Damage-based seismic design and performance assessment of reinforced concrete structures

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DAMAGE-BASED SEISMIC DESIGN AND PERFORMANCE ASSESSMENT OF REINFORCED CONCRETE STRUCTURES

WEI JIANWU

**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY**

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WEI JIANWU

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

**A thesis submitted to Nanyang Technological University
in fulfillment of the requirements for the degree of Doctor of
Philosophy**

2007

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ABSTRACT

Inspections after major earthquakes in the last two decades revealed that most buildings designed to conform to modern seismic design codes could survive moderate to severe earthquakes; however, the high level of spread structural damage was often excessively costly to repair. To address the concerns about economical losses, the concept of performance based seismic design (PBSD) emerged. Within the general framework of PBSD, the design and evaluation of building structures on the basis of quantified damage constitutes a fundamental aspect. However, there is still a lack of effective and systematic methodology for the quantification of structural damage and implementation of the quantified damage in the design and evaluation process for real applications. Besides, as a special class of seismic performance evaluation, there is a significant lack of understanding of the likely damage to nonseismically designed structures in low-to-moderate seismicity regions, considering the actual seismic behaviour of such type of structures and a realistic seismic demand.

This thesis is aimed to address some of the important aspects within the framework of PBSD, with focus on the damage-based design and evaluation, including nonseismically designed structures. Both analytical and experimental studies are conducted.

In the first part of the thesis, a coherent methodology is proposed for damage-based design and evaluation, including the generation of damage-based inelastic spectra and the framework for implementation on actual structural systems. A damage-based strength reduction factor (R_d) is introduced to associate the seismic demand with the structural damage in an explicit manner. A comprehensive investigation into the R_d spectra is carried out on the influences of pertinent parameters, including site conditions, earthquake magnitudes, source-to-site distances as well as hysteretic patterns. For practical use, unified empirical formulas are developed.

To implement the methodology for actual frame structures, two alternative methods are put forward and investigated through nonlinear static and seismic time-history analyses, one based on the modal pushover analysis, and the other based on the characterization using a newly proposed storey capacity factor. The first method, combining the effects from the first two modes, can provide a reasonable prediction of the damage distribution. On the other hand, the storey capacity factor is also found to correlate well with the damage distribution in regular as well as irregular frames.

The inter-storey drift remains to be an important measure concerning structural as well as nonstructural damage. To reflect the variation of the influences of the storey overstrength and stiffness on the inter-storey drifts with the increase of the inelastic response (ductility), the general storey capacity factor is modified so that the strength and stiffness contributions vary with the ductility level. Subsequently, a simple procedure is derived to predict inter-storey drift distribution from the spectral displacement using the modified storey capacity factor.

With regard to the performance and damage evaluation of non-seismically designed frame structures, a comparative shake table experiment is conducted on two companion frames of different detailing with a representative long-duration ground motion of increasing intensities. The potential problems with typical nonseismic details are examined with respect to realistic (low-to-moderate) seismic demands and for intermediate performance levels. In addition, the determination of important modeling parameters for the dynamic analysis of such structures is discussed based on the observed responses. The significance of cumulative damage under long-duration ground motions is highlighted.

Finally, the effectiveness of the commonly used FRP repairing technique for frame structures in recovering the structural performance is investigated using the shake table tests. The potential change of failure mechanisms, the drawback of debonding of FRP strips, and the implications of these issues on the performance evaluation of FRP-repaired frame structures are evaluated.

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LIST OF SYMBOLS

a_1, a_2	Constants for regression
b_1, b_2	Constants for regression
A_{et}	Extension reinforcement area
A_g	Gross concrete area
A_s	Reinforcement area
b_j	Effective joint width
c	Constant in Jeong and Iwan's damage model, or damping coefficient
c_1, c_2	Constant for regression
c^*	Damping coefficient of equivalent SDOF system
COV	Coefficient of variation
C	Damping matrix
d_b	Diameter of longitudinal bars
D	Damage index
D_i	Target damage level
D_K	Global degradation index
D_k	Modified column lateral stiffness
D_{PA}	Damage index from the Park and Ang damage model
D_s	Drift spectrum
D_{SDOF}	Damage level
E_h	Cumulative hysteretic energy
E_h^*	Dissipated hysteretic energy
E_{Pi}	Energy dissipated by primary half-cycle
E_{Si}	Energy dissipated by follower half-cycle
E_U	Maximum energy dissipated under monotonic loading
EI	Bending stiffness
f	Current fundamental frequency
f_0	Initial fundamental frequency
f'_c	Concrete cylinder compressive strength
f_y	Yield strength of the longitudinal bar

F_e	Elastic strength demand
$F_{y,\mu}$	Yield strength for ductility level μ
$F_{y,D}$	Yield strength for damage level D
F_y	Yield force or yield strength
ΔF	Strength reduction from elastic strength F_e
h	Storey height
h_c	Column depth
H	Building height
i_c	Linear stiffness
i_{OS}	Storey overstrength factor
$(i_{OS})_{ave}$	Mean value of storey overstrength factor
$(i_{OS})_{min}$	Minimum value of storey overstrength factor
i_{ns}	Normalized storey stiffness factor
$(i_{ns})_{ave}$	Average value of normalized storey stiffness factor
$(i_{ns})_{min}$	Minimum value of normalized storey stiffness factor
i_{sc}	Storey capacity factor
$(i_{sc})_{ave}$	Average value of i_{sc}
$(i_{sc})_{min}$	Minimum value of i_{sc}
K	Current lateral stiffness
K_0	Initial elastic stiffness
K_e	Secant stiffness
l_{em}	Extension length to joint
m	Mass of SDOF system
m^*	Mass of SDOF system
m_e	Effective mass
m_m^{req}	Required mass of the test model
m_p	Mass of prototype structure
M_s	Surface magnitude
M_w	Moment magnitude
M_y	Yield moment
$\mathbf{M}$	Mass matrix
n_0	Axial force ratio
n_f	Number of cycles to failure

N	Axial load
N_s	Number of storeys
PGA	Peak ground acceleration
q^*	Force of the equivalent SDOF system
q_y^*	Yield strength
$\mathbf{Q}$	Storey force vector
$\mathbf{Q}_y$	Yield storey force vector
R^2	Square of the regression correlation coefficient
R_μ	Ductility-based strength reduction factor
R_D	Damage-based strength reduction factor
R_f	Closest distance to the surface projection of the fault rupture
s_0	Spacing of transverse reinforcement
S_a	Elastic acceleration response
S_d	Elastic displacement response
T	Initial natural period of the system
T_0	Limiting period
T_e	Effective period
T_{eq}	Equivalent initial period
T_g	Predominant period
$\ddot{u}_g$	Ground acceleration
u^*	SDOF displacement
u_m^*	Maximum displacement attained
u_t	Top displacement
u_y^*	Yield displacement
u_u^*	Displacement capacity under the monotonic loading
$\mathbf{u}$	Displacement (relative) vector
v_g	Maximum ground velocity
V_b	Base shear of the structure
V_{jh}	Joint shear force
V_n	Normalized storey shear
V_R	Available storey shear strength
V_S	Design storey shear force
V_y	Yield base shear

w_i	Weighting factor
α_1	Post-yield stiffness ratio
α_r	Strength regularity index
α_s	Stiffness degradation parameter
α_{sc}	Regularity index
β	Constant parameter in the Park & Ang damage model
β_1	Mode participation factor
β_2	Elastic concentration factor of inter-storey drifts
β_3	Amplification factor of inelastic displacement
β_4	Elastic concentration factor of inter-storey drifts
β_e	Energy-based strength degradation parameter.
δ_m	Maximum displacement
δ_y	Yield displacement
δ_u	Ultimate displacement capacity
Δ_d	Maximum displacement
Δ_e	Elastic displacement
Δ_m	Maximum displacement
Δ_u	Ultimate displacement under monotonic loading
Δ_y	Yield displacement
$\{\phi\}$	Displacement shape
λ	Geometric scale factor
γ	Energy-related parameter
γ_p	Pinching parameter
Γ	Modal participation factor
η	Modification factor of column stiffness
η_1, η_2	Damage combination factors
η_s	Modification factor of column stiffness
μ	Displacement ductility
μ_u	Ultimate (monotonic) displacement ductility
θ_m	Maximum rotation attained during the loading history
$\theta_{\max}$	Maximum storey drift
$\theta_{r,peak}$	peak roof drift
θ_{sc}	Predicted storey drift

θ_y	Yield rotation
θ_u	Ultimate rotation capacity
ρ_t	Volumetric ratio of transverse reinforcement
ξ	Viscous damping ratio
ω_n	Natural frequency of SDOF system

CHAPTER 1

INTRODUCTION

1.1 Background

1.1.1 Damage-based seismic design and evaluation

Lessons from recent earthquakes indicate that, despite many years of worldwide efforts in earthquake-related research and practice, earthquakes continue to cause substantial damage and loss of life in many parts of the world. Although many buildings designed to conform to modern seismic design codes did not collapse during the earthquakes, the level of damage to structures was unexpectedly high. For example, a reinforced concrete building, called Jeunesse Rokko, suffered widespread damage in the 1995 Kobe earthquake (Moment magnitude $M_w=6.9$) due to the formation of a beam-yielding mechanism, which is commonly regarded as the most desirable earthquake resisting mechanism. Although the building appeared to be structurally sound after the earthquake, high cost to repair the damage at many beam ends and the perceived poor appearance from extensive repair work made the owner decide to demolish the building (Matsumori and Otani 1998). This example shows clearly that buildings designed in accordance with appropriate structural safety considerations might survive severe earthquakes without collapse; however, the cost of repairing the spread damage due to inelastic response can often be excessive and uneconomic.

In addition to the high cost of repairing the structural damage, the cost of business interruption due to loss of the building functions (damage to nonstructural elements

and equipment in buildings) have also been very significant in past earthquakes (EERI 1994).

In response to the growing concerns about losses due to building damage, the concept of performance-based seismic design (PBSD) has emerged (SEAOC 1995; ATC 1996; NEHRP 1997). Within the framework of PBSD, it is required that multiple design objectives be explicitly considered in terms of desired performance levels and expected levels of earthquake hazard. This idea has been widely accepted as the core of next generation of seismic design codes (Fajfar and Krawinkler 1998). However, the implementation of full performance based design in real design practices is still at a preliminary stage. One of the key difficulties lies on the quantification of building/structural damage and an effective way to take quantified damage into consideration in the seismic procedure. Several provisional documents such as Vision 2000 (SEAOC 1995) and FEMA 273 (NEHRP 1997) adopt the inter-storey drift as a basis for definition of performance/damage levels. However, as pointed out by Ghobarah (2004), it is an oversimplification to relate drift of the structure to damage since the damage is influenced by the accumulation and distribution of the structural damage, failure mode of the elements, the number of cycles and the duration of the earthquake, among other factors.

A comprehensive treatment of performance criteria has to take into consideration the structural as well as nonstructural damage, including damage to building contents. As far as structural damage is concerned, the damage index obtained from an appropriate damage model is more realistic than a single deformation measure. In this respect, the development of damage-based inelastic spectra, instead of the traditional ductility based inelastic spectra, is desired for purpose to explicitly relate the seismic demands to a target level of structural damage. Subsequent procedures are needed to evaluate the distribution of damage in multi-storey systems.

1.1.2 Performance evaluation of nonseismically designed structures

Along with the development of performance-based seismic design for new buildings, the actual performance of nonseismically designed (gravity-designed) structures has also received much attention in recent years. As a matter of fact, reinforced concrete (RC) building structures in many low to moderate seismic regions, such as the eastern and central United States, have historically and are still typically designed only for gravity load. Such structures generally possess reinforcement details that conform to the ordinary code of practice, but do not conform to modern seismic provisions. Previous studies (Aycardi et al. 1994; Bracci et al. 1995) indicated that such structures may still possess an inherent lateral strength capacity that may be mobilized to resist moderate earthquakes; but the deficient detailing can lead to questionable structural performance during severe seismic activity. In Japan, older (pre-1980) reinforced concrete structures, in general, sustained enormous damage in the 1995 Kobe earthquake (Scawthorn and Yanev 1995) since the Japanese building and other structural codes were significantly modified in 1981, to require more ductile detailing and other improvements. Designers in New Zealand and in many other countries are also being confronted with the task of assessing the seismic performance of existing RC structures designed before the introduction of modern seismic codes.

In Singapore, the current design practice following the British Standards does not impose any specific seismic design requirements. With the great Sumatra fault and the subduction zone 350km away at the closest point, there is some rising concern about the possibility that building structures in this low seismicity region could be subjected to some damaging level ground excitation if some worst-case scenario earthquake would occur.

Although some experimental studies have been conducted to evaluate the general behaviour of non-seismically designed structures (El-Attar et al. 1991; Bracci et al. 1995), there is still a lack of comparative information concerning the seismic behaviour of structures with different possible detailing arrangements. There is also

a need to examine the seismic performance of such structures under different levels of seismic hazard, especially for low-to-moderate ground motions.

1.2 Research objectives

For performance-based seismic design and evaluation, appropriate methodologies and effective procedures are needed to achieve satisfactory control of the structural damage to acceptable levels for different levels of earthquake hazard. Meanwhile, experimental observations are needed to improve the understanding of the actual seismic performance of building structures, especially those with nonseismic details, under realistic seismic ground motions. To these ends, the present study has been conducted targeting at the following objectives:

- 1) to propose damage-based inelastic spectra to relate the seismic (strength) demand to structural damage for damage-based design and performance evaluation, and to investigate the effects of earthquake characteristics, hysteretic patterns as well as other pertinent parameters on such spectra.
- 2) to examine the correlation between the damage indices of SDOF and MDOF systems, and subsequently propose a suitable methodology for prediction of damage distributions in multi-storey RC frame structures.
- 3) to propose a procedure for prediction of seismic inter-storey drifts with a newly developed storey capacity factor, taking into account the varying influences of storey overstrength and stiffness with increasing ductility levels.
- 4) to experimentally investigate the comparative seismic performance of nonseismically designed frame structures under realistic seismic ground motions, with shake table tests. In particular, the effect of long duration ground motions on cumulative damage is highlighted.
- 5) to experimentally investigate the effectiveness of commonly used repairing and retrofitting techniques with fibre reinforced polymer (FRP) in restoring the structural performances, especially in terms of strength, stiffness and deformability after moderate damage.

The originality of this study lies in the following aspects. a) the importance of damage-based inelastic spectra in structural performance (in terms of structural damage) design and evaluation, although recognized by many researchers, has not been realized in the past with a coherent methodology for the generation of such spectra and implementation for real structural systems. This study develops a complete methodology and implementation framework covering the generation of damage-based inelastic spectra for the determination of the seismic demand in design applications, to their application in a full structural damage evaluation on multi-storey frame systems. b) The physical influences of the storey overstrength and stiffness on the inelastic response distribution (in particular the inter-storey drift) are explicitly expressed in the proposed storey capacity factor evaluation with consideration of the varying significance of strength and stiffness with inelastic response (ductility) levels. c) Viewing the potential problems of nonseismic design details from the perspective of realistic (low-to-moderate) seismic demands is a significant point. This would allow the actual performance of non-seismically detailed frames be assessed in a more realistic manner. Such an idea is then applied in the assessment of the general performance of representative non-seismically detailed frames by means of the shake table tests. New insights are brought into the potential issues with non-desired details.

1.3 Thesis organization

This thesis consists of eight chapters. Chapter 1 introduces the research background and objectives. Chapter 2 presents a literature review of the recent developments of performance-based seismic design, with particular focus on the damage models and damage based performance evaluation. The performance of structures with nonseismic details is reviewed extensively, with a systematic characterization of the behaviour of such structures based on previous experimental data.

Chapter 3 is devoted to the development of damage-based inelastic demand spectra for damage based design and evaluation. The inelastic spectra are expressed in terms of the damage-based strength reduction factor, R_D . The influences of

important parameters such as site conditions, earthquake characteristics, as well as hysteretic patterns are examined and a generalized empirical expression of the mean R_d spectra is established.

Chapter 4 presents a methodology for prediction of damage distributions through two alternative methods involving pushover analysis and the storey capacity factor.

Chapter 5 describes a procedure for prediction of seismic inter-storey drifts with an inelastic response-dependent storey capacity factor.

Chapter 6 presents the seismic damage and performance of structures with nonseismic details investigated in a comparative shake table experiment. The seismic performance of the repaired frames and the effectiveness of FRP wrapping on restoration of structural properties are addressed in Chapter 7. Finally, the main conclusions of this research study and recommendations for future research are summarized in Chapter 8

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter presents a comprehensive review of the literature on three main topics related to this study, namely, a) framework of performance-based seismic design, and the existing methods for estimation of displacement demands; b) structural damage evaluation and existing damage models for quantifying damage in a reinforced concrete structure under seismic loading; and c) as a special class of performance and damage evaluation problems, the nonseismically designed structures are summarized and the previous experimental studies related to columns, beam-column joints and reduced scale structural models for this category of structures are reviewed.

2.2 Overview of performance-based seismic design

From the lessons of earthquake damages in recent decades, it is widely recognized that the current design procedure needs to be improved to contemplate multiple design objectives expressed in terms of desired performance levels and expected levels of earthquake hazard. Such a procedure, which targets at explicitly controlling the damage in structural and nonstructural components as well as the facilities equipped in buildings, is generally called performance-based seismic design (PBSD).

Vision 2000 (SEAOC, 1995) outlined the general framework of performance-based seismic design. An important first step was the development of the performance matrix as shown in Figure 2.1. In this document, the seismic performance objectives are defined as the “coupling of expected performance level with expected levels of seismic hazard”. Four performance levels were identified, namely:

- **Fully Operational:** No damage, continuous service.
- **Operational:** Damage is light. Most operations and functions can resume immediately. Repair is required to restore some non-essential services. Structure is safe for immediate occupancy. Essential operations are protected.
- **Life Safety:** Damage is moderate. Selected building systems, features or contents may be protected from damage. Life safety is generally protected. Structure is damaged but remains stable. Falling hazards remain secure. Repair is possible.
- **Near Collapse:** Damage is severe. Structural collapse is prevented. Nonstructural elements may fall. Repair is generally not possible.

Figure 2.1 Relationship between earthquake design level and performance level (after SEAOC 1995)

Similar frameworks of performance-based design and evaluation were subsequently proposed in ATC 40 (ATC 1996) and FEMA 365 (BSSC 2000). The implementation of performance-based seismic design has attracted wide spread attention in the structural engineering communities in many countries (Fajfar and Krawinkler 1998; Teran-Gilmore 1998; Priestley 2000; Xue 2000; Ghobarah 2001; Chopra and Goel 2001; Chandler and Lam 2001; Bertero and Bertero 2002).

In addition to those functional and safety related performance levels, Priestley (2000) suggested that a damage control performance level be defined to account for repair costs. The 1995 Kobe earthquake gave a good lesson on the excessive cost that could incur from repairing the damage due to inelastic behaviour, although the structure survived the earthquake event without collapse (Otani 1997). Kawashima (1997) suggested that the residual deformation be taken into account in defining the damage control performance level to reduce the repairing cost.

For practical applications, the performance level (structural and nonstructural performance) has generally been related to structural deformation such as the interstorey drift ratio in building structures. However, as pointed out by Ghobarah (2004), it is an oversimplification to relate structural damage to drift, since the state of a structure is actually influenced by the cumulation and distribution of the structural damage, failure mode of the elements, the number of cycles and the duration of the earthquake, among other factors. The importance of cumulative damage in structural design has also been highlighted in some other recent studies (Bertero and Bertero 2002; Teran-Gilmore 2004; Teran-Gilmore and Jirsa 2005; Khashaei 2005).

As a matter of fact, PBSd is essentially a reliability-based design/evaluation tool. Nevertheless, it is necessary and instrumental to establish the relevant criteria and the evaluation methodologies in a deterministic context, especially for concerns over the performance of the structural components, before incorporating the probabilistic considerations. The primary objective of this thesis is to establish such a deterministic framework with the structural performance being represented by a

damage index. Although not specifically taken into account in the present study, it is useful at this point to provide a brief overview of the sources of uncertainties and the potential analysis methods for the general PBSO. In broad terms, these may be divided into two categories, as follows:

i) Uncertainties associated with “capacities”

- in the mechanical properties (strength, modulus, etc) of the materials, such as steel reinforcement and concrete;
- in the nonlinear behaviour, such as displacement/rotation capacities, of structural members and joints, etc;
- in the behavioural limits of non-structural components and building contents

ii) Uncertainties associated with seismic demands

- in the amplitudes of the earthquake input (e.g., PGA of the ground motions etc.)
- in the dynamic characteristics of the earthquake input, such as the dominant frequencies and primary duration of the ground motions.

A lot of research effort has been made in attempt to quantify the above uncertainties and establish the corresponding probabilistic models (e.g., Bertero and Bertero 2002, Lu and Gu 2004). With the availability of appropriate probabilistic models for the relevant demands and capacities, a reliability-based PBSO would be able to proceed.

2.3 Damage-based design incorporating structural cumulative damage

Experimental and field evidences suggested that the mechanical characteristics of a structure deteriorate under inelastic reversals during long and severe ground motions. In recent years, increasing attention has been directed to the cumulative damage, and there is a tendency to use cumulative damage as a performance criterion to control the local structural damage (Bertero and Bertero 2002; Teran-Gilmore 2004; Teran-Gilmore and Jirsa 2005; Khashaee 2005). Earlier studies have shown that the plastic energy dissipated by the structure during a ground motion can

be used to characterize the cumulative damage for design purposes (Fajfar 1992, Krawinkler and Nassar 1992, Bertero et al. 1996). Due to the cumulative damage, the maximum deformation capacity of a structure under dynamic load reversals would be lower than the ultimate deformation capacity under monotonic loading.

2.3.1 Damage evaluation at structural level

The nonlinear time history analysis is the most direct approach for prediction of seismic damage to a structure. The damage of structural members can be calculated first by employing a damage model based on the predicted seismic responses and structural capacity. The damage at the storey and entire structure levels can then be obtained by a weighting average scheme. This procedure has been employed in many previous studies (Park et al. 1987; Hatamoto 1990; Gunturi and Shah 1992; Stone and Taylor 1994; Kunnath et al. 1995a, 1995b; Bracci et al. 1995). More recently, Kappos and Manafpour (2001) proposed a methodology for performance-based design of RC buildings, where the time history analyses are performed using the ground motions compatible to the design spectrum. The obtained local damage is checked with the performance criteria for two distinct limit states.

However, to perform nonlinear time history analyses is tedious and costly, especially when uncertainties in the ground motion input needs to be considered. To reduce the analysis effort, the damage estimation may also be carried out by the nonlinear static analysis (pushover). Fajfar and Gašperšič (1996) proposed a procedure for seismic damage estimation of RC structures using the N2 method. In this procedure, the deformation and energy demands of the equivalent SDOF system are firstly determined from the inelastic pseudo-acceleration spectra and the γ parameter (related to hysteretic energy and ductility), respectively. These are then transformed to the actual structure (MDOF system) and distributed to the structural members. Finally, the damage indices of the local members, storeys and the entire structure can be calculated using a selected damage model. It is noted that in the energy distribution, the dynamic plastic energy is assumed to be proportional to the energy dissipated at the maximum displacement under monotonic loading.

Similar method was also proposed by Cruz and López (2004) for damage control design of RC frames.

2.3.2 Damage control through damage-based spectra (SDOF system)

a) Ductility-based strength reduction factor

In traditional seismic design, the structural damage is not explicitly considered in the design phase. An appropriate ductility is generally required to ensure the structure exhibit (ductile) inelastic behaviour under severe earthquakes. The lateral strength, $F_{y,\mu}$, is generally determined by reducing the elastic strength demand, F_e , by a strength reduction factor, R_μ , depending on the maximum displacement ductility, μ . Thus, the ductility-based strength reduction factor is defined as

$$R_\mu = \frac{F_e}{F_{y,\mu}} \quad (2.1)$$

Several expressions have been proposed for strength reduction factor spectra in previous studies (Newmark and Hall 1982; Riddell et al. 1989; Nassar and Krawinkler 1991; Uang 1991; Miranda 1993; Vidic et al. 1994; Lam et al. 1998). Investigations on strength reduction factors and their implication in seismic design have been reviewed in Miranda and Bertero (1994) and Borzi and Elnashai (2001).

As aforementioned, the displacement ductility alone is not sufficient to quantify the damage sustained in the cyclic response due to cumulative damage. As a matter of fact, the seismic damage should be characterized by considering both ductility and energy dissipation demands. The more reliable strength demand should be associated with the damage, and other structural properties such as stiffness and ductility classes (capacity). The influence of cyclic excursions and cumulative damage on the strength reduction in structures subjected to seismic loading has been evidenced in many recent studies (Fajfar 1992; Vidic and Fajfar 1995; Teran-Gilmore 1996; Krätzig and Meskouris 1997; Warnitchai and Panyakapo 1999; Tiwari and Gupta 2000; Teran-Gilmore and Avila 2001; Decanini et al. 2004).

Fajfar (1992) suggested that the actual response ductility μ be specified from the ultimate ductility capacity μ_u using the Park and Ang damage model (Park and Ang 1985), i.e.

$$\mu = \frac{\sqrt{1 + 4\beta\gamma^2\mu_u D_{PA}} - 1}{2\beta\gamma^2} \quad (2.2)$$

where D_{PA} is the prescribed damage level according to the Park and Ang damage model; β is the constant parameter in the damage model; μ_u is the ultimate ductility capacity under monotonic loading; and γ is the energy parameter. Subsequently, the lateral strength can be obtained from the aforementioned R_μ factor. In this way ($D_{PA} - \mu - R_\mu$), the cumulative damage can be taken into account in the seismic design.

It is noted that the actual ductility of a structure/structural member would be dependent upon the characteristics of the dynamic response histories, particularly the number of large inelastic cycles. However, for the sake of practical application, the (nominal) ductility factor may be defined either in terms of the monotonic ductility and the cyclic ductility associated with a simplified cyclic history. The monotonic ductility is usually considered as a reference ductility capacity in an equivalent evaluation, whereas the cyclic ductility to a certain extent reflects the actual inelastic behaviour under cyclic loading. Obviously, the cyclic ductility is always lower than the monotonic ductility for the reinforced concrete members. In this thesis, both ductility factors have been employed (designated as μ_u and μ , respectively). Similar use can also be found in the literature (Fajfar 1992).

b) Damage-based strength reduction factor

Besides the above indirect methods, the required strength can also be directly determined through a damage-based strength reduction factor, R_D . Similar to the R_μ factor, the R_D factor is defined as

$$R_D = \frac{F_e}{F_{y,D}} \quad (2.3)$$

where $F_{y,D}$ is the strength corresponding to a specified damage level D which can be defined by a damage model. In some recent studies (Warnitchai and Panyakapo 1999; Tiwari and Gupta 2000), the Park and Ang damage model was selected to calculate the damage of SDOF systems.

