Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Seismic design based on strength and ductility demands

A.A. Nassar & J. D. Osteraas
Failure Analysis Associates, Menlo Park, Calif., USA

H. Krawinkler
Department of Civil Engineering, Stanford University, Calif., USA

ABSTRACT: A dual-level limit state seismic design approach is presented in this paper. The collapse limit state is addressed by providing strength capacities that limit ductility demands in critical structural elements to their safe ductility capacities for severe earthquakes. The inelastic strength demands imposed on SDOF systems, derived from smoothed ground motion spectra using reduction factors that depend on both the target ductility ratio and period of the structure, are related to the base shear demands on MDOF systems. The serviceability limit state for moderate earthquakes imposes a lower bound on the stiffness and strength of structural elements in order to control structural and non-structural damage. Reconciliation between the inelastic demands of the collapse limit state and elastic demands of the serviceability limit state is considered.

1 INTRODUCTION

Current seismic design procedures (e.g. UBC (1991)) are based on elastic response spectra, which are modified versions of smoothed ground motion spectra. Allowable stress design spectra are derived from the elastic spectra using system dependent, but period independent, reduction factors (R_W). There are several conceptual problems with this approach: (a) the relationship between R_W and ductility demand is not constant: for equal ductility demand the strength reduction factor varies with elastic period of the system, (b) due to wide variations in "overstrength" in structures, the code provides inconsistent levels of protection against collapse during severe earthquakes, (c) true strength and ductility demands and capacities are hidden from the designer (i.e., the procedure is not transparent), and (d) damage control during moderate earthquakes is not directly addressed.

Safety (collapse) and serviceability limit states are addressed implicitly in current design procedures and have been addressed explicitly in some more progressive design guidelines. Likewise, critical parameters of structural response (inelastic response, ductility capacity and overstrength) are beginning to be addressed in new design procedures. Improved understanding of inelastic seismic demands on buildings provides the basis for examination of refined design procedures that more accurately and explicitly model inelastic structural response.

The objective of the reported research is to develop a design procedure that utilizes member ductility capacity and inelastic structural response as the basis of design at the collapse limit state. The safe ductility capacity of critical elements, e.g., a plastic hinge in a beam of a moment resisting frame or a shear link in an eccentrically braced frame, should take into account the elements' efficiency in dissipating hysteretic energy with minimum deterioration for prolonged strong motion durations, the consequence of failure of these elements, tolerated limits of drift for structural integrity, etc. This points out the dependence on the characteristics of the expected ground motions, e.g., severity, duration and frequency content, as well as the characteristics of the structure, e.g., period, structural system, strength and stiffness discontinuities, failure mechanisms, redundancy and detailing. These factors affect the number and magnitude of inelastic excursions, which in turn determine the cumulative damage experienced by a structural element.

In an attempt to crystallize the concept of "demand versus capacity" in seismic design, this paper outlines a procedure in which displacement, whether ductility or drift limitation, becomes the basic design parameter. The seismic design objective for the collapse limit state is to provide adequate strength for the structure in order to limit the ductility demands at critical structural elements to specified target values (ductility capacities) incorporating an appropriate margin of safety. The derived strength and stiffness of the structure is checked against the limitations imposed by the serviceability limit state. A reconciliation of demands imposed by the two limit states is required.

The proposed design methodology more realistically models inelastic structural response, results in structural designs with consistent levels of seismic safety, is more transparent, and readily accommodates new data and improved understanding of seismic behavior.

