PERFORMANCE BASIS OF GUIDELINES FOR EVALUATION, UPGRADE AND DESIGN OF MOMENT-RESISTING STEEL FRAMES

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SUMMARY

Following the 1994 Northridge earthquake, many welded, moment-resisting steel frame buildings were discovered to have brittle fractures of their beam-column connections. This behavior had not been widely anticipated, was contrary to the intended ductile behavior, and called to question the ability of such structures to reliably resist future earthquakes. On behalf of the Federal Emergency Management Agency, the SAC Joint Venture embarked on a program of analytical and laboratory investigations to determine the causes of this behavior, better understand earthquake response of frame structures, and develop design and construction recommendations that would provide structures capable of more reliable earthquake resistance.

Extensive laboratory investigations provided statistical information on the hysteretic behavior of beam-column connections. Analytical investigations provided statistical information on the response of frames incorporating such connections and the ability of analytical techniques to predict this response. These investigations permitted development of a reliability-based approach for evaluation of the probable performance of moment-resisting steel frame structures. This approach forms the basis for recommended design, evaluation, repair and upgrade criteria.

These design and evaluation criteria employ a Demand and Resistance Factor Design (DRFD) format that accounts for the randomness and uncertainty inherent in earthquake demand and structural capacity prediction. Two performance levels are addressed – a state of incipient collapse, termed Collapse Prevention, and a state of incipient damage. Interstory drift is adopted as the primary performance-prediction parameter. Acceptance criteria (resistance) are developed both for local (connection or element) and global (frame) behavior. Resistance factors, dependent on structural configuration, connection type and construction quality are applied to account for uncertainties and randomness in capacity prediction. Computed drift demands are factored, based on the analysis used, modelling assumptions and hazard characteristics to account for the uncertainties and variability inherent in drift demand response prediction. The quotient of factored capacity and demand indicates the level of confidence that desired performance will be attained within specified probability of non-exceedance.

Application of these procedures to buildings designed in accordance with current United States design standards for moment-resisting steel frame construction indicates that acceptable margins against global collapse are generally provided by these procedures. However, beam-column connections must be significantly more robust than previously anticipated to provide suitably high confidence that endangerment of life safety will not occur due to local failure of the gravity load carrying system. Also, these structures are far more susceptible to damage in more moderate, and frequent earthquake events than previously imagined.

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INTRODUCTION

Prior to the 1994 Northridge earthquake, structural engineers believed that buildings provided with lateral-force resisting systems composed of welded, moment-resisting, steel frames (WSMF) would be capable of very reliable seismic performance. It was expected that when such buildings were subjected to strong earthquake ground motion, structural damage would be limited to moderate yielding of the beams, columns and panel zones, perhaps with some localised buckling, and limited permanent lateral deformation. Because no WSMF structure had ever experienced earthquake-induced collapse, it was expected that such behavior was highly improbable.

Initial investigations of building performance following the Northridge earthquake tended to confirm these expectations. However, it soon became apparent that a number of WSMF buildings had experienced brittle fractures at the welded joints of some beam to column connections. In many cases this damage was subtle and consisted of a small crack through the weld (or heat affected zone) material at the beam flange to column flange joint. In other cases, the fractures extended into the column flanges. Sometimes such fractures followed a curved surface within the thickness of the column flange and resulted in the withdrawal of a large “diovt” of material from the face of the column flange. In other cases, the fractures progressed through the column flange and extended into the column webs, sometimes completely severing the column. Damage to shear tabs connecting beam webs to columns sometimes occurred at connections where welded joints had failed. In a few WSMF buildings large, permanent interstory drifts accompanied this damage.

