FEMA-273 SEISMIC REHABILITATION GUIDELINES THE NEXT STEP - VERIFICATION

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ABSTRACT

FEMA-273/274, *Guidelines and Commentary for Seismic Rehabilitation of Buildings* (ATC, 1996a., b.) represents a landmark in the practice of structural/seismic engineering. In addition to providing a long-needed consensus basis for the design of building seismic upgrades, it also is a first major step towards the development of performance-based design procedures for seismic resistance. Under the *Guidelines*, design is performed with the expectation that for specified levels of ground motion intensity, building performance will remain within anticipated levels. Performance levels are defined in terms of permissible damage to individual structural and nonstructural components and are intended to provide specific defined margins of safety. In order to accommodate this performance-based approach new analytical procedures and acceptance criteria were developed. While the *Guidelines* represent a significant advance in the practice of earthquake engineering, calibration of the analysis procedures and acceptance criteria to real building performance is clearly needed. The use of strong motion data obtained from instrumented buildings experiencing strong earthquake ground shaking will be an essential part of this process.

INTRODUCTION

In the early 1980s, the Federal Emergency Management Agency (FEMA) committed to a plan to facilitate the mitigation of existing hazardous buildings in the United States. This plan included the development of guidelines and resource documents that would permit engineers and building officials to evaluate the existing hazards, estimate the range of costs required to mitigate these hazards through building upgrade programs, and to actually implement such programs. Under this plan, a series of guideline and resource documents were developed. These included ATC-14 (ATC-1987), a methodology for engineering evaluation of buildings to determine if they posed unacceptable risks to life safety; ATC-21 (ATC-1988) a handbook to allow rapid estimation of the probability of severe damage to buildings, based on consideration of building type, age, configuration and siting; FEMA-172 (URS - 1992) a guideline indicating alternative techniques that could be used to implement seismic rehabilitation of buildings; FEMA-178 (BSSC-1992) an update to the evaluation guidelines first presented in ATC-1986 and later modified in ATC-22 (ATC-1989); FEMA-157 (Englekirk & Hart - 1988) a guide to the probable range of costs to perform seismic rehabilitation in buildings; and most recently, FEMA-273/274 (ATC-1996a., b.) *Guidelines and Commentary for Seismic Rehabilitation of Buildings*.

As the program for the mitigation of hazards in existing buildings matured through the 1980s and 1990s, its goals evolved. Initially, the program intent was to encourage owners of hazardous buildings to upgrade these structure so as to bring the earthquake posed risk to life safety to an acceptable level. The purpose of each of the earlier documents in the series including ATC-1986, ATC-1988, ATC-1990 and FEMA-178 was to permit the identification of buildings that would pose an undue risk to life safety either through collapse, generation of large falling debris or loss of ready egress capability. However, as engineers began to use these resource documents to evaluate buildings, and building owners contemplated programs to rehabilitate their buildings and reduce the existing seismic risk, these owners began to ask for alternatives with regard to the performance the rehabilitated buildings would provide. While some owners were most interested in protecting the safety of the occupants of their buildings, other owners were interested only in preventing their buildings from collapsing and still others, particularly institutions, utilities and large businesses, became interested in minimizing the risk of earthquake induced interruption of the use of their facilities. Therefore, when FEMA commissioned the development of the rehabilitation guidelines document, in 1991, it sought to produce a document which would permit engineers to address each of these performance goals that were commonly being requested by the public.

Although not originally intended as such, the development effort for the *Guidelines for Seismic Rehabilitation* was to become the vanguard for a long series of research and development efforts intended to develop performance-based seismic design procedures. FEMA turned to a partnership of the American Society of Civil Engineers (ASCE), the Building Seismic Safety Council (BSSC) and Applied Technology Council (ATC) to develop this document. ASCE was responsible for the development of a data base of pertinent research results from the literature for use by the Guideline writers, as well as to provide consultation on issues relating to archaic construction systems and historic preservation and to hold a series of workshops to obtain input on the development of the *Guidelines* from various stakeholder groups. BSSC managed the program and also provided a consensus balloting process for review and approval of the *Guidelines*. ATC was responsible for developing the actual *Guidelines* and *Commentary* documents, through the assembly of a large team of researchers and structural engineering practitioners.

The performance-based design approach adopted by the Guidelines has three basic components. The first of these is the specification of a performance objective, comprised of an anticipated performance level and the design earthquake ground motion at which this performance is to be achieved. Performance levels are defined in terms of permissible levels of damage to individual building components and were formulated with the intent of providing specific margins against failure. The second component of the performance-based design approach is structural analysis. Analysis predicts the demands (forces, deformations, etc.) on the individual components and is performed in order to determine if a design is capable of meeting the selected performance objectives. Acceptance criteria provide the third component of the performance-based design process. Acceptance criteria are limiting values of component forces and deformations, against which the component demands predicted by the structural analysis are evaluated, in order to determine design acceptability.

STRUCTURAL PERFORMANCE

Three discrete structural performance levels are defined in the *Guidelines*. These are respectively termed the Immediate Occupancy Level, the Life Safety Level and the Collapse Prevention Level. The Immediate Occupancy Level is a damage state in which the building has experienced only very limited damage, perhaps consisting of minor yielding and cracking of elements. Structures meeting this performance level would retain nearly all of their pre-earthquake strength and stiffness and would be structurally safe for immediate post-earthquake occupancy. Repairs, if required, could be performed at the convenience of the owner and occupants. The Life Safety Level is a damage state in which the building has experienced significant damage to its structural components including yielding, cracking, spalling, and buckling, and perhaps, limited fracturing. Buildings meeting this level have substantially reduced post-earthquake stiffness and may have somewhat reduced post-earthquake strength. Such structures will generally require structural repair and may be judged unsafe for post-earthquake occupancy until such repairs can be made or at least, temporary shoring and bracing is installed. The Collapse Prevention Level is a state in which extreme damage, short of collapse, has occurred to the structure. Buildings damaged to this extent may experience large permanent lateral drifts as well as extensive localized failures of structural components. The stiffness of such a structure would be substantially degraded and the lateral strength may also be significantly reduced. Such buildings would not be safe for post-earthquake re-occupancy and may not be economically practical to repair and restore to service.

Both the Collapse Prevention and Life Safety performance levels have associated with them, a target margin against collapse. Margin may be thought of as the inherent factor of safety between the loading that the structure is designed to resist and the loading that would produce failure. When buildings are subjected to strong earthquake ground shaking, they generally behave in an inelastic manner. Although the building code provisions for earthquake design have typically been based on providing minimum specified levels of lateral force-resisting capacity in a structure, force is not a particularly useful design parameter for predicting the margin of a structure that is responding within the inelastic range of behavior.

