Effects of Near-Field Ground Motion on Building Structures

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CUREE-Kajima Joint Research Program
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1. INTRODUCTION

1.1. Statement of Problem

Near-field ground motions have caused much damage in the vicinity of seismic sources during recent earthquakes (Northridge 1994, Kobe 1995). There is evidence indicating that ground shaking near a fault rupture is characterized by a short-duration impulsive motion that exposes the structure to high input energy at the beginning of the record. This pulse-type motion is particularly prevalent in the "forward" direction, where the fault rupture propagates towards the site at a velocity close to the shear wave velocity. The radiation pattern of the shear dislocation of the fault causes the pulse to be mostly oriented perpendicular to the fault, causing the fault-normal component of the motion to be more severe than the fault-parallel component (Somerville, 1998). This phenomenon requires consideration in the design process for structures that are located in the near-field region, which is usually assumed to extend about 10 to 15 km from the seismic source (1996 SEAOC Bluebook).

Near-field ground motions exhibit special response characteristics that are different from the response characteristics of "ordinary" ground motions. This is shown in Fig. 1.1, which compares velocity response spectra of near-field and ordinary ground motions. The solid line (denoted as 15-D*) represents the mean velocity spectrum of a set of ordinary ground motions whose individual spectra resemble the 97 UBC soil type SD spectrum. The other lines correspond to the velocity spectra of individual near-field ground motions from different events. The figure illustrates significant variations in the response of SDOF systems to near-field ground motions. Every near-field record shows unique characteristics that distinguish it from others. The figure also indicates that near-field ground motions impose seismic demands on structures that are several times those imposed by US design level "ordinary" ground motions.

The response of MDOF structures to near-field ground motions also demonstrates special properties. Figure 1.2 compares the story ductility demands of a 2-second 20-story MDOF structure subjected to near-field and ordinary ground motions. The base shear strength of this structure is defined by the base shear coefficient $\gamma = \frac{V_y}{W} = 0.15$. The heavy solid line represents the mean story ductility demands for the same set of ordinary ground motions as shown in Fig. 1.1. The uniqueness of the MDOF response to near-field records is again prevalent. Unlike ordinary ground motions, the distribution of the demands over the height of the structure is highly non-uniform for the near-field records. The severity of near-field ground
motions leads to ductility demands that are significantly larger than those for the ordinary records that represent code design ground motions.

The special response characteristics of near-field ground motions deserve much scrutiny. The development (or improvement) of design guidelines for structures close to a seismic source requires a thorough understanding of near-field response phenomena. The near-source factors incorporated in recent US codes are insufficient to solve the problem consistently, because they pay little attention to the physical response characteristics of near-field ground motions. It may also be necessary to modify the design shear force distribution over the height of the structure. Moreover, the emerging concepts of performance-based seismic design require a quantitative understanding of response at different performance levels, ranging from nearly elastic behavior to highly inelastic behavior associated with incipient collapse.

1.2. Objectives and Scope

This study addresses the elastic and inelastic response of SDOF systems and MDOF frame structures subjected to near-field ground motions. The global objective is to acquire quantitative knowledge on near-field ground motion effects. The results of this study are intended to identify salient response characteristics, to describe near-field ground motions by simple equivalent pulses, and to utilize the pulse response characteristics to define behavior attributes of structures when subjected to near-field ground motions. The ultimate goal is to develop design guidelines that provide more consistent protection for structures located in near-field regions.

A set of recorded near-field ground motions is utilized in the response investigations. The ground motions are introduced in Chapter 2, which also addresses the effect of directivity and various components of near-field motions. In order to derive general rather than specific information, generic rather than particular structures are used in the response evaluations. Chapter 3 presents a description of the generic frame structures used in this study and the assumptions made in their design.

Chapter 4 focuses on the elastic and inelastic response of structures to near-field ground motions. Salient near-field response characteristics and differences from characteristics of ordinary ground motions are identified. Global and story drift demands of the generic structures are investigated through a comprehensive parametric study that describes the variation of seismic demands with structure parameters such as fundamental period and base shear strength. The pulse-type properties of near-field ground motions provide motivation for representing these ground

Chapter 1
motions by a small number of simple pulses, which can significantly facilitate the process of response prediction and design. Such simple pulse shapes and their spectral properties are discussed in Chapter 5. Chapter 6 addresses the elastic and inelastic demands of structures subjected to the simple pulses using an extensive parametric study that takes into account the parameters of the structure and the pulse. Since near-field ground motions tend to impose large displacement demands on frame structures, giving rise to second-order demand amplification, P-delta effects are also addressed in this study. The issue of representing near-field ground motions by equivalent pulses is pursued in Chapter 7.

In Chapter 8 particular steel structure models are employed for verification and calibration purposes, and to assess the extent to which the results obtained from the generic structures can be generalized. Design implications for near-field ground motions are presented in Chapter 9, which summarizes the results of a statistical study that relates the design base shear to the magnitude and distance of the event. Improved distributions of the design story shear force over the height of the structure are also investigated.

Chapter 10 is concerned with the study of a set of near-field ground motions recorded during the 1995 Kobe Earthquake. Response of SDOF and MDOF structures to this record set is investigated, and equivalent pulses are established that can represent the near-field ground motions.

Many fundamental characteristics of near-field ground motions and their effects on frame structures have been identified and quantified in this study. But it is recognized that the near-field problem is very complex, and that more work is needed before a comprehensive understanding of all important aspects of the problem will be accomplished. This work attempts to address the most important issues concerning near-field ground motions and their response attributes in order to form a foundation on which to base future research and development of design guidelines.
Chapter 1

Introduction

Figure 1.1 Velocity Response Spectra of Near-Field and Ordinary Ground Motions

Figure 1.2 Story Ductility Demands for Near-Field and Ordinary Ground Motions
2. NEAR-FIELD GROUND MOTIONS USED IN THIS STUDY

2.1. Ground Motion Records

A set of 23 near-field ground motion records is utilized in this study. The designation and basic properties of these recorded ground motions are listed in Table 2.1. The first 10 ground motions in the table were assembled by Somerville for the SAC Steel Project (Somerville et al., 1997a). The other 13 near-field records listed in the table were provided by Somerville for the CDMG Strong Motion Instrumentation Program (Somerville, 1998). The ground motions are either recorded on soil or have been modified to NEHRP soil type SD conditions. The exception is IV40ivir, which is modified to soft soil conditions. The ground motions cover a moment magnitude range from 6.2 to 7.4 and a distance (closest distance from the fault) range from 0.0 to 10.0 km. A complete set of ground time history traces is presented in Appendix A for the fault-normal component of the records with forward directivity.

In all near-field time histories there should be static displacements due to the static dislocation field of the earthquake. However, most recording systems do not adequately record the permanent displacements, which are filtered out of the recordings in the course of processing. Somerville has not attempted to retain the static displacement field in any of the time histories, with the exception of the Lucerne recording of the 1992 Landers earthquake (LN921ucr). This time history has been modified by Graves (1996) compared to the version of Iwan and Chen (1994) to include geodetically defined static displacements.

2.1.1. Directivity Effects

The record set includes recordings with both forward and backward rupture directivity. If the rupture propagates towards the site, the recording at the site will show forward-directivity effects. Since the propagation occurs at a velocity that is close the shear wave velocity, most of the seismic energy from the rupture arrives at the site in a large short-duration pulse of motion at the beginning of the record (Somerville et al., 1997b). This large pulse is mostly oriented in the fault-normal direction on account of the radiation pattern of shear dislocation on the fault. Figure 2.1 illustrates ground time history traces for the fault-normal component of a near-field ground motion (LN921ucr) that was recorded in the forward-directivity region during the 1992 Landers earthquake. The large pulse of motion is clearly observed in the velocity and displacement time histories.
If the rupture propagates away from the site, the recording at the site will show backward-directivity effects. Records with backward directivity exhibit long-duration motions that have low amplitudes at long periods (Somerville et al., 1997b). Figure 2.2 presents the time histories for a ground motion (LN92josh) that was recorded in the backward-directivity region of the Landers earthquake. As can be seen, this record does not show the pulse-type characteristics typical of records with forward directivity. Instead, the seismic energy arriving at the site is scattered throughout a long-duration ground motion. It is also observed that the maximum ground acceleration, velocity, and displacement of this backward-directivity record are significantly smaller than their corresponding values of the forward-directivity record LN92lucr, even though LN92josh is recorded at a station that is closer to the epicenter of the Landers earthquake. This study focuses only on the response characteristics of near-field ground motions with forward directivity.

2.1.2. Ground Motion Components

Figure 2.3 illustrates ground velocity and displacement traces for the fault-normal and fault-parallel components of the near-field record NR94rrs. This ground motion, which is recorded in the forward-directivity region of the 1994 Northridge Earthquake, shows a large pulse of motion in the time range from 2 to 3 sec. of the fault-normal trace. As pointed out earlier, the fault-normal component of the motion is much more severe than the fault-parallel one due to the radiation pattern of the shear dislocation. Therefore, the orientation of the structure with respect to the fault direction will determine the severity of the ground motion that the structure may experience in the near-field region of a fault rupture.

In order to obtain a better understanding of the effect of structure orientation, Fig. 2.4 shows ground velocity and displacement time histories for two rotated components of the same ground motion. These components, which are rotated by 45° with respect to the fault direction, are obtained by combining the fault-normal and fault-parallel time histories. It can be seen that the two rotated components also exhibit pulse-type characteristics. The time history trace of one of the rotated components is very similar to that of the fault-normal component (Fig. 2.4(a)). Thus, it appears that pulse-type characteristics are not particular only to the fault-normal direction. The study of the time history traces also suggests that the rotated components are relatively severe. The severity of the rotated components is further addressed using spectral values.
2.2. Elastic Spectra of Near-Field Ground Motions

Figure 2.5 illustrates acceleration (elastic strength demand), velocity, and displacement spectra of the near-field ground motion NR94rrs, whose ground time histories were illustrated earlier. Each graph includes the spectra for the fault-normal, fault-parallel, and the two 45° rotated components of this ground motion. All spectra are computed for 2% damping. The figure clearly shows the large difference between the fault-normal and fault-parallel components. These results as well as the elastic spectra of other near-field ground motions with forward directivity (see Appendix A) indicate that the fault-normal component is much more severe than the fault-parallel component. When these two components are rotated by 45°, the difference in the spectra becomes smaller, but one of the two rotated components still will impose demands close to (and sometimes even higher than) those associated with the fault-normal component. This pattern is consistent for all of the near-field records with forward directivity studied here (Appendix A). Thus, when a 3-D structure composed of frames in two perpendicular directions is subjected to a near-field ground motion, frames in one of these two directions will always be exposed to excitations with an intensity level close to that of the fault-normal component. This provides sufficient justification for focusing on the fault-normal component of near-field ground motions in this study.

Another important observation from the spectra is the existence of a predominant peak in the fault-normal velocity spectrum of most of the near-field records. However, some of the records used in this study have more than one clear velocity peak. Later in Chapter 7, it is shown that identifying the predominant peak of the velocity response spectrum is the key to estimating the period of the pulse contained in the near-field record.

As pointed out earlier, some of the ground motions used in this study are originally recorded on rock and have been analytically converted into soil motions (Somerville, 1998). Figure 2.6 shows the elastic response spectra of the fault-normal component of the near-field record KB95kobj, which has been modified from rock to soil conditions. The figure compares the elastic spectra of the ground motion before and after the modification is made. As the figure indicates, the spectral values of the original and converted records are almost identical in the period range T < 0.7 sec. However, at longer periods the spectral values of the converted soil motion vary between 1.6 and 1.9 times those of the original rock ground motion.
2.2.1. Comparison with Ordinary Ground Motions

A reference set of 15 “ordinary” records is utilized for comparison purposes. These records, which were used in past studies (Seneviratna and Krawinkler, 1997), are scaled in a way such that the spectrum of each individual record matches the 97 UBC soil type S_D spectrum with a minimum error, using discrete periods in the range from 0.6 to 4.0 seconds (constant velocity range). The mean acceleration response spectrum of the 15 scaled records, referred to as 15-D* (mean), is shown in Fig. 2.7 together with the 97 UBC soil type S_D spectrum (Z = 0.4). Thus, on average, these 15-D* time histories are reasonable representations of presently employed design ground motions in the US.

Figure 2.8 presents the mean velocity and displacement spectra of the 15-D* records superimposed on the velocity and displacement spectra of several of the near-field records with forward directivity. This figure is presented for two reasons: first, to illustrate great variations in the response spectra that have to be expected from near-field ground motions, and second, to put the severity of near-field ground motions in perspective with present design ground motions. Maximum values of spectral velocities and displacements of the near-field records are several times those of the mean of the design ground motions. This indicates that near-field records can impose very large demands that need to be considered in the design process. The response of MDOF structures to the near-field ground motions represented by these spectra is discussed in Chapter 4.
Table 2.1 Designation and Properties of Near-Field Ground Motions Used in this Study

<table>
<thead>
<tr>
<th>Designation</th>
<th>Earthquake</th>
<th>Station</th>
<th>Directivity</th>
<th>Magnitude</th>
<th>Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>TB78tab</td>
<td>Tabas, 1978</td>
<td>Tabas</td>
<td>backward</td>
<td>7.4</td>
<td>1.2</td>
</tr>
<tr>
<td>LP89lgpc</td>
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<td>Los Gatos</td>
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<td>3.5</td>
</tr>
<tr>
<td>LP89lex</td>
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<td>Lexington</td>
<td>forward</td>
<td>7.0</td>
<td>6.3</td>
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<tr>
<td>CM92petr</td>
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<td>Petrolia</td>
<td>backward</td>
<td>7.1</td>
<td>8.5</td>
</tr>
<tr>
<td>EZ92erzi</td>
<td>Erzincan, 1992</td>
<td>Erzincan</td>
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<td>6.7</td>
<td>2.0</td>
</tr>
<tr>
<td>LN92lucr</td>
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<td>Lucerne</td>
<td>forward</td>
<td>7.3</td>
<td>1.1</td>
</tr>
<tr>
<td>NR94rrs</td>
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<td>Rinaldi</td>
<td>forward</td>
<td>6.7</td>
<td>7.5</td>
</tr>
<tr>
<td>NR94sylm</td>
<td>Northridge, 1994</td>
<td>Olive View</td>
<td>forward</td>
<td>6.7</td>
<td>6.4</td>
</tr>
<tr>
<td>KB95kobi</td>
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<td>JMA</td>
<td>forward</td>
<td>6.9</td>
<td>0.6</td>
</tr>
<tr>
<td>KB95tato</td>
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<td>Takatori</td>
<td>forward</td>
<td>6.9</td>
<td>1.5</td>
</tr>
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<td>El Centro</td>
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<td>10.0</td>
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<td>1.2</td>
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<tr>
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<td>Port Island</td>
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<tr>
<td>LN92josh</td>
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<td>Joshua Tree</td>
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<td>7.4</td>
</tr>
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<td>LP89corr</td>
<td>Loma Prieta, 1989</td>
<td>Corralitos</td>
<td>backward</td>
<td>7.0</td>
<td>3.4</td>
</tr>
<tr>
<td>MH84andd</td>
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<td>Anderson D</td>
<td>forward</td>
<td>6.2</td>
<td>4.5</td>
</tr>
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<td>Coyote L D</td>
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<td>6.2</td>
<td>0.1</td>
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<td>2.4</td>
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<td>NR94newh</td>
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<td>NR94nord</td>
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<td>9.2</td>
</tr>
<tr>
<td>NR94spva</td>
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<td>Sepulveda</td>
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<td>8.9</td>
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Figure 2.1 Ground Acceleration, Velocity, and Displacement Time Histories of Fault-Normal Component of Record LN921ucr with Forward Directivity
Figure 2.2 Ground Acceleration, Velocity, and Displacement Time Histories of Fault-Normal Component of Record LN92josh with Backward Directivity
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Figure 2.6 Comparison of Elastic Spectra of Original (Rock) and Converted (Soil) Motions for Fault-Normal Component of Record KB95kobj
Figure 2.7 Mean Acceleration (Elastic Strength Demand) Spectrum of Reference Set of Records (15-D*) Superimposed on 97 UBC Soil Type S_D Spectrum
Figure 2.8 Velocity and Displacement Response Spectra of Near-Field Ground Motions and Reference Ground Motions
3. SDOF AND MDOF SYSTEMS USED IN THIS STUDY

3.1. SDOF Systems

Fundamental studies are carried out with elastic and inelastic SDOF systems in order to capture basic response characteristics that differentiate near-field ground motions from "ordinary" ground motions. The elastic period $T$ of the SDOF system is varied at closely spaced intervals to provide accurate spectral information within the range of interest. For recorded ground motions the period range is between 0 and 4.0 seconds, and for basic pulse-type ground motions the primary range of interest for $T/T_p$ is between 0 and 3.0, where $T_p$ is the period of the pulse. For all systems a damping ratio of $\xi = 2\%$ is used rather than the more customary value of 5%. The reason is that the focus of the study is on steel frame structures for which 5% damping is difficult to justify.

Inelastic SDOF systems are defined by a non-degrading bilinear skeleton curve and basic hysteresis rules. The yield strength is denoted as $F_y$, and the strain-hardening ratio is represented by $\alpha$. Unless noted otherwise, a value of $\alpha = 0.03$ is used to model hardening that is representative of typical steel frame structures.

3.2. MDOF Systems

3.2.1. Properties of Generic Structure

One of the main objectives of this study is to quantify the seismic demands of multistory frame structures subjected to near-field ground motions and simple pulses. To achieve this goal, a generic 2-dimensional frame structure is used whose strength and stiffness properties can be tuned to specific requirements in order to facilitate interpretation and generalization of response results. In this generic structure, the fundamental elastic period $T$ is a variable, but the number of stories is kept constant at 20. It was considered impractical to vary the number of stories because of the emphasis on pulse loading which is characterized by a pulse period $T_p$ rather than a specific numerical value of $T$ that can be associated with a specific number of stories.

In its physical configuration, the generic structure constitutes a single-bay moment-resisting frame whose story strengths and stiffnesses are tuned to specific requirements that are discussed in the next section. Inelastic deformations are permitted only at the ends of the beam in each story and at the base of the columns. Thus, the basic plastic hinge mechanism under lateral loads...
involves all stories, with no individual story mechanism allowed. This mechanism is illustrated in Fig. 3.1.

The following assumptions are made in the design of the generic model structure:

- Floor mass is the same in every story and at the roof level.
- Story height is the same in every story.
- Bay width is twice the story height.
- Beam and column moments of inertia are the same in each story.
- Only flexural deformations are considered.
- The variation of moment of inertia over the height is tuned such that the later defined SRSS lateral load pattern results in a straight-line deflected shape of the structure.
- The beam bending strength in each story is tuned such that under the SRSS lateral load pattern simultaneous yielding occurs in all stories.
- The effect of gravity load moments on plastic hinge formation is not considered.
- A bilinear non-degrading hysteresis model with a 3% strain-hardening ratio is used at all plastic hinge locations.

For time history analyses, Rayleigh damping is used to obtain a damping ratio of 2% at the first mode period T and at 0.1T. All MDOF structural analyses in this study are performed using the DRAIN-2DX computer program (Prakash et al., 1993).

As discussed in Section 6.5, a generic 3-story frame structure is also used in a sensitivity analysis to assess the demands for short-period MDOF structures subjected to near-field ground motions. The same assumptions and procedures introduced in this chapter are employed in the design of the 3-story structure. Furthermore, a pilot study is carried out to quantify the response obtained from frame models of real steel structures in order to validate the seismic demands derived from generic structures (Chapter 8).

3.2.2. Design Load Pattern

In order to establish story stiffness and strength properties, a design lateral load pattern and base shear strength are required. The base shear yield strength is varied according to specific objectives of the analysis and is discussed later. Given the base shear yield strength, the individual story shear yield strengths are tuned to the story shear forces obtained from the design load pattern. As a result, all stories will yield simultaneously if the lateral loads follow the
design load pattern. Thus, global and story “shear force - drift” relationships obtained from a pushover analysis with the design load pattern will mimic the bilinear shape corresponding to the SDOF systems summarized in Section 3.1.

In previous studies by Nassar and Krawinkler (1991), and Seneviratna and Krawinkler (1997), the UBC seismic load pattern was used for stiffness and strength design of generic models. In this study it was decided to utilize a load pattern that is based on dynamic properties rather than code assumptions. A load pattern was selected for this purpose which is based on story shear forces obtained from the SRSS modal superposition method. The SRSS analysis requires the selection of a design spectrum. It is assumed that the design spectrum follows a 1/T shape for acceleration (or constant velocity) at all modal periods that contribute significantly to the SRSS combination. This assumption, together with the requirement that the deflected shape under the design load pattern should be a straight line, results in the story shear force and design load patterns illustrated in Figs. 3.2 and 3.3. Unlike the UBC “triangular” load pattern, the SRSS pattern does not show a linear variation over the height of the structure. Particularly, the roof lateral load is significantly larger than the lateral load at the next lower floor level.

Since the story shear forces obtained from the SRSS combination depend on relative story stiffnesses, an iterative procedure is required to tune the element stiffnesses so that a straight-line deflected shape is obtained under the SRSS load pattern. Basic dynamic properties of the generic structure (period ratios, effective masses, and modal participation factors) that fulfill the stiffness design requirements are listed in Table 3.1.

3.2.3. Design for P-Delta Effects

It is expected that dynamic P-delta effects will be of major concern for structures subjected to the large displacement pulses of near-field ground motions, particularly if inelastic interstory drifts become large and lead to drifting (displacement amplification) of the seismic response. Thus, P-delta effects should be incorporated explicitly in the design of the generic structure.