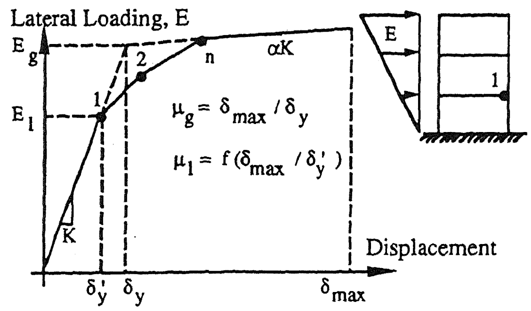


Figure 1. Lateral load-displacement response of a structure

2 DESIGN PROCEDURE

The inelastic multi-linear lateral load-displacement response of a structure subjected to a static incremental load can be idealized as a bilinear system as shown in Figure 1. The multi-linear curve marks the formation of different plastic hinges throughout the structure until a mechanism develops. The objective of the proposed design procedure is to estimate the required lateral load capacity, E_g, of the structure in order to limit the global (structure) ductility demand μ_g to a pre-determined target value, that will in turn limit the local (member) ductility demands μ_l to their corresponding capacities.

The evaluation of ductility demands imposed on different structural elements for a given global ductility demand is non-trivial, as it strongly depends on the sequence of redistribution of loads in the structure as plastic hinges form. A step-by-step non-linear incremental static analysis is required to estimate the relationships between local and global ductility demands. More realistically, although not currently feasible for design purposes, these relationships should be based on the dynamic response of structure during severe earthquakes. Further research is warranted in this area.

Basic information on the inelastic dynamic behavior of real structures are often derived from simplified SDOF models. In the proposed approach the SDOF model is defined by the first mode period of the structure, T, and the strain hardening ratio α as shown in Figure 1. Representation of the structure by a simple SDOF system serves the following purposes:

- Permits the use of the extensive database of information available for SDOF systems.
- Simplifies the design procedure by separating the multi-mode effect of MDOF system from other parameters. The multi-mode effect is dealt with later.
- Permits bench marking this procedure with the one already implemented in current seismic codes.

With regards to the assessment of strength and ductility demands for bilinear and simplified stiffness degrading models, results are not very sensitive to the hysteretic model used (Nassar & Krawinkler (1991)). However, this conclusion cannot be generalized for strength degrading models.

Figure 2 schematically shows the implementation of the proposed seismic design procedure summarized as follows:

(a) The ductility capacity of critical structural elements is estimated first taking into account the cumulative damage effects of inelastic excursions experienced by these elements during severe ground motions. This can be achieved by using an appropriate cumulative damage index, e.g., the Normalized Hysteretic Energy (NHE = HE / F_y δ_y) shown in Figure 2(a).

(b) Assuming the cumulative damage effect for critical structural elements can be represented by bilinear SDOF systems, the ductility capacity of these elements can be estimated for a given "tolerated" damage level (dashed curve in Figure 2(a)). The corresponding global ductility capacity is a function of the most critical element (local) ductility capacity as illustrated in Figure 1. Figure 2(b) illustrates the global ductility capacity spectrum, where the period T is the fundamental period of the structure.

(c) Knowing the target ductility ratio μ_g (denoted μ from here on) and the period T, the strength reduction factor $R_y(\mu)$ (defined as the ratio of elastic to inelastic strength demands for an SDOF system for a given target ductility ratio μ; $R_y(\mu) = F_{y,e} / F_y(\mu)$) can be derived from the developed R-μ-T relationships for bilinear SDOF systems discussed later (Figure 2(c)).

(d) The inelastic strength demand for the SDOF system can be obtained by scaling smoothed ground motion spectra (e.g., the ATC-3-06 (1978) ground motion spectrum) by the derived strength reduction factor, $R = R(\mu,T)$ (Figure 2(d)).

(e) Because of participation of higher modes and concentration of energy dissipation in some critical elements of the structure, the obtained SDOF inelastic strength demand has to be modified by system dependent modification factors, to account for the multi-mode effects of MDOF structures (Figure 2(e)). On the average, structures designed to this base shear demand would experience ductility demands that do not exceed ductility capacities in critical structural elements during severe earthquakes.