Following discovery of this damage a program of testing of large-scale, welded, beam-column assemblies was conducted on behalf of the American Institute of Steel Construction and the Getty Foundation [Engelhardt and Sabol, 1994]. This testing initially focused on the typical welded connections prescribed by the building code for use in earthquake-resistant WSMF construction and fabricated using contemporary standard practice. This testing dramatically demonstrated that such connections were incapable of reliably developing any significant inelastic deformation and would consistently develop brittle fractures like those found in buildings affected by the Northridge earthquake, at loading approximating the yield strength of the assembly. The Structural Engineers Association of California, the Applied Technology Council and the California Universities for Research in Earthquake Engineering then formed the SAC Joint Venture, and with funding provided by the Federal Emergency Management Agency (FEMA), led a nation-wide effort to discover the causes of this brittle connection behavior and to develop improved connection details that would be capable of more reliable inelastic performance. This effort culminated with the publication of *Interim Guidelines* [SAC, 1995] that provided preliminary consensus opinion as to the causes of the unanticipated behavior together with interim recommendations for improved design and construction practice. These guidelines recommended that connection details for new construction be demonstrated by approved cyclic testing procedures to be capable of sustaining at least 0.03 radians of plastic rotation demand. The guidelines also provided information on a series of connection details that limited testing had indicated to be capable of meeting this criterion. The selection of the 0.03 radian acceptance criteria was based on the observation that laboratory specimens meeting this criteria generally failed in a ductile rather than brittle manner and the qualitative belief that such deformations exceeded any demands likely to be induced in buildings by credible earthquakes.

The SAC *Interim Guidelines* recommendations were rapidly extended and adopted into the building codes and practice. The 1997 edition of the *NEHRP Recommended Provisions* [BSSC, 1997a] and the AISC seismic design specification [AISC, 1997], the resource documents for the building code provisions governing design of steel frame buildings, defined three classes of moment-resisting frame structures termed Ordinary (OMF), Intermediate (IMF), and Special (SMF). SMF construction is permitted for buildings of any height and for regions of any seismic hazard. The provisions for SMF design generally follow the SAC *Interim Guidelines* recommendations, requiring connections capable of providing minimum 0.03 radians of plastic rotation capacity. IMF and OMF structures are intended for use in zones of lower seismicity where anticipated seismic demands are reduced. The design provisions for these structures respectively permit connections with demonstrated plastic rotation capacity of 0.02 and 0.01 radians. It is important to note however, that no quantitative demonstration of the adequacy of these requirements to provide suitable seismic performance in building structures was performed as part of the development of those provisions.

Following the publication of the *Interim Guidelines,* SAC received supplemental funding from FEMA to perform extensive additional investigations into the earthquake behavior of WSMF structures and to develop final, performance-based recommendations for evaluation of existing structures and design of new structures. The investigations conducted by SAC in this phase 2 program include extensive laboratory research into the properties of base materials and welds, the parameters controlling connection performance and the hysteretic behavior of a wide range of bolted and welded, partial and full strength, partially restrained and fully restrained connection assemblies. The research also included extensive non-linear analytical investigations into the response of WSMF structures to different ground motions and the effect on this response of different connection hysteretic behaviours. These investigations are reported elsewhere in these proceedings. This paper describes the statistical approach used to develop connection and system design criteria with defined capability of meeting performance goals. The resulting *SAC Design Criteria* [SAC, 2000] are scheduled for publication in early 2000.

PERFORMANCE BASIS

Recently, code and design guideline documents in the United States, including the *NEHRP Rehabilitation Guidelines* [FEMA, 1997] and *NEHRP Recommended Provisions* have moved towards adoption of a performance-based design approach. The general concept of this performance basis is that designs should be capable of providing a high level of confidence that damage will not exceed certain limits, given that design ground motions are experienced. The permissible level of damage is termed a performance level while the combined specification that the performance level not be exceeded for a specific ground shaking hazard is termed a performance objective. For example, the *NEHRP Recommended Provisions Commentary* [BSSC, 1997b] states a performance objective that buildings designed and constructed to the criteria for ordinary occupancy buildings should not collapse when subjected to maximum considered earthquake (MCE) ground shaking demands. These MCE demands are generally defined as having a 2% probability of exceedance in 50 years. The *NEHRP Rehabilitation Guidelines* permit users to specify performance objectives by coupling any of four performance levels or ranges (collapse prevention, life safety, damage control, immediate occupancy) with a specific ground motion hazard. However, no attempt has ever been made to quantify in a realistic manner, the confidence levels provided by the *NEHRP Recommended Provisions* or the *NEHRP Rehabilitation Guidelines* to actually satisfy the implied design criteria.