To simulate P-delta effects, identical gravity loads are assigned to each story. This implies that axial column forces due to gravity loads increase linearly from the top to the bottom of the frame. The magnitude of the story gravity load is determined so that in the first story the elastic second-order interstory drift is 10% of the first-order interstory drift under the SRSS lateral loads. In the elastic range the consequence of incorporating P-delta effects is a 10% reduction in elastic stiffness in the first story, and a smaller reduction in higher stories.
The effect of P-delta on the inelastic response is illustrated in Fig. 3.4, which shows (a) base shear versus roof displacement, and (b) base shear versus first story displacement diagrams obtained from a pushover analysis. Results without and with consideration of P-delta effects are presented. If P-delta effects are neglected (without P-delta), the global and interstory strain-hardening stiffnesses are between 3.6% and 3.7% of the elastic stiffness. This value is different from the 3% strain hardening assumed at plastic hinge locations, because the columns remain elastic after the beam plastic hinges have formed, and contribute to the stiffness in the post-elastic range. Incorporating P-delta effects decreases the elastic stiffness by 10%, and decreases the strain-hardening ratio from +3.7% to -14.4% for the global response, and from +3.6% to -2.8% for the first story response. The large effect on the global response is due to the cumulative nature of the global displacement response (summation of all story drifts). The fact that the decrease of post-elastic stiffness in the first story is less than 10% of the elastic stiffness is attributed to the change in the deflected shape of the structure once a mechanism has formed.
Table 3.1 Basic Dynamic Properties of Generic Structures

<table>
<thead>
<tr>
<th>Mode #</th>
<th>$T_1 / T_1$</th>
<th>Effective Mass, %</th>
<th>Participation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.000</td>
<td>78.5</td>
<td>1.37</td>
</tr>
<tr>
<td>2</td>
<td>0.371</td>
<td>10.9</td>
<td>0.59</td>
</tr>
<tr>
<td>3</td>
<td>0.225</td>
<td>4.1</td>
<td>0.37</td>
</tr>
<tr>
<td>4</td>
<td>0.159</td>
<td>2.1</td>
<td>0.26</td>
</tr>
<tr>
<td>5</td>
<td>0.121</td>
<td>1.3</td>
<td>0.18</td>
</tr>
<tr>
<td>6</td>
<td>0.096</td>
<td>0.8</td>
<td>0.14</td>
</tr>
<tr>
<td>7</td>
<td>0.078</td>
<td>0.6</td>
<td>0.11</td>
</tr>
<tr>
<td>8</td>
<td>0.065</td>
<td>0.4</td>
<td>0.10</td>
</tr>
<tr>
<td>9</td>
<td>0.055</td>
<td>0.3</td>
<td>0.09</td>
</tr>
<tr>
<td>10</td>
<td>0.047</td>
<td>0.2</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Figure 3.1 Plastic Hinge Mechanism for Generic Frame Structure under Lateral Loads
Figure 3.2 Story Shear Force Pattern Based on SRSS Combination

Figure 3.3 Lateral Load Pattern Based on SRSS Story Shear Forces
Figure 3.4 Global and First Story Pushover Results with and without P-Delta
4. RESPONSE OF STRUCTURES TO NEAR-FIELD GROUND MOTIONS

This part of the study is devoted to an evaluation and quantification of the elastic and inelastic response of SDOF and MDOF structures of different periods subjected to near-field ground motions. Attempts are made to characterize important response characteristics of near-field records. The near-field ground motions introduced in Chapter 2 and the structure models introduced in Chapter 3 are utilized in the response evaluations. Structures with different base shear strength are investigated to identify near-field behavior patterns at different structure performance levels. The reference ground motion set presented previously is used to emphasize major differences in the inelastic response of MDOF structures subjected to near-field and ordinary ground motions.

4.1. Elastic Response of MDOF Structures

4.1.1. Elastic Base Shear Demands

Examples of maximum elastic base shear forces of the generic structures subjected to the fault-normal component of near-field ground motions are presented in Fig. 4.1. Each graph compares the maximum MDOF base shear forces for different fundamental periods with the corresponding SDOF elastic strength demand spectrum.

There is a rather close agreement between the MDOF and SDOF results. The general observation is that the MDOF base shear is smaller than the first mode SDOF strength demand if higher-mode effects are not important, and is larger than the SDOF demand if higher-mode effects are important (large peaks in spectrum at second and/or third mode periods). The design implication is that if the elastic design spectrum incorporates the effects of near-field ground motions, elastic MDOF base shear demands follow patterns similar to those for ordinary ground motions.

4.1.2. Elastic Shear Force Distribution Over Height of Structure

Figures 4.2 and 4.3 compare the SRSS story shear force pattern, which was used in the design of the generic structure, with the patterns obtained from (a) time history analyses, and (b) SRSS modal combinations for ground motions NR94rrs and KB95kobj, using MDOF systems with various fundamental periods T. The results of the time history analyses indicate that for
structures with a short fundamental period \((T \leq 1.0 \text{ sec. for NR94rrs and } T \leq 1.5 \text{ sec. for KB95kobj})\), the patterns are smooth and relatively close to the SRSS pattern. On the other hand, the distributions of the story shear forces over the height for long-period systems differ significantly from the SRSS pattern and exhibit the effect of a wave traveling up the structure. It appears that the traveling wave effect dominates the MDOF response of structures whose fundamental period is longer than a particular value that depends on the properties of the pulse contained in the near-field ground motion. In this context, the distribution of story shear forces provides useful information for estimating the effective pulse period of a near-field ground motion. The issue of identifying effective pulses for near-field ground motions is pursued more rigorously in Chapter 7.

The results presented in Figs. 4.2 and 4.3 also indicate that the SRSS modal superposition technique can capture only partially the traveling wave effect. For short-period structures, the distribution of story shear forces obtained from the SRSS analysis is close to the corresponding distribution obtained from the time history analysis, whereas for long-period structures larger differences can be seen. The reason is that in long-period structures the wave traveling up the structure gives rise to higher-mode effects, which are not taken into account accurately by the SRSS modal combination.

The significant deviation of the story shear force pattern obtained from a time history analysis from that of the SRSS pattern indicates that presently employed design shear force patterns will lead to early yielding in upper stories for structures with a long fundamental period. The reason is a traveling wave effect caused by the pulse-type nature of near-field ground motions. The effect of this traveling wave on the inelastic demands of MDOF structures is investigated in Section 4.2.2.

4.1.3. Elastic Roof Displacement Demands

Figure 4.4 presents ratios of the elastic MDOF roof displacement demand to the first-mode spectral displacement, \(\delta_{\text{roof,max}}/S_d\), for different periods \(T\) and the fault-normal component of typical near-field records. Each graph includes two curves, one for MDOF systems in which P-delta effects are neglected, and the other for systems with P-delta effects. When P-delta effects are considered, the fundamental period of the structure slightly elongates because the secondary effects reduce the effective stiffness of the structure. The first mode participation factor (PF₁) is 1.37, which is equivalent to \(\delta_{\text{roof,max}}/S_d\) when only the first mode of the structure is taken into
account. Different patterns of deviation from this reference value for different ground motions emphasize the uniqueness of the near-field records.

The general pattern is that the ratio oscillates about the predicted value of PF₁ for relatively short periods, and usually exceeds the predicted value by a large amount at long periods. This observation does not comply with the results of the study performed by Seneviratna and Krawinkler (1997) for ordinary ground motions. This indicates that higher-mode effects are more significant in long-period MDOF systems subjected to near-field ground motions.

The results also indicate that if the period elongation due to secondary effects is accounted for in the computation of the first mode spectral displacement (Sₐ), the ratio of δₘₐₓ/Sp will be close to the corresponding ratio obtained when P-delta effects are neglected.

4.2. Inelastic Response of SDOF and MDOF Structures

In this part of the study inelastic demands of SDOF and MDOF systems subjected to near-field ground motions are investigated. For MDOF systems, the base shear yield strength is quantified by a base shear coefficient, γ, defined as

\[
\gamma = \frac{V_y}{mg} = \frac{V_y}{W}
\]

(4.1)

where Vᵧ is the base shear strength, g is the acceleration of gravity, and W and m are the seismically effective weight and mass of the structure, respectively. Once the base shear strength is defined, the distribution of strength over the height follows an SRSS story shear force pattern obtained from a constant velocity spectrum, which was discussed in Section 3.2.2. The yield strength of SDOF systems is defined using the same coefficient, γ, and substituting the SDOF yield strength, Fᵧ, for the base shear strength, Vᵧ, in Eq. 4.1.

4.2.1. SDOF Systems

Displacement Time History:

Figure 4.5 illustrates inelastic displacement response time histories for SDOF systems with different fundamental periods subjected to the two near-field ground motions LP89lex and NR94rrs. In each case the strength of the SDOF system is selected such that a ductility ratio (μ =
u_{\text{max}}/u_y) of 6 is obtained. The displacement time history values are normalized by the yield displacement \( u_y \) for the corresponding structure period and input ground motion. The figure clearly demonstrates that the response to the near-field ground motions is one-sided, with no more than two large inelastic excursions followed by small elastic cycles. These pulse-type response characteristics differentiate near-field ground motions from ordinary ground motions. These observations invite the conjecture that the response of structures to near-field records can be replicated using pulse shapes as the input motion. Identifying such pulse shapes is one of the main objectives of this study.

**Constant Ductility Strength Demand Spectra:**

Figure 4.6 shows examples of elastic (\( \mu = 1 \)) and constant ductility inelastic strength demand spectra for the fault-normal component of near-field records NR94rrs and KB95kobj. The inelastic spectra are presented for target ductility ratios \( \mu = 2, 3, 4, 6, \) and 8. Similar to observations made in past studies (Nassar and Krawinkler, 1991, and Rahnama and Krawinkler, 1993), the humps of the elastic spectra diminish and even disappear at large ductility ratios. At the same time, the smaller peaks and valleys of the inelastic spectra shift to lower periods, which can be rationalized by the fact that the “effective period” of the structure elongates when the ductility increases.

**Ductility Demands for Various Strength Levels and Periods:**

Figures 4.7 and 4.8 present the ductility demands of SDOF systems, subjected to near-field records NR94rrs and KB95kobj, versus (a) the normalized yield strength \( \gamma \), and (b) the inverse of \( \gamma \), which is a relative measure of ground motion severity. In order to evaluate the effect of the structure period, the variation of the ductility demands with yield strength is presented for selected period values \( T = 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 \) and 4.0 seconds.

In many cases the ductility demand varies almost linearly with the inverse of structure strength (or \( 1/\gamma \)), particularly in the large ductility range, which indicates a linear \( R-\mu \) relationship. Deviations from a linear \( 1/\gamma-\mu \) relationship are noted primarily in the range of small ductility demands.
4.2.2. MDOF Systems

The story ductility demands are used here as the basic performance parameter of MDOF structures. The story ductility ratio is defined as the maximum interstory drift normalized by the story yield drift, i.e., $\mu_i = \frac{\delta_{\text{max},i}}{\delta_{y,i}}$. The story yield drift is obtained from the static pushover under the SRSS story shear force pattern. Distributions of story ductility demands over the height of the structure are studied to evaluate the story response characteristics of MDOF frame structures subjected to near-field ground motions. Maximum story ductility demands (maximum of all stories) are also presented, which can be directly compared with the SDOF ductility demands given in the previous section. The story ductility demands are utilized also to evaluate the importance of dynamic P-delta effects.

Story Ductility Demands Over Height:

Story ductility demands obtained from near-field records need to be put in perspective with the demands obtained from ordinary ground motions. For this purpose, Fig. 4.9 illustrates the distribution of story ductility demands over the height of structures subjected to the near-field records whose velocity and displacement response spectra are shown in Fig. 2.8. The demands are computed for an MDOF system with a fundamental period $T = 2.0$ sec. and base shear strength coefficients of $\gamma = 0.4$ and $\gamma = 0.15$, which represent a relatively strong and a relatively weak structure, respectively. For comparison purposes, the mean story ductility demands obtained from the reference record set 15-D* (see Section 2.2.1) are superimposed.

As the results indicate, for most of the records the maximum story ductility demand occurs in the top portion of the structure when the structure is strong (large $\gamma$), whereas a migration of ductility demands toward the base takes place when the structure becomes weaker (or the ground motion becomes more severe). This migration of ductility demands and the consequent concentration of demands at the base, which occur for structures whose first mode period is longer than the period of the pulse contained in the ground motion, are basic phenomena that characterize near-field ground motions. The graphs also indicate that, in the mean, an SRSS-based strength design results in a relatively uniform ductility distribution for ordinary ground motions (15-D*), whereas the same design causes great variations of ductility demands over the height when the structure is subjected to near-field ground motions.

Strength-dependent distributions of story ductility demands over the height for structures subjected to near-field records NR94rrs and KB95kobj are shown in Figs. 4.10 and 4.11. Each
figure illustrates the variation of ductility distribution as the base shear strength changes, covering a range from elastic behavior to large ductility demands. This variation is shown for structures with two different fundamental periods, i.e., (a) $T = 0.5$ sec. (shorter than the effective pulse period), and (b) $T = 2.0$ sec. (longer than the effective pulse period). For long-period systems (part (b)) the consistent observations are: (1) the occurrence of maximum ductility demands in upper stories for relatively strong structures, (2) the consequent stabilization of the demand in the upper portion, and (3) the migration of demands toward the base as structures become weaker. However, these phenomena are not observed for short-period structures (part (a)). For short-period structures, the maximum ductility demands occur close to the base even at high strength values, indicating that the traveling wave effect is specific to structures with a long period (longer than the period of the effective pulse).

The reason for the early inelastic behavior in the top portion of long-period strong structures is that, as Figs. 4.2(a) and 4.3(a) indicate, the traveling wave effect in long-period systems causes the elastic shear forces in upper stories to be the first to reach the story shear capacities (which follow an SRSS pattern). This leads to premature yielding of the upper stories and translates into significant ductility demands in the top portion of long-period structures. On the other hand, in short-period structure, the shear forces in lower stories exceed the provided capacities first, resulting in large ductility demands in the bottom portion of the structure.

**Maximum Story Ductility Demands:**

Near-field and ordinary ground motions are again compared in Fig. 4.12, which shows the maximum story ductility demand (maximum of all stories) versus the base shear strength of the structure for $T = 1.0$ and 2.0 sec. For $T = 2.0$ sec. several of the curves exhibit a kink (rapid change in slope) around a maximum ductility demand of about 3 to 4, which is the range in which the maximum story ductility demand migrates from the upper portion of the structure to the base. This phenomenon is evident for ground motions whose effective pulse period is shorter than the structure period of 2.0 seconds. This phenomenon is not observed for short-period structures ($T = 1.0$ sec.) because the pulse period of all ground motions is $\geq 1$ sec. and the maximum ductility occurs near the bottom of the structure at all strength levels (i.e., no migration of ductility demands takes place). Again, a comparison of ductility demands between near-field records and the mean of 15-D* records looks alarming.

A comprehensive assessment of maximum story ductility demands of MDOF systems subjected to individual near-field records NR94rrs and KB95kobj can be obtained from the $\gamma-\mu_{\text{max}}$ curves.
presented for different periods in Figs. 4.13 and 4.14. In each figure the top diagram is for structures in which P-delta effects are neglected, and the bottom diagram pertains to structures in which P-delta effects are included. These plots can also be compared with the corresponding plots presented for SDOF systems (Figs. 4.7 and 4.8).

As observed previously, many of the $\gamma - \mu_{\text{max}}$ curves for long-period systems include kinks, which indicate a migration of ductility demands from the top stories to the base. These kinks become more noticeable in the $1/\gamma - \mu_{\text{max}}$ diagrams, which are shown in Figs. 4.15 and 4.16 for the same near-field records. These graphs clearly show a steep slope for long period structures around a ductility of 4, which is even vertical (no increase in ductility demand with a decrease in strength) in some cases. In this range of strength the maximum ductility in the upper stories stabilizes and grows no further as the strength is reduced, whereas the ductility demands at the base increase and finally exceed the upper story demands.

The ground motion severity (or the structure strength) level corresponding to the migration of ductility is a critical level for the effect of P-delta on the maximum ductility demand. A comparison of parts (a) and (b) of Figs. 4.13 and 4.14, and also 4.15 and 4.16, reveals that for long-period structures, in which the migration phenomenon is observed, P-delta effects are insignificant when the structure is sufficiently strong such that the maximum story ductility occurs in upper stories. However, once the base shear strength is reduced to a level at which the maximum demand occurs at the base, P-delta effects become significant as evidenced by the large difference in slope between the $1/\gamma - \mu_{\text{max}}$ curves without and with P-delta effects.

The maximum ductility demands without and with P-delta effects are also compared in Fig. 4.17 for KB95kobj and selected periods in both the $\gamma - \mu_{\text{max}}$ and $1/\gamma - \mu_{\text{max}}$ domains. It can be seen that in the latter domain P-delta effects decrease the slope of the curves, which simply means larger story drifts for a given base shear strength. This decrease is more significant for low-strength structures.

Figures 4.13 to 4.17 demonstrate that P-delta effects are large for short-period structures even when the structure is relatively strong. The reason is that for structures with a short period the migration phenomenon, which is the consequence of the traveling wave effect, does not occur, and the lower stories sustain the largest demands even when the structure is strong. Whenever large displacement demands occur at the bottom of the structure, where gravity loads have their highest values, significant P-delta effects should be expected.
Base Shear Strength Demands for Target Ductility:

The MDOF $\gamma$-$\mu_{\text{max}}$ diagrams presented previously can be used to estimate ductility demands of a structure subjected to a near-field record for a given strength level. Inversely, they can be used to determine the base shear strength required to limit the maximum story ductility to a target value. For a given fundamental period $T$, such base shear strength demands can be obtained from straight-line interpolations of the data points presented in the $\gamma$-$\mu_{\text{max}}$ graphs (e.g., Figs. 4.13 and 4.14) in order to compute $\gamma$ for a target $\mu_{\text{max}}$ value. Figure 4.18 illustrates examples of the so obtained base shear strength demand spectra for the fault-normal component of near-field records NR94rrs and KB95kobj and target story ductility ratios $\mu_{\text{max}}$ ranging from 1 to 8.

If the design objective is to limit the maximum story ductility to a specific target value, graphs such as those presented in Fig. 4.18 can be used to obtain the required base shear strength for structures with various first mode periods. These graphs are directly comparable with the SDOF constant ductility strength demand spectra (Fig. 4.6). If SDOF and MDOF systems responded identically to near-field ground motions, Figs. 4.6 and 4.18 would look the same. However, a comparison of the corresponding diagrams in these two figures shows that, for a given ductility ratio, long-period MDOF structures require significantly higher strength than their SDOF counterparts. On the other hand, at short periods the strength demands of MDOF systems are close to (and sometimes smaller than) the corresponding demands of SDOF systems with the same period. Again, the dividing line between “short” and “long” periods is the period of the pulse contained in the ground motion (1.0 sec. for the NR94rrs record).

Strength Demands for Rotated Components:

The elastic response spectra of the rotated components of near-field ground motions were evaluated in Section 2.2. It was shown that the elastic response for one of the rotated components may be comparable to the response associated with the fault-normal component. In this part of the study the inelastic response of MDOF structures to the rotated components of near-field records is investigated. Base shear strength demand spectra similar to those presented in Fig. 4.18 can be computed for the rotated components by employing the interpolation scheme discussed earlier. If the strength demand obtained from a rotated component is divided by the corresponding value obtained from the fault-normal component for the same $T$ and $\mu$ values, the resulting ratios represent the required base shear strength for the rotated component as a fraction of the strength required for the fault-normal component. Examples of such ratios are illustrated...
in Figs. 4.19 and 4.20 for the 45° components of near-field ground motions NR94rrs and KB95kobj.

The results indicate that one of the 45° components exposes the structure to strength demands almost as high as those obtained from the fault-normal component. The ratios larger than unity for one of the rotated components of KB95kobj (Fig. 4.20(b)) highlight the severity of this component. One could argue for a strength reduction factor of about 0.8 for the 45° rotated component, but this argument does not apply consistently. In view of the many uncertainties and unknowns involved in quantifying near-field effects, it is argued that the focus on the fault-normal component as a representative component is justified.

The major observation that summarizes the investigations presented in this chapter is that the response of structures to near-field ground motions has unique characteristics, which set them apart from ordinary ground motions. The response clearly shows pulse-type characteristics that are specific to individual ground motions and strongly depend on the structure period and strength. The effect of the structure period on the response has to be put in perspective with the effective period of the pulse contained in the near-field ground motion. In the next chapter, attempts are made to define simple pulse motions whose response attributes are similar to those of near-field records.
Figure 4.1 Elastic MDOF Base Shear and SDOF Strength Demands for Fault-normal Component of Near-Field Ground Motions
Figure 4.2 Normalized Elastic Story Shear Demands Obtained from Time History and SRSS Analyses for Record NR94rrs
Figure 4.3 Normalized Elastic Story Shear Demands Obtained from Time History and SRSS Analyses for Record KB95kobj
Figure 4.4 Ratio of Elastic MDOF Roof Displacement Demand to First Mode Spectral Displacement for Near-Field Records
Figure 4.5 Normalized Inelastic SDOF Time Histories for Various Periods and $\mu = 6$
SDOF Strength Demands for Constant Ductility
NR94rrs, Bilinear, $\alpha = 3\%, \xi = 2\%$

- $\mu = 1$
- $\mu = 2$
- $\mu = 3$
- $\mu = 4$
- $\mu = 6$
- $\mu = 8$

T (sec)

(a) Record NR94rrs

SDOF Strength Demands for Constant Ductility
KB95kobj, Bilinear, $\alpha = 3\%, \xi = 2\%$

- $\mu = 1$
- $\mu = 2$
- $\mu = 3$
- $\mu = 4$
- $\mu = 6$
- $\mu = 8$

T (sec)

(b) Record KB95kobj

Figure 4.6 Elastic and Inelastic SDOF Strength Demand Spectra for Constant Ductility Ratios for Near-Field Ground Motions
Figure 4.7 Ductility Demands for SDOF Systems Subjected to Record NR94rrs; Various Periods
Figure 4.8 Ductility Demands for SDOF Systems Subjected to Record KB95kobj; Various Periods
Figure 4.9 Story Ductility Demands for Near-Field Ground Motions and Reference Ground Motions; Structure Period $T = 2.0$ sec

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Figure 4.10 Dependence of Distributions of Story Ductility Demands on Base Shear Strength for Record NR94rrs
Figure 4.11 Dependence of Distributions of Story Ductility Demands on Base Shear Strength for Record KB95kobj
Maximum MDOF Ductility Demands
15-D* vs. Recorded Near-Field, T = 1.0 sec, without P- Δ

(a) T = 1.0 sec.

Maximum MDOF Ductility Demands
15-D* vs. Recorded Near-Field, T = 2.0 sec, without P- Δ

(b) T = 2.0 sec.