To apply the R_D factor in seismic design, the R_D spectra are required to be established with respect to the initial period (T) of a structure for a series of damage levels (D) and ductility capacity classes (μ_u). To provide reliable R_D spectra, the influences of ground motion characteristics such as earthquake magnitudes, source-to-site distances and soil conditions need to be investigated using the actual ground motion records. Meanwhile, the selection of hysteretic patterns and structural detailing could also play an important role in the generation of the R_D spectra.

The R_D spectra can also be used in a reverse manner to estimate the SDOF damage or damage potential when the strength reduction from the elastic demand is known (Bozorgnia and Bertero 2003).

2.3.3 Hysteretic models

An appropriate hysteretic model is crucial in the prediction of the inelastic behaviour of a structure by a nonlinear time history analysis. Several hysteretic models (e.g., Clough 1966; Takeda et al. 1970) have been developed in the past to describe the actual behavioral stages of an element or a structure, such as initial or elastic, cracking, yielding, stiffness and strength degrading stages, and crack and gap closures. A more general hysteretic model was proposed by Park et al. (1987). This hysteretic model incorporates three parameters, α_s , β_e and γ_p in conjunction with a trilinear moment-curvature envelope to establish the rules under which

inelastic loading reversals take place. The values of the three parameters determine the properties of the stiffness degradation, strength deterioration (hysteretic energy-induced) and pinching behaviour, respectively. Figure 2.2 depicts an example hysteretic pattern.

The three-parameter hysteretic model has been calibrated and adopted in many previous studies (Kunnath et al. 1992; Bracci et al. 1995; Kunnath et al. 1995; Valles et al. 1996). A variety of hysteretic patterns can be achieved through a combination of these three parameters. For example, using $\alpha_s=2.0$, $\beta_e=0.1$ and $\gamma_p=1.0$ would yield a good simulation of the inelastic behaviour of normal RC members (Kunnath et al. 1990; Kunnath et al. 1995).

Figure 2.2 Three-parameter Park model: (a) example hysteretic loading path; (b) stiffness degradation; (c) strength deterioration; (d) pinching (Park et al. 1987; Valles et al. 1996)

2.3.4 Typical damage models

Damage indices are valuable tools to quantify the structural damage for damage estimation in seismic design and evaluation, by which different design or retrofit approaches can be compared objectively. This section presents a review of typical damage models.

Numerous studies have been conducted in the past to formulate appropriate damage indices capable of quantifying numerically the level of damage caused by an earthquake. Damage indices may be formulated locally, using the hysteretic characteristics of a particular member or cross-section (Park and Ang 1985; Chung et al. 1989; Krätzig et al. 1989; Chai et al. 1995; Rao et al. 1998), or globally for an entire structure, usually by considering changes in the modal parameters of the structure (Dipasquale and Cakmak 1988; Ghobarah et al. 1999). According to Williams and Sexsmith (1995), the cumulative damage models can be classified into three categories:

- (a) Deformation-based cumulative indices (e.g., Banon et al 1981; Stephens and Yao 1987; Jeong and Iwan 1988; Chung et al. 1989);
- (b) Energy-based cumulative indices (e.g., Gosain et al. 1977; Krätzig et al. 1989);
- (c) Combined indices (e.g., Park and Ang 1985; Bracci et al. 1989; Kunnath et al. 1992).

Typical cumulative damage models in the individual categories are introduced as follows.

Jeong and Iwan's Model (Jeong and Iwan 1988)

Jeong and Iwan (1988) proposed a damage model using the concept of classical low-cycle fatigue formulation. The number of cycles to failure at a given ductility, n_f , is found from a relationship similar to the Coffin-Manson law:

$$n_f \mu^s = c \quad (2.4)$$

where the constants s and c are suggested to be 6 and 416, respectively. The damage index is found using Miner's rule for combining the effects of cycles at varying amplitude:

$$D_{II} = \sum_i \frac{n_i}{n_{f,i}} = \frac{n_i \mu_i^s}{c} \quad (2.5)$$

Krätzig's Model (Krätzig et al. 1989)

Krätzig et al. (1989) proposed a damage index in terms of energy ratios. The energy dissipated during the cyclic process is normalized with respect to the energy dissipated under monotonic loading. The damage index reaches 100 percent at the ultimate displacement and zero percent damage at zero displacement. The governing equation for the damage index is

$$D_Q^\pm = \frac{\sum E_{Pi}^\pm + \sum E_{Si}^\pm}{E_U^\pm + \sum E_{Si}^\pm} \quad (2.6)$$

where E_{Pi} is the energy dissipated by a primary half-cycle; E_{Si} is the energy dissipated by a follower half-cycle; E_U is the maximum energy dissipated under monotonic loading; and $\pm$ indicates the positive or negative phase of cyclic deformation. Using Eq.(2.6), for each positive and negative phase of displacement, two D_Q values can be determined. A unique damage indicator is thus specified as

$$D_{KMM} = D_Q^+ + D_Q^- - D_Q^+ D_Q^- \quad (2.7)$$

As indicated in Eq.(2.6), this damage model is capable of accounting for both deformation and fatigue-type damage, since a high value of the damage index can be generated either by a single high-amplitude cycle or by repeated cycling at lower amplitude.

Park & Ang damage model (Park et al. 1985)

The Park & Ang damage model accounts for damage due to maximum inelastic excursions, as well as damage due to the cumulative effect of many inelastic cycles. Both components of damage are linearly combined. The damage index for a structural element was defined as:

$$D_{PA} = \frac{\delta_m}{\delta_u} + \beta \frac{E_h}{F_y \delta_u} \quad (2.8)$$

where δ_m is the maximum deformation; δ_u is the ultimate deformation; F_y is the yield strength; β is the constant parameter and E_h is the cumulative hysteretic energy. The constant β reflects the effect of cyclic loading on structural damage. It can be considered as the rate of damage accumulation through hysteretic energy dissipation. $\beta=0.1$ has been suggested for nominal strength deterioration (Park et al. 1987).

A modified version of the Park & Ang model was developed in terms of moment and rotation (Kunnath et al. 1992).

$$D_{MPA} = \frac{\theta_m - \theta_y}{\theta_u - \theta_y} + \beta \frac{E_h}{M_y \theta_u} \quad (2.9)$$

where θ_m is the maximum rotation attained during the loading history; θ_u is the ultimate rotation capacity; θ_y is the yield rotation; M_y is the yield moment; β is taken as the strength degradation parameter β_e in the hysteretic model as introduced in the above section; and E_h is the dissipated hysteretic energy at the member end.

Among the damage models mentioned above, the Park and Ang model is conceptually more attractive since it combines the effect of maximum deformation and the effect of cyclic loading by the hysteretic energy in an explicit manner. The Park and Ang model has been calibrated against a relatively large experimental database and used in many previous studies (Park et al. 1987; Gunturi and Shah

1992; Stone and Taylor 1994; Kunnath et al. 1995a, 1995b; Bracci et al. 1995). For the above reasons, the present study also adopts this damage model to quantify the damage of SDOF systems and the actual structures. More detailed discussion on the Park and Ang damage model will be presented in Chapter 3 and 4 where the model is applied.

2.4 Prediction of seismic deformation demands

At the current stage, deformation is still used as a primary parameter for performance design and evaluation. As a matter of fact, for very well detailed structures subjected to short-duration ground motions, the cumulative damage may play an insignificant role; and in such cases, the damage indices with or without considering the cumulative damage could make a marginal difference. Therefore, the prediction of seismic deformation demands constitutes an important task in the performance based design and evaluation.

2.4.1 Nonlinear static (pushover) analysis

The nonlinear static (pushover) analysis is a popular tool for predicting seismic demands of existing and newly designed structures. This approach has been adopted in the current structural evaluation provisions such as FEMA-356 (BSSC 2000) for the estimation of seismic demands in conjunction with seismic response spectra. The pushover analysis may be classified into the following three categories (Chintanapakdee and Chopra 2003):

a) Ordinary pushover analysis

The ordinary pushover analysis uses an invariant lateral force distribution and it is widely used in design offices. It is expected that using several lateral force distribution patterns would be adequate to bound the inelastic responses of structures. As suggested in FEMA-356 (BSSC 2000), at least two lateral force patterns should be selected among the recommended patterns, namely, effective design lateral forces (ELF), the first mode, uniform, and SRSS distributions. This

kind of pushover analysis is simple and yet can provide some essential response quantities for low-to-medium rise structures. However, the pushover analysis with an invariant lateral force pattern is not capable of effectively reflecting the higher mode effects and the redistribution of inertia force due to structural yielding.

b) Adaptive pushover analysis

To overcome the limitation of the ordinary pushover analysis, the adaptive pushover analysis was developed to follow more closely the time-variant distributions of inertia forces (Bracci et al. 1997; Gupta and Kunnath 2000). Although such method may provide better estimates of seismic demands, its implementation is complicated for design applications.

c) Modal pushover analysis (MPA).

More recently, the modal pushover analysis (MPA) was proposed for seismic demand estimation (Chopra and Goel 2002). In the MPA procedure, the higher mode effects are taken into account by performing the pushover analyses with lateral force patterns related to a sufficient number of modes of vibration, and the final demand is determined by combining the peak ‘modal’ responses with an appropriate modal combination rule, such as the Square Root of the Sum of the Squares (SRSS) rule. Comparing the median results from the MPA procedure and the time history analyses using individual ground motions, the MPA method was shown to provide better estimates than the first ‘mode’ pushover analysis.

2.4.2 Methods for estimation of seismic displacement demands

The existing methods for approximately estimating the seismic displacement response of structures may be classified into three categories (Lu et al. 2006): (1) based on static pushover analysis (Fajfar & Fischinger 1989; ATC 1996; NEHRP 1997; Chopra and Goel 2002); (2) based on analytical solution of simplified structural model (Saiidi and Sozen 1981; Iwan 1997); and (3) based on regression function from the results of response history analysis (Miranda 1999; Gupta & Krawinkler 2000).

Category 1

With a static nonlinear pushover analysis, an analyst may acquire detailed information about the distribution of local response quantities including the storey drift and element end rotation. Higher mode effect can also be reflected by combining the contribution from the first two or three modes with different lateral force distributions. Through appropriate transformation, an equivalent SDOF system can be obtained. Using the response spectrum in conjunction with a strength reduction factor, the inelastic acceleration and displacement demand on the equivalent SDOF system can be established. The inelastic displacement is then converted back to the MDOF system, through another push-over analysis if necessary, to determine the deformation demands at the storey and member levels.

The static pushover analysis procedure recommended in FEMA273 (NEHRP 1997) is based on the assumption of fundamental-mode controlled response and invariant lateral force distribution. A modal pushover analysis (MPA) procedure (Chopra & Goel 2002) can be performed to incorporate higher mode effects and redistribution of inertia forces after structural yielding.

Category 2

The methods in this category simulate the structure by a SDOF model or some other simplified models. Based on structural dynamics theory, the analytical solutions of the displacement response of these simplified models are obtained. These methods are generally simple in concept and computationally economical. Some methods, e.g. Saiidi & Sozen (1981), also require a static nonlinear pushover analysis to provide the information of global behavior for the transformation from MDOF model to SDOF model.

Saiidi & Sozen (1981) reported a simple analytical model with low computational cost for the estimation of displacement histories of multistory RC structures subjected to strong ground motions. This so-called Q-Model was based on the following two simplifications: (1) reduction of a MDOF structural system to a SDOF oscillator; (2) approximation of the varying incremental stiffness properties

of the entire structure by a single nonlinear spring. Applying an inverted triangular lateral force distribution, a nonlinear static pushover analysis was employed to obtain the deformation shape and the force-displacement relationship of the structure. Subsequently, a very simple hysteretic model was used to define the incremental stiffness properties.

For structures subjected to near-field ground motions, the coherent pulse-like nature of the near-field ground motion time history may cause the maximum response of the structure to occur before a resonant mode-like response can build up. Therefore, the response spectrum based on SDOF model may not be able to adequately estimate the concentration of the inter-storey drift demand of near-field type ground motions. Acknowledging this problem, Iwan (1997) developed a linear drift spectrum based on a continuous shear-beam model rather than a SDOF model. For the continuous shear-beam structure, the inter-storey drift ratio is equal to the shear strain $\partial u / \partial y$, where $u = u(y, t)$ denotes the displacement relative to the base; y and t are the vertical distance measured from a certain point of the shear-beam to the base and time, respectively. The coordinate of drift spectrum is represented by the maximum shear strain or inter-storey drift ratio $D = \max_{y,t} |\partial u / \partial y|$.

Comparison of the elastic response spectrum (ERS) and drift spectrum showed that for structures with a short period (less than 2.0s), the single-mode ERS approximation gives satisfactory results for inter-storey drift ratios. However, the single-mode ERS significantly underestimates the drift demand for longer period structures. It has also been found that the results of the drift spectrum are in general agreement with results of detailed numerical analyses of prototype structures.

Category 3

Methods falling into this category can also be referred as the coefficient methods. Several modification factors, derived from regression analysis of time history analysis results, are used to account for the MDOF effect, inelasticity effect, P-delta effect, drift concentration effect, and so on. Such procedures are generally more

straightforward and easier to apply in design applications than those of the previous two categories, provided that appropriate parameters reflecting the true nature of structural properties are selected and then derived by regression analysis from sufficient time history analysis results.

Miranda (1999) proposed an approximate method to estimate the maximum lateral deformation demands in multi-storey buildings responding primarily in the fundamental mode. In this method, the multi-storey building is modeled as an equivalent continuum structure consisting of a combination of a flexural cantilever beam and a shear cantilever beam. The simplified model was used to investigate the ratio of the spectral displacement to the roof displacement and the ratio of maximum inter-storey drift ratio to the roof drift ratio. Four parameters were proposed to obtain nonlinear inter-storey drift ratio from elastic displacement spectrum. Thus, the roof drift and the maximum inter-storey drift ratio can be expressed as

$$\Delta_{roof} = \beta_1 \beta_3 S_d \quad (2.10)$$

$$\theta_{max} = \beta_2 \beta_4 \frac{\Delta_{roof}}{H} \quad (2.11)$$

where S_d is the displacement spectrum ordinate, β_1 is the mode participation factor, β_2 is the amplification factor accounting for the concentration of inter-storey drifts, β_3 is the nonlinear amplification factor from the maximum inelastic displacement to maximum elastic displacement; for elastic response, $\beta_3 = 1$, and β_4 is the factor accounting for the increase of concentration of inter-storey drift ratio due to nonlinear response. β_4 is related to the number of stories, nonlinear response level and type of storey mechanism.

The storey where the maximum elastic inter-storey drift ratio occurs can be located by the solution of a differential equation describing the displacement behavior of the simplified elastic structural model. However, the weak storey during inelastic response may not be the same as that within elastic range, especially for a structure with significant irregularity in the distribution of storey stiffness and strength. It is

desired to find a quick way to determine the weak storey in both elastic and nonlinear response stages.

Gupta & Krawinkler (2000) evaluated the modification factors related to the estimation of seismic roof and inter-storey drift demands. The spectral displacement demand is related to the roof drift demand for multi-storey structures using three modification factors, accounting respectively for MDOF effects, inelasticity effects, and P-delta effects. It was found that the roof displacement demand of an inelastic structure can be estimated with good confidence, provided that the P-delta effect is sufficiently contained to avoid drifts associated with negative post-yield storey stiffness. The relationship between the roof and maximum storey drift demand was found to depend on the height of the structure, and the distribution of storey drifts over height depends strongly on ground motion and structure characteristics. No simple rules for the generation of distributions were recommended, and a pushover analysis is required to provide details of structural behavior.

2.5 Performance of structures with nonseismic details

As a special case, the performance of non-seismically designed structures, as seen in many low-to-medium seismicity regions, has attracted increasing attention in recent years due to an increase of knowledge about the real seismic hazard and awareness about the possible economic consequences of earthquake induced damage to buildings and infrastructure systems.

Experimental data forms the basis for understanding the likely behavior of nonseismically detailed structures in the event of an earthquake and developing more reliable analytical models to predict the seismic damage to these structures. In this section, previous experimental studies pertaining to nonseismic design, including tests of columns, beam-column joints and reduced scale structural models are reviewed. The performance of such components is characterized in terms of strength and ductility capacity as well as failure mechanism.

2.5.1 Performance of columns with nonseismic details

To maintain the stability of the structures and their axial load carrying capacities during an earthquake, most seismic design codes adopt the strong column-weak beam concept to ensure the formation of major plastic hinges in beams. Adequate confining reinforcement is provided in the potential plastic hinge regions in columns to achieve ductile behaviour. However, plastic hinge may still occur in columns, especially at the bottom of the ground storey columns.

In the literature, only a limited amount of experimental data about nonseismically designed columns or lightly reinforced columns are available. In the study by Saatcioglu and Ozcebe (1989), one of 14 full-scale (350mm) square RC columns, designated as U2, was designed to conform to ACI 318-83 requirements for an ordinary structural member that does not form part of a ductile moment-resisting frame in a seismically active region. The seismically designed specimen U4 is also selected herein for comparison purpose.

Several lightly reinforced columns were examined by Watson and Park (1994). In their study, the columns were designed with less quantity of confining reinforcement in the potential plastic hinge regions than recommended by ACI 318-89 and NZS 3101 for zones of high earthquake risk. The original column units consisted of two end columns and one central stub that simulated the presence of a beam, a footing, or a pile cap. For convenience of comparison with other column specimens, the column units considered herein have been converted to cantilever columns and the effect of some tilting of the central stub is ignored.

In a study by Aycardi et al. (1994), two one-third reduced scale reinforced concrete columns were designed in accordance with ACI 318-89, but without the seismic provisions. The lap splices was placed in the potential plastic hinge of one column and the corresponding spacing of transverse reinforcement was reduced by a half. The columns were tested under simulated seismic loading to investigate their inelastic behavior.

Table 2.1 summarizes the above-mentioned columns with nonseismic details, where n_0 is axial force ratio and ρ_t is the volumetric ratio of transverse reinforcement. A common feature in typical nonseismically detailed columns is the low quantity of transverse reinforcement compared with the requirement of ACI 318-89. Besides, lap splices often occur in the potential plastic region. Typical lateral force vs. displacement relationships from the above test investigations are shown in Figure 2.3. Discussion on the observed behavior follows.

Table 2.1 Selected columns with nonseismic details

Reference	Specimen	ρ_t (%)	ρ_t (required in ACI 318- 89) (%)	n_0	Stirrup pattern	Designation	Calculated Ductility
Saatcioglu and Ozcebe (1989)	U2	0.85	1.90	0.16		Saat89u2	2.0
Watson and Park (1994)	Unit 2	1.10	2.00	0.3		Soes8602	6.0
	Unit 3	0.80	2.20	0.3		Soes8603	5.3
	Unit 5	1.06	1.83	0.5		Wat89u5	4.2
	Unit 6	0.57	1.97	0.5		Wat89u6	3.4
Aycardi et al. (1994)	1	0.73 ^a	1.63	0.39		Ayca9401	4.4
	2	0.37 ^b	1.63	0.3		Ayca9402	5.8

Notes: a- with lap splices; b- without lap splices

(a) Saat89u2

(b) Saat89u4

(c) Soes8602

(d) Soes8603

(e) Wat89u5

(f) Wat89u6

Figure 2.3 Lateral load vs. displacement relationships of selected column specimens (Saatcioglu and Ozcebe 1989; Watson and Park 1994; Aycardi et al. 1994)

Figure 2.3 (continued)

2.5.1.1 Strength and ductility capacity

The capacity of the aforementioned columns subjected to simulated earthquake loading can be characterized in terms of their ductility and strength. The yield displacement is defined in a way as shown in Figure 2.4, in which the curve is the average of the envelope curves in the first and third quadrants of the original load vs. displacement hysteretic relationship.

Figure 2.4 Definition of displacement ductility

As shown in Figure 2.3, Specimen Saat89u2 exhibited rapid strength deterioration after reaching a ductility factor of 2.0, and this was attributed to low transverse confinement level and the poor tie configuration that contained only perimeter

hoops without crossties. However, in the study by Watson and Park (1994), the ductility factors of Unit 2, 3, 5 and 6 are found to be 6.0, 5.3, 4.2 and 3.4, respectively. In these cases, the achieved ductility is not apparently low despite the relatively small quantity of the transverse confining reinforcement. Significant pinching was observed for Specimen Ayca9401. This may be attributed to the column longitudinal reinforcement lap splice taking place in the plastic hinge region. The ductility is 4.4 at the failure stage. Specimen Ayca9402 without lap splices showed superior energy-dissipation characteristic and the achieved ductility reached 5.8 at the failure stage.

The above summary of the relevant test results tend to indicate that the ductility factors in columns with nonseismic details need not be considerably low as far as the behavior at the column level is concerned. However, as can be expected, the ductility factors decrease as the level of axial load increases. Further experimental evidences are required to arrive at a better quantification of the ductility capacity in the nonseismically detailed columns.

2.5.1.2 Effect of axial load

Axial force has a significant effect on the strength degradation. As illustrated in Figure 2.3, Column Saat89u2 deteriorated rapidly and the strength loss resulting from the three cycles at $2\Delta_y$ was approximately 50 percent. For Unit 2, 3, 4 and 5 (Watson and Park 1994), the ductility capacities decrease with the increase of axial load although higher lateral strength was obtained for the columns under higher level of axial load. Among these four columns, the lowest ductility factor was found to be 3.4 due to higher axial force ratio of 0.5.

From the viewpoint of failure mechanism, failure due to rupture of the longitudinal bar and transverse hoop occurred to Specimen Soes8602 and Soes8603 with an axial force ratio of 0.3. However, for Specimen Wat89u5 and Wat89u6, the eventual failure modes mainly featured the buckling of the longitudinal reinforcement due to high axial force ratio of 0.5. On the other hand, both Specimen

Ayca9401 and Ayca9402, under axial force ratio of 0.39 and 0.3, respectively, failed due to the crushing of concrete as a result of lack of confinement in the plastic hinge region, along with buckling of the longitudinal bars. Specimen Ayca9401 failed above the lap splice since the stirrups have been doubled within the lap splice region, and Specimen Ayca9402 failed about 76mm ($0.75h_c$) from the base. In both cases $s_0/d_b = 17.8$ is considerably greater than the anti-buckling requirement of $s_0/d_b = 8$ recommended by ACI 318-89.

The axial force generally accelerates the strength and stiffness degradation of columns under reversed cyclic loading. Due to insufficient transverse confinement in the non-seismic columns, the buckling of longitudinal reinforcement can be a common failure mechanism for the columns even under moderate axial force ratio of 0.3.

2.5.2 Performance of beam-column joints with nonseismic details

A number of studies on seismic performance of beam-column joints with nonseismic details or substandard reinforcing details have been carried out (Englekirk and Huang 1992, Beres et al. 1992, Aycardi et al. 1994, Hakuto et al. 2000, Li et al. 2002). Interior and exterior connections with structural detailing according to the old design codes or in-situ surveys were investigated.

2.5.2.1 Typical detailing of nonseismic beam-column joints

The performance of some full-scale interior and exterior joints with nonseismic details was studied by Beres et al. (1992) in support of the assessment of the seismic risk in regions of moderate seismicity, such as the central and eastern United States. The structures were generally designed according to earlier versions of the ACI 318 codes. In their study, the column stirrup spacing of 203mm exceeded the upper limit of 0.25 times the minimum member cross-sectional dimension as specified by ACI 318-89. The presence of the lap splicing in the vicinity of joints also did not conform to the modern seismic design.

Aycardi et al. (1994) performed unidirectional, quasistatic lateral load tests on one exterior and one interior 1/3-scale beam-column joint designed only for gravity loads. The specimens included a slab and transverse beams on both sides. The reinforcement detailing was similar to the cases tested by Beres et al. (1992).

Hakuto et al. (2000) investigated the performance of some poorly detailed reinforced concrete interior and exterior beam-column joints, typical of pre-1970s designed moment-resisting frames in New Zealand. In this case, the nonseismic details featured no transverse reinforcement in the joint core. All the specimens were tested without axial load.

Reinforced concrete structures consisting of wall-like wide column elements are very common in regions of low to moderate seismicity and actually form the predominant structural systems in Singapore and Malaysia. The seismic behavior of four full-scale reinforced concrete interior beam-wide column joints with nonseismic details were investigated by Li et al. (2002). The specimens were designed according to BS8110 without consideration of seismic loading.

Based on the nonseismic details reported in the above studies and the comparison with modern seismic provisions such as ACI Code (ACI 2002), the common features of nonseismic detailing in reinforced concrete structures may be summarized as follows:

- 1) Columns may be weaker than the adjacent beams, potentially leading to a soft storey or column sidesway mechanism;
- 2) Column main bars are lap spliced just in the potential plastic hinge region above the floor level;
- 3) Minimal transverse reinforcement in columns for shear and confinement, particularly in the plastic hinge region;
- 4) Beam bottom bars are terminated and lap spliced within the joint core;
- 5) There is little or no joint transverse reinforcement within the joint core; and
- 6) For the exterior joint, the bottom beam bars may not achieve adequate development length, or the hooks at the ends of beam longitudinal bars are bent

out of the joint core in some cases.

The above features are schematically illustrated in Figure 2.5. The individual features of these selected specimens are summarized in Table 2.2 and discussion of the observed behavior follows.

