Phase I Research Summary

1.1 Methodology

Input was sought from investigators on relevant published research as well as on research in progress and unpublished insights and perspectives. This chapter summarizes the information obtained from these investigators and a partial review of relevant literature in relation to the issues identified by the Working Group.

Various mechanisms were relied upon to obtain the information presented. Announcements in professional newsletters informed the engineering community of the project and directed interested persons to the ATC web site, where the project is described and a form for researcher input is available. Individually addressed email was sent to researchers identified by the Working Group soliciting input by way of meetings, telephone interviews, or in written form. Members of the Working Group met with investigators in face-to-face meetings at locations across the country from March through June 2001. Telephone interviews and email discussions were used when meetings could not be arranged. In all, over 50 researchers in the United States, Europe, and Japan were individually contacted, and over 30 provided input through many contact hours of interviews that took place. This was supplemented by email exchanges and discussions and presentations that took place at a recent workshop on performance-based earthquake engineering[1]. Most of the researchers that responded to our requests for input are in structural engineering or risk and reliability; the disciplines of geotechnical engineering and engineering seismology are not represented as well.

The summary that follows is a composite of information submitted by researchers or obtained from interviews and information obtained from published and pending articles and reports (listed in the References section). A summary addresses the main issues identified by the Working Group and identifies general areas of agreement among researchers.

1.2 Classification of Analysis Methods

This report focuses on various methods of inelastic analysis. Although other methods exist, emphasis is given to the nonlinear static procedures known as the Capacity Spectrum Method, as described in ATC-40 (1996), and the Displacement Coefficient Method, as described in FEMA-273 (1997) and FEMA-356 (2000).

The various inelastic analysis methods can be categorized based on

(1) the approximations used to model the structural system ("equivalent" SDOF (considering one or more modes), stick models, "fishbone" models, 2D planar models, and 3D models), and

[1] Working Group A of the Third US-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforced Concrete Building Structures, held in Seattle, Washington, August 16-18, 2001.

(2) the technique used to represent or estimate demands (elastic spectra, approximate inelastic spectra, one or more elastic or nonlinear time histories, or energy-based methods).

Both the Capacity Spectrum Method and Displacement Coefficient Method use "equivalent" SDOF systems to estimate the response of an inelastic system based on the response of elastic systems, with response of the elastic systems represented by smooth elastic design spectra. The techniques used in these methods for estimating response do not require that spectrum compatible ground motions be determined.

MacRae and Maffei observe that methods for estimating the response of inelastic systems based on elastic response spectra may be classified based on use of

(1) the initial elastic period and corresponding elastic spectral displacement. (used with lateral force reduction factors, R-μ-T relations, and the Displacement Coefficient Method), or

(2) the secant stiffness period and corresponding elastic spectral displacement (used with the Capacity Spectrum Method and other equivalent linearization techniques)

The relationship between the initial and secant stiffness of an oscillator can be expressed as a function of the ductility response. The critical distinction between these methods is not whether an oscillator is referred to by its initial or secant stiffness, but whether the inelastic displacement estimate is based on the response of an elastic oscillator having the initial or secant stiffness period. Displacement estimates made with the Displacement Coefficient Method use the initial stiffness together with R-μ-T relations to estimate the response of the inelastic oscillator, while displacement estimates made with the Capacity Spectrum Method are based on the peak response of an elastic system whose stiffness is equal to the secant stiffness of the yielding oscillator. This distinction is important because the graphic presentation of a method can be separated from the underlying relationships used to estimate response. Inelastic spectra (computed based on R-μ-T relations) can be displayed in a Capacity Spectrum format to allow the secant stiffness to be used, while the variant of the Capacity Spectrum Method known as Yield Point Spectra allows the initial stiffness to be used with the same R-μ-T relations. In both cases, the underlying displacement estimate is based on the initial stiffness, because the R-μ-T relationships were formulated based on the initial stiffness.

1.3 Nonlinear Static Procedures

1.3.1 Description of Procedures

1.3.1.1 Capacity Spectrum Method

Description: The peak displacement a nonlinear system is estimated to be at the intersection of the capacity curve and an elastic response spectrum that is reduced to account for energy dissipated by the yielding structure. The underlying basis of the Capacity Spectrum Method (CSM) is the concept of an "equivalent linear" system, wherein a linear system having reduced stiffness and increased damping is used to estimate the response of the nonlinear system.

Advantages: Several appealing features of the CSM were identified: (1) the intersection of "capacity" and "demand" curves suggests a form of dynamic equilibrium is being addressed by the procedure; (2) the influence of strength and stiffness on peak displacement is clearly represented by the graphic nature of the procedure, (3) the equating of hysteretic energy dissipation to viscous damping energy dissipation provides an appealing (though somewhat specious) theoretical basis that suggests the method is based on fundamental relations.

Drawbacks: Some drawbacks to the Capacity Spectrum Method are: (1) ongoing questions relating to the accuracy of displacement estimates, both for far-field motions and for near-field motions, (2) the iterative procedure is time-consuming and may sometimes lead to no solution or multiple solutions (Chopra), and (3) uncertainty related to whether the method can be applied to site specific spectra that depart from the standard spectral shape (Iwan).

Accuracy: Some investigators find the CSM overestimates displacement response while others find the CSM underestimates displacement response. Albanesi et al. (2000) find significant disparities between estimates of response made with the CSM, equal energy, and equal displacement assumptions and the results from nonlinear time history analyses. Chopra and Goel (1999, 2000) report that this procedure significantly underestimates displacement response for a wide range of periods and ductility values, relative to the Newmark Hall and other R-μ-T relations. Tsopelas (1997) finds that the CSM either accurately estimates or overestimates the mean displacements obtained from nonlinear time history analysis. MacRae observes that the CSM overestimates the effective damping for a given ductility and thus reduces the 5% damped spectrum too much for a given level of damping. Iwan reports that the CSM is not inaccurate in a mean sense, but the scatter in displacement estimates is large because the combination of effective stiffness and damping used in the CSM is not optimal. Iwan et al (2000) report that the use of equivalent viscous damping to account for inelastic behavior in the CSM yields satisfactory results for the limited period ranges where a resonance build-up type of response occurs, and that the CSM is not generally valid for near-field ground motions. Akkar and Gulkan (2000) report that the CSM underestimates response to near field earthquakes. Freeman notes the intersection point is least ambiguous when the capacity and demand curve form a sharp intersection; where the curves approach each other gradually, the expected displacement may be less certain. The range of findings on the accuracy of the CSM may reflect the various methodologies used in the studies—the method is intended to apply to a smooth elastic response spectrum, but often is assessed using actual records that have jagged response spectra. The assessment of the accuracy of the method is likely to vary with the ground motions used to evaluate it, with clear differences emerging, for example, for near-field motions relative to far-field motions.

Theory: The sense of scientific theory resulting from equating hysteretic and viscous-damping energies is both a strength and a weakness of the method. Section 2.3.2.1 discusses several of the theoretical bases that have been used in equivalent linearization. The need for empirically-determined coefficients (e.g. to account for structure framing type (Valley)) adds an element of empiricism to the method. The use of spectral reduction factors to be applied to a designated spectral shape makes it unclear if the method is even applicable to site specific spectra that depart from the designated spectral shape (Iwan).

Enhancements: Improvements to the implementation of the CSM have been suggested by some investigators. For example, Albanesi et al. (2000) suggest the use of a variable damping response spectrum, in which the damping level increases as the ductility of the system increases. Potential enhancements to the method involve using recalibrated ductility-damping-spectral reduction factor relationships or, more directly, ductility-spectral reduction factor relationships in the so-called Direct Capacity Spectra Method (MacRae and Tagawa, 2001), the use of inelastic spectra based on R-μ-T relationships, using the same graphic presentation of the Capacity Spectrum Method (Chopra and Fajfar), or the use of inelastic spectra based on R-μ-T relationships, plotted with yield displacement on the abscissa, in a format known as Yield Point Spectra (Aschheim).