Figure 4.12 Base Shear Strength vs. Maximum Ductility Demand for Near-Field Ground Motions and Reference Ground Motions
Figure 4.13 Base Shear Strength vs. Maximum Story Ductility Demands for Record NR94rrs
Figure 4.14 Base Shear Strength vs. Maximum Story Ductility Demands for Record KB95kobj

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Figure 4.15 Inverse of Base Shear Strength vs. Maximum Story Ductility Demands for Record NR94rrs.

(a) without P-delta Effects

(b) with P-delta Effects
Figure 4.16  Inverse of Base Shear Strength vs. Maximum Story Ductility Demands for Record KB95kobj

(a) without P-delta Effects

(b) with P-delta Effects

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Figure 4.17 Effect of P-Delta on Maximum Story Ductility Demands for Record KB95kobj
Figure 4.18 MDOF Base Shear Strength Demand Spectra for Target Maximum Story Ductility; NR94rrs and KB95kobj Records
MDOF Strength Demand Ratios for Constant Ductility
NR94rrs, $0.707(FN+FP)$ vs. Fault-Normal, $\xi = 2\%$, no $P-\Delta$

(a) FN+FP Component

(b) FN-FP Component

Figure 4.19 Comparison of Base Shear Strength Demands for Rotated and Fault-Normal Components of Near-Field Record NR94rrs
MDOF Strength Demand Ratios for Constant Ductility
KB95kobj, 0.707(FN+FP) vs. Fault-Normal, $\xi = 2\%$, no $P_\Delta$

(a) FN+FP Component

MDOF Strength Demand Ratios for Constant Ductility
KB95kobj, 0.707(FN-FP) vs. Fault-Normal, $\xi = 2\%$, no $P_\Delta$

(b) FN-FP Component

Figure 4.20 Comparison of Base Shear Strength Demands for Rotated and Fault-Normal Components of Near-Field Record KB95kobj
5. PULSE-TYPE SEISMIC INPUT

As discussed in Chapters 2 and 4, pulse-type characteristics are discernible in both ground time history traces and the response of structures to near-field ground motions with forward directivity. It was also shown that near-field ground motions come in large variations, which make a consistent evaluation of near-field effects difficult and cumbersome. Even though a near-field ground motion cannot be represented precisely by any simple pulse shape, a study of basic pulses can be very useful. If simple pulse models can be found that represent near-field ground motions with reasonable accuracy, the process of design or response evaluation will be significantly facilitated. Furthermore, the study of simple pulses along with real ground motions can provide a more transparent picture of near-field response properties, and leads to a better understanding of the near-field phenomena.

The objective of this part of the study is to introduce such simplified representations of near-field ground motions. Various pulse shapes are presented and their spectral values are evaluated. Three basic pulse shapes are utilized for this purpose. Other variations are also investigated in addition to the three basic pulses. A comprehensive evaluation of response of structures to several of these pulse-type input motions is presented in the next chapter.

5.1. Basic Pulse Shapes

The following three pulses (Figs. 5.1 to 5.3) are used as the basis for a representation of the pulse characteristics of near-field ground motions. These pulses are fully defined by a pulse shape and two parameters, i.e., the pulse period $T_p$ and a pulse intensity parameter, which can be either the maximum pulse acceleration $a_{g,\text{max}}$ or the maximum pulse velocity, $v_{g,\text{max}}$.

**One-directional half-pulse, P1.** In this pulse the ground experiences a non-reversing displacement history that is generated through a single cycle of acceleration input. In the basic pulse P1 the acceleration input is represented by a single square wave, which results in a triangular velocity half-cycle and a second-order one-directional displacement history. The peak ground acceleration, PGA or $a_{g,\text{max}}$, the peak ground velocity, PGV or $v_{g,\text{max}}$, and the peak ground displacement, PGD or $u_{g,\text{max}}$, are related as follows:

\[
v_{g,\text{max}} = \frac{a_{g,\text{max}} T_p}{4}
\]

(5.1)
Two-directional pulse, P2. In this pulse the ground experiences a reversing displacement history that is generated through a double cycle of acceleration input. In the basic pulse P2 the acceleration input is represented by the square wave shown in Fig. 5.2, which results in a triangular velocity cycle and a second-order reversing displacement history. The peak ground velocity, \( v_{g,max} \), and the peak ground displacement, \( u_{g,max} \), are also given by Eqs. 5.1 and 5.2.

Multiple pulses, P3. This pulse sequence is generated by the acceleration history shown in Fig. 5.3. It is utilized to investigate the effect of repeated pulses on response parameters. The peak ground velocity, \( v_{g,max} \), is given by Eq. 5.1, but the peak ground displacement, \( u_{g,max} \), is only half of that of the previous pulses, i.e.,

\[
 u_{g,max} = \frac{a_{g,max} T_p^2}{32} 
\]  

(5.3)

In all three cases the pulse period \( T_p \) is defined as the duration of a full velocity cycle. Thus, in P1 the duration of motion is only \( T_p/2 \). The effect of P1 and P2 on the response of a continuous and elastic shear building was studied by Hall et al. (1995). In this study the pulses will be used for a comprehensive evaluation of the elastic and inelastic response of SDOF and MDOF systems.

5.1.1. Elastic Response Spectra

Elastic strength (acceleration), velocity, and displacement demand spectra for the three basic pulses P1, P2, and P3 are presented in Fig. 5.4. Each graph shows the spectra for all three pulses. The period axis is normalized by the pulse period \( T_p \), and the spectral ordinates are normalized by the corresponding ground motion peak value. All spectra are computed for 2% damping.

The elastic strength demand (acceleration response) spectrum for P1 exhibits closely spaced peaks and valleys for small \( T/T_p \) ratios and attains a maximum dynamic amplification factor of 3.75 at \( T/T_p = 0.5 \), i.e., at a period equal to the duration of the one-directional half-pulse. The spectral acceleration values for P2 are equal to those of P1 at periods for which the maximum response occurs at times \( \leq T_p/2 \), whereas they exceed the P1 values at all periods for which the maximum response occurs at times \( > T_p/2 \). The maximum dynamic amplification factor is
attained at $T/T_p = 0.75$ and is equal to 4.7. The largest dynamic amplification factors are obtained for P3, and they occur at periods for which maximum response is attained during the second velocity cycle. The harmonic nature of P3 leads to very large dynamic amplification factors around $T/T_p = 1.0$.

The damaging nature of P3 is evident also from the velocity and displacement spectra. Considering that for a given period the expected damage is likely proportional to displacement, the presented displacement spectra provide a means to rank the damage potential of the three pulses. When doing this, it must be considered that the normalizing peak ground displacement $u_{g,max}$ is equal for P1 and P2, but is only half as large for P3. Thus, if equal ground acceleration or velocity is used as a basis for comparison of pulse effects, the normalized displacement spectral values for P3 should be divided by a factor of 2.

It is worth noting that the displacement spectra for P1 and P2 do not show a clear spectral peak. Only for P3, which consists of two displacement cycles, the duration of motion is long enough to generate high dynamic amplification around $T/T_p = 1.0$.

The general conclusion to be drawn from the pulse response spectra is that the structural response is sensitive to the pulse shape and the relative magnitude of structure to pulse period, $T/T_p$. There are definite patterns in the spectra, but these patterns are not simple and need to be inspected carefully. Moreover, the elastic spectra are inadequate to assess damage potential and need to be supplemented by inelastic spectra and by the response evaluation of MDOF systems in which multi-mode effects are present. This is the subject of Chapter 6.

5.2. Other Pulse Shapes

A number of assumptions had to be made to define the basic pulses. In the three basic pulses discussed previously, the acceleration history is described by square waves. Moreover, each pulse consists of a certain number of velocity cycles, i.e., P1, P2, and P3 contain one, two, and five velocity half-cycles, respectively. An important issue that needs to be addressed is whether a reasonably large variety of pulse-type ground motions can be represented by these three basic pulses or more pulse shapes need to be considered.

The objective of this part of the study is to assess the response characteristics of SDOF systems subjected to pulse inputs of different shapes and properties than the three basic ones, and to evaluate the necessity of considering additional pulse shapes. For this purpose, elastic spectral
quantities of pulses with triangular acceleration histories and pulses with a different number of cycles from that of the three basic ones are evaluated. The inelastic response of MDOF structure to these alternative pulse shapes will be investigated in the next chapter.

5.2.1. Triangular Pulses

The acceleration history of the three basic pulses is described by square waves. This results in a very low ratio of PGA/PGV of $4/T_p$, and assumes that the rise time of the acceleration pulse approaches zero. Even for distinct near-field pulses these are extreme and likely unrepresentative conditions. Modified versions of the basic pulses P1 and P2 are utilized to study the effect of rise time on response parameters. These modified pulses, denoted as P1.1 (a variation of P1), and P2.1 (a variation of P2) are illustrated in Figs. 5.5 and 5.6.

If the peak parameters (PGA, PGV, and PGD) of the pulse variations are normalized by the corresponding parameter of the basic pulses and the PGV ratio is set equal to 1.0, it is observed that no difference exists in the peak ground displacements, but that in both cases the PGA of the modified pulse is twice that of the basic pulse. Thus, the PGA/PGV ratio is $8/T_p$, which is in the range of values observed in "ordinary" records from past earthquakes (Lawson, 1996) - if $T_p$ is in the order of 1 sec. or shorter.

Elastic Response Spectra:

The effect of these pulse modifications on spectral response is documented in Figs. 5.7 and 5.8. The spectra of the modified pulses are shown in heavy lines and the spectra of the corresponding basic pulses are shown in light lines. Each pulse response spectrum is normalized by the peak value of the pulse input. These peak values are shown in Figs. 5.1, 5.2, 5.5, and 5.6. Since the PGV and PGD values of both modified pulses are identical to those of the basic pulses, and the normalized velocity and displacement spectra do not differ by much, it is concluded that the pulse modifications have a relatively small effect on spectral velocity and displacement responses. This appears to be not so for spectral acceleration responses. The normalized spectral accelerations (dynamic amplification factors) for the modified pulses are much smaller than those for the basic pulses. However, it needs to be considered that the PGA values of the modified pulses are twice as high as those of the basic pulses. Once this is considered, it is observed that the difference in spectral accelerations between modified and basic pulses is not drastic at most periods.
In summary, the spectral response of the modified pulses with triangular acceleration histories can be adequately represented by their corresponding basic pulses. The effect of these triangular pulses on the response of MDOF structures is studied in the next chapter.

5.2.2. Pulse Histories with Different Duration

The period of the basic pulses (T_p) is defined as the time needed to complete a full velocity cycle. Based on this definition, the duration of basic pulse histories P1, P2, and P3 is 0.5, 1.0, and 2.5 times the pulse period, respectively. There are pulse histories with different duration whose response properties may need to be evaluated. Two additional pulses, P4 and P5, are introduced and studied here for this reason. The acceleration time histories for P4 and P5 are represented by the square waves shown in Figs. 5.9 and 5.10. The duration of these two pulse histories is 1.5T_p for P4 and 2.0T_p for P5. It should be noted that the duration of these new pulses is between those of P2 and P3. Thus, it is useful to compare the response characteristics of P4 and P5 with those of the basic pulses P2 and P3.

Elastic Response Spectra:

Elastic strength (acceleration), velocity, and displacement demand spectra of all five pulses P1 to P5 are compared in Fig. 5.11. The spectral ordinates are normalized by the corresponding time history peak values. The spectra of P4 and P5 exhibit patterns similar to those of P3 but with smaller peak values. The normalized spectral displacements for P4 and P5 are bounded by the corresponding values for P2 and P3. However, this does not hold true at all periods for normalized spectral acceleration and velocity values. Since the new spectral responses are not always close to those of basic pulses P1, P2, and P3 in the full range of T/T_p, more investigation is necessary to quantify the response characteristics of pulses P4 and P5. Inelastic demands of MDOF structures subjected to pulse shapes P4 and P5 are evaluated in the next chapter.
Figure 5.1 Pulse P1 Ground Acceleration, Velocity, and Displacement Time History
Figure 5.2 Pulse P2 Ground Acceleration, Velocity, and Displacement Time History
Figure 5.3 Pulse P3 Ground Acceleration, Velocity, and Displacement Time History
Figure 5.4 Elastic Strength, Velocity, and Displacement Demand Spectra for P1, P2, and P3
Ground Acceleration Time History
Pulses P1 and P1.1

Ground Velocity Time History
Pulses P1 and P1.1

Ground Displacement Time History
Pulses P1 and P1.1

Figure 5.5 Pulse P1.1 (Modification to Pulse P1)
Ground Acceleration Time History
Pulses P2 and P2.1

Ground Velocity Time History
Pulses P2 and P2.1

Ground Displacement Time History
Pulses P2 and P2.1

Figure 5.6 Pulse P2.1 (Modification to Pulse P2)
Elastic SDOF Strength Demands
Pulses P1 and P1.1, $\xi = 2\%$

Elastic SDOF Velocity Demands
Pulses P1 and P1.1, $\xi = 2\%$

Elastic SDOF Displacement Demands
Pulses P1 and P1.1, $\xi = 2\%$

Figure 5.7 Comparison of Strength, Velocity and Displacement Demands for P1 and P1.1
Figure 5.8 Comparison of Strength, Velocity and Displacement Demands for P2 and P2.1
Figure 5.9 Pulse P4 Ground Acceleration, Velocity and Displacement Time History
Figure 5.10 Pulse P5 Ground Acceleration, Velocity and Displacement Time History
Figure 5.11 Elastic Strength, Velocity and Displacement Demand Spectra for P1 to P5
6. RESPONSE OF STRUCTURES TO PULSE-TYPE SEISMIC INPUT

The objective of this part of the study is to evaluate the response of elastic and inelastic structures to pulse-type seismic input, and to achieve a fundamental understanding of response characteristics that can be utilized later (in Chapter 7) in the representation of near-field ground motions by equivalent pulses. The SDOF and MDOF structures introduced in Chapter 3 and the pulse-type ground motions defined in Chapter 5 are employed in the pulse response evaluations.

First, the elastic response of MDOF structures to the pulse-type input motions is investigated. Elastic base shear demands and the distribution of story shear demands over the height of the structure are evaluated. The goal is to identify the response characteristics and patterns that near-field ground motions and simple pulses have in common. Then, comprehensive response studies are carried out on inelastic SDOF and MDOF structures subjected to the basic pulses. Story ductility demands are used to evaluate the performance of inelastic MDOF systems. Inelastic roof and story drift demands are also evaluated. Finally, a sensitivity analysis is performed that utilizes a generic 3-story structure to assess the response of short-period structures to pulse-type input motions.

6.1. Elastic Response of MDOF Structures to Pulse-Type Input

The basic assumptions and procedures employed in the design of the generic 20-story structure were introduced in Section 3.2. In the pulse study, MDOF analyses are performed primarily for \( T/T_p = 0.375, 0.50, 0.75, 1.0, 1.5, 2.0, \) and \( 3.0 \), where \( T \) is the structure fundamental period and \( T_p \) represents the pulse period defined in Section 5.1.

6.1.1. Deflected Shapes of Structure

Snapshots of the deflected shapes of the generic MDOF structures subjected to the basic pulses at intervals of \( T_p/4 \) are shown in Figs. 6.1 to 6.4. The deflected shapes serve to illustrate the effect of pulse loading on the structural response at specific time steps. Figure 6.1 correspond to a structure with \( T = 0.5T_p \) subjected to \( P_1 \). It can be seen that for this short-period structure the deflected shape is similar to a first mode shape, signifying that higher-mode effects are insignificant and the dynamic behavior is controlled mostly by the first mode. The maximum roof displacement value of \( 0.55u_{g,\text{max}} \) occurs at \( t = T_p/2 \), i.e., the beginning of the free vibration phase.
Figures 6.2 to 6.4 illustrate deflected shapes for a structure with $T = 2T_p$ subjected to all three basic pulses. For all pulses the deflected shape, which is far from a straight line, clearly shows higher mode contributions and the effect of a transient wave traveling up the structure. The deflected shapes for P1 and P2 are identical until $T_p/2$, at which time the structure subjected to P1 enters the free vibration phase. The roof displacement at this instance is very close to the ground displacement, which is at its maximum. Under P1 the structure reaches its maximum roof displacement in the free vibration phase at $t = T_p$ because enough energy is stored in the structure during the half-pulse input to increase the roof displacement by 30% during the first free vibration reversal. The structure subjected to P2 also reaches its maximum roof displacement at $t = T_p$, but because of the reversal of ground displacement, the maximum roof displacement of $2.3u_{g,max}$ is significantly larger than the corresponding value for P1. The largest roof displacement of $3.9u_{g,max}$ is observed for the structure subjected to P3. It occurs at $t = 1.5T_p$, the time of the peak of the second ground displacement cycle. Thus, the harmonic nature of P3 has a larger effect on the roof displacement compared to P2, which has only one displacement pulse.

6.1.2. Maximum Elastic Base Shear Force

Today's design procedures are based on specification of a base shear and its distribution as story shear forces over the height of the structure. The base shear is usually evaluated from the first mode spectral strength demand, with modifications applied to account for higher-mode effects. Thus, an evaluation of base shear demands and their relation to the first mode spectral values contributes to the understanding of pulse response.

The maximum elastic base shear of the generic structures with different periods is presented in Fig. 6.5. The base shear is normalized by $m.a_{g,max}$, so that the results are comparable with SDOF strength demands. This comparison is made in Fig. 6.6, which illustrates the ratio of MDOF base shear to SDOF strength demand at the first mode period. As can be seen the ratio is sensitive to the pulse type and $T/T_p$, and may easily exceed 1.0 for larger $T/T_p$ ratios, signifying large higher-mode effects. In general, the variation of the maximum elastic base shear for each pulse has the same pattern as its corresponding strength demand spectrum. The base shear demand is highest at relatively small $T/T_p$ ratios. The maximum base shear value for P3 occurs at $T/T_p = 1.0$, which can be attributed to the harmonic nature of this pulse. A similar observation has been made by Rahnama and Krawinkler (1993) in their study on response to soft soil motions, which have characteristics similar to those of P3.
Study on Continuous Shear Building with Uniform Properties:

Hall et al. (1995) studied the effects of pulses P1 and P2 on an undamped continuous shear building with uniform shear stiffness over the height, based on wave propagation theory. They presented their results for specific T/T_p values in terms of shear strain at the base of the building. In this study their approach is generalized to include also pulse P3, and the maximum base shear is calculated for the range of T/T_p from 0 to 3.0. The results for base shear demands obtained from a closed form solution are presented in Fig. 6.7, along with the time history of the base shear force for P2 and T/T_p = 0.75 in Fig. 6.8. A relatively consistent correlation is observed between the results of the continuous shear building and those of the generic structure. The base shear of the continuous shear building has in general larger values compared to the generic 20-story structure. In part, this can be attributed to the fact that no damping is considered in the shear building model. However, it is believed that the continuous shear building attracts base shears closer to the corresponding spectral values than the generic 20-story structure even without damping. The reason is that the shear building has uniform stiffness over the height.

6.1.3. Distribution of Elastic Story Shear Over Height

Figures 6.9 to 6.11 compare the SRSS story shear pattern, used in the design of the structures, with story shear force patterns obtained from (a) time history analyses and (b) SRSS modal combinations using the corresponding pulse spectrum for pulses P1 to P3 and various T/T_p ratios. For structures with T/T_p ≤ 1.0 the dynamic shear force patterns are relatively smooth, but for structures with T/T_p > 1.0 the patterns show a clear effect of a wave traveling up the structure. This effect is evident for P1, more evident for P2, and most evident for P3. In the last case the maximum elastic story shear force for T/T_p = 2.0 occurs about 2/3rd up the height of the structure rather than at the base. Thus, for long-period structures (T/T_p > 1.0) designed according to a standard SRSS story shear strength distribution, early yielding has to be expected in the upper stories.

The results also indicate that for MDOF systems with T/T_p > 1.0, the SRSS modal combination is not a good substitute for the dynamic time history analysis. This observation provides an indication that spectral analysis may not capture all important response characteristics of pulse-type ground motions, once higher-mode effects become important.

These results confirm observations made in Section 4.1.2 for near-field ground motions, and indicate that for pulse-type ground motions and structures whose fundamental period is larger
than the effective pulse period, the design shear force distribution over the height may need to be modified compared to presently assumed design patterns (triangular, parabolic, or SRSS). However, it should be considered that this conclusion applies only to elastic or nearly elastic structures, and at lower performance levels (highly inelastic systems) the distributions of demands over the height of the structure may change significantly. The issue of design shear force patterns is pursued in Chapter 9.

6.1.4. Maximum Elastic Roof Displacement

Seneviratna and Krawinkler (1997) have performed statistical correlation studies between the roof displacement of elastic MDOF structures and the spectral displacement of the first mode SDOF systems. Their conclusion was that - for the ordinary ground motions used in their study - there is a strong correlation with a small scatter between these two quantities. For all periods, and for frames as well as wall systems, the mean roof displacement is very close to and usually slightly larger than the spectral displacement multiplied by the first mode participation factor, PF. This simply means that roof displacement is dominated by first mode vibrations.

It turns out that the same conclusion cannot be drawn for structures subjected to pulse-type excitations. This is illustrated in Fig. 6.12, which shows plots of the ratio of elastic MDOF roof displacement to the first mode spectral displacement, \( \delta_{\text{roof,max}}/S_d \), for different period ratios \( T/T_p \) and pulses. Each graph shows two curves, one for MDOF systems in which P-delta effects are neglected, and one in which they are included. The ratio is close to the value of PF for \( T/T_p \leq 1.0 \), and consistently exceeds this value by a significant amount for large \( T/T_p \) ratios. These observations are in agreement with the results of the study on near-field records summarized in Section 4.1.3. This demonstrates that for pulse-type and near-field input motions and large \( T/T_p \) ratios, higher-mode effects play a larger role than for ordinary ground motions.

6.2. Ductility Demands for Inelastic Structures

The properties of the SDOF and MDOF structures used in the inelastic response evaluations were summarized in Chapter 3. In the pulse study a strength parameter \( \eta \) is utilized to define yield strength values. For MDOF systems the base shear strength coefficient \( \eta \) is defined as:

\[
\eta = \frac{V_y}{m \frac{g_{\text{p, max}}}{\text{max}}}
\]

(6.1)
where \( V_y \) = base shear yield strength of MDOF system
\( m \) = total seismically effective mass of system
\( a_{g,\text{max}} \) = maximum acceleration of input pulse

The yield strength of SDOF systems is defined using the same coefficient, \( \eta \), but substituting the SDOF yield strength, \( F_y \), for the base shear strength, \( V_y \), in Eq. 6.1.