The approach to characterising performance adopted by the *SAC Design Criteria* is subtly different from that adopted by both the *NEHRP Rehabilitation Guidelines* and *NEHRP Recommended Provisions*. The *NEHRP Rehabilitation Guidelines* define performance objectives as having a high, but unquantified confidence of not exceeding a certain performance level conditioned on the occurrence of ground shaking with a defined of probability of exceedance. The performance objectives adopted by the *SAC Design Criteria* are stated as having a defined level of confidence of less than a specific probability of exceeding the desired performance levels, considering all levels of ground shaking that may occur. Two specific levels of performance are considered. The first, termed Collapse Prevention, is a state of incipient local or global collapse. It occurs at those levels of interstory drift at which either 1) local beam-column connections are damaged to the extent that they loose ability to carry gravity loads, e.g. through failure of the shear connection; and 2) the frame system develops dynamic P-delta instability and initiates a global collapse. Since the life safety consequences of global collapse are more severe than those of local beam-column connection failures and possible local, partial collapse, the *Design Criteria* seeks to provide greater confidence against the potential of global failure than it does local failure. In a manner that is analogous to the approach taken by the NEHRP *Recommended Provisions,* which seek a high level of confidence that collapse would not occur given that ground shaking demands with a 2% exceedance probability in 50 years are experienced; the *SAC Design Criteria* provide at least a 95% confidence of less then a 2% probability of collapse in 50 years.

The second performance level addressed by the *SAC Design Criteria* is that state, in which structural damages, consisting of either permanent deformation or connection fracture, initiates. This performance level is termed incipient damage. As the current practice of United States building codes is to address only the protection of life safety, the *SAC Design Criteria* are not intended to provide specific probabilities of exceeding the incipient damage state. However, they do permit users to specify a desired probability of nonexceedance for the incipient damage state and to then calculate for a given design, the confidence provide of achieving such performance.

PERFORMANCE EVALUATION PROCEDURE

Basic Approach

Interstory drift is used as the primary parameter to predict performance. Clearly, interstory drift is closely related to the development of P-delta instability and therefore is a logical response parameter for the prediction of global collapse. Similarly, the amount of interstory drift experienced by a structure is also closely related to the amount of local deformation induced on individual beam-column connections and therefore is also closely related to local performance. Using this parameter, the basic approach adopted to evaluate performance becomes one of determining if the probability of developing interstory drifts that exceed the interstory drift capacity of the structure, for a given performance level, exceeds the desired probability, taking into account the uncertainties and randomness inherent in predicting both structural response and capacity. This adopted approach is based upon foundation work developed in two topical investigations [Wen and Foutch, 1997], [Jalayer and Cornell, 1998] performed as part of the SAC phase 2 program.

