APPENDIX A: DETAILED PROCEDURES FOR PERFORMANCE EVALUATION

A.1   Scope

This appendix provides detailed procedures for evaluating confidence levels associated with the ability of damaged WSMF structures to resist collapse in levels of ground shaking likely to occur in the period immediately following a major earthquake. These detailed procedures are provided as a supplement to the level 2 evaluation procedures in Chapter 5. They may be used to demonstrate enhanced levels of confidence with regard to the ability of a particular damaged structure to resist collapse, relative to the confidence levels that may be derived using the more simplified procedures of Chapter 5. The procedures of this appendix are required as a supplement to the Chapter 5 procedures for structures that are classified as irregular, considering the effects of the damage sustained.

> *Commentary: Chapter 5 provides the basic procedures for a level 2 evaluation, using factored demand-to-capacity ratios to indicate a level of confidence with regard to a damaged building's ability to resist collapse in that level of ground shaking likely to occur in the period immediately following a damaging earthquake. The tabular values of demand and resistance factors and confidence indices contained in Chapter 5 were derived using the procedures presented in this appendix, applied to the performance evaluation of a suite of 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 building configurations, that is, irregular structures not well represented by the model buildings used as the basis for the values contained in Chapter 5.*

A.2   Performance Evaluation Approach

A.2.1 Confidence of Ability to Withstand Collapse

The evaluation procedures contained herein permit estimation of a level of confidence associated with the ability of a damaged building to withstand collapse for the levels of ground shaking likely to be experienced within a year following a damaging earthquake.

> *Commentary: The probability that a damaged building may experience collapse as a result of ground shaking likely to occur in the year following a damaging event is calculated as the integral over a year's time of the probability that damage will exceed the collapse capacity of the structure. Mathematically, this may be expressed as:*

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

where:

P(D>PL) =   *Probability of damage exceeding the collapse level in a period of "t" years, taken as 1 year*

$P_{D>PL}(x)$ =   *Probability of damage exceeding a collapse 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 1 year following the first damaging event*

Vulnerability may be thought of as the capacity of the structure to resist collapse. 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 inelastic deformations. Ground accelerations associated with the seismic hazard, and the resulting 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 the probability that damage could exceed a collapse 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 expressed 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 evaluation procedures of Chapter 5 are based on the typical characteristics of standard buildings. Since they are based on the capacity characteristics of typical rather than specific structures, the procedures contained in Chapter 5 inherently 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 procedures of Chapter 5, it may be found that for a specific structure, there is only a 30% level of confidence that the structure is capable of resisting collapse for the levels of ground shaking likely to be experienced in a period of a year following a damaging event. This rather low level of confidence may be more a function of the uncertainty inherent in the procedures used to estimate the probability of collapse than the actual inadequate capacity of the building to resist collapse. 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 an 80 % or 90% level of confidence, in resisting collapse. The difference in such findings can mean the difference between deciding that a building must be vacated or that it can continue to be occupied.

In both the procedures of this appendix and Chapter 5, the uncertainties associated with estimation of the intensity of ground motion have been neglected. These uncertainties can be high, on the order of those associated with structural performance. Thus, the confidence estimated using these procedures is really a confidence with regard to structural performance, given an assumed seismicity, dominated by a single event, consisting of a repeat of the original damaging event, within a period of a year following the initial damaging shock. It is believed that this assumed seismicity is conservative, but credible.

A.2.2 Basic Procedure

As indicated in Chapter 5, a demand and resistance factor design (DRFD) format is used to associate a level of confidence with the probability that a building will be able to resist collapse in the level of ground shaking anticipated in the year following a damaging earthquake. The basic approach is to determine a confidence parameter, λ, which may then be used, with reference to Table 5-7, to determine the confidence level that exists with regard to performance estimation. The confidence parameter, λ, is determined from the factored demand-to-capacity equation:

$$\lambda = \frac{\gamma \gamma_a D}{\phi C} \tag{A-2}$$

where:

C =   median estimate of the capacity of the structure. This estimate may be obtained either by reference to default values contained in the tables of Chapters 5, 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

λ =   a confidence index parameter from which a level of confidence can be obtained by reference to Table 5-7.