Results obtained from nonlinear time history analyses are presented mostly in terms of \( \eta-\mu \) (strength-ductility) diagrams, and for MDOF systems with a given strength parameter \( \eta \) by means of plots that show the distribution of story ductility demands over the height of the structure.

### 6.2.1. SDOF Systems

**Displacement Time History:**

Normalized displacement response time histories of SDOF systems subjected to typical near-field ground motions were presented in Fig. 4.5. It was shown that the near-field response has clear pulse-type characteristics. The issue to be addressed is whether the basic pulses also exhibit similar response characteristics. It is expected that pulse-type ground motions will cause a response that is characterized by either a single large excursion in one direction or by a single large cycle with comparable positive and negative excursions. The question is how the period ratio \( T/T_p \) and pulse type affect this response behavior.

Typical response time histories of inelastic SDOF systems (for a pre-defined maximum ductility of 8) subjected to the three basic pulses are illustrated in Fig. 6.13 for periods of \( T/T_p = 0.5 \) and \( 1.0 \). The responses differ significantly between the two selected periods and among the three pulse types. For systems with \( T/T_p = 0.5 \), \( P1 \) causes a one-sided response with a full displacement reversal, which results in a very small residual displacement. Much of the inelastic displacement reversal occurs during the free vibration phase after a time \( T_p/2 \). The response to \( P2 \) reaches a maximum during the second half of the pulse, in the direction opposite to the maximum response to \( P1 \), and a significant residual displacement is evident. The response to \( P3 \) exhibits two cycles with large inelastic displacements, but very little residual displacement. For systems with \( T/T_p = 1.0 \) the response to all three pulses results in significant residual displacements. \( P1 \) and \( P2 \) cause only one cycle of large inelastic response, whereas \( P3 \) generates two large inelastic cycles.
Constant Ductility Strength Demand Spectra:

The inelastic strength demand spectra for the three basic pulses are shown in Fig. 6.14. Spectra are presented for predefined ductility ratios of $\mu = 1$ (elastic), 2, 3, 4, 6, and 8. The spectral ordinates are defined in terms of the strength coefficient $\eta$ defined previously.

The strength demand spectra follow expected patterns insofar that the inelastic spectra for larger $\mu$ values become much smoother than the elastic ones. This implies that the effects of the large peaks and valleys in the short period range are smoothed because of the period elongation of the inelastic systems. These patterns are also observed in the inelastic strength demand spectra of near-field ground motions as well as ordinary ground motions.

Ductility Demands for Specified Strength:

Figures 6.15 to 6.17 presents $\eta-\mu$ (strength vs. ductility demand) diagrams for selected values of $T/T_p = 0.375, 0.50, 0.75, 1.0, 1.5, 2.0,$ and $3.0$ for pulses P1 to P3. These graphs illustrate the ductility demand of an SDOF system as a function of its yield strength, for given $T/T_p$ values. The results exhibit a similar pattern for all three pulses, and indicate a relationship of the type $\mu = a/\eta$, which corresponds to a linear increase in ductility with the inverse of strength.

A clearer picture of this relationship can be obtained by plotting the inverse of $\eta$, rather than $\eta$, on the vertical axis. The inverse of $\eta$ has a linear relationship with $a_{g,\max}$, which is a measure of the intensity of the ground motion, i.e., $1/\eta = (W/F_y) a_{g,\max}/g$. This equation shows that weakening the structure is equivalent to intensifying the ground motion. A typical graph of $1/\eta$ versus $\mu$, for pulse P2, is shown in Fig. 6.18. The type of analysis this diagram represents is called Incremental Dynamic Analysis (IDA).

The results obtained for the three pulses for specific $T/T_p$ values can be directly compared because the peak ground acceleration, $a_{g,\max}$ is the same. A one-to-one comparison of the ductility demands for the three basic pulses is shown in Fig. 6.19, which shows $\eta-\mu$ and $1/\eta-\mu$ graphs for $T/T_p = 1.0$ and $3.0$. The results show significant pulse-type dependence at $T/T_p = 1.0$, whereas the demands at $T/T_p = 3.0$ are almost independent of the pulse type. A general comparison of the results for different pulses indicates that for a given strength level and $T/T_p$ ratio, P2 always causes a larger or at least equal ductility demand compared to P1. The reason is that pulses P1 and P2 have identical ground time histories up to $t = T_p/2$, and P1 comes to rest.
thereafter. Likewise, ductility demands for P3 are always larger than or equal to those for P1. However, this inequality does not always hold true between P2 and P3.

6.2.2. MDOF Systems

Story Ductility Demands Over Height:

As shown in Section 4.2.2, the inelastic response of MDOF systems to near-field ground motions has special and peculiar characteristics, which set these motions apart from ordinary ground motions. The results presented in Section 6.1 for elastic systems also provide evidence that the response of structures with $T > T_p$ to pulse-type ground motions is characterized by a traveling wave effect. The objective of this section is to evaluate the inelastic response of MDOF structures to the basic pulse inputs, and to identify the similarities between the MDOF response to pulse-type and near-field ground motions. Inelastic response is described here by the story ductility ratio, $\mu_i$, defined as $\mu_i = \delta_{\text{max},i}/\delta_{\text{y},i}$, where $\delta_{\text{max},i}$ is the story drift demand, and $\delta_{\text{y},i}$ denotes the story yield drift.

Distributions of story ductility demands over the height of the structure, neglecting P-delta effects, are illustrated in Figs. 6.20 to 6.22 for pulses P1 to P3 and $T/T_p$ values of 1.0 and 2.0. Each graph clearly shows the variation of the ductility distribution with the structure strength coefficient, $\eta$. Ductility ratios, $\mu$, less than unity imply that the story shear force demand is smaller than the provided shear strength in the story.

Since the strength of the MDOF systems is tuned to a story shear strength pattern corresponding to the SRSS modal combination for a $1/T$-type acceleration spectrum ("SRSS Pattern" in Figs. 6.9 to 6.11), the dynamic story shear force patterns shown in Figs. 6.9(a) to 6.11(a) indicate that for $T/T_p > 1.0$ yielding will start in upper stories. Accordingly, Figs. 6.20(b) to 6.22(b) show that the ductility demands are highest at these locations for relatively strong structures (large $\eta$ values). However, for weak structures (small $\eta$ values) a clear migration occurs of maximum ductility demands to the bottom of the structure. As the structure strength is reduced, the maximum ductility demands in the top portion of the structure seem to stabilize, whereas in the lower stories the ductility demands grow rapidly.

The consistency of this phenomenon is surprising and deserves much scrutiny because it is one of the fundamental MDOF response characteristics of pulse-type and near-field ground motions. Clearly, it has to do with the traveling wave effect that occurs primarily for $T/T_p > 1.0$. At $T/T_p$
=1.0 this phenomenon starts to occur but is not yet very pronounced (see Figs. 6.20(a) to 6.22(a)).

Figure 6.23 compares the variation of story ductility demands with $1/\eta$ for the first and 15th stories, using P2 and $T/T_p = 1.0$ and 2.0. As pointed out previously, an increase in $1/\eta$ implies a decrease in strength or an increase in pulse severity, represented by $a_{g,\text{max}}$. Figure 6.23(b) indicates that for a structure with a fundamental period twice the pulse period, the maximum ductility demands occur in the top portion of the structure when the input motion is not severe (or the structure is strong). However, as the severity of the input pulse increases (or the structure becomes weaker) the ductility demand in this portion stabilizes around 3.0 and even becomes smaller, whereas the first story ductility continues to grow rapidly, so that beyond $1/\eta = 4.0$ the ductility demands at the bottom are larger. On the other hand, the distribution of story ductility demands for strong structures with $T/T_p = 1.0$ is more uniform than that for $T/T_p = 2.0$, and the premature yielding of the upper stories does not occur in this case (Fig. 6.23(a)).

Figure 6.23(b) also demonstrates that the stabilization of the ductility demands in the upper stories is not necessarily permanent; at low strength levels the ductility in the 15th story starts to grow again. It is important to take note of the constant slope of the $1/\eta$-$\mu$ line for the first story, for $\mu > 3$, and to note that the slope of this line is smaller than 1.0, which indicates that the rate of increase in ductility is larger than the rate of decrease in strength.

**Maximum Story Ductility Demands:**

A comprehensive picture of the maximum story ductility demand, which can occur in any story, can be obtained from the $\eta$-$\mu_{\text{max}}$ diagrams illustrated for various $T/T_p$ ratios in Figs. 2.24 to 2.26. These plots are presented in the same manner as the SDOF demands shown in Figs. 6.15 to 6.17. In each figure the top diagram is for structures in which P-delta effects are neglected, and the bottom diagram corresponds to structures in which P-delta effects are included.

Many curves, particularly those for $T/T_p > 1.0$, have a close to vertical range around a ductility of about 3 to 4, which is the range of migration of the maximum ductility demand from upper stories to the bottom story. For instance, for pulse P2 and $T/T_p = 1.5$ to 3.0, there is a range in which $\eta$ (strength) can be reduced by half without leading to an increase in the maximum ductility demands. A clearer perspective of this phenomenon may be obtained from Fig. 6.27, which shows $1/\eta$-$\mu_{\text{max}}$ plots for P2, using systems without and with P-delta effects. The range of stabilization of ductility demands is clearly evident, as is the linear relationship between $1/\eta$ and
μ_max, once the maximum ductility demand has migrated to the first story. Again, the slope of these lines is less than unity for T/T_p between 1.5 and 3.0.

P-delta effects can be assessed by comparing part (a) and part (b) of Figs. 6.24 to 6.26. The results indicate a consistent pattern for T/T_p > 1.0. As long as the MDOF system is strong enough to prevent migration of maximum ductility demands to the first story, the inclusion of P-delta effects does not make a significant difference. However, for weaker structures, in which the maximum ductility demand occurs in the first story, the effect of P-delta suddenly gains much on importance and may lead to significant amplification of maximum ductility demands. This is a very good reason to provide structures with sufficient strength to prevent migration of maximum ductility demands to the first story. Figure 6.28 permits a direct assessment of P-delta effects for P2 and various T/T_p values. It is noted that the P-delta effect is by far largest for T/T_p = 0.75, for which maximum ductility demands are always highest in the lower stories.

The effect of the pulse type on the maximum ductility demand is illustrated in Fig. 6.29, in which the demands for the three basic pulses are compared for T/T_p = 1.0 and 3.0. For T/T_p = 3.0 there are considerable differences in the demands for the range of strength in which the maximum story ductilities are in the upper stories, but the differences become small once the maximum demands have migrated to the bottom of the structure.

The statement made previously about stabilization of ductility demands has to be put in perspective. The results presented here are obtained for structures whose relative story shear strengths follow the SRSS shear force distribution for a 1/T-type design spectrum. Thus, the results are useful for performance evaluation of structures designed according to present practice. This is not to say that a strength design according to this shear force distribution is desirable. Desirable shear strength distributions are discussed in Chapter 9. Also, the results presented here for pulse-type inputs are relevant only if actual near-field ground motions can be represented by equivalent pulses. This issue is the main focus of Chapter 7.

**Base Shear Strength Demands for Target Ductility:**

The η-μ_max diagrams presented earlier provide comprehensive information that can be used to assess the response of MDOF structures to pulse-type ground motions. However, in design it would be more useful to rearrange this information in order to evaluate the base shear strength, η, required to limit the maximum story ductility demand, μ_max, to specific target values. Vertical cuts through the η-μ_max diagrams shown in Figs. 6.24(a) to 6.26(a), and using a linear
interpolation scheme, provide values for the MDOF strength demands. This representation is analogous to the SDOF strength demand spectra for constant ductility, presented in Fig. 6.14. Figure 6.30 illustrates the MDOF base shear strength demand spectra of the basic pulses for target maximum story ductility of \( \mu_{\text{max}} = 1, 2, 3, 4, 6, \) and 8. Inherent in these spectra is the assumption of a story shear strength distribution over the height according to the SRSS story shear force pattern. These spectra are very useful for near-field design because they provide the required strength for various targeted ductility levels, provided that the basic pulses introduced in this study can represent near-field ground motions.

In design it is often attempted to use SDOF strength and displacement demands to deduce corresponding MDOF demands. If the ordinates of the MDOF strength spectra, presented in Fig. 6.30, are divided by the corresponding values of the SDOF strength spectra, presented in Fig. 6.14, for the same ductility and period values, the ratio will quantify the MDOF/SDOF strength demand relationships for the pulse-type ground motions. These strength demand ratios are illustrated in Fig. 6.31 for the three basic pulses. This figure, which provides a comprehensive comparison of MDOF and SDOF strength demands, verifies many past observations. For instance, in the period range of \( T/T_p \leq 1.0 \), the ratio does not differ much from 1.0 for all target ductility ratios and input pulses, which indicates that in short-period structures the effect of higher modes is insignificant. It should be noted that in the short-period range the migration of maximum ductilities from the upper stories to the bottom does not take place and, therefore, the response of MDOF and SDOF structures to the pulse-type motions is comparable.

On the other hand, in the long period range (\( T/T_p > 1.0 \)) the strength ratio can grow rapidly and reach high values, implying that SDOF systems may greatly underestimate the base shear strength demand of MDOF structures. For a structures with a fundamental period \( T = 3T_p \) subjected to P2 or P3, the base shear strength required to limit the maximum ductility to a low value can be as high as five times the required strength for the corresponding SDOF system. However, the MDOF/SDOF strength ratio for the same structure subjected to P1 does not exceed 2.8.

The MDOF base shear strength demand spectra can also be utilized to quantify P-delta effects. The spectra illustrated in Fig. 6.30 are obtained from the \( \eta-\mu_{\text{max}} \) diagrams that neglect P-delta effects. Similar spectra can be produced from the \( \eta-\mu_{\text{max}} \) diagrams in which P-delta effects are included (Figs. 6.24(b) to 6.26(b)). Then, the ratios of the MDOF spectral values with P-delta effects to the spectral values without P-delta effects can be used to evaluate the effects of P-delta
on base shear strength demands. These ratios versus $T/T_p$ values are illustrated in Fig. 6.32 for different targeted maximum story ductilities.

It is not surprising that P-delta effects cause larger amplifications at larger ductility values. The general observation is that the amplifications are larger for short period structures. As shown in Fig. 3.4, gravity loads, which trigger P-delta effects, are large enough to cause a negative post-yield stiffness for the generic structures. In each cycle of the motion, when the displacement demands are large enough to enter this negative-stiffness region, the displacement response is amplified by a certain irreversible amount. Short-period structures usually undergo more cycles when they are subjected to ground motions, resulting in a larger cumulative amplification caused by P-delta effects. This is also the reason why P3 produces the largest P-delta amplifications of the three basic pulses. P3 consists of more cycles than the other two pulses, and for the same base shear strength, the structure that is subjected to P3 experiences more drifting (ratcheting) of displacement response. It is also important to note that P-delta amplifications are more significant when maximum ductility demands occur in the bottom story simply because the vertical gravity loads are larger at the base.

6.3. Displacement Demands for Inelastic Structures

6.3.1. Inelastic Displacement Demands for SDOF and MDOF Systems

SDOF Inelastic Displacement Demands:

Figure 6.33 illustrates the ratio of inelastic to elastic spectral displacement demands for the basic pulses. These ratios follow patterns observed for ordinary ground motions. In the short period range (small $T/T_p$), the inelastic spectral displacements are usually larger than the elastic ones, whereas the reverse is observed in the long period range. This pattern is particularly clear for P3, for which the ratio of $\delta_{\text{inelastic}}/\delta_{\text{elastic}}$ is significantly smaller than 1.0 around $T/T_p = 1.0$, passes through 1.0 at $T/T_p = 0.75$ (for all ductility ratios), and is larger than 1.0 for smaller $T/T_p$ ratios. A very similar pattern was reported by Rahnama and Krawinkler (1993) for spectra of soft soil ground motions. This is no surprise since the harmonic motion of P3 closely replicates a soft soil motion whose frequency content is dominated by a soil period $T_s$. 
MDOF Roof Inelastic Displacement Demands:

The evaluation of roof inelastic displacement demands is carried out through a two-step procedure. In the first step elastic roof displacement demands are related to spectral displacement values. Then, the effort is devoted to the assessment of a relationship between elastic and inelastic roof displacement demands. Information regarding the first part of this procedure was provided in Section 6.1.4.

The relative values of inelastic to elastic roof displacements generally follow patterns observed for inelastic to elastic spectral displacements (Fig. 6.33). This is illustrated in Fig. 6.34, which shows for different pulses the inelastic roof displacement of MDOF systems with different base shear strengths (various η) normalized by their corresponding elastic roof displacement. The inelastic roof displacements are larger than the elastic ones in the short period range (T/Tₚ < 0.75), and become much larger with decreasing T/Tₚ and η. The reverse is observed in the long period range, where the inelastic roof displacement demand is smaller than the elastic one and decreases when the structure becomes weaker. This pattern is consistently observed for all three pulses.

6.3.2. Inelastic Displacement Demands for Specific Periods

SDOF Systems:

A comprehensive picture of the displacement demands for SDOF systems subjected to P2 is presented in Fig. 6.35, which illustrates, for specific periods, the variation of the displacement demand with the strength of the system. The displacement demand is normalized by the peak ground displacement, and the strength coefficient η is used on the vertical axis. The results again indicate that for the period range of T/Tₚ > 0.75, the inelastic displacement demands are smaller than the elastic ones, and that for T/Tₚ < 0.75 this pattern is reversed. It is also observed that at a strength level corresponding to η = 0.25, the displacement demand is about 0.9uₙₚ, max regardless of the period. The displacement demands approach the peak ground displacement as the structure becomes very weak or the ground motion becomes very severe.

MDOF Systems:

Analogous to Fig. 6.35 for SDOF displacement demands, Fig. 6.36 provides comprehensive information on the elastic and inelastic roof displacement demands of MDOF structures.
subjected to P2. The graph illustrates the variation of the roof displacement demand with the structure base shear strength, \( \eta \). Roof displacements are normalized by the peak ground displacement of the pulse. It can be seen that in the short period range \( (T/T_p \leq 1.0) \) the corresponding MDOF and SDOF displacement demands are very close, whereas in the long period range \( (T/T_p > 1.0) \) the roof demands are significantly larger than the SDOF demands. This again highlights the significance of higher-mode effects for long-period structures subjected to pulse-type motions.

6.4. Investigation of Other Pulse Shapes

Pulses of different shape and duration than the three basic ones were introduced in Section 5.2 and their spectral values were evaluated. This section provides a complementary investigation, which assesses the inelastic response of MDOF structures to those alternative pulse-type motions. The ultimate objective is to determine whether the three basic pulses can also reasonably represent other pulse shapes. If so, it will not be necessary to further investigate the response characteristics of other pulse shapes.

6.4.1. Triangular Pulses

Variations of pulses P1 and P2 were introduced in Section 5.2.1 in order to investigate the effect of rise time on response parameters. These modified pulses, which have a triangular rather than square acceleration history, were denoted as P1.1 (a variation of P1) and P2.1 (a variation of P2) (Figs. 5.5 and 5.6). It was shown that the pulse modifications have a relatively small effect on spectral response. The effect of these pulse modifications on the inelastic response of MDOF structures is discussed here.

Distributions of story ductility demands over the height of the structure for the modified pulses are compared with those for the corresponding basic pulses in Figs. 6.37 and 6.38. This comparison is made for the period ratio \( T/T_p = 2.0 \). In order to study the effect of the structure strength, the top graph in each figure shows the story ductility demands for a strong structure (large \( \eta \)), while the bottom graph presents the story ductility demands for a weak system (small \( \eta \)). For all pulses the peak acceleration of the basic pulses, \( a_{g,\text{max}} \), is used to determine \( \eta \), even though the peak acceleration of the modified pulses is \( 2a_{g,\text{max}} \). This simply means that the presented results in each graph are the story ductility demands of the same structure (with the same \( V_y \)) subjected to a basic pulse and its modified version.
The results indicate that although there are some differences, the basic pulse represents the major response characteristics of the modified version with reasonable accuracy. In other words, the responses to square acceleration pulses with a peak acceleration $a_{g,\text{max}}$ are not significantly different from the responses to triangular acceleration pulses with a peak acceleration $2a_{g,\text{max}}$.

A similar comparison is presented in Figs. 6.39 and 6.40 for the maximum story ductility demands. The results exhibit a good agreement between the maximum story ductility demands of structures subjected to a basic pulse and its modified version. This good agreement is observed for the entire range of base shear strength.

On the basis of the results presented here for inelastic MDOF demands and those provided in Section 5.2.1 for spectral responses, the general conclusion is that the three basic pulses characterized by square-shaped acceleration histories are able to represent their modified versions. Therefore, there appears to be no need to further study the effect of pulse shapes with different acceleration rise times.

6.4.2. Pulse Input Motions with Different Duration

Pulse input motions of different duration from the basic pulses, i.e., P4 and P5, were introduced in Section 5.2.2 and their spectral quantities were evaluated. The elastic response does not provide sufficient evidence that the inelastic response properties of P4 and P5 can be represented by the three basic pulses. Thus, the inelastic response of MDOF structure to these pulse-type motions is investigated here.

Figures 6.41 and 6.42 present distributions of story ductility demands over the height of the structure for pulses P4 and P5 and $T/T_p$ values of 1.0 and 2.0. The distributions are presented for structures with different base shear strength values to show the effect of structure strength on the story ductility distribution patterns. A comparison between the ductility distributions shown in these figures with the distributions obtained for pulses P2 and P3 (Figs. 6.21 and 6.22) reveals that P2 and P3 can also represent the story ductility demands of structures subjected to pulses P4 and P5 with reasonable accuracy.