This approach adopted by the *SAC Design Criteria* is quite different from that adopted by the *NEHRP Rehabilitation Guidelines* and other recently developed performance-based design approaches. In the *NEHRP Rehabilitation Guidelines* performance is evaluated by analysing the structure for a ground motion with the desired exceedance probability and determining if the resulting deformation demands are less than the deformation capacity for the structure for the desired performance level. If the computed drift demands are less than the capacity, then it is declared that the structure has greater than the desired probability of nonexceedance for the desired performance. This procedure neglects the fact that as a result of both uncertainties and randomness, it is not possible for us to precisely predict either the drift produced by ground motion with a specific exceedance probability or the capacity of the structure. In order to account for the uncertainty and randomness inherent both in drift and capacity prediction, we will instead, integrate the interstory drift capacity to demand (C/D) ratio as a function of ground motion intensity over the hazard curve, which provides ground motion intensity as a function of probability of exceedance, in order to obtain the probability of exceedance for various capacity to demand ratios. If the computed C/D ratio at the desired probability of exceedance is equal to 1.0, than in the mean, the desired probability of exceedance for the structure meeting the target performance is attained. To the extent that the integrated C/D ratio exceeds 1.0, we are more confident that the desired performance is attained and to the extent that it is less than 1.0, we are less confident. In order to simplify discussion, the following sections discuss the integration of demand and capacity individually.

Interstory Drift Demand

Uncertainty is a measure of the error introduced into calculations as a result of our inability to precisely characterise reality due to imperfect knowledge. For example, although we can precisely predict the axial force that will cause a test coupon to yield in a testing machine if we know the yield strength of the material, until such time as we perform the test, we are uncertain as to what this yield strength is. We can estimate the strength and also the possible error associated with this estimate based on statistical studies of mill production records. However, we can not precisely define the strength until we perform tests on the specific material. In theory, uncertainty can always be reduced by performing more study. In our example, if we actually perform the strength test, we can precisely define the material strength, and reduce the uncertainty in strength prediction to near nil levels. Uncertainty in prediction of interstory drift demand results from inaccuracies in the analytical procedures used to estimate drift, termed procedure uncertainty (UP); inability to accurately model the structure, termed modelling uncertainty (UM); inability to precisely define the structure's effective damping (UD); inability to precisely define the strength of materials in the structure (US); and inability to precisely define the effective vertical (gravity) load at the time the earthquake occurs (UG). Of these various sources of uncertainty in drift demand prediction, procedure uncertainty (UP) is the most significant and damping uncertainty (UD) is somewhat significant. The other sources of uncertainty are relatively insignificant.

Randomness is a measure of our inability to precisely characterise reality resulting from our imperfect understanding of the factors that affect the phenomenon. For example, the attenuation relationships used to estimate ground accelerations have been developed based on regression analyses on databases of past instrumental recordings. These regression analyses demonstrate that there is statistically significant correlation between ground motion strength and earthquake magnitude, distance, soil type and fault type, but not perfect correlation. This is because ground shaking intensity is also dependent on other factors that we have not yet defined. Therefore, the ability of attenuation equations to predict ground motion is not perfect, even if distance, magnitude and the other factors that are currently defined are precisely known. The error resulting in these predictions due to these undefined parameters may be termed randomness. Generally, randomness can not be reduced, until the state of knowledge moves forward. In the prediction of drift demand, randomness arises primarily from the variation in structural response that occurs from one ground motion record to another, independent of the spectrum to which the ground motion records are scaled. That is, when two ground motion records, both of which match a given design spectrum, are analysed, using our most perfect analytical models and methods, they will likely produce two different predictions of drift demand. We term this randomness, record to record randomness (RR).

Although it would be possible for an individual engineer to characterise the extent of randomness and uncertainty inherent in a prediction of drift demand for a specific structure, through the performance of Monte Carlo simulations, it would be highly impractical to require this as part of a general purpose design or evaluation procedure. Therefore, as part of the SAC topical investigations, a suite of model WSMF buildings were designed and used as the basis for determining typical uncertainties and randomness inherent in drift predictions using varying analytical techniques and for structures of different sizes and design characteristics. The model buildings consisted of 3-story, 9-story, and 20-story structures, designed for different geographic locations, and under different design specifications. These were then analysed using linear static, linear dynamic, non-linear static and non-linear dynamic methods, and using analytical models of different complexity, to characterise the variation in predicted interstory drift demand due to the various sources. This has allowed the SAC project team to develop a quantified measure of both the randomness, ${\sigma_R}^2$, and uncertainty, ${\sigma_U}^2$, inherent in drift prediction depending on the analysis procedure employed and other factors. These measures of uncertainty are the squares of the standard deviations of the natural logarithms of drift prediction due to respectively, the random and uncertain factors. When drift as a function of ground motion intensity is integrated with ground motion intensity as a function of probability of exceedance, this produces a product of integration that includes these measures of randomness and uncertainty. We term this product of integration γ. It is given by the expression:

$$\gamma = e^{\frac{k}{2b}\left(\sigma_{U_D}^2+\sigma_{RD}^2\right)} \tag{1}$$

In this equation, *k* is the logarithmic slope of the hazard curve, i.e. the amount of change in the log of ground motion intensity with the log of the probability of exceedance, evaluated at the desired probability of exceedance in the performance statement; *b* represents the variation of drift with ground motion intensity (approximately 1, except when approaching unstable response) and the σ values are the measures of randomness and uncertainty in drift demand prediction as described above. These γ factors are tabulated in the *Design Criteria* for various analytical procedures, zones of seismicity and types of analytical models. In order to find the mean value of the drift demand at a desired probability of exceedance, an engineer using this procedure need only apply the appropriate γ factor to the calculated interstory drift demand, ΔD, obtained from a conventional analysis of the structure's response to ground motion with the desired exceedance probability.

Interstory Drift Capacity

Interstory drift capacity is dependent on both local and global behavior. For the case of the collapse prevention performance level, the local capacity is limited by that total deformation that causes a beam-column connection to loose vertical load carrying capacity. There are several modes of connection failure that can result in such behavior, depending on the connection configuration. In conventional welded moment connections this typically consists of failure of the beam flange to column weld, followed by failure of the shear connection between the beam web and column. Laboratory testing of many full-scale beam-column assemblies by SAC and other researchers has provided a database from which statistics on the interstory drift demands at which such failures occur can be obtained. From these statistics, and depending on the connection characteristics, it is possible to select a median value of interstory drift capacity for each connection type and failure mode as well as logarithmic standard deviations for these failure drifts. The variation in connection behavior observed in the laboratory tests is ascribed to randomness, and we term this measure of variation ${\sigma_R}^2$. Similar capacities and measures of variation can be obtained from the laboratory test data for the incipient damage performance level.

In order to determine global drift capacity for the collapse prevention performance level, a series of incremental dynamic analyses (IDAs) are performed using a mathematical model that represents the best estimates of stiffness, damping and hysteretic behavior for the structure. In order to perform the IDA, a ground motion acceleration record, representative of the magnitude and source characteristics that dominate the hazard at the desired exceedance probability is selected. This is scaled to a sufficiently low value of spectral response acceleration, at the fundamental period of the structure, such that the structural response is linear. A response history analysis is performed and the maximum predicted interstory drift value obtained. The ground motion record is then incrementally increased and the analysis repeated, accounting for non-linear response, when it occurs. The process of incrementing the strength of the record and re-performing the dynamic analysis is repeated until either structural instability is produced or the predicted drift demand is very large. A plot is then made of the predicted interstory drift maxima for each analysis against the spectral acceleration value to which the ground motion record was scaled for that analysis. The interstory drift capacity of the structure, for this record, is taken as the lesser of that interstory drift at which the slope of the IDA plot becomes flat or a value at which confidence is lost in the validity of the analysis. A flattening of the IDA response curve indicates that small increment in ground motion intensity produces a large increment of drift response, an indicator of unstable behavior. In our analyses we have assumed that when the slope of an IDA plot reaches 1/5 the slope in the range of elastic response, instability is indicated. The maximum interstory drift at which the analysis is considered valid is 10%. This process is repeated for a series of ground motion records, and the maximum interstory drift vs. spectral response acceleration curves for all of these runs are plotted on a common graph. Figure 1 is a representative series of IDA plots for a 9-story model building, subjected to five different Los Angeles, California ground motion records representative of the hazard at a 2%/50 year exceedance probability.

