SEISMIC DEMANDS FOR SDOF AND MDOF SYSTEMS

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ABSTRACT

Earthquake resistant design has the objective of providing structural capacities that exceed the demands imposed by severe earthquakes by a sufficient margin of safety. It is anticipated that future seismic codes will incorporate capacity / demand concepts explicitly in order to make the design process transparent and permit designs with a well defined and consistent level of protection. This research is concerned with the assessment of seismic demand parameters that are needed to implement a capacity / demand based seismic design approach. The objectives of the study are: (a) to assess the importance of different demand parameters, (b) to evaluate patterns in demand parameters that will improve our understanding of the physical phenomena involved in seismic response of structures, and (c) to provide statistical information on demand parameters that can be utilized to assess the performance of structures designed according to existing codes and implement the proposed capacity / demand design approach.

A comprehensive evaluation of seismic demand parameters is performed for bilinear and stiffness degrading Single Degree of Freedom (SDOF) systems. A less comprehensive but much more elaborate study is also performed on three types of Multi-Degree of Freedom (MDOF) structures. The purpose of this aspect of the study is to evaluate the modification that must be applied to strength demand parameters derived from simplified SDOF models in order to account for multi-mode effects in real structures.

In the SDOF study, the inelastic strength and cumulative damage demands are evaluated statistically for specified target ductility ratios. Such a statistical study can be attempted only for ground motions with similar frequency characteristics, such as rock and firm soil motions recorded not too close and not too far from the fault rupture. Strength demands are represented in terms of inelastic strength demand spectra or spectra of strength reduction factors. Expressions are developed that relate the strength reduction factor to period and target ductility ratio. In the MDOF study, it is found that the required strength for specified target ductility ratios depends strongly on the type of failure mechanism that will develop during severe earthquakes. Quantitative information is developed on relative strength requirements for three types of MDOF structures, showing the disadvantage of structures in which story mechanisms develop, and particularly the great strength capacities needed to control inelastic deformations in structures with weak stories.
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CHAPTER 1
INTRODUCTION

1.1 Motivation of Study

In structural design, engineers have been and will be faced with the apparent conflict between safety and economy, practicality and creativity, functionality and elegance, simplicity and comprehensiveness of design. The aim of design codes is to guide the engineer in such important decision making and, therefore, codes need to be transparent and reflective of the important parameters affecting the design of structures. The basic philosophy of Load and Resistance Factor Design (LRFD), in which for specified limit states the factored "loads" or demands are not to exceed factored "resistances" or capacities with an acceptable probability, was a major step forward as compared to the traditional working stress design. The factors applied to demands and capacities ought to be a function of, but not limited to, variability and importance of the parameters considered, consequence of failure, importance of the structure, etc. Limit state could refer to "serviceability" of the structure or the "ultimate" limit state before the structure collapses. To this end, earthquakes bring a major challenge to the structural engineering profession. Questions that still remain open are: What do we mean by seismic demands and capacities? Which demand and capacity parameters are most important to be considered in design, yet simple enough to implement in a design code? What are the acceptable limit states for different structures at local and global levels? Such questions are difficult to answer, especially when dealing with inherently uncertain and inter-dependent parameters such as those which govern seismic performance of structures.

In concept, earthquake protection is a simple capacity / demand issue with the objective of providing capacities that exceed the imposed demands by a sufficient margin of safety. Seismic demands describe the severity of the effects imposed by earthquakes. In this context a demand parameter is defined as a quantity that relates seismic input (ground motion) to structural response. Thus, it is a response quantity obtained by filtering the ground motion through a linear or nonlinear structural filter. A simple example of a demand parameter is the acceleration response spectrum, which identifies the strength demand for an elastic single degree of freedom (SDOF) system. The peak ground acceleration (PGA) is not a demand parameter; it is merely a ground motion parameter
independent of the structure's response. Seismic capacities describe the ability of the structure to sustain the imposed seismic demands. An example is the ductility capacity which is a measure of the maximum deformation capacity of a critical element or of the structure as a whole.

Present code designs are based on the use of empirical coefficients that obscure the design process and lead to inconsistent designs with unknown levels of protection. Even though present codes have served the profession well, the need exists for improvements in their design approach. These improvements should explicitly incorporate capacity / demand concepts which are transparent to the design process and provide consistent levels of protection for the given limit states. Such a design approach could take on the format summarized in Section 2.1. This capacity / demand based design approach formed the motivation for this study, which is intended to provide basic information needed for its implementation.

1.2 Objectives and Scope

This study is concerned with the assessment of seismic demand parameters that are needed to implement a capacity / demand based seismic design approach. Capacity issues are not addressed in this study, but it is assumed that ductility capacity is the basic design parameter. For reasons discussed in Chapter 2, inelastic strength and cumulative damage demands, including energy demands, then become the relevant demand parameters. The objectives of the study are: (a) to assess the importance of different demand parameters, (b) to evaluate patterns in demand parameters that will improve our understanding of the physical phenomena involved in seismic response of structures, and (c) to provide statistical information on demand parameters that can be utilized to assess the performance of structures designed according to existing codes and implement the proposed capacity / demand design approach.

Three sets of ground motion records are used in this research for the evaluation of seismic demand parameters. The first one is a set of 36 records from the 1987 Whittier Narrows Earthquake, which is used to study the attenuation characteristics of ground motion and demand parameters. The second one is a set of 15 records representative of ground motions recorded for soil type $S_I$ from Western U.S. earthquakes. This set is used to provide statistical information on demand parameters. The third set is a subset of the first one, consisting of 10 records taken at distances less than 30 km from the Whittier
Narrows epicenter. The results from the last data set are compared to those from the second one in order to assess differences in demand parameters derived from either a single earthquake or an ensemble of different earthquakes.

A comprehensive evaluation of demand parameters is performed on bilinear and stiffness degrading single degree of freedom (SDOF) systems. A less comprehensive but much more elaborate study is also performed on three types of multi-degree of freedom (MDOF) structures. The purpose of this study is to evaluate the modification that must be applied to strength demands derived from simplified SDOF models in order to account for multi-mode effects in real structures.

The seismic demand information derived from the Whittier Narrows Earthquake are used, together with strength capacity estimates of several types of code designed structures, to assess the damage potential of the Whittier Narrows ground motions. The term damage potential is used to describe the ability of the ground motions to inflict damage on structures. For code designed structures, the ductility demands associated with the strength capacity estimates are used as measure of damage. This aspect of the study is intended to demonstrate that the capacity / demand concept can be also utilized to assess the performance of existing structures.

It must be emphasized that this study focuses only on a small part of a big problem. The seismic demands are evaluated only for selected ground motions in rock and firm soils. No conclusions can be drawn on demands imposed by ground motions on structures located on soft soils. The great importance of the soil site conditions was clearly demonstrated again by the 1989 Loma Prieta Earthquake. Thus, much more work needs to be done in the context of demand evaluation for seismic design, considering that input ground motions in general are sensitive to source mechanism, source-site distance, orientation, travel path through geologic media, and local site conditions.
CHAPTER 2
SEISMIC DESIGN ISSUES

Current seismic code design is based on elastic "strength demand spectra" (e.g., the product of $ZCW$ in the U.S. 1988 $UBC$), which are scaled to design base shear spectra by means of system dependent but usually period independent reduction factors ($R_w$ in the 1988 $UBC$ or $R$ in the $ATC-3-06$). The elastic strength demand spectra are modified versions of smoothened soil dependent ground motion spectra (see Fig. 2.1), the primary modification being the raising of long period ordinates to account for multi-mode effects and provide more safety for multi-story structures. Even though designs based on this approach appear to be satisfactory, there are several conceptual problems with this approach. First, it is well established that the reduction factor ($R$-factor) depends on the system ductility ratio and period. Second, ductility capacities, whether at the global (structure) or local (member) level, are not explicitly addressed in the code. Third, the issue of damage control (serviceability) during moderate earthquakes is not separated from design against collapse during severe earthquakes. Fourth, the ductility demands that code designed structures may experience during severe earthquakes are not proportional to the relative $R$-factors because of the great variation in "overstrength" (reserve strength) in structures.

It is desirable that seismic design codes adopt a different and more transparent design approach that permits tuning of the design to the ductility capacities of different structural systems and elements that control seismic behavior. In addition, the issue of damage control deserves special consideration and should be separated from that of safety against collapse. The following section summarizes a proposed design approach that forms the motivation of this study on seismic demands for $SDOF$ and $MDOF$ systems.

2.1 Seismic Demand / Capacity Concept (Design Methodology)

The proposed seismic design approach can be concisely summarized as follows: A simple transparent reliability-based dual level limit state design approach, the objective of which is to provide capacities that exceed seismic demands by an adequate margin of safety.
The proposed design approach has to be simple to be adopted in a design code and transparent to the design process in which the designer has to explicitly consider demands versus capacities. The objective of design is to provide capacities that exceed seismic demands by an adequate margin of safety, taking into account the uncertainties inherent in both capacities and demands. Therefore, a reliability-based design approach is warranted. The serviceability limit state should be separated from the collapse limit state by using two design earthquake levels with different probability of occurrence (representing moderate and severe earthquakes, respectively). Although the designs for these two limit states are separated, both designs follow the same basic concept and a reconciliation of generated design constraints is done in the process (see Osteraa and Krawinkler, 1990). This study is not concerned with the issue of damage control (serviceability). It focuses only on design for safety against collapse during severe earthquakes.

In the design for safety against collapse, it is postulated that the ductility capacity of the critical structural elements be the basis for seismic design. For "brittle" elements (e.g., many types of connections, vertical load carrying columns that may buckle, etc.) the ductility capacity is 1.0, and for "ductile" elements it is a quantity to be determined from experimental / analytical studies. For the latter elements, the ductility capacity is a function of the load-deformation (hysteresis) history, i.e., the number, magnitudes and cumulative damage effects of all inelastic excursions (Krawinkler et al., 1983). It is proposed that in the design procedure, the element ductility capacity be modified (weighted) to account for anticipated cumulative damage effects. This points out the importance of strong motion duration, frequency content of ground motions, period and structural redundancy of the structure, since they all affect the number and magnitudes of inelastic excursions, which in turn determine the cumulative damage experienced by a structural component.

Fig. 2.2 schematically shows the proposed design approach. In order to derive design strength requirements, the element ductility capacities have to be transformed into story ductility capacities (sometimes a simple geometric transformation - see footnote on page 12), which are then used to derive "inelastic strength demands" for design (discussed later). The so derived strength demands identify the required ultimate strength of the structure. Recognizing that the design profession prefers to perform elastic rather than plastic design, the structure strength level may be transformed to the member strength level in order to perform conventional elastic strength design (by assuming a ratio of the ultimate strength of the structure to the strength level associated with the end of elastic response, shown as $E_g$ and $E_t$, respectively, in Fig. 2.2). Pilot studies have shown that this
transformation is usually not difficult but may require an iteration (Osteraas and Krawinkler, 1990). After this preliminary design an important step is design verification through a nonlinear static incremental load analysis (using a rational equivalent static load pattern in a "push-over" loading) to verify that the required structure strength ($E_g$) is achieved and to "assure" that brittle elements are not overloaded (ductility demand < 1.0).

There is much research to be done in order to implement this design approach, and the study summarized here is nothing but a small step in providing some of the basic concepts and data needed for this purpose. Clearly, there are many difficulties associated with such a design approach, some of which are listed as follows:

- Seismic demands and capacities are not well defined at this time. Demands are functions of capacities and they cannot be separated.

- Seismic demands depend on many parameters that cannot be all addressed at the same time without unduly complicating the design process.

- What are the acceptable limit states for different structures at the element (local) or structure (global) levels (not part of this study) ?

- How do results from SDOF systems relate to those of MDOF systems ?

It is hoped that this research will help address some of these points. Fig. 2.3 shows a step-by-step implementation of the proposed design approach which forms the backbone of this study. The implementation is summarized as follows:

- First, the structure ductility capacity needs to be weighted (modified) to account for the cumulative damage effects of earthquake ground motions (see Chapter 4). This can be achieved, for instance, by using an appropriate cumulative damage index, e.g., the Normalized Hysteretic Energy ($NHE = \frac{HE}{F_y\delta_y}$) spectra for constant ductility ratios and assuming a constant damage level as illustrated at the top of Fig. 2.3. Thus, the cumulative damage effects are indirectly incorporated in the design process by estimating the ductility capacity of the structure modelled as an SDOF system. Further modification of the ductility capacity may be employed to account for the structural redundancy, consequence of failure and importance of the structure. Much more research needs to be directed to this area.
• Using the fundamental period of the structure, \( T \), and its weighted ductility capacity, \( \mu \), the reduction factor, \( R \), can be evaluated from the \( R-\mu-T \) relationships developed in Chapter 4 assuming the structure can be modelled as an \( SDOF \) system. This reduction factor can then be used to scale the elastic strength demand spectra (i.e., the ground motion spectra) to obtain inelastic strength demands (see Chapter 4).

• System dependent modification factors are then applied to the derived \( SDOF \) inelastic strength demands to account for the multi-mode effects in \( MDOF \) systems (see Chapter 5). This step defines the structure strength demand, \( E_s \), which defines the strength capacity required in order to limit the ductility demands on the structural elements to the target ductility capacities.

• The local strength demand (associated with the end of elastic response), \( E_l \), is then estimated from the structure strength demand (a process which may require an iteration), the structure is designed employing conventional elastic strength design, and a nonlinear incremental load analysis is carried out to verify that the structure has indeed the required strength capacity and that brittle elements are not overloaded (ductility demand < 1.0) (see Osteraas and Krawinkler, 1990).

Much fundamental information needs to be developed on the following aspects in order to permit the described implementation:

1. Experimental data on the ductility capacities for different structural elements (not part of this study).

2. Statistical data on cumulative damage demand parameters needed to modify (weigh) ductility capacities.

3. Statistical data on inelastic strength demands for prescribed ductility capacities, using \( SDOF \) systems.

4. Statistical data on the effects of higher modes in \( MDOF \) systems, needed to modify the inelastic strength demands derived from \( SDOF \) systems.

The determination of seismic demands for prescribed ductility ratios is essential for the proposed design approach, since the ductility capacity is the basis for design.
In this study several sets of ground motion records (discussed in detail in Chapter 3) are utilized to evaluate the seismic demand parameters listed in points 2 to 4 above. Extensive nonlinear dynamic time history analyses were performed on SDOF and simplified MDOF systems to better understand and assess the importance of different seismic demand parameters for ground motions recorded on rock and firm soils. The seismic demand parameters analyzed in this study are summarized in the next section.

2.2 Seismic Demand Parameters

Seismic demands represent the requirements imposed by ground motions on relevant structural performance parameters. In a local domain this could be the demand on the axial load of a column or the rotation of a plastic hinge in a beam, etc. The localized demands depend on many local and global response characteristics of structures, which cannot be considered in detail in a study that is concerned with a global evaluation of seismic demands. In this study, only SDOF and simplified MDOF systems are used as structural models. Assuming that these models have a reasonably well defined yield strength, the following basic seismic demand parameters play an important role in assessing the damage potential of ground motions. All parameters discussed below were studied for both bilinear and stiffness degrading hysteresis models (see Section 4.1), except the cumulative damage parameters which were evaluated for bilinear systems only. Some of the terms used in these definitions are illustrated in Fig. 2.4.

**Elastic Strength Demand,** $F_{y,e}$. This parameter defines the minimum yield strength required of the structural system in order to remain elastic during a given earthquake ground motion. For SDOF systems, the elastic acceleration response spectra provide the needed information on this parameter, i.e., $F_{y,e} = ms_a$ (where $m$ is the mass and $s_a$ is the acceleration response spectral ordinate for the given elastic SDOF system).

**Ductility Demand,** $\mu$. This parameter is defined as the ratio of maximum deformation to the yield deformation for a system with a yield strength smaller than the elastic strength demand $F_{y,e}$.

**Inelastic Strength Demand,** $F_{\mu}$. This parameter defines the yield strength required of an inelastic system in order to limit the ductility demand to a target value of $\mu$. Note that the terms demand and capacity can be used interchangeably and are
inter-dependent, i.e., the system has either a strength capacity $F_y(\mu)$ and experiences a ductility demand $\mu$ or a ductility capacity $\mu$ imposing a strength demand $F_y(\mu)$.

**Strength Reduction Factor, $R_y(\mu)$**. This parameter defines the reduction in the elastic strength of the system that will result in a ductility demand of $\mu$. Thus, $R_y(\mu) = F_{y,e} / F_y(\mu)$. The strength reduction factor, or the $R$-factor, can be thought of as an effectiveness factor; the larger the $R$-factor, the smaller the inelastic strength demand.

**Energy Demands.** Repeated cyclic loading is known to have a detrimental effect on the inelastic response characteristics of a system. There are many attempts reported in the literature on the assessment of cumulative damage effects through energy terms or specific cumulative damage models. The energy terms evaluated in this study are:

- **Input Energy, $IE$**:
  - The total *absolute* energy imparted to the system by the input ground motion.

- **Damping Energy, $DE$**:
  - The energy dissipated in the system through viscous damping.

- **Hysteretic Energy, $HE$**:
  - The energy dissipated in the system through inelastic deformations.

- **Total Dissipated Energy, $TDE$**:
  - $TDE = DE + HE$.

The other two related energy terms are:

- **Kinetic Energy, $KE$**:
  - The energy associated with the *absolute* velocity of the system.

- **Recoverable Strain Energy, $RSE$**:
  - The recoverable elastic potential energy absorbed by the system.

At any instance, the following relationship holds true:

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1 The definition of the input energy depends on that of the kinetic energy, i.e., whether the absolute (total) or relative velocity is used. In this study the absolute input and kinetic energies are used. For more details see Uang and Bertero, 1988.
\[ IE = DE + HE + KE + RSE \] (2.1)

This energy equilibrium equation comes from integrating the equation of motion with respect to the relative displacement, i.e.,

\[ \int m\ddot{x}_t \, dx + \int c\dot{x} \, dx + \int f_s \, dx = 0 \] (2.2)

where
- \( m \) = mass of the system
- \( c \) = coefficient of viscous damping
- \( f_s \) = restoring force \((f_s = kx \text{ for elastic systems only})\)
- \( k \) = elastic stiffness
- \( x_t = x + x_g \) = absolute (or total) displacement of the mass
- \( x \) = relative displacement of the mass with respect to the ground
- \( x_g \) = earthquake ground displacement

The first term in Eq. (2.2) gives \((KE - IE)\), the second gives \(DE\), while the third gives \((HE + RSE)\). The energy terms discussed are defined as follows:

\[ DE = \int c\dot{x} dx = \int c\dot{x}^2 dt \] (2.3)

\[ HE = \int f_s dx - RSE \] (2.4)

\[ RSE = \frac{(f_s)^2}{2k} \] (2.5)

\[ KE = \frac{m(x_t)^2}{2} \] (2.6)

\[ IE = \int m\ddot{x}_t dx_g = DE + HE + KE + RSE \] (2.7)

**Cumulative Damage Demands.** Many cumulative damage models have been proposed in the literature. Chung et al., 1987, provides a summary of these models. One of the simple cumulative damage models (Krawinkler et al., 1983) is of the form:
\[ D(C,c) = C \sum_{i=1}^{N} (\Delta \delta_{pi})^c \]  

(2.8)

where \( D \) = cumulative damage index  
\( C, c \) = structural performance parameters  
\( N \) = number of inelastic excursions experienced during the earthquake ground motion  
\( \Delta \delta_{pi} \) = the plastic deformation range of excursion \( i \) (see Fig. 4.33)

For bilinear systems, the expression given in Eq. (2.8) reduces to the hysteretic energy if the coefficient \( C \) is taken as the yield strength \( F_y(\mu) \) and the exponent \( c \) is taken as 1.0.\(^1\) For components of steel structures, the exponent \( c \) was found to be in the range of 1.5 to 2.0 (Krawinkler et al., 1983). The coefficient \( C \) varies widely and strongly depends on the performance characteristics of the structure. In the comparative study performed here, \( C \) can be eliminated and the plastic deformation ranges \( \Delta \delta_{pi} \) can be normalized with respect to the yield deformation \( \delta_y \). Thus, relative damage can be evaluated from the following expression:

\[ D(c) = \sum_{i=1}^{N} (\Delta \delta_{pi} / \delta_y)^c \]  

(2.9)

This expression is evaluated for exponents of \( c = 1.0, 1.5, \) and 2.0, in this study. For more details on this and other cumulative damage parameters studied, see Section 4.4.

2.3 Strength Capacity for Code Designed Structures

The standard procedure in U.S. seismic design codes is to prescribe equivalent static lateral loading, with a design base shear and a rational distribution of forces over the height of the structure, for designing structures for earthquake effects. In the 1988 SEAOC Blue Book (SEAOC, 1988), the design base shear is derived by dividing a modified elastic

\(^1\) This statement is strictly true only for bilinear systems with no strain hardening. For systems with strain hardening there can be some discrepancies depending on how unsymmetric the hysteresis loops are with respect to positive and negative yielding. The error, for a strain hardening of 10\%, is typically less than 1\% for the results presented in this study.
strength demand by a reduction factor $R_w$, which results in a base shear that is to be combined with gravity and other loads at the allowable stress design level. The design base shear is much lower than the elastic strength demand (the $R_w$ factor is as high as 12) because it is recognized, and implicitly considered in the codes, that for many reasons the element and structure strengths are larger than needed to resist the design base shear alone and that structures will respond inelastically during severe earthquakes, thus reducing the dynamic forces attracted by the structure.

The actual strength capacities of the elements and the structure as a whole are needed to be able to assess the performance of different structural systems during earthquakes. These are the strength capacities needed to identify the ductility demands and other seismic demands caused by ground motions. As the following discussion shows, the current seismic codes do not provide consistent reserve strength capacities for different structural systems. This section is devoted to the issue of actual strength capacities versus design strength for simple generic structures to help understand the implications associated with designing these structures to code design levels.

In its simplest form, the lateral load - lateral interstory displacement can be modelled as shown in Fig. 2.5, where $E$ represents the design base shear given by the seismic code, $E_l$ represents the lateral strength capacity associated with the strength capacity of the weakest member of the structure (end of elastic response of the structure), and $E_g$ represents the lateral strength capacity of the structure\(^1\). All increases in lateral capacity beyond the seismic design force level $E$ are denoted here as overstrength. The ratio $E_l/E$

\(^1\) For example, the trilinear load - displacement response in Fig. 2.5 may represent the response of an x-braced frame, where $E_l$ is associated with the capacity of the compression member and $E_g$ is associated with the tensile capacity of the other member. Element ductility demand refers to the extensional ductility demand on the braces. For this simple x-braced system, the story displacement and brace elongations are linearly related and, therefore, the element ductility ratio is given by the ratio $\frac{\delta_{max}}{\delta_y}$ as shown in Fig. 2.5. This ratio, denoted as the local ductility ratio $\mu_l$, is used here to assess member ductility demands even though it is understood that this may be an approximation for many cases which are not as simple as the x-braced system used for illustration.
is viewed as a member (local) overstrength factor, and $E_g/E$ as a structure (global) overstrength factor.

The overstrength present in real structures, the difference between $E$ and both $E_l$ and $E_g$, varies widely, depending on the material and type of the structural system, the structural configuration, the number of stories, detailing, and the kind and date of the code to which the structure was designed. Overstrength may come from a variety of sources, of which the following is an incomplete list:

1. Difference between the design level and required member strength (e.g., allowable versus yield stresses in steel, load factors in reinforced concrete).

2. Effects of other loads in load combinations (e.g., effects of gravity loads on the required member strength).

3. Effects of discrete member sizes (e.g., available steel sections).

4. Effects of underestimating member strength capacities in the design process (e.g., conservative models for predicting member strength, actual versus nominal material strength properties).

5. Effects of code minimum requirements (e.g., minimum reinforcement ratio in shear walls).

6. Effects of stiffness (drift) requirements on member strength (e.g., many steel moment frames are governed by drift limitations).

7. Effects of desired uniformity of members for constructibility (e.g., equal sizes of steel braces or equal thickness of shear walls over several stories).

8. Architectural considerations (e.g., excess shear wall area).

9. Effects of structural elements that are not considered as part of the lateral load resisting system (e.g., columns in flat plate structures with shear walls).

10. Effects of nonstructural elements (e.g., nonstructural infill walls).

11. Redistribution of internal forces in the inelastic range (e.g., strength difference between the formation of the first plastic hinge and a mechanism
in frame structures, difference between strength of braces in tension and compression).

It is not within the scope of this study to evaluate all these contributions to overstrength in detail; in fact, it cannot be done accurately except on a case-by-case basis. Nevertheless, the following discussion presents an attempt of a rough assessment of overstrength for several types of code designed structures. The work on steel structures summarized here has already been reported elsewhere (Osteraas and Krawinkler, 1990), but the work on reinforced concrete moment resisting frame structures is new and is discussed in more detail.

It must be emphasized that the presented values for strength capacity are rational, but rough, estimates based on a series of assumptions that may not always apply. The real strength of specific structures may differ considerably from the presented values. More often than not the estimates will be on the low side because not all of the listed sources of overstrength could be considered. The presented information has to be viewed in the context of a global assessment of seismic demands versus capacities.

The following discussion pertains to estimates of the strength capacities \( E_l \) and \( E_g \) of several types of steel and reinforced concrete structures. These structures are referred to as "generic" structures although it is recognized that no two structures are alike and significant variations may exist in properties of apparently similar structures. The objective is to get a feeling for the strength capacity and overstrength of widely used lateral load resisting systems for buildings with different numbers of stories (or natural periods).

Only those structural systems which lend themselves to a systematic evaluation of overstrength were selected for this study. Such systems include moment resisting and braced frame structures. Shear wall and masonry structures had to be excluded because of their great variations in lateral resistance.

All types of generic structures selected in this study have the same plan view, consisting of three 24 ft bays in the \( N-S \) direction and five 24 ft bays in the \( E-W \) direction. The plan view is shown in Fig. 2.6. The strength of the structures was evaluated in the \( N-S \) direction only. The following types of generic structures were selected:

1. \textit{CMF88}. Reinforced concrete moment resisting frame structures designed according to the 1988 \textit{SEAOC} Blue Book (\textit{SEAOC}, 1988) with \( R_w = 12 \), in which every frame is part of the lateral load resisting system.
2. **CMF68.** Reinforced concrete moment resisting frame structures designed according to the 1968 *SEAOC* Blue Book (*SEAOC*, 1968) with \( K = 0.67 \), in which every frame is part of the lateral load resisting system.

3. **SMF88.** Steel moment resisting frame structures designed according to the 1988 *SEAOC* Blue Book with \( R_w = 12 \), in which every frame is part of the lateral load resisting system.

4. **SPF88.** Steel perimeter frame structures designed according to the 1988 *SEAOC* Blue Book with \( R_w = 12 \), in which only the perimeter frames contribute to lateral resistance.

5. **SBF88.** Steel structures in which lateral resistance is provided by x-braced bays at the perimeter and designed according to the 1988 *SEAOC* Blue Book with \( R_w = 8 \). Such structures are permitted only to a height of 160 feet. The bracing of these structures is shown in dashed lines in Fig. 2.6.