This conclusion is verified by a comparison of the maximum ductility demands. For pulses P4 and P5, $\eta-\mu_{\text{max}}$ diagrams for various $T/T_p$ ratios are illustrated in Figs. 6.43 and 6.44. Again, the maximum story ductility demands are fairly close to the corresponding ductility demands of structures subjected to pulses P2 and P3 (Figs. 6.25(a) and 6.26(a)). Accordingly, the maximum
story ductility demands for P4 and P5 are also adequately represented by P2 and P3. This exempts P4 and P5 from further scrutiny and indicates that the three basic pulses P1, P2, and P3 are reasonable representations of a variety of pulse-type ground motions of different shapes and duration. Therefore, later in Chapter 7, only these three pulse shapes are used as equivalent pulses to represent near-field ground motions.

6.5. Sensitivity of Inelastic Demands to Number of Stories

The generic 20-story frame model introduced in Section 3.2 has been used extensively in the MDOF response evaluations, even for short-period structures (structures with small \( T/T_p \) ratios). However, using a 20-story model to quantify the response of structures with a short fundamental period (e.g., \( T < T_p \)) may be questionable. Stiff frame structures normally have a small number of stories and therefore few degrees of freedom, which may lead to a different contribution of higher modes to the response compared to the 20-story frame. Furthermore, for a structure with a small number of stories, distributions of story stiffness and shear strength over the height are not as smooth as those for the generic 20-story structure. This could result in different response characteristics in short-period structures that may not be represented by a model whose properties vary almost continuously over the height. Results of a sensitivity analysis are summarized here that is intended to assess the effect of the number of stories on the seismic demands of short-period structures.

As shown in Chapter 7, the period of the pulse contained in near-field ground motions, \( T_p \), is usually longer than about 1.0 second. This makes the period range of \( T/T_p \leq 1.0 \) the emphasis of this part of the study because normal frame structures with a fundamental period shorter than one second are likely to be misrepresented by a 20-story frame model. A generic 3-story structure is developed for this purpose, and its response to pulse-type ground motions is compared to that of the 20-story structure.

6.5.1. Generic 3-Story Structure

The generic 3-story structure utilized in this part of the study is designed according to the same rules and assumptions as used for the 20-story structures discussed in Section 3.2. As with the 20-story model, the story stiffnesses are tuned such that a straight-line deflected shape is achieved under a load pattern that is based on the story shear forces obtained from an SRSS modal combination. Figure 6.45 compares the SRSS story shear force pattern for the 3-story
structure with the corresponding pattern for the 20-story structure. The figure indicates a relatively large lateral force at the roof level for the 3-story frame caused by higher mode effects.

Various demands of the 3-story structure subjected to P2 are evaluated and are compared with the corresponding demands of the 20-story frame in order to assess the usefulness of the results presented previously in the short period range (primarily T/T_p ≤ 1.0).

6.5.2. Ductility Demands

Story Ductility Demands Over Height:

Figures 6.46 and 6.47 illustrate the distributions of story ductility demands over the height of (a) the 3-story and (b) the 20-story structures with T/T_p = 0.375 and 0.75 subjected to P2. Each graph presents the distributions for various base shear strength coefficients η, ranging from almost elastic to highly inelastic behavior.

As expected for the short period range, the maximum story ductility in the 20-story system occurs in the bottom story even for nearly elastic structures, and no migration of maximum ductilities from the upper stories to the bottom is observed. In strong systems the story ductility demands for the 3- and 20-story systems vary almost uniformly over the height, and the demands are close for systems with the same strength coefficients. When the strength of the system is reduced, the ductility distributions in the 20-story system become highly non-uniform with rapidly growing demands in the bottom story, whereas the distributions in the 3-story structure remain more uniform with the top story experiencing slightly larger ductility demands. The general observation is that for frames with fewer stories, the distribution of story ductility demands over the height of the structure is more uniform.

Maximum Story Ductility Demands:

Figure 6.48 compares η-μ_max and 1/η-μ_max diagrams for SDOF, 3-story, and 20-story systems with T/T_p = 0.375 and 0.75 subjected to P2. For a given strength coefficient η, each curve provides the maximum story ductility demand (maximum of all stories). For strong systems (large η) close proximity is observed among the maximum story ductility demands of the three systems. When the structures become weaker, the differences between the demands of the systems increase, but the differences are not very significant in any case. It appears that unlike the distributions of story ductility over the height, the maximum story ductility demands do not
depend strongly on the number of stories. However, the location of the maximum demand is not identical for different structures; e.g., for weak systems the maximum ductility occurs in the bottom story of the 20-story structure, whereas the location of the maximum demand in the 3-story structure is the upper story.

**Base Shear Strength Demands for Target Ductility:**

Similar to the procedure followed for the 20-story structure, the base shear strength of the 3-story structure required to limit the maximum story ductility demands to specific target values can be obtained from the $\frac{\mu}{\mu_{\text{max}}}$ diagrams of the type shown in Fig. 6.48(a) by using a linear interpolation algorithm. Such base shear strength demand spectra are compared in Fig. 6.49 for the 3- and 20-story structures, and (a) small and (b) large target story ductility values. This comparison evaluates the extent to which the generic 20-story structure can be utilized to estimate the demands of stiff structures with fewer stories. Although the graphs illustrate the spectra for fundamental periods up to $T = 2T_p$, the primary range of interest for stiff structures is $T/T_p \leq 1.0$.

The results indicate small differences between the base shear strength demands of the 3- and 20-story structures for small target ductilities. At large ductilities the differences increase somewhat, with the maximum difference being at $T/T_p = 0.75$. For given period and target ductility values, the strength demands of the 3-story system are always smaller than those of the 20-story frame, with the exception of $T/T_p > 1.25$ and $\mu_{\text{max}} = 4$, which is associated with the migration phenomenon in the 20-story frame. Overall, the results imply that the generic 20-story structure can reasonably represent the base shear strength demands of stiff structures with a smaller number of stories.

**6.5.3. Inelastic Displacement Demands**

**Roof Displacements:**

Inelastic roof displacement demands can be used to investigate to what extent the generic 20-story structure represents the global response of stiff structures that have a small number of stories. Figure 6.50 illustrates the variations of inelastic roof displacement demands with base shear strength for the 3-story structure with $T/T_p$ ratios ranging from 0.375 to 2.0, subjected to P2. This diagram can be compared with Fig.6.36, which presents the inelastic roof displacement demands of the 20-story structure. This comparison indicates that in the long period range ($T/T_p$
> 1.0) the displacement demand of the 20-story structure is larger than that of the 3-story structure for given \( \eta \) values. This can be attributed to larger contributions of higher-mode effects in the 20-story frame. However, the differences are small in the short period range.

Figure 6.51 provides a direct comparison of the displacement demands of SDOF, 3-story, and 20-story systems in the short period range, i.e., \( T/T_p = 0.375 \) and 0.75. The results show acceptable differences between the demands for given base shear strength values. In particular, the displacement demands of the 3- and 20-story structures are close at all strength levels indicating that the generic 20-story frame can be utilized to estimate the roof displacement demands of stiff structures with few stories.

**Story Drift Angle:**

The story ductility demand is defined as the story drift demand normalized by the story yield drift. The focus of this part of the study is on story drift angle demands, which are defined as the story drift normalized by the height of the story. The story drift angle demands are not directly comparable between structures that have the same fundamental period and base shear strength but different heights. For example, in a simple case of two elastic SDOF structures of the same period and different height, subjected to the same ground motion, the displacement demands are expected to be identical rather than the drift angles. Thus, if the story drift angles are normalized by the roof drift angle (roof displacement divided by the structure height), the normalized demands can be directly compared between structures that do not have the same number of stories or height.

Figures 6.52 and 6.53 compare distributions of the normalized story drift angle demands (\( \theta_i/\theta_{\text{roof}} \)) over the height of the 3- and 20-story structures subjected to P2 in the short period range, i.e., \( T/T_p = 0.375 \) and 0.75. The distributions are presented for various \( \eta \) values to illustrate the effect of base shear strength. The figures show that the distributions of story drift angle demands over the height of the structure are significantly more uniform for the 3-story frame than for the 20-story frame. In other word, in the 3-story frame all stories contribute to the roof displacement almost equally, whereas in the 20-story frame the contribution of the bottom stories is much larger than that of the top stories.

It is deduced that even though the roof displacement is rather insensitive to the number of stories of the structure, the distribution of the story drift over the height is sensitive to this parameter. In structures that have a larger number of stories, the distribution tends be less uniform. The
general conclusion is that the generic 20-story structures can be used to estimate the global demands (e.g., base shear strength or roof displacement) of stiff structures that have a small number of stories. However, when story-level demands (e.g., story ductility or story drift) are asked, special consideration should be given to the number of stories of the given structure.
Figure 6.1 Elastic Deflected Shape of MDOF Structure with $T/T_p = 0.5$, Pulse P1
Deflection with Respect to Ground

$t = T_p / 4$

Deflection with Respect to Ground

$t = T_p / 2$

Deflection with Respect to Ground

$t = 3 T_p / 4$

Deflection with Respect to Ground

$t = T_p$

Deflection with Respect to Ground

$t = 5 T_p / 4$

Deflection with Respect to Ground

$t = 3 T_p / 2$

Figure 6.2 Elastic Deflected Shape of MDOF Structure with $T/T_p = 2.0$, Pulse P1
Figure 6.3 Elastic Deflected Shape of MDOF Structure with $T/T_p = 2.0$, Pulse P2
Figure 6.4 Elastic Deflected Shape of MDOF Structure with $T/T_p = 2.0$, Pulse P3
Figure 6.5 Maximum Base Shear for Elastic Structures Subjected to Pulse-Type Inputs

Figure 6.6 Ratios of Maximum Base Shear to SDOF Strength Demand for Specific Periods
Figure 6.7 Maximum Base Shear for Continuous Shear Building Subjected to P1 to P3

Figure 6.8 Base Shear Time History for Shear Building with $T/T_p = 0.75$, Pulse P2
Figure 6.9 Normalized Elastic Story Shear Demands for Pulse P1
Figure 6.10 Normalized Elastic Story Shear Demands for Pulse P2
Figure 6.11 Normalized Elastic Story Shear Demands for Pulse P3
Figure 6.12 Ratio of Elastic MDOF Displacement to First Mode Spectral Displacement for Pulses P1, P2, and P3
Figure 6.13 Response of Inelastic SDOF Systems to Pulse-Type Input Motions

(a) $T/T_p = 0.5$

(b) $T/T_p = 1.0$
Figure 6.14 Elastic and Inelastic Strength Demand Spectra for Three Basic Pulses
Figure 6.15 Ductility Demands of SDOF Systems Subjected to Pulse P1

Figure 6.16 Ductility Demands of SDOF Systems Subjected to Pulse P2
Figure 6.17 Ductility Demands of SDOF Systems Subjected to Pulse P3

Figure 6.18 1/η–μ Diagram for SDOF Systems Subjected to Pulse P2
Figure 6.19 Effect of Pulse Type on Ductility Demands
Figure 6.20 Story Ductility Demands for Pulse P1, Various Values of $\eta$
Figure 6.21 Story Ductility Demands for Pulse P2, Various Values of $\eta$
Figure 6.22 Story Ductility Demands for Pulse P3, Various Values of $\eta$

(a) $T/T_p = 1.0$

(b) $T/T_p = 2.0$

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Figure 6.23 Comparison of Story Ductility Demands in Different Stories for Pulse P2
Figure 6.24 Base Shear Strength vs. Maximum Story Ductility Demands for Pulse P1

Chapter 6  
Response of Structures to Pulse-Type ...
Maximum MDOF Story Ductility Demands
Pulse P2, SRSS Pattern, without P-Δ

(a) without P-delta Effects

Maximum MDOF Story Ductility Demands
Pulse P2, SRSS Pattern, with P-Δ

(b) with P-delta Effects

Figure 6.25 Base Shear Strength vs. Maximum Story Ductility Demands for Pulse P2
Maximum MDOF Story Ductility Demands
Pulse P3, SRSS Pattern, without P-Δ

(a) without P-delta Effects

Maximum MDOF Story Ductility Demands
Pulse P3, SRSS Pattern, with P-Δ

(b) with P-delta Effects

Figure 6.26 Base Shear Strength vs. Maximum Story Ductility Demands for Pulse P3
Maximum MDOF Story Ductility Demands
Pulse P2, SRSS Pattern, without P-Δ

(a) without P-delta Effects

Maximum MDOF Story Ductility Demands
Pulse P2, SRSS Pattern, without P-Δ

(b) with P-delta Effects

Figure 6.27 $1/\eta_{\mu_{\text{max}}}$ Diagrams for MDOF Structures Subjected to Pulse P2
Figure 6.28  Effect of P-Delta on Ductility Demands of MDOF Structures Subjected to P2

Figure 6.29  Effect of Pulse Type on Maximum Ductility Demands of MDOF Structures
Figure 6.30 MDOF Base Shear Strength Demand Spectra of Basic Pulses for Target Story Ductility Ratios from 1 to 8
Figure 6.31 Ratio of MDOF to SDOF Strength Demands for Basic Pulses
Figure 6.32 Effect of P-Delta on MDOF Base Shear Strength Demands for Basic Pulses
Figure 6.33 Ratios of Inelastic to Elastic SDOF Displacement Demands for Basic Pulses
Figure 6.34 Ratios of Inelastic to Elastic Roof Displacement Demands for Basic Pulses
Figure 6.35 Inelastic SDOF Displacement Demands for Pulse P2

Figure 6.36 Inelastic Roof Displacement Demands for Pulse P2
Figure 6.37 Comparison of Story Ductility Demands for Pulses P1 and P1.1, T/T_p = 2.0
Figure 6.38 Comparison of Story Ductility Demands for Pulses P2 and P2.1, $T/T_p = 2.0$

(a) Strong Structure, $\eta = 1.0$

(b) Weak Structure, $\eta = 0.1$
Figure 6.39 Comparison of Maximum Story Ductility Demands for Pulses P1 and P1.1

Figure 6.40 Comparison of Maximum Story Ductility Demands for Pulses P2 and P2.1
Figure 6.41 Story Ductility Demands for Pulse P4, Various Values of $\eta$
Story Ductility Demands
Pulse P5, SRSS Pattern, $T/t_p = 1.0$, without $P$.

(a) $T/t_p = 1.0$

Story Ductility Demands
Pulse P5, SRSS Pattern, $T/t_p = 2.0$, without $P$.

(b) $T/t_p = 2.0$

Figure 6.42 Story Ductility Demands for Pulse P5, Various Values of $\eta$
Figure 6.43 Base Shear Strength vs. Maximum Story Ductility Demands for Pulse P4

Figure 6.44 Base Shear Strength vs. Maximum Story Ductility Demands for Pulse P5
Figure 6.45 SRSS Story Shear Force Pattern for Generic 3- and 20-Story Structures
Figure 6.46 Comparison of Story Ductility Demands of 3- and 20-Story Structures, T/Tₚ = 0.375
Figure 6.47 Comparison of Story Ductility Demands of 3- and 20-Story Structures, $T/T_p = 0.75$
Figure 6.48 Comparison of Max. Ductility Demands of 3- and 20-Story Structures, T/T_p = 0.75

Response of Structures to Pulse-Type...
MDOF Strength Demands for Constant Ductility
Pulse P2, SRSS Pattern, $\mu = 1, 2, 3, \xi = 2\%$, without P.Δ

(a) $\mu_{\text{max}} = 1, 2, \text{and } 3$

MDOF Strength Demands for Constant Ductility
Pulse P2, SRSS Pattern, $\mu = 4, 6, 8, \xi = 2\%$, without P.Δ

(b) $\mu_{\text{max}} = 4, 6, \text{and } 8$

Figure 6.49 Comparison of Base Shear Strength Demands of 3- and 20-Story Structures
Figure 6.50 Inelastic Roof Displacement Demands of 3-Story Structure

Figure 6.51 Comparison of Inelastic Roof Displacement Demands of 3- and 20-Story Structures
Figure 6.52 Comparison of Normalized Story Drift Angle Demands of 3- and 20-Story Structures for $T/T_p = 0.375$
Figure 6.53 Comparison of Normalized Story Drift Angle Demands of 3- and 20-Story Structures for T/Tp = 0.75
7. REPRESENTATION OF NEAR-FIELD GROUND MOTIONS BY EQUIVALENT PULSES

In Chapters 2 and 4 it was shown that near-field ground motions come in large variations. This variety very much complicates the evaluation or prediction of structural response unless near-field ground motions can be represented by a small number of simplified motions that can reasonably replicate important near-field response characteristics. An inspection of the time history records (especially velocity and displacement) of near-field ground motions reveals their impulsive characteristics (see Appendix A). The study of similarities between the response of structures subjected to near-field records and simple pulses also provides much evidence that within certain limitations near-field records can be represented by equivalent pulses of the type introduced in Chapter 5.

However, it is not reasonable to expect that perfect equivalence can be established between near-field ground motions and simple pulses for the full range of interest. Near-field records usually contain high frequency components that have little to do with the characteristics of the long-period high-energy pulse generated by the propagation of fault rupture. As will be shown in this chapter, in the very short period range these high frequency contents make the pulse-record equivalence questionable. Furthermore, in the very long period range it is also likely that other phenomena (e.g., basin effects or recorder limitations) contaminate the record.

This chapter presents procedures that can be used to identify the parameters of the predominant pulse contained in near-field ground motions. Using these guidelines, equivalent pulses are established for the fault-normal component of the records with forward directivity introduced in Chapter 2. Then, the capability of the equivalent pulses to replicate the salient SDOF and MDOF response attributes of near-field ground motions is evaluated. Finally, equivalent pulses are established and evaluated for the rotated components of the near-field records.

7.1. Matching of Near-Field Ground Motions to Equivalent Pulses

It is necessary to determine the period range in which simple pulses can represent near-field records with reasonable confidence. As shown in Fig. 5.4, the elastic strength demand (acceleration) spectra of the basic pulses demonstrate a relatively deep valley at $T/T_p = 0.25$, where $T$ is the fundamental period of the structure and $T_p$ is the period of the pulse. The valley, however, disappears in the inelastic strength spectra on account of post-yield period elongation, and even turns into a wide peak at very large ductilities. Furthermore, there are a number of
close peaks and valleys in the elastic spectrum of the simple pulses in the period range of $T/T_p < 0.25$. These observations are incompatible with the spectra of recorded near-field ground motions (see Appendix A), indicating questionable equivalence between near-field records and simple pulses in the very short period range, i.e., $T/T_p \leq 0.25$. Hence, it is postulated that the equivalence between a near-field record and a basic pulse can be reasonably established within the range of $T/T_p$ from 0.375 to 3.0.

Even though much effort has been devoted to establishing such equivalence through a consistent procedure, it cannot be claimed that the outcome of this procedure is an equivalent pulse that can perfectly represent the near-field ground motion. For many of the records used in this study an equivalent pulse has been successfully established, but not all near-field records have become part of this success. It is important to note that the use of equivalent pulses is an approximation to a very complex problem. If this approximation proves to be reasonably effective, it can significantly simplify the process of predicting near-field demands. The extent of this approximation is evaluated in Section 7.2.

### 7.1.1. Parameters of Equivalent Pulses

The basic pulses introduced in Chapter 5, i.e., P1, P2, and P3 are utilized here as equivalent pulses for near-field ground motions. It was shown that the other pulse motions (triangular pulses and pulses with different duration) are adequately represented by the three basic pulses for practical purposes. In order to establish an equivalent pulse for a record, three parameters need to be evaluated; pulse type (P1, P2, or P3), pulse period $T_p$, and pulse severity. The peak acceleration of the square wave acceleration history, $a_{g,max}$, is used for the latter purpose (see Figs. 5.1 to 5.3). In the following discussion the $a_{g,max}$ of the equivalent pulse is referred to as the effective acceleration, $a_{eff}$. These three parameters completely characterize the equivalent pulse, and therefore its time history and response properties can be derived accordingly.

The equivalent pulse severity can also be quantified by the (effective) peak velocity because the pulse peak acceleration and velocity are related, i.e., $v_{eff} = a_{eff}T_p/4$. As shown in the next section, the effective velocity is a more stable pulse severity measure since it is usually close to the recorded peak ground velocity of the ground motion. Nevertheless, the effective acceleration is more useful for design purposes due to its direct relationship with the structure strength coefficient $\eta = V_y/(m.a_{eff})$. 

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*Representation of Near-Field Ground ...*
7.1.2. Procedure for Matching

In this study the equivalent pulse parameters are estimated using engineering rather than seismological considerations. As pointed out before, the equivalent pulse is by no means a precise representation for near-field ground motions. In many cases, inspection, common sense, and judgmental decisions need to be employed in order to arrive at a final value for the equivalent pulse parameters. Some of these decisions are validated using a sensitivity analysis.

**Pulse Type and Pulse Period:**

Mostly judgment is employed to decide on the pulse type, based on an inspection of the time history trace, and on a comparison between ground motion and pulse spectral shapes (primarily velocity and displacement spectra). An inspection of the ground displacement and velocity time histories of (fault-normal components of) the records investigated in this research reveals that none of the near-field ground motions exhibits motions of the type represented by pulse P1, which has a half velocity cycle with a permanent ground displacement. Therefore, only equivalent pulses of the type P2 (full cycle) and P3 (multi-cycle) are employed to represent the records.

Since the velocity response spectra of basic pulses P2 and P3 demonstrate a clear hump at $T/T_p = 1$ (see Fig. 5.4), the pulse period $T_p$ for a near-field record is identified from the location of a global and clear peak in the velocity response spectrum. Typical examples are illustrated in Figs. 7.1 to 7.3, which show the velocity and displacement spectra of the three basic pulses superimposed on the velocity and displacement spectra of near-field records NR94rrs, KB95kobj, and KB95tato. In these figures the pulse and ground motion spectra are compared in a normalized domain. The period of the structure is normalized by the pulse period and the spectral ordinates are normalized by corresponding time history peak values. In most cases a narrow range for $T_p$ could be identified rigorously. But in some other cases, in which the peak is not clear or there are two or more peaks (e.g., Fig. 7.3), judgment has to be employed to decide on a final value. A sensitivity analysis is used to evaluate these decisions.