The underlying relationships are discussed in Sections 2.3.2.1 and 2.3.2.2. Direct Displacement Based Design is a design version of the CSM (Kowalsky) and is discussed in Section 2.3.1.4

1.3.1.2 Displacement Coefficient Method

Description: The peak displacement of a nonlinear system is estimated as the peak displacement of an elastic system multiplied by a series of coefficients. Of primary interest here are the coefficients C_1= the ratio of the peak displacement of the inelastic system and the peak displacement of the elastic system having the same period of vibration; C_2, which accounts for the effect of pinching in the load-deformation relation; and C_3, which accounts for second-order (P-Delta) effects. These coefficients can be determined empirically from studies of the response of SDOF oscillators. In particular, C_1 can be derived from R-μ-T relationships. The coefficient C_1 equals 1 for long period systems, representing the equal displacement approximation. In the Displacement Coefficient Method (DCM), stiffness has a large effect on peak displacement response, while strength has little effect on the displacement estimates for short period systems and no effect on the displacement estimates for long period systems, as long as the system is not too weak.

Advantages: Some advantages of the method are that (1) it is direct and simple to apply, and (2) it is based on R-μ-T relationships that are generally accepted.

Drawbacks: The Displacement Coefficient Method has received little attention in the literature, relative to the Capacity Spectrum Method. Thus, potential drawbacks of the method may not be as apparent. The product formulation for representing the effects of strength, pinching, and P-Delta effects may be questioned. Krawinkler stated that P-Delta effects in multistory structures can not be accounted for accurately using simplified procedures, and realistic spectra should be used for soft soil sites, rather than using a coefficient approach.

Accuracy: Many compromises were required to transform research results into the FEMA-273 nonlinear static procedure (Krawinkler). The C_1 factor is smaller than research indicates, as noted in FEMA-274. Miranda (2001) points out that the C_1 term should be derived from oscillator response values and not from the R-μ-T relations that are based on these responses, to avoid statistical bias in the results. MacRae and Tagawa (2001) note that the coefficient C_2 should approach unity as the strength of the pinched system approaches the strength required for elastic response.

Song and Pincheira (2000) find that the FEMA-273 recommendations provide conservative estimates of the displacement amplification factors for degrading oscillators with periods greater than 0.3 sec on firm soils, and are unconservative at shorter periods.

Lew and Kunnath (2000) compare demands computed using the LSP, LDP, NSP, and NDP of FEMA-273 with the acceptance criteria of this document for an instrumented 7-story reinforced concrete frame building (the Holiday Inn, Van Nuys, California) subjected to the SAC 10/50 ground motions. A triangular load pattern was used in the pushover analysis, and member plastic rotations were calculated from chord rotations as suggested in FEMA-273. The beam plastic rotation demands determined this way were similar to the mean beam plastic rotations determined by nonlinear dynamic analysis, with pushover analysis underestimating the plastic rotation demands in the columns relative to those determined by nonlinear dynamic analysis, particularly in the upper stories.

Enhancements: The Displacement Coefficient Method could be improved by deriving C_1 factors directly from nonlinear response data. The expression for C_1 could be made a function of the degree of degradation of the oscillator load-deformation response and the degree of P-Delta effects present. MacRae and Tagawa (2001) suggest improved C_1 factors to account for near-field effects. See the discussion in Sections 2.3.2.2 and 2.3.2.3.

1.3.1.3 Drift-Based Approaches

Simple methods to consider drift in the design of multistory buildings have been put forward by Sozen and his coworkers. Lepage (1998) describes a method for estimating peak drift, in which an effective period and an elastic displacement spectrum determined for 2% damping are used with a linear model of the structure. Browning (2000) uses the technique to determine a target period to limit the expected roof drift and interstory drifts during a design level event.

1.3.1.4 Direct Displacement Based Design

Kowalksy considers Direct Displacement Based Design to be a design-oriented implementation of CSM. Rather than estimating peak displacements, a limit on displacement is used to determine the required properties of the system. Lepage noted that this method has been criticized for overestimating the effective damping present, but at the same time, Fenwick finds that uncracked properties make the members stiffer than is considered in the method. As a result, the higher damping tends to compensate for the use of a more flexible building, resulting in reasonable displacement estimates. Chopra and Goel (2001) report that the use of linear elastic spectra with increased damping, as recommended by Priestley, does not work well in comparison with an inelastic design spectrum derived using the Newmark-Hall relations.

Filiatrault and Folz (2001) have adapted this procedure to wood frame construction. Because wood softens gradually, a sharp yield point does not exist. This makes the use of R-μ-T relations very difficult, while approaches that use the CSM format can handle softening more easily.

1.3.1.5 Other Methods

Manheim and Sattary (personal communication, 2001) describe an unpublished method in which an effective acceleration is determined based on the duration and amplitude of the largest velocity pulses in a ground motion. The effective acceleration is related to the work done in deforming an elastoplastic oscillator, and this work takes place over a time interval termed the “hang time.” The potential development of various mechanisms is considered.

1.3.2 Fundamental Bases and Relationships

The Capacity Spectrum Method and Displacement Coefficient Method have different graphical representations and rely on different underlying relationships to estimate the response of nonlinear systems based on an elastic response spectrum. The Capacity Spectrum Method relies on the concept of equivalent linearization while the Displacement Coefficient Method uses R-μ-T relationships, where R= the strength required for elastic response divided by the effective yield strength of the system and μ= the displacement ductility response of the system. These fundamental relations are reviewed in the following. In principle, either method may be modified to use either fundamental relationship (equivalent linearization or R-μ-T relations) to estimate response.

1.3.2.1 Equivalent Linearization

The basis of the Capacity Spectrum Method is the idea that the peak response of an inelastic system can be estimated as the peak response of a linear elastic system having reduced stiffness and increased damping. Different approaches have been taken to determine the properties of the "equivalent" linear system. In some cases, theoretical relationships between the energy dissipated by material nonlinearity and the energy dissipated by viscous damping are used, while in others, empirical calibrations are used to identify the viscous damping (and, in some cases, stiffness) that result in the best estimates of peak displacement response. This section reviews various conceptual approaches that have been taken and discusses empirical observations that bear on the hypothesis that viscous damping is a suitable surrogate for the energy dissipated by hysteretic behavior in nonlinear systems.

Empirical Methods: Equivalent linearization requires that the stiffness and viscous damping of the equivalent linear system be established. A nonlinear system having μ=4, for example, can be represented by a linear system having stiffness equal to the secant stiffness at maximum deformation and sufficient damping to cause the peak displacement response to equal the peak displacement of the nonlinear system. Iwan observes that the secant stiffness is a lower bound to the stiffnesses that could potentially be selected, and that for each admissible equivalent stiffness, there is an associated damping level that results in the desired peak displacement. Thus, the challenge is to identify the combination of stiffness and damping that results in an unbiased estimate of the peak response and that minimizes the variance in the estimates, which can be significant.

Tagawa and MacRae (2001) identify effective damping values by adjusting the damping of an elastic system, having a period based on the secant stiffness, to obtain peak displacements equal to the peak displacement of the inelastic system. MacRae reports that negative values of substitute damping are required for some combinations of oscillator characteristics and ground motion records to match the peak displacement of an inelastic oscillator, although the mean values tend to be somewhere between the ATC-40 and Japanese Building Standard Law versions of CSM.