Figure 2.5 Typical nonseismic details of interior and exterior beam-column connections

Table 2.2 Beam-column joints with nonseismic details

Reference	Specimen	Reinforcing details	Axial force Ratio n_0	Ductility
Englekirk and Huang (1992)	Interior	(a), (d)	0.16	4.0
Beres et al. (1992)	Interior	(a), (b), (c), (d), (e)	0.39	2.9
	Exterior	(a), (b), (c), (d), (f)	0.39	2.7
Aycardi et al. (1994)	1/3 reduced scale interior Stage 1	(a), (b), (c), (d), (e)	0.13	3.5
	1/3 reduced scale exterior	(a), (b), (c), (d), (e) (f)	Varied load	3.0
Hakuto et al. (2000)	Interior O1	(a), (c), (e)	0	2.0
	Interior O4	(c), (e)	0	4.0
	Exterior O6	(c), (e)	0	10.0
	Exterior O7	(c), (e), (f)	0	2.0
Li et al. (2002)	Interior A1	(a), (c), (e)	0	2.0
	Interior A2	(c), (e)	0	2.0

2.5.2.2 Failure mechanisms

For typical nonseismically detailed interior joints, the initiation of damage and the subsequent strength degradation may be attributed to the high joint shear in the absence of transverse reinforcement in the beam-column joint core. Shear failure of the joint core eventually occurred to Specimen O1 (Hakuto et al. 2000) and Specimen A1 (Li et al. 2002). In both cases, the theoretical flexural strength of the column was lower than that of the beam. This resulted in severe joint diagonal tension cracking and bond deterioration along the beam and column bars. However, the lack of proper seismic details in some other aspects may lead to some other failure mechanisms as well. For Specimen O4 (Hakuto et al. 2000), shear failure of the beams occurred first during the cyclic loading as only little transverse reinforcement was provided in the beams. Specimen A2 (Li et al. 2002) failed due to beam flexural behavior which was characterized by much wider beam flexural cracks at the column faces and concrete spalling in the beam compression zones. Meanwhile, under high level of axial load ($0.39 f'_c A_g$), the buckling of the column longitudinal bars at the lightly confined splice region was reported by Beres et al. (1992).

For exterior joints, the compression strut failure mechanism of the joint core may not be able to develop because of inadequate reinforcement details in the joint core or in the adjacent beam. Unit O6 (Hakuto et al., 2000) failed due to the shear failure in the beam at the final stage of testing. However, the other exterior joint (Unit O7), which was identical except that the ends of the hooks of the beam longitudinal bars were bent out of the joint core, failed in shear after the diagonal tension cracks in the joint core widened and connected to the bond splitting cracks along the column outer bars. The same result as Unit O7 was also reported by Beres et al. (1992).

2.5.2.3 Ductility capacity

Based on the measured lateral force versus lateral displacement relationship, the

ductility capacity of the entire joint specimen can be evaluated. For Unit O1, as reported by Hakuto et al. (2000), shear failure of the joint core occurred at a ductility of 2 with strength degradation of about 27 percent in accordance with the failure criteria commonly employed in New Zealand Code. In this case, the diameter of the longitudinal beam bars to column depth ratio was as high as 1/12.5. This resulted in severe bond slip of the longitudinal beam bars through the joint. For Unit O4, shear failure of the beams commenced during the first loading cycle at a displacement ductility of factor of 4. With the retrofitted beams, the joint was retested and eventually failed at the displacement ductility factor of 6. However, Unit O7 failed in shear shortly after the development of diagonal tension cracking at a ductility factor of 2. The theoretical horizontal load strength of the beam for Unit O7 was even not reached in either direction of loading.

Similar results were reported by Li et al. (2002). For Specimens A1 and A2, severe strength degradation occurred at a ductility of 2. In both cases, significant pinching in the hysteretic loops was observed due to severe bond deterioration along the beam and column bars and the joint diagonal tension cracking.

As was expected, the ductility capacities of the beam-column joints with nonseismic details are quite limited. Based on the results from the previous joint tests as summarized above, it seems to be reasonable to expect an ultimate ductility factor of around 2 for such joints under reversed cyclic loading.

2.5.2.4 *Shear strength capacity*

Although the initiation of damage in some specimens was attributed to reasons other than high joint shear as mentioned in Section 2.5.2.2, the maximum joint shear stress v_{jh} may still be treated as a good basis for evaluating the joint strength capacity,

$$v_{jh} = \frac{V_{jh}}{b_j h_c} \quad (2.12)$$

where V_{jh} is the joint shear force; b_j is the effective joint width as defined in Figure 2.6; and h_c is the column depth.

Figure 2.6 Definition of effective joint width (Paulay and Priestley 1992)

Based on the analysis of the joint shear stress from a number of test results, the relationship of the maximum joint shear stress v_{jh} and the concrete cylinder compressive strength f'_c at the stage of interior joint failure was proposed by Hakuto et al. (2000). It can be expressed as

$$v_{jh} = 0.17 f'_c, \text{ or } 1.0 \sqrt{f'_c} \text{ (MPa)} \quad (2.13)$$

In the study by Li et al. (2002), the maximum nominal horizontal shear stress v_{jh} in the as-built interior joints (A1 and A2) are $0.15 f'_c$ (or $0.84 \sqrt{f'_c}$ MPa) and $0.11 f'_c$ (or $0.61 \sqrt{f'_c}$ MPa), respectively. A relatively high value of $0.20 \sim 0.24 f'_c$ (or $1.0 \sim 1.2 \sqrt{f'_c}$ MPa) was obtained for interior joints with continuous beam bottom bars in the study by Beres et al. (1992). Despite some variations, however, generally speaking all the results obtained from different test programmes showed a reasonable correlation with Eq.(2.13). It has been concluded that the shear failure of beam-column joints without transverse reinforcement is due to extensive diagonal

tension cracking that eventually leads to diagonal compression failure in the joint core.

2.5.2.5 *Effects of lap splices*

Whether lap splices in the beam bottom bars would affect the joint behavior or not mainly depends on the lap splice length. Li et al. (2002) reported that the lap splice detail in the joint core were not detrimental to the performance of beam bottom bars in their as-built interior joint specimens. This is probably due to the fact that a sufficient lap length of more than 300mm was provided. Similar observation was also made by Englekirk and Huang (1992). In their study, the failure of the lap spliced beam bottom bars also did not happen as a long lap splice length of $30 d_b$ was adopted for the anchorage of beam bottom bars, where d_b is the diameter of the beam longitudinal reinforcement. However, problems may arise if the splice length is insufficient. In the full scale beam-column joint tested by Beres et al. (1992), the discontinuous bottom beam reinforcement extended only about 150mm ($6.8 d_b$) into the joint. As a result, the beam bottom reinforcement pulled out from interior joint. Similarly, the pull-out of the beam bottom reinforcement from the exterior joint was observed in the 1/3 scale model test conducted by Aycardi et al. (1994), where the anchorage length was only 50mm which was equal to $8.7 d_b$.

For the lap splices in the column just above the floor level, splitting failure are likely to occur during reversed cyclic loading. The splitting patterns, as reported by Lukose et al. (1982), are mainly determined by the rebar size and spacing. For Specimen A1 (Li et al., 2002), the lap splice of the column main bars failed during the testing. The splitting plane at failure extended completely across the plane of the splices leading to a simultaneous failure of the splices at one level. Note that this specimen had a large rebar size and small clear splice spacing equal to 75mm or $3.0 d_b$. For Specimen A2 which had large clear spaces between adjacent column bars, the lap splice failure of column main bars did not occur.

2.5.2.6 Effect of axial force

In some of the tests discussed above (Beres et al. 1992; Englekirk and Huang 1992 and Aycardi et al. 1994), the axial forces were applied to the columns of the interior and exterior joints, and the corresponding axial force ratios were 0.39, 0.17 and 0.13, respectively. In general, as mentioned by Beres et al. (1992), higher level of axial force reduced the deformations, produced higher initial stiffness and an increase in capacity, but it also resulted in the buckling of the column longitudinal bars at the lightly confined splice region and accelerated the shear crack extension within the exterior joint.

2.5.3 Overall behavior of structures from reduced scale model tests

Bracci et al. (1995) conducted a three-storey 1/3-scaled model test to evaluate the overall performance of structures with strength and reinforcing details compatible with regular (nonseismic) provisions of ACI 318-89. The model was tested using simulated earthquakes representing minor, moderate, and severe risks for low to moderate earthquake zones.

The ratio of base shear to structure weight was 6.5 percent for minor shaking and 15.3 percent for severe shaking. The overall structural response was dominated by weak column-strong beam behavior since the slab had a significant contribution toward the moment capacity of beams. Some pullout of positive beam reinforcement was observed in the exterior joint, which can be attributed to inadequate anchorage development in the exterior joint. There is no other joint failure observed during severe shaking. It was concluded that generally there existed inherent lateral strength and flexibility of such structures which can resist minor earthquake without significant damage. For moderate to severe earthquakes, these structures may still withstand the seismic forces but at the expense of substantial side-sway deformations that can exceed the recommended limits. For instance, large storey drifts of more than 2.0 percent occurred on the first and second floors under severe shaking of 0.3g.

A similar model at a 1/8 scale was investigated by Beres et al. (1992) on the shaking table of Cornell University to study the overall flexibility and stiffness deterioration of lightly reinforced concrete frame buildings designed to resist gravity loads only. Similar conclusions as mentioned above were reached. However, the $P-\Delta$ effect was highlighted since non-seismically designed reinforced concrete building with no infill walls may experience large lateral deformations during a moderate earthquake.

Although the experimental studies mentioned above provide a general evaluation of the behaviour of non-seismically designed structures under seismic conditions, there is still a lack of comparative information concerning the behaviour of structures with different possible detailing arrangements. Such structures also need to be viewed in the context of intermediate performance levels as the probable moderate seismic demand could be more concerned for low-to-moderate seismic regions.

2.6 Concluding remarks

Structural damage estimation, along with prediction of peak displacement response, constitutes an important part in the performance based design and performance evaluation. Despite numerical studies on the prediction of seismic displacement demand, continued efforts are required to improve the effectiveness and reliability of the prediction approaches, especially for multi-storey (MDOF) systems. The evaluation of damage in the context of performance based design is still at a developing stage, and a systematic methodology from the establishment of damage-based demand spectra to the distribution of damage in a multi-storey system requires focused study.

With regard to the spectral demand at the SDOF system level, the emerging damage-based strength reduction factor (R_d) is capable of associating the design strength with the desired damage level in a direct manner. Further study is needed

to investigate the effect of the earthquake characteristics such as earthquake magnitude, source-to-site distance and site condition as well as hysteretic patterns and structural detailing. As the damage-based spectra can be employed in a reversed manner to evaluate the potential damage, it is possible to develop a procedure for a rapid estimation of damage in multi-storey systems on the basis of the equivalent SDOF damage identified from the damage-based spectra.

Nonseismically-designed structures have various drawbacks from a strong earthquake point of view. However, experimental evidences on individual components and beam-column joint assemblages tend to indicate that these structures possess some inherent ductility and not all the potential problems with the non-seismic details will pose threats at low to moderate response levels. Therefore, the assessment of non-seismically designed structures with respect to the corresponding seismic hazard levels may be more of a performance and damage issue than safety issue. For this special class of performance and damage evaluation problems, more experimental data about the potential lateral strength and deformation capacities of nonseismically designed structures, especially under realistic seismic ground motions, are needed.

CHAPTER 3

DAMAGE-BASED STRENGTH REDUCTION FACTOR FOR STRUCTURAL DESIGN INCORPORATING PERFORMANCE CONSIDERATIONS

3.1 Introduction

The requirement for ductile behaviour is the foundation in the modern seismic design. With adequate ductility, a certain degree of damage can be tolerated during moderate to severe earthquakes as far as safety of the structure is concerned. Accordingly, the required design strength can be reduced from the elastic strength demand, for example through a response modification factor, R , as adopted in the 2003 NEHRP recommended provisions (BSSC 2003). The existing specifications of the R factor or the likes are based on engineering judgment and accumulated experiences from the past earthquakes and they are intended to account for ductility, overstrength, redundancy and damping of the structural system. Much research has been conducted (e.g., Uang 1991; Whittaker et al. 1999) in attempt to correlate the response modification factor with the above factors. While the influences of the other factors appear to involve more ambiguities in quantitative terms, the ductility, being a primary contributor, has a clear physical connection to the response modification factor. The present study focuses on the ductility related response reduction.

In the literature, the response reduction factor based solely on ductility is known as ductility-based strength reduction factor (SRF), denoted as R_μ . R_μ is defined as the ratio of the elastic strength demand F_e to the inelastic strength demand $F_{y,\mu}$ by which the required ductility on the system is μ for a prescribed level of ground

motions. The R_μ factor has been the subject of numerous investigations (e.g., Newmark and Hall 1982; Krawinkler and Nassar 1992; Miranda 1993; Vidic et al. 1994). The design spectra built using the R_μ factor are known as ductility-based demand spectra.

As a matter of fact, the cumulative damage resulting from the inelastic cycles also plays an important role in determining the damage state of a structure. In the past years, several approaches have been proposed for constructing the demand spectra for ultimate state design taking into account the cumulative damage. The equivalent ductility method proposed by Fajfar (1992) permits designers to choose an acceptable level of structural damage in an explicit manner. Krawinkler and Nassar (1992) and Jean and Loh (1998) proposed similar approaches by introducing a weighted ductility factor and an allowable ductility factor, respectively. The core idea among the above approaches is that, the cumulative damage can be accounted for by modifying (reducing) the available ductility and subsequently, the SRF (R_μ) can be specified through an appropriate $R_\mu - \mu - T$ relationship based on the modified ductility. T is the natural period of the system.

The demand spectra constructed from the above procedures may be considered as indirect damage based. Some other studies have proposed to employ a damage model directly to determine the seismic demand for a target damage level (Cosenza et al. 1993) or to check the validity of the traditional seismic design for ultimate state with respect to damage (Warnitchai and Panyakapo 1999).

From another perspective, the development towards the performance-based design poses a need to re-examine the strength demand from a broader scope of satisfying multiple performance targets. It is generally understood that the performances of a system are closely related to displacements; however, at inelastic stages the displacement response (including the history) in turn depends on the yield strength (or the response reduction factor in a normalized way), in addition to the stiffness. Therefore, the determination of the design strength in the context of performance based design needs not only to satisfy the ultimate state requirement as in the

conventional ultimate limit state design, but also to take care of its implications on other performance targets concerning structural performances. As will be elaborated later, this can be made possible by constructing damage based strength demand spectra for different damage levels.

In this chapter, the construction of direct damage based demand spectra in terms of the strength reduction factor (R_D factor) and their implications on the performance based design are discussed first. A series of R_D spectra for various damage levels are generated for a large collection of earthquake ground motions divided in different ways of grouping, and the effects of local site conditions, earthquake magnitudes, source-to-site distances and hysteretic patterns on the proposed R_D spectra are examined. Finally, the overall mean and statistical dispersion of the R_D spectra for different levels of damage are obtained, from which simple empirical formulas are proposed.

3.2 Damage-based strength reduction factor and its implications on structural performances

Similar to the ductility-based response reduction factor, R_μ , the damage-based strength reduction factor, R_D , is defined as

$$R_D = \frac{F_e}{F_{y,D}} \quad (3.1)$$

where F_e is the elastic strength demand; and $F_{y,D}$ is the inelastic strength required to limit the inelastic response of the system to a specified damage level D for a given system ductility capacity (class), μ_u . In most of the previous studies on this subject, the strength demand is solely associated with the ultimate limit state, which by default should correspond to $D = 1.0$ or a value close to 1.0, depending on the interpretation of the “ultimate state” in terms of damage.

The relationship of the R_D factor with the inelastic displacement, available ductility and the damage level may be conceptually illustrated in Figure 3.1 for a generic

system. In the diagram, line $o-a-e$ represents the envelope response (positive side) if the system remains elastic; line $o-a-b$ represents the envelope inelastic response (positive side), with point b defining the dynamic ductility demand. The shaded area indicates (schematically) the cumulative energy dissipation during all inelastic cycles. If the system has an available ductility capacity (herein we define the ductility capacity to be that under monotonic loading to avoid ambiguities. Other definition of ductility capacity is also possible) corresponding to point c , the damage incurred in the system by the inelastic response $o-a-b$ and the cumulative energy E_h can be determined by an appropriate damage model, as will be discussed later. Expressing the damage in general normalized terms, it has

$$D = D(\mu_u, \mu, \bar{E}_h) \quad (3.2)$$

where μ_u is the available ductility capacity, $\mu_u = \Delta_u / \Delta_y$; μ is the maximum dynamic ductility demand $\mu = \Delta_m / \Delta_y$; and $\bar{E}_h$ is the normalized hysteretic energy, $\bar{E}_h = E_h / F_y \Delta_y$.

Figure 3.1 Schematic relationship between strength reduction and inelastic response/damage

Apparently, the inelastic response of the system will depend on the yield strength (i.e., the R_D factor), the hysteretic behaviour, as well as the input ground motion and the dynamic properties (the natural period for a SDOF system) of the system. On the structural side, it becomes clear that, given elastic properties of the system that may be determined beforehand, the R_D factor will play a governing role in determining the inelastic response and thereby the damage status of the system so that for any specific ground motion input,

$$D = D(\mu_u, R_D) \quad (3.3)$$

It can be expected that for a given ductility class μ_u , higher R_D value generally results in higher structural damage D , while for a given R_D , the structural damage decreases as the ductility class increases. Thus, to a certain extent the structural damage can be controlled by selecting an appropriate pair of μ_u and R_D . Of course, the determination of the detailed relationship is a complex process as it involves the (cumulative) inelastic response which in turn depends on the characteristics of the ground motions. In the present study, we adopt a statistical approach to establish such a relationship, as will be described later.

In a conventional design, the design strength demand is solely determined according to the ultimate limit state requirement. In the context of performance based design, this may not always be the case when the potential effect of strength on the evaluation of damage is taken into account. This is because, although the hazard level (say $S_{a,i}$ in terms of spectral acceleration, where “ i ” denotes the performance level) corresponding to an intermediate performance level is lower than the ultimate level, the allowable structural damage D_i is also lower, implying a lower $R_{D,i}$. As a result, it is possible that the strength demand for the “ i -th” performance level, $S_{a,i}/R_{D,i}$, becomes larger than that for the ultimate level, $S_{a,u}/R_{D,u}$. Figure 3.2 illustrates two hypothetical scenarios for the ductility class of $\mu_u=4$ and $\mu_u=8$. The R_D factors in Figure 3.2(a) and the corresponding design strengths in Figure 3.2 (b) are calculated for the damage levels (I, II, III, IV and V) specified in Table 3.1.

According to the definition of R_D factor, the design strength (in terms of the base shear coefficient) F_y/W is equal to S_a/R_D , where W is the system weight. It can be observed that the design strength tends to be governed by an intermediate performance level.

Figure 3.2 Illustrative strength demands for different performance levels.
The top axis indicates the corresponding hazard levels (spectral accelerations)

Under the performance design framework, it is therefore appropriate to take into account the potential implications of the design strength on structural damage at other performance levels, in addition to the ultimate state consideration. In terms of the inelastic response spectra, this necessitates the generation of the R_D spectra for a range of damage levels for different ductility class systems.

As a matter of fact, the availability of a complete set of damage based R_D spectra could also be used for a rapid assessment of the general structural damage state for an existing structure or a design after proportioning. By calculating the actual R_D value for a given earthquake hazard level, taking into account the overstrength in the actual design, the expected damage level can be estimated from the R_D - T - μ_u - D curves in a reverse manner.

3.3 Definition of structural damage and its limit values for different Performance Levels

Performance-based frameworks such as FEMA-356 (BSSC 2000) provide the performance levels in terms of inter-storey drift ratio. In the present study, the performance of a generic structural system is defined using a damage index so that the cumulative damage in the structural system can be taken into account. A general review of typical damage models has been presented in Chapter 2. The well-known Park and Ang damage model, which was initially defined by Park and Ang (1985) and modified later by Kunnath et al. (1992), is employed and herein introduced in more detail. This damage model consists of a linear combination of normalized maximum displacement and hysteretic energy dissipation. Using the notation of ductility, it can be written as

$$D = \frac{\mu - 1}{\mu_u - 1} + \beta \frac{E_h}{F_y \Delta_y} \frac{1}{\mu_u} \quad (3.4)$$

where β is a constant representing the rate of damage cumulation through hysteretic energy due to cyclic loading. β may be associated with the strength degradation of hysteretic behaviour. For poorly detailed systems, β should take higher value. Negro (1997) evaluates the typical β values concerning the global behavior of structures through experimental assessment. For high ductility structures, a β value of 0.07 was suggested. In the present study, β is taken equal to 0.1 to represent a basic ductile design unless specified otherwise.

The damage index expressed in Eq. (3.4) has been calibrated against a large collection of observed seismic damage (Park et al. 1985; Park et al. 1987; De Leon and Ang 1994; Stone and Taylor 1994). It is generally demonstrated that $D = (0.4 \sim 0.5)$ represents a limit state for repairable damage, while D approaching 1.0 represents total collapse.

In terms of performances, there exist several versions of performance definitions. For example, FEMA-356 (BSSC 2000) proposed four performance levels, namely *Operational (OP)*, *Immediate Occupancy (IO)*, *Life Safety (LS)*, and *Collapse Prevention (CP)*, with descriptive damage states and proposed drift limits. Each performance level is associated with a particular earthquake hazard level, defined in accordance with a specified probability of occurrence in a given return period (50 years), namely *Frequent (F)*, *Occasional (O)*, *Rare (R)*, and *Very Rare (VR)*.

By associating the damage levels specified in FEMA-356 with the available calibration results of the Park-Ang damage model, the range of the damage index for each performance level may be given as shown in Table 3.1. It is noted that an additional performance level (damage control, DC) has been added to complete the picture, as suggested by some researchers (Priestley 2000). Thus, five performance limit states, namely from I to V (see Table 3.1), are considered.

Table 3.1 Damage index ranges for different performance levels

Performance level	Degree of damage	Damage index range (global level)
I: Operational	Negligible	0<0.1
II: Immediate occupancy	Minor	0.1-0.2
III: Damage control	Moderate	0.2-0.5
IV: Life safety	Severe	0.5-0.8
V: Collapse prevention	Near collapse	0.8-1.0
Loss of building	Collapse	>1.0

3.4 Procedure for construction of damage-based R_D spectra and discussion

For any ground motion input, a set of R_D spectra can be constructed for various levels of damage, given a pre-selected ductility class represented by a (monotonic)

ductility capacity μ_u . Figure 3.3 shows the flowchart for the calculation of the damage-based R_D factor by a series of SDOF systems. Firstly, a SDOF system is defined by specifying an initial natural period, T , and mass, m . In the present calculation, the mass is taken as unity and the viscous damping ratio, ξ , is assumed to be 5% in all cases. After evaluating the initial stiffness (K_0), the natural frequency (ω_n), and the damping coefficient (c), the elastic strength F_e under the given ground motion can be calculated by solving the governing equation of the SDOF system. Then by reducing F_e with a strength decrement ΔF , a yield strength F_y ($F_y = F_e - \Delta F$) and the corresponding damage index D can be calculated. Perform iterations by gradually reducing F_y (increasing ΔF) until the target damage level D_i is achieved. This gives rise to one spectral point on the R_D curve for the damage level D_i . Move on to a new SDOF system with another period and repeat the above procedure, and so forth.

Figure 3.3 Procedure for calculating a R_D factor

The three-parameter Park hysteretic model (Park et al. 1985), as introduced in Chapter 2, is adopted in the analysis to model the hysteretic behaviour of the system. A typical pattern similar to that shown in Figure 3.1 is employed. The post-yield stiffness is assumed to be 5% of the initial stiffness of the system. The effect of different hysteretic patterns on the R_D spectra will be addressed in the later part of this chapter.

As an example, the R_D spectra for the El Centro 1940 N-S ground motion record and a record from the 1995 Kobe earthquake are constructed. The two ground motion records and their elastic acceleration response spectra are plotted in Figure 3.4. Typical R_D spectra are shown in Figure 3.5. Generally speaking, the R_D spectra may be divided into three period segments, a) 0~0.5s, with small but increasing R_D values, b) 0.5~2s, moderate to high R_D values, and c) >2s, high R_D values. In-between b) and c) there appears to be a transition period range but it could be sensitive to individual ground motions.

For a comparison between the R_D spectra and the classical ductility based spectra, the ductility based spectra, R_μ , are also generated such that each curve corresponds to a constant ductility level. Because the R_μ spectra are meant for the ultimate limit state when the specified ductility is reached, the comparable R_D counterpart should be that corresponding to $D=1.0$ with the available ductility equal to that considered in the R_μ spectra. The comparison is shown in Figure 3.6. As can be seen, the R_D spectra are always lower than the R_μ spectra. Except for the lower ductility case ($\mu=2.0$), the ratio between the R_D and R_μ spectra is on the order of 0.8~0.9, with noticeable fluctuation at certain period ranges.

Similar procedure can be performed to generate the R_D spectra for other ground motion records, henceforth, the mean spectra and statistical dispersion for different categories of ground motions can be obtained.

a) Accelerograms (left = El Centro 1940; right = 1995 Kobe)

b) 5%-damped acceleration spectra normalized to PGA

Figure 3.4 Accelerograms and response spectra of two example ground motions

a) El Centro-NS (1940 Imperial Valley earthquake)

b) Shin Osaka-00 (1995 Kobe earthquake)

Figure 3.5 R_D spectra for the two example earthquake ground motions

a) El Centro 1940-NS

b) Shin Osaka-00

Figure 3.6 Comparison between R_D (damage-based, $D=1.0$) and R_μ (ductility-based) spectra

3.5 Generation of damage based spectra for different groups of ground motions

3.5.1 Earthquake ground motions considered

A total of 396 horizontal earthquake acceleration time histories recorded from 24 different earthquakes are used in this study, which have been obtained from PEER Strong Motion Database (PEER 2004). All the ground motions selected have common characteristics as follows:

- (1) They are recorded on accelerographic stations where detailed information on the geological and geotechnical conditions at the site is available. This enables the classification of the recording site.
- (2) They are recorded on free field stations or in the first floor or basement of low-rise buildings with negligible soil-structure interaction effects.
- (3) In these records, at least one of the two orthogonal horizontal components has Peak Ground Acceleration (PGA) larger than 0.05g.

The selected records cover a time span from 1952 (Kern County) to 1999 (Chi-Chi, Taiwan). As shown in Figure 3.7a), most of the records are within a surface wave magnitude (M_s) range from 5.2 (Point Mugu, 1973) to 7.8 (Kocaeli, Turkey, 1999), and a closest distance to the horizontal projection of the rupture (distance to rupture, R_f) range within 160 km. The scatter plot for PGAs and distances in Figure 3.7b) illustrates an overall trend that would match known attenuation relationships for firm sites; higher maximum ground accelerations correspond to recording sites with short distances to rupture, and with increasing distance, the maximum ground acceleration decreases. Peak ground accelerations range from 0.05g to 0.6g with the exception of a few records. More details about these database are provided in Appendix A.