(f) Finally, recognizing that the design profession prefers to perform elastic strength design, a reduced base shear demand (denoted by E_l in Figure 2(f)) is used for proportioning strengths and stiffnesses of structural members, assuming an appropriate lateral force distribution along the height of the structure. The ratio E_g/E_l defines the "overstrength" of the structure resulting from inelastic redistribution of internal forces. This step requires an estimation of this overstrength ratio in order to define E_l. The stiffness and strength requirements for inter-story drift limitations imposed by the serviceability limit state are reconciled at this stage with those imposed by the collapse limit state. A nonlinear incremental load analysis is then performed to verify that the structure has indeed the required global strength capacity (E_g) and that brittle elements are not overloaded (Osteraas & Krawinkler (1990)).

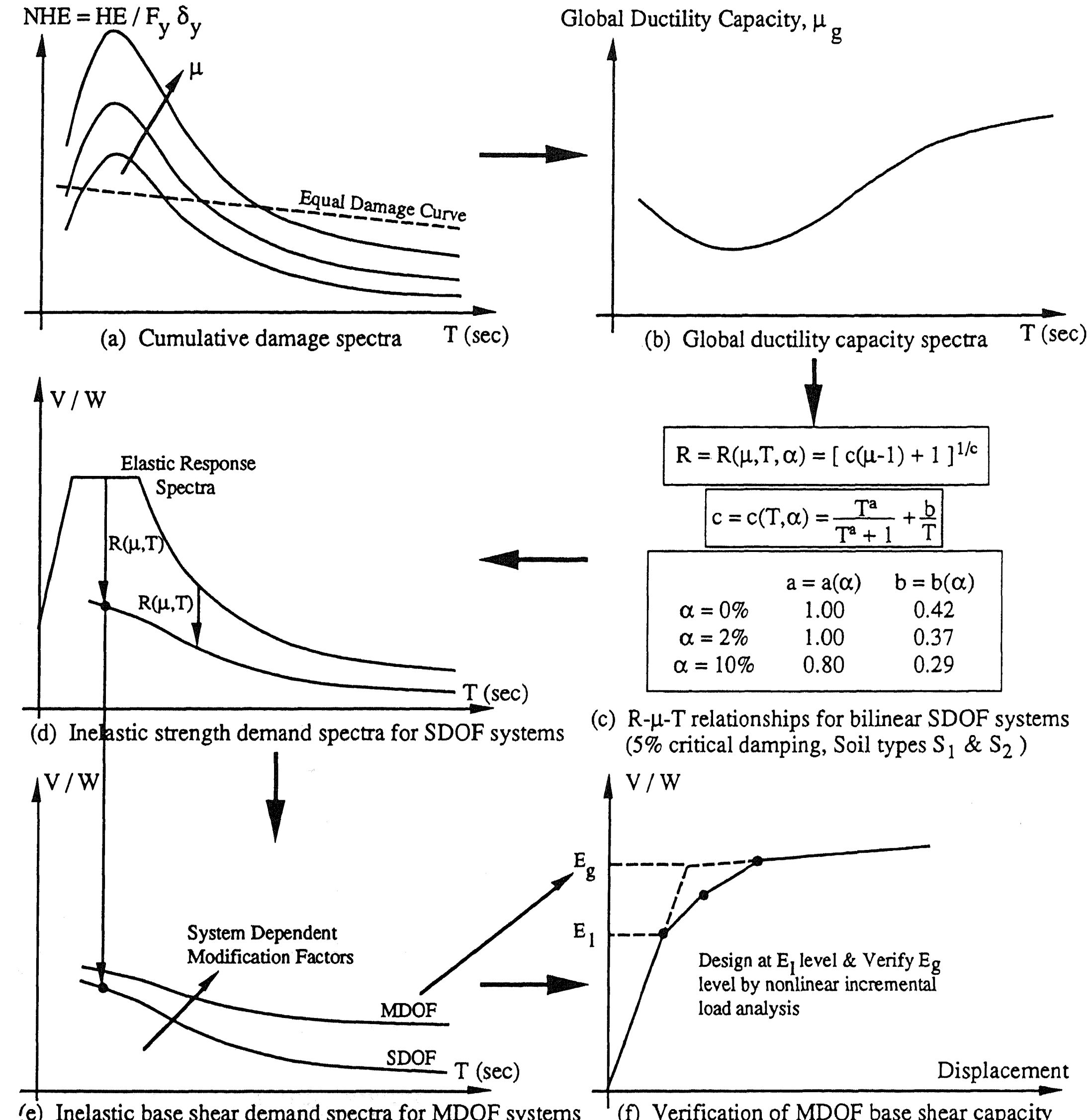


Figure 2. Implementation of the proposed seismic design procedure

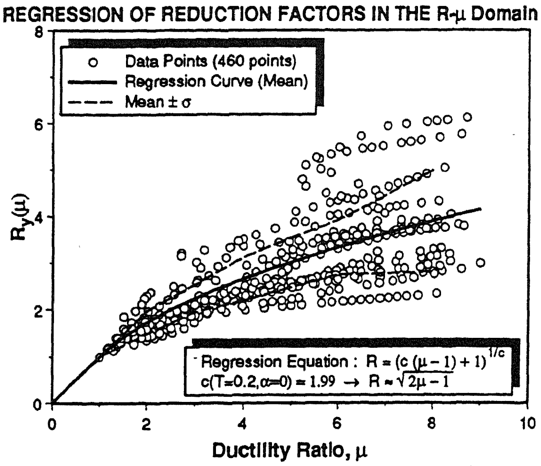
3 STRENGTH & DUCTILITY RELATIONSHIPS