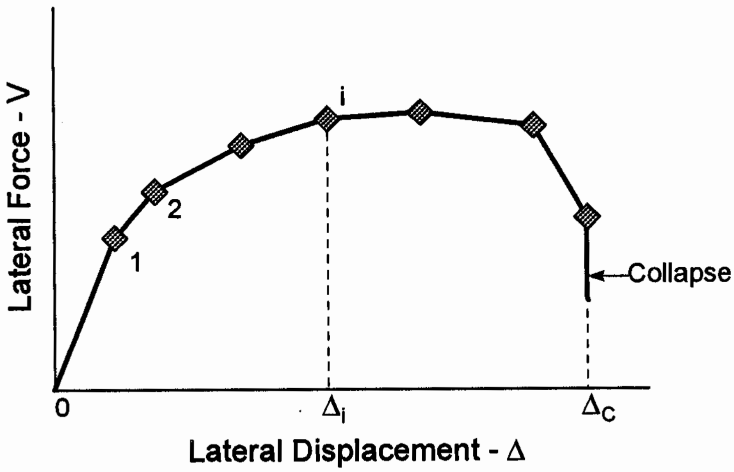

Figure 1 - Lateral Force - Displacement Curve

Figure 1 depicts the force-deformation behavior of a simple structure with the ability to respond to ground shaking in a non-linear, or inelastic manner. The vertical axis of this figure is the applied lateral force on the structure, V, and the horizontal axis is the resulting lateral displacement of the structure. At point "0" there is no lateral loading of the structure. The portion of the curve between points "0" and "1" represents the range of elastic behavior, in which no damage has occurred and the structure retains all of its strength and stiffness. As the structure is loaded beyond point "1" on the curve, damage events such as cracking, yielding, or buckling of elements, indicated by the symbol "◆" in the figure, start to occur. Each damage event results in a degradation of the structure's lateral stiffness and may also result in a degradation of lateral strength. As can be seen from the figure, once several damage events have occurred, lateral force becomes a relatively insensitive parameter by which to judge the structure's performance, since a wide range of different damage states can occur at a relatively constant level of applied lateral force. Because of this, FEMA-273 selected lateral displacement as the basic parameter by which structural performance is judged or controlled. In the figure, at any point "i" on the curve, the margin against failure can be defined as the ratio of the displacement at which collapse is likely to occur, Δ_c, to the displacement at point "i", Δ_i.

Figure 2 presents a curve similar to that of Figure 1, except that the FEMA-273 performance levels have been superimposed on the curve. As can be seen, the Collapse Prevention performance level is defined to have a margin of 1.0 against collapse. That is, the maximum lateral displacement induced in a structure meeting the Collapse Prevention performance level should not exceed the lateral displacement at which collapse is likely to occur. The Life Safety performance level is defined to have a margin of 1.33. This implies that the maximum lateral displacement induced into a structure meeting the Life Safety level is 1/1.33 or 3/4 of the lateral displacement at which collapse is likely to initiate. The Immediate Occupancy level does not have a specific margin associated with it, but instead, is achieved by maintaining damage to individual components of the structure to very low levels.

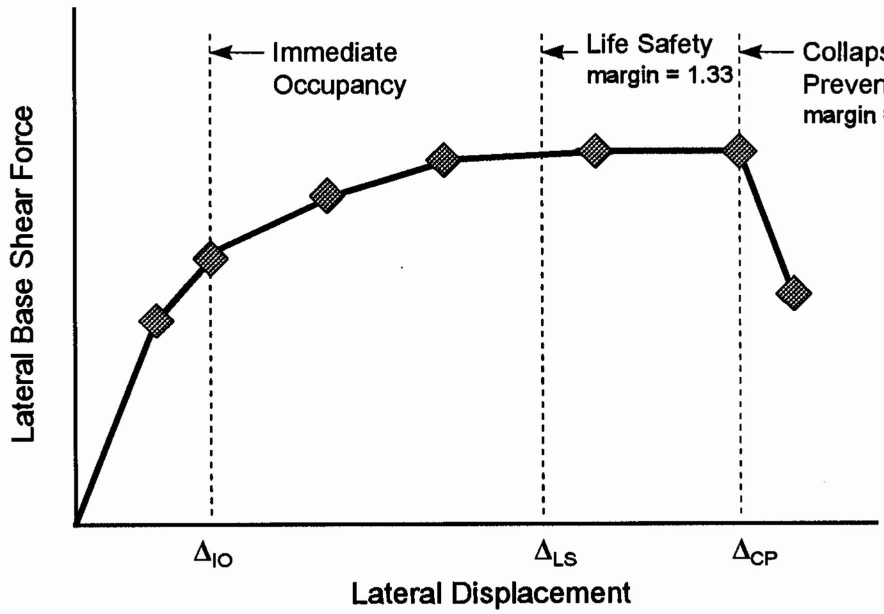

Figure 2 - FEMA-273 Performance Levels and Margins

STRUCTURAL RELIABILITY CONCEPTS

It is important to note that it is very difficult to predict a specific lateral displacement at which a structure will collapse, particularly in the case of earthquake induced collapse. There are a number of parameters that effect the displacement at which a structure will collapse. Some of these we can readily understand and predict, given the state of current engineering technology. For example, the amount of gravity load that a structure is supporting effects the amount of lateral displacement it can experience without developing P-Δ instabilities and collapsing. It is relatively easy for us to understand this relationship and also to predict the amount of gravity load a structure is supporting. There are other parameters that effect the likelihood of structural collapse at a given lateral displacement, the effect of which we can understand, but the actual values of which we have difficulty predicting. One such parameter is the yield or compressive strength of the actual materials present in the structure. We can understand that if the structural materials have relatively large strength, compared to the design values, that it will be less likely for the structure to collapse at a given displacement. However, without extensive testing of the properties of each of these materials, we have difficulty predicting what the actual strength of these materials in the structure is. There are still other parameters that affect the displacement at which a structure will collapse that we can neither understand nor predict, given today's technology and state of practice. As we continue to study the behavior of earthquakes and structures affected by them it is our hope that we can eventually understand all of the parameters that affect collapse and become better at predicting values for them.