**Pulse Severity:**

Various procedures were investigated to determine the effective pulse acceleration $a_{eff}$, the simplest one being the estimation of $a_{eff}$ from the elastic displacement spectra (equating pulse and ground motion spectral displacement at $T = T_p$). However, inconsistent results were
obtained when the so estimated values were used to compute ductility demands for MDOF systems. The reason is that when $a_{\text{eff}}$ is determined based on elastic spectra only, no consideration is given to inelastic MDOF response characteristics. Ultimately, a rigorous process is employed whose objective is to minimize the differences between the maximum story ductility demands obtained from the near-field ground motion and the equivalent pulse. In summary this procedure includes the following steps:

1. Compute the $\eta_{i\mu_{\text{max}}}^i$ curves for the appropriate pulse type for $T/T_p = 0.375, 0.5, 0.75, 1.0, 1.5, 2.0,$ and $3.0$.
2. Compute the $\gamma_{i\mu_{\text{max}}}^i$ curves for the near-field record for $T/T_p = 0.375, 0.5, 0.75, 1.0, 1.5, 2.0,$ and $3.0$.
3. For each $T/T_p$ value, convert the $\eta_{i\mu_{\text{max}}}^i$ curve of the pulse into a $\gamma_{i\mu_{\text{max}}}^i$ curve [$\gamma = (a_{\text{eff}} / g)\eta$] and find best-fit values for $a_{\text{eff}}$ by minimizing the relative differences between the two $\gamma_{i\mu_{\text{max}}}^i$ curves. The minimization technique is explained in more detail next.
4. Obtain final values for $a_{\text{eff}}$ by averaging the best-fit values for the seven period ratios.

The relative difference between the $\gamma_{i\mu_{\text{max}}}^i$ curves of the pulse and near-field record at a given ductility value of $\mu_i$ is defined as:

$$e_i = \left| \frac{\gamma_i^E - \gamma_i^P}{\gamma_i^E} \right| = \left| 1 - \frac{\gamma_i^P}{\gamma_i^E} \right| = \left| 1 - \left( \frac{a_{\text{eff}}}{g} \right) \eta_i^P \right|$$

(7.1)

where $\gamma_i^E$ and $\gamma_i^P$ represent the strength coefficient corresponding to the ground motion and the pulse, respectively, at $\mu_i$. The difference (error) values $e_i$ are calculated for discrete ductility ratios in the range of interest. Then, using the least-squares method, the best-fit value for $a_{\text{eff}}$ is evaluated such that the differences between the two curves are minimized, i.e., $\Sigma(e_i)^2$ is minimum. But, minimizing the sum of the squared errors is the same as minimizing the mean of the squared errors, which is equal to

$$\text{Mean}[(e_i)^2] = \text{Var}[e_i] + (\text{Mean}[e_i])^2 = (\text{Mean}[e_i])^2(1+V_e^2)$$

(7.2)

where $\text{Var}[e_i]$ is the variance of the error and $V_e$ represents the coefficient of variation of the error in the range in which the difference between the two curves is minimized. This is again equivalent to minimizing the square root of the quantity shown in Eq. 7.2, which can be written as:
This shows that by using the least-squares method, the quantity given by Eq. 7.3, which is a combination of the mean error and the error dispersion, is minimized. This quantity can be used as a good measure to assess the final proximity of the two $\gamma$-$\mu_{\text{max}}$ curves resulting from the error minimization procedure. Smaller values for this quantity imply a closer match between the two curves.

Through the procedure described above, values for $a_{\text{eff}}$ can be obtained that minimize the differences between the $\gamma$-$\mu_{\text{max}}$ curves corresponding to the pulse and record in a given range of ductility, $\mu_{\text{max}}$. In this study the following ranges of $\mu_{\text{max}}$ are investigated:

- $\mu_{\text{max}} = 1$ to 10, which covers the full range of interest.
- $\mu_{\text{max}} = 4$ to 10, which represents behavior at a low performance level.
- $\mu_{\text{max}} = 1.0$, which represents behavior at a high performance level.

The results of this procedure for the fault-normal components of the near-field ground motions introduced in Chapter 2 are presented in Table 7.1. For each ductility range and $T/T_p$ ratio, the best-fit value for $a_{\text{eff}}$ is shown as well as the quantity given by Eq. 7.3, the mean error, and the error coefficient of variation. The results are tabulated only for the near-field records with forward directivity. The Landers (Lucerne) record (LN921ucr) is omitted from Table 7.1 because it has a pulse period of greater than 4 seconds, which may be contaminated by instrument errors. Besides, the matching procedure for this record would require computing the demands of structures with unreasonably long periods (e.g., $T = 3T_p > 12.0$ sec.).

As explained in the last step of the procedure, the final value for $a_{\text{eff}}$ is achieved by averaging the best-fit values for the seven period ratios. Figure 7.4 illustrates the variations of the $a_{\text{eff}}$ values obtained for the records using different $T/T_p$ values and the full period range of interest (1 to 10). For each record the $a_{\text{eff}}$ values are normalized by their average. To make the picture more readable, the results are presented in two separate graphs, each corresponding to 7 near-field records. As can be seen, the $a_{\text{eff}}$ values obtained from the matching procedure are not very sensitive to $T/T_p$. This justifies averaging the values in order to arrive at a single pulse severity measure for a given near-field ground motion.
The results presented in Table 7.1 are summarized in Table 7.2 by averaging the $a_{\text{eff}}$ values obtained for various $T/T_p$ ratios. The table lists pulse type and period, as well as effective acceleration and peak velocity of the equivalent pulse for the three ductility ranges. As expected, the values of $a_{\text{eff}}$ differ somewhat but not by a large amount among the three ductility ranges. Even though the effective acceleration is used as the basic pulse parameter, the more relevant parameter for structural response evaluation is the peak velocity ($v_{\text{eff}} = a_{\text{eff}} T_p / 4$). The results indicate that in most of the cases the peak velocity of the equivalent pulse is within 20% of the peak ground velocity (PGV) of the near-field record (Table 7.2). Thus, it may be feasible to use the peak ground velocity of the near-field record, $v_{g,\text{max}}$, to estimate the pulse severity parameter (i.e., $a_{\text{eff}} = 4v_{g,\text{max}}/T_p$) rather than employing the complex procedure outlined here.

### 7.2. Evaluation of Equivalent Pulses

In the previous section the parameters of equivalent pulses were identified for the near-field records investigated in this study. An important issue that needs to be addressed is that to what extent this representation is reasonable, practical, and reliable. It should be considered that simple pulse representation of complex near-field records is not expected to be precise because near-field ground motions are affected by so many complicated seismological phenomena and, therefore, come in very large variations. The objective is not to develop pulses that can accurately replicate recorded ground motions, but to develop pulses that can reasonably simulate predominant response characteristics of structures located in the near-field region of a fault rupture.

In this section demands of structures subjected to near-field records are compared to the corresponding demands obtained from the equivalent pulses of those records. The equivalent pulse parameters identified for the ductility range from 1 to 10 are utilized for this purpose. The main goal is to evaluate the quality of the pulse-record equivalence established previously.

#### 7.2.1. Comparison of SDOF Response Time Histories

In this section elastic and inelastic responses of SDOF systems subjected to the near-field record NR94rrs are compared qualitatively with the corresponding responses for Pulse P2. As summarized in Table 7.2, the near-field record NR94rrs has an equivalent pulse of type P2 with a period of $T_p = 1.0$ sec. Figure 7.5 illustrates the inelastic displacement response time history normalized by the maximum ground displacement for Pulse P2 and $T/T_p$ ratios of 1.0 and 2.0.
Each graph represents the ground displacement time history (thick line) together with the displacement response time history (thin line) of a system with a ductility of $\mu = 6$.

Figure 7.6 shows the ground and response time histories for the near-field record NR94rrs in a very similar manner. The ground displacement time history of NR94rrs looks different from the time history of P2, with the greatest difference being its two-sided displacement. However, it should be noted that the ground displacement value at the first negative peak (at $t = 2.0$ sec) is almost half the value at the following positive peak. Furthermore, the slight slope of the ground displacement time history in the time period $t \leq 2.0$ sec. implies very low ground velocity, whereas the following impulsive motion ($2.0$ sec. $< t < 3.0$ sec.) is associated with much higher ground velocity. Therefore, the positive ground displacement pulse, which has a peak at about $t = 2.7$ sec., will most likely dominate the initial negative motion, and control the response of the structure. The response time histories, superimposed on the ground time histories, support this hypothesis. It is observed that for both $T/T_p$ values, before the positive pulse strikes the structure ($t < 2.0$ sec.), the response displacement is negligible, and then rapidly increases.

An inspection of the displacement response time histories for pulse P2 (Fig. 7.5) and the record NR94rrs (Fig. 7.6) reveals that there is a clear correlation between the response to the near-field ground motion and its equivalent pulse. Since $T_p$ is 1.0 sec. for NR94rrs, the time scales of the two displacement time histories are directly comparable. There are, however, some differences, which should be expected when such simple pulse shapes are utilized. For example, for $T/T_p = 2.0$ the residual displacement for the pulse is positive, whereas the residual displacement for the record is negative. The reason is that unlike the pulse, the ground in the record does not return rapidly to its original position ($u_g = 0$) after the positive peak.

Figure 7.7 presents a comprehensive comparison of the elastic displacement response time histories for pulse P2 and near-field record NR94rrs. The elastic displacement time history of each structure is normalized by its maximum value. The time scales of the two graphs are comparable since $T_p = 1.0$ sec. for NR94rrs. To facilitate the comparison of the two sets of time histories, the time scale for the record starts at $t = 2.1$ sec., the moment at which the predominant displacement pulse of the near-field ground motion comes into play. Despite existing differences, a relatively close correlation is evident between the corresponding time histories of the two graphs at all $T/T_p$ values. A similar picture is presented in Fig. 7.8 for inelastic SDOF systems with a ductility of 6. The inelastic displacement time history of each structure is normalized by its yield displacement. The similarities between the displacement time histories for the near-field record and pulse are apparent. A closer match between the responses to the
near-field record and pulse is observed for more flexible structures (particularly elastic), which is verified by an inspection of the response spectra.

7.2.2. Comparison of Story Ductility Demands

Figure 7.9 compares the converted $\gamma$-$\mu_{\text{max}}$ curve for the equivalent pulse with the ground motion $\gamma$-$\mu_{\text{max}}$ curve for records NR94rrs and KB95tato. The averaged $a_{\text{eff}}$ value obtained in the ductility range of 1 to 10 is used to convert the pulse $\eta$-$\mu_{\text{max}}$ curve into a $\gamma$-$\mu_{\text{max}}$ curve. This comparison is made for three period ratios $T/T_p = 0.5, 1.0, \text{ and } 2.0$. In general the results show a reasonable agreement between the two $\gamma$-$\mu_{\text{max}}$ curves indicating that the equivalent pulse appears to be capable of replicating maximum story ductility demands of MDOF systems subjected to near-field records with reasonable accuracy. This compatibility is assessed more comprehensively in the next section using MDOF strength demand spectra for given story ductility ratios.

Examples of the distribution of story ductility demands over the height of the structure obtained from a near-field record and the equivalent pulse are presented in Figs. 7.10 and 7.11 for cases of small and large ductility demands. The results are presented for structures with different $T/T_p$ values, subjected to near-field records NR94rrs and KB95kobj. Although some differences exist, it is observed that the equivalent pulse captures important response characteristics of the near-field records, particularly the migration of ductility demands from the top to the bottom portion of flexible structures ($T/T_p = 2.0$).

7.2.3. Sensitivity to Pulse Type and Period

In the process of identifying equivalent pulses, judgment had to be employed in many cases to decide on the pulse type and a final value for the pulse period $T_p$. Those decisions are evaluated here using a sensitivity analysis, which assesses the quality of the pulse-record equivalence when different pulse types or different values for $T_p$ are adopted. In each case the pulse severity is computed again using the selected pulse type and $T_p$, and the matching procedure described in Section 7.1.2. The final objective is to determine which alternative for the pulse type or $T_p$ value leads to a closer match between the record and equivalent pulse.

In order to assess the quality of the match, base shear strength demands for given story ductility ratios are utilized. As shown in Sections 4.2.2 and 6.2.2, the base shear strength required to limit story ductility demands to a given value can be obtained from $\gamma$-$\mu_{\text{max}}$ (for records) or $\eta$-$\mu_{\text{max}}$ (for pulses) curves employing a linear interpolation scheme. As a result, base shear strength demand
spectra (\(\gamma-T\) curves), such as those presented in Fig. 4.18, are obtained. If the parameters of the equivalent pulse (\(T_p\) and \(a_{\text{eff}}\)) are known, the \(\gamma-T\) curves can be converted into \(\eta-T/T_p\) curves using the relation between \(\gamma\) and \(\eta\), i.e., \(\eta = \gamma/(a_{\text{eff}}/g)\). Examples of so converted \(\eta-T/T_p\) curves are presented in Fig. 7.12 for near-field records NR94rrs and KB95kobj.

These graphs are directly comparable with base shear strength demand spectra (\(\eta-T/T_p\) curves) for the basic pulses, i.e., Fig. 6.30. Thus, if the ordinates of the record \(\eta-T/T_p\) curves are divided by the corresponding values of the pulse \(\eta-T/T_p\) curves, the resulting ratios can be used to assess the capability of the equivalent pulse to estimate the base shear strength demands of a structure subjected to the near-field record for various \(T/T_p\) and story ductility values. Example of these ratios are shown in Fig. 7.13, which compares the record KB95kobj with an equivalent pulse with a \(T_p\) value of 0.9 sec. and of type (a) P2 and (b) P3. A ratio larger than 1.0 for given \(T/T_p\) and \(\mu\) values indicates that in order to limit story ductility demands to \(\mu\), more base shear strength is needed for the record than for the equivalent pulse. In other words, the equivalent pulse underestimates the required base shear strength in this particular case. A perfect match between the records and equivalent pulse is implied when the record-to-pulse strength ratio is equal to one.

Figure 7.13 compares P2 and P3 as the equivalent pulse for the near-field record KB95kobj to determine which one better represents this ground motion. As can be seen, there are differences between the near-field record and equivalent pulses of both types. While P2 may have a closer match with the record for some \(T/T_p\) and \(\mu\) values, P3 represents the record better for some other values of \(T/T_p\) and \(\mu\). However, overall P3 is a better representative for the near-field record KB95kobj. Similar to this record, many other ground motions investigated in this study exhibit large record-to-pulse strength ratios for a story ductility of 4 and \(T/T_p > 1.0\), indicating that the equivalent pulse underestimates the base shear strength demands. The reason lies in the migration phenomenon that occurs in long period structures. As the strength of the structure is reduced, the migration of high ductility demands to the bottom of the structure, which starts at a story ductility ratio between 3 to 4, occurs at a somewhat faster rate for the pulse than for the ground motion. Therefore, the story ductility of 4 at the bottom of the structure in reached at a smaller base shear strength value for the pulse. Nonetheless, this is only a local discrepancy and does not have an impact on the strength demands for other ductility values.

The next issue that needs to be addressed is the sensitivity of the equivalent pulse representation to the value chosen for \(T_p\). As discussed in Section 7.1.2, the equivalent pulse period, \(T_p\), is identified based on the location of a clear and global peak in the velocity response spectrum. But
not all near-field ground motions exhibit a single clear peak in their velocity spectra. For example, the velocity spectrum of the record NR94rrs contains two humps at periods around 1.0 and 1.35 sec. (see Fig. 2.5). In order to choose one of these two candidates for the period, Fig. 7.14 illustrates the record-to-pulse strength demand ratios for this record assuming that the value of $T_p$ is (a) 1.0, and (b) 1.35 sec. It appears that $T_p = 1.0$ sec. leads to a closer match between the equivalent pulse P2 and the near-field record NR94rrs.

7.3. Equivalent Pulse for Rotated Components

In Chapter 2 it was shown that rotated components of near-field ground motions with respect to the fault direction have spectral values nearly as large as those associated with the fault-normal component. Time history traces indicate that the rotated components have pulse-type characteristics similar to those of the fault-normal component. These pulse-type characteristics and their effects on MDOF response can be investigated more rigorously using the procedures introduced in this chapter for detection and identification of equivalent pulses. In this section attempts are made to identify equivalent pulses for the $45^\circ$ components of the near-field record NR94rrs.

Figure 2.5 displays a global peak in the velocity spectra of both $45^\circ$ components of NR94rrs, indicating that as with the fault-normal component, the rotated components also contain pulses of a period around $T_p = 1.0$ sec. Similar to the fault-normal component, the rotated components are represented by equivalent pulses of type P2. Following the procedure presented in Section 7.2.1 for estimating the equivalent pulse severity, effective pulse acceleration values of 0.58g and 0.47g are achieved for the $45^\circ$ components ($a_{\text{eff}} = 0.72g$ for the fault-normal component). In other words, the severity of the pulse contained in one of the rotated components is almost 80% of the severity of the pulse contained in the fault-normal component. This signifies that $45^\circ$ components of near-field records may still be quite severe.

Figure 7.15 presents the record-to-pulse strength demand ratios for the rotated components of NR94rrs to illustrate the quality of the match between these records and the equivalent pulses identified previously. A comparison of these results with those corresponding to the fault-normal component (Fig. 7.14(a)) reveals that the response characteristics of the rotated record with $a_{\text{eff}} = 0.58$ are very similar to those of the fault-normal component. Furthermore, this rotated component is relatively well represented by the equivalent pulse.
7.4. Estimation of Structure Response to Near-Field Ground Motions

An evaluation of structure response to near-field ground motions requires dynamic analyses that are computationally expensive. Since every near-field record has unique properties, the response results for one ground motion are not useful in evaluating the response to a different ground motion. The equivalence established in this chapter between near-field ground motions and simple pulses can be utilized in order to estimate the demands of MDOF structures with little effort. A multi-step procedure is presented in this section that can be used to estimate the maximum story ductility, roof drift, and story drift angle demands of a given frame structure subjected to a specific near-field record. The response of structures to simple pulses presented in Chapter 6 provides basic information for this procedure, which consists of the following steps:

1. Obtain the following structural properties:
   - Fundamental Period, $T$
   - Base shear strength coefficient, $\gamma = V_y/W$

2. Obtain the following ground motion properties:
   - Peak ground displacement, $u_{g,\text{max}}$
   - Peak ground velocity, $v_{g,\text{max}}$
   - Pulse type (from the time history and spectral shapes, see Section 7.1.2)
   - Pulse period, $T_p$ (from the velocity spectrum, see Section 7.1.2)
   - Pulse severity, $a_{\text{eff}} = 4v_{\text{eff}}/T_p$ (use $v_{\text{eff}} = v_{g,\text{max}}$)

3. Calculate the following quantities:
   - Period ratio, $T/T_p$
   - Pulse strength parameter, $\eta = (g/a_{\text{eff}})\gamma$

Presuming that the given near-field ground motion is best represented by pulse P2, the demands of structures subjected to P2, as presented in Chapter 6, are used to determine seismic demands as follows:

4. Using the $T/T_p$ and $\eta$ values obtained in step 3, estimate the maximum story ductility demands from Fig. 7.16. For $T/T_p$ values between those given in the graph, the maximum story ductility demand can be obtained using linear interpolation.
5. Using the T/T<sub>p</sub> and η values, and u<sub>g,max</sub> from step 2, estimate the roof displacement demand, δ<sub>roof,max</sub>, from Fig. 7.17. For T/T<sub>p</sub> values between those given in the graph, the roof displacement demand can be obtained using linear interpolation.

6. Using the T/T<sub>p</sub> and η values, estimate story drift demands, u<sub>i</sub>, as follows:
   - Calculate roof drift angle, θ<sub>roof</sub> = δ<sub>roof,max</sub>/H, where H is the structure height.
   - Obtain the drift angle ratio for the story under consideration, λ = θ<sub>i</sub>/θ<sub>roof</sub>, from Fig. 7.18. Interpolate for T/T<sub>p</sub> values between those given in the graphs.
   - Calculate u<sub>i</sub> = λ<sub>i</sub>h<sub>i</sub>/θ<sub>roof</sub>, where h<sub>i</sub> is the height of the story under consideration.

It is important to note that P-delta effects are not accounted for in the results obtained from this procedure. While maximum story ductility demands and especially roof displacement demands are not very sensitive to the number of stories, story drift demands can be significantly affected by the number of stories (Section 6.5). Therefore, for structures with a small number of stories, Fig. 7.18, which represents demands of a 20-story structure, may not provide a good estimate of the story drift demand.
Table 7.1 Properties of Equivalent Pulses for Recorded Near-Field Ground Motions

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<th>Rec/Pls</th>
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<th>( a_{\text{eff}} ) (g)</th>
<th>( \text{sqr}(L_{\mu_2}) )</th>
<th>( \mu_4 %, V_e % )</th>
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Chapter 7
Table 7.1 (Cont’d) Properties of Equivalent Pulses for Recorded Near-Field Ground Motions

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Table 7.2 Equivalent Pulses for Recorded Near-Field Ground Motions (Summary)

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Figure 7.1 Determination of Pulse Period (and Pulse Type) from Elastic Spectra, Record NR94rrs
Figure 7.2 Determination of Pulse Period (and Pulse Type) from Elastic Spectra, Record KB95kobj
Figure 7.3 Determination of Pulse Period (and Pulse Type) from Elastic Spectra, Record KB95tato
Figure 7.4 Variation of Equivalent Pulse Effective Acceleration with $T/T_p$,
Ductility Range of 1 to 10
Figure 7.5 Ground and Inelastic SDOF Displacement Time Histories for Pulse P2 and $\mu = 6$
Figure 7.6 Ground and Inelastic SDOF Displacement Time Histories for NR94rrs and $\mu = 6$

(a) $T = 1.0$ sec.