The Building Research Institute studied the CSM for adoption into the Japanese building code. The Japanese implementation (Otani, 2000) uses a lower amount of damping, h_{eq}, than the ATC-40 implementation. The effective damping is a function of ductility, μ. For reinforced concrete and steel members in flexure,

$$h_{eq} = \frac{1}{4}\left(1 - \frac{1}{\sqrt{\mu}}\right) + 0.05 \qquad \text{(Eq. 1)}$$

and for reinforced concrete members with pinching or slip related to shear cracking or bar slip,

$$h_{eq} = \frac{1}{5}\left(1 - \frac{1}{\sqrt{\mu}}\right) + 0.05 \qquad \text{(Eq. 2)}$$

Energy Methods: Judi et al. (2000) summarize the concepts of equivalent damping and substitute damping. Equivalent damping comes from Jacobsen (in 1930) and is applicable to nonlinear systems subjected to sinusoidal displacement histories. Substitute damping was defined by Gulkan and Sozen (1974) as the viscous damping needed in an elastic structure to dissipate the same amount of energy input to a structure responding inelastically to an earthquake ground motion, where the elastic system has a fundamental period based on the secant stiffness of the inelastic structure at its peak displacement. Gulkan and Sozen worked with stiffness-degrading systems. Hudson, working with bilinear systems responding to earthquake ground motions reportedly found (in 1965) that substitute damping values were approximately 1/3 of the counterpart equivalent damping values obtained for harmonic motion, reflecting the reduced amount of energy dissipation associated with the relatively few cycles at or near the peak displacement for earthquake shaking relative to a uniform sinusoidal displacement history.

The Capacity Spectrum Method assumes that the energy dissipated by nonlinear behavior can be equated to the energy dissipated by a linear elastic system undergoing simple harmonic oscillations at the peak displacement response. The stiffness of the linear elastic system is set equal to the secant stiffness of the nonlinear system at the peak displacement.

Following the same concept employed for evaluating nonlinear site response in the SHAKE program, Powell suggests that the secant stiffness be evaluated at 70-80% of the maximum displacement, since there may be only one or two cycles at or near the peak displacement, with most cycles having peak displacements that are substantially less than the peak displacement. Powell suggests this would result in a smaller period and smaller effective damping relative to the ATC-40 Capacity Spectrum Method, and therefore would result in larger displacement estimates. The Perform-2D program allows the equivalent linear stiffness to be set to a proportion of the peak displacement. Another tweak to the CSM would be to not reduce the elastic spectra for higher damping at longer periods, to obtain results consistent with inelastic spectra.

Several observations can be cited that question the hypothesis that equivalent damping should be obtained by equating hysteretic energy dissipation to viscous energy dissipation: (1) oscillators with different hysteretic properties can have the same peak displacement (Kowalsky); (2) as the post-yield stiffness changes from positive to negative, there is a disproportionate increase in displacement response amplitudes (Aschheim); (3) nonlinear elastic systems (such as rocking walls) have zero hysteretic energy dissipation, yet peak displacements are not much greater than systems with full energy dissipation, suggesting that the amount of hysteretic energy dissipation has little to do with the peak displacement amplitude (Miranda); (4) initially undamaged and initially damaged oscillators were found to have nearly identical peak displacements, indicating that differences in the energy dissipated through hysteretic losses has little effect on peak displacement response (Aschheim and Black). It would appear that the reduction in stiffness associated with nonlinear response breaks the build up of resonance that drives the elastic spectral ordinates to their peaks. Thus, both R-μ-T relations and effective damping relations may be sensitive to the nature of the input motion, with larger R factors being indicated for longer duration motions and smaller R-factors being indicated for motions dominated by one or two strong pulses, such as near-field motions.

<u>Accuracy of Effective Damping Relationships</u>: Iwan notes there may be some sensitivity of the optimal effective stiffness and damping values to the suite of ground motions used. Miranda reports that he is finding that Iwan and Gates (1979) relations produced relatively small mean errors on the peak displacement and the Gulkan and Sozen damping is not very accurate. Fenves, however, reports that the Gulkan and Sozen damping is good for reinforced concrete structures. Kowalsky indicates that effective damping may differ for near-field motions containing significant velocity pulses. R. Goel reports that Building Standard Law underestimates damping and leads to overestimates of displacements. Stanton expressed concern that the baseline value of damping, to which the equivalent or substitute damping is added, is not necessarily 5% and might be lower. MacRae has recalibrated the Capacity Spectrum Method and finds the scatter is similar to a recalibrated version of the Displacement Coefficient Method, except for periods above 1.5 sec, where the recalibrated CSM has greater scatter.

<u>Spectral Reduction Factors</u>: Reduction factors to be applied to smoothed elastic design spectra to establish spectral amplitudes for larger amounts of effective damping are tabulated for the Capacity Spectrum Method. Tagawa and MacRae (2001) find the actual reduction associated with

elastic response for a given damping level is not as large as is determined in ATC-40. While the Capacity Spectrum Method specifies (a) a ductility-effective damping relationship and (b) an effective damping-spectral reduction factor relationship, it is possible to establish a direct relationship between ductility and the spectral reduction factor (MacRae and Tagawa, 2001). MacRae reports that less scatter results when this single relationship is used, in the so-called Direct Capacity Spectra Method. Spectral reduction factors for systems with supplemental damping are recommended by Ramirez et al. (2000).

1.3.2.2 R-μ-T Relationships

The coefficient C_1 of the Displacement Coefficient Method was derived from an R-μ-T relationship. Such relationships are usually determined by statistical analysis of the computed response of a large number of SDOF oscillators having prescribed load-deformation relationships to actual ground motion records. There is general agreement on the form of the R-μ-T relationship (e.g. Miranda and Bertero, 1994), although there are some differences in the relationships determined by various investigators. There is significant variability in the R-factors determined for individual records. Larger R-factors generally can be expected for long duration motions that allow resonance to build up the elastic response, and smaller R-factors can generally be expected for systems subjected to predominantly pulse type motions. Soft soil sites may have R and C_1 factors that differ from those for firm sites, as discussed by Miranda and Ruiz-Garcia. The effects of supplemental damping on C_1 values are described by Ramirez et al. (2001).

Most investigators have determined R-μ-T relationships based on statistics computed on R, and have further determined displacement ratios such as C_1 by algebraic manipulation of R-μ-T relationships. Miranda observes that: (1) when required strengths are to be determined by applying R factors to elastic spectral amplitudes, the parameter of interest is R^{-1}, and (2) because the expected R factor for a given ductility level is not equivalent to the expected ductility that results from a given R factor, research studies should determine coefficients such as C_1 from the oscillator response data and not by algebraic manipulation of the R-μ-T relationship. This is further complicated by the non-monotonic nature of the strength-ductility relationship, in which different strengths may result in the same ductility response, with R factors usually based on the largest strength associated with a desired ductility level. Cuesta et al. (2001) minimize the error in estimated strengths, and find that R-factors should be stated in relation to a characteristic period of the ground motion, that is, as R-μ-T/T_g relationships, an observation also made by Vidic et al. Even so, there is some ambiguity in the identification of the characteristic period of a site, because ground motions recorded in different horizontal directions or in different earthquakes may display different characteristic periods.

1.3.2.3 The Choice Between Equivalent Linearization and R-μ-T Relationships

While some investigators find the direct computation of R-factors to be more direct than the use of equivalent linearization, Fenves observed that the averaging of R-factors over many ground motions to obtain R-μ-T relationships distances the relationships from the actual dynamics, and effective damping relationships may be as good. Fajfar noted that both approaches involve approximations, but R-μ-T relationships are easier to use, in part because no iteration is required, and most people accept the equal displacement approximation that is expressed in many R-μ-T relationships for periods greater than T_g. Miranda has shown that methods based on displacement modification and equivalent linearization can lead to small mean errors in peak displacement estimates, but significant scatter exists in the estimates made by either approach. Fajfar (1999) and Chopra and Goel (1999, 2000) have recommended R-μ-T relationships be used for reducing the elastic response spectra in the CSM. Aschheim and Black (2000) have also recommended the use of these relationships with smoothed elastic design spectra or the display of the actual, jagged, constant ductility spectra of a suite a ground motions, in the Yield Point Spectra format.

The accuracy of peak displacements estimates varies significantly from one motion to another. Fajfar believes the only reasonable approach is to provide upper and lower bounds. Miranda suggested, for example, providing median responses along with the 25^{th} and 75^{th} percentiles.