All designs were performed according to the applicable code, using approximate methods of analysis to estimate the design strength and stiffness requirements. The following assumptions were made in the design process:

- The story height is 12 ft for all stories.
- The number of stories is varied from 2 to that value for which either the computed period exceeds 2.0 sec or a code height limitation is reached.
- The period of the structures is computed according to the applicable code.
- The lateral resistance is governed by behavior in the first or second story.
- For frame structures the strong column - weak girder concept is adopted.
- Reduced live loads are represented by a uniformly distributed load of 30 psf.
- Dead loads are assumed to be 70 psf for steel structures and 100 psf for reinforced concrete structures.
• For moment resisting frame structures, interior frames are assumed to represent the structure. For SBF88 structures, the bracing system is placed on the exterior frame lines.

• In the 1988 SEAOC designs, the soil factor $S$ was taken as 1.2 (soil type $S_2$), which better represents the alluvial soils on which the Whittier Narrows ground motions were recorded.

• Concentrated plasticity is assumed at the girder-column joints. No plastic hinges are allowed within the girder spans. No strain hardening was assumed once a plastic hinge develops.

These structures can be viewed as well designed structures, which follow basic code requirements, have no excessive waste in their member sizes, and have no undesirable features such as plastic hinges in columns or weak connections. The information presented in this section on strength capacities of generic structures, together with the information on inelastic seismic demands are utilized to assess the damage potential of strong ground motions (see Section 4.5).

Reinforced concrete moment resisting frame structures were designed according to the 1988 and 1968 SEAOC Recommendations. The main difference between the two designs lies in the different code equations used to estimate the period and design base shear coefficients. Compared to the design of generic steel structures, the following differences are noted:

• Dead loads are assumed to be 100 psf.

• Negative moments are calculated at the faces of columns, which are assumed to be 1 foot away from the column centerline.

• Drift does not govern the design of reinforced concrete structures.

• The minimum girder positive moment capacity at the support is 50% the negative moment capacity.

• The actual capacity of the girder section is assumed to be the nominal bending strength, excluding the $\phi$ factor.
• Girders are assumed to be sized to design bending strength requirement, i.e., no oversize factor was used.

• Load Resistance Factored Design (LRFD) versus Allowable Stress Design (ASD).

The girders were designed according to code prescribed load combinations and using standard design procedures. The negative design bending strength was always governed by the load combination \(1.4[D + L + E]\). The positive design bending strength of the girders at the column faces was governed by the requirement \(M^+ > 0.5M^-\) for 2 to 13 story \(CMF88\) and 2 to 20 story \(CMF68\) structures. For taller structures the load combination \(0.9D - 1.4E\) governed.

The overstrength was evaluated as shown in Fig. 2.7. The \(E_l\) strength is defined as the base shear at which the structure develops the first plastic hinge at one of the girder sections (numbered 1 to 6 in Fig. 2.7) of the first story. Unlike steel structures, the first plastic hinge may develop when the critical combination of unfactored gravity and superimposed seismic moments exceed either the positive or negative nominal bending strength capacity of the girders at the faces of the columns. The lateral strength at the \(E_l\) level is governed by the weakest section, the section with the minimum critical reserve moment. The critical reserve moment, at a given section, is the governing difference between the unfactored gravity and seismic moment, and the positive / negative nominal bending strength capacity of the section. The negative moments are critical where the lateral moments add to gravity and vice versa. For example, the critical reserve moments for Sections 2, 4, and 6 are the differences between moments due to full dead plus live plus design lateral load \([D + L + E]\) and the negative nominal bending strength capacity. For Sections 1, 3, and 5, the critical reserve moments are the difference between moments due to dead loads alone minus design lateral load \([D - E]\) and the positive nominal bending strength capacity. It turned out that the load combination \([D + L + E]\) governs at Section 6 for the \(E_l\) level for 2 to 5 story \(CMF88\) and 2 to 10 story \(CMF68\) structures. For taller structures, \([D - E]\) governs at Section 1.

The \(E_g\) strength is defined as the base shear at which the structure develops a story mechanism. At this stage, all girder sections at the first floor have developed plastic hinges. The \(E_g\) strength is governed by the sum of all the reserve moments in girders of the first story. It is independent of the sequence at which plastic hinges develop (for plastic
hinges with no strain hardening only). As the last sketch in Fig. 2.7 shows, it is quite possible that plastic hinges form within the span of the girders. However, this condition was not considered in this strength evaluation.

The details of the design of steel structures can be found in Osteraaas and Krawinkler, 1990. The yield stress of all structural elements was taken as 36 ksi. In the selection of member steel sections, a discrete member size factor of 1.1 was applied to account for the fact that steel members are available only in discrete sizes.

The design process for the steel moment frame structures (SMF88) is in most aspects similar to that of the CMF88 structures; the only major differences being the use of allowable stress design rather than strength (LRFD) design and the criterion that the negative girder bending strength at the column face is equal to, rather than half, the positive one. The theoretical bending strength was related to the allowable stress design moment by a factor of 1.13 \((F_Y / F_b)\), where \(F_b\) is the allowable bending stress and the factor 1.13 (Shape Factor = \(Z_x / S_x\)) accounts for the difference between yield moment and plastic moment capacity of the section.

The design process for the perimeter frame structures (SPF88) is identical to that for the SMF88 except that the girder tributary area is smaller for gravity loads (exterior perimeter frames versus interior moment resisting frames) and considerably larger for lateral loads (2 perimeter frames versus 6 moment resisting frames).

The less the gravity loads contribute to the total (gravity+lateral) design load effect, the smaller the overstrength factors \(E_I / E\) and \(E_g / E\) become. Provided everything else remains the same, braced frames have the smallest overstrength (since the bracing system carries no gravity loads) and interior moment resisting frames have the largest.

In the design of the braced frame structures (SBF88), the \(x\)-braces are located on the exterior frame lines. In the design of braces and columns, all important 1988 SEAOC criteria (reduced allowable compressive stress for braces, \(3R_w / 8\) requirement for columns, etc.) were considered. The \(E_I\) strength is based on buckling of the compression braces, and the strength capacity increase from \(E_I\) to \(E_g\) level is based on the difference between the tensile and compressive strength of the braces, disregarding post-buckling deterioration of compression braces. Because of the 160 ft height limitation, only structures up to 13 stories (\(h_n = 156\) ft) are considered. Using the period equation \(T = 0.02h_n^{0.75}\), this corresponds to a period of 0.88 seconds.
Results from this strength evaluation are summarized in Figs. 2.8 to 2.12. Fig. 2.8 shows the $E_l$ and $E_g$ strength capacities of the reinforced concrete frame structures, CMF68 and CMF88. The strength capacities are expressed as base shear coefficients. The figure shows a significant difference in strength capacities between the CMF68 and CMF88 structures at both the $E_l$ and $E_g$ levels. The corresponding overstrength factors $E_l/E$ and $E_g/E$, which identify the increase in strength above the seismic design force level, are shown in Fig. 2.9. The overstrength factors are larger for the CMF68 structures even though their strength capacities are smaller. The differences come primarily from the fact that the code seismic design force for the CMF68 structures is considerably smaller than that of the CMF88 structures. Fig. 2.9 clearly shows that the overstrength factors increase significantly for short period structures, the reason being the increasing effect of gravity loads, compared to lateral loads, on the girder design.

Fig. 2.10 shows the strength capacities $E_l$ and $E_g$ for the three types of steel structures, SBF88, SMF88 and SPF88. At the $E_l$ level (start of inelastic behavior) the SPF88 is stronger than the SMF88, but only because drift limitations become an overriding consideration. The strength of all SPF88 structures, regardless of the number of stories, is governed by drift, and by a larger margin for structures above 5 stories high (due to more stringent drift requirements beyond $T = 0.7$ sec). For the SMF88 structures, only those above 4 stories high are governed by drift. The $E_l$ strength of the SBF88 structures, in which drift was not a consideration, is not much higher than those of SMF88 and SPF88 structures.

Important conclusions can be drawn from the structure strength capacity $E_g$. For all periods, the SMF88 structures have the highest structure (global) strength. Their strength is considerably higher than that of the SPF88 structures even though seismic codes treat them both the same. The noted difference in strength has implications on the expected global ductility demands, as discussed in Section 4.5.

The most striking result is the low structure strength capacity ($E_g$) of the SBF88 structures. For all periods, this strength is lower than that of the SMF88 structures, even though the SBF88 structures are designed for a 50% higher base shear ($R_w = 8$ vs. $R_w = 12$). The simple reason is that SMF88 structures have a much higher overstrength than SBF88 structures. Again, this has major implications on the expected global ductility demands.
The relative amount of overstrength for the three types of generic steel structures can be evaluated from Fig. 2.11, which shows the overstrength factors $E_t / E$ and $E_g / E$. The overstrength is smallest for $SBF88$ structures and largest for $SMF88$ structures mainly because of the differences in tributary areas for gravity and lateral loads. For the same reason, the overstrength factors for $SMF88$ structures rapidly increase with decreasing period (decreasing number of stories).

In Fig. 2.12, the strength capacities $E_t$ and $E_g$ of the five types of generic steel and reinforced concrete structures are compared. The strength capacities of steel moment resisting frames ($SMF88$) are much larger than those of their reinforced concrete counterparts ($CMF88$). The large difference come from two sources. First, most of the $SMF88$ structures are governed by drift requirements, which increased the member strengths significantly. Second, the $CMF88$ structures have a smaller structure reserve strength because the positive bending strength of the girders involved in the mechanism is only half as large as the negative one, whereas in the $SMF88$ structures girder positive and negative bending strengths are equal.

$CMF68$ structures have the smallest strength capacities, whether at the global ($E_g$) or local ($E_t$) levels. The upper bound is defined by the $SMF88$ structures for the global strength capacities and by both $SBF88$ and $SPF88$ structures for the local strength capacities.

The rapid increase of strength capacities with decreasing period, noted in Fig. 2.12, is very beneficial in the context of code design. Statistical studies (discussed later) clearly show that, in order to obtain equal ductility demands, the required strength of structures in the short period range (about 0.5 sec and shorter) increases strongly with decreasing structural period. However, for this period range, most codes have a plateau value for the seismic base shear coefficient, that is, short period structures are designed for the same base shear regardless of period. Were it not for the rapid increase in strength capacities, short period structures would be subjected to excessive ductility demands.

Different conclusions may be derived if the number of stories is used instead of period. However, the comparison based on period is relevant since the seismic demands are dependent on period and not the number of stories. Fig. 2.12 shows the number of stories associated with each type of structure (shown as symbols for 2, 6, 10, 14 and 18 stories).
Base Shear Coefficient V/W

ATC Smoothened Ground Motion Spectrum for 5% Damping (Elastic Response Spectrum)

$\times 2.5$ (ATC) or $2.75$ (UBC)

R or $R_w$

Design Level

$\alpha \left( \frac{1}{T^{2/3}} \right)$

$\alpha \left( \frac{1}{T} \right)$

To account for MDOF Effects

$T$ (sec)

Fig. 2.1 Current Seismic Code Design Procedure
Fig. 2.2 Proposed Seismic Design Procedure
Fig. 2.3 Implementation of the Proposed Seismic Design Procedure
Fig. 2.4 Some Seismic Demand Parameters
Global Ductility Ratio \( \mu_g = \frac{\delta_{\text{max}}}{\delta_y} \)

Local Ductility Ratio \( \mu_l = \frac{\delta_{\text{max}}}{\delta_y} \)

\( = \mu_g \left( \frac{E_g}{E_l} \right) \)

Fig. 2.5 Simplified Structural Model

Fig. 2.6 Plan View of Generic Structures Used in this Study
STRENGTH CAPACITIES OF R/C MRF STRUCTURES

MOMENTS DUE TO GRAVITY LOADS

Design Level $E_L$

Base Shear = $E_L$

MOMENTS DUE TO LATERAL LOADS @ $E_L$ LEVEL

MOMENTS DUE TO GRAVITY + LATERAL LOADS @ $E_L$ LEVEL

INCREMENTAL MOMENTS FROM $E_L$ TO $E_g$ LEVEL

MOMENTS DUE TO GRAVITY + LATERAL LOADS @ $E_g$ LEVEL

Fig. 2.7 Development of Strength Capacities of Reinforced Concrete Moment Resisting Frame Structures
STRENGTH CAPACITIES $E_I$ & $E_g$ FOR CMF68 & CMF88

Concrete Structures

Base Shear Coefficients $E_I$ & $E_g$

$T$ (sec)

Fig. 2.8 Strength Capacities $E_I$ and $E_g$ for 1968 and 1988 Generic R/C Moment Resisting Frame Structures

OVERSTRENGTH FACTORS $E_I / E$ & $E_g / E$ FOR CMF68 & CMF88

Concrete Structures

Overstrength Factors $E_I / E$ & $E_g / E$

$T$ (sec)

Fig. 2.9 Overstrength Factors $E_I / E$ and $E_g / E$ for 1968 and 1988 Generic R/C Moment Resisting Frame Structures
STRENGTH CAPACITIES $E_I$ & $E_g$ FOR SBF88, SMF88 & SPF88

Steel Structures

Fig. 2.10 Strength Capacities $E_I$ and $E_g$ for 1988 Generic Steel Structures

OVERSTRENGTH FACTORS $E_I / E$ & $E_g / E$ FOR SBF88, SMF88 & SPF88

Steel Structures

Fig. 2.11 Overstrength Factors $E_I / E$ and $E_g / E$ for 1988 Generic Steel Structures
STRENGTH CAPACITY $E_i$ FOR CONCRETE & STEEL STRUCTURES

CMF68, CMF88, SBF88, SMF88, SPF88

Base Shear Coefficient $E_i$

$T$ (sec)

(a) $E_i$ Level

STRENGTH CAPACITY $E_g$ FOR CONCRETE & STEEL STRUCTURES

CMF68, CMF88, SBF88, SMF88, SPF88

Base Shear Coefficient $E_g$

$T$ (sec)

(b) $E_g$ Level

Fig 2.12 Comparison of Strength Capacities for All Generic Structures Used in this Study
CHAPTER 3
GROUND MOTION RECORDS

3.1 Introduction

In order to provide seismic demand information that should prove useful for direct application to design of structures located on rock or firm soils, three data sets of strong motion records were used in the nonlinear dynamic time history analyses for SDOF systems (see Chapter 4).

The first data set, discussed in detail in Section 3.2, is used to study the attenuation characteristics of the ground motion parameters and seismic demand parameters. This set includes ground motions from 36 strong motion stations, recorded during one earthquake, namely, the Whittier Narrows Earthquake of October 1, 1987, in and around Los Angeles, California.

The second and third data sets, discussed in detail in Section 3.3, are used for a statistical evaluation of patterns in seismic demand parameters. The influence of important factors, e.g., period, strain hardening and hysteresis models on these demand parameters are comprehensively analyzed. The second data set (see Subsection 3.3.2), includes 15 "typical" ground motions (denoted hereinafter as 15s records) from different Western U.S. earthquakes, recorded on firm soils (soil type $S_1$). The purpose of the third data set (see Subsection 3.3.3), which is a subset of the first data set with only 10 representative ground motions from the Whittier Narrows Earthquake (denoted hereinafter as 10w records), is to evaluate the dependency of statistical results on the characteristics of one earthquake compared to the average of different earthquakes. Also, the Whittier Narrows ground motions were recorded on alluvial soils which are closer to soil type $S_2$ than $S_1$.

The second and third data sets, the 15s and 10w records, were also used as input ground motions for studying the behavior of simplified MDOF systems in Chapter 5. The purpose of this part of the study is to evaluate the multi-mode effect of MDOF systems, evaluate the 1988 UBC equivalent static load pattern, and characterize the seismic demand parameters for three distinct types of MDOF models.
3.2 Whittier Narrows Ground Motions and their Attenuation

3.2.1 CSMIP (CDMG) Records

The Whittier Narrows Earthquake of October 1, 1987, provided an extensive set of ground motions and damage information. This data set is used to evaluate basic ground motion and seismic demand parameters, search for patterns in their characteristics, study their attenuation, and assess the damage potential of the Whittier Narrows Earthquake by superimposing seismic demand versus capacity, as discussed in Section 4.5.

The overriding consideration in the selection of records was that each record could be viewed as a "free-field" record. Therefore, only records from instrument shelters or single-story buildings were considered in order to avoid records that could be considerably contaminated by structural feedback. From the three extensive collections of records (CDMG, USGS, USC) that were obtained, only the CSMIP (California Strong Motion Instrumentation Program) stations maintained by CDMG (California Division of Mines and Geology) were utilized in this study. Very few of the USGS (U.S. Geological Survey) records qualify as "free-field" records, and the USC (University of Southern California) maintained records, from the Los Angeles Strong Motion Accelerograph Network, would have been made available only at a prohibitive cost.

Ground motions recorded at 36 CSMIP stations qualified as "free-field" records. The locations of these CSMIP stations as well as the epicenter of the October 1, 1987 Earthquake are shown in Fig. 3.1. The stations are numbered in alphabetical order (see Table 3.1). The majority of the stations are located to the west of the epicenter, which hampered plotting any comprehensive contour plots all around the epicenter for ground motion and seismic demand parameters of interest.

Table 3.1 summarizes basic information on the two horizontal components of the CSMIP records utilized in this study. The stations are ordered in increasing epicentral distance.

The two horizontal components of the acceleration and velocity time histories at each station were vectorially combined to give a 2-D vector trace. The orientation of the peak vector ground accelerations and velocities is illustrated in Fig. 3.2. The figure shows the spatial attenuation of accelerations and velocities, and clearly demonstrates a radial pattern of the vectors, with the epicenter as the focal point.
Acceleration, velocity and displacement time histories of two records are shown in Figs. 3.3 and 3.4. The Tarzana record, which has an exceptionally large PGA of 0.54g at an epicentral distance of 44 km, represents a predominantly harmonic motion in the acceleration, velocity and displacement time histories, with a period of 0.3 seconds (see Fig. 3.3). The harmonic nature of the motion is also evident from the ratios of PGA / PGV and PGV / PGD, which are very close to the angular frequency $\omega = 2\pi / 0.3 = 21$ radians / sec. The Tarzana site was not studied in detail, but the harmonic motion leads one to believe that there was considerable site amplification at the period of 0.3 seconds.

The Alhambra record, on the other hand, is a more typical record with a broader frequency band (see Fig. 3.4). The Alhambra record was taken at an epicentral distance of 7 km.

The harmonic characteristics of the Tarzana ground motion is also evident from the acceleration response spectra shown in Fig. 3.5 (a). The spectra exhibit a single peak at 0.3 seconds and very little energy content at longer periods. The spectra of the Alhambra record, shown in Fig. 3.5 (b), exhibit quite different characteristics which are more typical of acceleration response spectra.

The unique characteristics of the Tarzana record are also evident in the vector traces of the acceleration and velocity resultants of the two horizontal components. These traces, shown in Fig. 3.6, exhibit a predominant direction of motion in the South-East to North-West direction. It is speculated that seismic waves may have reflected off the deep depression of the bedrock in the Los Angeles basin, which runs along the South-East to North-West direction, converging towards Tarzana (see Fig. 3.7). All these characteristics, and an evaluation of all other records, lead to the conclusion that the Tarzana record is exceptional for this earthquake.

### 3.2.2 Attenuation of Ground Motion Parameters

The following ground motion parameters were evaluated for all 36 CSMIP records:

- **PGA**: Peak ground acceleration of record
- **RMSA**: Root-mean-square acceleration of record
- **PGV**: Peak ground velocity of record
\[ I : \text{ Arias Intensity of record} \]
\[ D_{sm} : \text{ Strong motion duration of the record} \]

The parameters used here are well defined, except for the strong motion duration \( D_{sm} \). Many different methods for evaluating the strong motion duration of an acceleration time history are proposed in the literature (e.g., Bolt, 1973, Trifunac and Brady, 1975, Vanmarcke and Lai, 1977, McCann and Shah, 1979). Several methods, which can be categorized as follows, were used for an initial evaluation of strong motion duration.

1. **Cut-off Methods (absolute or relative):** In these methods a threshold acceleration is used to determine the strong motion duration, using either an absolute value, e.g., 2, 5, or 10% \( g \) (Bolt, 1973), or a relative value such as 10% PGA. In all cases the strong motion duration is defined as the time between the first and last exceedance of the given threshold. In the literature, these methods are often referred to as bracketed methods; as a matter of preference this name is chosen to describe the method discussed next.

2. **Bracketed Method (Trifunac & Brady 1975):** In this method the integral of the square of the acceleration time history \( E(T) \) is plotted vs. time \( T \). This *monotonic* function, \( E(T) \), grows from 0 to a maximum \( E_m \), the maximum "energy" imparted by the earthquake at the end of the record (Arias Intensity = \( (\Pi/2g) \) \( E_m \), see Arias, 1970). The strong motion duration is defined as the time it takes \( E(T) \) to grow from 5% to 95% of \( E_m \).

\[
E(T) = \int_{0}^{T} a^2(t) \, dt \tag{3.1}
\]

\[
E_m = E(T_{100\%}) = \int_{0}^{T_{100\%}} a^2(t) \, dt \tag{3.2}
\]

\[
D_{sm} = T_{95\%} - T_{5\%} \tag{3.3}
\]

3. **RMSA Method (McCann and Shah 1979):** This method is based on the cumulative root-mean-square function \( (RMSA(T)) \) of the acceleration time history. The end of the strong motion duration is defined as the last time the RMSA reaches a
local maximum in time. The beginning of the strong motion is obtained in the same manner, but using the reversed time history.

\[
RMSA(T) = \sqrt{\int_0^T \frac{a^2(t) \, dt}{T}} = \sqrt{\frac{E(T)}{T}}
\]  

(3.4)

Table 3.2 lists the strong motion durations obtained by the above mentioned different methods. Table 3.3 lists the RMSA and Arias Intensity for the whole and strong motion duration (defined by the RMSA method) of the records. The information given in Tables 3.1 to 3.3 is for the two horizontal components at each of the 36 CSMIP stations.

Table 3.4 summarizes the information on the 33 Whittier Narrows ground motions chosen for the regression analysis discussed later in this subsection. The components with the larger PGA were chosen as representative of the Whittier Narrows ground motions for studying the attenuation of ground motion and seismic demand parameters. No unique measure for choosing the "larger" component could be found, but in most cases the component with the larger PGA was also the component with the larger RMSA, PGV and I. Thus, the definition of the "larger" component was based, somewhat arbitrarily, on the larger PGA of the two components. From the 36 records listed in Table 3.4, the three shaded records were eliminated from the regression analysis for the following reasons:

(a) The Tarzana record for its peculiarity discussed in Subsection 3.2.1.

(b) The Downey record because of its exceptionally large velocity pulses compared to all other records. The Downey station is located where the bedrock is deepest (30,000 ft), as shown in Fig. 3.7.

(c) The Mt. Wilson record because it is the only rock-site record.

From here on, the notation 36w refers to the 36 Whittier Narrows records before excluding the above listed three records, and 33w refers to the Whittier Narrows records after excluding the three records. In all cases, these notations refer to the components with the larger PGA, except when specifically mentioned otherwise.

A comparison between the component and vector PGA and PGV is shown in Figs. 3.8 and 3.9 as well as Table 3.1. The vector peak ground accelerations and velocities (denoted by VPGA and VPGV in the table) are usually not much different than their larger
component counterparts, and the times at which the vector and component peak values occur are almost the same. The Tarzana record clearly sticks out at an epicentral distance of e = 44 km. The high peak velocity of the Downey record is evident at e = 17 km in Fig. 3.9.

Fig. 3.10 illustrates the information given in Table 3.2, comparing the different methods for estimating the strong motion duration, $D_{sm}$. Part (a) of the figure shows $D_{sm}$ as defined by the Cut-off, both absolute and relative, and Bracketed methods for the 36w records. Part (b) shows the strong motion duration as defined by the RMSA method for the two horizontal components of the 36w records. Except for the Absolute Cut-off method, the strong motion durations increase with epicentral distance and show a similar trend even though values differ much depending on the method used. The Absolute Cut-off method gives a decreasing $D_{sm}$ with epicentral distance because of the attenuation of the ground motion.

Fig. 3.10(b) shows that the RMSA method gives relatively small differences in strong motion duration between the two components of each record. For this reason, and because the RMSA method gives average results compared to the other methods employed, the RMSA method was used to define the strong motion duration for all records used in the later discussed regression analysis of ground motion and seismic demand parameters.

A comparison between the maximum RMSA of the whole and strong motion duration of the 36w records is shown in Fig. 3.11. The RMSA at the end of the strong motion duration is also plotted. The maximum RMSA of the strong motion duration is always greater than that of the whole record. This is because the portion of the record preceding the strong motion duration, adds to the averaging time (the denominator in Eq. (3.4)) but not much to the energy imparted by the earthquake (numerator). For the same reason, the RMSA at the end on the whole record, shown in Table 3.3 and not in Fig. 3.11, is always a lower bound estimate of RMSA.

The difference in Arias intensity between the whole and strong motion duration of the 36w records, illustrated in Fig. 3.12, is negligible. This justifies the use of the strong motion duration of the records in lieu of the whole records for the dynamic time history analysis discussed in Chapters 4 and 5. The RMSA and Arias Intensity given in Table 3.4 are for the strong motion duration.
It was initially attempted to look at the spatial variations of ground motion and seismic demand parameters. These attempts turned out to be futile because the CSMIP records represent too small a sample set to draw definite conclusions on spatial variations. Contour plots were very sensitive to the boundary conditions imposed on the interpolating function and totally different conclusions could be drawn on spatial variation. Trifunac, 1988, reported on the spatial variations of PGA, using the records from 68 stations of the Los Angeles Strong Motion Accelerograph Network, and showed that the ground motions were largest to the south and north-west of the epicenter.

With these constraints, the CSMIP records were utilized to evaluate the attenuation with epicentral distance alone, regardless of station location with respect to the epicenter. It is recognized that such an attenuation disregards variations in geological and site conditions. Although the surface geology of the Los Angeles basin is rather complex, the justification for disregarding geographic location (in addition to the inability to consider it explicitly) is that most of the area in which the records were taken is covered with a deep layer of recent or quaternary alluvium. Only records at alluvial sites are used in this study.

For attenuation purposes, the independent variable \( y \) (the ground motion parameter) was regressed (least square error method) versus the distance \( r \), using the following relationship:

\[
\log y = a + d \log r + kr
\]

where \( r = (e + h)^{0.5} \)

- \( e \) = epicentral distance
- \( h \) = focal depth of earthquake (estimated as 14 km)
- \( a, d, k \) = regression parameters.

This equation is of the form proposed by Joyner and Boore, 1988, without considering the soil site correction factor. Joyner and Boore set the value of \( d \) equal to -1.0, whereas in this study \( d \) was used as a free regression parameter. The only constraint on the regression parameters was that \( k \) had to be negative and was taken as zero if it turned out to be positive in an unconstrained regression analysis.

According to Joyner and Boore it would be appropriate to replace \( e \) by the distance from the station to the nearest surface projection of the fault rupture. However, the epicentral distance was used instead, in this regression analysis, because the fault rupture
zone for the Whittier Narrows earthquake was short and almost all stations are located sufficiently far away from the rupture zone to render the error insignificant.