The foregoing procedure is based on extensive knowledge of the relationships between strength (relative to elastic demand) and ductility demands for typical ground motions.

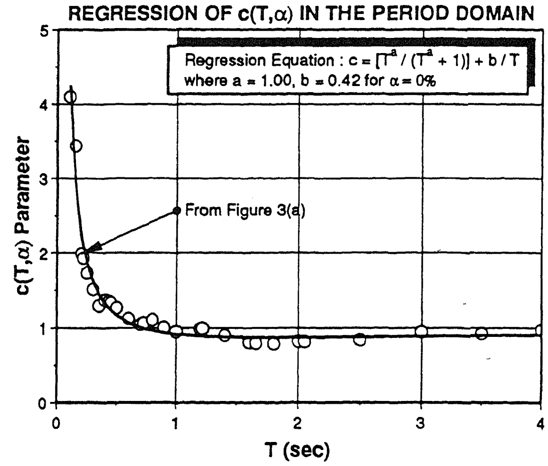
Statistical studies performed on 15 "typical" Western U.S. strong ground motions recorded on firm soils, for both SDOF and MDOF systems, provided the required relationships.

The R-μ-T relationships (referred to in Figure 2(c)), allow the derivation of inelastic strength demands for SDOF systems, given the elastic period T and target ductility ratio μ. A two-step nonlinear regression analysis was performed on the strength reduction factor $R_y(\mu)$, obtained from 39,000 time history analyses of bilinear SDOF systems with periods ranging from T = 0.1-4.0 sec, target ductility ratios μ = 1-8 and strain hardening ratios α = 0, 2 and 10%. Least square fit regression analysis was first carried out on R versus μ for constant periods T (Figure 3(a)). Then the effect of period was evaluated in the second step for different strain hardening ratios (Figure 3(b)). Figure 4 shows the 3-D perspective view (part (a)) and the three planes (parts (b) to (d)) of the developed R-μ-T relationships.