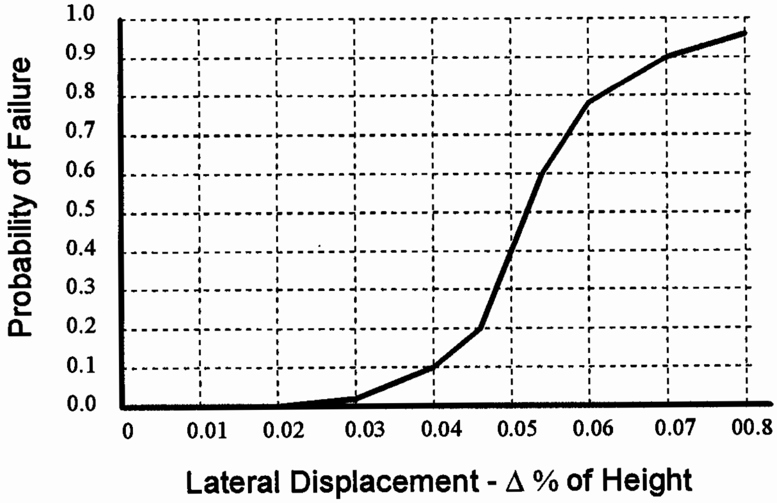

Figure 3 - Typical Structural Fragility

Since we do not currently have perfect knowledge of the parameters that control the displacement at which a structure is likely to collapse nor an ability to predict values for all of them, it is necessary to express the displacement at which collapse is likely to occur, Δ_c, in a probabilistic manner. In order to do this, reliability analysts typically use the concept of a fragility curve. A typical fragility curve is shown in Figure 3. As shown in the Figure, this curve expresses that at lateral displacements equal to approximately 2% of the story height, or less, there is a negligible probability of collapse for the structure represented by this curve while at lateral displacements equal to 8% of the story height, collapse is almost a certainty. This variation in the capacity of the structure to resist collapse as predicted by lateral displacement, Δ, in this case ranging from 2% to 8%, is a measure of our lack of understanding and inability to perfectly predict the behavior and results in what appears to be behavioral randomness.

This randomness has commonly been observed in past earthquakes. For example, it is not uncommon to find discussion in earthquake reconnaissance reports of cases of two or more apparently similar structures, located near each other, and apparently having experienced similar ground motion, but exhibiting markedly different post-earthquake damage states.

In the previous discussion our current inability to perfectly predict the lateral displacements at which collapse will be induced in a structure were discussed and expressed as an apparent randomness in structural behavior. In addition to our inability to perfectly predict the displacement at which a structure will collapse, we also have an inability to perfectly predict the lateral displacement that an earthquake will induce in a structure. Prediction of the ground motion characteristics that will result from even a specific magnitude earthquake along a fault located a known distance from the site has associated with it a great deal of uncertainty. Further, given an estimate of the ground motion response spectrum or even time histories from a given earthquake, there is an uncertainty, or potential error, associated with our ability to estimate the lateral displacement induced in the structure by this ground motion.

In structural reliability analysis, the uncertainties associated with our ability to perfectly predict the behavior of a structure are often dealt with by establishing levels of confidence around our prediction. Figure 3 presented a fragility function for a typical structure, in which the randomness of structural behavior was represented. Figure 4 presents an expanded fragility function for this structure in which both this randomness and also the uncertainty inherent in our ability to predict failure are expressed. The uncertainties are represented by a family of fragility curves respectively representing the confidence associated with our prediction of structural behavior. In the Figure, three levels of confidence are indicated. One curve, representing our best estimate of the failure probabilities for the structure is labeled as a median confidence level. Located above the median curve is a curve representing a 90% confidence level of non-exceedance. That is every point on the 90% curve indicates a probability of failure, at a given lateral displacement, that we are 90% confident will not be exceeded. Similarly, the lower curve represents a 10% confidence level of non-exceedance. Though curves representing three specific confidence levels have been shown in the Figure, in reality an continuous spectrum of curves representing different confidence levels could be drawn.

The effect of considering uncertainty in evaluation of structural fragility is to further increase the variation in behavior already resulting from randomness. In the earlier discussion, related to Figure 3, it was indicated that there was a negligible probability of collapse of the structure if it experienced an earthquake induced lateral displacement equal to 2% of its height. If we add in uncertainties, such as our ability to predict the lateral displacement induced in the structure, then Figure 4 tells us that once our prediction of lateral displacement demand exceeds about 1.5% of story height, there starts to be a non-negligible probability of collapse. Similarly, Figure 3 indicated a near certainty of collapse at a lateral displacement demand of 8% of story height, while Figure 4 indicates that a significant potential for collapse not to occur at predicted lateral displacement demands in excess of 8%.

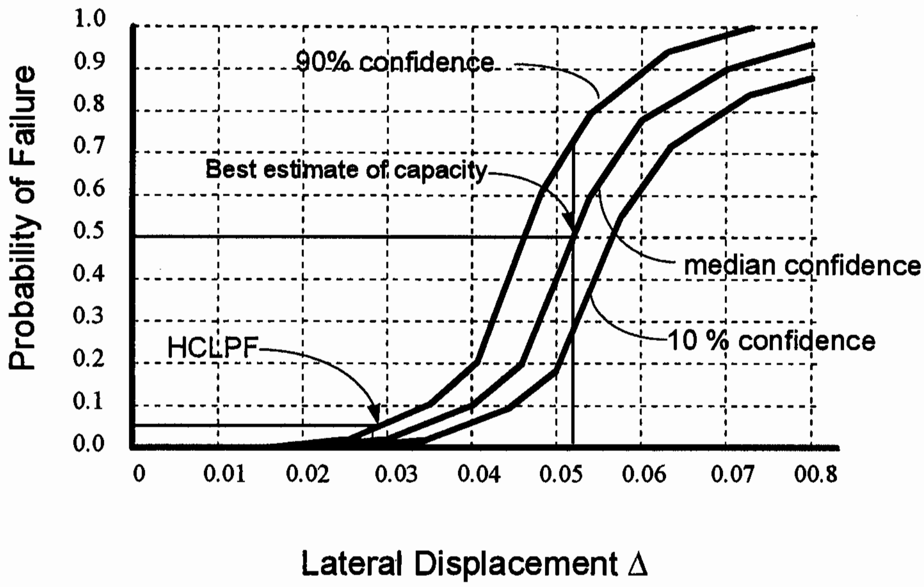

Figure 4 - Randomness and Uncertainty in Structural Fragility Evaluation

Two important points are represented in Figure 4. One of these occurs on the curve for a median confidence level, at a 50% probability of failure. This represents the best estimate of the lateral displacement demand at which collapse will occur, in this case, a lateral displacement of 5.2% of the structure's height. Although this may be our best estimate of the loading that would actually produce collapse, it would be an inappropriate point upon which to base a design since half of our structures would collapse if designed to this performance. The second point occurs at a somewhat arbitrarily selected 90% confidence level of less than a 5% chance of failure. This point, which in the figure has a value of 2.8% of structural height as a lateral displacement demand, is termed the point of High Confidence of Low Probability of Failure or HCLPF. The HCLPF is a more reasonable point upon which to base a design in that only a very few structures would actually be expected to fail when subjected to HCLPF loading.