(b) $T = 2.0$ sec.
Figure 7.7 Normalized Elastic SDOF Displacement Time Histories for Various Periods
Figure 7.8 Normalized Inelastic SDOF Displacement Time Histories for Various Periods and $\mu = 6$
Maximum Story Ductility Demands
$T / T_s = 0.50, a_{en} = 0.72 g, \mu = (1,10)$, without $P - A$

Maximum Story Ductility Demands
$T / T_s = 0.50, a_{en} = 0.86 g, \mu = (1,10)$, without $P - A$

Maximum Story Ductility Demands
$T / T_s = 1.00, a_{en} = 0.72 g, \mu = (1,10)$, without $P - A$

Maximum Story Ductility Demands
$T / T_s = 1.00, a_{en} = 0.86 g, \mu = (1,10)$, without $P - A$

Maximum Story Ductility Demands
$T / T_s = 2.00, a_{en} = 0.72 g, \mu = (1,10)$, without $P - A$

Maximum Story Ductility Demands
$T / T_s = 2.00, a_{en} = 0.86 g, \mu = (1,10)$, without $P - A$

(a) NR94rrs and Pulse P2
(b) KB95kobj and Pulse P3

Figure 7.9 Matching of $\gamma - \mu_{max}$ Curves for Identification of Best Fit $a_{eff}$ for NR94rrs and KB95kobj

Chapter 7 Representation of Near-Field Ground ...
Figure 7.10 Comparison of Story Ductility Demands Obtained from Record NR94rrrs and its Equivalent Pulse
Figure 7.11 Comparison of Story Ductility Demands Obtained from Record KB95kobj and its Equivalent Pulse
Figure 7.12 Converted Base Shear Strength Demands for Target Story Ductility
MDOF Strength Demand Ratios for Constant Ductility
KB95kobj, Fault-Normal, \( T_p = 0.9 \) sec, \( a_{\text{eff}} = 0.85 \) g, \( \zeta = 2\% \), no \( P_\Delta \)

![Graph (a) Equivalent Pulse P2](image)

(b) Equivalent Pulse P3

MDOF Strength Demand Ratios for Constant Ductility
KB95kobj, Fault-Normal, \( T_p = 0.9 \) sec, \( a_{\text{eff}} = 0.86 \) g, \( \zeta = 2\% \), no \( P_\Delta \)

![Graph (b) Equivalent Pulse P3](image)

Figure 7.13 Record-to-Pulse Strength Demand Ratios for Specific Target Ductility Ratios; Different Equivalent Pulse Types, Record KB95kobj
Figure 7.14 Record-to-Pulse Strength Demand Ratios for Specific Target Ductility Ratios; Different Equivalent Pulse Periods, Record NR94rrs
MDOF Strength Demand Ratios for Constant Ductility
NR94rrs, 0.707(FN+FP), $T_p = 1.0$ sec, $a_{eff} = 0.58$ g, $\xi = 2\%$, no P-Δ

(a) FN+FP Component

MDOF Strength Demand Ratios for Constant Ductility
NR94rrs, 0.707(FN-FP), $T_p = 1.0$ sec, $a_{eff} = 0.47$ g, $\xi = 2\%$, no P-Δ

(b) FN-FP Component

Figure 7.15 Evaluation of Equivalent Pulse for Rotated Components of Near-Field Records, Record NR94rrs
Figure 7.16 Estimation of Maximum Story Ductility Demands Based on Pulse P2

Figure 7.17 Estimation of Roof Displacement Demands Based on Pulse P2
Figure 7.18 Estimation of Story Drift Demands Based on Pulse P2, Various $T/T_p$ values
8. STUDY OF MODELS OF STEEL STRUCTURES

The results presented in previous chapters will be of value only if the generic structures introduced in Section 3.2 can represent a variety of real-world MDOF frame structures. However, real multi-story frame structures may not quite respond to earthquake ground motions in the same way as the simplified generic systems. In addition to frame geometrical configuration parameters such as the number of bays and stories, span length, or story height, there are a large number of factors that differentiate the simple generic frames from real structures. The design of the generic frames is based on many simplifying assumptions that do not necessarily agree with reality in all cases. For instance, contributions from panel zones, floor slabs, and non-structural elements to the stiffness and strength of the system are ignored, and the generic design is based on particular distributions of story stiffness and strength over the height of the structure.

On the other hand, employing sophisticated structural models that can fully account for all existing elements and effects of the system is not practical in many cases. If advantage can be taken of generic structures to predict the seismic demands of real structures with reasonable accuracy, a great deal of computational time and effort can be saved. Therefore, it is important to assess the usefulness of predictions obtained from the generic structures for various seismic demand quantities. In this chapter the correlation between demands is investigated for the generic frames and more realistic structural models subjected to near-field ground motions in order to justify and validate the use of generic models in this study.

8.1. Models of Steel Structures Used in This Study

Two steel moment resisting frame (SMRF) models (3- and 9-story) are utilized, which were thoroughly analyzed in a past study carried out as part of the SAC Steel Program (Gupta, and Krawinkler, 1999). The 2-dimensional frame models, which are referred to as “LA 3-story” and “LA 9-story”, correspond to the perimeter moment resisting frames of two buildings located in Los Angeles. The geometrical configurations of these two models are illustrated in Fig. 8.1. The LA 3-story frame has four bays, while the LA 9-story frame contains five bays. Moreover, the LA 9-story structure has a basement, and is laterally restrained at the ground level (for a detailed description see Gupta, and Krawinkler, 1999).

These models are bare frames of the type M2 in the SAC project, in which the dimensions, stiffness, strength, and inelastic shear distortion of panel zones are considered. The panel zones
are modeled using a combination of standard beam-column elements and trilinear rotational springs at each joint (Gupta, and Krawinkler, 1999). The interior (gravity) frames, which include shear (simple) connections, are not modeled. However, second-order (P-delta) effects due to gravity loads on these frames are taken into account by linking a virtual column to the main frame, which carries the vertical loads tributary to the interior gravity frames.

8.2. Inelastic Static Analysis and Calibration of Generic Structures

Inelastic static (pushover) analysis is utilized here to calibrate the properties of the generic frames such that their demands can be directly compared with those of LA 3-story and LA 9-story structures. The 3-story frame introduced in Section 6.5 is selected as the generic counterpart of the LA 3-story structure, whereas the generic 20-story frame, which has been extensively studied throughout this research, is compared with LA 9-story. In addition to the SRSS lateral load pattern discussed previously, the 1994 NEHRP load pattern with \( k = 2 \) is used here in pushover analyses. In this load pattern, the lateral load applied to the \( i \)-th story of the frame is given as:

\[
F_i = \frac{w_i h_i^2}{\sum_{j=1}^{n} w_j h_j^2} V
\]

(8.1)

where \( h_i \) and \( w_i \) are the height and seismically effective weight of the \( i \)-th story, respectively, and \( V \) represents the base shear.

The global pushover plots, i.e., normalized base shear force (base shear normalized by structure seismic weight, \( V/W \)) versus roof displacement are shown in Figs. 8.2 and 8.3 for LA 3-story and LA 9-story structures. Each graph illustrates the global nonlinear behavior of the structure subjected to the SRSS and NEHRP load patterns with and without consideration of P-delta effects. The pushover curve for the LA 3-story structure exhibits a bilinear shape, which indicates that all plastic hinges develop within a relatively narrow range of displacement. On the other hand, the plastic hinge formation in the LA 9-story frame occurs over a much wider range of displacement, resulting in a smoother transition from elastic behavior to a mechanism. Furthermore, the SRSS load pattern leads to higher yield strengths compared to the NEHRP pattern.
Even though a post-yield strain-hardening ratio of 3.0% is assigned at the element level, the global strain-hardening ratio amounts to about 3.7% for the LA 3-story frame, and 4.1% for the LA 9-story frame in the absence of P-delta effects. However, once second-order effects are taken into account, the global strain-hardening ratio decreases to 0.3% for the LA 3-story frame, and to negative values for LA 9-story. The reason is that the gravity load in the 3-story structure is not large enough to cause a negative post-yield stiffness, whereas for the 9-story frame, beyond a certain displacement value, negative post-yield slopes are observed. An indication of the significance of P-delta effects on the global behavior of frame structures can be obtained from the first story “stability coefficient” defined as:

$$\theta_1 = \frac{P\Delta_1}{Vh_1}$$  \hspace{1cm} (8.2)

where P is the total vertical gravity load, \(\Delta_1\) is the elastic first story drift caused by the base shear V, and \(h_1\) denotes the height of the first story. The value of \(\theta_1\) for the LA 3-story and LA 9-story structures is 3.4% and 7.1%, respectively. These values help to explain the difference in the post-yield stiffness of the two frames.

For comparison purposes, the generic 3-story structure is tuned to have a fundamental period equal to the fundamental period of the LA 3-story structure, i.e., 0.99 sec. (without P-delta effects). Likewise, the generic 20-story structure with a fundamental period of 2.17 sec. is compared with the LA 9-story structure. Since the story strengths of the generic structures follow the SRSS load pattern, only the pushover curves that are based on the SRSS load pattern are used for strength calibration. Even though the pushover curves of the LA structures, unlike those of the generic frames, do not exhibit a distinct yield point, the base shear strength can be estimated from the intersection of the extended elastic and inelastic branches. This results in strength coefficient values \(\gamma = V_y/W = 0.35\) and 0.21 for generic 3- and 20-story structures, respectively.

In order for the response of the generic frames to be comparable with the response of the SAC structures, P-delta effects should also be compatible between the two systems. For this purpose, the amount of the vertical load acting on the generic frame is determined such that the first story stability coefficient is equal to the corresponding value for the SAC structure, i.e., \(\theta_1 = 3.4\%\) and 7.1% for the 3- and 9-story frames. However, this merely means that static P-delta effects in the first story of the SAC and generic frames are at the same level as long as the systems behave elastically. The stability coefficients in higher stories of the two structures are not necessarily
similar. Besides, once plastic hinges form, the deflected shape of the structure will deviate from the elastic deflected shape, and therefore the distribution of secondary effects over the height of the structure may change significantly (Gupta and Krawinkler, 1999). The differences between the dynamic response of the generic frames and that of the SAC steel structures are investigated in the next section.

Figures 8.4 and 8.5 compare the global pushover curve (base shear vs. roof displacement) of the calibrated generic structures with the corresponding pushover curve of the SAC structures under NEHRP and SRSS load patterns (a) ignoring P-delta effects, and (b) considering P-delta effects. A close correlation between the result of the generic and LA 3-story structures is observed, which demonstrates the similarity of their global static behavior in both elastic and inelastic ranges. However, this is not true for the correlation between the generic 20-story and LA 9-story structures in the post-yield region when P-delta effects are accounted for. The differences are even more significant for the SRSS load pattern at large displacements (see Fig. 8.5 (b)). This discrepancy can be attributed mainly to the different deflected shapes of the two structures after plastic hinges have developed. Nevertheless, if the NEHRP load pattern is used, the generic and “realistic” pushover curves are comparable for a relatively long range of displacement. This also demonstrates the sensitivity of the inelastic static behavior of multi-story frame structures to the shape of load patterns employed in the pushover analysis. The reflection of these differences in dynamic response at global and story levels is investigated in the following section.

8.3. Inelastic Dynamic Analysis

In this section the dynamic response of the SAC structures to near-field ground motions is compared with the response obtained using the generic models. The objective is to assess the degree to which the response of generic structures can be used to estimate the response of real frame structures to near-field ground motions.

8.3.1. Roof Displacement

Figure 8.6 compares roof displacement time histories of the generic and LA 3-story structures subjected to near-field records (a) LP89lex, and (b) KB95tato, considering P-delta effects. As can be seen, the correlation is very good, which is not surprising when the compatibility of the pushover curves for these two structures is considered (Fig. 8.4).
Figure 8.7 compares roof displacement time histories of the LA 9-story structure with those of the calibrated generic 20-story frame subjected to the same near-field records as used in Fig. 8.6. In the early phases the two responses are almost identical, but there are differences in the time histories once the main pulse of the ground motion drives the structures into the inelastic range. Larger differences between the response of the two structures are observed for the record LP89lex (T_p = 1.0 sec) than for KB95tato (T_p = 2.0 sec), which can be attributed to the larger T/T_p value for LP89lex (T/T_p ≈ 2.2). At large T/T_p values (larger than 1) higher modes play a more important role in dynamic response, and therefore the generic 20-story and LA 9-story structures, which have different higher-mode characteristics, tend to respond differently to the same near-field ground motion. In spite of the existing differences, it is observed that the maximum roof displacements are very close, and that the global response of the two structures is in the same phase.

In order to investigate the response differences between the SAC and generic structures at different performance (inelasticity) levels, the input ground motion (i.e., LP89lex) is scaled using an intensity multiplier. A sequence of dynamic analyses that utilizes the same record with gradually increasing intensity is called “incremental dynamic analysis” (IDA). Figure 8.8 illustrates the roof displacement demands for the generic and LA 3-story frames versus the intensity multiplier (which equals unity for the original record) (a) ignoring P-delta effects, and (b) considering P-delta effects. Figure 8.9 compares the generic 20-story and LA 9-story structures in the same manner.

The demands obtained from the generic models are very close to those obtained from the SAC models in all cases. This demonstrates that generic structures can accurately estimate the global demands (roof displacement) with much less computational effort compared to more realistic, but complicated, models. It is also worth noting that when P-delta effects are taken into account, the roof displacement demand grows more rapidly with the ground motion severity. This decrease in the slope of the IDA curve is especially evident for the LA 9-story structure (Fig. 8.9) whose post-yield stiffness is negative giving rise to secondary effects.

8.3.2. Story Drift Angle

In order to avoid complexity associated with the definition of story yield displacement, story drift angles (story drift demand divided by story height) are used here to represent story-level displacement demands. To allow a direct comparison of structures with different heights, as
discussed in Section 6.5.3, the drift angle demands are normalized by the roof drift angle (roof displacement divided by structure height).

Figure 8.10 compares the normalized drift angle demands for the calibrated generic and LA 3-story structures subjected to records LP89lex and KB95tato. The distribution of the demands over the height is almost uniform, which implies that the deflected shapes are close to a straight line. This is not surprising when it is considered that the response of the 3-story structures is mainly controlled by the first mode, and that the first mode shape is close to a straight line. The results show that the normalized drift angle demands of the generic and SAC structures are relatively close. This is an indication that the generic 3-story structure can be utilized to estimate story drift demands with reasonable accuracy.

Figure 8.11 compares normalized drift angle demands of the generic 20-story structure with those of the LA 9-story structure in the same manner. Although the general pattern of the two distributions is similar, significant differences are observed, particularly in the bottom portion of the structure. The results indicate that the distribution of story drift angle demands over the height of the structure is dependent on the number of stories.

If the input ground motion is scaled using an intensity multiplier, as in the last section, the distributions of normalized story drift angle demands over the height of the structure can be investigated at different performance levels. Figures 8.12 and 8.13 show these distributions for the generic and LA 3-story frame models, respectively, with and without taking P-delta effects into account. In both structures P-delta effects do not appear to alter the distribution significantly, and the distributions are not far from uniform, indicating that the deflected shapes are close to a straight line. It is worth noting that for the generic structure the third story drift angle demand is smaller than the first story demand at high performance levels (low severity ground motion), whereas at lower performance levels (severe ground motion) this pattern is reversed (Fig. 8.12). This “rotation” effect was also observed in the response of the generic 3-story structure to basic pulses (see Figs. 6.52 and 6.53). This effect, however, is not as clear in the response of the LA 3-story structure to the same near-field ground motion (Fig. 8.13).

Figures 8.14 and 8.15 compare normalized story drift angle distributions for the generic 20-story and LA 9-story structures subjected to the near-field record LP89lex. The distributions demonstrate the migration phenomenon that was previously identified as a major near-field response characteristic of MDOF structures with $T/T_p > 1.0$ ($T/T_p \approx 2.2$ in this case). That is, for low severity ground motions (or strong structures) maximum story drift demands occur in the
upper portion of the structure, and as the severity increases (or the strength decreases), large
story drift demands migrate to the bottom. As can be seen, P-delta effects amplify the lower
story drift demands significantly after the migration of large demands to the bottom has taken
place. It is also observed that for a given ground motion intensity level, the distribution of
normalized drift angle demands is more uniform for the LA 9-story structure than for the generic
20-story frame.

From the results presented in this section and those given in Section 6.5, it is concluded that the
distribution of normalized drift angle demands depends on the number of stories (degrees of
freedom), in addition to the $T/T_p$ ratio and structure strength. For a 3-story structure this
distribution is rather uniform at all performance levels (see Figs. 8.12 and 8.13), whereas for a
20-story structure the distribution is highly non-uniform and strongly depends on ground motion
severity (or structure strength) (see Fig. 8.14). It can be deduced that as the number of stories
grows, the distribution of story drift angle demands over the height of the structure becomes less
uniform.

In summary, the results of the study presented in this chapter indicate that generic structural
models can be utilized to estimate the global demands for MDOF structures with good accuracy.
However, quantifying the story-level demands of a frame structure may be questionable when
the number of stories of the generic model employed is significantly different from that of the
structure under consideration.
Figure 8.1 Floor Plans and Elevations of 3- and 9-Story LA Model Buildings
(Gupta and Krawinkler, 1999)
Figure 8.2 Global Pushover Curves for LA 3-Story Structure, Different Load Patterns

Figure 8.3 Global Pushover Curves for LA 9-Story Structure, Different Load Patterns
Figure 8.4 Comparison of Pushover Curves for Generic and LA 3-Story Structures
Roof Displacement vs. Normalized Base Shear
LA 9-Story and Generic 20-Story, $y = 0.21$, without P-$\Delta$

(a) without P-delta

Roof Displacement vs. Normalized Base Shear
LA 9-Story and Generic 20-Story, $y = 0.21$, with P-$\Delta$

(b) with P-delta

Figure 8.5 Comparison of Pushover Curves for Generic 20-Story and LA 9-Story Structures
Figure 8.6 Comparison of Roof Displacement Time Histories for Generic and LA 3-Story Structures
Figure 8.7 Comparison of Roof Displacement Time Histories for Generic 20-Story and LA 9-Story Structures
Figure 8.8 Comparison of Roof Displacement Demands for Generic and LA 3-Story Structures; Record LP89lex
Figure 8.9 Comparison of Roof Displacement Demands for Generic 20-Story and LA 9-Story Structures; Record LP89Iex

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Figure 8.10 Comparison of Normalized Story Drift Angle Demands for Generic and LA 3-Story Structures
Figure 8.11 Comparison of Normalized Story Drift Angle Demands for Generic 20-Story and LA 9-Story Structures
Figure 8.12 Normalized Story Drift Angle Demands for Generic 3-Story Structure; Record LP89lex
Figure 8.13 Normalized Story Drift Angle Demands for LA 3-Story Structure; Record LP89lex
Figure 8.14 Normalized Story Drift Angle Demands for Generic 20-Story Structure; Record LP89lex
Figure 8.15 Normalized Story Drift Angle Demands for LA 9-Story Structure; Record LP89lex
9. DESIGN CONSIDERATIONS FOR NEAR-FIELD GROUND MOTIONS

In the previous chapters it was shown that structures designed according to current seismic codes or guidelines might experience excessively large demands or undesirable distributions of demands over their height when subjected to near-field ground motions. The development of design guidelines was not a major objective of this project, but was investigated in related studies. Selected findings of the related studies are summarized here. Attempts are made in these studies to develop design guidelines that can provide more protection for structures located in the near-field region of a seismic source. A summary is provided here of the results of a study that relates the parameters of the equivalent pulse discussed in Chapter 7 to site/source properties (i.e., magnitude and distance) using a statistical approach. Once this relationship is established, advantage can be taken of the response of structures to simple pulse-type motions in order to develop design recommendations or to predict response to future near-field ground motions. Base shear strength demands and distributions of story shear strength over the height of the structure are briefly discussed in this chapter. A more comprehensive discussion on design issues is provided in Somerville et al. (1999).

9.1. Base Shear Strength Demands

The values of the equivalent pulse parameters for the near-field records used in this study were summarized in Table 7.2. A regression analysis can be employed to evaluate the magnitude ($M_w$) and distance ($R$) dependence of these parameters. Since relatively small sets of near-field ground motions are available for this purpose, the results of such a regression analysis should be interpreted with caution. Nevertheless, this analysis was carried out using a combination of the recorded ground motions in Table 7.2 and a set of synthetic near-field records with forward directivity (Somerville et al., 1999). Using the properties of the equivalent pulse identified for these records, and employing a formulation proposed by Somerville (Somerville, 1998), the following regression equations are obtained:

$$\log_{10} T_p = -1.76 + 0.31 M_w$$  \hspace{1cm} (9.1)

$$\log_{10} v_{\text{eff}} = -2.03 + 0.65 M_w - 0.47 \log_{10} R$$  \hspace{1cm} (9.2)

This formulation assumes that the pulse period ($T_p$) is only a function of moment magnitude ($M_w$), and the pulse effective velocity ($v_{\text{eff}}$) is a function of both $M_w$ and the shortest distance from the site to the fault ($R$). Records with $R$ values smaller than 3 km. were not used in the
derivation of Eq. 9.2 on account of the logarithmic form of this equation, which results in unreasonable values for $v_{\text{eff}}$ at small R values. Figures 9.1 and 9.2 illustrate these equations together with the data points to which the line in Eq. 9.1 and the surface in Eq. 9.2 are fitted. In Fig. 9.1 some of the circles correspond to more than one data point with identical $M_w$ and $T_p$ values. Since the ground motions used in the regression analysis come from different events with different faulting mechanisms and geology, a large scatter is observed, particularly for $T_p$. Even though the large scatter may be interpreted as a lack of confidence in predicting $T_p$ values using Eq. 9.1, it will be shown that a limited variation in $T_p$ does not have a large effect on the base shear strength demands obtained from the equivalent pulse approach. In Fig. 9.2 the solid circles identify the data points used in the regression analysis, and the empty circles represent points on the regression surface with the same R and $M_w$ values as the solid circles. A short distance between a solid and empty circle on the same vertical line indicates a close match between the regression surface and the data point.