Figure 3.7 Magnitude-PGA-Rupture distance range of earthquake ground motions considered

3.5.2 Effect of site conditions on the R_D spectra

It is generally known that the local site conditions could have marked influence on the structural response. The effect of the local soil condition on elastic design spectra has been reflected in current seismic codes such as 2003 NEHRP provisions (BSSC 2003). However, as a relative measure, the R_D factor may not necessarily exhibit the same effect of the site conditions as the elastic response spectra.

For this evaluation, the site classification criterion in 2003 NEHRP provisions is adopted herein. The method is based on the average shear wave velocity and specifies five site classes, namely, A - hard rock site, B - rock site, C - very dense soil and soft rock site, D - stiff soil site, and E - soft soil site. In this study, only the ground motions recorded on rock and firm soil sites are considered and they are divided into three groups, namely, S_{AB} (Site Classes A + B), S_C , and S_D . The soft soil site is not included due to the limitation of available data.

Using the procedure described in Section 3.4, by reducing progressively the yield

strength from the elastic strength demand, a series of R_D factors corresponding to different damage levels are computed for a family of 72 SDOF oscillators with the initial period T from 0.05 to 3.0s. Repeating the same process for each ground motion record, the spectra for different damage levels for a given ductility class can be generated. After the spectra are generated for all individual ground motions in each site group, the results are combined to form the mean spectra and the dispersion characteristics (standard deviation).

Figure 3.8 plots a set of mean R_D spectra for the three site groups of S_{AB} , S_C and S_D . For the convenience of comparison, these spectra are also normalized with respect to the overall mean R_D spectra for all the firm site groups under consideration, and the normalized results are plotted in Figure 3.9.

As can be seen in Figure 3.9, site group S_{AB} exhibits relatively higher R_D values in the shorter period range ($<1.5s$) and lower R_D values in the longer period range ($>1.5s$). This implies that for these sites neglecting the effects of local site conditions on R_D would result in certain overestimation of the inelastic strength demand in the shorter period and underestimation in the longer period range. An opposite trend is observed for group S_D , whereas for group S_C , the mean R_D spectra are almost identical to the overall mean R_D spectra. In general, the normalized R_D spectra increase as the damage level increases; and for the same damage level, the normalized R_D spectra increase with an increase of the ductility class. However, the variation of different sites from the overall mean spectra are typically less than 10% for a ductility class range of 2.0 to 8.0 at $D=0.8$ in the whole period range. Therefore, the effect of site conditions (excluding soft soil site) on the R_D spectra may be neglected. It should be noted again that the basic effect of the site conditions is expected to be reflected in the site-dependent elastic response spectra and R_D is a reduction factor from the elastic spectra.

Figure 3.8 Group mean R_D spectra for different firm site conditions

Figure 3.9 Ratio of group mean to overall (all groups) mean R_D spectra

3.5.3 Effect of earthquake magnitude

For this evaluation, all ground motions in the data set are combined and then re-divided into three groups in accordance with the earthquake magnitude (surface magnitude, M_s), as follows: a) $5.2 < M_s < 6.6$ (100 records), b) $6.7 < M_s < 7.1$ (162 records), and c) $7.4 < M_s < 7.8$ (134 records). The respective mean R_D spectra for $D=0.5$ and 0.8 with $\mu_u=6.0$ are illustrated in Figure 3.10a), while the normalized curves with respect to the overall mean spectra are shown in Figure 3.10b). It can be observed that lower R_D factors are produced consistently in case of larger magnitude earthquakes, while lower magnitude earthquakes generally yield higher R_D factors. The difference of R_D factors among the groups increases as the damage level increases at the same ductility level. Clearly, using the overall mean R_D spectra would result in underestimation of inelastic strength demand for higher magnitude earthquakes and overestimation for lower magnitude earthquakes. The range of difference is generally between 10~20% for $D=0.5$ and 0.8 with $\mu_u=6.0$.

3.5.4 Effect of distance to rupture

For this evaluation, the whole set of ground motions are divided into three distance groups, as follows: a) $R_f < 20$ km (short distance range, 136 records), b) $20.1 < R_f < 55$ km (intermediate distance range, 130 records), and c) $R_f > 55$ km (relatively long distance range, 130 records). Figure 3.11a) shows the mean R_D spectra corresponding to $D=0.5$ and 0.8 and $\mu_u=6.0$ for the three distance groups. The normalized curves with respect to the overall mean spectra are shown in Figure 3.11b). It can be observed that lower R_D factors are produced for the ground motions of longer distance, while the ground motions of shorter distance generally result in higher R_D factors. However, the variation of distance does not appear to cause much difference in the mean R_D spectra, and the difference is generally within 10% of the overall mean spectra.

a) Group mean R_D spectra

b) Ratio to overall mean R_D spectra

Figure 3.10 Influence of earthquake magnitude on R_D spectra

Figure 3.11 Influence of distance to rupture on R_D spectra

To a large extent, the effects of local site condition, earthquake magnitude and distance may be explained by the ground motion duration, as the duration is closely related to the damage induced by the dissipated hysteretic energy (Chai et al. 1998). Longer duration ground motions of comparable intensities generally result in higher energy-induced (cumulative) damage and consequently lead to lower R_D factors, and vice versa. In the literature (Hernandez and Cotton 2000; Novikova and Trifunac 1994), the ground motion duration was characterized as a function of earthquake magnitude, distance, and local site condition. In general, larger magnitude, longer distance and softer soil yield longer duration, and consequently

result in lower R_D factors, which agree with the observations in the above comparisons. Based on the present results, the effect of earthquake magnitude on the R_D factors appears to be more significant. It is also noted that the soft soil condition (S_E) may induce other complications, but this is not included in the present evaluation due to limited records available.

3.5.5 Effect of hysteretic patterns

In the above discussions, the basic (typical) degrading hysteretic pattern, as illustrated in Figure 3.1, is used to generate all the above R_D spectra. For different materials and structural configurations, the global hysteretic behaviour would be different and hence it is reasonable to investigate the effect of different hysteretic patterns on the R_D factors.

The basic degrading pattern with nominal stiffness degradation, strength degradation and pinching is defined using the three parameter Park model with $\alpha_s=2.0$, $\beta_e=0.1$ and $\gamma_p=1.0$. For comparison purpose, the elasto-perfectly-plastic (EPP) model (post-yield stiffness ratio $\alpha_p=0$) and the bilinear model are considered. Besides this two non-degrading patterns, another three degrading patterns are also selected to represent a severe level of stiffness degradation, strength degradation and pinching, respectively. Table 3.2 summarizes the selected representative hysteretic patterns. The ground motions for Site S_C are used to represent the firm site group. In the present evaluation, the constant β , related to the hysteretic energy-induced damage portion in the Park & Ang damage model, is taken to be 0.1 for general cases.

Table 3.2 Selected representative hysteretic patterns

No.	α_s	β_e	γ_p	α_p	μ_u	Model description
1	2	0.1	1.0	0.05	8	Basic (nominal) degrading
2				0	8	EPP
3				0.15	8	Bilinear
4	0	0.1	1.0	0.05	8	Severe stiffness degradation
5	2	0.3	1.0	0.05	8	Severe strength degradation
6	2	0.1	0.3	0.05	8	Severe pinching

Figure 3.12 Hysteretic patterns for JOS000 ground motion

To illustrate the hysteretic behaviours, the computed hysteretic responses for the record (JOS000) at Joshua Tree Fire Station, California during the Landers Earthquake ($M_s=7.5$, June 28, 1992) are shown in Figure 3.12. The basic degrading pattern in Figure 3.12(a) is associated with typical reinforced concrete structures. The EPP and bilinear patterns as illustrated in Figure 3.12 (b)~(c) are treated as a simplified model to represent the basic hysteretic behavior without stiffness and strength degradation. The last three patterns (Figure 3.12(d)~(e)) exhibit severe stiffness degradation, strength degradation and pinching, respectively.

Figure 3.13 illustrates the influence of hysteretic patterns on the R_D factors at the damage level of $D=0.4$. As can be seen in Figure 3.13(a), the influence of the three basic hysteretic patterns is period-dependent. The dividing period value is around 0.5 second. In the short period range, the degrading models can result in lower R_D factors, while the bilinear and EPP models result in higher R_D factors. The trend appears to be reversed in the longer period range, whereby higher R_D factors are associated with the degrading hysteretic model. Compared with the bilinear model, the EPP model always produces lower R_D factors as the post-yield hardening is ignored in the EPP model. Generally speaking, the R_D spectra produced for $D=0.4$ vary only slightly for different hysteretic models.

Figure 3.13 Comparison of R_D spectra from different hysteretic Patterns

Further detailed comparison is made between the basic degrading pattern and those three patterns with severe stiffness degradation, strength degradation and pinching. As shown in Figure 3.13(b), the variation of hysteretic patterns generally has a negligible effect on the R_D spectra for firm sites.

The above observation seems to somewhat contradict to what one would expect from a physical viewpoint. Judging from the basic definition of the damage index defined in Eq. 3.4, which is widely used in the literature, the observation is numerically explicable. This is because for a well detailed ductile design, β takes a lower value (say 0.05), implying smaller rate of damage accumulation, as opposed to a higher value (say 0.2) for a poorly detailed case. However, the total amount of energy dissipation during the response would be higher for the design with a better hysteresis pattern than that for a poorer design. Therefore, the combination of a lower β value with a higher energy dissipation, and vice versa, plus the statistical effect, gives rise to what appears to be an negligible effect of the hysteretic pattern on the damage- R_D spectra. The evaluation of the effect of the β factor will be discussed later in Section 3.6.2. Further probe of this issue requires a re-visit of the rationale of the damage index, especially the denominator with regard to the energy induced damage. This is beyond the present scope of study.

3.6 Proposed R_D spectra and comparison with R_μ spectra

3.6.1 Overall mean R_D spectra, COV and proposed empirical expressions

Based on the observations described above, the R_D spectra for different groupings of the ground motions do not vary significantly, except to some extent for different earthquake magnitude groups. Therefore, it is reasonable to construct the overall

(grand) mean R_D spectra that could be used as a uniform reference. Modification coefficients may be incorporated for special groups of ground motions if necessary.

Figure 3.14 presents the overall mean R_D spectra. Typical COV (coefficient of variation) curves are shown in Figure 3.15. As can be seen, the R_D spectra exhibit a steep slope in the shorter period range and become almost constant in the longer period range. The limiting period is about 1.0s. As expected, for a given ductility class μ_u , higher R_D factors are produced for higher structural damage, D , while for a given damage level D , the R_D factor increases as the ductility class increases.

The similarity in the shape of the overall mean R_D curves permits the use of a unified expression of these curves by a simple empirical formula as a function of T , D and μ_u , i.e.,

$$R_D = f(T, D, \mu_u) \quad (3.5)$$

The boundary limits can be identified in a similar way as that for the ductility-based spectra (e.g., Miranda and Bertero 1994), as follows:

$$R_D(T = 0, D, \mu_u) = 1 \quad (3.6)$$

$$R_D(T, D = 0, \mu_u) = 1 \quad (3.7)$$

$$R_D(T = \infty, D, \mu_u) = \bar{R}_D \quad (3.8)$$

It is noted that in the large period range, R_D will approach a constant value, $\bar{R}_D$, as a function of damage level and ductility class only, and it can be determined by a regression analysis from the actual R_D spectra.

Figure 3.14 Comparison of the computed R_D spectra using Eq. (3.9) with the original spectra

Figure 3.15 Comparison of regressed COV lines and actual mean COV spectral curves

A three-step nonlinear regression analysis is carried out on the R_D factors: firstly, the function of R_D vs. the structural period T is regressed for a series of damage indices and ductility classes; secondly, the effect of damage indices on the first step coefficients is evaluated for several ductility classes; and finally, the second step coefficients are expressed as a function of ductility class. The following equations of $R_D - T - \mu_u - D$ are obtained for the overall mean spectra for the firm sites under consideration:

$$R_{D, Firm} = 1 + A_0 (1 - e^{-B_0 T}) \quad (3.9a)$$

$$A_0 = (a_1 + a_2 \mu_u) D \quad (3.9b)$$

$$B_0 = b_1 + \frac{b_2}{\mu_u D} \quad (3.9c)$$

where $a_1 = -0.68$, and $a_2 = 0.91$, $b_1 = 2.09$ and $b_2 = 2.55$ for the basic ductile design under consideration with $\beta = 0.1$.

The computed R_D spectra using Eq. (3.9) are compared with the actual mean spectra in Figure 3.14. A good match is observed for all damage indices and ductility classes.

The dispersion of the R_D spectra, as represented by the COV shown in Figure 3.15, appears to be constant almost over the entire period range of interest. However, they increase with increasing damage level and the ductility class. Therefore, the COV curves may be approximated as a function of D and μ_u only. Based on the regression analysis, it follows:

$$COV = c_1 [(\mu_u - 1) D]^{1/2} \quad (3.10)$$

where $c_1 = 0.18$ herein. Typical comparison between Eq. (3.10) and the original

COV curves is seen in Figure 3.15.

It has been shown in Section 3.5.3 that the earthquake magnitude has appreciable influence on the R_D spectra, and such influence does not vary markedly with the period, but depends on damage level and the ductility capacity of the system. Based on these considerations and following a regression analysis, the following modification factor is proposed to take into account the earthquake magnitude effect on the R_D spectra:

$$R_D = C_m R_{D, Firm} \quad (3.11a)$$

$$C_m = \begin{cases} c_2 \mu_u D + 1 & \text{low mag.} \\ 1.0 & \text{moderate mag.} \\ -c_2 \mu_u D + 1 & \text{high mag.} \end{cases} \quad (3.11b)$$

where $c_2 = 0.03$ for cases with $\beta = 0.1$.

3.6.2 R_D spectra for low and high ductility designs

The R_D spectra produced in the above sections are for the basic ductile design assuming $\beta = 0.1$. For very well detailed ductile designs, β could take a low value of 0.05, whereas for poorly detailed structures, β could be as large as 0.2. For this reason, the R_D spectra are also constructed for the upper and lower cases with $\beta = 0.05$ and 0.2, respectively. It should be noted that in real situations the selection of β value is generally dependent on the ductility class μ_u ; but for the sake of completeness, herein the R_D spectra are constructed for the same wide range of μ_u for all cases.

Figure 3.16 compares the spectra with $\beta = 0.05$, 0.1, and 0.2 for two ductility classes. It can be seen that the R_D values consistently increase with a decrease of the β value.

Regression analysis indicates that the same general formulas as expressed in Eqs.(3.9) and (3.10) are also suitable for the mean R_D and COV in the cases with $\beta=0.05$ and 0.2 . The coefficients of a_1 , a_2 , b_1 and b_2 are summarized in Table 3.3. Figure 3.17 shows a typical comparison of the regression formula for R_D with the actual mean spectrum curves for $\beta=0.05$ and 0.2 , respectively. A good fit is observed.

Figure 3.16 Comparison of R_D spectra for different β values

Figure 3.17 R_D spectra and their regression curves for $\beta=0.05$ and 0.20

Table 3.3 Summary of coefficients in the proposed mean R_D spectra and COV for different ductility class structures

Regressor	High ductility $\beta=0.05$	Moderate ductility $\beta=0.1$	Low ductility $\beta=0.2$
a_1	-0.87	-0.68	-0.46
a_2	1.04	0.91	0.73
b_1	2.19	2.09	1.96
b_2	2.6	2.55	2.52
c_1	0.16	0.18	0.20
c_2	0.03	0.03	0.03

3.6.3 Comparison of proposed R_D spectra with R_μ spectra at ultimate limit state

As mentioned before, the previous R_μ spectra ($R_\mu - \mu - T$ relationships) (e.g., Miranda and Bertero 1994) are associated with the ultimate limit state, and a comparison with the present R_D spectra is only meaningful for a comparable damage state, which, for simplicity, may be taken as $D=1.0$. The R_D spectra calculated using Eq. (3.10) are compared with the R_μ spectra proposed by various investigators (Newmark and Hall 1982; Krawinkler and Nassar 1992; Miranda and Bertero 1994) in Figure 3.18. It is noted that the $R_\mu - \mu - T$ relationship consistent with the Newmark and Hall inelastic response spectra was given by Chopra (1995).

Figure 3.18 Comparison of the proposed R_D spectra with the previous R_μ spectra at ultimate limit state

In general, the R_D and R_μ spectra exhibit similar trends but the R_D values are consistently lower than the R_μ values for the same class of ductility, as expected. For a lower ductility class such as 2.0, the difference is limited. However, for higher ductility classes such as 6.0 and 8.0, the difference can be quite significant (up to about 25%). Relatively speaking, the R_μ spectra derived from Newmark and Hall inelastic response spectra (Chopra 1995) shows a better comparison with the proposed R_D spectra. The R_μ spectra from Miranda & Bertero (1994) compare well with the proposed R_D spectra in the small period ($<0.5s$) and large period ($>2.0s$) ranges. The constantly higher R_μ spectra from Krawinkler and Nassar (1992) may be attributable to the different considerations in the ductility capacity. The R_μ factors proposed in Krawinkler and Nassar (1992) are based on equivalent cyclic ductility; however, in the present calculation the ductility capacity refers to the monotonic ductility capacity. Given an appropriate conversion from the monotonic to cyclic ductility capacity, the comparison between the Krawinkler & Nassar's R_μ spectra and the current R_D spectra is expected to be quite close.

3.7 Summary and conclusions

The strength reduction factor is inherently related to the inelastic response of a structure at the ultimate limit state as well as at other response levels. The construction of damage-based R_D spectra for various damage levels allows for a more rational determination of the design strength demand, taking into consideration the desired damage limits for multiple performance targets. The damage based R_D spectra can also be used in a reverse manner for a rapid estimation of the potential damage to a system when the actual strength of the system and a particular earthquake intensity are available.

The R_D spectra are generated for a large set of ground motions with different groupings to examine the effects of site condition, earthquake magnitude, distance to rupture and hysteretic pattern on the R_D spectra. The effect of the earthquake magnitude is relatively more significant, such that higher magnitude earthquakes tend to give rise to lower R_D factors for the same damage level. The influences of the distance to rupture and hysteretic patterns are generally negligible. As far as firm sites (S_{AB} , S_C and S_D) are concerned, it is observed that the local site condition does not appear to have significant effect on the mean R_D spectra. However, it should be noted here that the overall demand can still be site-dependent due to the site-dependence of the basic elastic spectra.

A uniform overall mean R_D spectrum is thus proposed for all firm sites, with a modification to take into account the influence of earthquake magnitudes. Regression formulas are constructed for the mean R_D spectra as a function of the natural period of the system, the damage level and the ductility class (capacity) for moderate, low and high ductility designs. The uncertainty is represented by the coefficient of variation, which is given as a function of the damage level and the ductility capacity.

It should be pointed out that soft soil site condition is not included in this study; therefore, the proposed R_D spectra are not expected to apply to soft soil sites. Besides, the connection of the R_D spectra to performance based design is viewed only from the perspective of structural damage. For nonstructural components and building contents, the damage states may depend more on the storey drift ratio (Gu and Lu 2005) and floor acceleration.

In the present study, the mean values of different variables are used to illustrate the proposed methodology and facilitate comparisons. To develop probabilistic PBSB guidelines, the uncertainties as mentioned in Section 2.2 should be taken into account. As suggested by Bertero and Bertero (2002), a probabilistic design approach could be adopted for design, where the design equation could be reduced to the load factor design assuming that all the random variables as deterministic (and equal to the mean value) with the exception of the earthquake demands because the large uncertainties in the seismic demand. A more sophisticated reliability-based design and analysis may be performed through statistical simulations and involving fragility curves (e.g. Lu and Gu 2004).

CHAPTER 4

EVALUATION OF SEISMIC DAMAGE OF MULTI-STOREY RC FRAMES WITH DAMAGE-BASED INELASTIC SPECTRA

4.1 Introduction

The analysis of seismic behaviour of structures in terms of damage has received much attention in recent decades (e.g., Park et al. 1987; Ghobarah et al. 1999). In a performance-based design era, a damage measure may be used as a performance criterion as well as a basis for decision-making with regard to repairing and retrofitting. An evaluation of seismic damage can be carried out by performing detailed nonlinear time history analyses in conjunction with an appropriate damage model, so that both the overall damage status and the distribution of damage can be obtained. However, to perform nonlinear time history analyses is tedious and costly, especially when uncertainties in the ground motion input needs to be considered. In this respect, a simple and yet rational method based on well-constructed probabilistic response spectra is desired.

Similar to the widely known approach of deducing the displacement response of a multi-degree-of-freedom (MDOF) system from a single-degree-of-freedom (SDOF) displacement (e.g., Whittaker et al. 1998; Miranda 1999; Fajfar 2000; Gupta and Krawinkler 2000; Chopra and Goel 2000), a method to deduce the MDOF damage from SDOF damage can be envisaged. As far as structural damage is concerned, an estimation of the overall damage of a MDOF system may be made first from the equivalent SDOF damage, taking into account the dynamic properties of the MDOF system. Subsequently, the distribution of damage throughout the MDOF system can be estimated based on the structural characteristics of the MDOF system. With such

a scheme, an evaluation of the seismic damage to a MDOF system can be made conveniently when the damage spectra for SDOF systems become available.

Several issues need to be investigated before the above outlined SDOF-to-MDOF damage estimation scheme can be put forward. First, it is necessary to establish whether a reasonable correlation exists between SDOF and MDOF systems in terms of the overall damage, and how such correlation may be affected by MDOF system parameters. Second, there must be an appropriate way to define the system properties (regularity) concerning the distribution of inelastic response or damage. Finally, it is necessary to investigate how to distribute the structural damage by a rapid approach such as a nonlinear static analysis and how to correct the ‘static’ damage distribution to well represent that under actual ground motions from a perspective of mean values.

This chapter proposes a methodology for estimating the damage in multi-storey frames from the damage spectra of SDOF systems, and presents the associated studies with nonlinear static and dynamic time history analysis. A series of representative frame structures and a large set of earthquake ground motions are selected for the parametric studies in establishing the correlations between the actual structural damage in multi-storey frames and the damage in their equivalent SDOF systems. Two alternative approaches for predicting the distribution of damage in the frames, one based on the modal pushover analysis, and the other based on the evaluation of the so-called storey capacity factor (Lu et al 2006), are investigated and the appropriate procedure of using these approaches are proposed.

4.2 Theory and basic considerations

4.2.1 Definition of damage in SDOF and MDOF systems

A review of typical damage models has been presented in Chapter 2. In the present study, the modified version of the Park and Ang damage model (Park and Ang 1985; Kunnath et al. 1992) is selected and discussed herein in detail for quantifying the

damage under seismic loading. This damage model has been widely used and calibrated according to previous experimental data (Park and Ang 1985; Park et al. 1987). It is worth noting, however, that the proposed methodology is not restricted to the currently selected damage model; other models that are capable of providing reasonable damage estimation at different levels can also be applied.

4.2.1.1 SDOF damage under seismic loading

For an equivalent SDOF system, the damage index is defined in terms of the displacement and the hysteretic energy as,

$$D_{SDOF} = \frac{u_m^* - u_y^*}{u_u^* - u_y^*} + \beta \frac{E_h^*}{q_y^* u_u^*} \quad (4.1)$$

where the asterisk is used to indicate an equivalent SDOF system. u_m^* is the maximum displacement attained; u_y^* is the yield displacement; u_u^* is the displacement capacity under the monotonic loading; E_h^* is the dissipated hysteretic energy; q_y^* is the yield strength; and β is the model parameter.

The typical elastic and inelastic response terms of a SDOF system have been schematically illustrated in Chapter 3, and they are repeated in Figure 4.1 for association with the present derivations. In this demand/capacity damage model, u_m^* and E_h^* represent the demands, and q_y^* , u_u^* and β represent the capacities. The β value reflects the effect of the cumulative energy dissipation in the total damage. Low β values (such as $\beta=0.05$) are usually associated with well-detailed ductile RC structures (Park et al., 1987), as in such cases the influence of low-cycle fatigue is small and hence the seismic damage is mainly governed by the maximum displacement. On the other hand, for poorly detailed RC structures (such as nonseismically designed structures), high β values are appropriate. In the present study, a moderate β value of 0.1 is considered as a general case.

Figure 4.1 Schematic of elastic and inelastic responses of a SDOF system

4.2.1.2 MDOF damage under seismic loading

For a structural (MDOF) system, the member damage index is firstly calculated, and the storey damage and the global damage are then determined according to an energy-based weighting scheme. The local damage index of a structural member is defined as (Kunnath et al. 1992):

$$D = \frac{\theta_m - \theta_y}{\theta_u - \theta_y} + \beta_e \frac{E_h}{M_y \theta_u} \quad (4.2)$$

where θ_m is the maximum rotation attained; θ_y is the yield rotation; θ_u is the rotation capacity; β_e is the model parameter representing the rate of energy-induced damage of the member due to cyclic loading; E_h is the dissipated hysteretic energy in the section and M_y is the yield moment. Here, θ_m and E_h represent the demands, and M_y , θ_u and β_e represent the capacities. The physical implication of β_e is similar to the β described in the previous section. At the member level, the definition can also be expressed in terms of moment-curvature quantities, assuming that shear deformation remains elastic.

The damage at a storey and the entire system (global) levels depends on both the

magnitude of local damage and the damage distribution within the storey and entire structure. The storey and global damage indices can be calculated by combining the component damage indices through a weighting procedure. In general, the weighted damage can be evaluated by

$$D = \frac{\sum_i w_i D_i}{\sum_i w_i} \quad (4.3)$$

where D is the weighted damage index, D_i is the local damage index; and w_i is the weighting factor. Due to the significance of the severest local damage in the status of overall damage (for example when a local region reaches a critical state the entire system may be declared as failed), the weighting factor is usually inclined towards larger local damage such that larger local damage weighs heavier in the overall damage. In the Park and Ang damage model (Park et. al 1985), the weighting factor is tied to the amount of plastic strain energy dissipation, E_h . This approach is adopted herein. Thus, when calculating the storey-level damage index, D_i and w_i are taken as the component damage index and the dissipated hysteretic energy of the i -th component in the storey under consideration, respectively. When calculating the global damage index, D_i and w_i are taken as the storey damage index and storey hysteretic energy dissipation, respectively.