Typical results of the regression analysis for the attenuation of ground motion parameters are shown in Figs. 3.13 and 3.14. Fig. 3.13 shows the PGA data points for the 33w records (as listed in Table 3.4) and the least square fit regression curve. Also shown is the corresponding regression curve for the vector PGA as well as Joyner-Boore relationship for a magnitude 5.9 earthquake. Fig. 3.14 shows the corresponding information for the PGV. The Whittier Narrows Earthquake generated peak ground accelerations that were higher than predicted by Joyner and Boore, particularly at larger epicentral distances. To a lesser extent, the same applies to peak ground velocities except at close epicentral distances.

Fig. 3.15 illustrates the relative attenuation of four basic ground motion parameters. The curves are normalized with respect to the shown regressed values at 6 km epicentral distance. The rate of attenuation for PGA, PGV and RMSA (maximum for strong motion duration) is similar, whereas the Arias Intensity (for the strong motion duration), I, attenuates at a much faster rate because it is proportional to the square of the RMSA. For this earthquake, one is tempted to suggest typical values for \( \frac{PGA}{RMSA} \) of about 3.0, and for \( \frac{PGA}{PGV} \) of about 17 / sec, irrespective of epicentral distance.

Regression analysis was also performed on the strong motion duration \( D_{sm} \) (defined by the RMSA method), but employing a second order polynomial (versus the epicentral distance \( e \)) rather than the relationship given by Eq. (3.5). The data points and regression curve, shown in Fig. 3.16, show an increase in strong motion duration with epicentral distance. The strong motion duration becomes an important parameter when cumulative damage is evaluated, an issue to be discussed later in Subsection 4.4.5.

3.3 Ground Motion Records Used for Statistical Study

3.3.1 Introduction

Two other data sets of strong motion records, the 15s and 10w records, were chosen for the statistical studies performed on SDOF (Chapter 4) and MDOF (Chapter 5) systems. It needs to be emphasized that the 15s data set is used here as the primary set of records for statistical evaluation, while the 10w data set is mainly for comparison and
evaluating the dependency of results on the characteristics of one earthquake compared to the average of several "typical" earthquakes.

In order to group records with different "severity" (whether it be defined by PGA, PGV, RMSA or Arias Intensity) in one data set, there should be some sort of scaling available if and when it is needed. Scaling is vital for dimensional (unit dependent) parameters, e.g. strength demand, $F_y(\mu)$, energy terms, etc. Scaling is irrelevant for dimensionless (unitless) parameters which are independent of ground motion severity (or scale). Tuning the seismic demand parameters to a constant target ductility ratio, $\mu$, as opposed to a constant yield level, renders the dimensionless parameters independent of ground motion severity (or scale). For example, the strength reduction factor, $R_y(\mu)$, for a given strong motion record is independent of the scaling of the record. Scaling the input acceleration time history by a factor of two will equally double both the elastic and inelastic strength demands, giving the same ratio, $R_y(\mu) = F_{y,e} / F_y(\mu)$. NHE (Normalized Hysteretic Energy, $HE / F_y\delta_y$) and energy ratios like $HE / TDE$ and $IE(max) / IE$ are all examples of dimensionless parameters that are independent of ground motion severity (only because results the are tuned to constant target ductility ratios).

The purpose of scaling is twofold. First, to allow averaging of results that depend on ground motion severity. Second, to evaluate how well the spectra of the ground motions used represent, individually or in average, typical smoothened ground motion spectra as that proposed in ATC-3-06. Many different methods for scaling to equal "severity" are proposed in the literature. In this study the following two methods were explored:

- Scaling to a constant PGA, which allows a better control on the short period range ($T \approx 0.1-0.5$ sec) of the ground motion spectrum, often referred to as the constant acceleration range.

- Scaling to a constant PGV, which allows a better control on the period range ($T \approx 0.5-4.0$ sec) of the ground motion spectrum, often referred to as the constant velocity range.

For seismic zone 4, the ATC-3-06 document proposes the acceleration and velocity related coefficients, $A_q = A_v = 0.4$, corresponding to an EPA (Effective Peak Acceleration) of 0.4g and EPV (Effective Peak Velocity) of 12 in / sec. Therefore, scaling was done to
$PGA = 0.4g$ and $PGV = 12$ in / sec for comparison, as discussed in the coming two subsections.

### 3.3.2 15 Typical $S_I$ Records

Table 3.5 shows the basic information for the 15s data set comprised of 15 typical strong motion records from 9 different earthquakes that occurred in California and Washington State between 1934 and 1983. The earthquakes range in magnitude from 5.7 to 7.7. The epicentral distances of the stations range from 12 to 64 km and the strong motion duration ranges from 5.6 to 19.8 sec. The definition used for strong motion duration was that of RMSA method (McCann and Shah, 1979). The record peak values ($PGA$, $PGV$) are given in Table 3.5, before and after scaling to $PGA = 0.4g$ and $PGV = 12$ in / sec. More detailed information on the records is given in Hadidi-Tamjed, 1988. Fig. 3.17(a) shows the mean elastic response spectra for the 15s records scaled to a $PGA = 0.4g$ and $PGV = 12$ in / sec, together with the $ATC-S_I$ ground motion spectrum for 5% critical damping for comparison. Fig. 3.17(b) shows the corresponding mean $+ \sigma$ spectra for the 15s records. The following can be observed for the 15s records:

- Scaling the elastic response spectrum of the 15 $S_I$ ground motions to a $PGA$ of 0.4g results in a good match between the average spectrum and the $ATC-S_I$ spectrum up to a period of 1.0 sec. Beyond that period, the average spectrum underestimates the $ATC-S_I$ spectrum significantly.

- Scaling the elastic response spectrum of the 15 $S_I$ ground motions to a $PGV$ of 12 in / sec significantly underestimates the $ATC-S_I$ spectrum.

Because of the better match demonstrated by scaling to a constant $PGA$, this scaling was chosen for the parametric study on $SDOF$ systems for parameters that depend on seismic severity, namely, the strength demand, $F_y(\mu)$, and the different energy terms. All other $SDOF$ results (Chapter 4) and those of the $MDOF$ systems (Chapter 5) are independent of this scaling.

### 3.3.3 10 Near-Source Whittier Narrows Records

The 10 $CSMIP$ records closest to the epicenter of the Whittier Narrows Earthquake, excluding Downey and Mt. Wilson records for reasons discussed in Subsection 3.2.2,
were chosen as a representative subset of the Whittier Narrows ground motions for the statistical studies in Chapters 4 and 5. Near-source records have the least effect of travel distance and soil filtering on the frequency characteristics of ground motions. Such effects are evident from Fig. 4.10 (see Subsection 4.2.1).

Basic information on the Whittier Narrows records used in this data set, the 10w records, is included in Table 3.4. Components with the larger PGA are used and their epicentral distance range from 7 to 30 km (from the alhambra to pomonaff records in Table 3.4, excluding the shaded records).

Fig. 3.17 shows the mean and mean + \( \sigma \) spectra for the 10w records scaled to \( PGA = 0.4g \). The Whittier Narrows records have most of their energy concentrated in the short period range and very little in the long period range. Except for short periods, the Whittier Narrows spectrum severely underestimates the \( ATC-S_1 \) spectrum, and even more so the \( ATC-S_2 \) spectrum, which better represents ground motions recorded on alluvial soils. This helps explain the peculiarity of results obtained from this data set, compared to those from the 15s records, especially for the \( MDOF \) systems where higher modes are excited.
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* PGV obtained by numerical integration of digitized acceleration records

Table 3.5 15 Typical Western U.S. Ground Motion Records Used in This Study
Fig. 3.1 Locations of CSMIP Whittier Narrows "Freefield" Ground Motion Stations Used in this Study
(a) Peak Vector Accelerations

(b) Peak Vector Velocities

Fig. 3.2 The Orientation of the Peak Vectors for the Whittier Narrows Records
(a) Acceleration Time History

(b) Velocity Time History

(c) Displacement Time History

Fig. 3.3 Time Histories for the Tarzana Cedar Hill Nursery Record (90°)
ACCELERATION TIME HISTORY

VELOCITY TIME HISTORY

DISPLACEMENT TIME HISTORY

(a) Acceleration Time History

(b) Velocity Time History

(c) Displacement Time History

Fig. 3.4 Time Histories for the Alhambra Fremont School Record (270°)
ACCELERATION RESPONSE SPECTRA

Tarzana (90°)

(a) Tarzana Cedar Hill Nursery Record (90°)

ACCELERATION RESPONSE SPECTRA

Alhambra (270°)

(b) Alhambra Fremont School Record (270°)

Fig. 3.5 Site Specific Acceleration Response Spectra for 2, 5 and 10% Damping
Fig. 3.6 Vector Traces of the Horizontal Components of Tarzana Cedar Hill Nursery Record
Fig. 3.7 The Depth of Bedrock in the Los Angeles Area
(Reference: Yerkes (USGS), 1965)
Fig. 3.8 Comparison of Record Component and Vector PGA for 36w Records

Fig. 3.9 Comparison of Record Component and Vector PGV for 36w Records
COMPARISON OF STRONG MOTION DURATIONS - (36w)
Bracketed & Cut-off Methods, 36w Records, Components with Larger PGA

Strong Motion Duration, $D_{sm}$ (sec)

Epicentral Distance, $e$ (km)

(a) Bracketed, Absolute and Relative Cut-off Methods

COMPARISON OF STRONG MOTION DURATIONS - (36w)
RMSA Method, 36w Records, Longer & Shorter Components

Strong Motion Duration, $D_{sm}$ (sec)

Epicentral Distance, $e$ (km)

(b) RMSA Method

Fig. 3.10 Comparison of Strong Motion Durations Using Different Methods (36w Records)
COMPARISON OF RMSA - (36w)
36w Records, Components with Larger PGA

Compared to RMSA, the plot shows the variation of RMSA with epicentral distance. The graph includes three curves:
- RMSA @ End of Strong Motion Duration
- Maximum RMSA for Strong Motion Duration
- Maximum RMSA for Whole Record

Fig. 3.11 Comparison of RMSA for 36w Records

COMPARISON OF ARIAS INTENSITY - (36w)
36w Records, Components with Larger PGA

The plot illustrates the variation of Arias Intensity with epicentral distance. The plot consists of two curves:
- Arias Intensity for Strong Motion Duration
- Arias Intensity for Whole Record

Fig. 3.12 Comparison of Arias Intensity for 36w Records
ATTENUATION OF PGA - (33w)
33w Records, Vectors & Components with Larger PGA

Epicentral Distance, e (km)

Fig. 3.13 Attenuation of PGA for 33w Records

ATTENUATION OF PGV - (33w)
33w Records, Vectors & Components with Larger PGA

Epicentral Distance, e (km)

Fig. 3.14 Attenuation of PGV for 33w Records
Fig. 3.15 Normalized Attenuation Curves for PGA, PGV, RMSA and Arias Intensity (33w Records)

Fig. 3.16 Regression of Strong Motion Duration Defined by the RMSA Method (33w Records)
Fig. 3.17 Scaled Elastic Response Spectra for 15s and 10w Records
CHAPTER 4
SEISMIC DEMANDS FOR SDOF SYSTEMS

4.1 Models, Assumptions and Analysis Procedures

The three data sets discussed in Chapter 3 are used to gain a better understanding of the significance of the different seismic demand parameters for SDOF systems. It is hoped that this study will provide a platform for the demand versus capacity seismic design approach, outlined in Section 2.1, in which it is assumed that the ductility capacity, at the structure (global) or element (local) level, is the basic design parameter and the design objective is to provide sufficient strength capacity so that the ductility demands do not exceed the capacities by an adequate margin of safety during the "design" earthquake. Therefore, seismic demands presented in this chapter are for prescribed target ductility ratios rather than given yield levels. As a consequence, dimensionless demand parameters become independent of the ground motion severity (scale).

Nonlinear dynamic time history analyses were carried out on SDOF systems using NONSPEC, a nonlinear dynamic analysis program for SDOF systems (Mahin and Lin, 1983). NONSPEC was modified to perform iterations on yield levels in search of those that give the prescribed target ductility ratios used in this study (μ = 2, 3, 4, 5, 6 and 8). The process of determining the yield level for specified ductility ratios has to be done very carefully and may involve several trials because the relationship between yield level and ductility demand (Fy-μ) is not necessarily a monotonic function. This is illustrated for one specific case in Fig. 4.1 (a). For this record and the selected structural period of 0.9 seconds, there exist three yield levels that result in a ductility demand of 3.0. Clearly, it is only the highest of the three yield levels that defines the strength demand Fy(3). Interpolation between widely spaced Fy-μ data points could have led to very erroneous results. Fig. 4.1 (b) illustrates that the three systems with different yield levels have indeed the same ductility demand in the time history response.

Two hysteresis (load-deformation) models (shown in Fig 4.2) were used, namely, the bilinear and modified Clough stiffness degrading models. The bilinear model is commonly used to represent the behavior of structural elements which exhibit insignificant strength or stiffness degradation, e.g., flexural response of a compact steel beam (where
lateral and torsional buckling is not of concern). The modified Clough stiffness degrading model (Mahin and Bertero, 1975) is used to represent the behavior of elements which have a memory limited to the largest positive and negative excursions of past loading cycles and may be representative of reinforced concrete elements in which the opening / closing of cracks leads to pinching of the hysteresis loops.

The nonlinear dynamic time history analyses were performed for the following 51,504 permutations:

- For a total of 48 digitized records comprising the three data sets discussed in Chapter 3 (the 33 Whittier Narrows ground motions (for the 33w and 10w data sets) and the 15 $S_T$ ground motions typical of Western U.S. earthquakes (the 15s data set)).
- For target ductility ratios $\mu = 1$ (elastic), 2, 3, 4, 5, 6 and 8.
- For strain hardening $\alpha = 0$, 2, and 10%.
- For the two hysteresis models discussed before, namely, the bilinear and stiffness degrading models.
- For 29 discrete periods ranging from $T = 0.1$ to 4.0 sec.

Only the strong motion portion plus 2 seconds on either side of the records are used in the dynamic time history analysis. The strong motion duration is defined by RMSA method discussed in Section 3.2. The added portions, if available, are to allow the system to vibrate freely before and after the strong motion portion.

Strength and energy demand parameters were evaluated for both the bilinear and stiffness degrading models, while the cumulative damage parameters (see Section 4.4) were evaluated for the bilinear model only. The reason being that cumulative damage parameters depend on the plastic deformation range, which becomes undefined for the stiffness degrading model.

Whenever possible, the results for the seismic demand parameters, listed in Section 2.2, are presented here as spectra for prescribed target ductility ratios. In most cases, information is presented in the following sequence:
- **Site specific data** from an individual record, using the **bilinear model with 10% strain hardening** as an example.

- **Attenuation** of seismic demand parameters with epicentral distance, using the Whittier Narrows ground motions (the 33w records discussed in Section 3.2) and the **bilinear model with 10% strain hardening**.

- **Statistical study** on the 15s and 10w data sets (see Section 3.3). Results are presented for both the **bilinear and stiffness degrading models with 0, 2 and 10% strain hardening**. Gaussian (Normal) distribution is assumed to represent the scatter of data. The mean and mean plus (or minus) standard deviation (mean ± σ) are presented for the bilinear model with 10% strain hardening. The effects of using different strain hardening ratios (α = 0 and 2%) and different hysteresis models (stiffness degrading versus bilinear) are presented as ratios to the results obtained from the bilinear model (the ratios given are always the **mean of the ratios** not the ratios of the means).

All results are given for **5% critical damping**; damping was not varied as a parameter in this study.

For the purpose of a global assessment of seismic demand parameters, the regression analysis used in the attenuation study for the Whittier Narrows ground motions and the statistical study on the 15s and 10w data sets are expected to diminish the effects of local site conditions and amplify the behavioral patterns in these parameters.

The objective of the attenuation study presented here is threefold. First, to identify attenuation characteristics of important seismic demand parameters and evaluate them in conjunction with the attenuation characteristics of the ground motion parameters. Second, to identify patterns in seismic demand parameters that can be generalized for the purpose of improving our understanding of seismic input and its effect on structures. And, third, to provide quantitative information on seismic demands that can be used together with information on capacities to assess the damage potential of the Whittier Narrows ground motions.

The process employed in the regression analysis for the attenuation of seismic demand parameters versus epicentral distance is simple but very computation intensive. For each parameter identified in Section 2.2, and for all the variables employed in the study
(e.g., different target ductility ratios, periods, etc.), the data points from the 33 Whittier Narrows (33w) ground motions were used for an independent regression analysis. The regression curves were then used to derive distance dependent spectral information, or distance independent information, if appropriate. The attenuation relationship used is given by Eq. (3.5), which was also used for the attenuation of ground motion parameters in Subsection 3.2.2. The least square error method is employed in the regression analysis. The regressed plus standard deviation spectra, like the ones shown in Figs. 4.6 to 4.9 (b), are obtained by adding the distance independent standard deviation to the regressed parameter in the log-log equation given by Eq. (3.5).

Statistical averaging is performed on the 15s and 10w data sets to evaluate patterns and variations of the aforementioned seismic demand parameters irrespective of epicentral distance. At this point scaling of ground motions may be required for certain parameters as discussed in Section 3.3.

Because the Whittier Narrows Earthquake has very little energy in the long period range (see Fig. 3.17), the attenuation results shown in the figures are limited to a period \( T = 2.0 \) sec and target ductility ratios \( \mu \leq 4 \). On the other hand, the statistical results of the 15s records were extended to a period \( T = 4.0 \) sec and target ductility ratio \( \mu = 8 \) (the same applies to the 10w data set for comparison).

4.2 Strength Demands

Strength demands are expressed here as a base shear coefficient, thus, \( F_y(\mu) / W \) represents yield level over seismically effective weight. The elastic strength demand spectrum is identical to the acceleration response spectrum. The inelastic strength demand spectrum represents the period dependent yield level required to limit the ductility ratio to a prescribed value of \( \mu \).

4.2.1 Attenuation of Elastic and Inelastic Strength Demands

The results discussed here are all based on bilinear SDOF systems with 10% strain hardening whose yield levels are adjusted to give preselected ductility demands of \( \mu = 1 \) (elastic), 2, 3 and 4. The corresponding yield levels are, therefore, the strength demands, \( F_{y,e}, F_y(2), F_y(3), F_y(4) \). The data set used here is the 33 Whittier Narrows (33w) ground motions discussed in Section 3.2.
Fig. 4.3 shows two site specific elastic and inelastic strength demand spectra for the Alhambra Fremont School record (270° component), which is the closest to the epicenter, and the L.A. Obregon Park record (0° component), which had the largest PGA except for the Tarzana record. The two stations are close to each other, located 7 km to the north-west and 10 km to the west of the epicenter, respectively. The elastic spectra for the two records are similar, with one large peak in the short period range and two other discernible humps at longer periods. In the inelastic spectra the peaks diminish and essentially disappear for larger ductility ratios. This phenomenon is observed consistently for all records and can be explained with the change in effective period of inelastic systems. For instance, a system with an initial period of 0.2 seconds (peak of elastic spectrum) will experience a high force demand and will yield early; however, once yielded, the effective period will lengthen and the strength demand on the system will be low because of the steep decreasing slope of the elastic strength demand spectrum. As a consequence, the shapes of the elastic and inelastic strength demand spectra are not necessarily similar, and are in fact rather dissimilar if the elastic spectrum has steep peaks. Further discussion on this issue will be presented in Section 4.3, which addresses strength reduction factors. The Alhambra record will be used from here on to illustrate site specific patterns for different demand parameters.

Fig. 4.4 shows a typical regression curve for the inelastic strength demand $F_y(\mu = 4)$ versus epicentral distance for a period $T = 0.5$ sec. The figure shows typical scatter of data points associated with the regression analysis on strength demands. Fig. 4.5 shows the regression curves for $F_y(\mu = 4)$ normalized with respect to values at epicentral distance $e = 6$ km, for discrete periods. Superimposed on the graph is the normalized regression curve for the PGA (see Section 3.2). The short period inelastic strength demands attenuate at a rate very similar to the PGA, whereas the long period strength demands attenuate much faster (more discussion to follow).

Figs. 4.6 to 4.9 show the attenuation of the elastic and inelastic strength demand spectra with epicentral distances ranging from $e = 6$ to 80 km. Regressed as well as regressed plus standard deviation spectra are presented to give a sense of the scatter associated with the regression analysis. Superimposed on the graphs for elastic strength demands (Fig. 4.6) are curves of the 1988 UBC Code values for the product $ZC$ (for soil types $S_1$, $S_2$, and $S_3$), which are measures of the elastic strength demand for severe earthquakes implied by code design. Superimposed on the graphs for inelastic strength demands (Figs. 4.7 to 4.9) is a curve for the product $ZC / 12$ (for soil type $S_2$), which is a measure of the code seismic design force level for ductile structural systems. Fig. 4.10 is a
summary of the regressed spectra of all the elastic and inelastic strength demands for epicentral distances ranging from $e = 6$ to $80$ km, normalized with respect to the corresponding PGA (regressed PGA at the same epicentral distances). The *elastic* strength demand normalized with respect to PGA is the Dynamic Amplification Factor (DAF).

The following general observations can be made from the mentioned figures:

1. The shapes of the strength demand spectra are relatively smooth.

2. The elastic spectra (Fig. 4.6) exhibit one large protuberance in the short period range, which widens with increasing epicentral distance.

3. The high values of elastic strength demand in the short period range of near-source spectra help explain the large damage experienced by stiff masonry buildings in Whittier and other nearby communities.

4. The inelastic strength demand spectra (Figs. 4.7 to 4.9) attenuate at a rate that is similar to that of the elastic spectra, and also show a relatively high demand for short period structures.

5. If one would consider modern code seismic design forces as a measure of strength capacity (e.g., $ZC / 12$, as shown in the figures), then the damage potential of the ground motions appears to be very large. However, damage observations do not confirm high ductility demands for modern structures. The reasons for this apparent discrepancy are discussed in Section 4.5.

6. Fig. 4.10 shows the attenuation of normalized elastic and inelastic strength demand spectra for the Whittier Narrows records, superimposed on the ATC-3-06 normalized ground motion spectra for soil types $S_f$ and $S_2$. The normalized elastic strength demand (DAF) severely underestimates that of the ATC-$S_f$ in the medium and long period range, and more so for soil type $S_2$ which represents the alluvial soils in and around Los Angeles.

7. The shapes of the normalized spectra, presented in Fig. 4.10, show unusual characteristics of the Whittier Narrows ground motions. It is usually expected that medium and long period components of the ground motion attenuate slower than high frequency components, leading to spectra in which the medium and long period DAFs increase with distance from the
source. The Whittier Narrows ground motions show the opposite behavior. For periods longer than 0.6 sec, the DAFs decrease with epicentral distance. This pattern can also be seen in the normalized inelastic strength demand spectra. Thus, it must be concluded that the Whittier Narrows Earthquake, which shows little energy content in the long period range near the source and even much less at larger distances, is an earthquake with uncommon long-period characteristics.

A comparison between site specific and regressed strength demand spectra is presented in Fig. 4.11. The four parts of this figure show site specific elastic and inelastic strength demand spectra (solid curves), together with the corresponding regressed spectra at each site (dashed curves). Alhambra and Featherly Park (Fig. 4.11 (a) and (c)) are typical records for which the site specific spectra are close to the regressed ones. The L.A. Obregon Park record shows the same trend as Alhambra, but has spectral values that are significantly larger than the corresponding regressed ones. Fig. 4.11 (d) clearly shows the unusual strength demands of the Tarzana record (which was excluded from the regression analysis). The site specific demands exceed the demands obtained from regression analysis by about a factor of 10 in the short period range. This is quite different from the Featherly Park record, which is at a similar epicentral distance (40 km versus 44 km) but to the south-east of the epicenter rather than the north-west.

### 4.2.2 Statistical Study on Strength Demands

Statistical evaluation of elastic and inelastic strength demands was performed on the other two data sets, namely, the 15s and 10w records, irrespective of epicentral distance. Fig. 4.12 shows the mean elastic ($\mu = 1$) and inelastic ($\mu = 2, 3, 4, 5, 6$ and 8) strength demand spectra scaled to $PGA = 0.4g$ for the 15s data set for strain hardening $\alpha = 0$ and 10%, respectively. Fig. 4.13 shows the corresponding graphs for the 10w data set. Results presented in Figs. 4.12 and 4.13 are for bilinear systems.

The following observations can be made from these figures:

- There is a hump in most of the inelastic strength demand spectra for the 15s records at $T = 0.2$ sec, whereas the corresponding 10w spectra monotonically decrease with period. This is due to the fact that the peak elastic values for the 15s records occur at longer periods ($T = 0.2$ to 0.3
sec) than for the 10w records \((T = 0.15 \text{ sec})\). As an SDOF system with a very short period, e.g. \(T = 0.1 \text{ sec}\), becomes inelastic and its effective period lengthens, it traverses the large peak of the 10w records faster than that of the 15s records and takes little notice of the existence of the large steep peak in the 10w elastic spectrum.

- For the same target ductility ratio, systems with no strain hardening require slightly larger strengths than strain hardening systems. This is attributed to the fact that the former drift more and, therefore, require higher strength to limit the ductility demand to a prescribed value.

A more detailed discussion on the effect of strain hardening and hysteresis model (bilinear versus stiffness degrading) on the inelastic strength demand follows in the next section on strength reduction factors. It is convenient to express this effect in terms of ratios of reduction factors. Since the elastic strength demand, \(F_{y,e}\), is independent of strain hardening and hysteresis model, the ratio of strength reduction factor is the inverse of the ratio of strength demands.

### 4.3 Strength Reduction Factors

The strength reduction factor \(R_y(\mu)\) is the ratio of the elastic strength demand imposed on an SDOF system to the inelastic strength demand for a specific target ductility ratio. Thus, it is the ratio of spectral ordinates of the elastic and inelastic strength demand spectra. It can be thought of as an effectiveness factor that shows how much the yield level capacity of a given elastic SDOF system can be reduced, by allowing the system to behave inelastically, within the limits of a predefined ductility ratio.

Seismic design may be based on either inelastic strength demand spectra of the type discussed in the previous subsection or on the elastic spectra and associated strength reduction factors employed to scale the elastic spectra to inelastic strength levels corresponding to a given target ductility ratio. The latter approach is more convenient in the context of code design since smoothened ground motion (elastic strength demand) spectra, like those given in the ATC-3-06, have already been developed. This approach is even more appealing for the proposed seismic design procedure discussed in Section 2.1, which is based on two "design" earthquakes with different probability of occurrence. Provided that the reduction factors are not very sensitive to spectral shape, the same reduction factors
can be used with different elastic spectra. Such a procedure is more adept to firm soils (characterized by soil type $S_1$ and $S_2$). For soft soils, this procedure is not feasible because of the dependency of both elastic and inelastic strength demands, and therefore the strength reduction factor, on the local site conditions.

The database generated in this study allowed a comprehensive study on strength reduction factors for firm soils. The next subsection discusses both site-specific and attenuation characteristics of the $R$-factors, while Subsections 4.3.2 discuss the effect of different parameters on the $R$-factors. A nonlinear least square best fit relationships for the $R$-factor, as a function of the important parameters on which it depends, are developed bilinear systems subjected to the 15s records, in Subsection 4.3.3.