1.3.3 Near-Field Effects on SDOF Systems

Near field motions are motions that contain one or several large velocity pulses, usually originating from the superposition of waves emanating from the fault as the rupture progresses towards a site. Short period systems experience the near-field pulses as impulses. The large velocity pulses can cause the elastic spectra to be larger. The R factors associated with such pulses are smaller, in general, than those associated with motions in which resonance contributes to the elastic spectral amplitudes. It is now appreciated that structures with periods less than the characteristic period of the pulse may be severely affected (Krawinkler). Long period structures may experience large interstory drifts associated with the large amplitude ground motion reversals

Iwan et al. (2000) observe that larger displacement amplification factors and smaller strength reduction factors are indicated for structures having fundamental periods less than the predominant period of the near field ground motion, relative to far-field cases. Baez and Miranda (2000) find that displacement amplification factors (the peak displacement of an inelastic system having a specified ductility divided by the peak displacement of an elastic system having the same initial stiffness) are up to about 20% larger, on average, for near field sites, with fault normal amplifications being larger than fault parallel amplifications. MacRae and Tagawa (2001) recommend an R-μ-T relation for near field motions that changes with directivity.

1.3.4 Equivalent SDOF Systems

Both the Capacity Spectrum Method and the Displacement Coefficient Method use an “equivalent” SDOF model to represent the resistance of the structure to deformation as it responds in its predominant “mode.” Various techniques have been recommended for determining the properties of the “equivalent” SDOF system, but the resulting “equivalent” SDOF systems may differ and hence violate the notion of equivalence. In particular, the relationships used in FEMA-273 and ATC-40 result in different SDOF systems. In many cases the period of the SDOF analogue differs from the natural period of vibration of the structure (Aschheim). For structures in which the predominant mode of response involves a change in the shape vector (e.g. once a plastic hinge develops at the base of a slender structural wall), further adjustments in the post-yield capacity curve of the SDOF analogue may be indicated, although such refinements may be overly precise given the approximate nature of the displacement estimate. To resolve this issue, some “equivalent” SDOF techniques to consider include those by Rothe and Sozen (1983), Miranda (1991), Qi and Moehle (1991), Fajfar and Gaspersic, (1996), Villaverde (1996), Han and Wen (1997), and Chopra and Goel (2001b).

1.3.5 Behavior Mode Effects

Many structures will not exhibit the full hysteretic loops that are often used in analytical research studies. The presence of stiffness degradation, pinching, strength deterioration, and foundation rocking may influence peak displacement estimates. The general consensus appears to be that moderate levels of stiffness degradation and pinching will cause peak displacements of short period systems (below 0.3 to 0.5 sec) to increase above those determined for bilinear systems. Negative values of post-yield stiffness, arising either due to the load-deformation behavior of the component or the presence of P-Delta effects, can cause substantial increases in peak displacement, as can rapid strength deterioration.

Gupta and Kunnath (1998) investigated stiffness degradation and pinching, and found that "severe" degradation causes only structures with periods less than about 0.5 sec to have mean displacements substantially greater than elastic values. Gupta and Krawinkler (1998) find that peak displacements increase with the introduction of pinching, with the increase becoming larger with decreasing initial period. In this study, the ratio of peak displacement for a pinching model to the peak displacement without pinching was nearly independent of the ductility of the system. This study finds that peak displacements increase substantially as the post-yield stiffness becomes increasingly negative, and the increase is larger for weaker systems. However, the effect of negative post-yield stiffness on increasing peak displacement is reduced in the presence of pinching.

Song and Pincheira (2000) discuss effects of pinching, stiffness degradation, and negative post-yield stiffness on displacement response. They find that the equal displacement approximation is generally applicable to degrading systems for periods greater than a characteristic period (T_g) of the ground motion. Peak displacements were generally larger than those of non-degrading systems for periods less than T_g. For systems on rock or firm soil, displacement amplification factors of 2 were found at T=0.3 sec, with even larger values possible at shorter periods or on soft soils. Fischinger reports that the shape of the hysteretic loop is important for short period systems and for cases with negative post-yield stiffness or strength deterioration. Otani agreed that loop shape will not affect peak response amplitudes in the displacement-controlled portion of the spectrum.

Miranda reports that the period T_c that represents a breakpoint in the R-μ-T relationship depends on μ, as provided for in the Vidic et al. R factor relationship. Miranda reports that the shifting of the period at which the R factors change is moderate for cases of pinching and stiffness degradation, but is large for strength deterioration.

Miranda expressed dislike for the coefficient in CSM and DCM that accounts for loop shape. Miranda (2000) has determined displacement amplification ratios for oscillators located on firm sites, having bilinear and stiffness degrading load-deformation responses. Sause reportedly is determining similar parameters for nonlinear elastic systems, for use with precast post-tensioned walls.

1.3.6 MDOF/Inelastic Mechanism Effects

The use of "equivalent" SDOF systems to characterize the nonlinear response of multistory structures potentially may be misleading if higher modes should play a significant role in the response or if mechanisms should develop that were not identified in the nonlinear static (pushover) analysis. Higher modes may influence the mechanisms that develop, and different excitations potentially may cause different mechanisms to develop. This may be more pronounced in buildings in which mechanism strengths or modes of failure are not well separated.

An example of the latter case would be the development of shear failures in beams or columns due to higher mode forces, in a structure that developed a ductile mechanism in a pushover analysis. Analytical studies have focused on only a limited number of case study buildings; sufficient research to address these issues is not presently available.

Gupta and Krawinkler (2000a) relate the peak inelastic drifts observed in steel frame buildings to the elastic response of a SDOF oscillator through a series of factors that account separately for roof drift relative to SDOF response, the development of inelasticity, the presence of P-delta effects, the ratio of interstory to average roof drift, and the relation between element deformations and interstory drift. Three-, nine-, and twenty story steel moment frame buildings were subjected to the components of the SAC 2/50, 10/50, and 50/50 ground motions that were oriented at 45 degrees to the fault-normal and fault-parallel directions. They report that a good estimate of the ratio of elastic roof drift to the first mode spectral displacement is given by the first mode participation factor, but for structures with periods greater than 2 sec they advise use of 1.1 times the first mode participation factor. The effects of inelasticity on roof drift for the MDOF structures are consistent with and similar to the effects observed for SDOF systems. For the period range considered, inelasticity tended to cause peak drifts to be about 70 to 80% of the elastic values, at the median level. The MDOF inelasticity factor tended to become smaller with an increase in roof drift. This was explained as possibly being the result of a concentration of interstory drift demand in a few stories leading to a reduction in roof drift. Gupta and Krawinkler (2000b) find that P-Delta is a relatively benign phenomenon except if the ground motion drives the structure into the range of negative post-yield stiffness, at which point large increases in displacement may occur. The ratio of peak story drift to peak roof drift is strongly dependent on the ground motion and structure characteristics. Median values of this ratio increase from about 1.2 for low-rise structures to 2.0 for mid-rise structures to about 2.5 to 3.0 for tall structures, for the structures and motions considered. The drift patterns observed for these structures suggest that a common drift distribution that can be generalized does not exist.

Foutch and Shi (1998) report the results of nonlinear dynamic analyses of steel frame buildings for the SAC steel program in which the beam plastic hinges were modeled with different load-deformation models. Steel moment frame buildings were analyzed that ranged from 3 to 9 stories in height. Eight hysteretic models were considered for the beam plastic hinges: bilinear (with and without strength degradation), stiffness degrading (with and without strength degradation), pinched stiffness degrading (with and without strength degradation), fracturing, and bilinear elastic. The plastic hinge model did affect response histories at the connections. The effect on the maximum story ductility demand relative to the maximum story ductility demand for the non-degrading bilinear model is as follows: a maximum increase of 10-20% for the non-pinching hysteretic models, a maximum increase of 20-30% for the pinching hysteretic models, and a maximum increase of 30-50% for the bilinear elastic model (which has no hysteretic energy dissipation). The Foutch and Shi results may be applicable to buildings that develop desirable mechanisms; Aschheim expects that a weak-story system having degrading column hinges would have much worse performance than the buildings described above.