The effect of participation of higher modes for regular MDOF systems without closely spaced elastic modal periods is summarized in Figures 5 to 7. Figure 5

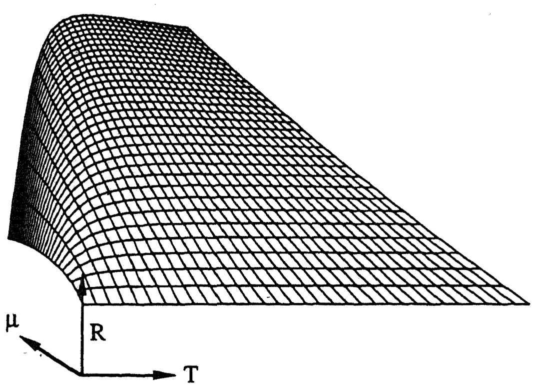


(a) R-μ regression for T = 0.2 sec, α = 0%

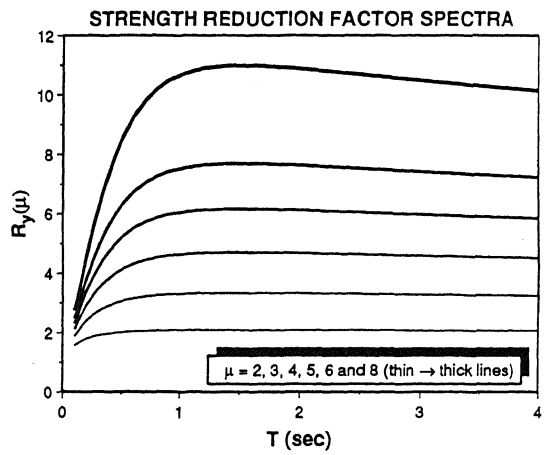


(b) c(T,α)-T regression for α = 0%

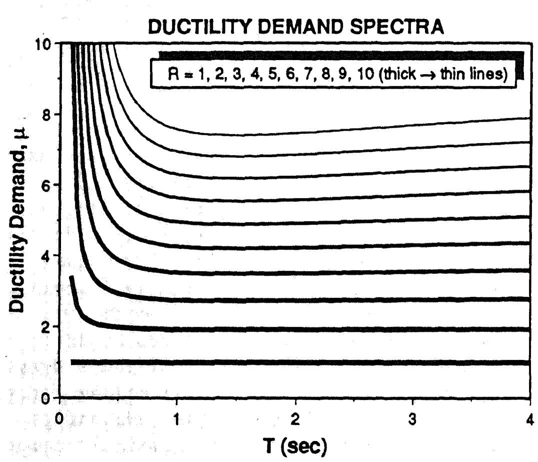
Figure 3. R-μ-T regression



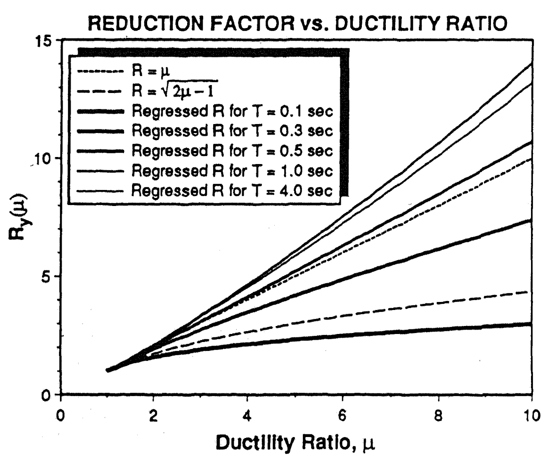
(a) A 3-D perspective view of the developed R-μ-T relationships