4.3.1 Attenuation of Strength Reduction Factors

The results discussed here are all based on bilinear SDOF systems with 10% strain hardening, whose yield levels are adjusted to give preselected ductility demands of $\mu = 2$, 3 and 4. The data set used here is the 33 Whittier Narrows (33w) record set discussed in Section 3.2.

Fig. 4.14 shows the site specific strength reduction factors ($R$-factors) for ductility ratios of 2, 3 and 4, using the two Whittier Narrows records whose strength demand spectra are shown in Fig. 4.3. As the figure shows, the $R$-factors are not constant with period and not linearly related to $\mu$. By comparing Figs. 4.14 and 4.3, it can be noted that, in general, in the short period range, the peaks and valleys in the $R$-spectra correspond to peaks and valleys in the elastic strength demand spectra, for reasons discussed in Subsection 4.2.1. Thus, the strength reduction factor is large at periods at which the elastic strength demand is large, and vice versa, which results in relatively smooth inelastic strength demand spectra even for elastic spectra with large peaks and valleys.

Since both the elastic and inelastic strength demands are assumed to follow a log-log attenuation with epicentral distance (see Eq. (3.5)), their ratios should also attenuate in the same manner. Attenuating the site specific reduction factors, using the log-log regression given in Eq. (3.5), should and did give the same results as are obtained from the ratios of the regressed elastic and inelastic strength demand spectra discussed in Subsection 4.2.1. A typical regression curve for the $R$-factor versus epicentral distance, together with the data points for $\mu = 4$ and $T = 0.2$ sec, is shown in Fig. 4.15. The reduction factor is
not very sensitive to epicentral distance. This is also evident from Fig. 4.16, which illustrates the regressed $R$-spectra for different target ductility ratios and epicentral distances $e = 6$ and 80 km. Superimposed in dashed curves are the later developed $R-\mu-T$ relationships (see Subsection 4.3.3) for the 15 typical Western U.S. ground motions (referred to as 15s records). The Whittier Narrows $R$-factors are larger than predicted by the developed $R-\mu-T$ relationships. This could be attributed to the steep elastic strength demand spectra of the Whittier Narrows records (Fig. 4.13) and the fact that upon yielding the effective period of the system lengthens and the resulting inelastic strength demand drops sharply.

### 4.3.2 Statistical Study on Strength Reduction Factors

The following discussion is based on a statistical evaluation of data for both the 15s and 10w data sets for bilinear and stiffness degrading models using 0, 2 and 10% strain hardening.

Figs. 4.17 to 4.19 show the mean and mean - $\sigma$ of strength reduction spectra for the 15s records for bilinear systems with target ductility ratios $\mu = 2, 3, 4, 5, 6$ and 8 for strain hardening $\alpha = 0, 2$ and 10 %, respectively. The corresponding spectra for the 10w data set are shown for $\alpha = 10\%$ only in Fig. 4.20. Mean - $\sigma$ values are shown here as conservative estimates of the strength reduction factor with only 16% chance of data falling below these values. Figs. 4.21 and 4.22 show the effect of strain hardening and hysteresis models, respectively, on the $R$-factors.

As noted before, the ratios of the strength reduction factors for different parameters is the inverse of the ratios of inelastic strength demands for the same parameters.

The following observations can be made from Figs. 4.17 to 4.22:

- The reduction factors approach $R = 1$ for very short periods and $R = \mu$ for very long periods (to be explained in the next subsection). Note that, in average, the $R$-spectra exceed the $R = \mu$ lines for intermediate periods and asymptotically approach the $R = \mu$ lines from above (unlike what is assumed in Hidalgo and Arias, 1990).

- Despite the fact that the elastic and inelastic strength demand spectra for the 15s and 10w data sets are quite dissimilar, their $R$-spectra are not much
different. This supports the concepts of using $R$-spectra in design codes in conjunction with elastic strength demand spectra to obtain inelastic strength demand spectra.

- The reduction factors for $\alpha = 2\%$ are approximately half way between those for $\alpha = 0$ and $10\%$, suggesting that even small values of strain hardening improve (increase) the strength reduction factor significantly. An elastic-perfectly plastic SDOF system ($\alpha = 0\%$) drifts much more than a system with some strain hardening ($\alpha = 2$ to $10\%$), and therefore require larger strengths to reach the same target ductility ratio $\mu$.

- The effect of strain hardening on the strength reduction factors is illustrated in Fig. 4.21 as ratios of the $R$-factors for bilinear systems with 0 or 2% to 10% strain hardening for the 15s records. $R_y(\mu)(\alpha=0\%) / R_y(\mu)(\alpha=10\%)$ is in the order of 0.8 to 0.9. $R_y(\mu)(\alpha=2\%) / R_y(\mu)(\alpha=10\%)$ is in the order of 0.9 to 1.0. Therefore, the effect of strain hardening on the inelastic strength demands of bilinear SDOF systems is noticeable but not predominant.

- The effect of hysteresis model on the $R$-factors is shown in Fig. 4.22 as ratios of $R$-factors of stiffness degrading to bilinear systems with 0 and 10% strain hardening for the 15s records. It is interesting to note that, except for very short period systems, the stiffness degrading model allows higher strength reduction factors than the bilinear model, for systems without strain hardening. This difference diminishes with strain hardening. This is a very interesting result in that it suggests that the stiffness degrading model behaves "better" than the bilinear model, i.e., it has a smaller inelastic strength demand for the same ductility ratio. This effect can be explained by realizing that the stiffness degrading model spends less "time" in the minimum stiffness (post-yielding stiffness = $\alpha K$) region of the hysteresis loop compared to the bilinear model. Therefore, the bilinear model has a greater chance of drifting and requires more strength capacity to limit deformations to the same target ductility ratio.

There are important implications in the observation that the differences in strength demands between bilinear and stiffness degrading Clough models are usually small and in most cases the degrading model gives favorable results. Much effort is often devoted to
refined hysteresis modelling for elements and structures. With regards to assessment of
ductility (or strength) demands, this effort may not be warranted provided that stiffness
degradation is of a type similar to that described by the Clough model.

4.3.3 Regression Analysis of Strength Reduction Factors

Strength reduction factors (R-factors) have taken on an important role in seismic
design as they are used in many codes, either explicitly or implicitly, to derive seismic
design forces from elastic response spectra. Using R-factors has many advantages,
particularly if it can be shown that these factors are stable parameters for typical S1 and S2
ground motions. The relatively small differences in R-factors for the 15s and 10w data sets
(see Figs. 4.19 and 4.20) appear to confirm this. Therefore, an effort was made in this
study to derive expressions that relate the R-factors to basic SDOF system parameters.

The results in this study show that the strength reduction factor depends strongly on
the target ductility ratio \( \mu \), and the period \( T \), and to a much lesser degree on the strain
hardening and type of hysteresis model. A study was carried out on the 15s data set to
formulate regression equations representing the R-factor as a function of the
aforementioned parameters for bilinear SDOF systems with 5% critical damping. For a
given strain hardening, \( R \) becomes a function of period \( T \) and target ductility ratio \( \mu \), i.e.,
\( R = R(\mu, T, \alpha) = R(T, \mu) \). A database of 39,000 points (about 30 \( R-\mu \) points for each of the
15 ground motions, 29 periods and 3 strain hardening ratios) in the \( R-\mu-T \) domain,
obtained from the SDOF dynamic time history analyses, made this regression analysis
possible. A two-step nonlinear regression analysis (referred to in the figures as first and
final regression) is carried out in 2-D domains, first regressing \( R \) versus \( \mu \) for discrete
periods, \( T \), and then evaluating the effect of period in the second step. This process is
repeated for the three strain hardening ratios \( \alpha = 0, 2 \) and 10%.

In the \( R-\mu \) domain, the most commonly used relationships are:

\[
R = \mu \quad \text{equal displacement method} \quad (4.1)
\]
\[
R = \sqrt{2 \mu - T} \quad \text{equal energy method} \quad (4.2)
\]

Osteraas and Krawinkler, 1990, proposed a 2-parameter equation for \( R-\mu \) in the following
form:
\[ R = (c(\mu - 1) + I)^x \]  \hspace{1cm} (4.3)

where \( x \) and \( c \) are functions of the period \( T \), and strain hardening \( \alpha \). The slope at \( \mu = 1 \) is:

\[ \frac{\partial R}{\partial \mu} (\mu = 1) = xc \]  \hspace{1cm} (4.4)

Since \( \mu \leq 1 \) defines the elastic behavior, the slope of the \( R-\mu \) relationship at \( \mu = 1 \) is assumed here to be 1.0 and therefore \( x \) is taken as \( 1/c \) in this study. The parameter \( c \), which is a function of period \( T \) and strain hardening \( \alpha \), is first regressed in the \( R-\mu \) domain using the following equation:

\[ R = (c(\mu - 1) + 1)^{1/c} \]  \hspace{1cm} (4.5)

where \( c = c(T, \alpha) \)

Eq. (4.5) comprises both Eqs. (4.1) and (4.2) by using \( c = 1.0 \) and \( c = 2.0 \), respectively. In addition, the following relationships should also be satisfied:

\[ R(\mu, T \rightarrow 0) \rightarrow I \]  \hspace{1cm} (4.6 a)

\[ R(\mu, T \rightarrow \infty) \rightarrow \mu \]  \hspace{1cm} (4.6 b)

\[ R(\mu = 1, T) = I \]  \hspace{1cm} (4.6 c)

Eq. (4.6 a) represents the extreme case of a very stiff system (yield displacement \( \delta_y \rightarrow 0 \)) for which a very small reduction in its strength capacity brings the system to the prescribed target ductility ratio, i.e., \( R \rightarrow I \). Eq. (4.6 b) is based on the other extreme case of a very soft system for which the maximum relative displacement tends to the peak ground displacement, \( PGD \), regardless of the system yield level (i.e., equal displacements). Eq. (4.6 c) applies to elastic systems. Eqs (4.6 a-c) require that the \( c \) parameter fulfill the following conditions:

\[ c(T \rightarrow 0, \alpha) \rightarrow \infty \]  \hspace{1cm} (4.7 a)

\[ c(T \rightarrow \infty, \alpha) \rightarrow 1 \]  \hspace{1cm} (4.7 b)

Examples of the nonlinear regression in the \( R-\mu \) domain are shown in Fig. 4.23 for periods \( T = 0.2 \) and 1.0 sec and strain hardening \( \alpha = 0\% \). The data points, shown as small circles, are obtained from the iterations performed in the \( R-\mu \) domain in search of the
R-factors (or inelastic strength demands, $F_\gamma(\mu)$) that give the prescribed target ductility ratios $\mu = 1, 2, 3, 4, 5, 6$ and 8, as discussed in Section 4.1 (about 30 iterations per ground motion record x 15 records). By trial and error on the $c$ parameter, the regression curve, shown as the middle solid curve, was obtained by minimizing the sum of the square of the errors of the data points about the regression curve. This procedure was repeated for all periods (29 periods spanning $T = 0.1 - 4.0$ sec) and strain hardening ratios ($\alpha = 0, 2$ and 10%).

Also shown in the same figure are dashed curves for the standard deviation of the data points about the regression curve. The standard deviation increases with the target ductility ratio $\mu$. All data points converge to $R = 1$ at $\mu = 1$.

A Gaussian (Normal) distribution is assumed to represent the scatter of data points about the regression curve, with the standard deviation gradually increasing from zero at $\mu = 1$ to a maximum at $\mu = 8$. In order to obtain such gradual increase in standard deviation, the means ($m_1$), shown as black circles in Fig. 4.23, and standard deviations ($\sigma_1$) of the data points (about the mean values $m_1$) at discrete ductility ratios were first evaluated. There are at least 15 data points (corresponding to 15 ground motion records) on the vertical lines $\mu = 1, 2, 3, 4, 5, 6$ and 8 in Fig. 4.23 (with a tolerance in ductility ratios of $\pm 2\%$). Since the regression curves will not necessarily pass through the means ($m_1$) of the data points at discrete ductility ratios, the standard deviation of the data points about the regression curve (referred to as $\sigma_2$) is evaluated at the discrete ductility ratios as follows:

$$\sigma_2 = \sqrt{\sigma_1^2 + (m_1 - m_2)^2}$$  \hspace{1cm} (4.8)

where $m_2$ is the regressed value at the given ductility ratio, i.e., the ordinates of the middle solid curve at the discrete ductility ratios $\mu = 1, 2, 3, 4, 5, 6$ and 8.

Fig. 4.24 shows the $c$ parameters obtained from the above mentioned regression, plotted versus period $T$ for 0 and 10% strain hardening. Each regression curve of the type shown in Fig. 4.23 gives a single point in Fig. 4.24 for a given period $T$ and strain hardening $\alpha$. Another independent regression analysis (referred to as the final regression in the figures) is performed in the $c-T$ domain to smoothen the obtained data points.

Besides trying to satisfy Eq. (4.7), the regression equation for $c(T, \alpha)$ should also allow the tendency of data points (see Fig. 4.24) to approach the $c = 1.0$ line from below as
$T$ tends to infinity (corresponding to the $R$-factors approaching $R = \mu$ from above in Fig. 4.17 to 4.19). The proposed $c(T, \alpha)$ equation is as follows:

$$c(T, \alpha) = \frac{T^a}{1+T^a} + \frac{b}{T}$$

(where $a$ and $b$ are functions of $\alpha$) \hspace{1em} (4.9)

The first term is zero at $T = 0$ and approaches $c = 1.0$ from below for long periods, while the second term is infinity $T = 0$ and goes to zero for long periods. A separate nonlinear regression analysis was carried out, by trial and error on the parameters $a$ and $b$, to minimize the sum of the square of the errors of the $c$ points about the regression curve (in the period domain). This regression was done for 0, 2 and 10% strain hardening. The correlation coefficient was in the order of 96%). The following values for $a$ and $b$ were obtained:

- for $\alpha = 0\%$ : $a = 1.00$ \hspace{1em} $b = 0.42$ \hspace{1em} (4.10 a)
- for $\alpha = 2\%$ : $a = 1.01$ \hspace{1em} $b = 0.37$ \hspace{1em} (4.10 b)
- for $\alpha = 10\%$ : $a = 0.80$ \hspace{1em} $b = 0.29$ \hspace{1em} (4.10 c)

Fig. 4.25 summarizes the results of the $R-\mu-T$ regression analysis. The first regression in the $R-\mu$ domain, separated the period and strain hardening dependency into the $c$ parameter. The final regression in the $c-T$ domain separated the strain hardening dependency into $a$ and $b$.

Fig. 4.26 shows a 3-D perspective view of the generated $R-\mu-T$ surface for $\alpha = 10\%$ using Eqs. (4.5), (4.9) and (4.10). The surface is plotted for $T = 0.1$ to 4.0 sec and $\mu = 1$ to 10. This surface is the least square best fit for about 13,000 points. It is evident that the $R-\mu-T$ relationship is highly nonlinear, especially in the short period range. The developed $R-\mu-T$ relationships serve the part of the main objectives of this study (refer to Fig. 2.3). They are to be used to scale down the elastic response spectra to obtain the inelastic strength demand for $SDOF$ systems, knowing their period $T$, target ductility $\mu$ and strain hardening $\alpha$. The obtained inelastic strength demand spectra will then be modified to account for multi-mode effects of $MDOF$ systems as discussed in Chapter 5.

Fig. 4.27 shows the evolution of regression analysis in the $R-T$ domain for 0 and 10% strain hardening. The dashed curves show the mean $R$-spectra using the mean of data points at discrete ductility ratios $\mu$ (i.e., $m_j$ represented by the black circles in Fig. 4.23).
The dotted curves show the mean $R$-spectra after the first step regression, i.e., using the $c$ points (representing the middle solid curves in Fig. 4.23 or the circles in Fig. 4.24) obtained from the nonlinear regression in the $R$-$\mu$ domain and before regressing $c$ versus the period $T$. The solid curves show the mean $R$-spectra after the final regression of $c$ versus the period $T$ (represented by the solid curves in Fig. 4.24). It should be noted that the first step regression (comparing the dotted vs. dashed curves) tends to underestimate the mean of the data points at discrete ductility ratios for $\mu = 2 - 5$. This is because regression in the $R$-$\mu$ domain gives more weight to data points at large ductility ratios (see Fig. 4.23).

Fig. 4.28 illustrates the variation associated with the $R$-$\mu$-$T$ regression analysis described above, for 0 and 10% strain hardening. The mean $\sigma (m_f - \sigma_j)$ curves represent conservative estimates of the $R$-factors with only 16% chance of data falling below these values. The dashed curves in Figs. 4.27 and 4.28 are the same as those shown in Figs. 4.17 and 4.19.

The solid curves in Figs. 4.27 and 4.28 represent one plane of the 3-D view shown in Fig. 4.26. Figs. 4.29 and 4.30 show the other two planes. Fig. 4.29 shows the regressed ductility demand spectra for constant reduction factors for 0 and 10% strain hardening. In concept, the ductility demands imposed on short period structures designed for period independent $R$-factors could be very high. Fortunately, as discussed in Section 2.3, short period structures have much larger overstrength than long period structures, and therefore the ductility demands are less than those implied by this figure. The figure also illustrates that the ductility demand is higher for systems without strain hardening.

Fig. 4.30 shows the regressed $R$-$\mu$ relationships for discrete periods. Superimposed are the two well-known relationships given by Eqs. (4.1) and (4.2). $R = \mu$ (or $c = 1$ in Fig. 4.24) is a conservative approximation for medium and long period systems, whereas $R = \sqrt{2\mu - T}$ (or $c = 2$ in Fig. 4.24) is a poor relationship for all but one specific period.

The $R$-$\mu$-$T$ relationships developed here may be used in conjunction with smoothened ground motion spectra, like those given by ATC-3-06 for strong motions on firm soils (i.e., soil types $S_1$ and $S_2$), to obtain inelastic strength demand spectra. Fig. 4.31 illustrates such spectra for soil types $S_1$ and $S_2$ for constant ductility demand $\mu = 2, 3, 4$ and 8. It can be seen that the plateau in the short period range of the ATC ground motion
spectra almost disappears from the inelastic strength demand spectra because of the highly nonlinear $R-\mu-T$ relationship in this range.

4.4 Energy and Cumulative Damage Demands

4.4.1 Introduction

It is well established from experimental work and analytical studies that strength and stiffness properties of elements and structures deteriorate during cyclic loading. Materials, and therefore elements and structures, have a memory of past loading time history, and the current deformation state depends on the cumulative damage effect of all past states. In concept every excursion causes damage, and damage accumulates as the number of excursions increases. The damage caused by elastic excursions is usually small and negligible in the context of seismic behavior. Thus, only inelastic excursions need to be considered, and from those the large ones cause significantly more damage than smaller ones (however, smaller excursions are much more frequent). In damage evaluation, inelastic excursions are not counted as they appear in the time history but re-arranged according to one of the low-cycle fatigue cycle counting methods (Fuchs and Stephans, 1980), so that small excursions are considered as interruptions to larger ones.

For low-cycle fatigue, the rain-flow cycle counting method was found to be the best suited one for seismic demand evaluation (Krawinkler et al., 1983). The rain-flow cycle counting method is illustrated schematically in Fig. 4.32. The deformation time history is drawn on a vertical time axis pointing downwards. Rain flow is thought to be initiated at the beginning of the time history and at each point of reversal. Every rain drop flows downwards until it either meets another drop from above or reaches a peak which is the starting point of a reversal leading to a peak that exceeds the point from which the rain drop was initiated. The horizontal distance between the beginning and end points of each rain-flow is counted as a half cycle deformation range or excursion. These ranges are numbered, in order of their occurrence, in the line diagram below the time history illustrated in Fig. 4.32. Note that every part of the deformation time history is counted only once. These ranges are quite different from those in the time history and contain four closed cycles (namely, excursions: 4 / 7, 5 / 6, 9 / 10 and 11 / 12 in Fig. 4.32). For each deformation range, the elastic portion can be separated from the plastic one. The plastic deformation ranges (denoted as $\Delta \delta_p$) so identified, together with the number of inelastic excursions, $N$, provide basic information needed for cumulative damage modelling. A
computer program was developed to extract these parameters and others listed later, from the deformation time history.

The cumulative damage model summarized in Section 2.2 (see Eqs. (2.8) and (2.9)) is based on the two hypotheses of Manson-Coffin relationship and Miner's rule (Krawinkler et al, 1983). The first hypothesis postulates that for constant amplitude cycling, the number of excursions to failure, \( N_f \), and the plastic deformation range, \( \Delta \delta_p \), are related by the following equation:

\[
N_f = C^{-1} (\Delta \delta_p)^{-c}
\]  

(4.11)

where \( C \) and \( c \) are structural performance parameters that have to be determined experimentally. The equation implies that on a log-log plot, the relationship between \( N_f \) and \( \Delta \delta_p \) is linear.

The second hypothesis is Miner's rule of linear damage accumulation, which postulates that the damage per excursion is \( 1 / N_f \), and that the damage from excursions with different plastic deformation ranges, \( \Delta \delta_{pi} \), can be linearly combined, thus giving Eq. (2.8). Eq. (2.9), the normalized version of Eq. (2.8), is preferred for reasons discussed in Section 2.2.

If these hypotheses and models were accurate, a total damage of \( D(C,c) = 1.0 \) would constitute failure. Because of the known shortcomings of Miner's rule (neglect of mean deformation and sequence effects) and the scatter in the structural performance parameters \( C \) and \( c \), the limit value of damage that constitutes failure is a random variable.

Fig. 4.33 illustrates the following cumulative damage parameters, which are evaluated in this study for bilinear systems only:

- \( D(c) = \Sigma (\Delta \delta_{pi} / \delta_y)c \), the cumulative damage index. This parameter is evaluated for \( c = 1.0, 1.5 \) and \( 2.0 \). \( D(1.0) \) is the sum of the plastic deformation ranges.

- \( N \), the number of inelastic excursions.

- \( \Sigma (\Delta \delta_{pi} / \delta_y) / N \), the mean of the plastic deformation ranges.
• \((\Delta \delta_p / \delta_y)_{max}\), the maximum plastic deformation range, which contributes most to damage. This parameter cannot exceed the value \(2(\mu - 1)(1 - \alpha)\) as shown in Fig. 4.33.

• \(\delta_{p,mean} / \delta_y\), the mean plastic deformation, which is the centroid of the plastic deformation ranges about the zero displacement axis. It is an indicator of drifting. For a perfectly symmetric loading history, \(\delta_{p,mean} / \delta_y\) is equal to zero. For a prescribed target ductility ratio \(\mu\), \(\delta_{p,mean} / \delta_y\) cannot exceed the value \([\mu(1+\alpha) + (1-\alpha)] / 2\).

For bilinear systems, the hysteretic energy \((HE)\) and the sum of the plastic deformation ranges can be related as follows:

\[
HE = F_y \sum_{i=1}^{N} \Delta \delta_{pi}
\]

\[
NHE = \frac{HE}{F_y \delta_y} = \sum_{i=1}^{N} \frac{(\Delta \delta_{pi})}{\delta_y} = D(1.0)
\]

where \(NHE\) = Normalized Hysteretic Energy. The approximation associated with Eqs. (4.12) and (4.13) is in the order of 1% in this study (for closed hysteresis loops the error is equal to zero). Note that the normalizing factor \(F_y \delta_y\), for a unit mass system, depends on the ground motion record, period \(T\) and target ductility ratio \(\mu\) as follows:

\[
F_y \delta_y = \frac{F^2_y(\mu) T^2}{4\pi^2}
\]

The other energy demand parameters discussed in Section 2.2 are also of relevance to cumulative damage assessment. As will be shown later, there are clear patterns associated with energy and cumulative damage parameters that may be of direct importance and use for the proposed seismic design procedure discussed in Section 2.1. Such parameters are to be used to weigh the ductility capacity of different structural systems to account for cumulative damage effects (see Fig. 2.3).

Subsection 4.4.3 discusses the attenuation of energy and cumulative damage parameters for the Whittier Narrows ground motions (33w records) using bilinear \(SDOF\) systems with 10% strain hardening. Subsection 4.4.4 discusses the statistical analysis
performed on the other two data sets (the 15s and 10w records) for bilinear and stiffness degrading models with 0, 2 and 10% strain hardening.

 It must be emphasized that the results shown here are based on the response of SDOF systems whose yield strength has been tuned to result in constant ductility demands for each record. Thus, the yield strength and yield displacement of each system depend on the record as well as the system's period and selected ductility ratio. Also, it should be emphasized that all energy calculations, and the subsequently discussed graphs of energy terms, are presented per unit mass. Energies for real systems can be obtained by multiplying the presented results by the mass of the system.

4.4.2 Specific Observations on Energy and Cumulative Damage

General information on energy terms is derived from energy time histories of the type shown in Figs. 4.34 and 4.35. The figures show additive energy terms in sequence of $DE, TDE = DE + HE$ (for inelastic systems only), $DE + HE + KE$ and $DE + HE + KE + RSE$. The latter sum amounts to the energy imparted to the system by the ground motion (input energy, $IE$). Note that the $KE$, and consequently the $IE$, is based on the absolute rather than relative velocities (see Uang and Bertero, 1988).

Results presented in this subsection are for bilinear systems with 10% strain hardening. Observations discussed here are site-specific and cannot be generalized. For observations on the attenuation characteristics of energy and cumulative damage demands, see Subsection 4.4.3, and for a statistical evaluation see Subsection 4.4.4.

Fig. 4.34 shows the energy time histories for the Alhambra record, which is typical of most Whittier Narrows ground motions. The maximum input energy, $IE_{max}$, occurs usually at the end of the ground motion, especially for long period systems. For short period systems ($T < 0.2$ sec), there are some exceptions and the maximum may occur within the record. Excluding these exceptions, typical Whittier Narrows records are not overpowered by large peaks in the kinetic energy, observed often in records with large velocity pulses. The major exception is the previously discussed Downey record, which has two very large velocity pulses. The effect of these pulses is evident in the energy time histories shown in Fig. 4.35. In this case, the maximum input energy, $IE_{max}$, for the elastic as well as inelastic systems (even for $\mu = 4$) occurs within the record and is characterized by large peaks in the kinetic energy. This peculiarity supports the decision to
exclude the Downey record from the regression analysis on ground motion as well as strength and energy demand parameters.

The KE and RSE diminish near the end of the ground motion record. Therefore, the difference between IE and TDE also diminishes for both elastic and inelastic systems. The DE and KE are smaller for inelastic systems, implying smaller relative and absolute velocities, respectively.

From this point onwards, all energy terms refer to values at the end of the used portion of the record (the strong motion duration plus 2 sec on either side), except if specifically mentioned otherwise as for the maximum input energy, IE$_{\text{max}}$, which may occur within the record.

Fig. 4.36 shows the input energy spectra for elastic and inelastic systems for both the Alhambra and Downey records. Each figure shows four curves, IE$_{\text{max}}$ and IE for elastic and inelastic ($\mu = 4$) systems. For the Alhambra record, IE$_{\text{max}}$ and IE are close to each other for elastic systems and almost identical for inelastic systems. However, for the Downey record, the two curves diverge, particularly in the short period range, where IE$_{\text{max}}$ is much greater than IE. Comparing Figs. 4.36 (a) and 4.3 (a) shows that the peaks in the elastic input energy spectra of the Alhambra record correspond to those in the elastic response spectrum, though their relative magnitude increases with period. The peaks in the elastic input energy spectra tend to diminish for inelastic systems. It is interesting to note that the predominant peak in the input energy spectra for the Downey record is shifted more towards the long period range, compared to typical records like Alhambra (see the mean IE spectra for the 15s and 10w records in Fig. 4.62).