In previous discussion, structural margin was described as the ratio of lateral drift demand at collapse, Δ_c, to that produced by a given design demand level, Δ_i. At this point, it is possible to redefine this margin as being the ratio of the HCLPF lateral drift demand to the demand, Δ_{HCLPF}, to that produced by a given design demand level, Δ_i. Although this concept was not specifically expressed in this manner, it was clearly considered in the development of the FEMA-273 *Guidelines* and accompanying *Commentary*. The *Commentary* notes that there are no guarantees that a structure rehabilitated using the procedures contained in the *Guidelines* will actually meet the expected performance levels when subjected to the design earthquake ground motions and that perhaps, one or two such structures out of every hundred, may fail to meet the expected performance.

The randomness and uncertainties inherent in the prediction of structural behavior are specifically dealt with in the *Guidelines*. Uncertainty is evaluated with the use of adjustment factors, selected based on the type of analysis performed and the engineer's knowledge of the way in which the structure was actually constructed. Randomness is evaluated in the selection of ground motion loading criteria and in the evaluation of component capacities. These issues are discussed in the following sections.

STRUCTURAL ANALYSIS

Once a target performance level has been selected for a structure and a preliminary design developed, the Guidelines require that an analysis be performed to determine the structural demands and their acceptability. Four analytical procedures are contained within the Guidelines. These include a linear static procedure (LSP), a linear dynamic procedure (LDP), a nonlinear static procedure (NSP) and a nonlinear dynamic procedure (NDP).

Linear Static Procedure. The LSP is the least accurate of the four procedures, and therefore, introduces the greatest amount of uncertainty into the process of predicting performance. It is also the simplest procedure. It was developed to provide a counterpart to the equivalent lateral force (ELF) procedure commonly used by designers of new buildings under the code provisions. Though the LSP resembles the ELF procedure in many ways, there are significant differences. Both the ELF and LSP are simplified applications of the response spectrum analysis technique in which the dynamic properties of the structure (structural period, mode shape, modal participation) are taken based on simple default values rather than being calculated through rigorous analysis. However, while the ELF is used to proportion structures with a minimum lateral force-resisting strength, the LSP is used to predict the amount of lateral displacement demand that the structure will experience from a design earthquake motion.

The LSP starts with estimation of the structural period, T, for the building. This may be calculated using principles of structural mechanics, such as the Rayleigh equation, by performing a dynamic analysis, or by using simple approximate period formulae, similar to those contained in the building code. Once the building's period has been estimated, the spectral acceleration, from a 5% damped acceleration response spectrum, S_a, is obtained. If real buildings were simple single degree of freedom systems, it would be possible to obtain the spectral displacement, and consequently, the lateral displacement demand directly from the known values of the spectral response acceleration, S_a and the structural period, T, using the relationship:

$$S_d = \frac{T^2}{4\pi^2} S_a \tag{1}$$

Then, it would be possible to adjust the calculated spectral displacement demand, which represents the response the structure would experience if it remained elastic, to the displacement that it will experience when it exhibits inelastic behavior. This adjustment is possible because over the years, a number of researchers have performed a series of analyses in which they have compared the predicted response for a simple, elastic, single degree of freedom system, to that obtained from more complex models using nonlinear time history analyses. These studies have permitted the development of a series of correlation coefficients that relate the displacement predicted by elastic response spectrum analysis to that predicted by the inelastic time history analyses. These coefficients are a function of the modal properties of the structure (period, and mode shape), the effective damping inherent in the structure (a function of the ductility demand

and the fullness of the hysteretic loops for the structure), and the extent to which strength degrades with increasing repeated cyclic motion. To account for these effects, FEMA-273 adopts a modified form of equation (1) expressed as follows:

$$\Delta = C_0 C_1 C_2 C_3 \frac{T^2}{4\pi^2} S_a \qquad (2)$$

In equation (2), Δ is the lateral drift demand induced in the structure by the ground motion, C_0 is a coefficient to adjust spectral displacement to structural displacement, based on the modal shape properties of the structure; C_1 is a correlation coefficient that relates the response of a perfectly elastic structure, to a structure that has elastic-perfectly plastic inelastic response characteristics; C_2 is a correlation coefficient that adjusts the estimated inelastic displacement demand for the effects of pinching in the hysteretic behavior of the structure; and C_3 is a correlation coefficient that adjusts the inelastic displacement demand for the effects of strength degradation in the structure. These effects are illustrated in Figure 5.