Given the earthquake event parameters R and $M_w$, the pulse parameters $T_p$ and $a_{g,\text{max}} = \text{a}_{\text{eff}} = 4v_{\text{eff}}/T_p$ can be estimated from Eqs. 9.1 and 9.2, and the $\eta - T/T_p$ curves presented in Fig. 6.30 can be converted into $\gamma - T$ curves [$\gamma = (a_{g,\text{max}}/g)\eta$]. The $\gamma - T$ curves represent MDOF base shear strength demand spectra for specified target ductilities. Examples of such base shear strength demand spectra are presented in Figs. 9.3 and 9.4 for various combinations of magnitude and distance, using the $\eta - T/T_p$ curves for pulses P2 and P3. Superimposed on each graph is a $2/(3T)$ curve, which represents a fit to the design base shear coefficients of 239 high-rise steel buildings constructed in Japan (Nakashima et al., 1992) multiplied by an overstrength factor of 2. The graphs illustrate the magnitude and distance dependence of the base shear strength demands obtained from the equivalent pulse approach, and put these demands in perspective with the values used in present practice for steel structures. If the $2/(3T)$ curve provides a good estimate for the available base shear strength of steel structures designed in Japan, it can be deduced that in small events ($M_w = 6$) these structures will behave elastically regardless of their period and their distance from the seismic source. On the other hand, in severe events and especially within a short distance from the source, the response of the steel structures may be highly inelastic. For instance, for $M_w = 7$ and $R = 3$ km., a structure with a period of 2 seconds is expected to experience maximum story ductility demands of around 5, whereas the ductility demands are expected to be around 3 for a structures with a period of 4 seconds. In larger-magnitude events the story ductility demands of steel structures designed according to current design codes are expected to be excessively large.
The large scatter of the data shown in Fig. 9.1 provides little confidence in estimating $T_p$ as a function of magnitude. The response sensitivity to a variation in $T_p$ is illustrated in Fig. 9.5, which shows the strength demand spectra for pulse P3, target story ductility ratios of 2 and 8, $M_w = 7.0$, and $R = 3$ km. In this figure $T_p$ is varied around the value obtained from Eq. 9.1 for $M_w = 7.0$ (i.e., 2.6 sec.). As can be seen, the equivalent pulse strength demands are affected to various degrees, depending on $T$ and $\mu$, and it becomes a matter of judgment in what range the scatter in strength demands is acceptable.

9.2. Distribution of Story Strength Demands Over Height

As discussed in Chapter 3, an SRSS-based story shear strength distribution over the height has been assigned to the generic structure. Figures 4.9 and 6.20 to 6.22, among others, indicate that this strength pattern leads to large variations of ductility demands over the height of the structures subjected to near-field and pulse-type ground motions. Therefore, the standard SRSS pattern may not be the most suitable one for protection against near-field effects. An ideal story shear strength distribution would result in uniform story ductility over the height of the structure. The results of a pilot study on pulse-type ground motions (Somerville et al., 1999) indicate that story shear strength distributions for uniform ductility are strongly dependent on the target ductility ratio, particularly for structures with $T/T_p > 1.0$. This is illustrated in Fig. 9.6 for pulse P2 and different target ductility ratios. The SRSS pattern is also superimposed in this figure. The figure shows that for long-period structures ($T/T_p = 2.0$) and close to elastic behavior ($\mu = 1$ and 2), relatively high strength is required around $2/3$ up the structure to control ductility demands in the top portion, whereas for a target ductility of 3 or larger the bottom portion needs to be very strong, and the required strength decreases rapidly with height. Thus, vastly different strength patterns are obtained as a function of the target ductility ratio. Moreover, different patterns are obtained also for structures with $T/T_p < 1.0$ (Somerville et al., 1999). The conclusion is that no single story shear strength pattern will provide consistent protection at all performance levels and for structures with different periods.

Presuming that the primary concern is to prevent excessive ductility demands in very severe events, it appears to be appropriate to strengthen the bottom portion of the structure compared to the standard SRSS story strength pattern. Such a strengthening technique was investigated using the story strength pattern shown in Fig. 9.7. In this option, the lower 30% of the structure is strengthened compared to the SRSS design with a linear strength increase leading to 40% extra strength at the base. The story ductility demands for structures with $T/T_p = 1.0$ and 2.0, designed using the new shear strength pattern, are shown in Fig. 9.8. The graphs can be compared directly
with Fig. 6.21 to evaluate the benefits achieved by adding the extra strength, which is obviously associated with extra cost.

The benefits in reducing the maximum ductility demands are evident for the structure with $T/T_p = 2.0$. The demands become more uniform over the lower portion and the maximum demand decreases by a factor larger than the strength increase factor of 1.4 in most cases. The benefits for the structure with $T/T_p = 1.0$ are not as evident. For this structure the ductility distribution without strengthening (Fig. 6.21(a)) is already rather uniform over the lower portion, and the effect of strengthening is to reduce the demand at the very bottom. But the reduction in the maximum ductility demand is minor because the effect of strengthening diminishes at the level at which no strengthening is provided. More results, including the strengthening effects for structures with P-delta effects and the study of other story strength patterns, are provided in Somerville et al. (1999).
Pulse Period - Magnitude Relationship
Near-Field Ground Motions with Forward Directivity

\[
\log_{10} T_p = -1.76 + 0.31 M_w
\]

Figure 9.1 Dependence of Equivalent Pulse Period on Magnitude

Effective Pulse Velocity

Figure 9.2 Dependence of Equivalent Pulse Velocity on Magnitude and Distance
Figure 9.3 Magnitude and Distance Dependence of Base Shear Strength Demands for Equivalent Pulse P2
Inelastic Base Shear Strength Demands
Pulse P3, $M_a = 6.0$, $R = 10$ km, $T_e = 1.3$ sec, $a_{max} = 0.08$ g, $\xi = 2\%$, no P.A.

Inelastic Base Shear Strength Demands
Pulse P3, $M_a = 6.0$, $R = 3$ km, $T_e = 1.3$ sec, $a_{max} = 0.14$ g, $\xi = 2\%$, no P.A.

Inelastic Base Shear Strength Demands
Pulse P3, $M_a = 6.5$, $R = 10$ km, $T_e = 1.8$ sec, $a_{max} = 0.14$ g, $\xi = 2\%$, no P.A.

Inelastic Base Shear Strength Demands
Pulse P3, $M_a = 6.5$, $R = 3$ km, $T_e = 1.8$ sec, $a_{max} = 0.14$ g, $\xi = 2\%$, no P.A.

Inelastic Base Shear Strength Demands
Pulse P3, $M_a = 7.0$, $R = 10$ km, $T_e = 2.6$ sec, $a_{max} = 0.18$ g, $\xi = 2\%$, no P.A.

Inelastic Base Shear Strength Demands
Pulse P3, $M_a = 7.0$, $R = 3$ km, $T_e = 2.6$ sec, $a_{max} = 0.18$ g, $\xi = 2\%$, no P.A.

(a) $R = 10$ km  
(b) $R = 3$ km

Figure 9.4 Magnitude and Distance Dependence of Base Shear Strength Demands for Equivalent Pulse P3
Figure 9.5  Sensitivity of Base Shear Strength Demands to $T_p$ Obtained from Equivalent Pulse P3 for $M_w = 7.0$ and $R = 3$ km
Figure 9.6 Story Shear Strength Patterns Leading to Constant Ductility Over Height, Pulse P2

Figure 9.7 SRSS and Modified Story Shear Strength Patterns
Figure 9.8 Story Ductility Demands for Strengthened Structures; Pulse P2
10. STUDY OF KOBE GROUND MOTIONS

This chapter includes the results of a study performed on 11 near-field ground motions recorded during the 1995 Kobe earthquake. The main objective of this study is to investigate the pulse-type response characteristics of SDOF and MDOF structures subjected to the fault-normal component of the Kobe ground motions, and to establish and evaluate equivalent pulses for the records in the context discussed in the previous chapters.

10.1. Ground Motions Used

The designation and basic properties of the Kobe ground motions are listed in Table 10.1. These records and their station locations were obtained from Japanese sources (see Acknowledgements). All stations are designated with forward directivity because they are in the forward direction with respect to the Awaji Island epicenter. Figure 10.1 illustrates the locations of the recording stations in the Kobe area. The ground motions were originally recorded as acceleration, except for AMC and KBU, which were recorded as velocity. The central difference method is used to convert the velocity records into acceleration. The acceleration and velocity time histories of the fault-normal component of the ground motions are presented in Fig. 10.2. Most of these time history traces exhibit characteristics that are close to those of a multiple pulse (pulse P3). Three of the records introduced here correspond to the Kobe ground motions studied in the previous chapters, i.e., KOB, TKT, and PR1, which correspond to KB95kobj, KB95tato, and KB95kpi1, respectively. However, KOB is a rock record, whereas KB95kobj has been analytically transformed into a stiff soil motion whose spectral values are higher than those of KOB at periods longer than 0.7 sec (see Fig. 2.6). The difference between KB95kpi1 and PR1 is that PR1 has been recorded at a depth of 83 meters.

10.2. Elastic Response Spectra

Figure 10.3 shows the acceleration (elastic strength), velocity, and displacement response spectra for the fault-normal and fault-parallel components of the Kobe records. As with the records investigated previously, a global hump is observed in most of the fault-normal velocity spectra, which corresponds to the period of the predominant pulse contained in the near-field record. The spectral values for the fault-normal component are consistently larger than their fault-parallel counterparts. This is in agreement with the results presented in Chapter 2 using a different set of near-field ground motions.
10.3. Equivalent Pulses

Based on the procedure discussed in detail in Chapter 7, attempts are made here to establish equivalent pulses for the fault-normal component of several of the Kobe records, and to evaluate the quality of the equivalent pulse representation. Table 10.2 summarizes the properties of the equivalent pulse for the records investigated. The table lists the pulse type, pulse period, as well as the effective acceleration and peak velocity of the equivalent pulse for three ductility ranges of (a) $\mu = 1$ (high performance level), (b) $\mu = 4$ to 10 (low performance level), and (c) $\mu = 1$ to 10 (the entire range of interest). The results show that the peak velocity of the equivalent pulse ($v_{\text{eff}}$) and the peak ground velocity (PGV) of the record are similar, but not as close as for most of the records discussed in the previous chapters.

Figures 10.4 to 10.7 are intended to assess how accurately the equivalent pulse can represent maximum story ductility demands of an MDOF structure subjected to several of the Kobe near-field ground motions. The figures compare the base shear strength demand obtained from the record with the corresponding demand obtained from the equivalent pulse for target maximum story ductility ratios. The effective pulse acceleration associated with the entire ductility range of interest ($\mu = 1$ to 10) is used. The graphs illustrate record-to-pulse strength demand ratios versus $T/T_p$, which is equal to unity for an ideal case, where the pulse is a perfect match for the record. For instance, a ratio larger than one for given $T/T_p$ and $\mu$ values indicate that the equivalent pulse underestimates the strength demand obtained from the record in that particular case. As can be seen, depending on the $T/T_p$ value and the ductility ratio of interest, the equivalent pulses may underestimate or overestimate the strength demand. In other words, there is no equivalent pulse that can perfectly represent near-field ground motions in the entire range. However, the results show that in many cases the accuracy of such a representation may be within an acceptable range for practical purposes.

In many cases the equivalent pulse appears to underestimate the strength demands for a maximum story ductility of 4 in the period range of $T/T_p > 1.0$. As shown in Chapters 4 and 6, the ductility of 4 is around a sensitive threshold, where maximum ductility demands migrate from the top to the bottom portion of structures with $T/T_p > 1.0$. The migration phenomenon occurs slightly more rapidly for simple pulses than for near-field ground motions in some cases, resulting in smaller base shear strength required to limit the story ductility demand to 4. Nevertheless, this is merely a local discrepancy and does not affect the strength demands associated with other ductility ratios.
In order to verify the choice of the pulse type, pulses P2 and P3 are compared as the equivalent pulse to determine which one of these pulses leads to a closer match with the ground motion. A comparison of parts (a) and (b) of Figs. 10.4 to 10.7 demonstrates that choosing either of the pulse shapes has advantages for some periods and ductilities, and disadvantages for others. Therefore, the suitable pulse shape depends on the period and ductility range of interest. However, one can make the choice by employing judgment, considering the match in the entire range, and inspection of the ground time histories. For example, a comparison of Figs. 10.4(a) and 10.4(b) shows that pulse P3 is generally a better match for the record EKB.
Table 10.1 Designation and Properties of Kobe Records, $M_w = 6.9$

<table>
<thead>
<tr>
<th>Designation</th>
<th>Station</th>
<th>Directivity</th>
<th>Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>AMC</td>
<td>Amagasaki</td>
<td>forward*</td>
<td>8.7</td>
</tr>
<tr>
<td>EKB</td>
<td>Higashi Kobe Bridge</td>
<td>forward*</td>
<td>2.1</td>
</tr>
<tr>
<td>FKA</td>
<td>Fukiai, Osaka Gas</td>
<td>forward*</td>
<td>0.7</td>
</tr>
<tr>
<td>KB3</td>
<td>NTT Building</td>
<td>forward*</td>
<td>1.1</td>
</tr>
<tr>
<td>KBU</td>
<td>Kobe University</td>
<td>forward*</td>
<td>0.3</td>
</tr>
<tr>
<td>KOB</td>
<td>JMA Kobe</td>
<td>forward*</td>
<td>0.6</td>
</tr>
<tr>
<td>KOJ</td>
<td>Harbar Research Institute</td>
<td>forward*</td>
<td>1.5</td>
</tr>
<tr>
<td>PR1</td>
<td>Port Island -83 m</td>
<td>forward*</td>
<td>2.6</td>
</tr>
<tr>
<td>RKI</td>
<td>Rokko Island</td>
<td>forward*</td>
<td>4.7</td>
</tr>
<tr>
<td>TKT</td>
<td>JR Takatori</td>
<td>forward*</td>
<td>1.2</td>
</tr>
<tr>
<td>TZK</td>
<td>JR Takarazuka</td>
<td>forward*</td>
<td>1.1</td>
</tr>
</tbody>
</table>

* with respect to the Awaji Island epicenter

Table 10.2 Equivalent Pulses for Kobe Records (Fault-Normal Component)

<table>
<thead>
<tr>
<th>Rec. Name</th>
<th>Pulse Type</th>
<th>$T_p$</th>
<th>$a_{eff}$ (g)</th>
<th>$V_{eff}$ (cm/s)</th>
<th>$a_{eff}$ (g)</th>
<th>$V_{eff}$ (cm/s)</th>
<th>$a_{eff}$ (g)</th>
<th>$V_{eff}$ (cm/s)</th>
<th>PGV (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EKB</td>
<td>P3</td>
<td>2.5</td>
<td>0.23</td>
<td>141</td>
<td>0.20</td>
<td>123</td>
<td>0.20</td>
<td>123</td>
<td>96</td>
</tr>
<tr>
<td>RKI</td>
<td>P3</td>
<td>2.5</td>
<td>0.19</td>
<td>116</td>
<td>0.15</td>
<td>92</td>
<td>0.15</td>
<td>92</td>
<td>74</td>
</tr>
<tr>
<td>TKT</td>
<td>P3</td>
<td>2.0</td>
<td>0.45</td>
<td>221</td>
<td>0.38</td>
<td>186</td>
<td>0.40</td>
<td>196</td>
<td>153</td>
</tr>
<tr>
<td>TZK</td>
<td>P2</td>
<td>1.8</td>
<td>0.26</td>
<td>115</td>
<td>0.17</td>
<td>75</td>
<td>0.18</td>
<td>79</td>
<td>91</td>
</tr>
</tbody>
</table>

Figure 10.1 Locations of Kobe Ground Motion Stations Used in This Study
Figure 10.2 Acceleration and Velocity Time Histories of Kobe Ground Motions
Figure 10.2 (Cont’d) Acceleration and Velocity Time Histories of Kobe Ground Motions
Figure 10.2 (Cont’d) Acceleration and Velocity Time Histories of Kobe Ground Motions

(i) RKI

(j) TKT

(k) TZK

Study of Kobe Ground Motions
Figure 10.3 Elastic Acceleration, Velocity, and Displacement Spectra of Kobe Ground Motions
Figure 10.3 (Cont'd) Elastic Acceleration, Velocity, and Displacement Response Spectra
Figure 10.3 (Cont’d) Elastic Acceleration, Velocity, and Displacement Response Spectra
Figure 10.4 Comparison of Strength Demands of Ground Motion EKB and Equivalent Pulses
Figure 10.5 Comparison of Strength Demands of Ground Motion RKI and Equivalent Pulses
Figure 10.6 Comparison of Strength Demands of Ground Motion TKT and Equivalent Pulses
MDOF Strength Demand Ratios for Constant Ductility
TZK, SRSS Pattern, $T_p = 1.8$ sec, $a_{efl} = 0.18$ g, $\xi = 2\%$, without $P$.Δ

Figure 10.7 Comparison of Strength Demands of Ground Motion TZK and Equivalent Pulses
11. CONCLUSIONS

The results of the study summarized here are intended to provide a basic understanding of the important attributes that characterize near-field ground motions and their effects on the response of elastic and inelastic SDOF and MDOF structural systems. It is necessary to identify the response characteristics that set near-field ground motions aside from “ordinary” ground motions whose effects on the response of structures have been considered, either explicitly or implicitly, in presently employed design procedures. A comprehensive evaluation of the seismic demands of SDOF and MDOF structures subjected to near-field and pulse-type ground motions is presented in this work. Much effort is devoted to representing near-field ground motions using the properties of simple equivalent pulses. Modifications to design procedures are proposed utilizing the equivalent pulses in order to provide more protection for structures subjected to near-field ground motions. Advantage is also taken of the equivalent pulses to estimate the response of MDOF structures to given near-field ground motions through a relatively simple procedure.

A number of assumptions had to be made in the analyses performed in this study. The results should be interpreted or generalized within the assumptions made. This study focuses on the most important issues that set the response of structures to near-field ground motions apart from that to “ordinary” ground motions. Although the results presented here shed light on many important aspects of the near-field problem, it is emphasized that much more research needs to be done to provide final answers to this complicated problem. This study has lead to conclusions that can be summarized as follows:

- The fault-normal component of near-field ground motions with forward directivity is characterized by a short-duration severe impulsive motion that can be identified in the time history traces of ground velocity and displacement, and in the shape of the response spectra. The 45° rotated components also exhibit pulse-type characteristics, and at least one of them is almost as severe as the fault-normal component.

- Near-field ground motions come in great variations and can expose structures to demands much higher than those predicted by current seismic codes.

- In many aspects the response of structures to pulse inputs differs greatly from that to ordinary ground motions.
• SRSS modal combinations cannot capture all important elastic response characteristics of structures subjected to near-field or pulse-type ground motions when the structure fundamental period (T) is longer than the (equivalent) pulse period (T_p). A traveling wave effect, which is a fundamental characteristic of the response of long-period MDOF structures to near-field ground motions, is not adequately taken into account by standard spectral analyses.

• For long-period structures (T > T_p) designed according to present design guidelines, the distribution of elastic story shear forces over the height is sensitive to the ratio T/T_p and may cause shear forces in upper stories that are higher than the base shear. These high elastic shear forces result in early yielding of upper stories when the structure is relatively strong. When the structure strength is reduced, the story ductility demands stabilize in the upper portion and the maximum ductility demand migrates to the base.

• The traveling wave effect, which causes highly non-uniform distributions of ductility demands over the height, should be incorporated in the design of long-period structures located in near-field regions.

• For short-period structures (T ≤ T_p) the traveling wave effect is not predominant and the maximum story ductility demands occur in the bottom portion even for strong structures.

• For long-period structures (T > T_p) subjected to near-field or pulse-type ground motions, P-delta effects are not very large when the structure is strong enough to prevent the migration of the maximum ductilities from the upper portion of the structure to the base. When the maximum ductility demand occurs at the base, the effect of P-delta gains much on importance and dynamic instability is a distinct possibility. Since for short-period structures the maximum ductility always occur close to the base, P-delta effects are important even when the structure is strong.

• Inelastic roof displacements are larger than elastic ones in the short period range (T/T_p < 0.75), and become larger with a decrease in structure strength. The reverse is observed in the long period range, where inelastic roof displacement demands are smaller than the elastic ones and decrease when the strength is reduced.

• The generic models used in this study can reasonably represent the global demands (e.g., base shear strength and roof displacement demands) of a large variety of MDOF frame
structures subjected to near-field ground motions. However, when story-level demands (e.g., story ductility or story drift) are to be estimated, consideration should be given to the number of stories of the structure.

- There are clear similarities between the response of frame structures to near-field ground motions and the response to pulse-type excitations. Within the approximate period range of $0.375 < T/T_p < 3.0$, the salient response characteristics of near-field ground motions can be represented by simple equivalent pulses, which are fully defined by a pulse type, a pulse period, and a single pulse severity parameter. These equivalent pulses can be used to assess the response of frame structures to near-field ground motions in a consistent manner.

- Preliminary models have been developed that describe the site seismic hazard in terms of equivalent pulse parameters. These models relate the pulse period and the pulse effective velocity to the magnitude of the event and the shortest distance from the site to the fault. These models, together with the pulse MDOF strength demand spectra for constant ductility, can be utilized to develop base shear strength demand spectra for design.

- Suitable distributions of story shear strength over the height of the structure depend on the performance objectives and the $T/T_p$ ratio of the structure under consideration. For long-period structures ($T/T_p > 1.0$) subjected to very severe ground motions, strengthening of the bottom stories (compared to a standard SRSS distribution) will significantly reduce the maximum story ductility demands. For short-period structures ($T/T_p \leq 1.0$) strengthening of the lower half of the structure is necessary to achieve an effective reduction in the maximum demands.
APPENDIX A. TIME HISTORY TRACES AND ELASTIC SPECTRA OF NEAR-FIELD GROUND MOTIONS

The set of 23 near-field ground motions used in this study is introduced in Chapter 2. Fifteen of the ground motions, which are extensively utilized in the response studies, are records with forward directivity. Figure A.1 illustrates the ground acceleration, velocity, and displacement time histories of the fault-normal component of these 15 records.

Figure A.2 shows the acceleration (elastic strength), velocity, and displacement spectra of the near-field records whose fault-normal time histories are presented in Fig. A.1. Each graph shows the spectra for the fault-normal, fault-parallel, and two 45° (rotated with respect to the fault direction) components of the ground motion. The spectral values are computed using a damping ratio of $\xi = 2\%$. 

Figure A.1 Ground Acceleration, Velocity, and Displacement Time Histories
Figure A.1 (Cont'd) Ground Acceleration, Velocity, and Displacement Time Histories
Figure A.1 (Cont’d) Ground Acceleration, Velocity, and Displacement Time Histories
Figure A.1 (Cont'd) Ground Acceleration, Velocity, and Displacement Time Histories
Figure A.2 Elastic Acceleration, Velocity, and Displacement Response Spectra
Figure A.2 (Cont'd) Elastic Acceleration, Velocity, and Displacement Response Spectra
Figure A.2 (Cont’d) Elastic Acceleration, Velocity, and Displacement Response Spectra
Figure A.2 (Cont'd) Elastic Acceleration, Velocity, and Displacement Response Spectra
REFERENCES