The seismic input energy imparted to an inelastic system during an earthquake is dissipated by both viscous damping (DE) and inelastic deformations (HE). Fig. 4.37 shows the hysteretic energy, HE, spectra for the Alhambra record for $\mu = 2, 3, 4$. It is noted that the hysteretic energy spectra are not very sensitive to the target ductility ratio $\mu$.

It is a matter of importance in design to identify the proportions in which DE and HE contribute to the total dissipated energy, TDE. The ratio HE / TDE for the Alhambra record (Fig. 4.38) shows that the contribution of HE to TDE increases slowly with the target ductility ratio $\mu$, though the HE spectra (Fig. 4.37) do not change that much with the target ductility ratio. This suggests that the DE decreases with the target ductility ratio, i.e., smaller relative velocities.
Fig. 4.39 shows the sum of the plastic deformation ranges $D(c) = \Sigma(\Delta \delta_p / \delta_y)$, which is also (approximately, within 1% error) equal to the normalized hysteretic energy $NHE$, for the Alhambra record. A comparison of $HE$ with $NHE$, in Figs. 4.37 and 4.39 respectively, shows that $HE$ is not very sensitive to the target ductility ratio, while $NHE$ is, and the shape of the $HE$ spectra resembles that of the inelastic IE spectra (Fig. 4.36 (a)).

Fig. 4.40 shows the cumulative damage index spectra, $D(c) = \Sigma(\Delta \delta_p / \delta_y)^c$ for $c = 2$, for Alhambra. Comparing Figs. 4.40 and 4.39 shows that the $NHE$ is indeed a good approximation of the relative damage given by $D(2.0)$. This conclusion was also confirmed for $c = 1.5$. This important result will be further discussed later.

Fig. 4.41 shows the maximum plastic deformation range, $(\Delta \delta_p / \delta_y)_{max}$ for Alhambra and Fig. 4.42 shows the mean plastic deformation, $\delta_{p,\text{mean}} / \delta_y$. The closer the mean plastic deformation is to zero, the more symmetric the hysteresis loops are. It is interesting to note by comparing Figs. 4.39, 4.41 and 4.42 that the peaks in $\delta_{p,\text{mean}} / \delta_y$ correspond to valleys in both $NHE = \Sigma(\Delta \delta_p / \delta_y)$ and $(\Delta \delta_p / \delta_y)_{max}$ and vice versa. This confirms that the more the system drifts in one direction, the smaller the normalized hysteretic energy dissipated and the plastic deformation ranges become. Note that this is true only for results tuned to the same target ductility ratio.

4.4.3 Attenuation of Energy and Cumulative Damage Parameters

All results presented in this subsection are for bilinear SDOF systems with 10% strain hardening. The attenuation of energy and cumulative damage parameters is evaluated in the same fashion as that of the strength demands discussed in Subsection 4.2.1. Samples of the regression of $HE$ and $IE$ versus epicentral distance, for $T = 0.2$ sec and target ductility ratio $\mu = 4$, are shown in Fig. 4.43. The attenuation follows the same pattern as that of aforementioned ground motion and strength demand parameters.

Regressed $HE$ spectra evaluated at different epicentral distances are shown in Fig. 4.44 for $\mu = 2$ and 4. The $HE$ spectra are not very sensitive to the target ductility ratio. They are rather similar in shape and magnitude, and all spectra, regardless of distance, show that the demand first increases and then decreases with period. This pattern can be explained by considering that the hysteretic energy per unit mass for a bilinear system can be expressed approximately by (see Eqs. (4.13) and (4.14)):
\[ HE = \frac{1}{4\pi^2} T^2 F^2_x(\mu) \sum_{i=1}^{N} (\Delta \delta_{pi} / \delta_y) \] (4.15)

It can be deduced from Figs. 4.7 to 4.9 that the product \( T^2 F^2_x(\mu) \) has a similar shape as that seen in Fig. 4.44. The term \( \Sigma(\Delta \delta_{pi} / \delta_y) \) varies from record to record and could not be correlated with epicentral distance. However, except for short periods, in average this term was not strongly period dependent for constant target ductility ratios \( \mu \). Thus, the \( HE \) spectra of Fig. 4.44 have a shape that resembles that of the product \( T^2 F^2_x(\mu) \).

Fig. 4.45 shows the regressed \( IE \) spectra for elastic and inelastic (\( \mu = 4 \)) systems, evaluated at different epicentral distances. The shape of the \( IE \) spectra is similar to that of the \( HE \) spectra, though they decrease with target ductility ratio, mainly due to decrease in the \( DE \) and \( KE \).

For the Whittier Narrows records (and for that matter for all other records used in this study), the difference between the \( IE \) and \( TDE \) is negligible at the end of the used portion of the records (defined by the strong motion duration plus two seconds at either end). This justifies the use of the strong motion portion of the record in lieu of the whole record in the dynamic time history analysis.

Also, the maximum input energy usually occurs at the end of the used portion of the records (i.e., \( IE_{max} = IE \)), except for short period structures (\( T < 0.2 \) sec). However, this finding cannot be generalized to ground motions with large velocity pulses, where \( IE_{max} \) may considerably exceed the \( IE \).

The ratio \( HE / TDE \), which identifies the fraction of total energy dissipation consumed by hysteretic action, was found to be a rather stable parameter for all records regardless of epicentral distance. In fact, it is the parameter with the least standard deviation as shown in Fig. 4.46 (a). Fig. 4.46 (b) shows the mean \( HE / TDE \) spectra for \( \mu = 2, 3 \) and 4 for the 33 Whittier Narrows (33w) records, irrespective of epicentral distance. The figure indicates that for the Whittier Narrows Earthquake, the ratio \( HE / TDE \) is not very sensitive to the period and increases slowly with the target ductility ratio \( \mu \).

Distance independent mean values of cumulative damage parameters were also evaluated for the Whittier Narrows records. The cumulative damage index
\[ D(c) = \Sigma(\Delta \gamma_p / \Delta \gamma) \] was evaluated together with several other related parameters as discussed in Subsection 4.4.1.

The ratio \( D(c) / D(1.0) \) is an indication of how different the cumulative damage index is, from the NHE (represented by \( D(1.0) \)). Fig. 4.47 (a) shows the variation of the ratio \( D(c) / D(1.0) \), irrespective of epicentral distance. Fig. 4.47 (b) shows the means of the same ratio for \( c = 1.0, 1.5 \) and 2.0 and for target ductility ratios \( \mu = 2, 3 \) and 4. The following can be concluded from the Fig. 4.47:

- In average, the ratio \( D(c) / D(1.0) \) is period independent with relatively small standard deviation. The ratio \( D(c) / D(1.0) \) and its variation increases with the parameter \( c \). This implies that the NHE (or \( D(1.0) \)) is a very good cumulative damage index for a given \( c \). This has an important implication on cumulative damage prediction and whether it is justifiable to use the more elaborate and complicated cumulative damage models. Note that NHE does not require any re-arrangement of inelastic excursions.

- Although the NHE is a good cumulative damage index, it cannot be used to compare cumulative damage of systems with different \( c \) values. Fig. 4.47 (b) can be used for such purposes.

4.4.4 Statistical Study on Energy and Cumulative Damage Parameters

Statistical analysis of the aforementioned energy and cumulative damage demand parameters was carried out on the 15s and 10w data sets. Except for dimensional energy terms, results are independent of the strong motion severity (scale) because they are tuned to prescribed ductility ratios \( \mu = 1, 2, 3, 4, 5, 6 \) and 8. Dimensional energy terms are presented for records scaled to \( PGA = 0.4g \), as was done for elastic and inelastic strength demands. Cumulative damage parameters are evaluated for bilinear systems only, whereas energy demand parameters are evaluated for both bilinear and stiffness degrading models. The mean and mean plus (or minus) standard deviation (mean ± \( \sigma \)) are presented for the bilinear model with 10% strain hardening. The effects of using different strain hardening ratios (\( \alpha = 0 \) and 2%) and different hysteresis models (stiffness degrading versus bilinear) are presented as ratios to the results obtained from the bilinear model.

Fig. 4.48 shows spectra for the number of inelastic excursions, \( N \), for both the 15s and 10w data sets. The figure for the 15s records shows that for constant target ductility
ratio a bilinear SDOF system will experience many more excursions in the short period range \( T = 0.2 \text{ sec} \) and the spectra drop sharply towards long period. The 10w records show quite a different pattern, with long period values much higher than those of the 15s records. This must be attributed to the low energy content of the Whittier Narrows Earthquake in the long period range, which resulted in very low strength demands, \( F_y(\mu) \), in order to obtain the prescribed ductility ratios, which in turn led to more inelastic excursions.

Figs. 4.49 and 4.50 show the sum of the plastic deformation ranges, \( \Sigma(\Delta \delta_p/\delta_y) \) = NHE, and the mean of the plastic deformation ranges, \( \Sigma(\Delta \delta_p/\delta_y) / N \), respectively, for the 15s records. Comparing Figs. 4.49 and 4.48 (a), the steep decay in the number of inelastic excursion curves is not reflected in the sum of plastic deformation ranges, which implies that, in average, the excursions in the short period range must be smaller. This is confirmed in Fig. 4.50. The computed mean values of \( \Sigma(\Delta \delta_p/\delta_y) \) are larger than anticipated for long period systems from past studies (Hadidi-Tamjed, 1988).

The maximum plastic deformation range, \((\Delta \delta_p / \delta_y)_{max}\) is shown in Fig. 4.51 for both the 15s and 10w records. As illustrated in Fig. 4.33, \((\Delta \delta_p / \delta_y)_{max}\) cannot exceed the value \( 2(\mu-1)(1-\alpha) \). Fig. 4.51 shows that \((\Delta \delta_p / \delta_y)_{max}\) is almost period independent and similar for both data sets. The ratio of \((\Delta \delta_p / \delta_y)_{max}\) to \( 2(\mu-1)(1-\alpha) \) is in the order of 0.75 to 0.80. These values can be used to estimate the maximum plastic deformation range, which causes much of the cumulative damage, directly from the target ductility ratio \( \mu \) and strain hardening \( \alpha \).

The mean plastic deformation, \( \delta_{p,mean}/\delta_y \), which is an indicator of drifting of the SDOF system, is shown in Fig. 4.52 for the 15s records. Part (a) is for 0% strain hardening and (b) is for 10%. \( \delta_{p,mean}/\delta_y \) cannot exceed the value \( (\mu(1+\alpha) + (1-\alpha))/2 \) for a given target ductility \( \mu \) and strain hardening \( \alpha \) (see Fig. 4.33). Fig. 4.52 shows that bilinear systems with 10% strain hardening drift much less than those with no strain hardening, particularly in the short period range (which has implications on the corresponding hysteretic energies - see discussion on Fig. 4.56).

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1 Note that all deformation parameters are normalized with respect to the yield displacement \( \delta_y \), which is a function of both the period of the system and the input ground motion.
The ratio of cumulative damage indices, $D(c) / D(1.0)$, where $D(c) = \Sigma (\Delta \delta_p / \delta_p)^c$ and $D(1.0) = NHE$, is of significance in addressing the question of how good $NHE$ is as a cumulative damage index. Figs. 4.53 and 4.54 show that, for $c$ values less than 2.0, the ratio $D(c) / D(1.0)$ is relatively insensitive to period and is similar for both data sets. Thus, $NHE$ can be used as a relative cumulative damage index and this index can be weighted for different ductility ratios and $c$ values on hand of the data presented in Figs. 4.53 and 4.54.

Therefore, it becomes of interest to study carefully the variation and effect of important parameters on the $NHE$. Fig. 4.55 shows the mean and mean $+ \sigma$ spectra of the $NHE$ for the 15s records. Fig. 4.55 (a) is almost identical to Fig. 4.49 with discrepancies in the order of 1%. Fig. 4.56 illustrates the effect of both strain hardening and hysteresis models on $NHE$. For bilinear systems, the $NHE$ for 0% strain hardening is always less than or equal to that of 10% strain hardening, because such systems tend to drift more and dissipate less hysteretic energy for the same ductility ratio. The difference is larger for short period systems. On the other hand, stiffness degrading systems tend to dissipate more $NHE$ than bilinear systems and much more so for short period systems. This is because the stiffness degrading model undergoes many more inelastic excursions than their bilinear counterparts.

Fig. 4.57 shows the mean and mean $+ \sigma$ spectra of the $HE / TDE$ ratio for bilinear systems with 10% strain hardening for the 15s records. The variation in this parameter is small and except for short periods, it is not very sensitive to both period and target ductility ratio. Comparing the mean spectra of $HE / TDE$ for the 15s and 33w records (Figs. 4.57 (a) and 4.46 (b)), shows that the ratio is smaller for the Whittier Narrows Earthquake (33w records), especially for long periods. This, again, is attributed to the low energy content of the Whittier Narrows Earthquake in this period range.

The effect of strain hardening and hysteresis model on the $HE / TDE$ ratio is shown in Fig. 4.58 for the 15s records. Except for very short periods, strain hardening has no significant effect on $HE / TDE$ for bilinear systems. Stiffness degrading systems tend to have higher $HE / TDE$ ratios, especially for short periods and higher ductility ratios.

As was the case for the Whittier Narrows records, and except for short period systems, the maximum input energy, $IE_{max}$, for the 15s records usually occurs at the end of the records. Fig. 4.59 shows $IE_{max} / IE$ for elastic and inelastic bilinear systems. The
ratio $IE_{max} / IE$ is large for very short period systems, which is due to large peaks in the kinetic energy.

Fig. 4.60 shows the effect of hysteresis model on $IE_{max}$. Except for very short periods, the maximum input energy for the stiffness degrading model is less than that of the bilinear.

It is needs to be emphasized that both data sets were scaled to $PGA = 0.4g$. Results presented on energy terms, as well as the previously discussed elastic and inelastic strength demands, depend on the severity (scale) of the ground motions. The mean hysteretic and input energy spectra for both the 15s and 10w records are shown in Figs. 4.61 and 4.62, respectively. The spectra are for bilinear systems with 10% strain hardening. Both the $HE$ and $IE$ spectra are not very sensitive to the ductility ratio and are similar in shapes. The shapes of the $HE$ spectra are very different from those of the $NHE$ spectra (Fig. 4.55 for the 15s records), which illustrates the significant effect of the normalizing factor $F_y \delta_y$ on the $NHE$ spectra. Also, the $HE$ and $IE$ spectra for the 15s records are almost three times as large as their 10w counterparts, although the strength demand spectra (see Figs. 4.12 and 4.13) are comparable.

4.4.5 Effect of Strong Motion Duration on Cumulative Damage

In concept, every excursion the system experiences causes damage, and damage accumulates as the number of excursions increases. The longer the strong motion duration, the more cumulative damage is inflicted on the system. Elastic excursions cause very little damage compared to inelastic ones and are, therefore, neglected. The number, sequence and relative magnitude of the inelastic excursions have an important effect on cumulative damage. However, these parameters not only depend on the characteristics of the strong ground motion, but also on the period, yield level and hysteretic model of the system, as well. Therefore, the issue of correlating cumulative damage with strong motion duration is not a simple one, and we come to the conclusion that there is no unique definition of the "effective" strong ground motion duration that applies to all structural systems.

All our attempts to find an "effective" strong motion duration parameter that correlates well with cumulative damage did not lead to satisfactory results. More research needs to be directed towards characterizing system dependent "effective" strong motion duration as a function of the aforementioned parameters.
In order to provide some insight into the complexity of the problem, the effect of strong motion duration, defined by the \textit{RMSA} method (see Section 3.2), on important cumulative damage parameters is illustrated in Figs. 4.63 and 4.64 for the Whittier Narrows Earthquake (33w records).

Fig. 4.63 illustrates the effect of strong motion duration on both the number of inelastic excursions, \(N\), and the cumulative damage index, \(D(1.0) = NHE\), for bilinear systems with 10\% strain hardening and target ductility ratio \(\mu = 4\). The figure shows that for this earthquake a trend appears to exist in both parameters towards a linear increase with strong motion duration, irrespective of the system's period. The pattern is more pronounced for \(N\) than for \(NHE\).

Fig. 4.64 shows the effect of strong motion duration on the cumulative damage index, \(D(1.0) \approx NHE\), for discrete periods and different ductility ratios. The pattern noted in the previous figure is no longer evident.

4.5 Damage Potential of the Whittier Narrows Earthquake

4.5.1 Ductility Demands

A direct way of relating seismic demands and capacities is to superimpose the estimates of strength capacities \(E_g\) of the five types of generic structures discussed in Section 2.3, on the regressed spectra of strength demands for different ductility ratios derived from the Whittier Narrows strong motion records (see Subsection 4.2.1). Figs. 4.65 and 4.66 show the estimated strength capacities of the five types of generic structures superimposed on the regressed elastic and inelastic strength demands at epicentral distances of 10 and 20 km, respectively. At an epicentral distance of 10 km (Fig. 4.65), it can be seen that, even with the large overstrength available in the generic structures, there are period ranges in which all of the generic structures are expected to experience global inelastic deformations. It is fortunate that the overstrength is highest in the short period range where the strength demands are highest. At an epicentral distance of 10 km the global ductility demands are not expected to exceed a value of 2 except for the \textit{CMF68}.

The attenuation of ductility demands can be evaluated by comparing Fig. 4.65 with 4.66, which shows similar information at an epicentral distance of 20 km. At this distance the global ductility demands are negligible except for the \textit{CMF68}. This does not mean that the local ductility (member ductility) demands vanish, because member yielding is defined
by the $E_l$ level and the $E_l$ strength may be considerably smaller than the $E_g$ strength (see Fig. 2.3).

Information of the type presented in Figs. 4.65 and 4.66, together with the attenuation relationships for strength demands discussed in Section 4.2.1, can be utilized to assess the attenuation of ductility demands for different types of generic structures as shown in Figs. 4.67 to 4.69. The attenuation equations for the elastic and inelastic strength demands are evaluated at a given period, $T$, and a given epicentral distance, $e$, giving four values, $F_{y,e}$, $F_{y}(2)$, $F_{y}(3)$ and $F_{y}(4)$ associated with the target ductility ratios $\mu = 1, 2, 3$ and 4, respectively. In the $F_y-\mu$ domain, where $\mu$ (the ordinate) is the ductility demand associated with the strength demand $F_y$ (the abscissa), cubic spline interpolations were fitted to connect the four data points. The global ductility ratio, $\mu_g$, is first evaluated as the interpolated ductility ratio corresponding to the strength capacity $E_g$. The local ductility ratio, $\mu_l$, is then evaluated as $\mu_l = \mu_g E_g / E_l$. If $E_g$ is greater than $F_{y,e}$, then $\mu_g < 1$ (elastic) and $\mu_l = F_{y,e} / E_l$. The fact that $\mu_l$ is obtained from a different equation depending on whether $E_g$ is smaller or greater than the elastic strength demand $F_{y,e}$, explains the kinks in the local ductility demand versus epicentral distance curves in Figs. 4.67 to 4.69 at the points where the global ductility ratios reach a value of 1.0.

Figs. 4.67 to 4.69 show typical examples of the attenuation of ductility demands with epicentral distance for 2, 5, and 10 story buildings of all five generic types. Note that the results are presented for buildings with specified numbers of stories, and not specified periods. In fact, the periods shown on the figures for buildings with an equal number of stories may vary significantly. The periods shown are those computed from the appropriate code equations. The figures show the attenuation of global ductility demands ($\delta_{max} / \delta_y$ in Fig. 2.5) as well as local ones ($\delta_{max} / \delta_y$ in Fig. 2.5). The demands vary significantly with structural type and number of stories. The local ductility demands are significant, particularly for CMF68 structures. For these structures localized inelastic behavior is predicted for epicentral distances up to 50 km (Figs. 4.67 and 4.68 (a)).

Information of the type presented in Figs. 4.65 to 4.69 permits an assessment of the damage potential of the Whittier Narrows earthquake. Based on the information generated in this study it must be concluded that this earthquake was a relatively severe test for modern structures located near the source. At specific sites, at which localized site conditions led to greater amplification of motions than indicated by the regressed spectra, the predicted ductility demands would be even considerably higher. Since little damage has been reported for modern multi-story buildings in the Whittier Narrows earthquake, this
raises the question whether these ductility ratios were tolerated without much visible damage, or whether structures have much more overstrength than anticipated, or whether our models for predicting performance are overly conservative. The answers to this question are not yet clear.

4.5.2 Observed versus Predicted Damage

Several damage surveys and specific damage studies on the Whittier Narrows earthquake have been reported in the literature. For instance, two issues of the EERI Earthquake Spectra are in part devoted to this topic (Earthquake Spectra, 1988/1 and 1988/2). Based on the published information the conclusion can be drawn that the types of structures investigated in this study (modern frame type structures) have performed well in this earthquake, with few exceptions. Structural damage was noted primarily in single family residences, unreinforced masonry structures, tilt-up structures, and other mostly low-rise structures which were not of the types investigated in this study.

The predominance of damage in low-rise construction comes as no surprise if one considers the high strength demands, elastic as well as inelastic, evident for short period structures in the regressed strength demand spectra discussed in Subsection 4.2.1. Even with the existence of large overstrength (in relation to a customarily assumed code design levels), inelastic demands have to be anticipated for short period structures near the epicenter. Short period implies here about 0.2 seconds and less, since the elastic as well as inelastic strength demands in all near-source spectra decrease rapidly for structures with periods above 0.2 seconds. Unfortunately, structures with very short periods could not be considered in the study on generic structures because of their large variability in strength. The strength capacity of such structures has to be evaluated on a case-by-case basis, but the seismic demand information discussed in Chapter 2 is general and can be applied for all cases.

Several instrumented multi-story buildings subjected to the Whittier Narrows earthquake were the subject of specific studies (e.g., Fenves 1989, Filippou 1989, Moehle 1989, and Pardoen 1989). From the studied structures, three fall, to some extent, within the range of generic structures investigated here. These structures are:

1. Los Angeles-Sears Warehouse, CSMIP Station #463 (Filippou 1989)
   5-story reinforced concrete structure
   Lateral load resisting system: ductile perimeter frame
   Designed according to the 1970 Los Angeles Building Code
Epicentral distance: 14 km.

   20-story reinforced concrete structure
   Lateral load resisting system: ductile moment resisting frames
   Designed according to the 1966 Los Angeles Building Code
   Epicentral distance: 28 km.

3. Los Angeles-Hollywood Storage Bldg., CSMIP Station #236 (Fenves 1989)
   14-story reinforced concrete structure
   Lateral load resisting system: moment resisting frames
   Designed in 1925
   Epicentral distance: 25 km.

To our knowledge, no structural damage was reported in any of the three buildings. Nevertheless, the three buildings can be used to illustrate a few important points with regard to the demand (average demands based on regression analysis) and capacity (estimated capacity of generic structures) evaluation discussed here.

Fig. 4.70 shows the $E_l$ and $E_g$ capacity estimates for CMF68 structures superimposed on the elastic demand spectra for the epicentral distances of the three structures ($e = 14, 25, \text{and} \ 28 \text{ km}$). For structure #2 ($e = 28 \text{ km}$, 1968 code period $T = 2.0 \text{ sec}$) it can be seen that the estimated capacities $E_l$ and $E_g$ are much larger than the elastic demands and, therefore, no damage is expected. This is supported by the observation that the fundamental periods in the two principal directions, 2.31 and 2.05 sec as estimated by Filippou, are close to the period on which the generic design was based (2.0 sec). Fig. 4.70 also indicates that structure #3 ($e = 25 \text{ km}$, "code" period $T = 1.4 \text{ sec}$) is in a safe range in which no damage is expected. However, the observation made from the figure may be misleading since the estimated fundamental periods in the two principal directions, 1.90 sec in the transverse direction and 0.62 sec in the longitudinal direction (Fenves, 1989), are quite different from period on which the generic design was based (1.4 sec). Comparing the estimated capacities of 14-story buildings with the elastic demand at $e = 25 \text{ km}$ and $T = 0.62 \text{ sec}$, one would have to expect considerable inelastic deformations in the structure. The reason why no damage was observed, which is also the reason why the real period (0.62 sec) is so different from the "design" period, is the presence of stiff and strong infill panels in the two exterior longitudinal frames. These walls are both detrimental and beneficial, as they shift the period into a range of much higher seismic demand but also strengthen the structure sufficient to prevent damage in this earthquake.

The preceding paragraph illustrates some of the issues that have to be considered in an attempt to assess damage potential by means of the simplified demand and capacity
models proposed here and implemented in this chapter through superposition of strength demands and capacities. A derivation of specific conclusions from this superposition has to be done with caution and with the following issues in mind. Information of the type illustrated in Figs. 4.65 to 4.69 provides a global picture of damage potential, based on a direction and site independent attenuation of ground motions and a general evaluation of structure strength that cannot account for the great variation of strength and stiffness properties of real structures.

When drawing conclusions on specific structures the following issues have to be considered:

1. The site-specific seismic demands may be quite different from those obtained by regression.

2. The fundamental period of the structure may be quite different from the "design" period on which the capacity estimates are based.

3. The true strength capacity of the structure may be quite different from the estimated one. However, if structural damage is observed in a case in which the generic structure strength exceeds the elastic strength demand, a structural weakness is indicated. This weakness can only be found through a case-specific study.

4. The demand evaluation is based on the response of SDOF systems and no consideration is given to higher mode effects.

5. The excitation at the base of the structure may be different from the free-field ground motion at the site.

In a global damage evaluation the first three issues cannot be addressed. However, general information can be generated on the last two issues. Issue 5 is not a subject of this study, but issue 4 will be discussed in Chapter 5.
STRENGTH CAPACITY vs. DUCTILITY DEMAND - (bi-10)

Alhambra (270°), Bilinear, T = 0.9 sec, α = 10%

Strength Capacity, $F_y(\mu) / W$

(a) Nonmonotonic Relationship Between $F_y(\mu)$ and $\mu$

DISPLACEMENT TIME HISTORIES FOR DIFFERENT STRENGTH CAPACITIES - (bi-3.10)

Alhambra (270°), Bilinear, T = 0.9 sec, $\mu = 3$, α = 10%

(b) Displacement Time Histories for the Three SDOF Systems with $\mu = 3$ in Fig. 4.1(a)

Fig. 4.1 Problems in Determining the Inelastic Strength Demand, $F_y(\mu)$
(a) Bilinear Model

(b) Modified-Clough Stiffness Degrading Model

* Reloading at point A follows the path ABC.
* Reloading at point D follows the path DBC if DB's slope is larger than DC, else it follows DC.