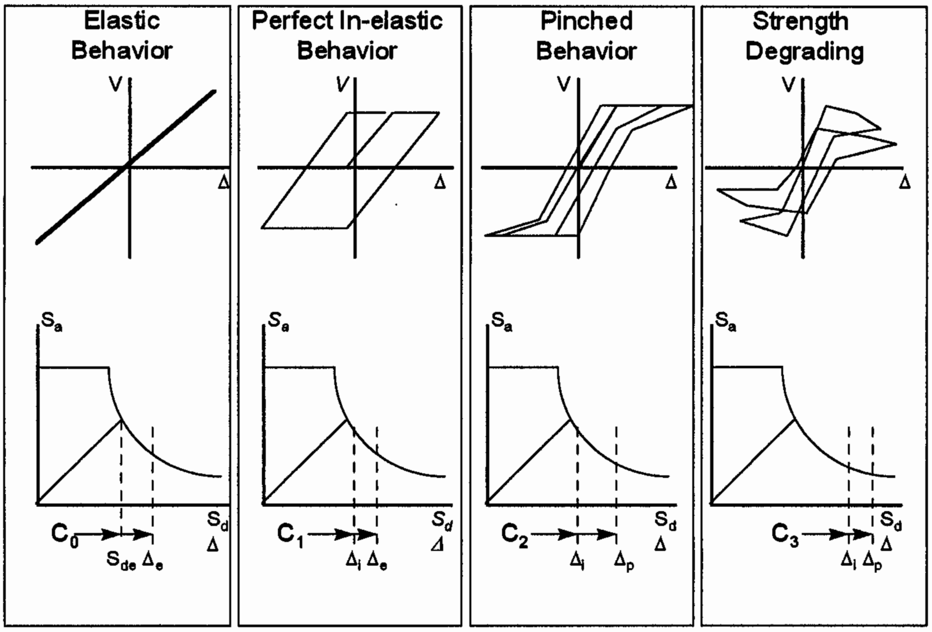

Figure 5 - Correlation of Elastic and Inelastic Behaviors

Structures are complex assemblies of components, including walls, diaphragms, braces, frames, etc., each of which has different capacity to resist earthquake induced lateral displacement. This assembly is made even more complex when new components, provided to increase the building's earthquake resistance are added, since these will typically be detailed in accordance with contemporary practice and will have often have significantly larger lateral displacement capacity than do the brittle elements of many existing buildings. Therefore, the use of a single lateral displacement, Δ, as implied by equation (2) was found not to be useful for real existing buildings. Instead it was decided to evaluate structural performance based on the effects of lateral displacement demand on the individual elements.

In order to do this, an equivalent lateral force, V, is calculated, which when applied to an elastic model of the structure, will provide the same displacement in the structure as predicted by equation (2). This force is calculated from the equation:

$$V = C_0 C_1 C_2 C_3 S_a W \tag{3}$$

where the terms C_0, C_1, C_2, C_3, and S_a have the same meaning and use as in equation (2), and W is the effective weight of the structure. This equivalent lateral force is applied to a mathematical model of the structure, which is then analyzed for static response under this force to determine the total lateral displacement as well as the individual forces and deformations on the various structural components. Two basic assumptions are made at this point. One is that the distribution of deformation demands in the real structure, which is behaving in an inelastic manner is the same as the distribution of deformation demands predicted by this elastic static analysis. For structures that are reasonably regular, and in which the inelastic behavior is distributed throughout the structure, this is a reasonable assumption. The second assumption is that the ratio of the force calculated in each of the structural components from this elastic, static analysis, to the yield capacity of the component, can be used as a direct measure of the inelastic ductility demand on the component. The accuracy of this assumption was not explored in depth by the developers of the Guidelines, but was felt to be a necessary simplification for the LSP.

With the above two assumptions in place, the ability of a design to satisfy the intended performance is judged by comparing the computed force demands on each structural component to a permissible strength capacity. For components capable of significant ductility, the permissible strength capacity is taken as a value that is greater, by a factor m, than the expected yield strength of the component. The *Guidelines* provide tabulated values for these m factors, for different types of structural components and for each of the various performance levels. For components that are not capable of significant ductility and instead are subject to brittle failure if excessively loaded, the forces predicted by the linear analysis are adjusted, by a coefficient C_4, to account for the fact that forces in a structure that responds in-elastically are lower than those in a structure of similar stiffness but that has adequate strength to respond elastically. These adjusted forces are then evaluated against a lower-bound estimate of the probable strength of the component. Further discussion of these issues is contained in the discussion on acceptance criteria, below.

If a structure is very irregular and has large ductility demands, the use of the LSP introduces a great deal of uncertainty into the prediction of demands on the various components. Also, since the LSP is based on the response of a single degree of freedom system, it can be quite inaccurate (introducing uncertainty) when the response of the structure has significant participation by higher modes. Therefore, the Guidelines prohibit the use of the LSP in highly irregular structures, subjected to large ductility demands and strongly recommend that it not be used for structures with significant higher mode participation.

Linear Dynamic Procedure. The LDP is very similar to the LSP, except that instead of performing an elastic static analysis, using lateral forces intended to represent the inertial forces that would be calculated in a dynamic analysis, a dynamic analysis is actually performed. The dynamic analysis can consist either of an elastic response spectrum analysis or of a suite of elastic time history analyses. In a manner similar to the building code, if time history analyses are used at least three analyses, using different time histories, scaled to match the design response spectrum must be performed, and the maximum of each of the member forces and structural displacements calculated from the three analyses are used to check acceptability of performance. If as many as seven analyses are performed, using different time histories scaled to match the design spectrum, then the average of the member forces and structural displacements predicted by the suite of analyses may be used to evaluate design acceptability. This approach is taken as a means of dealing with the uncertainty inherent in the use of any single ground motion record to represent the effects of a future earthquake. Due to the complexity involved in performing these multiple analyses, as well as the difficulty of reviewing and interpreting the results of a time history analysis, most designs employing the LDP will use the response spectrum method of analysis.

Just as with the LSP, when an LDP is performed it is necessary to adjust the results of the analysis to correlate with the probable behavior of the structure when it behaves in-elastically. Therefore, the member forces and structural displacements predicted from the elastic dynamic analysis are scaled by the combined term $C_1C_2C_3$, with each of these coefficients having the same value and same intent previously described for the LSP. It is not necessary to modify the results of an LDP by the coefficient C_0 because the effects of modal participation are directly accounted for by the dynamic analysis. The acceptability of structural performance is evaluated in the same way previously described for the LSP.

Nonlinear Static Procedure. The NSP is a type of analysis commonly known as a push-over analysis. It is intended to reduce the uncertainty inherent in both of the linear analysis procedures, by directly accounting for the non-linear behavior of the structure. It is performed as a series of linear static procedures, using sequentially modified versions of the mathematical structural model to represent the degradation in stiffness that the structure experiences as it is subjected to larger lateral deformations and becomes damaged. In the process of performing these analyses, a push-over curve, similar to those shown in Figures 1 and 2 is plotted for the structure, allowing direct evaluation of the structure's performance.

To perform an NSP a mathematical model of the structure is developed. The model must include the effects (stiffness and strength) of all elements that are significant to the behavior of the building. An element is significant if it contributes a significant amount of stiffness or strength to the overall structure, or, if it is likely to be damaged as a result of earthquake induced lateral deformation of the structure. This initial structural model is analyzed for an arbitrary static lateral force. The forces induced in each member of the model due to this lateral force is determined and compared against a best-estimate of the yield capacity of the various members, to provide demand-capacity ratios (DCRs). These DCRs are reviewed to determine the largest value. The member with the largest DCR is the one that will first experience damage or yielding, when the structure is subjected to an increasing lateral displacement. The entire analysis is scaled by a factor such that the largest value of the computed DCRs for the members has a value of 1.0, indicating that at the lateral displacement predicted by this scaled analysis, yielding in the structure initiates. The forces in each member are at this load step are recorded and the value of the total applied lateral force and corresponding displacement are plotted as the first damage event on the pushover curve.