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 $P\text{-}D$ 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 ability of a building to resist collapse for the level of ground shaking likely to occur in the year immediately following an earthquake, the following steps are followed:

1. **A best estimate of the ground shaking intensity that caused the initial damage in the building is developed.** This can be done by reference to instrumental recordings of ground motion at the building or nearby sites, the use of standard attenuation relations, or ground shaking contour maps. For the purpose of this evaluation, it is assumed that an event of similar intensity at the building site is likely to occur within a one year period. Ground shaking intensity should be characterized by a 5% damped elastic response spectrum.

2. **A mathematical structural model is developed to represent the damaged building structure.** Note that since damage can result in unsymmetrical structural response, it may be necessary to develop several models. The model(s) are then subjected to a structural analysis, using any of the methods contained in Chapter 5. This analysis predicts the median estimates of maximum interstory drift demand, maximum column compressive demand, and maximum column-splice tensile demand, for the assumed repeat of the original damaging earthquake.

3. **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 5 for regular structures, or the detailed

procedures of Section A.6 of this appendix may be used. These detailed procedures are
required for irregular structures or for regular structures that have been made irregular by the
damage they have sustained. Interstory drift capacity for moment-resisting connections may
be estimated using the default values of Chapter 5, for typical connection types, or 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 5.

4. **A factored-demand-to-capacity ratio λ 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 λ.
   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 5 are used to determine either demand or
   median capacity estimates, than the corresponding values of the demand factors γ and
   resistance factors ϕ should also be determined in accordance with the procedures of that
   chapter. If the procedures of this appendix are used to determine median estimates of
   demand or capacity, then the corresponding demand and resistance factors should be
   determined in accordance with the applicable procedures of this appendix.

5. **Evaluate confidence.** The confidence obtained with regard to the ability of the structure to
   meet the performance objective is determined using the lowest of the λ values determined in
   accordance with Step 4 above, back-calculated from the equation:

$$\lambda = e^{-b\beta_{UT}(K_X - k\beta_{UT}/2)}$$
(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,

β_{UT}=   an uncertainty measure equal to the vector sum of the logarithmic standard deviation
         of the variation 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. This is taken as having a value of 5, representative of the assumed
         seismicity for the year following a damaging earthquake,

K_X =   standard Gaussian variate associated with the 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 5-7 provides a solution for this equation, for various values of the parameters λ and
β_{UT}.

The values of the parameter b_{UT} used in Equation A-3 and Table 5-7 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:

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

where b_{u_i} 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.5 and A.6 indicate how to determine b_{u_i} values associated with demand estimation, beam-column connection assembly behavior, and building global stability capacity prediction respectively.

A.3   Determination of Hazard Parameters

In order to implement these postearthquake evaluation procedures, it is necessary to obtain an estimate of the 5% damped, linear response spectrum for the original damaging earthquake, and to obtain from that response spectrum, an estimate of the spectral response acceleration, S_{aT1} at the fundamental period of the damaged building.

A.4   Determination of Demand Factors

The demand variability factor, g, and analytical 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 and probable intensity of ground shaking during the year following the initial damaging earthquake.

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 demands 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. 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_{DR}.

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

$$g = e^{\frac{k}{2b}b_{DR}^{2}}$$

(A-5)

where:

   k is the logarithmic slope of the hazard curve, taken as having a value of 5.

   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_{DU}.

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:

$$C_{B} = \frac{\text{demand predicted by nonlinear time history analysis}}{\text{demand predicted by analysis method}}$$

(A-6)

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_{DU}.