Naeim et al (2000) also investigated the response of steel moment frame buildings for the SAC steel project. Three-, nine-, and twenty-story steel moment frame structures were investigated using a variety of hysteretic models. Stiffness degradation with slip or pinching was modeled in addition to bilinear response. Strength deterioration was modeled but results were not described. Severe stiffness deterioration increased interstory drifts and residual interstory drifts, with interstory drifts often increasing in the upper stories, and sometimes reducing in the lower stories. The authors suggest this may be attributed to higher modes causing the upper stories to go through many more cycles of sufficient amplitude to be affected by deterioration of the load-deformation response. The observed increases tended to be larger than those observed by Foutch and Shi. Stiffness degradation generally reduced force demands. Slip often reduced lower and upper story interstory drift demands, although increases were observed for some combinations of building height, city, and ground motion intensity level. Slip tended to decrease story shears and overturning moments.

Iwan et al. (2000) used a shear building model to compare the Capacity Spectrum Method with nonlinear dynamic analysis for near-field motions. They find that for structural periods shorter than the ground pulse duration, the MDOF building exhibits a fundamental mode type of response, and higher mode contributions to drift and shear demands are negligible. For long period structures, the large displacement and velocity pulses of near field records cause greater participation of the higher modes, resulting in the potential for very misleading results if a single mode analysis is used. Large interstory drifts were observed at the base, during the forward movement of the ground motion, and were again observed at the upper stories during a large reversal of the ground motion, associated with wave propagation through the structure. This latter case is not associated with the development of the maximum roof drift and does not correspond to a first mode shape, and thus a fundamental mode analysis would not detect these drifts. The investigators conclude that the use of a single-mode "equivalent" system provided a reasonable estimate of the maximum roof displacement regardless of building period, degree of nonlinearity, or distribution of stiffness, even for pulse type motions, but estimates of interstory drift for tall buildings (fundamental period significantly greater than the ground pulse duration) were poor, particularly in the upper stories.

1.3.7 Pushover Analysis

Nonlinear static (pushover) analysis is used to quantify the resistance of the structure to lateral deformation and to gauge the mode of deformation and intensity of local demands. Various techniques have been recommended, including the use of constant lateral force profiles and the use of adaptive and multimodal approaches. Pushover techniques provide useful information on the overall characteristics of the structural system and can be used to identify some (but not necessarily all) of the likely mechanisms. Because the prescribed loading used in pushover analyses can not represent the potential range of loading experienced in dynamic response, the results obtained by pushover analyses at best represent an approximation of the nonlinear behavior expected to develop in the response to earthquake ground motions. The utility of pushover analyses is less clear for systems having discontinuities in strength and stiffness. Furthermore, results may be misleading where multiple collapse mechanisms potentially may develop because mechanism strengths are not well separated, or where different modes of behavior potentially may develop, for example, if higher modes cause demands to approach or exceed the capacities of strength-controlled components.

Pushover techniques are useful to estimate peak displacement response in conjunction with the use of "equivalent" SDOF systems. While higher modes typically have a small or negligible contribution to displacements, higher modes can significantly affect interstory drifts, plastic hinge rotations, story shears, and overturning forces. The contribution to interstory drifts stems directly from the higher mode shapes being more tortuous and therefore having a greater contribution to interstory drift. Consequently, estimates of interstory drift based on a first mode pushover analysis is prone to be inaccurate as the number of stories increases. Pushing to a target displacement will not develop the maximum interstory drifts in each story because the maximum values in each story do not occur simultaneously, and the sum of the individual maximum interstory drifts may be twice the peak roof displacement, depending on the mechanism that develops (Krawinkler). Where weak stories exist, some evidence suggests that pushovers tend to overestimate the weak story drifts, while underestimating interstory drifts at other locations. Although there are limitations on the accuracy of first mode pushover analyses for interstory drift estimates, it should be noted that common code equivalent lateral force procedures provide limits on the interstory drifts calculated using nearly first mode lateral force distributions.

The application of lateral forces in a pushover analysis is preferred to applying a prescribed displacement pattern because the former allows softening of the structure to develop and allows story collapse mechanisms to develop. Many techniques involving application of lateral forces have been used. The simplest technique uses a fixed lateral force profile, with lateral forces being proportional to the mass and mode shape amplitude at each floor. An updated load vector would be more likely to identify concentrations of damage, although this presumes that first mode response is dominant—higher modes may dominate or may influence mechanism development in the first mode.

Other techniques update the lateral force profile to adapt to the softening structure, use a fixed lateral force profile that is modified from the first mode pattern to account for higher modes, or combine the results obtained from independent pushover analyses in each of several modes. Adaptive techniques that update the lateral load vector can make the updated load vector be proportional to the current displaced shape or to the current first mode (based on the current stiffness properties of the structure) or may make the increment in lateral loads proportional to the current displaced shape or mode shape. The displaced shape changes more quickly than does the mode shape (Valley). Inconsistencies can be introduced if the load vector is updated without updating the mode participation and mass participation factors used for determining the properties of the "equivalent" SDOF system. Methods that consider higher modes must contend with uncertainty in the amplitudes and algebraic signs of the higher modes, along with their timing relative to the first mode peaks. The question of how simple or complex a pushover technique to use depends on one's analysis objectives. Simple techniques can provide very valuable but incomplete information, while techniques that are more complex are still unable to represent the full range of response that potentially may develop.

Valley and Harris (1998) describe the development of a static pushover curve by repeated elastic analyses, with members removed sequentially as deformations exceed the member yield or ultimate capacities, and with loads reapplied in accordance with updated Ritz vectors. Reinhorn describes multimodal procedures (1997) that rely on updated modal properties. These were developed in more detail for simple and irregular structures in DeRue (1998) and Barron Corvera (2000). Bracci et al. (1997) also determined demand estimates based on the instantaneous dynamic properties of the structure; Gupta and Kunnath (2000) coupled the use of the instantaneous dynamic properties and the elastic spectral ordinates of the ground motion to determine incremental lateral forces to be applied in the pushover analysis. More recently, Kunnath has looked at sums and differences of modes. Elnashai (2000 and 2001) also has applied adaptive techniques that make use of the instantaneous modal properties, and is able to follow the S_a vs S_d plot obtained in Incremental Dynamic Analysis reasonably well. Reinhorn suggests that the multimode pushover force distribution can be simplified to a linear distribution that is unique for each structure. Saitoh (2001) describes an "Inelastic Modal Deformation Analysis" method in which a lateral deformation pattern is evaluated by modal analysis using equivalent stiffness and hysteretic damping factors associated with inelastic response. Kunnath reports that even adaptive pushover techniques fail to capture the response of some stories in some buildings.

Sasaki et al. (1998) perform pushover analyses independently in each of several modes using invariant lateral force distributions, to identify the potential for higher modes to cause mechanisms to develop. Black and Aschheim (2000) combined the peak displacements and interstory drifts determined independently for the first two modes using SRSS combinations, and observed significant disparities between the peak interstory drifts and the SRSS estimates. This procedure is termed a Modal Pushover Analysis (MPA) by Chopra and Goel (2001b), who consider up to three or five modes. Chopra suggests that SRSS combination rules may be used for all computed quantities (e.g. member forces and moments), not just displacements and interstory drifts. Chopra and Goel (2001c) demonstrate that median estimates of interstory drift of the SAC buildings are improved by the use of three modes for the 9-story buildings and five modes for the 20-story buildings, with baseline values established by nonlinear dynamic analysis. Errors in the interstory drift estimates were larger, in general, than the errors associated with response spectrum analysis of linear elastic buildings and were largest for the "Los Angeles" buildings, which generally had larger interstory drift responses than the "Seattle" and "Boston" buildings. Patterns of the distribution of median interstory drifts of the "Los Angeles" and "Seattle" 9- and 20- story buildings differed. Chopra and Goel (2001b) also put forward an Uncoupled Modal Response History Analysis (UMRHA), in which dynamic response histories determined for each "equivalent" SDOF system are summed algebraically in time, and maximum values are determined from the summed response history. If nonlinearities are absent, the MPA and UMRHA approaches are equivalent to the traditional response spectrum and linear dynamic analysis methods, respectively; the interaction of modes introduces errors that make the techniques approximate for nonlinear systems.