Fig. 4.2 Hysteretic Models Used in this Study
SITE SPECIFIC STRENGTH DEMAND SPECTRA - (bi-10)

Alhambra (270°), Bilinear, α = 10%

(a) Alhambra Fremont School Record (270°)

SITE SPECIFIC STRENGTH DEMAND SPECTRA - (bi-10)

L.A. Obregon Park (0°), Bilinear, α = 10%

(b) Los Angeles Obregon Park Record (0°)

Fig. 4.3 Site Specific Strength Demand Spectra
Fig. 4.4 Regression of Inelastic Strength Demand $F_y(\mu = 4)$ for $T = 0.5$ sec (33w Records)

Fig. 4.5 Comparison of Normalized Attenuation Curves for PGA and $F_y(\mu = 4)$ (33w Records)
Fig. 4.6 Attenuation of Elastic Strength Demand Spectra, $F_{y,e}$ (33w Records)
ATTENUATION OF INELASTIC STRENGTH DEMAND SPECTRA, $F_y(\mu = 2) - (33w.bi-2.10)$

33w Records, Bilinear, $\alpha = 10\%$, Regressed Spectra

- $e = 6, 10, 20, 30, 40, 50, 80$ km
- ZC / 12 (1988 UBC - $S_2$)

(a) Regressed Spectra

ATTENUATION OF INELASTIC STRENGTH DEMAND SPECTRA, $F_y(\mu = 2) - (33w.bi-2.10)$

33w Records, Bilinear, $\alpha = 10\%$, Regressed+$\sigma$ Spectra

- $e = 6, 10, 20, 30, 40, 50, 80$ km
- ZC / 12 (1988 UBC - $S_2$)

(b) Regressed+$\sigma$ Spectra

Fig. 4.7 Attenuation of Inelastic Strength Demand Spectra, $F_y(\mu = 2)$ (33w Records)
ATTENUATION OF INELASTIC STRENGTH DEMAND SPECTRA, $F_y(\mu = 3)$ - (33w.bi-3.10)

33w Records, Bilinear, $\alpha = 10\%$, Regressed Spectra

![Graph](image)

- $e = 6, 10, 20, 30, 40, 50, 80$ km
- ZC / 12 (1988 UBC - $S_2$)

(a) Regressed Spectra

ATTENUATION OF INELASTIC STRENGTH DEMAND SPECTRA, $F_y(\mu = 3)$ - (33w.bi-3.10)

33w Records, Bilinear, $\alpha = 10\%$, Regressed+$\sigma$ Spectra

![Graph](image)

- $e = 6, 10, 20, 30, 40, 50, 80$ km
- ZC / 12 (1988 UBC - $S_2$)

(b) Regressed+$\sigma$ Spectra

Fig. 4.8 Attenuation of Inelastic Strength Demand Spectra, $F_y(\mu = 3)$ (33w Records)
ATTENUATION OF INELASTIC STRENGTH DEMAND SPECTRA, $F_Y(\mu = 4)$ - (33w.bi-4.10)

33w Records, Bilinear, $\alpha = 10\%$, Regressed Spectra

(a) Regressed Spectra

ATTENUATION OF INELASTIC STRENGTH DEMAND SPECTRA, $F_Y(\mu = 4)$ - (33w.bi-4.10)

33w Records, Bilinear, $\alpha = 10\%$, Regressed+$\sigma$ Spectra

(b) Regressed+$\sigma$ Spectra

Fig. 4.9 Attenuation of Inelastic Strength Demand Spectra, $F_Y(\mu = 4)$ (33w Records)
ATTENUATION OF NORMALIZED ELASTIC & INELASTIC STRENGTH DEMAND SPECTRA, $F_y(\mu) / PGA$ - (33w.bi-10)

33w Records, Regressed Spectra for $e = 10, 30, 50, 80$ km, Bilinear, $\alpha = 10\%$

Fig. 4.10 Attenuation of Normalized Elastic and Inelastic Strength Demand Spectra (33w Records)
Fig. 4.11 Comparison of Site Specific vs. Regressed Strength Demand Spectra
MEAN ELASTIC & INELASTIC STRENGTH DEMAND SPECTRA - (15s.bi-00)

15s Records, Scaled to PGA = 0.4g, Bilinear, \( \alpha = 0\% \)

\[ F_y(\mu) / W \]

\( T \) (sec)

(a) For Bilinear Systems with No Strain Hardening

MEAN ELASTIC & INELASTIC STRENGTH DEMAND SPECTRA - (15s.bi-10)

15s Records, Scaled to PGA = 0.4g, Bilinear, \( \alpha = 10\% \)

\[ F_y(\mu) / W \]

\( T \) (sec)

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.12 Mean Elastic and Inelastic Strength Demand Spectra for 15s Records

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MEAN ELASTIC & INELASTIC STRENGTH DEMAND SPECTRA - (10w.bi-00)

10w Records, Scaled to PGA = 0.4g, Bilinear, $\alpha = 0\%$

(a) For Bilinear Systems with No Strain Hardening

MEAN ELASTIC & INELASTIC STRENGTH DEMAND SPECTRA - (10w.bi-10)

10w Records, Scaled to PGA = 0.4g, Bilinear, $\alpha = 10\%$

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.13 Mean Elastic and Inelastic Strength Demand Spectra for 10w Records
SITE SPECIFIC STRENGTH REDUCTION FACTORS - (bi-10)

Alhambra (270°), Bilinear, α = 10%

(a) Alhambra Fremont School Record (270°)

SITE SPECIFIC STRENGTH REDUCTION FACTORS - (bi-10)

L.A. Obregon Park (0°), Bilinear, α = 10%

(b) Los Angeles Obregon Park Record (0°)

Fig. 4.14 Site Specific Strength Reduction Factors

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Fig. 4.15  Regression of Strength Reduction Factor $R_y(\mu = 4)$ for $T = 0.2$ sec (33w Records)

Fig. 4.16  Comparison of Regressed Strength Reduction Factors for 33w & 15s Records
Fig. 4.17  Strength Reduction Factors for Bilinear Systems with No Strain Hardening (15s Records)
Fig. 4.18 Strength Reduction Factors for Bilinear Systems with 2% Strain Hardening (15s Records)
Fig. 4.19 Strength Reduction Factors for Bilinear Systems with 10% Strain Hardening (15s Records)
Fig. 4.20 Strength Reduction Factors for Bilinear Systems with 10% Strain Hardening (10w Records)
EFFECT OF STRAIN HARDENING ON R-FACTORS - (15s.bi-00/bi-10)

15s Records, Bilinear, \((\alpha = 0\%) / (\alpha = 10\%)\), Mean

\(R_y(\mu) / R_y(\mu)_{(bi-10)}\)

\[\mu = 2, 3, 4, 5, 6, 8 \text{ (thin \to thick lines)}\]

(a) Ratio of R-Factors for Bilinear Systems with 0% to 10% Strain Hardening

EFFECT OF STRAIN HARDENING ON R-FACTORS - (15s.bi-02/bi-10)

15s Records, Bilinear, \((\alpha = 2\%) / (\alpha = 10\%)\), Mean

\(R_y(\mu) / R_y(\mu)_{(bi-10)}\)

\[\mu = 2, 3, 4, 5, 6, 8 \text{ (thin \to thick lines)}\]

(b) Ratio of R-Factors for Bilinear Systems with 2% to 10% Strain Hardening

Fig. 4.21 Effect of Strain Hardening on the Strength Reduction Factors for 15s Records
(a) Ratio of R-Factors for Stiffness Degrading to Bilinear Systems with No Strain Hardening

(b) Ratio of R-Factors for Stiffness Degrading to Bilinear Systems with 10% Strain Hardening

Fig. 4.22 Effect of Hysteresis Model on the Strength Reduction Factors for 15s Records
REGRESSION OF STRENGTH REDUCTION FACTORS IN THE R-µ Domain - (15s.bi-00)

15s Records, Bilinear, T = 0.2 sec, α = 0%, Least Square Error Method in c(T, α)

- Data Points
- Mean at Constant µ (m₁)
- Regression Curve (m₂)
- m₂ ± σ₂

Regression Equation: \[ R = \left( c \( \frac{\mu - 1}{T} \) + 1 \right)^{1/c} \]
\[ c(T=0.2, \alpha=0) = 1.99 \rightarrow R = \sqrt{2\mu - 1} \]

Ductility Ratio, µ

(a) For Bilinear Systems with T = 0.2 sec, α = 0%

REGRESSION OF STRENGTH REDUCTION FACTORS IN THE R-µ Domain - (15s.bi-00)

15s Records, Bilinear, T = 0.9 sec, α = 0%, Least Square Error Method in c(T, α)

- Data Points
- Mean at Constant µ (m₁)
- Regression Curve (m₂)
- m₂ ± σ₂

Regression Equation: \[ R = \left( c \( \frac{\mu - 1}{T} \) + 1 \right)^{1/c} \]
\[ c(T=0.9, \alpha=0) = 1.01 \rightarrow R = \mu \]

Ductility Ratio, µ

(b) For Bilinear Systems with T = 0.9 sec, α = 0%

Fig. 4.23 Regression of Strength Reduction Factors in the R-µ Domain for Discrete Periods (15s Records)

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REGRESSION OF $c(T, \alpha)$ IN THE PERIOD DOMAIN - (15s.bi-00)

15s Records, Bilinear, $\alpha = 0\%$, Least Square Error Method in $a(\alpha)$ & $b(\alpha)$

Regression Equation: $c = \frac{T^a}{[T^a + 1]} + \frac{b}{T}$
where $a = 1.00$, $b = 0.42$ for $\alpha = 0\%$

From Fig. 4.23(a)

From Fig. 4.23(b)

T (sec)

(a) For Bilinear Systems with No Strain Hardening

REGRESSION OF $c(T, \alpha)$ IN THE PERIOD DOMAIN - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Least Square Error Method in $a(\alpha)$ & $b(\alpha)$

Regression Equation: $c = \frac{T^a}{[T^a + 1]} + \frac{b}{T}$
where $a = 0.80$, $b = 0.29$ for $\alpha = 10\%$

T (sec)

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.24 Regression of $c(T, \alpha)$ Parameter in the Period Domain (15s Records)
Fig. 4.25 Summary of R-μ-T Relationships Developed for Bilinear Systems with 0, 2 and 10% Strain Hardening (15s Records)
**Comparison of Different Stages of Regression on R-Factors - (15s.bi-00)**

15s Records, Bilinear, $\alpha = 0\%$, Data (Mean), First & Final Regression

(a) For Bilinear Systems with No Strain Hardening

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**Comparison of Different Stages of Regression on R-Factors - (15s.bi-10)**

15s Records, Bilinear, $\alpha = 10\%$, Data (Mean), First & Final Regression

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.27 Comparison of Different Stages of Regression on Strength Reduction Factors (15s Records)
VARIATION IN REgressed STRENGTH REDUCTION FACTORS - (15s.bi-00)

15s Records, Bilinear, $\alpha = 0\%$, Regressed vs. Data Mean-$\sigma$ ($m_1 - \sigma_1$)

- $\mu = 2, 3, 4, 5, 6$ and $8$ (thin → thick lines)
- solid = regressed, dashed = data (mean-$\sigma$)

$R_y(\mu)$ vs. $T$ (sec)

(a) For Bilinear Systems with No Strain Hardening

VARIATION IN REgressed STRENGTH REDUCTION FACTORS - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Regressed vs. Data Mean-$\sigma$ ($m_1 - \sigma_1$)

- $\mu = 2, 3, 4, 5, 6$ and $8$ (thin → thick lines)
- solid = regressed, dashed = data (mean-$\sigma$)

$R_y(\mu)$ vs. $T$ (sec)

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.28 Variation in Regressed Strength Reduction Factors for 15s Records
REGRESSED DUCTILITY DEMAND SPECTRA FOR CONSTANT R-FACTORS - (15s.bi-00)

15s Records, Bilinear, $\alpha = 0\%$

$R = 1, 2, 3, 4, 5, 6, 7, 8, 9, 10$ (thick $\rightarrow$ thin lines)

(a) For Bilinear Systems with No Strain Hardening

REGRESSED DUCTILITY DEMAND SPECTRA FOR CONSTANT R-FACTORS - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$

$R = 1, 2, 3, 4, 5, 6, 7, 8, 9, 10$ (thick $\rightarrow$ thin lines)

(a) For Bilinear Systems with 10% Strain Hardening

Fig. 4.29 Regressed Ductility Demand Spectra for Constant Reduction Factors (15s Records)
(a) For Bilinear Systems with No Strain Hardening

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.30 Regressed R-μ Relationships for Discrete Periods (15s Records)
Fig. 4.31 Inelastic Strength Demand Spectra Using Regressed R-μ-T Relationships
Fig. 4.32 Rain-flow Cycle Counting Method

\[ N = \text{Number of Inelastic Excursions} = 4 \]

\[ \delta_{p,\text{mean}} = \frac{\sum_{i=1}^{N} \Delta \delta_{pi} \delta_{pi}}{\sum_{i=1}^{N} \delta_{pi}} \]

\[ D(c) = \sum_{i=1}^{N} (\Delta \delta_{pi} / \delta_{y})^c \]

\[ \text{NHE} = \frac{\text{HE}}{F_y \delta_y} = D(1) = \sum_{i=1}^{N} (\Delta \delta_{pi} / \delta_{y}) \]

Fig. 4.33 Cumulative Damage Parameters
ELASTIC ENERGY TIME HISTORY

Alhambra (270°), T = 0.2 sec

(a) For an Elastic SDOF System with T = 0.2 sec

INELASTIC ENERGY TIME HISTORY - (bi-4.10)

Alhambra (270°), Bilinear, T = 0.2 sec, μ = 4, α = 10%

(b) For an Inelastic Bilinear SDOF System with T = 0.2 sec, μ = 4 and α = 10%

Fig. 4.34 Energy Time Histories for the Alhambra Fremont School Record (270°)
ELASTIC ENERGY TIME HISTORY

Downey (180°), T = 0.2 sec

- DE
- DE+KE
- DE+KE+RSE = IE

(a) For an Elastic SDOF System with T = 0.2 sec

INELASTIC ENERGY TIME HISTORY - (bi-4.10)

Downey (180°), Bilinear, T = 0.2 sec, μ = 4, α = 10%

- DE
- DE+HE
- DE+HE+KE
- DE+HE+KE+RSE = IE

(b) For an Inelastic Bilinear SDOF System with T = 0.2 sec, μ = 4 and α = 10%

Fig. 4.35 Energy Time Histories for the Downey County Maint. Bldg. Record (180°)
SITE SPECIFIC ELASTIC & INELASTIC ($\mu = 4$) INPUT ENERGY SPECTRA - (bi-10)

Alhambra ($270^\circ$), Bilinear, $\alpha = 10\%$

(a) Alhambra Fremont School Record ($270^\circ$)

SITE SPECIFIC ELASTIC & INELASTIC ($\mu = 4$) INPUT ENERGY SPECTRA - (bi-10)

Downey ($180^\circ$), Bilinear, $\alpha = 10\%$

(b) Downey County Maintenance Building ($180^\circ$)

Fig. 4.36 Site Specific Elastic & Inelastic ($\mu = 4$) Input Energy Spectra ($IE_{\text{max}}$ & $IE$)
Fig. 4.37 Hysteretic Energy Spectra for the Alhambra Fremont School Record (270°)

Fig. 4.38 HE / TDE Spectra for the Alhambra Fremont School Record (270°)
SITE SPECIFIC CUMULATIVE DAMAGE SPECTRA, $D(1.0) = \Sigma (\Delta \delta_p / \delta_y) = \text{NHE} - (\text{bi}-10)$

Alhambra (270°), Bilinear, $\alpha = 10\%$

![Graph of SITE SPECIFIC CUMULATIVE DAMAGE SPECTRA, $D(1.0) = \Sigma (\Delta \delta_p / \delta_y) = \text{NHE}$](image)

Fig. 4.39 Cumulative Damage Index, $D(1.0) = \text{NHE}$, for the Alhambra Fremont School Record (270°)

SITE SPECIFIC CUMULATIVE DAMAGE SPECTRA, $D(2.0) = \Sigma (\Delta \delta_p / \delta_y)^2$

Alhambra (270°), Bilinear, $\alpha = 10\%$

![Graph of SITE SPECIFIC CUMULATIVE DAMAGE SPECTRA, $D(2.0) = \Sigma (\Delta \delta_p / \delta_y)^2$](image)

Fig. 4.40 Cumulative Damage Index, $D(2.0)$, for the Alhambra Fremont School Record (270°)
Fig. 4.41 Maximum Plastic Deformation Range Spectra for the Alhambra Fremont School Record (270°)

Fig. 4.42 Mean Plastic Deformation Spectra for the Alhambra Fremont School Record (270°)
REGRESSION OF HYSTERETIC ENERGY - (33w.bi-4.10)

33w Records, Bilinear, $T = 0.2$ sec, $\mu = 4$, $\alpha = 10\%$

Epicentral Distance, $e$ (km)

(a) Hysteretic Energy

REGRESSION OF INPUT ENERGY - (33w.bi-4.10)

33w Records, Bilinear, $T = 0.2$ sec, $\mu = 4$, $\alpha = 10\%$

Epicentral Distance, $e$ (km)

(a) Input Energy

Fig. 4.43 Regression of Hysteretic and Input Energies for $\mu = 4$ and $T = 0.2$ sec (33w Records)
ATTENUATION OF HYSTERETIC ENERGY DEMANDS - (33w.bi-2.10)
33w Records, Regressed Spectra for e = 6, 10, 20, 30, 40, 50, 60, 80 km, Bilinear, μ = 2, α = 10%

ATTENUATION OF HYSTERETIC ENERGY DEMANDS - (33w.bi-4.10)
33w Records, Regressed Spectra for e = 6, 10, 20, 30, 40, 50, 60, 80 km, Bilinear, μ = 4, α = 10%

Fig. 4.44 Attenuation of Hysteretic Energy Spectra for 33w Records
ATTENUATION OF ELASTIC INPUT ENERGY DEMANDS - (33w-e)

33w Records, Regressed Spectra for e = 6, 10, 20, 30, 40, 50, 60, 80 km

Input Energy Per Unit Mass (cm/sec)²

T (sec)

(a) For \( \mu = 1 \) (elastic)

ATTENUATION OF INPUT ENERGY DEMANDS - (33w.bi-4.10)

33w Records, Regressed Spectra for e = 6, 10, 20, 30, 40, 50, 60, 80 km, Bilinear, \( \mu = 4, \alpha = 10\% \)

Input Energy Per Unit Mass (cm/sec)²

T (sec)

(a) For \( \mu = 4 \)

Fig. 4.45 Attenuation of Input Energy Spectra for 33w Records
VARIATION IN HE / TDE - (33w.bi-4.10)
33w Records, Bilinear, $\mu = 4$, $\alpha = 10\%$, Mean±σ

HE / TDE

T (sec)

(a) Mean±σ Spectra for $\mu = 4$

HE / TDE SPECTRA - (33w.bi-4.10)
33w Records, Bilinear, $\alpha = 10\%$, Mean

HE / TDE

T (sec)

(b) Mean Spectra for $\mu = 2$, 3 and 4

Fig. 4.46 HE / TDE Spectra for 33w Records
Fig. 4.47 Ratio of Cumulative Damage Indices, $D(c) / D(1.0)$, for $c = 1.5$ and 2.0 (33w Records)
Fig. 4.48 Number of Inelastic Excursions for Bilinear Systems with 10% Strain Hardening
SUM OF PLASTIC DEFORMATION RANGES, $\Sigma(\Delta \delta_p / \delta_y) = NHE$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

$\Sigma(\Delta \delta_p / \delta_y) = NHE$

$T$ (sec)

$\mu = 2, 3, 4, 5, 6, 8$ (thin $\rightarrow$ thick lines)

Fig. 4.49 Sum of Plastic Deformation Ranges for Bilinear Systems with 10% Strain Hardening (15s Records)

MEAN OF PLASTIC DEFORMATION RANGES - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

$\Sigma(\Delta \delta_p / \delta_y) / N$

$T$ (sec)

$\mu = 2, 3, 4, 5, 6, 8$ (thin $\rightarrow$ thick lines)

Fig. 4.50 Mean of Plastic Deformation Ranges for Bilinear Systems with 10% Strain Hardening (15s Records)
(a) 15s Records

(b) 10w Records

Fig. 4.51 Maximum Plastic Deformation Ranges for Bilinear Systems with 10% Strain Hardening
MEAN PLASTIC DEFORMATION, \( \frac{\delta_p}{\delta_y} \)\text{mean} - (15s.bi-00)

15s Records, Bilinear, \( \alpha = 0\% \), Mean

\[ \mu = 2, 3, 4, 5, 6, 8 \] (thin \( \rightarrow \) thick lines)

T (sec)

(a) For Bilinear Systems with No Strain Hardening

MEAN PLASTIC DEFORMATION, \( \frac{\delta_p}{\delta_y} \)\text{mean} - (15s.bi-10)

15s Records, Bilinear, \( \alpha = 10\% \), Mean

\[ \mu = 2, 3, 4, 5, 6, 8 \] (thin \( \rightarrow \) thick lines)

T (sec)

(b) For Bilinear Systems with 10% Strain Hardening

Fig. 4.52 Mean Plastic Deformation for Bilinear Systems (15s Records)
RATIO OF CUMULATIVE DAMAGE INDICES, $D(1.5) / D(1.0)$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

RATIO OF CUMULATIVE DAMAGE INDICES, $D(1.5) / D(1.0)$ - (10w.bi-10)

10w Records, Bilinear, $\alpha = 10\%$, Mean

Fig. 4.53  Ratio of Cumulative Damage Indices, $D(1.5) / D(1.0)$, for Bilinear Systems with 10% Strain Hardening
RATIO OF CUMULATIVE DAMAGE INDICES, $D(2.0) / D(1.0)$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

$$D(2.0) / D(1.0) = \frac{\Sigma (\Delta \delta_p / \delta_y)^2}{\Sigma (\Delta \delta_p / \delta_y)}$$

$T$ (sec)

(a) Mean Spectra

RATIO OF CUMULATIVE DAMAGE INDICES, $D(2.0) / D(1.0)$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean+$\sigma$

$$D(2.0) / D(1.0) = \frac{\Sigma (\Delta \delta_p / \delta_y)^2}{\Sigma (\Delta \delta_p / \delta_y)}$$

$T$ (sec)

(b) Mean+$\sigma$ Spectra

Fig. 4.54 Ratio of Cumulative Damage Indices, $D(2.0) / D(1.0)$, for Bilinear Systems with 10% Strain Hardening (15s Records)
NORMALIZED HYSTERETIC ENERGY, $NHE = \Sigma (\Delta \delta_p / \delta_y)$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

$NHE = \frac{HE}{(F_y \delta_y)} = \frac{\Sigma (\Delta \delta_p / \delta_y)}{\Sigma \delta_y}$

$T$ (sec)

(a) Mean Spectra

NORMALIZED HYSTERETIC ENERGY, $NHE = \Sigma (\Delta \delta_p / \delta_y)$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean+$\sigma$

$NHE = \frac{HE}{(F_y \delta_y)} = \frac{\Sigma (\Delta \delta_p / \delta_y)}{\Sigma \delta_y}$

$T$ (sec)

(b) Mean+$\sigma$ Spectra

Fig. 4.55 Normalized Hysteretic Energy for Bilinear Systems with 10% Strain Hardening (15s Records)
EFFECT OF STRAIN HARDENING ON NHE - (15s.bi-00/bi-10)

15s Records, Bilinear, (α = 0%) / (α = 10%), Mean

μ = 2, 3, 4, 5, 6, 8 (thin → thick lines)

(a) Ratio of NHE for Bilinear Systems with 0% to 10% Strain Hardening

EFFECT OF HYSTERESIS MODEL ON NHE - (15s.dg-00/bi-00)

15s Records, Degrading / Bilinear, α = 0%, Mean

μ = 2, 3, 4, 5, 6, 8 (thin → thick lines)

(a) Ratio of NHE for Stiffness Degrading to Bilinear Systems with No Strain Hardening

Fig. 4.56 Effect of Strain Hardening and Hysteresis Model on NHE for 15s Records
HYSTERETIC / TOTAL DISSIPATED ENERGY, HE / TDE - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

$\mu = 2, 3, 4, 5, 6, 8$ (thin $\rightarrow$ thick lines)

(a) Mean Spectra

HYSTERETIC / TOTAL DISSIPATED ENERGY, HE / TDE - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean+$\sigma$

$\mu = 2, 3, 4, 5, 6, 8$ (thin $\rightarrow$ thick lines)

(b) Mean+$\sigma$ Spectra

Fig. 4.57 HE / TDE Spectra for Bilinear Systems with 10% Strain Hardening for 15s Records
EFFECT OF STRAIN HARDENING ON HE / TDE - (15s.bi-00/bi-10)

15s Records, Bilinear, ($\alpha = 0\%$) / ($\alpha = 10\%$), Mean

(a) Ratio of HE / TDE for Bilinear Systems with 0\% to 10\% Strain Hardening

EFFECT OF HYSTERESIS MODEL ON HE / TDE - (15s.dg-00/bi-00)

15s Records, Degrading / Bilinear, $\alpha = 0\%$, Mean

(b) Ratio of HE / TDE for Stiffness Degrading to Bilinear Systems with No Strain Hardening

Fig. 4.58 Effect of Strain Hardening and Hysteresis Model on HE / TDE for 15s Records
MAXIMUM INPUT ENERGY / INPUT ENERGY, $\text{IE}_{\text{max}} / \text{IE}$ - (15s.bi-10)

15s Records, Bilinear, $\alpha = 10\%$, Mean

$\mu = 1, 2, 3, 4, 5, 6, 8$ (thin → thick lines)

Fig. 4.59 $\text{IE}_{\text{max}} / \text{IE}$ Spectra for Bilinear Systems with 10% Strain Hardening (15s Records)

EFFECT OF HYSTERESIS MODEL ON $\text{IE}_{\text{max}}$ - (15s.dg-00/bi-00)

15s Records, Degrading / Bilinear, $\alpha = 0\%$, Mean

$\mu = 2, 3, 4, 5, 6, 8$ (thin → thick lines)

Fig. 4.60 Ratio of $\text{IE}_{\text{max}}$ for Stiffness Degrading to Bilinear Systems with No Strain Hardening (15s Records)
HYSTERETIC ENERGY SPECTRA, HE - (15s.bi-10)

15s Records, Bilinear, \( \alpha = 10\% \), Mean

\( \mu = 2, 3, 4, 5, 6, 8 \) (thin \( \rightarrow \) thick lines)

HE per Unit Mass (cm/sec\(^2\))

T (sec)

(a) 15s Records

HYSTERETIC ENERGY SPECTRA, HE - (10w.bi-10)

10w Records, Bilinear, \( \alpha = 10\% \), Mean

\( \mu = 2, 3, 4, 5, 6, 8 \) (thin \( \rightarrow \) thick lines)

HE per Unit Mass (cm/sec\(^2\))

T (sec)

(b) 10w Records

Fig. 4.61 Hysteretic Energy Spectra for Bilinear Systems with 10\% Strain Hardening
Fig. 4.62 Input Energy Spectra for Bilinear Systems with 10% Strain Hardening
EFFECT OF STRONG MOTION DURATION ON N - (33w.bi-4.10)

33 Records, Bilinear, $\mu = 4$, $\alpha = 10\%$

(a) Number of Inelastic Excursions, N

EFFECT OF STRONG MOTION DURATION ON $D(1.0) = NHE$ - (33w.bi-4.10)

