(0.2)^2} \tag{A-12}$$

   where:
   f_R = the resistance factor accounting for random behavior
   f_U = the resistance factor accounting for uncertainty in the relationship between laboratory findings and behavior in real buildings, and assumed in these *Recommended Criteria* to have a logarithmic standard deviation b_u of 0.2

A.6 Global Stability Capacity

For the Collapse Prevention performance level, in addition to consideration of local behavior, that is, the damage sustained by individual beams and beam-column connection assemblies, it is also important to consider the global stability of the frame. The procedures indicated in this section are recommended for determining an interstory drift capacity C and resistance factor f associated with global stability of the structure.

The global stability limit is determined using the Incremental Dynamic Analysis (IDA) technique. This requires the following steps:

1. Choose a suite of ten to twenty accelerograms representative of the site and hazard level for which the Collapse Prevention level is desired to be achieved.
2. Select one of these accelerograms and perform an elastic time-history analysis of the building. Determine a scaling factor for this accelerogram such that the elastic time history analysis would result in response that would produce incipient yielding in the structure. Determine the 5%-damped, spectral response acceleration S_{aT1} for this scaled accelerogram at the fundamental period of the structure. On a graph with an abscissa consisting of peak interstory drift and an ordinate axis of S_{aT1}, plot the point consisting of the maximum calculated interstory drift from the scaled analysis and the scaled value of S_{aT1}. Draw a straight line from the origin of the axes to this point. The slope of this line is referred to as the elastic slope, S_e
3. Increase the scaling of the accelerogram, such that it will produce mild nonlinear behavior of the building. Perform a nonlinear time-history analysis of the building for this scaled accelerogram. Determine the S_{aT1} for this scaled accelerogram and the maximum predicted interstory drift from the analysis. Plot this point on the graph. Call this point D_1.
4. Increase the scaling amplitude of the accelerogram slightly and repeat Step 3. Plot this point as D_2. Draw a straight line between points D_1 and D_2.
5. Repeat Step 4 until the straight line slope between consecutive points D_i and D_{i+1}, is less than 0.2 S_e. When this condition is reached, D_{i+1} is the global drift capacity for this accelerogram. If $D_{i+1} \geq 0.10$ then the drift capacity is taken as 0.10. Figure A-1 presents a typical series of plots obtained from such analyses.
6. Repeat Steps 2 through 5 for each of the accelerograms in the suite selected as representative of the site and hazard and determine an interstory drift capacity for the structure for each accelerogram.
7. Determine a median interstory drift capacity C for global collapse as the median value of the calculated set of interstory drift capacities, determined for each of the accelerograms. The median value is that value exceeded by 50% of the accelerograms.
8. Determine a logarithmic standard deviation b for random differences in ground motion accelerograms, using Equation A-10 of Section A.5.2. In this equation, x_i is the interstory drift capacity predicted for the i^{th} accelerogram, and n is the number of accelerograms contained in the analyzed suite.
9. Calculate the global resistance factor f_R due to randomness in the predicted global collapse capacity for various ground motions from the equation:

$$f_R = e^{-\frac{k}{2b}b^2} \tag{A-13}$$

where k and b are the parameters described in Section A.5.2 and b is the logarithmic standard deviation calculated in the previous step.