To obtain the next point on the pushover curve, the mathematical model is modified to reduce the stiffness of the member that has yielded to an appropriate post-yield value. Again, a static lateral force is applied to this modified mathematical model and an analysis is performed to determine the various member forces. These demand forces are compared to the residual capacities of each of the various members. The residual capacities are simply obtained by subtracting from the initial member capacity, the demand force obtained at the previous load step. DCRs are again computed for all of the members, using these residual capacities. Again, a scale factor is determined such that the largest DCR from this analysis will have a value of 1.0. The analysis is then scaled by this factor and the resulting forces and displacements from the scaled analysis are added to those obtained from the first analysis step. The total applied lateral force (force in the first scaled analysis plus the force in the second scaled analysis) and the total lateral displacement (lateral displacement form the first scaled analysis plus the lateral displacement from the second scaled analysis) are then plotted as the second damage event on the push-over curve.

This entire process is repeated, with the mathematical model revised each time to represent the reduction in stiffness that occurs with each successive damage event until it is found that the structure has become unstable, predicting the onset of collapse. This pushover curve represents one estimate of the capacity of the structure. One of the uncertainties inherent in this analysis approach is that the actual pattern of inertial forces, that will drive the real structure during an earthquake are being estimated. In order to bound the effects of this uncertainty, the Guidelines specify that when the NSP is used, two independent analyses should be performed, each using a different assumption as to the vertical distribution of lateral forces in the structure. One loading pattern is the inverted triangular distribution of forces, used in the ELF procedure in the building codes. The second pattern is that which results from a uniform inertial acceleration of the structure (rectangular distribution of loading). The triangular distribution is a reasonable approximation of the lateral forces that will occur when a building is responding in the elastic range, while the rectangular distribution is representative of the loading that will occur on a structure that has formed an inelastic soft story. Together, these analyses should bound the capacity of the structure.

The next step is to predict the demand produced by the design earthquake. In order to do this, the pushover curve obtained from the analyses are represented by an equivalent elastic-perfectly plastic curve. The stiffness of the elastic portion of this equivalent curve is obtained by drawing a line from the origin of the curve, through the point on the pushover curve that occurs at a lateral force that is 60% of the maximum lateral force obtained in the analysis. This is illustrated in Figure 6. The slope of the elastic portion of the idealized pushover curve is next determined. This slope represents the stiffness of an idealized single degree of freedom structure, having dynamic characteristics similar to that of the structure. The period of this idealized structure, T_e, is determined from the equation:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \qquad (4)$$

where T_i is the initial elastic period of the actual structure, K_i is the initial elastic stiffness of the actual structure, obtained as the slope of the initial segment of the push-over curve and K_e is the effective stiffness of the idealized elastic-perfectly plastic structure, as shown in Figure 6.

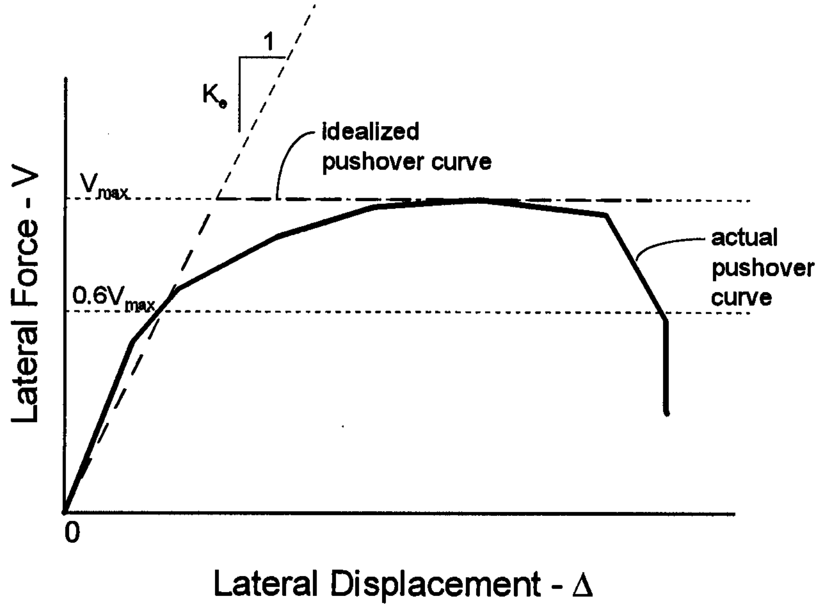

Figure 6 - Use of Pushover Curve to Determine Demand

Once the effective period, T_e, for the structure has been determined, the displacement demand is determined from equation (2), substituting T_e for T and evaluating S_a at period T_e. The coefficients C_0 and C_1 are evaluated in the same manner as for the linear procedures. The coefficients C_2 and C_3 are determined using the actual characteristics of the pushover curve.

Following determination of the demand, Δ, termed the target displacement in the *Guidelines*, it is necessary to determine the acceptability of the structural performance. This is performed by comparing the actual deformation demands for the various members at lateral displacement Δ, against the acceptance criteria presented in the *Guidelines* for the various performance levels.

The NSP entails a significant reduction in uncertainty, with regard to the prediction of building behavior, than do either of the linear procedures, except when the effects of higher modes are significant to the structure. If higher mode effects are significant, the NSP can introduce a significant error into the estimated performance. Therefore, for structures with significant higher mode response, the NSP is not recommended unless an LDP is also performed. Alternatively, an NDP analysis can be performed.

Nonlinear Dynamic Analysis. A nonlinear dynamic analysis is potentially the most accurate (least uncertain) of all of the various analysis procedures contained in the *Guidelines*. In this procedure, a non-linear mathematical model of the structure is developed and subjected to a series of time-history analyses. To the extent that it is possible to accurately model the building and the ground motion, many of the analytical uncertainties inherent in the other analysis procedures are eliminated with this approach. However, in reality it is not practical at the current time to reliably implement this approach for complex structures. There are currently few consensus models available to represent the behavior of the many of types elements that are found in our existing structures. Since relatively small changes in the way elements are modeled can have significant effect on the results predicted by such analyses, use of this technique can create the impression of highly certain results without actually providing such accuracy. Therefore, the Guidelines recommend that when this approach is employed, independent third party review of the assumptions and approach should be performed by a knowledgeable expert. As with the LDP, suites of several different ground motion time histories should be used in the analyses to account for the variation and uncertainty inherent in the prediction of the ground motion. Analysts employing this approach should also consider running sensitivity studies and suites of analyses to explore the uncertainty inherent in various modeling assumptions employed in the analysis.