33 Records, Bilinear, $\mu = 4$, $\alpha = 10\%$

(b) Cumulative Damage Index, $D(1.0) = \Sigma (\delta_p / \delta_y) = NHE$

Fig. 4.63 Effect of Strong Motion Duration on Cumulative Damage Parameters for $\mu = 4$ (33w Records)
EFFECT OF STRONG MOTION DURATION ON $D(1.0) = \text{NHE}$ - (33w.bi-4.10)

33 Records, Bilinear, $T = 0.2$ sec, $\alpha = 10\%$

$D(1.0) = \Sigma (\Delta \delta_p / \delta_y) = \text{NHE}$

$D_{sm}$ (sec)

(a) For $T = 0.2$ sec

EFFECT OF STRONG MOTION DURATION ON $D(1.0) = \text{NHE}$ - (33w.bi-4.10)

33 Records, Bilinear, $T = 2.0$ sec, $\alpha = 10\%$

$D(1.0) = \Sigma (\Delta \delta_p / \delta_y) = \text{NHE}$

$D_{sm}$ (sec)

(b) For $T = 2.0$ sec

Fig. 4.64 Effect of Strong Motion Duration on the Cumulative Damage Index for Discrete Periods (33w Records)
Fig. 4.65 Strength Capacities vs. Regressed Strength Demands for 33w Records at e = 10 km
(a) Generic 1968 and 1988 R/C Moment Resisting Frame Structures

(b) Generic 1988 Steel Structures

Fig. 4.66 Strength Capacities vs. Regressed Strength Demands for 33w Records at e = 20 km
ATTENUATION OF DUCTILITY DEMANDS FOR CMF68, CMF88

2-Story Concrete Structures, 33w Records

Ductility Ratio, $\mu$

Epicentral Distance, $e$ (km)

(a) Generic 1988 and 1968 R/C Moment Resisting Frame Structures

ATTENUATION OF DUCTILITY DEMANDS FOR SBF88, SMF88, SPF88

2-Story Steel Structures, 33w Records

Ductility Ratio, $\mu$

Epicentral distance, $e$ (km)

(b) Generic Steel Structures

Fig. 4.67 Attenuation of Ductility Demands for Generic 2-Story Structures (33w Records)
ATTENUATION OF DUCTILITY DEMANDS FOR CMF68, CMF88

5-Story Concrete Structures, 33w Records

Epicentral Distance, e (km)

Ductility Ratio, $\mu$

CMF68 - local ($T = 0.50$ sec)
CMF88 - local ($T = 0.65$ sec)
CMF68 - global
CMF88 - global

(a) Generic 1988 and 1968 R/C Moment Resisting Frame Structures

ATTENUATION OF DUCTILITY DEMANDS FOR SBF88, SMF88, SPF88

5-Story Steel Structures, 33w Records

Epicentral distance, e (km)

Ductility Ratio, $\mu$

SBF88 - local ($T = 0.43$ sec)
SMF88 - local ($T = 0.75$ sec)
SPF88 - local ($T = 0.75$ sec)
SBF88 - global
SMF88 - global
SPF88 - global

(b) Generic Steel Structures

Fig. 4.68 Attenuation of Ductility Demands for Generic 5-Story Structures (33w Records)
ATTENUATION OF DUCTILITY DEMANDS FOR CMF68, CMF88

10-Story Concrete Structures, 33w Records

Epicentral Distance, $e$ (km)

(a) Generic 1988 and 1968 R/C Moment Resisting Frame Structures

ATTENUATION OF DUCTILITY DEMANDS FOR SBF88, SMF88, SPF88

10-Story Steel Structures, 33w Records

Epicentral distance, $e$ (km)

(b) Generic Steel Structures

Fig. 4.69 Attenuation of Ductility Demands for Generic 10-Story Structures (33w Records)
Fig. 4.70  Strength Capacities for 1968 R/C Moment Resisting Frame Structures vs. Regressed Elastic Strength Demands at $e = 14$, 25 and 28 km (33w Records)
CHAPTER 5
SEISMIC DEMANDS FOR MDOF SYSTEMS

The previous chapter provided information on seismic demands for inelastic SDOF systems. This information is relevant for relative assessment of damage potential of ground motions, but needs to be modified to become of direct use for design of real structures, which are mostly multi-degree-of-freedom (MDOF) systems governed by several translational and torsional modes. For elastic MDOF systems, the combination of modal responses using SRSS, CQC, or other approaches, provides reasonable estimates of peak dynamic response characteristics. For inelastic MDOF systems, modal superposition cannot be applied with any degree of confidence and different techniques have to be employed in order to predict strength or ductility demands that can be used for design.

Story ductility ratio, defined as the maximum dynamic interstory displacement normalized by the interstory yield displacement, is used here as the basic deformation parameter and, therefore, the strength demand associated with the a specific story ductility ratio becomes the basic design parameter. The objective of the proposed design approach, discussed in Section 2.1, is to design structures with sufficient strength to limit the story ductility ratios to a prescribed value (see Figs. 2.2 and 2.3). This strength demand differs from that of the corresponding SDOF system because of higher mode and torsional effects, and depends on many other structural characteristics such as the member stiffness and strength distribution over the height of the structure, structural system redundancy and modes of failure of critical structural elements.

5.1 MDOF Models Used in this Study

The research discussed in this chapter is intended to provide some of the answers needed to assess strength demands for inelastic MDOF systems for comparison with their SDOF counterparts. The focus is on a statistical evaluation of systems that are regular from the perspective of elastic dynamic behavior. Realizing that no two structures are the same, and that the dynamic behavior of real structures depend on so many parameters, it was decided to focus on simplified MDOF models in order to gain insight into basic inelastic dynamic behavior patterns. Therefore, "regular" 2-dimensional single-bay frames with
widely spaced elastic modal periods were utilized to represent the behavior of three distinct structural models. The torsional effects of 3-dimensional structures are neglected and the results do not apply to structures with closely spaced modal periods.

The three models used are illustrated in Fig. 5.1. The "beam hinge" (strong column - weak beam) model, referred to as BH model from here on, represents structures (e.g., special moment resisting frames) in which plastic hinges will form in beams only (as well as supports). The "column hinge" (weak column - strong beam) model, referred to as CH model from here on, represents structures (e.g., braced frames and moment resisting frames designed to accommodate plastic hinges in columns) in which plastic hinges will form in columns only. The "weak story" model, referred to as WS model from here on, represents structures in which plastic hinges will form in columns of the first story only. This model represents the behavior of frames with a strength (not stiffness) discontinuity in the first story. Note that structures with infill walls in all but the first story will have both strength and stiffness discontinuities, which is not represented by the WS model.

The three models are designed to develop the structure mechanisms shown in Fig. 5.1 under the 1988 UBC equivalent static lateral load pattern, i.e., relative member strengths are tuned so that all shown plastic hinges develop simultaneously under this specific lateral load pattern. Note that the BH model cannot develop a story mechanism, the CH can develop one in any story and the WS can develop one only in the first story. The only difference between the CH and WS models is the strength discontinuity above the first story for the WS model, in which all stories remain elastic throughout the entire dynamic time history analysis discussed later.

The elastic member stiffnesses in each story are tuned so that, under the 1988 UBC equivalent static load pattern, the interstory drift in every story is identical, resulting in a straight line deflected shape. Furthermore, the stiffnesses are tuned such that the first mode period of each structure is equal to that given by the UBC as $T = 0.02h_n^{3/4}$ sec, where $h_n =$ total frame height in feet. As a consequence, the first mode shape for the three models is close to a straight line. This tuning is done for the three models in order to permit a direct comparison of dynamic analysis results. Only structures with 2, 5, 10, 20, 30 and 40 stories (story height $h = 12'$), ranging in period from $T = 0.217$ to $2.051$ sec, were studied. Pertinent data for the first five elastic modes of the CH (and WS) models are presented in Table 5.1. The modal periods for the BH models are the same for the first mode but deviate slightly for higher modes due to difficulties in precise stiffness tuning, which also led to slight deviation in modal masses.
A bilinear moment-rotation (M-θ) hysteresis model with strain hardening ratios \( \alpha = 0, 2 \) and 10% is assumed at each plastic hinge location. Therefore, under an incremental ("push-over") 1988 UBC load pattern, each structure will have a bilinear response identical to that of a bilinear SDOF system that was employed as one of the SDOF models in Chapter 4.

The base shear strength, \( V_y \), is varied for each structure and ground motion record so that it is identical to the inelastic strength demand \( F_y(\mu_t) \) of the corresponding SDOF system with the first mode period and the same strain hardening ratio, for SDOF target ductility ratios of \( \mu_t = 1, 2, 3, 4, 5, 6 \) and 8. Tuning the base shear capacity of the MDOF system to the inelastic strength demand of the corresponding SDOF system, subjected to the same ground motion record, permit a direct comparison of results for SDOF and MDOF systems. Because modal superposition is hardly feasible for inelastic MDOF systems, it is desirable to utilize SDOF demand estimates to predict MDOF demands. The inelastic strength demands can be evaluated for SDOF systems as discussed in Chapter 4. The question to be answered is how do the ductility demands of MDOF systems compare to the SDOF target ductility ratio \( \mu_t \), if the MDOF base shear capacities are identical to the inelastic strength demands \( (F_y(\mu_t)) \) of their equivalent SDOF systems. Or more relevant for design, the question becomes: how different should the base shear strength of the MDOF system be (assuming the code prescribed seismic load pattern over the height of the structure), compared to that of the corresponding SDOF system, in order to limit the maximum story ductility ratio in the MDOF system to the specified target ductility ratio \( \mu_t \) (see Fig. 5.2).

The following assumptions were made in the design and analysis of the MDOF models:

- The mass of each story is the same.
- The height of each story is the same.
- Concentrated plasticity at the ends of members, i.e., no yielding occurs within members.
- Even though the 1988 UBC uses different equations for estimating the natural periods of different structures, the same equation, \( T = 0.02h_n^{3/4} \) sec is used for the three models for comparison.
The \textit{P-delta} effect of gravity loads is not included explicitly and is assumed to be accounted for through a reduction in the strain hardening ratio $\alpha$.

The effect of gravity load moments on plastic hinge formation is not included. This simplifies the nonlinear structural response from a multi-linear to a bilinear one.

Axial load - bending moment ($P-M$) interaction is not considered, i.e., the bending strength at each plastic hinge location is assumed to be constant, irrespective of the axial load carried by a column.

The overturning moment at a given story is assumed to be the integral area of the story shears from the top of the structure to \textit{the base of the columns} of that story \textit{at a given instance}. No allowance was made for moments at the base of columns. Therefore, the calculated overturning moments are the externally applied moments irrespective of the type of model used.

Critical damping of 5\% was assumed for the first two modes in the time history analysis.

All \textit{MDOF} models, with their base shear strength capacities tuned as discussed previously, were subjected to the 15s and 10w records using a modified version of the \textit{DRAIN-2D} dynamic analysis program (Kanaan and Powell, 1973). A total of 9,600 nonlinear dynamic time history analyses were performed for the following permutations:

- A total of 25 ground motion records (the 15s and 10w records).
- Structures with 2, 5, 10, 20, 30 and 40 stories.
- For \textit{SDOF} target ductility ratios $\mu_t = 1$ (elastic \textit{SDOF}), 2, 3, 4, 5, 6 and 8.
- For the three structural models ($BH$, $CH$ and $WS$).
- For three strain hardening ratios $\alpha = 0$, 2 and 10\%.

In addition, elastic time history analyses ($\mu = e$) were performed as needed. Note that an \textit{MDOF} system designed for a base shear equal to that imposed on an equivalent \textit{elastic SDOF} will not necessarily remain elastic, therefore the distinction between $\mu_t = 1$ (elastic \textit{SDOF}) and $\mu = e$ (elastic \textit{MDOF}). From each dynamic time history analysis the following information was stored for evaluation:
• Story ductility ratios, $\mu_i$

• Maximum shear force in each story, $V_i$

• Maximum overturning moment in each story, $M_{OT,i}$

The maximum overturning moment in a story is not necessarily the same as the overturning moments resulting from maximum story shears since the story shear maxima may not occur at the same instance. Figures, in which the distribution of the aforementioned parameters are shown over the height of the structure, represent envelopes of story maxima for these parameters, irrespective of the time at which they occur.

Results presented in the following sections are mean values, using primarily the 15s data set and occasionally the 10w data set for comparison. In the text and the figures, the computed MDOF ductility ratios are referred to as $\mu$, whereas the target ductility ratio that defines the strength demand is referred to as $\mu_t$.

### 5.2 Ductility and Base Shear Demands

If an MDOF system were to behave exactly like its equivalent SDOF system, all resulting interstory ductility ratios would be equal to the SDOF target ductility ratio $\mu_t$. However, due to higher mode participation in MDOF systems, this is usually not the case. Subsection 5.2.1 addresses the question of how do the MDOF (interstory) ductility demands compare to the target ductility ratio of the equivalent SDOF system. Subsection 5.2.2 addresses the question of how much do we need to modify the base shear capacity of the MDOF system, compared to that of the equivalent SDOF system, in order to limit the maximum interstory ductility ratio to the same prescribed value (see Fig. 5.2).

#### 5.2.1 Story Ductility Demands

Fig. 5.3 (a) shows an example of the mean $\pm \sigma$ story ductility demands for 20-story BH structures, for an SDOF target ductility ratio of $\mu_t = 8$ and strain hardening $\alpha = 0\%$ for the 15s data set (thus the notation 15s.bh20-8.00). Part (b) of the figure shows the corresponding information for the CH model.

Fig. 5.4 shows the mean story ductility ratios for the three models (BH, CH and WS, respectively) with 0% strain hardening, base shear capacities equal to the inelastic
strength demand for SDOF systems with a target ductility ratio $\mu_t = 2$, and using the 15s records. Fig. 5.5 is similar to Fig. 5.4 except that the target ductility ratio $\mu_t$ is equal to 8. Fig. 5.6 is similar to Fig. 5.5 except for $\alpha = 10\%$. Fig. 5.7 is similar to Fig. 5.6 except for using the 10w rather than the 15s data set.

The results in Figs. 5.4 to 5.7 show consistent trends that can be summarized as follows:

- The pattern of the story ductility demands over the height of the structure is similar for the BH and CH structures. The distribution is rather uniform over height for 2 to 10 story structures; for taller structures the ductility demands are high in the lower and upper stories (except for very large ductility ratios ($\mu_t = 8$)), and decrease significantly in the middle stories. This pattern is due to the participation of higher modes whose elastic periods (see Table 5.1) coincide with the peaks of the elastic and inelastic strength demand spectra (Figs. 4.12 and 4.13).

- The ductility demands for the CH structures are consistently higher than the demands for the BH structures, particularly in the lower stories, even though both structures are designed for the same base shear strength. This is because a story mechanism can readily develop in a CH model, resulting in more drift.

- The deviation of story ductility ratios from those predicted by the equivalent SDOF systems, increases with the number of stories, the target ductility ratio and decreasing strain hardening. The largest deviation is associated with the WS model and the least with the BH model.

- The strain hardening ratio $\alpha$ has a significant effect on the ductility demands, as can be seen by comparing the results for $\alpha = 0$ and 10%. The elastic-perfectly plastic structures ($\alpha = 0\%$) create much larger ductility demands, particularly in the lower stories of tall structures, even though the base shear strength, $V_y = F_y(\mu_t)$, for $\alpha = 0\%$ is greater than that for $\alpha = 10\%$.

- The Whittier Narrows 10w data set tends to give higher story ductility demands and greater deviations from the (SDOF) target ductility ratios compared to the 15s data set, especially for taller structures. This can be attributed to the nature of the Whittier Narrows ground motions depicted in their elastic and inelastic strength demand spectra (see Fig. 4.13) which
shows that most of the seismic energy is concentrated in the short period range exciting higher modes of tall structures.

- The relative story strengths, which are in conformance with the 1988 UBC seismic load pattern, do not cause an equal ductility demand in each story. (It probably was not intended to do so).

- In most cases, the maximum story ductility ratio occurs in the first story, indicating that the 1988 UBC seismic load pattern generally provides good protection against excessive ductility demands in upper stories. There are notable exceptions, particularly for 5 to 20 story structures and lower ductility ratios, where the story ductility demands in the upper stories are sometimes higher than in the first story. This indicates the need for some (but not major) revisions of the code seismic load pattern. In the evaluation of the results discussed later it is assumed that the first story ductility ratio governs for design and that the required base shear strength is determined from this first story ductility ratio, \( \mu_1 \).

- The ductility demands for the WS model demonstrate the great problems inherent in these structures. Only the first story ductility ratios are actual ductility demands\(^1\), and they are much larger than those of the corresponding CH structures. It should be emphasized that in the first story, the WS and CH structures are identical, i.e., they have the same strength and story mechanism. Differences start above the first story, insofar that all other stories in the WS structures are "strong" and remain elastic whereas the strength of their counterparts in the CH structures are tuned to the first story strength and the code seismic load pattern. The high ductility demand imposed on the WS structures is attributed to the

\(^1\) "Ductility" values larger than 1.0 are shown in the upper stories of the WS structures to illustrate the increase in elastic forces, attracted in these stories, beyond the force level based on the code seismic load pattern. For instance, as shown in Fig. 5.5 (c), the elastic shear forces in the upper stories of the 40-story WS structure (with target ductility \( \mu_4 = 8 \) and \( \alpha = 0\% \)), is about 6 times that required by the code load pattern, even though the shear transfer to the upper stories is limited by the strength capacity of the first story. This increase is mainly caused by increased elastic vibrations above the first story.
concentration of energy dissipation in the first story. The demand is even higher for higher ductility ratios and no strain hardening.

- Even though the shear transfer in WS structures is limited by the shear capacity of the first story, the elastic vibration of the upper stories can attract large shear forces (not limited by any yield level) that transmit high axial forces to the first story. It is worth noting that the stories above the first do not vibrate as a rigid body, and that their interstory drift demand increases with strain hardening in the first story.

- Strain hardening leads to a more uniform variation in ductility demands over the height of the BH and CH models. Comparing Figs. 5.5 and 5.6 shows that the double curvature noted in the story ductility demands of tall structures with no strain hardening, almost disappears.

- For the same SDOF target ductility ratio \( \mu_t \), the Whittier Narrows 10w records give higher ductility demands than the 15s records. This is again attributed to the peculiarity of the Whittier Narrows Earthquake with most of its energy concentrated in the short period range, which leads to an increase in higher mode effects.

Fig. 5.8 shows the story ductility demands in the first story, \( \mu_1 \), of the MDOF systems for SDOF target ductility ratios \( \mu_t = 2 \) and \( 8 \) and strain hardening ratios \( \alpha = 0 \) and \( 10\% \), plotted against the first mode period of the MDOF systems. The target ductility ratios, indicated by the horizontal dashed lines, are the ductility ratios of the SDOF systems whose strength, \( F_Y(\mu_t) \), was used as base shear strength \( V_Y \) in the design of the MDOF systems. The following can be noted from Fig. 5.8:

- The deviations from the target ductility ratios exhibit consistent trends, being small for low-rise BH and CH structures and increase with the number of stories. However, the WS structures give very large ductility demands. Also, the difference in ductility demands for CH and BH models appears to be period independent (almost parallel lines in the figure). The maximum ductility demand for the 40 story structures may be as high as 10 times the target ductility ratio for the WS model, 3 times for the CH model and 2 times for the BH model.

- The significant effect of strain hardening is evident by comparing the results for strain hardening \( \alpha = 0 \) and \( 10\% \). Systems with strain hardening
(α = 10%) give consistently smaller ductility demands than those without (α = 0%). This is attributed to the fact that systems without strain hardening drift more and especially so for systems that can develop separate story mechanisms, i.e., WS and CH models.

5.2.2 Base Shear Demands

The results illustrated so far, clearly show that the ductility demands for MDOF systems differ significantly from those of the corresponding SDOF systems. The implication is that the base shear strength capacities of the MDOF systems must be modified compared to the inelastic strength demands (Fy(μ)) of SDOF systems, in order to limit the story ductility demand to the desired target values (see Fig. 5.2). The following discussion provides an approach to achieve this objective and illustrates several results. It needs to be emphasized that the results presented are for the 15 Sj (15s) ground motions, and, therefore, provide representative information for design of MDOF systems located on firm soils or rock. Also, results have to be put in context of the aforementioned assumptions listed Section 5.1.

For SDOF systems, the parameter that relates elastic to inelastic strength demands is the strength reduction factor Ry(μ) = Fy,el/Fy(μ). It is a convenient design parameter as it permits, in concept, the derivation of inelastic strength demand spectra from the acceleration response (elastic strength demand) spectra. The advantage of using strength reduction factors, which are strength demand ratios, is that they are dimensionless parameters that do not depend on the severity (scale) of ground motions and can be utilized to evaluate relative strength demands.

Working in the dimensionless R-μ domain, the modifications needed for the MDOF base shear capacities, in order to limit the story ductility demands at the base (assumed to be the maximum) to the prescribed target ductility ratio μt, can be readily evaluated. For a given number of stories (period) and strain hardening ratio, the R-μ relationship for any of the three types of MDOF systems and their equivalent SDOF system can be plotted as shown in Fig. 5.9. Point 1 refers to the strength capacity associated with an SDOF target ductility ratio μt. An MDOF system designed for that strength capacity will give a higher ductility demand μ' (shown as Point 2). In order to limit the MDOF ductility demand to μt, linear interpolation between data points similar to Point 2 is used to obtain Point 3. The ratio of the reduction factors at Points 1 and 3 is the reverse of the ratio of the strength
capacities of the SDOF and MDOF systems that will give the same target ductility ratio \( \mu_t \) (that is \( V_y(\mu_t) / F_y(\mu_t) \) in Fig. 5.2).

Fig. 5.10 illustrates typical \( R-\mu \) relationships for 2 and 40 story MDOF systems with 0 and 10% strain hardening, together with the corresponding SDOF \( R-\mu \) relationships (dashed curves). Data points on which the curves are based are obtained by using, as ordinates, the mean strength reduction factors for \( \mu_t = 1, 2, 3, 4, 5, 6, \) and 8 of the SDOF system with the first mode period of the MDOF systems (these values define the strength of both the SDOF and MDOF systems used in the analysis), and, as abscissa, the mean ductility ratios of the SDOF and MDOF systems. The data points shown on solid curves for the MDOF systems correspond to Point 2 in Fig. 5.9. Connecting the data points for each system with straight lines gives approximate \( R-\mu \) relationships for the SDOF as well as MDOF systems. As mentioned before, the ductility demand parameter used here for the MDOF systems is the story ductility demand in the first story of the structure, which in most cases is the maximum overall story ductility ratio.

The following observations are made from the \( R-\mu \) relationships in Fig. 5.10:

- The \( R-\mu \) relationships for the three MDOF models are relatively smooth curves with no apparent irregularities, justifying the use of linear interpolation between the \( R-\mu \) data points.

- The BH model has consistently the closest \( R-\mu \) relationships to that of the SDOF system, especially for strain hardening systems (\( \alpha = 10\% \)). Short period BH structures (2 to 10 stories high - see also Figs. 5.4 to 5.6) have ductility demands that are even smaller than those predicted by SDOF systems.

- The WS model shows the greatest deviation from the SDOF \( R-\mu \) relationships, particularly for systems with no strain hardening (\( \alpha = 0\% \)).

- The deviation of the MDOF \( R-\mu \) relationships from that of the SDOF system increases with the number of stories (period). The effect of strain hardening is greatest for the WS model and smallest for the BH model.

It remains to be answered how the information generated here can be incorporated in the design process. As outlined in Fig. 2.3, the inelastic strength demand spectra derived for SDOF systems need to be modified to account for the multi-mode effects of
MDOF systems in place of the SRSS or CQC modal combination procedures. For structures located on $S_I$ soil sites, and which fulfill the constraints summarized at the beginning of this chapter, the data generated in this study can be used to evaluate these modifications as illustrated in Fig. 5.9.

Fig. 5.11 (parts (a) through (h)) shows the modification factors, the ratios of the MDOF base shear strength demand $V_y(\mu_t)$ to SDOF inelastic strength demand $F_y(\mu_t)$, which are the reverse of the ratios of the corresponding reduction factors, that are required to limit the ductility demand for both systems to the target values $\mu_t = 2, 3, 4$ and 8, and for 0 and 10% strain hardening. The period axis includes the six different number of stories.

The dashed curve shown in each graph represents the modification factors envisioned by ATC-3 to account for the multi-mode effect of MDOF systems (that is, raising of the $1/T$ tail of the ground motion spectra to $1/T^2β$ in the design spectra as shown in Fig. 2.1). For soil type $S_I$ and $A_y = A_s$ this ratio, $r$, is given as:

$$r = \frac{1.2 A_y / T^{2β}}{1.22 A_y / T} \leq 2.5 A_s$$

(5.1)

According to the Commentary of the ATC-3-06 document (ATC-3-06, 1978), this ratio is intended to provide more safety for long period structures and account for higher mode effects. For periods exceeding 0.488 sec. this ratio amounts to $0.984T^{1/3}$. Note that this modification is independent of the target ductility ratio since ATC-3 assumes a constant reduction factor, $R$ (or $R_w$ in the 1988 UBC), irrespective of the expected ductility demand for the given structure.

Fig. 5.12 shows the modification factors, but plotted vs. the target ductility ratio $\mu_t$ instead of period, for 2 and 40 story structures with 0 and 10% strain hardening. The following observations can be made from Fig. 5.11 and 5.12:

- The required strength modifications for MDOF effects depend strongly on the target ductility ratio, the fundamental period of the system, the type of structural system, and the strain hardening ratio $\alpha$. For BH structures, i.e., structures that involve beam mechanisms in every story, the modification factor is smallest and is mostly in good agreement with the ATC-3 modification only if there is significant strain hardening ($\alpha = 10\%$). Even here the caveat exists that, for longer period structures and higher target ductility ratios, somewhat larger modifications are required than indicated.
by the ATC-3 approach. For short period BH structures, the base shear strength demand is consistently lower than the corresponding SDOF strength demand, indicating that MDOF effects are not important for this range.

- For CH structures, i.e., structures in which story mechanisms form within each story (e.g., plastic hinging in columns, or mechanisms in bracing systems), the MDOF strength demands are higher than for BH structures. Quantitative information can be obtained from the graphs. It can be seen that the required increase in strength compared to the BH structures is about the same regardless of fundamental period. The figures also clearly demonstrate that WS structures, i.e., structures with a weak first story, are indeed a great problem. Such structures require strength capacities that may be more than twice those required in BH structures in order to limit the story drift to the same target ductility ratio.

- A comparison between results for 0 and 10% strain hardening shows that matters get considerably worse if the structural system has no strain hardening, particularly for CH structures and longer periods. Systems without strain hardening drift more, and larger strength is required in order to limit the drift to a prescribed target ductility ratio. This effect is actually larger than indicated in the graphs, since the SDOF strength demands, which are used as normalizing factors, are already larger for elastic-plastic SDOF systems than for strain hardening SDOF systems (see Figs. 4.17 to 4.19). The differences between the results shown for 0 and 10% strain hardening in Fig. 5.11 and 5.12 come only from multi-mode effects.

- In general, the modification factor for MDOF vs. SDOF strength demands increases with target ductility ratios (except for 2-story structures which show an increase then decrease in modification factors for higher target ductility ratios) and decreases with strain hardening.