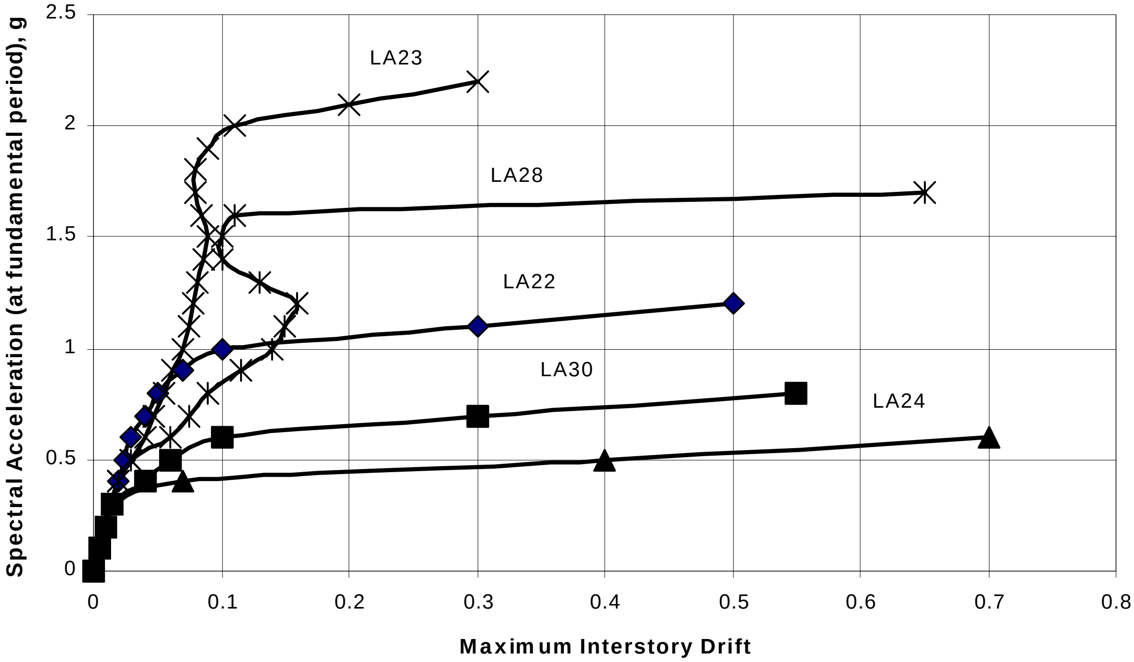

Figure A-1 Representative Incremental Dynamic Analysis Plots

10. Determine a resistance factor for global collapse from the equation:

$$f = f_U f_R = \mathrm{e}^{-\frac{k}{2b} b_U{}^2} f_R \tag{A-14}$$

where:

f_R is the global resistance factor due to randomness determined in Step 9.

b_U is the logarithmic standard deviation related to uncertainty in analytical prediction of global collapse prevention taken as having a value of 0.15 for low-rise structures, 3 stories or less in height; a value of 0.2 for mid-rise structures, 4 stories to 12 stories in height; and taken as having a value of 0.25 for high-rise structures, greater than 12 stories in height.

It is important that the analytical model used for determining the global drift demand be as accurate as possible. The model should include the elements of the moment-resisting frame as well as framing that is not intended to participate in lateral load resistance. A nominal viscous damping of 3% of critical is recommended for most buildings. The element models for beam-column assemblies should realistically account for the effects of panel zone flexibility and yielding, element strain hardening, and stiffness and strength degradation, so that the hysteretic behavior of the element models closely matches that obtained from laboratory testing of comparable assemblies.

Commentary: As noted above, accurate representation of the hysteretic behavior of the beam-column assemblies is important. Earthquake-induced global collapse initiates when displacements produced by the response to ground shaking are *large enough to allow P-D instabilities to develop. Prediction of the onset of P-D instability due to ground shaking is quite complex. It is possible that during an acceleration record a structure will displace to a point where static P-D instability would initiate, only to have the structure straighten out again before collapse can occur, due to a reversal in ground shaking direction.*

The basic effect of P-D instability is that a negative tangent stiffness is induced in the structure. That is, P-D effects produce a condition in which increased displacement can occur at a reduced lateral force. A similar and equally dangerous effect can be produced by local hysteretic strength degradation of beam-column assemblies (FEMA-355C). Hysteretic strength degradation typically occurs after the onset of significant local buckling in the beam-column assemblies. It is important when performing Incremental Dynamic Analyses that these local strength degradation effects, which show up as a concave curvature in the hysteretic loops in laboratory data, are replicated by the analytical model. Nonlinear analysis software that is currently commercially available is not, in general, able to model this behavior. These effects can be approximately accounted for by increasing the amount of dead load on the structure, to produce artificially the appropriate negative stiffness.