Aydinoglu (2001) extends pushover analyses to incorporate P-Delta effects on a story-by-story basis. A mass-normalized story geometric stiffness is used to represent P-Delta effects in place of the traditional story-based stability coefficient.

Iwan is working on load profiles that will better predict the spatial distribution of damage; Carr also is reported to be working on improved pushover techniques. Bracci recently has been doing pushovers on frames one story at a time. Deierlein suggested that one could apply a perturbation to a first mode load pattern, consisting of an additional force that is allowed to change its location over the height of the structure, to identify sensitivity in the development of the mechanism.

1.4 Nonlinear Dynamic Procedures

1.4.1 Simplified Models

Nakashima et al. have described the use of simplified models for nonlinear dynamic analysis. Much like the "notional frames" used by Sozen and Lepage, the generic frame or "fishbone" model consists of a single column with beams at every floor level extending halfway towards an adjacent column, with a roller supporting each beam at midspan. The model allows beam plastic hinges and story mechanisms to develop, much as they can in complete frames. The generic frame model, however, does not determine actions on individual members of the frame (Otani).

1.4.2 Incremental Dynamic Analysis

Incremental Dynamic Analysis determines peak response quantities (e.g. roof drift) by a series of independent nonlinear dynamic analyses of a structure subjected to one or more scaled ground motions. The scale factor is increased successively from a small initial value, and peak response quantities are plotted against a measure of the ground motion intensity. Two versions of IDA were identified. Cornell and Krawinkler plot peak interstory drift as a function of the scaled S_a at the fundamental period of the building ($S_a(T_1)$). Elnashai plots the peak roof displacement versus $S_a(T_1)$. Both investigators plot S_a on the vertical axis and the damage measure on the horizontal axis, to be consistent with the conventional plotting of deformations and displacements on the abscissa and forces on the ordinate. Cornell has developed relationships between the nonlinear static pushover capacity curve and the IDA curve that are implemented in a spreadsheet, allowing an engineer to observe the influence of changes in the capacity curve on response parameters, based on relationships embedded in the spreadsheet. Because these relationships reflect the behavior of the structure, it seems they must change as the relative distributions of strength, stiffness, and mass of the structure vary.

The strength of the IDA is that it captures aspects of the dynamic behavior of the system, and when done for a sufficient number of ground motions, reflects the range of damage that may result. Although investigators apparently have not used the technique to characterize the range of mechanisms that may potentially occur under different excitations, the ability to recover this information seems to be another benefit of this form of analysis. Interstory drifts are observed to increase dramatically when the intensity measure is large enough. Such an increase suggests the structure has reached its capacity, allowing the "capacity" of the structure to be characterized in terms of the intensity measure. (This is analogous to the capacity of a steel bar being measured by its ultimate strength, except that the capacity is expressed in terms of the spectral acceleration of a specific excitation waveform.) Typically, there is substantial scatter in the capacities determined in this way, reflecting variability in the response of the structure to different excitations.

Concern has been expressed regarding (1) the validity of scaling the ground motion amplitude uniformly (because high frequencies attenuate more rapidly as distance from the fault increases), (2) the uncertainty in establishing an accurate structural model, (3) ambiguity in the definition of "capacity," with Cornell focusing on the interstory drifts and Krawinkler now focusing on the value of the intensity measure ($S_a(T_1)$) at which the response parameter (or damage measure) seems to increase without limit, and (4) whether interstory drift is an appropriate parameter to monitor collapse, when collapse may be due to gravity loads acting, for example, after columns have failed in shear.

The IDA curves are interesting because of the peculiar dynamic response characteristics that are apparent in this representation, and may be useful for identifying variability in demands, the "capacity" of the structure, as well as the onset of collapse, subject to limitations on modeling. The usefulness of IDA for design verification has not been investigated yet.

1.5 Modeling Limitations

Accuracy in the estimate of response of a given structural model is of little value if the structural model itself is inaccurate. Issues relating to the accuracy of mathematical models used for estimating response include (1) evaluation of initial stiffness and strength; (2) uncertainty and variation in the actual material properties, dimensions, and dimensions of the as-built structure, (3) variation of the actual component strengths from calculated estimates, (4) the complexity of behaviors to be represented; and (5) limitations in our understanding and modeling of response to complex, inelastic loading histories.

Uncertainty in the initial stiffness and strength of a structure leads to further dispersion in the accuracy of the displacements estimated using the Capacity Spectrum and Displacement Coefficient Methods (Miranda). For reinforced concrete structures, there is ambiguity in how the period of vibration of the structure should be computed (Otani). An additional difficulty relates to actual compressive strengths exceeding the specified strength, leading to likely increases in the modulus of elasticity (Otani). Valley noted that of three tuned-mass-damper buildings that his firm designed, the estimate of period for one was sufficiently off that they had to redesign the tuned mass damper after construction. The Mexico City Building Code of 1976 modified the design spectrum to account for uncertainty in the period estimate (Chopra and Miranda).

Choices made by structural engineers in modeling of a structure can affect computed response. Krawinkler recalled that in the SAC project, a centerline model of a 20-story building was found to collapse in the presence of P-Delta, but had drifts of no more than about 5% when panel zones and gravity columns were modeled. Krawinkler noted that different investigators using different computer codes obtained very different results when first modeling buildings for the SAC project. Only when assumptions were made consistent were the results more or less identical. Diaphragm flexibility generally not been incorporated into simplified inelastic procedures; an approximate method is described by Nakaki (2000).

There are relatively few instances in which models have been developed of instrumented buildings that were heavily damaged by ground shaking. Kunnath et al. (2000) considered four instrumented buildings, of which two were moderately damaged. He finds that calibrating structural models to observed response is sensitive to mass and stiffness modeling assumptions. Kunnath reports that linear and nonlinear static procedures did not adequately predict interstory drift estimates, and no one procedure consistently gave good results. Islam et al. (1998) model the 7-story instrumented reinforced concrete building in Van Nuys, and find that extensive flexural cracking in the beams observed in the pushover analysis at the measured roof drift did not occur; the actual building had only minor flexural cracking at the lower level beams. Browning et al. (2000) report on the ability of various analysis procedures to estimate peak drifts and interstory drifts of this building, and the difficulty in matching locations of column shear failure.

Multiple actions (e.g. P, V, M) result in inelastic behaviors that are not well-understood and which are represented poorly in analysis software. Modeling of collapse requires careful attention to component degradation and may require that the assumptions of small displacement theory be supplanted by large displacement theory. The accuracy of computed predictions of collapse has not been established; even definition of a collapse limit state is ambiguous. Until future research better addresses these questions, inelastic procedures probably should focus on structures that are less severely damaged. Experience with IDAs suggests that degradation can be neglected for the performance objectives that limit significant damage.

1.6 Demand Characterization

The lack of an accepted and clearly-defined relationship between smoothed design spectra and the actual motions the spectra ostensibly represent creates difficulties in (1) evaluating the accuracy of inelastic procedures, (2) assessing variability in response estimates, and (3) establishing design ground motions for use in performance-based earthquake engineering.

Traditionally, smoothed design spectra were fit by judgment to the jagged elastic response spectra computed for real ground motions. Current approaches fit a smoothed design spectrum at T= 1 sec and at "short" periods, using values determined from a seismic hazard curve. The degree to which actual spectra may, and should, depart from a smoothed spectrum is not defined, yet the degree of variability surely affects the statistical distribution of peak displacements relative to estimates based on smoothed elastic response spectra. Scaling ground motions to precisely match a target design spectrum has been found to result in a systematic underestimate of inelastic response, because response amplitudes to the stronger ground motions are often disproportionately higher than those to weaker ground motions (Wen).

Cornell notes that demand is not a design spectrum but a set of earthquake events that cannot be collapsed into a single spectrum. Particularly for uniform hazard spectra, there does not seem to be a clear answer on how to choose records (Cornell). However, to represent record-to-record variability, it appears to be necessary to use recorded ground motions rather than synthetic motions. For design applications, Wen (2001) suggests using records based on regional seismicity—perhaps a Magnitude 8 earthquake at 40 km, a few M7.5 earthquakes at 20 km, a few M6 earthquakes at closer distances, etc. For the evaluation of inelastic procedures by ATC-55, Cornell suggested that a bin of records be used that have approximately the same magnitude, distance, and soil conditions; the average of these spectra could be used in place of the smoothed elastic spectrum used in the DCM and CSM.