- For the regular structures studied here, elastic MDOF systems attract lower base shears than those predicted by the equivalent SDOF systems.

These results clearly demonstrate that an estimation of the required strength in the design process needs to be based on criteria that consider the fundamental period and distinguish between different "failure" mechanisms and anticipated strain hardening. These
factors may significantly affect the ductility demand the structures will experience during an earthquake. When drawing quantitative conclusions from the presented data it needs to be emphasized that the presented results are based on mean values obtained from 15 firm soil records and, in their final presentation as shown in Figs. 5.11 and 5.12, are derived from straight line interpolation between discrete ductility values and discrete periods. These straight line interpolations may lead to considerable approximations for systems for which the SDOF and MDOF ductility demands differ significantly (e.g., WS structures with high target ductility ratios).

The foregoing discussion focused on a procedure that can be employed to derive design strength demands for MDOF systems from inelastic strength demand spectra of SDOF systems. The presented numerical results apply only within the constraints identified in this chapter and cannot be generalized without a much more comprehensive parametric study. The parameters that need to be considered include the frequency content of the ground motions (which may be greatly affected by local site conditions), the hysteretic characteristics of the structural models (strain hardening, stiffness degradation, strength deterioration, etc.), and the dynamic characteristics of the MDOF models (periods, mode shapes and modal masses of all important modes, as well as stiffness and strength discontinuities). Furthermore, it must be pointed out that the $R$-$\mu$ relationships for MDOF systems developed here are based on ductility demands in the first story. In several cases the maximum story ductility demand did not occur in the first story, but close to the top of the structure. This issue is not considered here, but can be addressed through modifications to the distribution of design story forces over the height for structures without stiffness and strength discontinuities (regular structures), or a case-by-case dynamic analysis for irregular structures.

5.3 Shear and Overturning Moment Demands

The parametric study on MDOF systems provided much statistical information on shear and overturning moment distribution for elastic as well as inelastic systems. The maximum story shears and overturning moments for all stories were obtained from the dynamic time history analyses for the different permutations discussed in Section 5.1. Depending on the purpose, the maximum dynamic story shears (or overturning moments) can be normalized in two ways: either by (a) the design base shear (or overturning moment at the base), to allow a direct comparison between dynamic and design (code) load patterns;
or by (b) the \textit{design story} shears (or overturning moments), for a story-by-story comparison.

In all cases, the design story shears (or overturning moments) are those obtained from a base shear equal to the strength demand of the corresponding \textit{SDOF} system and the 1988 \textit{UBC} seismic load pattern. All results in this section are presented for specific \textit{SDOF} target ductility ratios $\mu_i$; the corresponding \textit{MDOF} story ductility ratios can be obtained from Figs. 5.4 to 5.6.

\textbf{5.3.1 Story Shear Demands}

Fig. 5.13 shows an example of the mean $\pm \sigma$ variation of dynamic story shear force distribution, normalized with respect to the design story shears, for elastic 20-story \textit{BH} structures subjected to the 15s ground motions. Fig. 5.14 shows the mean distribution of the elastic dynamic story shears, normalized with respect to the design story shears, for 2 to 40 story structures. The figure shows that the dynamic story shears are somewhat higher than assumed by code distribution in the upper stories (for tall structures only) and lower in the lower stories. The values of less than 1.0 at the base indicate that the \textit{MDOF} systems attracted lower base shears than their equivalent \textit{SDOF} systems.

The picture is quite different for inelastic systems. Fig. 5.15 shows the same parameters as Fig. 5.14 but for inelastic systems with 0\% strain hardening and \textit{SDOF} target ductility ratio $\mu_t = 2$. Fig. 5.16 is for 0\% strain hardening and \textit{SDOF} target ductility ratio $\mu_t = 8$ and Fig. 5.17 is for 10\% strain hardening and \textit{SDOF} target ductility ratio $\mu_t = 8$. The following can be noted from Figs. 5.15 to 5.17:

- For \textit{CH} structures all stories yield, resulting in uniform ratios of 1.0 over the entire height of the structures for elastic-perfectly plastic systems ($\alpha = 0\%$), or ratios of $1.0 + \alpha (\mu_i - 1)$ at story $i$ for systems with strain hardening ratio $\alpha$ (see Figs. 5.4 to 5.6). At any given story, once the dynamic story shear reaches its yield capacity, the story yields and develops a story mechanism, thus preventing any further transfer of shear forces across this story unless there is some strain hardening. This is evident by comparing Fig. 5.16 with 5.17 parts (b) for \textit{CH} structures for 0 and 10\% strain hardening.
• For BH structures, the ratios are always greater than those of the CH structures and greater than 1.0 even for \( \alpha = 0\% \). This is due to the inability of the BH model to develop a story mechanism, thus attracting more story shear forces. For \( \alpha = 0\% \), the BH model can resist an additional 10-20% story shears due to this effect for \( \mu_t = 2 \) and up to 100% more for \( \mu_t = 8 \). Note that this would not occur if the BH model were incrementally loaded with the 1988 UBC load pattern, since a complete structure mechanism would develop.

• The large story shear forces attracted above the first weak story of the WS structures are illustrated in parts (c) of Figs. 5.15 to 5.17. In contrast to the CH model, making the stories above the first strong enough to remain elastic attracts much larger story shear forces, especially in the upper stories. These large shear forces may not pose a problem by themselves, since for these structures the cause of the weak story is excessive shear strengths in all upper stories, however, as will be discussed later, they generate very large overturning moments that have to be resisted in the first story. The first story will have to resist high axial forces on its columns, besides having to dissipate most of the seismic energy imparted to the structure. The \( P\text{-}delta \) effect may become of major concern for these structures. It is worth noting that the maximum story shears in the upper stories can reach up to 2 (or 6) times the shears obtained from the 1988 UBC load pattern for 0% strain hardening in the first story and \( \mu_t = 2 \) (or 8), and even more for 10% strain hardening.

Figs. 5.18 and 5.19 show story shear forces for 40-story structures with SDOF target ductility ratios of 2 and 8, normalized with respect to the design base shears. The thin lines refer to the UBC load pattern normalized to a value of 1.0 at the base. The following observations can be made from these figures:

• For inelastic BH and CH structures, the governing dynamic load pattern is closer to a rectangular (from the almost constant slope shear force diagram) rather than an inverted triangular shape. This is attributed to the full development of story shear capacities as illustrated in Fig. 5.15 to 5.17. On the other hand, the WS structures can be characterized by a 3-step load pattern with maximum intensity at the upper stories and minimum at the lower stories.
• The deviation of dynamic story shears from the design values increases with strain hardening, target ductility ratio, and towards the bottom of the structures. The deviation greatly depends on the structural system. It is largest for the WS structures and smallest for the CH structures.

5.3.2 Story Overturning Moment Demands

The issue of overturning moments is considered one of the most critical ones, as these moments control the flexural design of shear walls and the maximum axial forces that have to be resisted in columns of braced and moment resisting frames. Since columns under axial loads are usually brittle, conservative criteria have to be used in estimating overturning moments, and reductions in overturning moments have to be treated with great caution.

The shear force distributions discussed previously cannot be used directly to draw conclusions on overturning moments, because these forces represent maxima in each story that occur at different times. Therefore, in the dynamic time history analyses, the maximum overturning moment in each story was computed and stored for evaluation.

Fig. 5.20 shows the mean ± σ variation for the dynamic story overturning moments, normalized with respect to design overturning moments at each story, for elastic 20-story BH structures. The design overturning moments are the overturning moments obtained from the 1988 UBC load pattern with the base shear set equal to that of the equivalent SDOF system. It is interesting to note that the variation decreases towards the bottom of the structure. This is attributed to the cumulative nature of the overturning moments, which are the summation of the story shears, which are, in turn, the summation of story forces.

Fig. 5.21 shows the mean distribution of overturning moments, normalized with respect to design overturning moments at each story, for 2 to 40-story elastic MDOF systems. For elastic systems, the moments are close to the design values at the upper stories (usually not of much importance) and significantly smaller at the lower stories. This may justify the use of overturning moment reduction factors. However, it needs to be emphasized that there are two reasons for the relatively small values shown at the base of all structures. First, the base shears attracted by the elastic MDOF systems are smaller than the design base shears (see Fig. 5.14), which were based on the corresponding SDOF
systems. Second, the dynamic overturning moments are smaller than those based on the code seismic load pattern. The contribution of the latter can be estimated by comparing Figs. 5.14 and 5.21. For example, the mean base shear of 40-story structures is about 94% of the design base shear, and the mean overturning moment at the base is about 72% of the design overturning moment. Thus, the actual reduction in overturning moments is $0.72/0.94 = 0.77$.

For inelastic systems, a much discussed code design issue is whether it is necessary to design structures for overturning moments that are produced by the story shear capacities, or whether it is justifiable to use reduced overturning moments assuming that not all stories yield simultaneously.

Figs. 5.22 to 5.24 show the maximum dynamic story overturning moments for an SDOF target ductility ratio $\mu_t = 2$ and strain hardening $\alpha = 0\%$, for $\mu_t = 8$ and $\alpha = 0\%$, and for $\mu_t = 8$ and $\alpha = 10\%$, respectively. The information presented in these figures correspond to that on shear forces in Figs. 5.15 to 5.19. The following observations can be made from Figs. 5.22 to 5.24:

- The presented results provide much evidence that for structures whose story strengths are tuned to the design level (all BH and CH structures), simultaneous yielding in most stories is very likely and, therefore, no reductions in overturning moments should be applied. In the presented graphs, a value of 1.0 implies no overturning moment reduction, and, as the figures show, this value is reached or exceeded in most cases. If no strain hardening is present, the CH structures develop the maximum design overturning moments over the entire height of the structure (computed values close to 1.0), and the BH structures develop even higher demands in the upper stories, which increase with the target ductility $\mu_t$. For systems with strain hardening, the dynamic overturning moment at the base of the structure may be in the order of 50% higher than design values, for both the BH and CH structures.

- For inelastic weak story (WS) structures, the dynamic overturning moments become very large compared to the design overturning moments. For these types of structures, the ductility demands are very high (Figs. 5.4 to 5.6), the story shear forces are much larger than the shear strength of the first story (Figs. 5.15 to 5.17), and the overturning moments exceed those based
on first story strength and code seismic shear distribution by a large margin (Figs. 5.22 to 5.24). All these issues need to be considered in the design of weak story structures.

The results presented here give a clear indication that overturning moments in inelastic structures can be very large. If the story strengths are tuned to the code seismic load pattern and a base shears equal to that predicted by the equivalent SDOF systems for a prescribed target ductility ratio (as was done in all BH and CH structures), it should be assumed that all stories will yield simultaneously and the maximum overturning moments should be based on the shear strengths of all stories above, with due consideration given to strain hardening. No overturning moment reduction factors should be applied. In most real designs, the story strengths cannot be tuned exactly to the code seismic load pattern and individual stories may have a shear strength larger than required. In such cases the overturning moments will further increase. This study provides no information on the magnitude of this increase, as it depends on the relative strength of each story and cannot be generalized.

Extreme strength discontinuities, such as those in the WS structures, should be avoided, whenever possible, as they lead to excessive ductility and overturning moment demands that may be greatly amplified by the portions of the structure that remain elastic.
Table 5.1 Modal Periods and % Mass for MDOF Structures Used in this Study

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<th>Mode #</th>
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<th>10-STOREY</th>
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<th>30-STOREY</th>
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(a) Beam Hinge (BH) Model | (b) Column Hinge (CH) Model | (c) Soft Story (SS) Model

Fig. 5.1 Types of Structures Used in MDOF Study

Fig. 5.2 Modification in MDOF Strength Capacity to Achieve Equal Ductility Demands in SDOF and MDOF Systems
DYNAMIC STORY DUCTILITY DEMANDS, $\mu_i$ - (15s.bh20-8.00)

15s Records, BH Model, 20-Story, $\mu_i = 8$, $\alpha = 0\%$, Mean±$\sigma$

Story Ductility Ratio, $\mu_i = \delta_{dyn,i} / \delta_{y,i}$

(a) BH Model

DYNAMIC STORY DUCTILITY DEMANDS, $\mu_i$ - (15s.ch20-8.00)

15s Records, CH Model, 20-Story, $\mu_i = 8$, $\alpha = 0\%$, Mean±$\sigma$

Story Ductility Ratio, $\mu_i = \delta_{dyn,i} / \delta_{y,i}$

(b) CH Model

Fig. 5.3 Variation in Story Ductility Demands for 20-Story Structures for $\mu_i$ (SDOF) = 8, $\alpha = 0\%$ (15s Records)
Fig. 5.4 Dynamic Story Ductility Demands for $\mu_t(SDOF) = 2$, $\alpha = 0\%$ (Mean for 15s Records)
Fig. 5.5 Dynamic Story Ductility Demands for $\mu_\text{f(SDOF)} = 8$, $\alpha = 0\%$ (Mean for 15s Records)
Fig. 5.6 Dynamic Story Ductility Demands for $\mu_l(SDOF) = 8$, $\alpha = 10\%$ (Mean for 15s Records)
Fig. 5.7 Dynamic Story Ductility Demands for $\mu_{(SDOF)} = 8, \alpha = 10\%$ (Mean for 10w Records)
Fig. 5.8 Comparison Between First Story Ductility Demands for MDOF Structures and SDOF Target Ductility Ratios (Mean for 15s Records)
Fig. 5.9 R-\(\mu\) Relationships for SDOF and MDOF Systems (Explanation)
(a) 2 Stories (T = 0.217 sec), α = 0%

(b) 2 Stories (T = 0.217 sec), α = 10%

(c) 40 Stories (T = 2.051 sec), α = 0%

(d) 40 Stories (T = 2.051 sec), α = 10%

Fig. 5.10 R-μ Relationships for SDOF and MDOF Systems (Mean for 15s Records)
Fig. 5.11 Base Shear Modification for MDOF Effects vs. Period (15s Records)
BASE SHEAR MODIFICATION FOR MDOF EFFECTS, $V_y(MDOF) / V_y(SDOF)$ - (15s-4.00)

(c) $\mu_t = 4$, $\alpha = 0\%$

BASE SHEAR MODIFICATION FOR MDOF EFFECTS, $V_y(MDOF) / V_y(SDOF)$ - (15s-4.10)

(f) $\mu_t = 4$, $\alpha = 10\%$

BASE SHEAR MODIFICATION FOR MDOF EFFECTS, $V_y(MDOF) / V_y(SDOF)$ - (15s-8.00)

(g) $\mu_t = 8$, $\alpha = 0\%$

BASE SHEAR MODIFICATION FOR MDOF EFFECTS, $V_y(MDOF) / V_y(SDOF)$ - (15s-8.10)

(h) $\mu_t = 8$, $\alpha = 10\%$

Fig. 5.11(Cont'd) Base Shear Modification for MDOF Effects vs. Period (15s Records)
Fig. 5.12 Base Shear Modification for MDOF Effects vs. Target Ductility Ratio $\mu_t$ (15s Records)
Fig. 5.13 Variation in Maximum Dynamic Story Shears for Elastic 20-Story Structures (15s Records)

Fig. 5.14 Maximum Dynamic Story Shears for Elastic Structures (Mean for 15s Records)
Fig. 5.15 Max. Dyn. Story Shears (Normalized by $V_{\text{des},i}$) for $\mu_{\text{SDOF}} = 2, \alpha = 0\%$ (Mean for 15s Records)
Fig. 5.16 Max. Dyn. Story Shears (Normalized by $V_{\text{des,l}}$) for $\mu_\ell(\text{SDOF}) = 8$, $\alpha = 0\%$ (Mean for 15s Records)
Fig. 5.17 Max. Dyn. Story Shears (Normalized by $V_{\text{des},l}$) for $\mu(SDOP) = 8$, $\alpha = 10\%$ (Mean for 15s Records)
Fig. 5.18 Max. Dyn. & Design Story Shears (Normalized by $V_{\text{des, base}}$) for $\mu_\text{SDOF} = 2$, 40 Stories (Mean for 15s Records)
Fig. 5.19 Max. Dyn. & Design Story Shears (Normalized by $V_{des, base}$) for $\mu_1$ (SDOF) = 8, 40 Stories (Mean for 15s Records)
MAX. DYNAMIC STORY OVERTURNING MOMENTS, $M_{OT(dyn,l)}/M_{OT(des,l)} - (15s.20-e)$

15s Records, 20-Story, Elastic, Mean±σ

Fig. 5.20 Variation in Maximum Dynamic Story Overtuing Moments for Elastic 20-Story Structures (15s Records)

MAX. DYNAMIC STORY OVERTURNING MOMENTS, $M_{OT(dyn,l)}/M_{OT(des,l)} - (15s-e)$

15s Records, Elastic, Mean

Fig. 5.21 Maximum Dynamic Story Overtuing Moments for Elastic Structures (Mean for 15s Records)
Fig. 5.22 Max. Dyn. Story Overturning Mom. (Normalized by $M_{OT(\text{des.})}$) for $\mu_{(SDOF)} = 2$, $\alpha = 0\%$, (Mean for 15s Records)
MAX. DYNAMIC STORY OVERTURNING MOMENTS, $M_{OT(dyn,l)}/M_{OT(des,l)}$ *(15s.bh-8.00)*

(a) BH Model

MAX. DYNAMIC STORY OVERTURNING MOMENTS, $M_{OT(dyn,l)}/M_{OT(des,l)}$ *(15s.ch-8.00)*

(b) CH Model

MAX. DYNAMIC STORY OVERTURNING MOMENTS, $M_{OT(dyn,l)}/M_{OT(des,l)}$ *(15s.ws-8.00)*

(c) WS Model

Fig. 5.23 Max. Dyn. Story Overturning Mom. (Normalized by $M_{OT(des,l)}$) for $\mu_4$(SDOF) = 8, $\alpha = 0\%$, (Mean for 15s Records)
Fig. 5.24  Max. Dyn. Story Overturning Mom. (Normalized by $M_{OT\text{(des,}}$) for $\mu_1(\text{SDOF}) = 8, \alpha = 10\%$, (Mean for 15s Records)
CHAPTER 6
SUMMARY & CONCLUSIONS

This research is intended to provide basic information needed to implement a
ductility based seismic design approach. The concepts of this approach and its
implementation are outlined in Section 2.1. It is postulated that the ductility capacity of
critical structural elements is the basic design parameter, and the objective of design is to
provide the structure with sufficient strength capacity so that the ductility demands in these
elements are less than their allowable capacities. Target ductility capacities for structures
are established by modifying (weighing) member ductility capacities for anticipated
cumulative damage effects and transforming these member ductility capacities into story
ductility capacities which are used as measures of the structure ductility capacity. For the
so derived target ductility capacity the required structure strength (inelastic strength
demand) may be estimated from SDOF systems and appropriate modifications that account
for MDOF effects. Thus, implementation of this approach necessitates extensive
information on system dependent SDOF seismic demand parameters, including cumulative
damage parameters (in order to weigh ductility capacities), and system dependent MDOF
modifications. These topics are the focus of this study.

A comprehensive evaluation of seismic demand parameters is performed for bilinear
and stiffness degrading SDOF systems. In this study, the inelastic strength and cumulative
damage demands are evaluated statistically for specified target ductility ratios. Such a
statistical study can be attempted only for ground motions with similar frequency
characteristics, such as rock and firm soil motions recorded not too close and not too far
from the fault rupture. Strength demands are represented in terms of inelastic strength
demand spectra or spectra of strength reduction factors. Expressions are developed that
relate the strength reduction factor to period and target ductility ratio. Cumulative damage
demands are expressed in terms of energy quantities, number of inelastic excursions, and a
simple cumulative damage model. The attenuation characteristics of seismic demand
parameters are studied on hand of a single earthquake, the 1987 Whittier Narrows
earthquake, for which an extensive set of ground motions is available. The conclusions
drawn from this SDOF study are summarized as follows:
The strength reduction factors depend strongly on the target ductility ratio and period of the SDOF system and to a much lesser extent on the strain hardening and hysteresis model. The effect of damping is not studied here. The reduction factors are not sensitive to epicentral distance.

Smooth $R-\mu-T$ relationships are developed for typical $S_I$ ground motions through a regression analysis based on a database of 39,000 points. The $R-\mu-T$ relationships are highly nonlinear in the short period range and the relationship $R = \sqrt{2\mu - T}$ will give poor predictions for all but one specific short period. The relationship $R = \mu$ is a conservative approximation for long period systems.

The $R-\mu-T$ relationships can be used together with smooth ground motion spectra, such as those proposed in ATC-3-06 for soil types $S_I$ and $S_2$, to obtain inelastic strength demand spectra. Because of the high nonlinearity of the reduction factor in the short period range, the inelastic strength demand spectra, derived from utilizing the ATC-3-06 ground motion spectra and the developed $R-\mu-T$ relationships, do not show any signs of a plateau in the short period range.

The peaks in the elastic response spectra give a distorted view of the inelastic strength demands as they tend to disappear with increasing ductility ratios. The peaks and valleys in the strength reduction spectra usually coincide with those of the elastic strength demand spectra, which explains why the inelastic strength demand spectra are much smoother than the elastic ones.

Systems without strain hardening tend to drift more, thus requiring higher strength capacities for the same target ductility ratios. The difference between strength demands for 0 and 10% strain hardening is in the order of 10-20% for bilinear systems. The difference between strength demands for 0 and 2% strain hardening is in the order of 5-10%, i.e., small strain hardening can be very effective.

The differences in strength demands between bilinear and stiffness degrading Clough models are usually small and in many cases the degrading model gives favorable results (smaller strength demands). Much effort is often devoted to refined hysteresis modelling for elements and structures. With regards to assessment of ductility or strength demands, this effort may not be warranted.
provided that stiffness degradation is of a type similar to that described by the Clough model.

- Hysteretic and input energy spectra (per unit mass) are not very sensitive to the target ductility ratio and are quite similar in shape.

- A direct consequence of the increased drifting of systems without strain hardening is that such systems generally undergo smaller inelastic excursions and dissipate less hysteretic energy for a given target ductility ratio (see Figs. 4.39, 4.41 and 4.42).

- Stiffness degrading models tend to dissipate more hysteretic energy as they execute many more small inelastic excursions.

- The contribution of hysteretic energy to total dissipated energy (HE / TDE) is not very sensitive to period and increases only moderately with the target ductility ratio.

- Except for very short period systems, the maximum input energy occurs at the end of the ground motion records provided the ground motions are not overpowered by large velocity pulses.

- The elastic and inelastic strength demand spectra can give a distorted picture of hysteretic or input energy demands. This conclusion is based on a comparison of Figs. 4.12 and 4.13 with Figs. 4.61 and 4.62, respectively.

- Normalized hysteretic energy, \( NHE \), is a good index for comparing relative cumulative damage for systems with the same target ductility ratio.

- The number of inelastic excursions sharply increases with decreasing period, while the average plastic deformation range of the excursions decreases, but at a slower rate. The net result is an increase in \( NHE \) demand in the short period range, but the increase occurs at a slower rate than that of the number of inelastic excursions.

- For a given ductility ratio, the maximum inelastic excursion is almost period independent and not very sensitive to ground motion characteristics. It is in the order of 75 to 80% of \( 2(\mu-1)(1-\alpha) \), where \( \mu \) is the target ductility ratio and \( \alpha \) is the strain hardening ratio.
• Strong motion duration has an important effect on cumulative damage. The "effective" strong motion duration experienced by the system is a function of the frequency characteristics of the ground motion as well as structural response characteristics (period and target ductility ratio). Presently used definitions of strong motion duration, which do not consider structural response characteristics, cannot be used as general indicators of cumulative damage. More research needs to be directed towards formalizing system dependent strong motion duration parameters.

• The cumulative damage information presented in Chapter 4 may be used to weigh (modify) the ductility capacity of structures.

In the MDOF study three types of multi-degree of freedom structures are comprehensively analyzed for an evaluation of story ductility, shear, and overturning moment demands. The three MDOF models studied are: (a) BH (beam hinge) models, in which plastic hinges will form in beams only (as well as supports), (b) CH (column hinge) models, in which plastic hinges will form in columns only, and (c) WS (weak story) model, in which plastic hinges will form in columns of the first story only. The main objective of the MDOF study is to estimate the modifications required to the inelastic strength demands obtained from bilinear SDOF systems, in order to limit the story ductility demand in the first story of the MDOF systems to a prescribed value. The main conclusions derived from the parametric study of these MDOF systems are summarized as follows.

• MDOF story ductility demands differ significantly from those of the corresponding SDOF systems. The maximum ductility demands occur usually in the first story and are usually higher than those of the SDOF systems. The deviation of MDOF story ductility demands from the SDOF target ductility ratios increases with period (number of stories) and target ductility ratio, and decreases with strain hardening. MDOF systems that can develop story mechanisms tend to drift more.

• The required MDOF base shear capacity for specified target ductility ratios depends strongly on the type of failure mechanism that will develop in the structure during severe earthquakes. Quantitative information is developed on the relative strength requirements for three types of MDOF structures,
illustrating the disadvantage of structures in which story mechanisms develop, and particularly the great strength capacities needed to control inelastic deformations in structures with weak stories.

- For BH structures the strength modification factor, which relates the required base shear strength of MDOF structures to the strength demand predicted from SDOF systems for the same target ductility, is smallest and is mostly in good agreement with the ATC-3 modification (raising of the 1/T spectral ordinates to $1/T^{2/3}$), provided that there is significant strain hardening ($\alpha = 10\%$). Even here the caveat exists that for longer period structures and higher target ductility ratios somewhat larger modifications are required than indicated by the ATC-3 approach. For short period BH structures, the base shear strength demand is slightly lower than the corresponding SDOF strength demand, indicating that MDOF effects are not important for this range. Larger modifications are required for CH structures. The WS structures require much greater strengths due to the problems inherent in the weak story system. Quantitative information on these modification factors is presented in Chapter 5.

- For the regular structures studied here, elastic MDOF systems attract lower base shears than those predicted by the equivalent SDOF systems.

- Extreme strength discontinuities, such as those in the WS structures, should be avoided whenever possible, as they lead to excessive ductility and overturning moment demands that may be greatly amplified by the elastic vibration of the upper portions of the structure.

- The results of the MDOF study clearly indicate that overturning moments in inelastic structures can be very large. If the story strengths are tuned to the code required strength levels, it is likely that all stories will yield simultaneously and, therefore, the maximum overturning moments should be based on the shear strengths of all stories above with due consideration given to strain hardening. No overturning moment reduction factors should be applied. In most real designs, the story strengths cannot be tuned precisely to the code strength levels and individual stories may have a shear strength larger than required. In such cases the overturning moments will increase further. This study provides no information on the magnitude of this increase, as it depends on the relative strength of each story and cannot be generalized.
It must be emphasized that this study focuses only on a small part of a big problem. The seismic demands are evaluated only for selected ground motions in rock and firm soils. No conclusions can be drawn on demands imposed by ground motions on structures located on soft soils. The great importance of the soil site conditions was clearly demonstrated again by the 1989 Loma Prieta Earthquake. Thus, much more work needs to be done in the context of demand evaluation for seismic design, considering that input ground motions in general are sensitive to source mechanism, source-site distance, orientation, travel path through geologic media, and local site conditions.
REFERENCES


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