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

$$g_{a} = C_{B}e^{\frac{k}{2b}b_{DU}^{2}}$$

(A-7)

The analysis uncertainty factors presented in Chapter 5 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_{DU}, used as a basis for the analysis uncertainty factors presented in Chapter 5 are summarized in Table A-1. The bias factors C_B used in Chapter 5 are summarized in Table A-2. It is recommended that these default values for b_{DU} and C_B be used for all buildings.

Table A-1   Default Logarithmic Uncertainty b_{DU} for Various Analytical Methods

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

Table A-2   Default Bias Factors C_B

	Analysis Procedure			
	Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Type 1 Connections				
Low Rise (<4 stories)	0.65	0.80	0.85	1.00
Mid Rise (4 – 12 stories)	0.85	1.15	0.95	1.00
High Rise (> 12 stories)	1.0	1.0	0.85	1.00
Type 2 Connections				
Low Rise (<4 stories)	0.90	1.20	1.25	1.00
Mid Rise (4 – 12 stories)	1.00	1.30	1.35	1.00
High Rise (> 12 stories)	0.70	1.20	1.30	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, contained in Chapter 5 be used for most structures. The procedures
contained in this appendix are most useful for calculating capacities and capacity
factors.*

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 deformations 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 is the preferred demand
parameter for reporting laboratory data. This parameter is less subject to interpretation by
various testing laboratories and also has the advantage that it is approximately equal to the
interstory drift angle predicted by linear structural analyses. Refer to *FEMA 350*, *Recommended
Seismic Design Criteria for New Steel Moment-Frame Buildings* for additional information on
laboratory testing protocols and parameters for reporting test behavior.

Median drift angle capacities *C* and resistance factors f for common connection types are
presented in Chapter 5. These values were determined from cyclic tests of full-size connection
assemblies using the testing protocols indicated below. 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. degradation of moment-resisting capacity of the assembly to a value below the nominal
    moment-resisting capacity,

2. initiation of fracture of bolts, welds, or base metal that results in significant strength
    degradation of the assembly, and

3. complete failure of the connection, characterized by an inability of the connection to
    maintain integrity of the beam to column connection under gravity loading.

Based on this data, drift angle statistics are obtained for a state of incipient collapse, q_u. The quantity q_u, the *ultimate* capacity of the connection, which occurs at damage state 3, above, is used to evaluate the acceptability of local connection behavior.

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 (Krawinkler et al., 1997) 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 proximate 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.

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

Median drift angle capacities for the quantity 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 should be determine by the following procedure.

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

$$b = \sqrt{\frac{\sum_{i=1}^{n}\left(\ln x_i - \overline{\ln x_i}\right)^2}{n-1}} \tag{A-8}$$

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 due to randomness, the observed variation in
    connection behavior from laboratory testing using the equation:

$$f_R = e^{-2.5b^2} \tag{A-9}$$

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

$$f = f_U f_R = 0.9 f_R \tag{A-10}$$

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

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 (Cornell, 1999). 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 non-linear behavior of the structure. 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 for global collapse C as the median value of the calculated set of interstory drift capacities, determined for each of the accelerograms. Note that the median value is that value exceeded by 50% of the accelerograms.
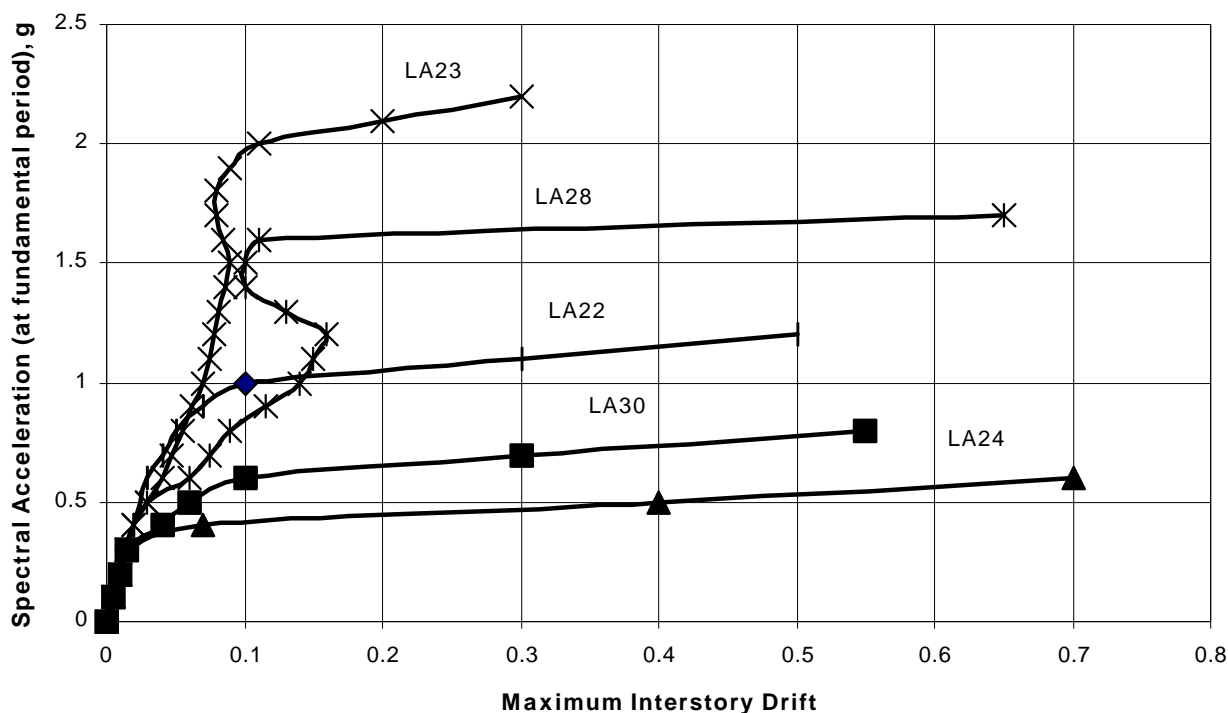

Figure A-1 Representative Incremental Dynamic Analysis Plots

8. Determine a logarithmic standard deviation for random differences in ground motion accelerograms, b_R, using Equation A-8 of Section A.5.2. In this equation, x_i is the interstory

drift capacities predicted for the ith accelerogram, and n is the number of accelerograms contained in the analyzed suite.

9. Calculate the global resistance factor $\boldsymbol{f}_R$ due to randomness in the predicted global collapse capacity for various ground motions from the equation:

$$\boldsymbol{f}_R = e^{-\frac{k}{2b}b^2} \tag{A-10}$$

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

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

$$\boldsymbol{f} = \boldsymbol{f}_U \boldsymbol{f}_R = e^{-\frac{k}{2b}b_U^{\,2}} \boldsymbol{f}_R \tag{A-11}$$

where:

$\boldsymbol{f}_R$ is the global resistance factor due to randomness determined in Step 9.

$\boldsymbol{b}_U$ is the logarithmic standard deviation related to uncertainty in the 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 steel moment 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 structures. 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-$\boldsymbol{D}$ instabilities to develop. Prediction of the onset of P-$\boldsymbol{D}$ instability due to ground shaking is quite complex. It is possible that an acceleration record will displace a structure to a point where static P-$\boldsymbol{D}$ instability would initiate, only to bring the structure back again before collapse can occur, due to a reversal in ground shaking direction.

The basic effect of P-$\boldsymbol{D}$ instability is that a negative stiffness is induced in the structure. That is, P-$\boldsymbol{D}$ effects produce a condition in which increased displacement can occur at 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 (IDA) 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. Increasing the amount of dead load on the structure, to produce artificially the appropriate negative stiffness, can account approximately for these effects.