4.2.2 Overview of damage-based inelastic spectra and their use for damage evaluation of SDOF systems

The damage of an equivalent SDOF system under seismic loading can be determined using the damage-based inelastic spectra with consideration of cumulative damage. An extensive study has been conducted in Chapter 3 for constructing the damage-based inelastic spectra in terms of the strength reduction factor (SRF, designated as R_D) for SDOF systems with a given ductility capacity. Here, a briefing is provided towards their use for damage estimation of SDOF systems.

The damage based R_D spectra are presented in the form of R_D - T (period) curves for SDOF systems of different ductility (capacity) classes and damage levels. Figure 4.2 illustrates a set of damage-based mean R_D curves produced according to the empirical formulas for the firm sites (Eqs.(4.4a-c)) which is a general case extracted from Eq.(3.9a-c) in Chapter 3. The R_D factor is expressed as a function of period (T), ductility capacity (μ_u) and damage level (D). Comparisons have shown that the R_D spectra are consistent with some well-known ductility-based SRF spectra, with reduced R_D values as a result of considering the cumulative damage. In Chapter 3, the damage-based R_D spectra have been used in determining the design base shear from an elastic response spectrum to achieve a targeted damage index for a given ductility class of the structure.

Figure 4.2 Typical damage-based inelastic spectra ($\mu_u=6$)

$$R_{D, Firm} = 1 + A_0 (1 - e^{-B_0 T}) \quad (4.4a)$$

$$A_0 = (-0.68 + 0.91\mu_u)D \quad (4.4b)$$

$$B_0 = 2.09 + \frac{2.55}{\mu_u D} \quad (4.4c)$$

In the present study, the same damage-based R_D spectra will be employed in a reversed manner to determine the damage for a given SDOF system. The SDOF system can be defined by the initial period T_{eq} , mass m^* , yield strength q_y^* , and the ductility capacity class μ_u . For a certain level of seismic hazard, the seismic demand in terms of elastic spectral response acceleration, $S_{ae}(T_{eq})$, can be obtained from the site-specific design spectrum. The damage-based R_D factor of the SDOF system can then be calculated as:

$$R_D = \frac{q_e^*}{q_y^*} = \frac{S_{ae}(T_{eq})m^*}{q_y^*} \quad (4.5)$$

Subsequently, with the known μ_u , T_{eq} and R_D , the damage of the equivalent SDOF in the seismic conditions concerned can be easily read from the R_D curves, such as those shown in Figure 4.2, or calculated using trial-and-error from the empirical equations (4.4a-c), or Eq. (3.9a-c) for a broad range of R_D factors.

4.2.3 Characterization of vertical profile of frames with storey capacity factor

Generally speaking, the distribution of seismic demand along the height of a structural frame depends on the strength and stiffness distributions. Several investigations into the seismic response of vertically irregular frames (Valmundsson and Nau 1997; Al-Ali and Krawinkler 1998; Chintanapakdee and Chopra 2004; Lu 2002) indicated that the distribution of deformation is primarily affected by the storey strength, and to a lesser extent, the storey stiffness, especially at advanced inelastic response stages. A scheme incorporating both stiffness and strength considerations, called “storey capacity” profile factor, has recently been proposed to characterize the concentration of inter-storey drift in multi-storey RC frames (Lu/Gu/Wei 2006). In the study presented in this chapter, the above scheme is extended for damage evaluation as a possible design analysis approach and its correlation with the damage distribution in multi-storey frames is investigated. A modified version of the storey-capacity scheme for prediction of drift concentration

to take into account the inelastic response levels will be discussed in Chapter 5. Preliminary analysis indicates that the modification on the storey capacity evaluation does not affect significantly the evaluation of damage. For simplicity, in this chapter we adopt the original definition of the storey capacity for the damage evaluation. An overview of the storey capacity scheme is given in what follows.

As mentioned, the frame regularity is often defined based on strength, such as the strength regularity index, α_r , adopted in Eurocode 8 (1996):

$$\alpha_r = \frac{(i_{OS})_{\min}}{(i_{OS})_{ave}} \quad (4.6)$$

where $(i_{OS})_{\min}$ and $(i_{OS})_{ave}$ are the minimum and mean values of storey overstrength factor, respectively. i_{OS} is given by:

$$i_{OS} = \frac{V_R}{V_S} \quad (4.7)$$

where V_R and V_S are the available storey shear strength and design storey shear force, respectively.

In EC8, V_R is generally evaluated as the sum of shear strengths of all columns in a storey, assuming that all columns yield at both ends. However, in typical seismic design, frames are required to have a strong-column weak-beam system. Therefore, a more precise calculation of the storey strength capacity should take into account hinging beams. In the present scheme, it is proposed to evaluate the bending moment that can be developed in a column according to the equilibrium around a joint, as illustrated in Figure 4.3(a).

Figure 4.3 Equilibrium and stiffness terms around beam-column joint

The maximum moment that may be developed in the column end k corresponding to yielding of the connecting beams can be approximately calculated as:

$$M_c^k = \begin{cases} M_{c,y}^k \times \frac{\sum M_{b,y}}{\sum M_{c,y}}, & \frac{\sum M_{b,y}}{\sum M_{c,y}} \leq 1 \\ M_{c,y}^k, & \frac{\sum M_{b,y}}{\sum M_{c,y}} > 1 \end{cases} \quad (4.8)$$

where, $M_{c,y}^k$ denotes the yield moment at column end k ; $\sum M_{b,y}$ and $\sum M_{c,y}$ are the summation of the yield moments of beams and columns connecting to the same joint, respectively. Moments at column top and bottom ends can be calculated using Eq.(4.8); thus, the maximum shear force that can be developed in the column can be obtained as:

$$V_c = \frac{M_c^{top} + M_c^{bot}}{h} \quad (4.9)$$

where h is the storey height. The maximum storey shear resistance (strength) can then be evaluated by summing up the representative shear resistances of all columns belonging to the storey.

On the other hand, the distribution of the lateral stiffness in a frame affects directly the drift distribution during the elastic stage and it can be expected that such effect will extend into certain inelastic response stage. Therefore, it is rational to include the effect of the storey lateral stiffness in the evaluation of the structural deformation behaviour. In a simple case with relatively rigid floors, the elastic lateral stiffness of a column can be expressed as:

$$K_c = \frac{12i_c}{h^2} = \frac{12EI}{h^3} \quad (4.10)$$

where, i_c is the linear stiffness defined as EI/h . EI is the bending stiffness.

Considering a general case with flexible floor beams, the D-value method (Muto 1974) is adopted to modify the storey lateral stiffness incorporating the effect of linear stiffness of beams. Referring to Figure 4.3(b), the modified lateral stiffness of an internal column at j^{th} storey ($j > 1$) can be expressed as:

$$D_k = \eta \frac{12i_c}{h^2} = \eta \frac{12EI}{h^3} \quad (4.11a)$$

$$\eta = \frac{K_s}{2 + K_s} \quad (4.11b)$$

$$K_s = \frac{i_1 + i_2 + i_3 + i_4}{2i_c} \quad (4.11c)$$

where, D_k is the modified column lateral stiffness and η is the modification factor; i_1 , i_2 , i_3 and i_4 are the linear stiffness of beams (see Figure 4.3(b)) and i_c is the linear stiffness of the column under consideration.

The uniformity in the distribution of the storey stiffness needs to be measured with respect to the storey shear demand. In the current scheme, a normalized dimensionless storey stiffness factor, i_{ns} , is defined as:

$$i_{ns} = \frac{h D_k}{V_n} \quad (4.12)$$

where, V_n is the normalized storey shear, respectively. The i_{ns} values, as inversed elastic storey drift ratios, reflect the relative stiffness distribution associated with a certain lateral force distribution.

Combining the contribution of both the storey overstrength and storey stiffness factors, the so-called storey capacity factor, i_{sc} , emerges, and for a general inelastic stage it is defined simply as a product of the two factors as:

$$i_{sc} = i_{os} \times i_{ns} \quad (4.13)$$

Accordingly, a frame regularity index with respect to the storey capacity factor, α_{sc} , can be evaluated as:

$$\alpha_{sc} = \frac{(i_{sc})_{\min}}{(i_{sc})_{ave}} \quad (4.14)$$

where, $(i_{sc})_{\min}$ and $(i_{sc})_{ave}$ are the minimum and average values of i_{sc} , respectively. It is expected that larger value of i_{sc} would result in relatively smaller inter-storey drift as well as storey damage. The correlation of the storey capacity factor and the distribution of damage will be investigated in the later part of this chapter

4.2.4 Transformation from MDOF system to SDOF system

In the literature, different procedures are available for transforming a MDOF system to a SDOF system (e.g., Chopra 1995; Krawinkler and Seneviratna 1998). Generally speaking, the response of a MDOF system can be described in the following differential equation,

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{Q} = -\mathbf{M}\mathbf{1}\ddot{u}_g \quad (4.15)$$

where $\mathbf{M}$ and $\mathbf{C}$ are the mass and damping matrices; $\mathbf{Q}$ denotes the storey force vector; $\mathbf{u}$ is the (relative) displacement vector; and $\ddot{u}_g$ is the ground acceleration. The formulation of the equivalent SDOF system is based on the assumption that the deflected shape of the MDOF system can be represented by a shape vector $\{\phi\}$ which remains the same throughout the time history, regardless of the level of deformation. Thus, the displacement vector can be defined as

$$\mathbf{u} = \{\phi\} u_t \quad (4.16)$$

where u_t represents the time variation of the displacements. Note that the assumed displacement shape $\{\phi\}$ is normalized so that the value at the top is equal to unity; and hence, u_t becomes the top displacement history. The governing differential equation of an MDOF system can be rewritten as

$$\mathbf{M}\{\phi\}\ddot{u}_t + \mathbf{C}\{\phi\}\dot{u}_t + \mathbf{Q} = -\mathbf{M}\mathbf{1}\ddot{u}_g \quad (4.17)$$

Premultiplying both sides of the equation by $\{\phi\}^T$, we obtain

$$\{\phi\}^T \mathbf{M}\{\phi\}\ddot{u}_t + \{\phi\}^T \mathbf{C}\{\phi\}\dot{u}_t + \{\phi\}^T \mathbf{Q} = -\{\phi\}^T \mathbf{M}\mathbf{1}\ddot{u}_g \quad (4.18)$$

Define a reference SDOF displacement u^* so that

$$u^* = \frac{u_t}{\Gamma} \quad (4.19)$$

where Γ is usually called the modal participation factor, which is defined as

$$\Gamma = \frac{\{\phi\}^T \mathbf{M}\{1\}}{\{\phi\}^T \mathbf{M}\{\phi\}} \quad (4.20)$$

The following can be obtained for the response of the equivalent SDOF system

$$m^* \ddot{u}^* + c^* \dot{u}^* + q^* = -m^* \ddot{u}_g \quad (4.21)$$

where m^* , c^* and q^* denote the properties of the equivalent SDOF system and are given by

$$m^* = \{\phi\}^T \mathbf{M}\{1\} \quad (4.22)$$

$$c^* = \{\phi\}^T \mathbf{C} \{\phi\} \frac{\{\phi\}^T \mathbf{M} \{1\}}{\{\phi\}^T \mathbf{M} \{\phi\}} \quad (4.23)$$

$$q^* = \{\phi\}^T \mathbf{Q} \quad (4.24)$$

Figure 4.4 Force-displacement relationship of MDOF and equivalent SDOF systems

As shown in Figure 4.4, the relationship of the yield quantities of MDOF and SDOF systems can be obtained accordingly, as

$$u_y^* = \frac{u_{t,y}}{\Gamma} \quad (4.25)$$

$$q_y^* = \{\phi\}^T \mathbf{Q}_y \quad (4.26)$$

$$V_y = \{1\}^T \mathbf{Q}_y \quad (4.27)$$

where $\mathbf{Q}_y$ is the yield storey force vector and V_y is the yield base shear. The initial period T_{eq} of the equivalent SDOF system can be computed as

$$T_{eq} = 2\pi \sqrt{\frac{u_y^* m^*}{q_y^*}} \quad (4.28)$$

It is noted that when the first mode shape is used as the shape function, T_{eq} resembles the first mode frequency of the MDOF system. The deformation at the ultimate stage can also be obtained accordingly, as

$$u_u^* = \frac{u_{t,u}}{\Gamma} \quad (4.29)$$

Thus, the ductility capacity corresponding to the ultimate state is the same for both systems.

4.3 Numerical study on overall damage distribution

4.3.1 General

In this section, the correlation of the global damage in the MDOF structural system (MDOF damage) and its equivalent SDOF system (SDOF damage) will be examined in both static and seismic conditions through the nonlinear static (pushover) and nonlinear time history analyses. The obtained MDOF-SDOF damage relationships in both conditions are compared and discussed in conjunction with the structural features characterized by the storey capacity factor. Both pushover analysis and nonlinear time history analysis are carried out using the program package IDARC 40 (Valles et al. 1996).

The computer program IDARC is essentially a two-dimensional analysis program for the analysis of the non-linear response of reinforced concrete buildings. The program is capable of modeling most structural elements, i.e. columns, beams and shear walls using the basic macro formulation. In general, column elements are modeled considering macromodels at the cross-section level with inelastic flexural deformations, and elastic shear and axial deformations. Beam elements are modeled using a nonlinear flexural stiffness model with linear elastic shear deformation. Shear wall elements are modeled taking into account the inelastic shear and bending deformations, with an uncoupled elastic axial component.

The inelastic behaviour at the member ends is represented by a trilinear moment-curvature envelop in conjunction with a hysteretic model considering stiffness and

strength degradation as well as pinching effect. The moment-curvature envelop is calculated using the Fiber Model, in which the concrete within the confined core area is modeled using the confined concrete stress-strain relationship proposed by Park and Paulay (1975). In this model, the spacing and confinement effect of transverse reinforcement is taken into account in terms of the volumetric ratio of confinement steel to core concrete.

The IDARC program provides four analysis options including a) Nonlinear Static Analysis; b) Nonlinear Pushover Analysis; c) Nonlinear Dynamic Analysis; d) Nonlinear Quasi-static Analysis. This program has been widely utilized and verified in some previous studies (Bracci 1995; Valles 1996).

4.3.2 Representative frames

Four pairs of regular and irregular reinforced concrete (RC) frames of 3, 7, and 12 storeys are considered. They are designated respectively as Frame 3-R, 3-I, 7-R, 7-I, 12-R-a, 12-I-a, 12-R-b, and 12-I-b (“3”, “7” and “12” indicate the number of storeys; “R” and “I” denotes Regular and Irregular frame, respectively). The regular (R) frames generally possess a regular overstrength distribution with respect to the design lateral forces, while the irregular (I) frames exhibit relatively lower overstrength at lower storeys. The frames are designed to comply with the NEHRP provisions (BSSC 2003). The general design information is summarized in Table 4.1, and the section dimensions and reinforcement details are given in Table 4.2. It is noted that frames 12-R-b, and 12-I-b are obtained by systematically weakening the beams in frames 12-R-a, and 12-I-a to ensure that plastic hinges only occur at the member ends of beams and the bottom ends of first storey columns under seismic loading.

Table 4.1 Summary of basic design information

<p>Concrete: $f_c' = 25\text{MPa}$</p> <p>Longitudinal reinforcement: Grade 60, $f_y = 414\text{MPa}$</p> <p>Transverse reinforcement: Grade 40, $f_y = 276\text{MPa}$</p> <p>Design code: NEHRP 2003</p> <p>Site soil: NEHRP S_D stiff soil</p> <p>Design spectrum: NEHRP spectrum</p> <p>Design peak acceleration: 0.3g</p> <p>Response modification factor: 4.0</p> <p>Frame system: Special Moment-Resisting Frame (SMRF)</p> <p>Rectangular hoops with D9.5 steel at spacing of 100mm</p> <p>Heights are 11.2, 25.2 and 42.7 meters for 3-, 7- and 12-storey frames, respectively.</p>
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Table 4.2 Section and reinforcement details of frames

Storey	Frame 3-R				Frame 3-I			
	Column		Beam		Column		Beam	
1	450×450	12D19.1/ 12D19.1	500×300	4D22.3/ 4D19.1	450×450	12D19.1/ 12D19.1	500×300	4D22.3/ 4D19.1
2	400×400	12D19.1/ 8D19.1	400×300	4D19.1/ 4D15.9	400×400	12D19.1/ 8D19.1	500×300	4D19.1/ 4D15.9
3	350×350	8D19.1/ 8D19.1	400×300	4D19.1/ 3D15.9	400×400	8D19.1/ 8D19.1	400×300	4D19.1/ 4D15.9

Storey	Frame 7-R				Frame 7-I			
	Column		Beam		Column		Beam	
1	600×600	12D22.3/ 12D19.1	500×300	4D22.3/ 4D19.1	600×600	12D22.3/ 12D19.1	500×300	4D19.1/ 4D19.1
2	600×600	12D22.3/ 12D19.1	500×300	4D22.3/ 4D19.1	600×600	12D22.3/ 12D19.1	500×300	4D19.1/ 4D19.1
3	500×500	12D22.3/ 12D19.1	450×300	4D22.3/ 4D19.1	550×550	12D22.3/ 12D19.1	500×300	4D19.1/ 4D19.1
4	500×500	12D22.3/ 12D19.1	450×300	4D22.3/ 4D19.1	550×550	12D22.3/ 12D19.1	500×300	4D19.1/ 4D19.1
5	400×400	12D22.3/ 12D19.1	400×300	4D19.1/ 4D19.1	500×500	12D22.3/ 12D19.1	500×300	4D19.1/ 4D19.1
6	400×400	12D19.1/ 12D19.1	400×300	4D19.1/ 4D19.1	500×500	12D19.1/ 12D19.1	500×300	4D19.1/ 4D19.1
7	400×400	12D19.1/ 12D19.1	400×300	3D19.1/ 3D19.1	500×500	12D19.1/ 12D19.1	500×300	4D19.1/ 4D19.1

Chapter 4 Evaluation of Seismic Damage of Multi-Storey RC Frames

Storey	Frame 12-R-a				Frame 12-I-a			
	Column		Beam		Column		Beam	
1	700×700	16D22.3/ 12D22.3	700×300	6D25.5/ 5D25.5	700×700	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5
2	700×700	16D22.3/ 12D22.3	700×300	6D25.5/ 5D25.5	700×700	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5
3	700×700	16D22.3/ 12D22.3	700×300	5D25.5/ 5D25.5	700×700	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5
4	700×700	16D22.3/ 12D22.3	700×300	5D25.5/ 5D25.5	700×700	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5
5	600×600	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5	700×700	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5
6	600×600	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5	700×700	16D22.3/ 12D22.3	650×300	5D25.5/ 5D25.5
7	600×600	12D22.3/ 12D22.3	600×300	5D25.5/ 4D25.5	600×600	12D22.3/ 12D22.3	600×300	5D25.5/ 4D25.5
8	600×600	12D22.3/ 12D22.3	600×300	5D25.5/ 4D25.5	600×600	12D22.3/ 12D22.3	600×300	5D25.5/ 4D25.5
9	500×500	12D22.3/ 12D22.3	550×300	4D25.5/ 4D25.5	600×600	12D22.3/ 12D22.3	600×300	4D25.5/ 4D25.5
10	500×500	12D22.3/ 12D22.3	550×300	4D25.5/ 4D25.5	600×600	12D22.3/ 12D22.3	600×300	4D25.5/ 4D25.5
11	500×500	12D19.1/ 12D19.1	500×300	4D22.3/ 4D19.1	600×600	12D19.1/ 12D19.1	600×300	4D22.3/ 4D19.1
12	500×500	12D19.1/ 12D19.1	500×300	4D22.3/ 4D19.1	600×600	12D19.1/ 12D19.1	600×300	4D22.3/ 4D19.1

Storey	Frame 12-R-b				Frame 12-I-b			
	Column		Beam		Column		Beam	
1	700×700	16D22.3/ 12D22.3	600×300	6D22.3/ 5D22.3	700×700	16D22.3/ 12D22.3	600×300	4D22.3/ 4D19.1
2	700×700	16D22.3/ 12D22.3	600×300	6D22.3/ 5D22.3	700×700	16D22.3/ 12D22.3	600×300	4D22.3/ 4D19.1
3	700×700	16D22.3/ 12D22.3	600×300	5D22.3/ 4D22.3	700×700	16D22.3/ 12D22.3	600×300	4D22.3/ 4D19.1
4	700×700	16D22.3/ 12D22.3	600×300	5D22.3/ 4D22.3	700×700	16D22.3/ 12D22.3	600×300	4D22.3/ 4D19.1
5	600×600	16D22.3/ 12D22.3	550×300	5D22.3/ 4D22.3	600×600	16D22.3/ 12D22.3	550×300	4D22.3/ 4D19.1
6	600×600	16D22.3/ 12D22.3	550×300	5D22.3/ 4D22.3	600×600	16D22.3/ 12D22.3	550×300	4D22.3/ 4D19.1
7	600×600	12D22.3/ 12D22.3	550×300	5D22.3/ 4D22.3	600×600	12D22.3/ 12D22.3	550×300	4D22.3/ 4D19.1
8	600×600	12D22.3/ 12D22.3	550×300	5D22.3/ 4D22.3	600×600	12D22.3/ 12D22.3	550×300	4D22.3/ 4D19.1
9	500×500	12D22.3/ 12D22.3	500×300	5D19.1/ 4D19.1	500×500	12D22.3/ 12D22.3	500×300	4D22.3/ 4D19.1
10	500×500	12D22.3/ 12D22.3	500×300	5D19.1/ 4D19.1	500×500	12D22.3/ 12D22.3	500×300	4D22.3/ 4D19.1
11	500×500	12D19.1/ 12D19.1	500×300	5D19.1/ 4D19.1	500×500	12D19.1/ 12D19.1	500×300	4D22.3/ 4D19.1
12	500×500	12D19.1/ 12D19.1	500×300	5D19.1/ 4D19.1	500×500	12D19.1/ 12D19.1	500×300	4D22.3/ 4D19.1

4.3.3 Evaluation of regularity configurations of frames

The structural regularity can be evaluated through the capacity factor as introduced in Section 4.2. For this purpose, two lateral force distributions are considered here:

Pattern-A

The first mode lateral force distribution, defined as the product of the diagonal mass matrix and the first modal shape, $\mathbf{p} = \mathbf{M}\{\phi\}_1$, is applied as recommended in the NEHRP provisions (BSSC 2000) for structural evaluation.

Pattern-B

For significantly irregular or high-rise structures, the higher mode effects tend to become more pronounced. To reflect such effects, a concentrated lateral force equal to 20% of the total base shear is applied at the top level while the remaining lateral force is distributed in a similar way as in Pattern-A. This lateral force pattern is considered here to induce larger storey shear force demand on the upper storeys, and hence reflect higher mode effects (Lu 2002).

Figure 4.5 illustrates the profiles of the storey shear overstrength factor, storey stiffness factor and the resulting capacity factor of each frame under the lateral force pattern Style-A. Since the inter-storey drift and storey damage is somewhat inversely preoperational to the storey strength and stiffness factors, the inverse of the storey capacity factors are also shown in the same figure. It is noted that inverse of the storey capacity factor is normalized so that the maximum value is equal to unity. It can be seen that the capacity factors tend to be uniformly distributed for the regular frames. The regularity indices (α_{sc}) are evaluated to be 0.85, 0.66, 0.72 and 0.61 for Frame 3-R, 7-R, 12-R-a and 12-R-b, respectively. For irregular frames, lower storey capacity factors exhibit in the lower storeys. The regularity indices for irregular frames 3-I, 7-I, 12-I-a and 12-I-b are found to be 0.70, 0.51, 0.48 and 0.44, respectively.

Besides, the capacity factors corresponding to the lateral force pattern Style-B are also evaluated for the 7-storey and 12-storey frames and they are illustrated in

Figure 4.6. It is observed that relatively lower capacity factors occur in the upper storeys as the Style-B pattern imposes larger storey shear force in the upper storeys.

(a) Frame 3-R

(b) Frame 3-I

(c) Frame 7-R

(d) Frame 7-I

Figure 4.5 Profiles of storey overstrength, stiffness and capacity factors under lateral force pattern-A

(e) Frame12-R-a

(f) Frame12-I-a

(g) Frame12-R-b

(h) Frame12-I-b

Figure 4.5 (continued)

Figure 4.6 Profiles of storey capacity factors under lateral force pattern-B

4.3.4 Pushover analysis and static damage

The pushover analysis is employed for the following purposes: 1) to determine the ultimate structural deformation capacity from the capacity curve (base shear vs. top displacement curve until a predefined ultimate state is reached); 2) to obtain the equivalent SDOF system; and 3) to provide preliminary information on the damage in the actual frame and in the equivalent SDOF system.

In the pushover analysis, the global response of a frame is described by the base shear vs. roof displacement curve. The ultimate state is controlled by the member failure when the ultimate curvature is reached. For example, Figure 4.7(a) shows the typical static response of Frame 7-R. In this graph, the actual capacity curve is idealized into a bilinear curve (in thicker line) according to the FEMA-356 procedure (BSSC 2000). Thus, the overall ductility capacity can be obtained as the ratio of the ultimate roof displacement to the yield displacement.

As described in Section 4.2, the equivalent SDOF can be determined via the transformation assuming an appropriate displacement shape. For simplicity and without losing generality, herein the first mode shape is adopted. Figure 4.7(b) shows the converted force-deformation relationship of the equivalent SDOF system.

The physical properties of the SDOF system include the normalized strength q^*/m^* , initial period T_{eq} and ductility capacity μ_u .

Figure 4.7 Typical static response of MDOF and SDOF systems in pushover analysis (Frame 7-R)

At each intermediate stage of the pushover analysis, the ratio of the maximum response to the ultimate capacity can be obtained, and this ratio effectively represents the “static” damage as the cumulative damage term in Eq.(4.1) and (4.2) is zero in a pushover analysis. The so-called static hysteretic energy (Fajfar and Gašperšič 1996), which is defined as the area under the monotonic moment-rotation curve of a member, is used in the energy-weighting scheme expressed in Eq.(4.3). At the same control roof displacement level, the “static” SDOF damage can also be computed without considering cumulative damage terms.

Figure 4.8 depicts the static damage results in the actual frame and the equivalent SDOF systems obtained from the pushover analysis. As shown in Figure 4.8(a), the damage in both systems exhibits almost a linear relationship with a slope equal to about 0.8. This means the overall damage in the actual frame is about 80% of that in the equivalent SDOF system. The distribution of damage in the actual frame as

obtained from the pushover analysis for different SDOF damage levels are plotted in Figure 4.8(b).