1.7 Applicability for Performance Based Earthquake Engineering

1.7.1 Role for Inelastic Procedures

Many researchers have focused on improving simplified analysis procedures with the goal of accurately representing response quantities determined in nonlinear dynamic analyses, with some operating under the notion that analysis and design are so intertwined that they cannot be separated. Other researchers view the role of analysis is to enable good design, acknowledging that even the best analyses are approximate and that approximate analyses are sufficient. Given uncertainty in the accuracy of the mathematical model of a structure and uncertainties in future ground motions, engineers often must rely on their judgment to interpret analytical results. There is a fundamental uncertainty in response amplitudes that applies to all analysis techniques because of variability in the R-μ-T relationship from one motion to another and variability in the elastic spectral ordinates, timing, and algebraic signs of the higher modes. Because even the best analysis techniques are prone to uncertainty with regard to performance under future earthquakes, there may be a role to be played by simplified analysis techniques.

Simplified inelastic procedures can be used for preliminary design, for identifying those structures that require more detailed evaluation, and for characterizing performance. Simple inelastic procedures can give good estimates of peak roof displacement, at least for regular structures in which response is dominated by the first mode under conditions where P-Delta effects are negligible. Estimates of interstory drift indices, story shears, and plastic rotations in relatively flexible buildings are prone to be inaccurate, due to higher mode contributions. Therefore, inelastic analysis procedures may be useful as a first approximation and to indicate when analyses of higher precision are needed. Elastic analysis procedures also can serve this purpose, although one would expect inelastic procedures to provide higher fidelity. Inelastic analysis procedures could be used to encourage capacity design approaches in new design. As with elastic design approaches, plastic hinges can be used to “absorb” errors in the analysis.

The profession is in the midst of a transition from force-based design approaches to displacement-based design approaches. A complete implementation of a displacement-based approach involves (1) determining displacement demand, (2) breaking down overall demand into local components, and (3) comparing local supply to demand (Bonacci). Simplified inelastic procedures can be used to move from force-based approaches (which are very imprecise but were useful for proportioning structures) to displacement-based approaches. Bonacci urges caution in rushing too rapidly to compare local demands and capacities, and cites as an example the difficulty in evaluating whether a stiffener will buckle when we may be 50% off on T_g and PGA. The desire for accurate analytical results must be balanced against the significant uncertainties in deformation capacities (Krawinkler). Furthermore, complicated techniques may be misused by engineers that are unfamiliar with them (Krawinkler).

Foutch suggested that if inelastic analysis techniques are used, they should be simple enough to be useful for conceptual design and may be useful for approximate analysis. Reinhorn suggested that one might use a simple technique to proportion the structure, then iteratively adjust the relative distributions of strength to ensure undesirable mechanisms will not form, and then follow with a more complex procedure to develop statistics on response. Miranda suggested that a simplified static procedure would be useful with estimates of dispersion, followed by nonlinear dynamic analyses to assess simulated response statistics. Aschheim suggests a simple inelastic analysis technique could be used for preliminary design, with nonlinear dynamic analysis being used to develop response statistics only for those structures where this comparison is deemed necessary (e.g. substantial irregularities, high importance, or to satisfy client requirements).

Because of uncertainties in the effects of higher modes, any simple procedure will require that prescriptive provisions are used to ensure that (1) desirable mechanisms form, with plastic hinges having sufficient ductility capacity to absorb uncertainties in plastic rotation demands arising from the presence of higher modes, (2) undesirable mechanisms (e.g. weak story mechanisms) will not form, even under the influence of higher modes, and (3) force-controlled components or modes of behavior have sufficient strength that forces associated with higher modes do not cause brittle failures to result. Variability due to higher modes can be expected to be a function of the number of stories as well as the spectral amplitudes at the higher mode periods. The separation of strengths required to prevent brittle modes of failure and undesirable mechanisms depends in part on the variability of material strengths in the as-built structure and the variability of actual strengths relative to calculated estimates.

Otani expressed concern about safety in view of the scatter in displacement estimates; the Japanese are using a modified form of Capacity Spectrum Method for checking the performance of designs that satisfy other criteria. Wilson expressed concern that nonlinear response spectra are not applicable to MDOF systems; response of 2D and 3D structures can and should be determined by nonlinear dynamic analysis, in part because only nonlinear dynamic analysis can really inform the engineer about the behavior of the systems being designed.

Lepage suggested that an iterative procedure could be used, wherein a variety of load patterns are used to determine a variety of deflected shapes and possibly a number of different mechanisms. If similar deflected shapes result, then the deflected shape would be used to determine the "equivalent" SDOF system for each of a suite of ground motions, recognizing that iteration will be required to identify the right shape to be used for different drift levels. Lepage also suggested an alternate approach in which linear estimates of roof displacement are coupled with a collapse mechanism analysis—if drifts concentrate in just a few stories, then all of the estimated drift would be assigned to those stories.

1.7.2 Design with simplified inelastic procedures

Design procedures have been formulated for use with three spectral representations. Direct Displacement Based Design uses the concept of effective damping to establish response spectra that are plotted on the same axes used in the Capacity Spectrum Method. The period of vibration (or stiffness) required to satisfy a performance objective is determined, along with a required strength. The use of effective damping is supplanted in design procedures recommended by Fajfar and by Chopra and Goel, who use inelastic spectra (based on R-μ-T relations) plotted on the same axes used in the Capacity Spectrum Method to estimate peak displacements and to determine required strengths. Black and Aschheim used Yield Point Spectra (based on R-μ-T relations or the actual jagged spectra associated with design ground motions) to determine the strength required to satisfy multiple performance objectives using admissible design regions. An iterative approach was suggested in which nonlinear static analyses are avoided entirely by relying only on design strengths and elastic properties.

Otani notes that one could use a nonlinear static procedure to get design moments for beam hinge regions, and then apply a factor of safety to design the columns to prevent or limit the development of plastic hinges in the columns.

1.7.3 Quantities to be determined and measures of performance

There is uncertainty in estimates of demands and capacities. Rather than compare very approximate values of local demands and capacities, some suggest that it may be preferable to focus on quantities that are of a more global nature, such as interstory drift (R. Goel). Estimates of deformation capacity are fairly crude; Krawinkler observed that the best measure of inelastic deformation capacity (total or plastic rotation, curvature ductility, etc.) has not even been identified yet.

One approach is to estimate peak interstory drifts as a factor times the average roof drift. For regular buildings, the factor varies with the number of stories and may not follow a consistent pattern over the height of the building (Gupta and Krawinkler, 2000a), and may depend on the ground motion (MacRae). Uetani and Tagawa (1998) reportedly find that interstory drifts concentrate less in structures in which the eigenvalues obtained during the nonlinear response are larger (more positive). Fenwick reportedly has introduced into the New Zealand Code an estimate of interstory drift equal to twice the drifts determined by elastic analysis. Interstory drifts for near-field motions appear to be related to ground motion reversals (Iwan), and might be better estimated using concepts of wave propagation theory rather than conventional modal response approaches.

The actual shears in a building can be significantly higher that those associated with development of capacity in the predominant mode. Dynamic shears, therefore, may be significantly higher than estimated by pushover analysis (e.g. see Eberhard and Sozen (1994) and a chapter by MacRae and Roeder). Rodriguez, Restrepo, and Carr reportedly found the second and higher modes respond essentially elastically, contributing to the shears associated with inelastic first mode response. Forces in reinforced concrete collectors may be poorly estimated by typical procedures because their larger stiffness in compression causes greater force to be carried in compression than in tension.