When the NDP is employed, the analysis directly predicts maximum values of member forces and deformation demands. These are compared against the acceptance criteria provided in the *Guidelines* to evaluate the predicted performance of the structure.

ACCEPTANCE CRITERIA

The acceptability of structural performance is evaluated on a structural component basis. In the *Guidelines*, structural components are classified as being force controlled or deformation controlled. Deformation controlled components are those that have significant ductility while force controlled components do not. In order to develop acceptance criteria for the document, an extensive literature search was conducted to determine typical hysteretic curves for the various types of structural components common in our existing buildings, based on past research. Where research data could be found for an element type, it was represented by a "backbone" curve. The backbone curve, illustrated in Figure 7, is an envelope of the hysteretic behavior of the component, taking into account the degradation that occurs under repeated cycles of motion to the same displacement level.

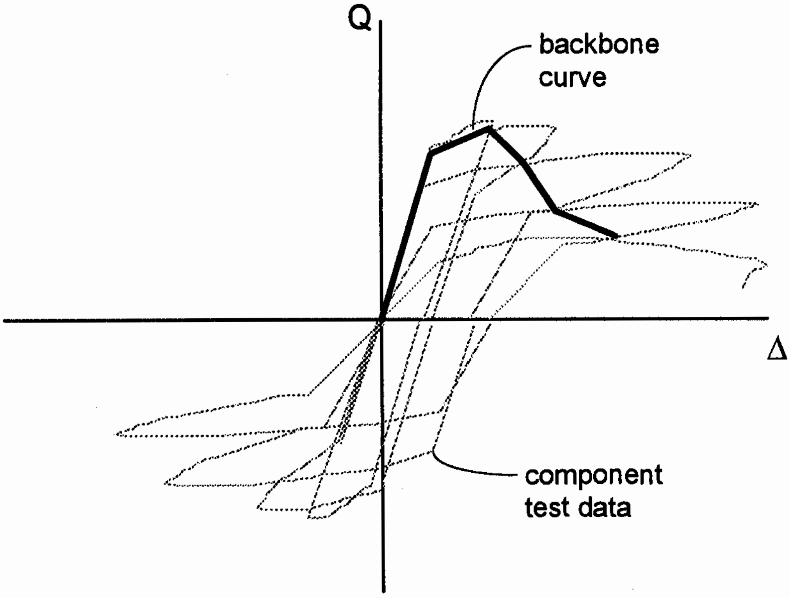

Figure 7 - Development of Backbone Curve from Hysteretic Data

Each of the backbone curves obtained from the available research data were represented by one of three idealized shapes, represented in Figure 8. The type 1 curve represents a ductile behavior in which there is an elastic range (points 0-1), a plastic range (points 1-2), and a strength degrading range (points 2-3). An example of a structural component that exhibits this type of behavior are steel beam-column connections with partially restrained joints. The Type 2 curve represents components with ductile behavior but a rapid degradation of strength after a limiting ductility is reached. Steel braces loaded in compression would be represented by such behavior. The type 3 curve represents a non-ductile or brittle behavior, such as may occur in an unconfined concrete element beam or column. Where specific research data could not be found on the behavior of a particular type of structural component, the developers of the *Guidelines* developed a "best-estimate" of such behavior based on judgment, thus introducing another source of uncertainty in the document.

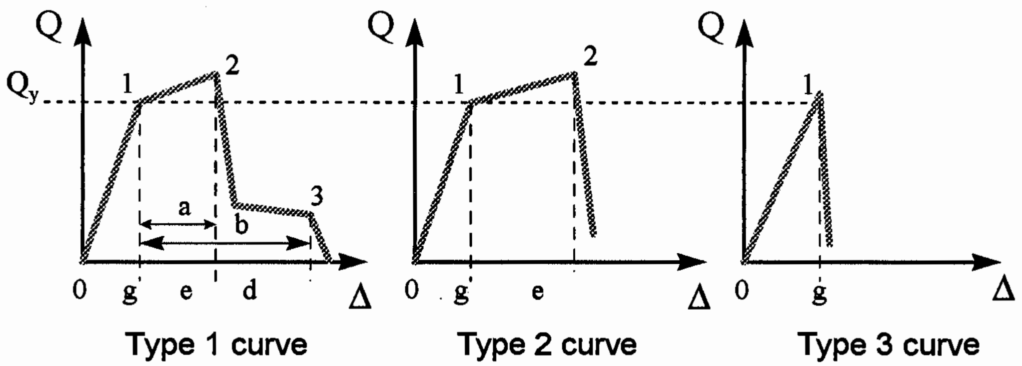

Figure 8 - Idealized Backbone Curves

The acceptance criteria (permissible forces and deformations) for the various structural components and performance levels were determined from these idealized backbone curves. As stated earlier, for Collapse Prevention performance, the intent is that there be a margin of 1.0 against failure. Therefore, for deformation controlled components, the acceptance criteria is that the predicted member deformation not exceed point "2" on the corresponding backbone curve. For force controlled components, the computed force on the component can not exceed a lower bound estimate of the yield (or ultimate) capacity of the component. For Life Safety performance, the desired margin has a value of 1.33. Therefore, the acceptance criteria for Life Safety performance is that the member deformation for deformation controlled elements not exceed 75% (1/1.33) times the deformation at point "2" of the corresponding backbone curves. For force controlled components, the total force on the component can not exceed 75% of a lower bound estimate of the yield (or ultimate) strength of the component. These permissible values are tabulated in the Guidelines, categorized by component type.

When either an NSP or NDP is performed, the component deformation demands and forces predicted by the analyses can directly be compared against the acceptance criteria obtained from the backbone curves and discussed above. However, when an LSP or LDP is performed, the analysis does not directly predict component deformation demands. Instead the analysis provides estimates of member forces, which often will exceed the member yield strength. In order to determine the acceptability of the behavior of deformation controlled components, the following equation is used when a linear procedure has been performed:

$$Q \leq m\kappa Q_{CE} \tag{5}$$

In equation (5), Q is the value of the total computed strength demand on the component from the analysis, including the effects of dead and live loads. Q_{CE} is the expected, or median, value of the yield strength of the component, Q_y in Figure 8. The coefficient κ is intended to reduce the permissible capacity of the structure to account for uncertainty resulting from a lack of knowledge as to how the structure is actually constructed. It has a minimum value of 0.75 when relatively little knowledge is available as to the construction of the structure to a maximum value of 1.0 when extensive knowledge of the structure's construction has been obtained. The coefficient "m" is a ductility coefficient, obtained from the equation:

$$m = 0.75\delta_i \tag{6}$$

where δ_i is the acceptable deformation for the component and performance level if a nonlinear analysis procedure is used, as has previously been described. The purpose of the 0.75 factor in equation (6) is to account for the additional uncertainty inherent in the prediction of component demands when a linear analysis procedure is used as opposed to a nonlinear procedure and also to account for the uncertainty inherent in the assumptions that a demand/capacity ratio computed on the basis of forces is an accurate representation of inelastic deformation demands.