Using the same procedure, the damage information for other frame cases can also be obtained. This will be discussed later in comparison with the results from the time history analysis.

Figure 4.8 Static damage of actual frame from pushover analysis vs. equivalent SDOF damage (Frame 7-R)

4.3.5 Time history analysis and dynamic damage

In the previous section, the correlation between damage in an actual frame and its equivalent SDOF system has been discussed briefly in the context of static pushover analysis. In this section, such correlation as well as the distribution of the damage in the actual frame is investigated by means of nonlinear time history analysis using real earthquake ground motions.

4.3.5.1 Selection of earthquake ground motions

A group of 30 earthquake ground motions are selected and summarized in Table 4.3. These ground motions are chosen from relatively large magnitude earthquakes at medium distances, namely $6.5 < M_s < 7.1$, $20 < R_f < 60\text{km}$, where M_s is the surface wave magnitude and R_f is the closest distance to the horizontal projection of the rupture. The soil condition is S_D defined for firm soil according to the NEHRP classification. The elastic response spectra with a 5% damping ratio of original records are shown in Figure 4.9.

4.3.5.2 Dynamic analysis on actual frames and their equivalent SDOF systems

The dynamic damage of each frame and its equivalent SDOF system can be calculated using nonlinear time history analyses. Both systems are subjected to the same ground motion. To facilitate the comparison, the 30 ground motions are scaled so that they would induce the same level of damage in the equivalent SDOF system for a particular frame. The SDOF damage, D_{SDOF} , is herein referred to as damage level. As the mechanical properties of the SDOF system vary for individual frames, the scaling factors should be different for different ground motions and different frames even for the same damage level (D_{SDOF}). The three-parameter hysteretic model (Park and Ang 1985), as shown in Figure 4.1, is employed to represent the typical hysteretic behaviour of beams and columns as well as the SDOF system in the analysis.

Figure 4.10 depicts typical inelastic responses of Frame 3-I and its equivalent SDOF system under a scaled I-ELC180 ground motion with $\text{PGA}=0.5\text{g}$. It can be seen that the response of the SDOF system represents reasonably the overall response of the actual frame. According to the definition given in Section 4.2, the SDOF damage and the actual overall of Frame 3-R are found, to be 0.5 and 0.45, respectively.

Table 4.3 Summary of earthquake ground motions

Date	Earthquake	Ms	Station	Dist. (km)	Component	PGA (g)	PGV (cm/s)	PGD (cm)
05/19/40	Imperial Valley	7.2	117 El Centro Array #9	12	I-ELC180	0.313	29.8	13.32
					I-ELC270	0.215	30.2	23.91
02/09/71	San Fernando	6.5	135 LA - Hollywood Stor Lot	23	PEL090	0.21	18.9	12.4
					PEL180	0.174	14.9	6.25
10/15/79	Imperial Valley	6.9	6621 Chihuahua	17.7	H-CHI012	0.27	24.9	9.08
					H-CHI282	0.254	30.1	12.89
10/15/79	Imperial Valley	6.9	6605 Delta	32.7	H-DLT262	0.238	26	12.06
					H-DLT352	0.351	33	19.02
10/15/79	Imperial Valley	6.9	5058 El Centro Array #11	12.6	H-E11140	0.364	34.5	16.07
					H-E11230	0.38	42.1	18.59
10/18/89	Loma Prieta		47380 Gilroy Array #2	12.1	G02000	0.367	32.9	7.15
					G02090	0.322	39.1	12.07
10/18/89	Loma Prieta	7.1	57382 Gilroy Array #4	24.3	G04000	0.417	38.8	7.09
					G04090	0.212	37.9	10.08
10/18/89	Loma Prieta	7.1	1028 Hollister City Hall	28.2	HCH090	0.247	38.5	17.83
					HCH180	0.215	45	26.1
10/18/89	Loma Prieta	7.1	1601 Palo Alto - SLAC Lab	35	SLC270	0.194	37.5	9.96
					SLC360	0.278	29.3	9.72
10/18/89	Loma Prieta	7.1	1695 Sunnyvale - Colton Ave.	27.5	SVL270	0.207	37.3	19.11
					SVL360	0.209	36	16.9
01/17/94	Northridge	6.8	90053 Canoga Park - Topanga Can	15.8	CNP106	0.356	32.1	9.13
					CNP196	0.42	60.8	20.17
01/17/94	Northridge	6.8	90057 Canyon Country - W Lost Cany	12.2	LOS000	0.41	43	11.75
					LOS270	0.482	45.1	12.58
01/17/94	Northridge	6.8	90006 Sun Valley - Roscoe Blvd	12.3	RO3000	0.303	22.1	7.84
					RO3090	0.443	38.2	10.04
01/17/94	Northridge	6.8	90003 Northridge - 17645 Saticoy St	13.3	STC090	0.368	28.9	8.44
					STC180	0.477	61.5	22.06
01/17/94	Northridge	6.8	90091 LA - Saturn St	23.2	STN020	0.474	34.6	6.55
					STN110	0.439	39	6.45

Figure 4.9 Response spectra of selected ground motions (damping ratio=5%). The mean spectra are shown by thicker lines.

Figure 4.10 Typical inelastic responses of SDOF and MDOF systems in time history analysis

4.3.5.3 Correlation of mean SDOF-MDOF damage

Repeating the above procedure, a total of 3840 time history analyses are performed for the 8 frames and their SDOF systems using the 30 ground motions scaled to 8 damage levels ($D_{SDOF} = 0, 0.1, 0.2, 0.4, 0.5, 0.6, 0.8, 1.0$). Figure 4.11 shows the scatter plots of SDOF damage vs. actual frame damage and the best-fit linear relationship for each frame. In these graphs, each data point represents a pair of SDOF and MDOF damage from the time history analyses using the same ground motion. Besides, the static damage relationship (in dashed line) as obtained from the pushover analysis is also plotted in the same graph for comparison.

In general, the actual global damage in each frame increases with an increase of the SDOF damage. Based on the best-fit linear relationship, the ratio between the global and SDOF damage is in the range of 0.7~0.8 in most cases. As a matter of fact, the relationship as obtained from the pushover analysis represents very well the statistical relationship established from the time history analysis. The above results indicate that the static damage relationship can be employed to represent the dynamic damage relationship in determining the global damage in an actual frame.

(a) Frame 3-R and 3-I

(b) Frame 7-R and 7-I

(c) Frame 12-R-a and 12-I-a

(d) Frame 12-R-b and 12-I-b

Figure 4.11 Correlation of damage of SDOF and MDOF systems in both pushover and time history analyses

Figure 4.12 Ratios of MDOF damage to SDOF damage

For simplicity, an approximate MDOF-to-SDOF damage ratio (γ_D) may be used to directly determine the global damage in an actual frame in lieu of a pushover analysis. Figure 4.12 summarizes the MDOF-to-SDOF damage ratios based on the best-fit linear lines for all frames with different regularity index. As can be seen, the damage ratio is rather stable and is generally independent of the regularity status and the number of storeys. Therefore, a constant ratio of 0.75 may be used for general frame cases.

4.4 Prediction of distribution of damage in multi-storey frames

Besides the global damage, the damage distribution along the frame height is also very important in seismic damage evaluation. It is reasonable to expect that the distribution of damage as obtained from a pushover analysis can represent to a certain extent the actual damage in the frame under seismic ground motions. On the other hand, the distribution of damage is also expected to correlate with the profile of the storey capacity factor, in a way such that a lower storey capacity factor would correspond to higher storey damage

Based on the above consideration, two possible methods are investigated for the distribution of damage in actual frames, one using a pushover analysis and the other based on the evaluation of the storey capacity factor. In the pushover method, both ordinary pushover analysis with a single lateral force pattern and modal pushover analysis with the first two modes are considered.

4.4.1 Method based on damage distribution from pushover analysis

4.4.1.1 *Pushover analysis with a single lateral force pattern (Ordinal pushover analysis)*

Figure 4.13 shows the static storey damage distributions from the pushover analysis (with lateral force pattern-A) as compared with the mean dynamic damage distributions of all frames at a damage level $D_{SDOF}=0.4$.

Generally speaking, for low-rise cases (Frame 3-R and 3-I), the static damage distribution is in good agreement with the dynamic distribution. With the increase of the number of storeys, the pushover analysis tends to underestimate the damage in the upper storeys due to the inability of representing higher mode effects. In certain cases such as frame 12-R-b, the pushover analysis also yields overestimation of damage in lower storeys.

As discussed in Section 4.3, the pattern-B lateral force pattern imposes increased force effects on the upper storeys, and this is expected to improve the pushover analysis in terms of representing higher mode effects. Figure 4.14 shows the static damage distribution in the 7-storey and 12-storey frames with the pattern-B lateral force. It can be seen that, comparing to the mean dynamic results, the static damage distributions have improved markedly, especially for the 7-storey frames. However, noticeable underestimation of damage in the upper storeys still exists in frames 12-R and 12-I. Further improvement would seemingly require the incorporation of higher modes in the pushover analysis.

Figure 4.13 Comparison of static damage distribution using 1st mode lateral force distribution with mean dynamic damage distribution at $D_{SDOF}=0.4$

Figure 4.14 Comparison of static damage distribution using pattern-B lateral force distribution with mean dynamic distribution at $D_{SDOF}=0.4$

4.4.1.2 Modal pushover analysis (first two modes)

For medium- to high-rise frames, higher mode effects may become significant under certain ground motions, thus affecting the damage distribution along the height of the structure. In such a situation, it is reasonable to expect that a combination of the damage distributions from a modal pushover analysis with consideration of the first two modes can yield a better prediction of the damage distribution. The combination may be expressed as

$$D = \eta_1 D_1^{mode} + \eta_2 D_2^{mode} \quad (4.30)$$

where D_1^{mode} and D_2^{mode} are the first and second modal damage distribution obtained from the modal pushover analysis corresponding to the same global damage; η_1 and η_2 are the damage combination factors. The general η_1 and η_2 values can be determined by fitting the combined damage distributions from the pushover analysis to the mean distributions obtained from the time history analyses for typical frame configurations, as will be discussed in what follows.

Figure 4.15 shows the mode shapes and the modal damage distributions of Frame 7-R, 12-R-a and 12-R-b at $D_{SDOF}=0.4$ as an example. As can be seen, in the first modal damage distribution, the damage is more concentrated at lower storeys. The reverse trend can be found in the second ‘modal’ damage distribution.

(a) First two modal shapes of Frame 7-R, 12-R-a and 12-R-b

(b) Modal damage distributions of Frame 7-R, 12-R-a and 12-R-b

Figure 4.15 First two modal shapes and modal damage distributions of Frame 7-R, 12-R-a and 12-R-b at $D_{SDOF}=0.4$

The modal damage combination factors (η_1 and η_2) obtained for frames 7-R, 7-I, 12-R-b and 12-I-b are summarized in Table 4.4. Figure 4.16 illustrates the variation of the damage combination factors with respect to the damage level. As can be seen in Figure 4.16(a), the η_1 factors, which represents the first mode contribution, falls in a range of 0.7~1.1, whereas the η_2 factors for the second mode contribution varies in a range of 0.1~0.5. η_1 tends to decrease with an increase of α_{sc} (more regular frames), and it also tends to decrease with an increase of the number of storeys. Similar trends also hold for η_2 (Figure 4.16 (b)). However, with an increase of the damage level, η_1 appears to decrease, while η_2 tends to increase.

Based on the above observations and by means of a regression analysis, η_1 and η_2 can be expressed by the following empirical formulas, as

$$\eta_1 = 1.02 + D(1.14 - 0.06N - 1.38\alpha_{sc}) \quad (4.31)$$

$$\eta_2 = (D + 1)(-0.67 + 0.04N + 0.97\alpha_{sc}) \quad (4.32)$$

where D is the damage level; N is the number of storey, and α_{sc} is the regularity index. The predicted η_1 and η_2 by Eqs.(4.31~32) are compared with the actual values on the right hand side of Figure 4.16. A satisfactory match is observed.

Table 4.4 Summary of modal damage combination factors

D_{SDOF}	7-R $\alpha_{sc}=0.66$		7-I $\alpha_{sc}=0.51$		12-R-b $\alpha_{sc}=0.61$		12-I-b $\alpha_{sc}=0.53$	
	η_1	η_2	η_1	η_2	η_1	η_2	η_1	η_2
0.2	1.02	0.2	1.050	0.030	0.908	0.43	0.990	0.244
0.4	1.0	0.38	1.035	0.080	0.863	0.5	0.980	0.354
0.6	0.97	0.43	1.020	0.140	0.75	0.59	0.970	0.390
0.8	0.89	0.49	1.012	0.205	0.74	0.592	0.940	0.410

(a) η_1 factor

(b) η_2 factor

Figure 4.16 Damage combination factors

With the above proposed η_1 and η_2 factors, Figure 4.17 shows damage distributions predicted by Eq.(4.30) using the modal pushover analysis in comparison with the mean distribution for 7- and 12-storey frames at different damage levels. In general, the predicted distributions agree favourably with the mean distribution from the time-history analysis. Some discrepancy is observed for Frame12-R-a and 12-I-b as shown in Figure 4.17(c)~(d). Further examination of the damage distribution when these frames are subjected to the second mode pushover analysis indicates that the corresponding damage is rather concentrated in the top 2~3 storeys due to the development of a mixed beam-column hinge mechanism (see Figure 4.15, the center plot, and Figure 4.18). On the other hand, the damage in the first mode pushover analysis is more concentrated in the lower storeys. The combinations of such two modal damage profiles inevitably create an underestimated damage at around the two-third height of the frame in these special cases. This demonstrates the potential shortcoming of using the modal pushover analysis considering the first two modes.

As will be discussed in Section 4.4, the above problem seems to be readily avoided in the simplified design-oriented method using the storey capacity factor.

For the more usual cases where a dominant beam hinging mechanism may be developed, apart from the storey damage distribution, the beam and column distributions can also be well predicted from the corresponding damage distributions in the pushover analyses. Figure 4.19 illustrates the average beam and column damage distributions at $D_{SDOF}=0.4$. As can be seen, the local static damage prediction also achieves a satisfactory agreement with the mean damage distribution.

(a) Frame 7-R

(b) Frame 7-I

(c) Frame 12-R-a

Figure 4.17 Comparison of storey damage distributions from pushover analyses and mean distribution from time history analyses

(d) Frame 12-I-a

(e) Frame 12-R-b

(f) Frame 12-I-b

Figure 4.17 (continued)

(a) Frame 12-R-a

(b) Frame 12-R-b

Figure 4.18 Plastic hinge pattern of Frame 12-R-a and 12-R-b under the second mode lateral force distribution at $D_{SDOF}=0.4$

(a) Frame 3-R and 3-I under lateral force pattern-A

(b) Frame 7-R and 7-I by modal pushover analysis

(c) Frame 12-R-a and 12-I-a under lateral force pattern-B

(d) Frame 12-R-b and 12-I-b by modal pushover analysis

Figure 4.19 Beam and column damage distributions of frames at $D_{SDOF}=0.4$
(The damage value given here is an average value of damage indices for each storey)

4.4.2 Method based on capacity factor distribution

The capacity factor introduced in Section 4.2 is a comprehensive measure of the relative storey strength and stiffness in a frame. Inspection of the storey capacity factor distributions in Figure 4.5 and the mean storey damage in Figure 4.13 reveals a close correlation, such that a lower storey capacity factor generally correlates with larger damage. As a matter of fact, the inversed storey capacity factor distribution is almost proportional to the mean damage distribution. This provides a potentially convenient means for a quick estimation of the damage distribution in a design environment.

As the overall damage of a frame is correlated with the damage in its equivalent SDOF system, the damage at a storey level can be directly associated with the SDOF damage via the storey capacity factor as

$$D_{sc} = (1/i_{sc})_n D_{SDOF} \eta_s \quad (4.33)$$

where $(1/i_{sc})_n$ is the normalized $1/i_{sc}$ such that the maximum value in all storeys equals unity, η_s is a coefficient relating the maximum storey damage to the SDOF damage. η_s is taken equal to 1.0. The rationale for this is that the SDOF damage is largely controlled by the critical (maximum) damage in the frame. Take the ultimate state for example, when the maximum damage in a storey reaches the critical state, i.e., $D_{sc,max} = 1.0$, the SDOF system is also regarded as reaching the ultimate state and hence D_{SDOF} is equal to 1.0 as well.

Figure 4.20 shows the damage distribution predicted by Eq.(4.33) as compared with the mean dynamic damage distribution. It can be observed that in most cases the predicted distributions of damage agree well with the dynamic results. The accuracy of the prediction is deemed acceptable for design evaluation purpose.

As mentioned in Section 4.2, the present storey capacity factor takes into account the influence of the storey stiffness and strength as constants throughout the entire

inelastic response range. A more appropriate treatment should reflect the decreasing influence of stiffness and increasing effect of storey strength with increase of the inelastic response. The likely variation of the effect of the strength and stiffness irregularity with respect to the ductility level will be discussed in the next chapter.

(a) Frame 3-R and 3-I

(b) Frame 7-R and 7-I

(c) Frame 12-R-a and 12-I-a

Figure 4.20 Comparison of damage distributions by capacity factors and mean distributions

Figure 4.20 (continued)

4.5 Summary of proposed procedure and application examples

4.5.1 Summary

The implementation of the proposed procedure for predicting the global damage and the damage distribution in a multi-storey frame is summarized as follows (see also Figure 4.21):

- (1) Perform a pushover analysis to obtain the capacity curve of the frame. Obtain the equivalent SDOF system. At the same time, establish the static MDOF-SDOF damage relationship based on the static damage results from the pushover analysis.
- (2) Obtain the SDOF damage from the damage-based inelastic spectra in conjunction with the elastic seismic demand. In a design environment, the elastic seismic demand may be established from the design spectrum.
- (3) Obtain the global damage of the frame from the MDOF-SDOF damage relationship established at Step 1, or simply using the approximate conversion factor of 0.75.
- (4) Determine the distribution of damage using either the modal pushover method or the storey capacity factor method.

Figure 4.21 Flowchart of proposed procedure for damage evaluation

4.5.2 Assumptions and limitations

The proposed approach is subjected to the following assumptions and limitations:

- The R_D spectra at the SDOF level are currently available for firm soil sites, so they may not be applicable to frames located on soft soil sites. Besides, near-field ground motions are not included in the scope of the present study, therefore the current R_D spectra may not be extendable to near-field scenarios.

- For a given damage level, the static damage distribution obtained from the pushover analysis is regarded as a reasonable representation of the damage distribution from the dynamic analysis in an average sense considering a series of consistent ground motions.
- The structural response is generally assumed to be dominated by a single mode and the displacement shape of the mode remains constant throughout the time history response. As such, the proposed approach is applicable to low-to-medium rise frame structure dominated by the first mode of vibration.
- For high-rise frame structures such as the 12-storey frames considered in the examples, a modal pushover analysis in accordance with the proposed damage combination factors is recommended.

4.5.3 Application examples

Frame 7-R is used as an example to illustrate the procedure for damage evaluation. Herein we assume a mean elastic response spectrum exists regarding the local seismic hazard. Otherwise, a design spectrum may be employed in determining the elastic spectral demand on the structure. For the present case, the mean elastic acceleration spectrum and the damage-based R_D spectra according to Eq.(4.4a-c) are shown in Figure 4.22.

Figure 4.22 Seismic hazard by mean elastic spectra

Step 1: pushover analysis for deformation capacity and transformation

The pushover analysis is performed with the lateral force pattern-A until the first member failure occurs. The capacity curve is obtained as shown in Figure 4.7(a). The equivalent SDOF system is deduced as given in Figure 4.7(b). The equivalent SDOF system properties include: $q_y^*/m^*=0.19g$, $T_{eq}=1.11s$ and $\mu_u=5.85$. The static MDOF-SDOF damage relationship is shown in Figure 4.8.

Step 2: SDOF damage

Given the SDOF properties, the elastic strength demand (q_e^*) is found from the elastic spectral acceleration (Figure 4.22(a)), $S_{ae} = 0.49g$, and the force reduction factor is then calculated as

$$R_D = \frac{q_e^*}{q_y^*} = \frac{0.49m^*g}{0.19m^*g} = 2.60 \quad (4.34)$$

With T_{eq} , μ_u and R_D , the SDOF damage can be read from the R_D spectra (see Figure 4.22(b)), thus $D_{SDOF} = 0.39$.

Step 3: MDOF damage

From the static MDOF-SDOF damage relationship (Figure 9), the global damage of the frame is obtained as $D=0.30$. Alternatively, the SDOF-to-MDOF damage conversion factor of 0.75 can be employed, yielding $D = 0.75 \times 0.39 = 0.30$.

Step 4: Damage distribution

a) Using modal pushover method

From the pushover analysis in Step 1, the first modal damage distribution can be extracted for a global damage equal to 0.30. Performing another pushover analysis for the second mode to the same global damage, the second modal damage

distribution can be obtained as shown in Figure 4.23(a). According to Eqs.(4.31~32), the modal damage combination factors are found to be $\eta_1=0.96$ and $\eta_2=0.36$. The combined damage distribution is shown in Figure 4.23(b) in comparison with the mean distribution from the time history analysis.

Figure 4.23 Modal damage distributions and final distributions

b) Using storey capacity method

The damage distribution can also be predicted by projecting the distribution of the inversed storey capacity factor. According to the definition in Section 4.2, the storey capacity factors of Frame 7-R can be obtained and normalized as shown in Figure 4.24(a). With the SDOF damage found in Step2, the damage distribution of Frame 7-R can then be deduced by Eq.(4.33). Figure 4.24(b) compares the predicted damage distribution with the mean distribution from the time history analysis.

Figure 4.24 Prediction of damage distribution with storey capacity factors

4.6 Concluding remarks

A methodology is developed in this chapter to estimate the seismic damage of multi-storey RC frames in terms of both the overall damage and the damage distribution. The damage of the equivalent SDOF system of a multi-storey frame under a certain seismic intensity defined by an elastic response spectrum can be found conveniently from the damage-based inelastic (R_D) spectra. The overall damage of the actual frame can then be deduced from the SDOF damage. Subsequently, the distribution of damage in the frame can be established taking into account the strength and stiffness characteristics of the structure.

Two alternative methods may be employed for the implementation of the proposed damage evaluation procedure, one based on the modal pushover analysis, and the other based on the characterization using the storey capacity factor. In both methods, an ordinary pushover analysis is carried out first to obtain necessary information for defining the equivalent SDOF system and also establishing the reference relationship between the SDOF damage and the actual frame damage. For the damage distribution, the first method requires a modal pushover analysis to establish the first and second modal damage distributions. Through a combination formula, the combined damage distribution can be obtained. In the second method,

the storey capacity factor is employed to allow for an approximate distribution of the damage along the frame height in a straightforward manner.

A group of representative multi-storey frames, ranging from 3-storey to 12-storey in height and including both regular and irregular cases, are selected and nonlinear time history analyses for a large group of ground motions are carried out to establish and verify the proposed methodology. The results indicate that the proposed procedure is capable of providing adequate damage estimates for general frame structures. It is observed that the relationship between the overall in an actual frame and the SDOF damage as established from the pushover analysis can well represent that under seismic ground motions. Furthermore, a constant SDOF-MDOF damage conversion factor of 0.75 seems to work well as an approximate for most cases.

With regard to the damage distribution, it may be concluded that, with a first-mode consistent lateral force pattern, the pushover analysis can generally provide a reasonable damage estimation for low-to-medium rise frames such as 3- and 7-storey frames considered in the present study. The consideration of a concentrated force (say 20% of total base shear) at the top level usually leads to a better prediction of the damage distribution. A more general method for the damage estimation, especially for high-rise frame structures, may be achieved via performing a modal pushover analysis in accordance with the damage combination factors recommended in this study.

For a rapid design evaluation, the use of storey capacity factor is found to be an acceptable alternative for predicting the damage distribution along the height of a frame structure.

CHAPTER 5

PREDICTION OF SEISMIC INTER-STOREY DRIFTS OF FRAME STRUCTURES WITH STOREY CAPACITY FACTOR

5.1 Introduction

In the context of performance-based design, the performance of a building depends not only on the damage state of structural members, which has been the subject of Chapter 3 and 4, but also on the damage of non-structural components and building contents. To a large extent, the non-structural damage can be associated with the maximum inter-storey drift. In this chapter, the method of predicting the inter-storey drift distribution using the storey capacity factor, as introduced in Chapter 4, is further developed to take into account the varying effect of storey overstrength and stiffness with different inelastic response (ductility) levels.

Numerous research investigations have been conducted for the evaluation of inter-storey drift under seismic ground motions (Miranda 1999; Gupta and Krawinkler 2000; Fajfar 2000; Chopra et al. 2003), and some typical results have been adopted in current seismic provisions such as FEMA-365 (BSSC 2000). The inter-storey drift is generally predicted from the spectral displacement. The standard procedure usually includes two steps: 1) to determine the inelastic roof displacement from the spectral displacement, and 2) to estimate the critical inter-storey drift from the above roof displacement considering general structural characteristics such as the structure type and the number of storeys. Among other parameters, the structural vertical regularity is a crucial factor that can influence significantly the distribution of the lateral displacement and inter-storey drift in a frame structure. Although this

factor is usually taken into account in many existing methods one way or the other, a direct quantification of the structural irregularity and its correlation with the inter-storey drift distribution have not been explicitly incorporated in the commonly used methodologies. On the other hand, due to architectural and functional requirements, many building structures are indeed designed with varying degree of vertical irregularities. Therefore, it is of practical significance to appropriately incorporate the irregularity factor in the estimation of the lateral displacement and inter-storey drift in a frame structure.

In the approach introduced in Chapter 4 for the evaluation of the damage/inelastic deformation distribution in a frame structure, the vertical irregularity of a frame is quantified on the basis of the storey strength and stiffness via the so-called capacity factor (i_{sc}), which is defined as a product of the storey overstrength factor (i_{os}) and the storey stiffness factor (i_{ns}). It has been found that the profile of the storey capacity factor correlates (inversely) with the inter-storey drift distribution reasonably in a moderate inelastic response stage. Generally speaking, larger capacity factors correspond to lower inter-storey drifts, and vice versa. However, it has also been observed from the associated nonlinear analysis that, comparing to the overstrength component, the storey stiffness component tends to have a decreased influence on the inter-storey drift distribution as the inelastic response (ductility level) increases. As a matter of fact, this trend is physically explicable; with the increase of the inelastic response, the elastic deformation contribution, which is directly associated with the stiffness, decreases in the total displacement responses. Therefore, a more appropriate treatment of structural irregularity as represented by the storey capacity factor has to take into account its dependency upon the inelastic response or ductility levels.