(b) R-T relationship



(c) μ-T relationship



(d) R-μ relationship

Figure 4. R-μ-T relationships

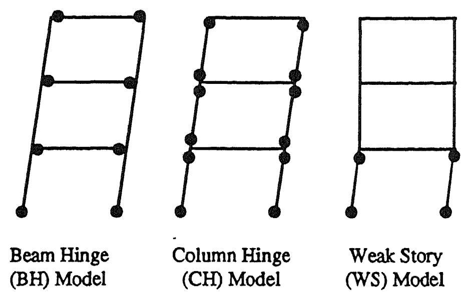


Figure 5. Types of structures used in the MDOF study

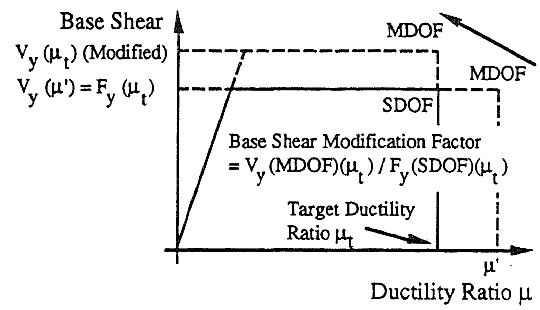
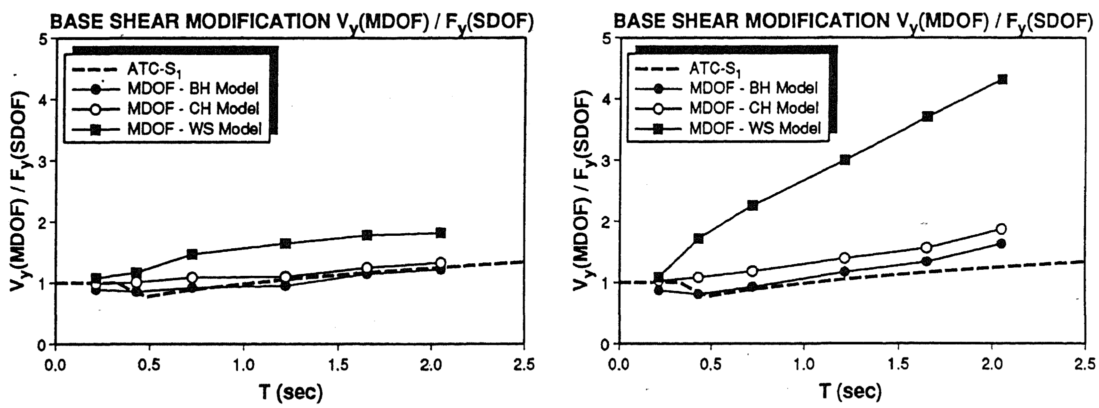


Figure 6. Modification in MDOF base shear demand to achieve equal ductility ratios in SDOF and MDOF systems



(a) $\mu_t = 2, \alpha = 10\%$ (b) $\mu_t = 8, \alpha = 10\%$

Figure 7. Base shear modification for same target ductility demand

shows the three distinct failure modes analyzed (Nassar & Krawinkler (1991)); namely, the beam hinge model, BH, representing structures in which plastic hinges form in beams only, the column hinge model, CH, representing structures in which individual story mechanisms can form, and the weak story model, WS, representing a structure with a strength (not stiffness) discontinuity in the first story.

Nonlinear dynamic time history analyses of these three structural systems were performed for heights ranging from 2-40 stories, subjected to 25 strong ground motions, for target ductility ratios μ_t = 1-8 and strain hardening ratios α = 0-10% (9,600 permutations in total). Relative story strengths and stiffnesses along the height of the structures were proportioned to story shears obtained from the UBC (1991) equivalent static lateral load distribution. The objective of the analysis was to study the story ductility, shear, and overturning moment demands for structures designed for a base shear equal to that estimated from SDOF systems (F_y(SDOF)) with periods equal to the fundamental period of the structure for a given target ductility ratio μ_t, subjected to the same strong ground motion. These results are then used to estimate what base shear capacity is required of MDOF systems (V_y(MDOF)) to limit the maximum inter-story ductility demand to the target value, μ_t (Figure 6).