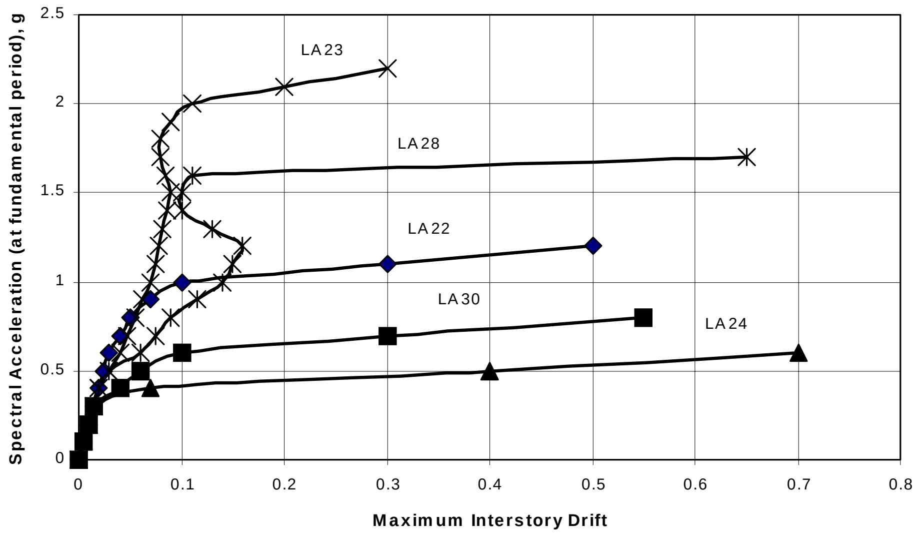

Figure 1:Typical Incremental Dynamic Analysis Plot

From IDA plots, such as those indicated in Figure 1, it is possible to obtain a median interstory drift capacity for the structure and statistics on the randomness in the interstory drift capacity resulting from record to record variation. This is again taken as $\sigma_R{}^2$. The amount of uncertainty introduced into the estimate of structural drift capacity obtained from the IDA process, due to modelling approach, hysteretic behavior assumptions, assumed damping and similar parameters, can be estimated by varying the assumptions, re-performing the analyses and obtaining statistics on the variation of the predicted capacity with variation in these parameters. The resulting measure of uncertainty is taken as $\sigma_U{}^2$. Once these measures of randomness and uncertainty have been developed, a product of integration of the capacity over the hazard curve is obtained in a manner similar to that previously described for the integration of the demand and the hazard curve. This yields a capacity factor, ϕ, given by the equation:

$$\phi = e^{-\frac{k}{2b}\left(\sigma_{UC}^2 + \sigma_{RC}^2\right)} \tag{2}$$

where the parameters *k*, *b*, $\sigma_U{}^2$, and $\sigma_R{}^2$ are as previously described. Finally, we are ready to determine the C/D ratio at the desired probability of exceedance, as:

$$\lambda_{con} = \frac{\phi \Delta_C}{\gamma \Delta_D} \tag{3}$$

where the values of γ and ϕ are as given by equations 1 and 2, respectively and Δ_D and Δ_C are respectively the computed interstory drift demands and capacities. The process for obtaining Δ_C described above is admittedly quite tedious. Since such calculations are beyond the level of effort a design engineer could reasonably be expected to perform, the *SAC Design Criteria* tabulates default values of $\phi\Delta_C$ for structures of different configuration and different zones of seismicity.