Previous studies (Al-Ali and Krawinkler 1998; Chintanapakdee and Chopra 2004) and preliminary analysis conducted in the present investigation have revealed that the roof displacement is generally not sensitive to the vertical irregularity of a frame structure. For this reason, in the present study the main focus will be placed on the

evaluation of the inelastic response dependent storey capacity factor in terms of its effect on the inter-storey drift distribution.

5.2 Proposed modification to the storey capacity factor for varying response level

As mentioned in Section 5.1, it can be expected from the physical point of view that the influence of the storey stiffness on the inter-storey drift distribution will decrease with increase of the inelastic response level. On the other hand, it can also be anticipated that the storey overstrength will have a lesser effect on the inter-storey distribution at a relatively low inelastic response stage. Herein we use the overall response ductility level, which is defined as the ratio of the maximum inelastic roof displacement at a particular response stage to the corresponding yield displacement, to represent the inelastic response level in the storey capacity factor evaluation.

To illustrate the above trends, a generic 7-storey irregular frame (designated as Frame 7-SK) is designed as a modified version of Frame 7-R in the series of representative frames considered in Chapter 4. This particular frame is purposely designed so that it exhibits a pronounced strength irregularity (non-uniformity of the overstrength factor) at the 2nd and 3rd storeys as compared to the remaining storeys, while marked stiffness irregularity occurs at the 4th and 5th storeys. This special configuration will allow the effect of strength and stiffness on the inter-storey drift distribution at different inelastic response levels to be clearly demonstrated.

Figure 5.1 depicts the storey overstrength, storey stiffness and storey capacity factors of Frame 7-SK. As can be seen, lower strength and stiffness factors appear at storeys 2~3 and storeys 4~5, respectively. As a result, the storey capacity factors exhibit lower values at the middle portion of the frame, especially at the 4th~5th storeys due to particularly low stiffness factors.

Figure 5.1 Storey overstrength, stiffness and capacity factors of frame 7-SK

Figure 5.2 Inter-storey drift distributions at different response levels in Frame 7-SK from pushover analysis

The frame 7-SK is subjected to a pushover analysis with the general lateral force pattern-A. Figure 5.2 shows the inter-storey drift distributions at different inelastic response levels, which are represented by the corresponding response ductility levels. Herein the ductility level, μ , is defined as the ratio of the maximum roof displacement at the particular response stage concerned, to the yield displacement. The yield displacement can be obtained by a bilinear idealization of the actual pushover curve as described in Chapter 4 following the procedure in FEMA 356 (BSSC 2000).

As expected, the inter-storey drift is generally governed by the storey stiffness distribution (represented by the storey stiffness factor in Figure 5.1) at the elastic stage (with an overall $\mu=0.5$), showing larger inter-storey drift at the 4th~5th storeys where the storey stiffness is relatively weak. On the other hand, the storey overstrength distribution tends to play a governing role at the advanced inelastic stage (e.g. $\mu>2$), such that larger inter-storey drifts occur around storeys 2~3 where the storey overstrength is lower, despite the marked lower storey stiffness factor in the upper storeys.

Due to the underlying physical reasons, the above observation can reasonably be extended to apply in general frame cases. Consequently, the use of a constant storey capacity factor, independent of the inelastic response level, is not capable of reflecting the changing effect of the stiffness and strength irregularity at different inelastic response or ductility levels.

In light of the above discussion and the pushover analysis, a general modification to the capacity factor may be expressed as

$$i_{sc} = \begin{cases} i_{ns} & \mu \leq 0.5 \\ \left(i_{os}\right)^{1-\frac{0.5}{\mu}} \times \left(i_{ns}\right)^{\frac{0.5}{\mu}} & \mu > 0.5 \end{cases} \quad (5.1)$$

The response limit for the sole influence of the storey stiffness is set at an overall ductility of 0.5 because beyond this level certain members in the frame will start yielding. By the use of the exponential expression, the decreasing effect of the storey stiffness (i_{ns}) and increasing effect of the storey overstrength factor (i_{os}) with increase of the response ductility level is well represented, as illustrated in Figure 5.3. The storey overstrength effect becomes dominant beyond a response ductility level of 2~3, which is consistent with the exploratory pushover analysis.

Figure 5.3 Variation of the exponentials of stiffness and strength factors against ductility level

Figure 5.4 Capacity factors at different ductility levels

Figure 5.4 depicts the storey capacity factors calculated according to Eq.(5.1) for four inelastic response (ductility) levels. It can be seen that the new capacity factors correlate well with the inter-storey drift distributions shown in Figure 5.2(b). At the elastic stage, relatively lower storey capacity factor appears at 4th~5th storeys, due to a lower storey stiffness factor, and correspondingly, larger interstorey-drift occurs in these storeys. On the contrary, at advanced inelastic stages, relatively lower storey capacity factor shifts to storeys 2~3, and correspondingly, larger inter-storey drift also occur in storeys 2~3.

Because of the sound correlation, the new capacity factor may be used directly to predict the inter-storey drift distribution at any response level of interests. As the storey capacity factor is inversely related to the inter-storey drift, the prediction of the inter-storey drift distribution may be expressed in a simple way as

$$\theta_{sc} = \left(1/i_{sc}\right)_n \theta_{\max} \quad (5.2)$$

where $(1/i_{sc})_n$ is the inversed capacity factor normalized so that the maximum value is equal to unity, $\theta_{\max}$ is the maximum inter-storey drift ratio.

Figure 5.5 compares the predicted inter-storey drift distributions from the capacity factors using Eq. (5.2) and the pushover analysis for frame 7-SK. It can be seen that the prediction of inter-storey drift distributions achieves a good agreement with that from the pushover analysis. With the new capacity factor, the varying effect of the storey stiffness and strength on the inter-storey drift distribution is well reflected from the elastic stage to the advanced inelastic response levels.

Further verification of the correlation between the new storey capacity factor and the inter-storey drift distribution with Eq. (5.2) is carried out on different frames by nonlinear time history analysis results, as will be described in the next section.

Figure 5.5 Comparison between predicted storey drift and those from pushover analysis

5.3 Verification of the modified storey capacity factor for inter-storey drift distribution with nonlinear time history analysis

Nonlinear time history analysis is carried out on Frame 7-SK for the same group of ground motions used in Chapter 4. The ground motions are scaled to various intensity levels to result in different levels of inelastic responses (or damage indices in the context of chapter 4). From the analysis for each ground motion at a given scaled intensity, the inter-storey drift distribution of the frame is obtained. The mean inter-storey drift distribution for all the ground motions at the particular intensity level is then obtained. This mean distribution is used to represent the seismic inter-storey drift distribution for the frame at the corresponding inelastic response level, which is represented by the ductility factor calculated as the ratio of the mean roof displacement to the yield displacement obtained from the pushover analysis.

Figure 5.6 compares the mean inter-storey drift distributions from the time history analyses and predicted distributions using Eq. (5.2) for Frame 7-SK at different response levels. It can be seen that the predicted inter-storey drift distributions have good agreement with the mean distributions at different ductility levels.

Following the same procedure, the mean inter-storey drift distributions of the eight frames analyzed in Chapter 4 are processed from their nonlinear analysis results, and the corresponding predictions using Eq. (5.2) are calculated. Figure 5.7 compares the predicted storey drift distributions with the mean distributions for all the eight frames. Generally speaking, the predictions using the new capacity factors show a consistent agreement with the mean seismic distributions at different ductility levels.

It is noted that the use of the storey capacity factor for the prediction of the inter-storey drift distribution provides only a relative distribution profile, given a known maximum (critical) inter-storey drift ratio $\theta_{\max}$. In the actual application for the complete prediction of the actual inter-storey drifts, it is necessary to estimate the maximum or critical inter-storey drift in the entire frame for a certain response level.

This may be achieved by relating the critical inter-storey drift to the roof drift, which can be estimated from a standard procedure based on the spectral displacement, through the regularity index (α_{sc}). This will be discussed in the section that follows.

Figure 5.6 Comparison between predicted storey drift distributions and mean distributions from time history analysis of Frame 7-SK

(a) Frame 3-R

(b) Frame 3-I

Figure 5.7 Comparison between predicted inter-storey drift distributions and the mean distributions from time history analysis

Figure 5.7 (continued)

(f) Frame 12-I-a

(g) Frame 12-R-b

(h) Frame 12-I-b

Figure 5.7 (continued)

5.4 Estimation of critical inter-storey drift ratio

In a recent study in which the author was also involved (Lu et al. 2006), it was found that good correlation exists between the inter-storey drift concentration factor ($\theta_{\max}/\theta_{r,peak}$, where $\theta_{r,peak}$ denotes the peak roof drift) and the regularity index (α_{sc}), which has been defined in Chapter 4 (see Eq.(4.14)). Generally speaking, the drift concentration increases with the increase of irregularity (decrease of the regularity index α_{sc}). Based on the analysis of six frames, an empirical relationship between the drift concentration factor and the regularity index was proposed, without considering the variation of structural irregularity with inelastic response level, as:

$$\theta_{\max}/\theta_{r,peak} = 1 + 1.2\left(\frac{1}{\alpha_{sc}} - 1\right) \quad (5.3)$$

In the present study, the relationship between the inter-storey drift concentration and the regularity index is further investigated to examine whether it is dependent upon the inelastic response (ductility) level. Figure 5.8 gives the scatter plots of the maximum inter-storey drift vs. peak roof drift for representative frame 3-R, 7-R, 7-I and 12-R-a under the selected 30 ground motions. It can be seen that the maximum inter-storey drift and the roof drift has a good linear correlation (shown with the solid line) with a fairly small scatter throughout the entire range of response. This indicates that the drift concentration factors are not sensitive to the level of inelastic response (ductility). Therefore, the use of a constant regularity index for the same frame is confirmed to be sound.

The inter-storey drift concentration factors as obtained from the best-fit linear relationships for all the nine frames (including frame 7-SK) are plotted in Figure 5.9, together with the previous data for other frames and the predictions by the empirical Eq.(5.3). It can be seen that all the data points show a consistent trend. With the addition of the data from the present study that are more in the relatively regular range ($\alpha_{sc} > 0.6$), it appears that there is a need to adjust the empirical formula to

maintain consistency with the entire set of data points. By means of a fitting analysis, the modified empirical expression is expressed as

$$\theta_{\max} / \theta_{r, peak} = 1.2 + 1.1 \left(\frac{1}{\alpha_{sc}} - 1 \right) \quad (5.4)$$

where the lower bound of 1.2 is considered to take into account the fact that even in a “perfect” case with $\alpha_{sc}=1.0$, a minimum degree of inelastic concentration could still occur due to the nature of dynamic response and variation of ground motions. Similar observations were also found in some previous studies (Gupta and Krawinkler 2000)

Figure 5.8 Scatter of maximum inter-storey drift vs. peak roof drift

Figure 5.9 Variation of $\theta_{\max}/\theta_{r,peak}$ with regular index α_{sc}

Miranda (1999) proposed a method to obtain nonlinear inter-storey drift ratio from elastic displacement spectrum, which has been described in more detail in Chapter 2. For a general comparison with Miranda's method, the critical drift concentration referred to in the present study can be expressed in Miranda's terms as

$$\theta_{\max}/\theta_{r,peak} = \beta_2\beta_4 \quad (5.5)$$

where β_2 is the amplification factor accounting for the concentration of inter-storey drifts, and β_4 is the factor accounting for the increase of concentration of inter-storey drift ratio due to nonlinear response. According to Miranda (1999), for the shear-type frames considered in this study, β_2 equals 1.5 for all cases. For a storey drift ductility of order of 3.0, β_4 can be calculated for the 3-, 7- and 12-storey frames, respectively. Finally, the drift concentration factor is found to be 1.65, 1.70 and 1.74 for the 3-, 7- and 12-storey frames, respectively, and the results are indicated in Figure 5.9.

It can be seen that the predicted drift concentration using Miranda's method represents the results for relatively regular frame cases with a regularity factor

around 0.7. It tends to underestimate the drift concentration for more irregular frames with a regularity index below 0.6.

Summarizing the procedures for a complete estimation of the inter-storey drift ratio and its distribution along the frame height, a flowchart for the implementation of the procedures is given in Figure 5.10.

Figure 5.10 Flowchart of proposed procedure for prediction of inter-storey drifts

5.5 Summary and conclusions

In this chapter, a simple method is proposed for the prediction of the inter-storey drift distribution on the basis of the modified storey capacity factor as a combination of the effects of the storey overstrength and stiffness, taking into account the inelastic response (ductility) levels. The procedure can be applied to predict seismic storey drift distribution in a preliminary design of new buildings or a rapid seismic evaluation of existing buildings.

The proposed method stems from the fact that the displacement response profile along the height of a frame, or the inter-storey drift distribution, depends on the vertical structural regularity profile, which in turn depends on the storey-shear overstrength and stiffness factors. It is physically clear that the influence of the strength and stiffness factors on the inter-storey drift will generally increase and decrease, respectively, with the increase of the inelastic response (ductility) level. A representative case study clearly demonstrates such a trend. On this basis, a modified storey capacity factor is proposed as a product of an overstrength term that increases with the response ductility and a stiffness term which decreases with the ductility in an exponential form.

Results from the nonlinear time history analysis on nine representative frames of 3, 7, and 12 storeys demonstrate that the modified storey capacity factor (i_{sc}) has good correlation with the inter-storey drifts in a consistent manner at different ductility levels, and the distribution of the normalized $1/i_{sc}$ can be directly employed to predict the inter-storey drift profile..

Further refinement of the approach to predict the inter-storey drift concentration factor as a function of the structural regularity index, which is defined on the basis of the storey capacity factor, is also carried out. Statistical results from the nonlinear time history analyses for a group of earthquake ground motions indicate that the inter-storey drift concentration factor is not sensitive to the level of inelastic response. Based on the enlarged dataset, an improved empirical formula is proposed for the prediction of the critical inter-storey drift.

CHAPTER 6

EVALUATION OF DAMAGE AND STRUCTURAL PERFORMANCE OF NONSEISMICALLY DESIGNED FRAMES – AN EXPERIMENTAL STUDY

6.1 Introduction

Structures designed before the introduction of modern seismic codes, or those located in low seismic regions are generally called nonseismically designed (gravity-designed) structures. Such structures are relatively vulnerable to seismic ground motions as compared with those conforming to modern seismic design codes. More recently, the actual performance of nonseismically designed structures, especially concerning the reliability and risk assessment, has received increasing attention in many parts of the world, such as the eastern and central United States, Japan, New Zealand and Singapore.

An extensive review of general performance of nonseismically designed structures and component connections has been presented in Chapter 2. It is felt that there is still a lack of comparative information concerning the behaviour of structures with different possible detailing arrangements. There is also a need to examine the behaviour of such structures in the context of a moderate seismic demand. This is a particularly important aspect since under a moderate ground shaking, which may be rightfully assumed as the worst case in regions where non-seismically designed structures are allowed to exist, some of the potential problem with undesired detailing, such as insufficient extension length, etc, may simply not occur. Indeed, the potential problem with gravity-designed buildings must be viewed from the

perspective of a realistic seismic demand. In the present study, special attention is placed in observing the behaviour of undesired detailings as to what extent they may cause problems at different levels of ground motions. To enhance the observation, a companion design which duplicates all the main longitudinal reinforcement of the nonseismically designed frame, but with some modifications in the arrangement of lap splices of longitudinal reinforcement and limited increase of stirrups in conformation to the intermediate seismic requirements in ACI 318-02 (ACI 2002), is also made as a comparison.

Another main objective of this experimental study is to acquire test data with regard to the serviceability and damage control performance levels of the frame structures. For building structures located in moderate seismic regions, structural collapse may not be of a primary concern; rather, good understanding about the potential degree of damage becomes more important. With the experiment, pertinent information with regard to the performance limit criteria can be evaluated.

An important characteristic of the seismic ground motions in low-to-moderate seismic regions is a longer duration. According to relevant studies on probable worst case scenario earthquakes for Singapore (e.g., Megawati and Pan 2002; Kirke and Hao 2004), the duration of ground motions in Singapore could be as long as 100s or more. Such a long duration should be treated only as a tendency but not a certainty. It basically says that the duration of low-to-moderate ground motions could be longer or much longer than that of strong ground motions. As such, the cumulative damage at relatively low displacement levels could play a significant role in the overall damage of structures. In the present study, particular attention is paid to the effect of cumulative damage on the structural performance.

In summary, the main objectives of the present experimental programme include: 1) to assess the seismic behaviour of the non-seismically designed frames under realistic earthquake demands, with particular attention to the possible extent of the adverse effect that may be caused by the undesired detailing; 2) investigate the damage pattern and limit criteria in the context of performance-based design; and 3)

to assess the effect of long-duration ground motion on the structural performance due to cumulative damage.

6.2 Comparison of reinforcement details in seismic and nonseismic designs

For different seismic hazard levels, building structures need to be designed and detailed to achieve different ductility levels. For example, ACI 318-02 (ACI 2002) classifies concrete moment resisting frames into three categories: Ordinary Moment Resisting Frames (OMRF); Intermediate Moment Resisting Frames (IMRF); Special Moment Resisting Frames (SMRF). ACI 318-02 also specifies the design and reinforcement detailing for each type of frames. The selection of the type of frames should be made according to the seismic hazard level or seismic design category under consideration. The seismic hazard levels can be classified into low, moderate and high levels, for example according to the seismic zones specified in UBC 97 (ICBO 1997). The seismic design categories are specified in IBC (ICBO 2000).

Obviously, it is only meaningful to examine the effects of reinforcement detailing on the actual seismic behaviour of structures with respect to the seismic hazard level concerned. For example, in a low seismicity region, such as Singapore, the likely seismic performance of building structures should be evaluated with respect to low-to-moderate ground motions expected. However, in the literature, most of the studies have focused on the ultimate states of the nonseismic detailed structures. In the present study, the serviceability and intermediate performance levels are particularly emphasized, considering the level of seismicity involved.

The requirements of detailing for nonseismic design are less stringent than those for seismic design. The typical nonseismic details in design practice have been introduced in Chapter 3 and the main features are repeated as follows, and schematically illustrated in Figure 6.1.

- 1) Columns may be weaker than the adjacent beams, leading potentially to a soft

storey or column sidesway failure mechanism if a large earthquake would ever occur;

- 2) Beam bottom flexural reinforcing bars are discontinuous at the interior joints and are simply terminated in the exterior joints;
- 3) Lap splices of column reinforcement are located in the potential plastic hinge region just above the floor level ;
- 4) There is minimal transverse reinforcement in columns for shear and confinement, even in potential plastic hinge regions;
- 5) There is little or no joint transverse reinforcement within the joint core.

Figure 6.1 Typical nonseismic details of interior and exterior beam-column connections

The above nonseismic details conform to the detailing requirements for Ordinary MRFs in ACI 318-02. It is meaningful to compare the minimum detailing requirements adopted in the local design practice that follows BS 8110 (BS 1995) with those in ACI 318-02. Table 6.1 summarizes several important detailing aspects concerning beams, columns and joints. The minimum requirements of BS 8110 are quite similar to ACI Ordinary category; however they are less stringent than the requirements for ACI Intermediate and Special MRFs. For this reason, the detailing that conforms to BS 8110 is selected to represent the nonseismic case, while the detailing within the specification of IMRF in ACI 318-02 is selected to

represent a somewhat comparable seismic case. In the experimental study, the above two cases are represented in two test frames, respectively. The test frame designed to the nonseismic code, BS 8110, is designated as Frame FC, while the modified version that is made to conform with ACI 318-02 detailing requirements is designated as Frame FM.

Table 6.1 Comparison of minimum requirements of reinforcing details

Reinforcing details	BS 8110 1985	ACI 318-02		
		Ordinary	Intermediate	Special
Beam bottom flexural reinforcement at the joint	(3.12.9.1) $A_{et} \geq 30\% A_s$, $l_{em} \geq \max(12d_b, d)$	(12.11.1) $A_{et} \geq 1/3 A_s$ $l_{em} \geq 6 \text{ in. (150 mm)}$	(21.10) No additional requirement.	(21.3.2.3) Lap splice of flexural reinforcement should not be placed in the potential plastic hinge region.
Beam transverse reinforcement	(3.4.5.5) $s_0 < 0.75d$	(11.5.4) $s_0 \leq \min(d/2, 24 \text{ in})$	(21.10.4) $l_0 = 2h_b$ $s_0 = \min(d/4, 8d_b, 24d_t, 12 \text{ in})$	(21.3.3) Same as Intermediate
Column transverse reinforcement	(3.12.7.1) $s_0 \leq 12d_b$	(7.10.5.2) $s_0 \leq \min(16d_b, 48d_t, h_c)$	(21.10.5) $l_0 = \max(h_c, 1/6H_n, 18 \text{ in})$ $s_0 = \min(h_c/2, 8d_b, 24d_t, 12 \text{ in})$	(21.4.4) $l_0 = \max(h_c, 1/6H_n, 18 \text{ in})$ $s_0 = \min(h_c/4, 6d_b, s_x)$
Lap splices in column	It can be placed just above the floor level.	It can be placed just above the floor level.	(21.10) No additional requirement	(21.4.3.2) Lap splices should be located within the center half of the member length.
Joint transverse reinforcement	Minimal	(11.5.5) Minimal	(21.10) No additional requirement	(21.5.2) The transverse reinforcement in the critical regions of the framing columns will be continued through the depth of the joint

Notes:

A_{et} : positive bar area extended in the support;	h_b : beam depth;
l_{em} : embedment length of A_{et} ;	h_c : column depth
A_s : positive reinforcement area;	d_b : diameter of longitudinal steel bars;
s_0 : spacing of transverse bars over l_0 ;	d_t : diameter of transverse steel bars;
l_0 : length corresponding to spacing bars;	H_n : net height of member;
d : effective depth of member;	s_x : longitudinal spacing of transverse bars.

6.3 Experimental programme

6.3.1 Test frames

A pair of two-storey one-bay frame structures, representing respectively the nonseismic detailing (Frame FC) and the intermediate seismic detailing (Frame FM), is considered in this study. Both frames have the same dimensions but different reinforcing details. Frame FC is designed for gravity loads ($1.4D + 1.6L$) according to BS 8110 (BSI 1985). A 2.5 kN/m^2 live load and a 1.5 kN/m^2 superimposed dead load are included in the design as specified in the design code. The characteristic cube strength of concrete is 30 MPa (Grade 30) and the characteristic yield strength of flexural steel reinforcement is 460 MPa (Grade 460). Frame FM is actually a modified version of Frame FC with consideration of the minimum intermediate seismic details according to ACI 318-02 (ACI 2002).

Figure 6.2 Dimensions of test frames (FC and FM, Unit: mm)

The test frames were exactly half-scale models of the prototype frames. The dimensions of the test frames are shown in Figure 6.2. In these planner test frames, The effect of the original floor slab on the beam stiffness and strength was reflected with a slab flange (800mm) which well exceeds the anticipated effective slab width ($1/4$ of the span length, or 625mm at the 50% reduced scale) as recommended in ACI 318-02. Figure 6.3 and Figure 6.4 illustrate the reinforcing details for Frame

FC and FM, respectively. The close-ups of the steel rebar arrangement are shown in Figure 6.5.

Figure 6.3 Reinforcing details of Frame FC (Unit: mm)

Figure 6.4 Reinforcing details of Frame FM (Unit: mm)

(a) Joint reinforcement of Frame FC

(b) Joint reinforcement of Frame FM

(c) Column reinforcement of Frame FC

(d) Column reinforcement of Frame FM

Figure 6.5 Reinforcement arrangement of test frames

6.3.2 Material properties

The ready-mixed concrete with a maximum aggregate size of 10mm was used to cast the test frames. The concrete control specimens (standard cubic and cylinder specimens) were cured in the same condition. The average cube and cylinder compressive strengths at the time when the frames were tested were 53.5MPa and 40.9MPa, respectively.

The reinforcement used in the test frames was of the same mechanical properties as in the prototype design. The longitudinal reinforcing steel in the test frames comprised T10 (10mm diameter) deformed bars and the stirrup steel was R6 (6mm diameter) plain round bars. Table 6.2 summarizes the mechanical properties of the steel bars average yield strength and yield strain according to the steel coupon tests.

Table 6.2 Measured reinforcing steel properties

Steel bar	Grade	Yield strength (MPa)	Yield strain	Ultimate strength (MPa)
T10	460	534	0.002871	601
R6	250	473	0.002563	569

6.3.3 Mass similitude requirement

The mass similitude requirement should be satisfied for a proper simulation of the dynamic loads. In this study, the materials in the model frames were made similar to the normal concrete and steel materials. Therefore, the similitude laws of the true replica (Harris and Sabnis 1999) apply in the reduced scale models to maintain the equivalence at the stress level. The required mass of the model should be:

$$m_m^{req} = m_p \cdot \frac{1}{\lambda^2} \quad (6.1)$$

where m_m^{req} is the required mass of the test model; m_p is the mass of prototype structure; and λ is the geometric scale factor. However, the model structure has a mass:

$$m_m^{model} = m_p \cdot \frac{1}{\lambda^3} \quad (6.2)$$

To compensate the difference between the required mass and actual mass of the test model, additional mass need to be used:

$$\Delta m = m_m^{req} - m_m^{model} = \frac{\lambda - 1}{\lambda^3} m_p = m_m^{model} \quad (6.3)$$

In the present 1:2 scale models ($\lambda=2$), the additional mass was calculated to be approximate 700 kg for each storey. In the experiment, pre-prepared concrete blocks ($300 \times 400 \times 300 \text{ mm}^3$ each) were used as the additional mass and they were fixed on the slab with bolts. Due to the constraint of the shake table capacity, the live load was not considered for mass similitude. The scale factors for the model response quantities are summarized in Appendix C.

6.3.4 Experimental setup and Instrumentation

The test set-up was arranged so that the two planar frames were tested side-by-side simultaneously. In-between the two frames a pair of couplers were used. The couplers enabled the two frames to support each other against incidental out-of-plane movement, but allowed free sliding between the two frames in the direction of excitation. Figure 6.6 shows the two test frames mounted on the shake table. The additional masses (8 pieces of concrete blocks each storey) required by the similitude laws were distributed on the floor slab.