Figure 7 shows two examples of the statistical average of the ratio of base shear demands on MDOF systems to those of the SDOF counterparts in order to limit the ductility demands in both systems to the same target values, μ_t. This ratio is used to scale the inelastic strength demand estimated from SDOF systems (Figure 2(e)). It is evident from Figure 7 that the ratio is dependent on the structural failure mechanism. Also, the ATC-3-06 modification, using $1/T^{2/3}$ decay instead of 1/T for the constant velocity portion of the elastic response spectra to account for multi-mode effects of MDOF systems (dashed lines in Figure 7), may be inadequate in many cases.

4 ASSESSMENT OF PRESENT CODE DESIGNS

The results discussed in the previous sections together with data presented in Osterass & Krawinkler (1990) and Nassar & Krawinkler (1991) can be used to provide a rough assessment of global ductility demands

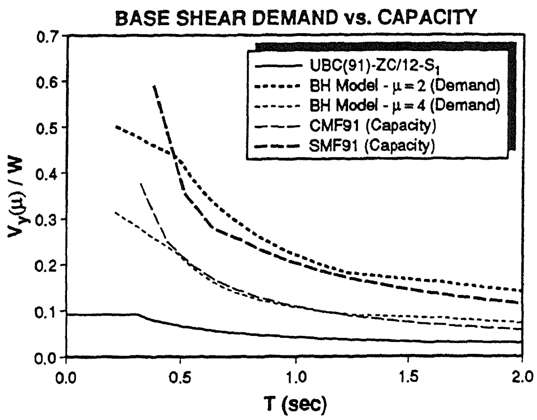


Figure 8. Comparison of base shear demands with estimated code capacities

anticipated for regular building structures designed according to the UBC (1991).

Using the procedure outlined in this paper, the smoothed ground motion spectrum on which the UBC base shear equation is based (for S_1 soil type and Z = 0.4), together with the R-μ-T relationships shown in Figures 2(c) and 4, and data on MDOF modifications of the type shown in Figure 7, can be utilized to compute the global strength demands for regular building structures for specific target ductility ratios. Two examples of the resulting strength demand spectra, for BH structures and target ductility ratios of 2 and 4 are shown in Figure 8. Also shown in this figure are estimates of the actual strengths of code designed steel and reinforced concrete moment resisting frame structures (SMF91 and CMF91), plotted against the first mode period given by the UBC. These strength estimates are derived considering various sources of overstrength, including effects of gravity loading, stiffness requirements, and inelastic redistribution of internal forces.

Figure 8 gives an indication of the period and system dependence of expected global ductility demands for code designed structures. Most notable is the great difference between demands for short and long period structures and between steel and reinforced concrete frame structures. Also, the code prescribed design base shear (solid curve in Figure 8) gives a distorted picture of base shear capacities of structures including overstrength. The main sources of these differences are the great variations in overstrength, an issue that is not considered in present code designs. This observation reinforces the argument that designs should be based on an explicit consideration of ductility demands and capacities, and estimates of structure strength that includes all important sources of overstrength.

5 SUMMARY AND CONCLUSION

Improved understanding of inelastic seismic demands for buildings provides the basis for examination of refined design procedure that more accurately and explicitly model structural response to seismic loading. Serviceability and collapse limit states should be addressed explicitly. For the collapse limit state, ductility capacity becomes the basic seismic design parameter. Relationships derived statistically from inelastic response of SDOF and MDOF systems provide a means of developing strength design criteria based on ductility capacity and other important structural characteristics (e.g., energy dissipation capacity). Current seismic design procedures, while implicitly addressing ductility capacity and demand, hide this critical aspect of seismic performance from the designer. The approach presented here, while more complicated than the current code approach, highlights the significance of ductility in seismic performance and provides the designer the means to explicitly evaluate demands and provide designs with requisite capacities.

ACKNOWLEDGEMENTS

This research is supported in part by the John A. Blume Earthquake Engineering Center at Stanford University, the Stanford/USGS Institute for Research in Earthquake Engineering and Seismology, the National Science Foundation, and a grant given by Kajima Corporation and administered by CUREe. The support provided by these organizations is gratefully acknowledged.

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