As previously, described, when the computed value of λcon has a value of 1.0, then in the mean, the probability of having poorer performance than desired is less than the target level. The specific level of confidence inherent in a performance prediction can be quantified by examining the uncertainties inherent in the estimate of demand and capacity. This is done by evaluating the computed value of λ_{con} with the expression:

$$\lambda_{con} = e^{\left[K_x \sigma_{UT} - \frac{k}{2b}\sigma_{UT}^2\right]} \qquad (4)$$

In equation 4, *k* and *b* are the slope of the hazard curve and the measure of variation of the drift response to hazard intensity, previously described; σ_{UT} is the total logarithmic standard deviation due to uncertain parameters computed as the square root of the sum of the squares of each of the σ_U computed in the capacity and demand analyses; and K_x is the number of logarithmic standard deviations above the median, assuming a lognormal distribution, at which varying levels of confidence are attained.

Application of Approach

In application, this rather complex procedure outlined above is greatly simplified and made no more difficult to apply than typical load and resistance factor approaches for design. In practice, using the *SAC Design Criteria*, the engineer would develop a mathematical model for his building structure. He would select a desired performance objective, e.g. less than a 2%/50 year probability of exceeding a collapse prevention state. Then he would obtain a ground motion corresponding with this exceedance probability (in this case 2%/50 years) and perform a structural analysis to estimate the interstory drift demand. Any of several analysis procedures can be used including an equivalent lateral force procedure, response spectrum analysis, linear or non-linear response history analysis, or pushover analysis. Depending on the type of analysis used, and the sophistication of the mathematical model, the engineer would select a demand factor, γ, from tables contained in the *SAC Design Criteria*. He would also obtain tabular values of the inter-story drift capacity, Δ_C and resistance factor ϕ, based on the types of connections present in the building, the building's configuration, the seismicity of the site, and the desired performance level. Having obtained these values, the engineer would compute λ_{con}, using equation (3). Finally, the engineer would solve equation (4) for K_x and enter a table that relates K_x to confidence level. The end result is a conclusion that there is *XX*% of confidence that the desired performance will be exceeded at lower probability than YY% in 50 years.

CONCLUSIONS

In the development of the *SAC Design Criteria*, a large number of analyses were performed using the procedures described above to determine if existing strength and stiffness criteria contained in United States building codes provide adequate levels of confidence with regard to protection against either global or local collapse. In general, it was determined that structures designed either as SMF or IMF systems provide greater than a 95% confidence of less than a 2% probability of global collapse initiation in 50 years. However, much lower levels of confidence are provided of less than a 2% probability of local impairment of gravity load carrying capacity in 50 years. As would be anticipated, the associated confidence is particularly low for structures constructed using connection details qualified for IMF systems.

As a result of these findings, the *SAC Design Criteria* provides no recommendations for IMF systems, as it is felt that these systems inherently provide too little safeguard against the potential for local collapse. It is anticipated that future editions of the building codes will either drop consideration of IMF systems or require that IMF systems be designed with larger stiffness and better drift control to provide for enhanced performance. The *SAC Design Criteria* provide a toolkit of pre-qualified connections, which are believed to be sufficiently robust to provide acceptable performance under the drift demands anticipated for SMF systems. These connections include reduced beam section, welded cover plate, welded flange plate, free flange, bolted end plate, bolted flange plate and bolted T-stub configurations. Each prequalification contained in the *SAC Design Criteria* includes limitations with regard to the materials, size of members, and design procedures that must be employed in sizing the welds, bolts and other components of the connection in order to meet the prequalification. Median drift capacities and resistance factors are tabulated for these connections to permit performance evaluation of structures employing these connections. In addition, the *SAC Design Criteria* provides a uniform procedure for qualifying connection details outside of the range of their prequalification as well as for developing qualifications for other types of connections.

The development of the *SAC Design Criteria* truly represents a milestone in the development of design procedures for seismic resistance. It is the first performance-based procedure that explicitly accounts for the uncertainty and variability inherent in performance prediction. It adopts a flexible reliability-based approach that can easily be adapted for application to other structural systems. Indeed, the American Institute of Steel Construction has expressed an interest in extending this approach to other steel frame systems, and similar approaches are being developed for application to composite steel and concrete structures.

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