Kunnath notes that plastic hinge rotation demands are calculated differently in different software programs. The post-yield stiffness, hinge lengths, and use of distributed or concentrated plasticity affects the values, as do the different solution strategies used by the programs. Estimates of yield and plastic rotation are often based on assuming points of inflection occur at midspan, leading to errors of 50 to 100%. The approximate nature of the demands estimated by any procedure makes comparison with estimated capacities less certain; significant improvements are needed to improve the reliability of estimates of local demands and capacities, to make their comparison meaningful.

1.7.4 Statistical measures and treatment of uncertainty

Performance may be evaluated in different ways and may include or exclude various types of uncertainties. For example, Wen determines the annual probability of exceeding drifts of various levels. Cornell's work for SAC focuses on the level of confidence in the hypothesis that the structure will satisfy a given performance objective, for ground motions that have a stated probability of exceedance. Uncertainty in the hazard is neglected in the SAC work, although Cornell et al. (2000) presents a theoretical formulation that accounts for uncertainty in the hazard. While it stands to reason that variations in capacities (strengths, deformation capacities) should have an influence on demands, current formulations neglect such interaction.

1.8 Conclusions (and Areas of General Agreement)

1.8.1 Relative accuracy of procedures

Researchers have reached different conclusions about the accuracy of displacement estimates made using the CSM, with some finding displacements to be overestimated and others finding displacements to be underestimated. This may stem from the use of different research methodologies, related to the absence of a clear definition of the relationship between a smooth elastic design spectrum and the jagged spectra of actual ground motions. When compared to well-accepted R-μ-T relations, the CSM tends to underestimate displacements in many cases. This can be attributed to an overestimation of effective damping and the use of excessive spectral reductions for a given level of damping. Empirical calibration of the method or the use of R-μ-T relationships in place of ductility-damping-spectral reduction factor relationships has the potential to improve the accuracy of the method.

Compromises made in the development of the DCM may have affected its accuracy. Some investigators find the DCM overestimates displacements, even for stiffness degrading oscillators, for periods greater than 0.3 to 0.5 sec, but is unconservative for shorter periods. Recalibration of the method potentially could improve its accuracy.

Different relationships may be appropriate for near and far field motions, and may be necessary for systems with degrading component response. An alternate approach is to apply bias factors to the displacement estimates to account for these effects, along the lines of what was done in the SAC steel project.

1.8.2 Fundamental Bases and Relationships

Both the CSM and DCM estimate response based on "equivalent" SDOF representations of multistory buildings. The methods use different graphical representations of response and use different fundamental relations to estimate the response of the nonlinear "equivalent" SDOF system relative to an elastic response spectrum. The graphical representations and the fundamental relationships used in these methods can be interchanged—that is, it is possible to use the effective damping relationships of the CSM in the conventional S_a vs T format and it is possible to use the coefficients (or R-μ-T relationships) of the DCM in the S_a vs. S_d format.

Research suggests that the R-μ-T or ductility-effective damping-spectral reduction factor relationships used in these methods should be different for near and far field motions, with smaller R factors generally being applicable to near-field motions. Some researchers have suggested the R-μ-T relationships should be restated to account for the frequency content of the ground motion as R-μ-T/T_g relationships, where T_g is a characteristic period of the ground motion. Different relationships are necessary for sites on soft soil deposits that give rise to nearly harmonic ground motions.

Various methods to obtain "equivalent" SDOF systems are available, and these can lead to different (non-equivalent) SDOF models. "Equivalent" SDOF systems can be derived to be equivalent to modal analysis for linear elastic response; superposition of these responses becomes approximate for nonlinear systems.

1.8.3 Behavior Mode Effects

Studies of SDOF structures show degradation can cause an increase in peak displacement response, particularly for periods below 0.3 to 0.5 sec. Studies of MDOF frame structures with degrading component models show greater effects in the upper stories of flexible buildings, where higher modes go through many cycles of sufficient amplitude to cause degradation. Degradation potentially may exacerbate displacements in weak story buildings. Duration generally may amplify the effects of degradation.

Rocking at the foundation level may affect response amplitudes and the distribution of damage throughout the structure. Where the entire foundation rocks as a rigid body, roof displacements and interstory drifts are generally increased by the rigid body rotation of the building-foundation system. Second-order (P-Delta) effects may become more prominent, and pounding (impact with adjoining structures) may become more of a concern.

1.8.4 MDOF/Inelastic Mechanism Effects

First mode "equivalent" SDOF systems are useful for estimating peak roof displacement for regular structures of varied heights subjected to both near and far field ground motions. Peak interstory drifts may be estimated accurately for low-rise structures. Taller frame structures tend to have smaller roof drifts relative to simple predictions when inelastic drift concentrates in just a few stories.

Higher modes affect interstory drifts, story shears, overturning moments, and plastic hinge rotations. The distribution of interstory drifts over the height of the structure apparently does not follow a pattern that can be readily generalized, and often differs for ground motions with near-field pulses. Dynamic shears are larger than those determined using pushover analyses with triangular or rectangular force distributions. Pushover analyses may not be able to identify the different mechanisms that potentially may form under different dynamic excitations. P-Delta effects appear to be minor so long as the post yield stiffness of the story load-deformation curves remains positive in the presence of the second order effects.

Component degradation may exacerbate demands where damage localizes, affecting the inelastic mechanism. This may cause interstory drifts to increase substantially in weak story systems and in the upper stories of taller buildings.

Various extensions to the inelastic procedures to account for higher modes using adaptive or multimodal pushover analyses have been suggested. Multimode approaches suffer from an uncertainty as to whether to combine the modes with positive or negative algebraic signs. The increased complexity of these methods must be weighed against the quality of the information obtained, relative to the information that can be obtained by nonlinear dynamic analysis.

Potential limitations due to irregularities are not well-established. Insufficient information is available on the use of simplified procedures where plan torsion or other 3D inelastic effects are present.

1.8.5 Characterization of Demand

There is no clear answer on how to choose ground motions to represent a smooth design spectrum. Because smoothed design spectra are anchored by just several spectral values (e.g. S_a at 0.2 and 1 sec), the relation between a smoothed design spectrum and actual, jagged, spectra computed for real ground motions is unclear. The lack of a definite relation makes it difficult to establish suites of ground motion records that are suitable characterizations of the smooth design spectrum. Overmatching of jagged spectra to a smoothed design spectrum leads to a systematic underestimation in response, because larger motions tend to cause disproportionately larger inelastic responses. The greater the spread of the actual spectra about a design spectrum, the greater the dispersion in computed response. Uncertainties in the timing, relative amplitudes, and signs of higher mode responses leads to a fundamental uncertainty in component demands.

R-μ-T and ductility-effective damping-spectral reduction factor relationships differ for far field and near field motions, and depend on the behavior mode. Different relationships are applicable to soft soil sites. Some researchers recommend that these relationships should be indexed against a characteristic period of the ground motion.

2 Potential Case Study Buildings

During interviews with researchers, the Working Group became aware of a number of instrumented buildings that may be good candidates for analytical studies. These are:

- Van Nuys Holiday Inn: Miranda is doing dynamic analyses in Drain 2DX. Pincheira supervised a thesis on this; Browning, Moehle, Li, and Lynn and Jirsa have a paper in Earthquake Spectra on this building; Kunnath has investigated this building. PEER uses this building as an example case study building; it is presently being modeled in OpenSees.
- SAC buildings: Steel frame buildings having different numbers of stories were designed, modeled, and analyzed for the SAC steel project.
- 11-story RC frame building damaged by Northridge, instrumented by CSMIP
- 13-story Blue Cross steel moment frame building, instrumented by CSMIP, damaged in Northridge, being looked at by Kunnath.
- Example building used in FEMA-307
- The Pacific Bell Annex, at 17th and Franklin in Oakland. Designed by Degenkolb in 1960; a dual system with a torsional irregularity.
- The Hollister Tilt-Up Warehouse, instrumented by CSMIP, with wood diaphragms.
- The Burlingame Hyatt
- The Barrington Building (Santa Monica) with damaged captive columns
- The Imperial County Courthouse
- One or more buildings being tested on the CERL shake table as part of the R-factor project

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