For force controlled components, acceptance is evaluated using the equations:

$$Q_G + \frac{Q_E}{C_0 C_1 C_2 C_3 C_4} \leq Q_{CL} \tag{7}$$

In the above equation Q_G is the force in the component due to gravity loads, Q_E is the computed force due to earthquake demands and the coefficient C_0 through C_4 have the meaning previously described. The quantity on the left side of equation (7) is intended to represent a best estimate of the actual force that can be delivered to a brittle component by the structure, considering its inelastic behavior. This is a very approximate estimation with a significant inherent uncertainty. Q_{CL} is a lower bound estimate, taken at a mean minus one standard deviation level, of the yield strength of the component.

THE NEED FOR VERIFICATION

The *Guidelines* represent a significant advancement in the state of practice for earthquake-resistant design of structures. The procedures take into account the actual nonlinear behavior of building structures, in a far more realistic manner than do the building codes, and also attempt to directly account for the randomness and uncertainties inherent in the evaluation process. However, because the *Guidelines* represent such a significant departure from past practice and also because the uncertainties have been accounted for in only a very approximate way, there is a real need to perform verification of the ability of the procedures to actually predict structural performance within the variance previously discussed, i.e., the expectation that when subjected to the design ground motions only 1% to 2% of the affected structures would experience worse performance than predicted.

The uncertainties inherent in the prediction of performance by the procedures contained in the *Guidelines* can be attributed to a number of different sources. Principal among these are our inability to accurately predict the character of ground motion that the structure will experience and our inability to accurately model the behavior of the structure when subjected to such ground motion. There is also significant uncertainty with regard to our ability to predict the capacity of components and of the structure itself. In addition to these uncertainties, there is a considerable apparent randomness in both the demand produced on structures and the capacity of structures to resist these demands. In developing the *Guidelines*, a conscious attempt was made to bound this randomness and these uncertainties by attempting to estimate the biases inherent in the various assumptions made in structural modeling and capacity evaluations and also by estimating the levels of confidence associated with various analytical procedures. However, these treatments of the variation were largely performed in a judgmental and qualitative manner, rather than as a result of rigorous reliability investigations. As a result, the procedures may currently contain significant biases, meaning that they may either significantly under-predict or over-predict the performance of buildings in a systematic matter.

If the *Guidelines* systematically over-predict building performance, than an undesirably large number of buildings rehabilitated in accordance with the *Guidelines* will fail to meet the intended performance objectives. Conversely, if the *Guidelines* systematically under-predict performance, than rehabilitation programs designed in accordance with the *Guidelines* will not be optimal in terms of the amount of rehabilitation work that is performed. Neither condition is particularly desirable. If actual performance is poorer than predicted, than a number of buildings may remain hazardous despite being rehabilitated. If the actual performance is better than predicted, then the excessive cost of implementing the *Guidelines*, relative to a more optimal approach will result in the mitigation of relatively fewer structures, and again a greater than desired risk. Clearly there is a need to verify the reliability actually provided by the procedures, with regard to the intended goals.

The use of strong motion data to test and calibrate the *Guidelines* is an essential part of this verification process. Ideally, in order to perform this verification there is a need for strong motion data from arrays that include both free-field and in-structure response recordings for a large number and wide variety of building types and site conditions, together with detailed documentation of the actual performance of the buildings from which this data is obtained. This data, if made available to the structural engineering community, will permit investigators to compare the performance predicted by the *Guidelines* to that actually experienced by real structures. It will also enable the accuracy and variation inherent in each of the individual steps followed in the *Guidelines* to be evaluated. In its simplest form, a comparison of the strong motion data with predicted response obtained from structural analyses can be used to determine the randomness and uncertainty introduced by the four analysis techniques with regard to prediction of lateral displacements. Such investigations can also be used as the basis for modifying the analytical technique so as to reduce the inherent variations as well as to determine the adequacy of limitations currently placed on each analysis technique based on the apparent limitations of reliability.

Strong motion data can also be used to verify the adequacy of acceptance criteria contained in the *Guidelines*. The acceptance criteria are based on values of acceptable deformation in various structural components. Where hysteretic data from laboratory investigations was available, it was used as the basis for determining acceptance criteria. Unfortunately, no data was available for many of the types of structural components commonly found in existing buildings. Consequently, the developers of the Guidelines were forced to rely on estimates of the probable behavior rather than actual laboratory data. To the extent that strong motion data permits the cyclic deformation demands on elements of real instrumented buildings to be understood, the behavior of these elements in response to these demands can serve to greatly supplement the existing laboratory data and provide a method to benchmark and adjust the acceptance criteria as appropriate.

It is important to note that a large number of analyses of buildings for which strong motion data is obtained must be performed to be useful in this verification process. As previously discussed, the prediction of structural capacity to resist earthquake ground motion inherently includes great variation and apparent randomness. Therefore the ability of a few strong motion data sets to either confirm or apparently call to question the validity of the *Guidelines*, is not by itself meaningful. It is anticipated that some buildings will not perform as well as predicted by the Guidelines and that many buildings will perform better than predicted. In order to provide true verification, it will be necessary to perform a statistically significant series of analyses of buildings for which strong motion data is available so that relative reliability of the *Guidelines* can be appropriately judged.

SUMMARY

FEMA-273/274, *Guidelines and Commentary for Seismic Rehabilitation of Buildings*, is a landmark document that breaks new ground in the practice of performance-based seismic engineering of buildings. It employs new analytical techniques that are far more rational than those employed in standard building code procedures as well as acceptance criteria tied directly to laboratory test data on typical components. However, the complexity of this document as well as the radical extent to which it has departed from previous structural engineering practice suggests that it is likely to need considerable verification and adjustment in order to provide the desired level of economy in structural rehabilitation projects while maintaining acceptable levels of reliability. Application of the *Guidelines* to instrumented buildings from which actual strong motion data has been obtained should be a key part of this verification and adjustment process.

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