A series of instruments were installed to record the dynamic responses of the two test frames during the seismic tests. Displacement transducers (DT) were installed to measure the absolute horizontal displacements at each floor level. Several pairs of DTs were also installed at the beam and column ends of the ground floor to

measure the plastic hinge rotations. Accelerometers (A) were installed to record the horizontal accelerations at each floor level. An additional accelerometer was also mounted in the middle of the top floor beam of each frame to monitor the out-of-plan motion. Figure 6.7 illustrates the instrumentation, which is the same for both test frames.

Besides, a number of steel strain gauges were mounted on the reinforcing bars in potential critical regions at the constriction stage and measurements were taken from these strain gauges during the tests. Figure 6.8 illustrates the arrangement of the steel strain gauges in the test frames.

In addition, several digital video recorders and cameras were used to record the dynamic behavior of the test frames during the tests.

(a) Front view

(b) Side view

Figure 6.6 Two test frames mounted on shake table

Figure 6.7 Instrumentation of the test frames (sensor labels for Frame FM are given in the brackets)

Figure 6.8 Arrangement of steel strain gauges

6.3.5 Selection of input excitation and test procedure

The selection of the ground motion input was aimed to reflect the characteristics of typical ground motions in low to moderate seismic regions, particularly a long effective duration. Structures with nonseismic details could be more vulnerable to ground motions with longer duration because of cumulative damage.

For this purpose, the ground motion component N70W recorded at Vina del Mar site during the 1985 Chilean earthquake was selected as the reference input ground motion for the shake table tests. The source of the selected Chile earthquake has a magnitude (M_s) of 7.8 and it occurred off the coast of central Chile. The epicenter was located approximately 80km southwest of Vina del Mar, the record site. Figure 6.9 plots the acceleration time history and its response spectrum. The ground motion is characterized by a long effective period of about 40 seconds. The N70W component is selected due to its relatively flat plateau in the acceleration response spectrum within the period range of interest (0.2-0.8s), as compared with the other horizontal component (S20W). For reduced scale test models with a length scale factor of 2, the input excitation is obtained by compressing the time scale of the original ground motion using a time scale factor of $1/\sqrt{2}$ according to the similitude laws.

Before the actual earthquake test, the shake table motion was calibrated to simulate the target reference ground motion. Figure 6.10 shows the reference (input) and achieved base acceleration time history for CHL-023 (CHL indicates Chile record, and 023 indicates PGA of 0.23g) as an example. Figure 6.11 compares the corresponding pseudo-acceleration and displacement response spectra. It can be seen that the achieved base motion matches the target reference record very well, and the response spectra are in close agreement in the period range of 0.2-0.8s within which the actual natural periods of the test frames in different response stages are expected to fall. Some slight discrepancy exists outside the above period range, but this is not deemed to induce significant influence on the shake table tests.

The original record has a peak ground acceleration (PGA) of 0.23g, but the actual test programme started with a reduced PGA to represent a lower level ground motion. Trial test indicated that for a base motion with PGA lower than 0.1g, the response in the test frames was low and it was well within the elastic range. For this reason, the actual test programme started with a PGA of 0.12g (CHL-012) which was about 50% of the original ground motion record. This was followed by a test with PGA=0.23g (CHL-023). It was expected that sensible damage would occur at this level of test. To further test the inelastic behaviour of the frames, the test intensity was then increased to 0.46g, twice of the original record. In other words, three main tests with PGA equal to 0.12g, 0.23g and 0.46g were performed to represent minor, moderate and severe seismic intensity, respectively. Table 6.3 summarizes the test sequence of this experimental programme. In addition, complementary random tests (white noise) were conducted at the initial stage and after each earthquake test to capture the changes of the dynamic properties, which serves as an indicator of the damage states.

Figure 6.9 Accelerogram and response spectra of the selected original ground motion

Figure 6.10 Reference and achieved base (shake table) accelerations (CHL-023)

Figure 6.11 Comparison of acceleration and displacement response spectra for reference and achieved base motions (5% damping, CHL-023)

Table 6.3 Experimental testing sequence for test frames

Test number	Name	Description	Purpose
Shake table calibration			
1	RND-A	Random, PGA=0.02g	Pretest structural identification
2	CHL-012	Chile, PGA=0.1g, half intensity of original record	Minor shaking,
3	RND-B	Random, PGA=0.02g	Structural identification
4	CHL-023	Chile, PGA=0.23g, original record	moderate shaking,
5	RND-C	Random, PGA=0.02g	Structural identification
6	CHL-046	Chile, PGA=0.46g twice intensity of original record	Severe shaking,
7	RND-D	Random, PGA=0.02g	Structural identification

6.4 Test results

6.4.1 Measured responses

A series of earthquake simulation tests with increasing intensities were performed to the companion test frames. In the course of the tests, the measured responses included: 1) storey acceleration; 2) storey displacement; 3) fundamental frequency; 4) steel strain at member ends; and 5) member end rotation. The first three items are discussed in this section in detail.

a) Acceleration and storey shear

Figures 6.12-6.14 show the storey shear time histories of the test frames for each test (CHL-012, CHL-023 and CHL-046). The storey shear is obtained from the measured storey acceleration multiplied by the storey mass (1480kg). Under low-to-moderate ground motions, Frame FC has lower peak storey shear than Frame FM as shown in Figures 6.12-6.13. In the CHL-023 test, the peak base shear of Frame FC is about 80% of that of Frame FM. In the severe test with $PGA=0.46g$, however the peak values of base shear are quite close for both frames as shown in Figure 6.14. In this case, the maximum base shears are found to be about 20 kN, and the base shear coefficient, as a ratio of the base shear to the effective weight, is about 0.2.

b) Storey displacements

Figures 6.15-6.17 show the storey displacement time histories of the test frames for all the three tests. The storey displacement is the relative displacement at each storey level to the base. In the first two tests (CHL-012 and CHL-023), Frame FC exhibited lower displacement response as compared with Frame FM. For example, under the moderate ground motion (CHL-023), the top displacement of Frame FC was 14mm (or 0.4% of the frame height), about half of that in Frame FM (31mm or was 0.88%). However, the displacement response became quite close in the severe test of CHL-046, for which the peak top displacements in both frames was about 60 mm, or 1.7% of the frame height.

Figure 6.12 Storey shear time histories for CHL-012

Figure 6.13 Storey shear time histories for CHL-023

Figure 6.14 Storey shear time histories for CHL-046

Figure 6.15 Storey displacement time histories for CHL-012

Figure 6.16 Storey displacement time histories for CHL-023

Figure 6.17 Storey displacement time histories for CHL-046

(The last 20-minute response was missed due to recording error)

Table 6.4 Summary of peak responses in earthquake tests

EQ simulation test	FC			FM		
	Peak storey displ. (mm)	Peak inter-storey drift ratio (%)	Peak storey shear (kN)	Peak storey displ. (mm)	Peak inter-storey drift ratio (%)	Peak storey shear (kN)
CHL -012	5	0.22	6.06	15	0.55	12.11
CHL -023	14	0.60	14.31	31	1.15	17.00
CHL -046	58	2.70	21.62	63	2.83	19.46

Figure 6.18 Spectral responses with fundamental periods (5% damping)

Table 6.4 summarizes the peak responses of the test frames in each test. As mentioned above, during low-to-moderate tests ($PGA=0.12g$ and $0.23g$), the base shear and top displacement of the nonseismically designed frame (FC) are lower than that of the modified version (FM). Although there could be some uncertain contributing factors, the most obvious reason for the difference in the response between the two frames is deemed to be the difference in stiffness due to the splice arrangement. This argument is actually supported by the fact that the most significant difference in response happened at elastic and relatively low inelastic stages, for which the stiffness plays a more significant role. It should be pointed out that, although the equivalent cracked storey stiffness differs only by 18% between

the two frames (will be discussed later), the actual difference around the verge of cracking could be much more pronounced than what this figure implies, since at this response level the FC frame might still be responding in the uncracked range while the FM frame had already cracked.

Moreover, the reduced period of FC frame due to the increase in stiffness could have also reduced the seismic demands. Figure 6.18 illustrates the elastic spectral responses of the two frames at the measured fundamental periods before the CHL-023 test. It can be seen that the seismic demands to Frame FC in terms of the spectral acceleration and displacement are about 30% lower than that to Frame FM.

Under such a compounded effect, it is not surprising to see a considerable difference in the displacement responses between the two frames up to a moderate inelastic level. The difference in the natural frequencies of the two frames will be discussed next.

c) Measured dynamic properties

In general, the structural dynamic characteristics include the following: natural frequencies, model shapes, equivalent viscous damping ratios, stiffness matrix, and modal participation factors. The fundamental natural frequency is usually a primary factor of interest because it affects the general spectral demand on the structure and it also represents the structural degradation of the test frames during the course of the tests. The dynamic identification was carried out through random vibration tests in conjunction with standard identification methods such as half-power bandwidth method (He and Fu 2001).

A wide-band (0-50Hz) white noise, as shown in Figure 6.19, was used to determine the dynamic characteristics of the test frames. The peak base acceleration was scaled to 0.02g to provide enough excitation such that the modes of vibration could be identified. Figure 6.20 shows the frequency response function (FRF) amplitudes of the storey accelerations recorded during the initial random test (RND-A). The

natural (resonance) modes can be clearly identified. The initial fundamental frequencies of Frame FC and FM are found to be 4.59Hz and 4.05Hz, respectively. Repeating the same procedure as mentioned above, the structural dynamic characteristics, including damping, were identified after each earthquake test. The results are summarized in Table 6.5. In general, Frame FC has higher fundamental frequencies than Frame FM, especially at earlier stages.

Figure 6.19 Random base excitation for modal identification

Figure 6.20 FRF curves of storey accelerations for RND-A

Table 6.5 Dynamic characteristics of test frames

Random test	FC				FM			
	Frequency	Damping	Modal shapes		Frequency	Damping	Modal shapes	
RND_A	4.59	4.87	1.00	-0.61	4.05	5.40	1.00	-0.66
	13.73	4.70	0.64	1.00	11.75	4.35	0.61	1.00
Chile012g								
RND_B	4.22	5.03	1.00	-0.58	3.77	6.52	1.00	-0.65
	12.72	5.50	0.62	1.00	10.81	4.89	0.59	1.00
Chile023g								
RND_C	3.92	5.52	1.00	-0.54	3.34	7.22	1.00	-0.58
	12.35	5.82	0.62	1.00	10.28	5.43	0.56	1.00
Chile04g								
RND_D	2.05	8.09	1.00	-0.75	1.86	9.94	1.00	-0.86
	9.47	6.02	0.79	1.00	7.48	7.01	0.81	1.00

6.4.2 Inelastic response and hysteretic behaviour

a) Development of structural damage

The progressive damage with increasing base excitation was carefully inspected after each test. It was observed that the damage was mainly concentrated at member ends of the ground storey.

During the test with $PGA=0.12g$, there was no clear cracks on the test frames (Figure 6.21). The first crack appeared at the bottom ends of the ground-storey columns in both frames under the moderate ground motion of $PGA=0.23g$, as shown in Figure 6.22. It can be seen that only a few cracks appeared on Frame FC, especially at the beam bottom side. For Frame FM, more cracks developed at both beam and column ends. It is worth noting that under the moderate ground motion, no crack in the joint regions was observed. The crack width was measured to be about 0.2mm at this stage.

Figure 6.23 depicts the crack patterns after the CHL-046 test ($PGA=0.46g$). It can be seen that under such a strong ground motion the test frames sustained severe damage. The damage was generally concentrated on the beam and column ends of the ground floor. Slight diagonal cracks also appeared in the joint regions of both frames.

As shown in Figure 6.23(a), Frame FC exhibited one major crack at both column bottom ends. Large cracks also occurred at the beam ends and these cracks extended into the slab flange. Nevertheless, no apparent bond failure (pull-out) of the beam bottom rebars at the beam-column interface was observed, although the steel rebars had an extension length into joint by only 15 times of the rebar diameter, which was shorter than that required by ACI 318-02 (ACI 2002). This may be explained by the fact that in these nonseismically designed frames, columns are relatively weak, therefore a lesser inelastic deformation demand would incur at the beam ends that frame into the joints.

The damage of frame FM is generally similar to that of frame FC, except that a severer crack region formed at the top end of a ground storey column with visible concrete spalling, as shown in Figure 6.23(b).

The overall crack patterns of both frames are shown in Figure 6.24-25.

(a) Frame FC

(b) Frame FM

Figure 6.21 Close-up of observed damage after CHL-012

(The wires around the joint were used for other measurement)

(a) Frame FC

(b) Frame FM

Figure 6.22 Close-up of observed damage after CHL-023 test

Figure 6.23 Close-up of observed damage after CHL-046 test

Figure 6.24 Crack patterns after CHL-023

Figure 6.25 Crack patterns after CHL-046

b) Hysteretic behaviour

Based on the measured acceleration and displacement responses at floor levels, the base shear vs. top displacement hysteresis relationships are obtained. Figure 6.26 shows the main hysteretic loops for the two test frames together with their backbone curves in both directions.

As can be seen, during the low-to-moderate earthquake tests (CHL-012 and CHL-023), both frames exhibited generally an elastic behaviour. The base shear of Frame FC during the CHL-023 test attained about 60% of the maximum base shear strength, whereas Frame FM reached almost the yielding state. As can be seen from Figure 6.24, only a few cracks occurred on the test frames. Frame FC appeared to sustain relatively lighter damage than Frame FM due to a lower seismic demand.

During the strong motion test with $PGA=0.46g$, both frames exhibited significant inelastic behavior. Because of a long-duration ground motion, the frames experienced a number of plastic reversals. As a result, remarkable strength degradation can be observed for both frames after the maximum base shear was reached. As shown in Figure 6.23, the first storey sustained much heavier damage than the second storey, exhibiting an apparent soft-storey mechanism. Major cracks

developed at member ends, while minor cracks were also observed in the joint regions.

Figure 6.26 Base shear vs. top displacement relationships

c) Stiffness degradation

The development of damage in the consecutive tests may be represented by the stiffness degradation with respect to the initial state. In this respect, a global degradation index may be defined as,

$$D_K = 1 - \frac{K}{K_0} \quad (6.4)$$

where K_0 and K are the initial and current lateral stiffness. Since the natural frequency of the frames is proportional to the square root of the lateral stiffness, the global degradation index can also be expressed in terms of fundamental frequency as

$$D_K = 1 - \left(\frac{f}{f_0} \right)^2 \quad (6.5)$$

where f_0 and f are the initial and current fundamental frequencies. Figure 6.27 shows the variation of the global (stiffness) degradation index with the increase of

the earthquake intensity during the tests. It can be seen that the global degradation rates in the two test frames are almost the same despite that the initial frequencies differed by about 10%. This indicates that the difference in the reinforcement detailing between the two test frames did not have much influence on the stiffness degradation. After the moderate earthquake test (CHL-023), the global degradation index is in an order of 0.3. This is consistent with the visual inspection that only minor cracking occurred at this stage of the tests. The damage index rapidly increased to 0.8 after the final test (CHL-046). Significant stiffness degradation can also be observed from the hysteretic loops shown in Figure 6.26.

Figure 6.27 Global stiffness degradation of test frames

Summarizing the observed response and damage, it may be concluded that the test frames were capable of resisting low-moderate earthquakes without major damage. During the moderate test of CHL-023, only minor damage was observed. In particular, Frame FC did not exhibit apparent undesired behaviour that would usually be expected due to its nonseismic detailing. As a matter of fact, Frame FC exhibited reduced displacement and increased stiffness at lower ground motion levels. Further discussion on the possible effect of nonseismic detailing on the initial stiffness, among other aspects, follows.

6.5 Further analyses and discussions on observed behaviour

6.5.1 Potential effect of detailing on initial lateral stiffness

As shown in Table 6.5, Frame FC possessed a larger initial natural frequency, or lateral stiffness, than Frame FM. This could affect the dynamic response of the frames to a varying extent depending on the frequency characteristics of the ground motions. Besides the normal variation of material properties, the longitudinal reinforcement details in the original non-seismically designed frame FC is also deemed to have a contribution to the increase of the stiffness in the particular frame.

As in typical non-seismic design, the longitudinal reinforcement in the columns of frame FC had lap splices in the bottom region where the bending effect is large under lateral loading. Referring to Figure 6.3, for an uncracked section, the additional steel area can increase the sectional rigidity by 10~30% for a reinforcement ratio in a range of 1~4%. Considering that in practical situations the concrete members could be precracked due to gravity loads, concrete shrinkage, and temperature effects, herein we take the effective flexural rigidity for the discussion.

Figure 6.28 Approximation of different rigidity distributions

To simplify the analysis, without losing generality, a column with fixed ends is considered herein as part of a moment-resisting frame. Two representative scenarios are considered, one with rebar lap splices at the bottom region of the column (representing frame FC), and another with lap splices at the mid-height region, representing frame FM. Figure 6.28 shows the two types of flexural rigidity distributions corresponding to the above two different scenarios, respectively, along with the moment distribution under a lateral force P applied at the top end of the column. As a reference, the lateral stiffness of the case with a uniformly distributed rigidity (EI) is calculated by

$$K_0 = \frac{12EI}{L^3} \quad (6.6)$$

In the first distribution (see Figure 6.28b), the lateral stiffness can be calculated through the horizontal deflection under an applied lateral force P . Using the moment area method,

$$\Delta_1 = \frac{P \left[(2L_1)^3 + L_s (4L_1^2 + 2L_1L + L^2) \right]}{12EI} + \frac{PL_s (4L_1^2 + 2L_1L + L^2)}{12EI_s} \quad (6.7)$$

where EI and EI_s are the flexural rigidities; L_s is the length of the EI_s region.

Let $\alpha_s = L_s / L$, $\beta_s = \alpha_s (4\alpha_s^2 - 6\alpha_s + 3)$, and $\gamma_s = EI_s / EI$, the lateral stiffness of the column can be obtained as,

$$K_1 = \frac{P}{\Delta_1} = \frac{12EI}{L^3 \left[1 + \left(\frac{1}{\gamma_s} - 1 \right) \beta_s \right]} \quad (6.8)$$

Similarly, the horizontal deflection for the second rigidity distribution (see Figure 6.28c) can also be obtained as

$$\Delta_2 = \frac{2PL_2(L_s^2 + L_sL + L^2)}{12EI} + \frac{PL_s^3}{12EI_s} \quad (6.9)$$

Thus,

$$K_2 = \frac{P}{\Delta_2} = \frac{12EI}{L^3 \left[1 + \left(\frac{1}{\gamma_s} - 1 \right) \alpha_s^3 \right]} \quad (6.10)$$

Figure 6.29 illustrates the lateral stiffness ratio, K_1/K_2 , for these two schemes with respect to α_s , which varies within 0~1.0, while γ_s is set in a reasonable range of 1~2. The stiffness ratio, K_1/K_2 , is always larger than unity.

Figure 6.29 Stiffness ratios between two different rigidity distributions

In the present case, α_s is 0.26 for both lap splice schemes, and γ_s is assumed to be 1.50 minor cracked sections. The resulting stiffness ratio K_1/K_2 is found to be 1.18 and subsequently the natural frequency ratio is 1.09 for a constant effective mass, proportional to the square root of the stiffness ratio. This value is in close agreement with the measured frequency ratio between Frame FC and FM. It is worth noting that the current columns are lightly reinforced. The stiffness difference could be even higher in case of increased amount of longitudinal reinforcement.

The above analysis leads to a general observation that a sensible increase in the lateral stiffness is likely to occur in non-seismically designed frames. As observed from the present experiment, this increase in stiffness will only start to diminish at advanced inelastic stages. As far as low to moderate seismicity is concerned, the increase in stiffness tends to become a pertinent factor that one needs to take into account in the evaluation of such type of non-seismic designs, especially in cases where the structure falls into a frequency (period) sensitive range.

Based on the present study, it is suggested that an increase of 10-40% in lateral stiffness be considered when lap splices are arranged in the column base regions. For ordinary columns with a reinforcement ratio of 1.5%~2%, an increase of 20% in lateral stiffness is appropriate. More precise calculation of the increase will need to take into account the actual reinforcement ratio and the damage (cracking) state of the affected regions. The calculation of the uncracked stiffness may be performed in a similar procedure as presented in this section, whereas the coefficient for cracked stiffness may be considered in accordance with the relevant code provisions regarding the stiffness of cracked RC members.

6.5.2 Lateral strength and ductility capacity

Figure 6.30(a) illustrates the base shear vs. top displacement envelopes of the test frames. These envelopes are obtained by averaging the backbone curve in both loading directions. It can be observed that the lateral strengths of the two frames are quite close. This indicates that for the deformation level at which the maximum base shear strength develops, the lap splice located at the column end regions does not adversely affect the development of the full lateral strength of the frame.

In the present analysis, a commonly-used definition, as shown in Figure 6.30(b), is employed to estimate the ductility capacity at the ultimate state. In this definition, the yield displacement is calculated in the way as shown in the figure, while the ultimate displacement is calculated at 20% strength degradation. Accordingly, the

global ductility capacity, defined as the ratio of the ultimate displacement to the yield displacement, is found to be 2.4 and 2.2 for Frame FC and FM, respectively. As can be expected, these overall ductility capacities are indeed lower than the desired levels for ductile RC structures. Apart from the non-ductile design, the cumulative damage from the large number of cycles during the long duration excitation also accelerated the degradation of the structure, leading to a reduced ductility. In a previous study (Lu 2002), the ductility capacity of a similar frame conforming to low ductility design but subjected to a fewer cycles of excitation (with El-Centro 1940 record) exhibited a ductility about 3.6.

Figure 6.30 Estimation of displacement ductility capacity

6.5.3 Performance criteria as compared with existing guidelines

The performance criterion for the types of frames under consideration is discussed herein in term of the interstorey drift, which is commonly adopted in existing seismic evaluation guidelines, such as FEMA 356 (BSSC 2000).

From the present experiment, Frame FC exhibited elastic behaviour under the moderate ground motion (CHL-023). The crack width at this stage is only about 0.2mm (see Figure 6.24) and the global stiffness degradation is about 25%. In the context of FEMA 356, the above state of damage in Frame FC can be categorized into Structural Performance Level S-1 (immediate occupancy). This performance

level refers to a state with very limited structural damage (e.g. minor hairline cracking, limited yielding in a few locations, no concrete crushing), while the vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness.

During the strong base excitation test of CHL-046, Frame FC experienced a peak interstorey drift ratio of 2.7%, which corresponds more or less to Structural Performance Level S-3 (life safety). However, the strength degradation at the maximum displacement, as shown in Figure 6.30, has well exceeded the commonly-accepted order of 20% strength degradation as an indication of reaching the ultimate state of the structure. The implication is that for a non-seismic designed frame, especially under long duration ground motions, the performance criterion in terms of inter-storey drift would have to be lowered significantly at apparent inelastic stages.

Compared with Frame FC, Frame FM experienced larger displacement response under CHL-023. As the maximum storey drift ratio (1.15%) has exceeded the limit value of 1% for the performance level of Immediate Occupancy, Frame FM may be categorized into the performance level S-2, namely the Damage Control level, which is defined to lie in-between the Immediate Occupancy and the Life Safety levels. Under CHL-046, however, both frames sustained similar damage that fell into the same Life Safety performance level.

6.6 Dynamic analysis

In this section, a non-linear time history analysis on the test frame is presented for the purpose to gain insights into appropriately modeling the low ductility frames for seismic response, particularly with regard to the selection of initial stiffness and hysteretic patterns. A verified structural model can be used to assist in the interpretation of observed response and damage as complementary to the measured responses.

6.6.1 Structural modeling

The general model is similar to the model used in the multi-storey frame analysis described in Chapter 4 and 5, using the computer program IDARC (Valles et al. 1996). The beam and column elements are modeled using beam-column elements with rigid joints. The inelastic behaviour induced by bending moment is simulated by the plastic hinge at the member end, and the shear deformation is assumed to be elastic for these moment-resisting frames. A 4% viscous damping ratio is considered, which is consistent with the measured value as shown in Table 6.5. The moment-curvature relationship is obtained through the fiber model analysis according to the actual properties of the test frames.

For the main purpose of simulating the inelastic response, the analysis is performed for the CHL-046 test. The initial flexural stiffness is adjusted so that the computed natural frequencies match the measured frequencies following the CHL-023 test (see Table 6.5). Table 6.6 summarizes the initial flexural stiffness (as a ratio to the uncracked gross sectional stiffness). The equivalent stiffness values recommended in typical building codes (ACI 2002; BSSC 2000; NZS 1995) are also listed in the same table for a comparison. It is noted that the actual values used in this particular analysis are generally lower than those according to the codes, and this is reasonable as the frames have already experienced certain damage in the first two tests.

Table 6.6 Equivalent initial flexural rigidity

Member	Equivalent flexural rigidity EI_{eq}/EI_g			
	Frame FM 1 st /2 nd storey	ACI 318-02	FEMA 356	NZS 1995
Beams	0.2/0.3	0.35	0.50	0.40 0.35(T-, L- beams)
Columns	0.2/0.5	0.70	0.70 0.50 ($n_0 \leq 0.3$)	0.80($n_0 > 0.5$) 0.60($n_0 = 0.2$) 0.40($n_0 = -0.05$)

Note: n_0 is the axial force ratio, and EI_g is the flexural rigidity of gross section.

Besides the initial stiffness, the hysteretic (inelastic) behaviour at the element end, especially the stiffness degradation and pinching, is important for a realistic simulation of the inelastic structural response. To highlight such an effect, two different hysteretic patterns, one as an ordinary case for ductile frames (designated as pattern 1) and another with a pronounced stiffness degradation (designated as pattern 2) are considered, as schematically illustrated in Figure 6.31. As observed during the strong base motion tests, major cracks formed at the column ends in a rather concentrated manner, implying significant stiffness degradation and pinching. Therefore, it is expected that pattern-2 hysteresis model will yield better results.

Figure 6.31 Hysteretic patterns considered at member ends

(left: $\alpha_s=5.0$, $\beta_e=0.1$, and $\gamma_p=1.0$ right: $\alpha_s=0.3$, $\beta_e=0.2$, and $\gamma_p=0.5$)

6.6.2 Comparison between measured and computed responses

Using the above structural model with two different hysteretic patterns, the nonlinear time history analysis is carried out on Frame FM for the CHL-046 (PGA=0.46g) ground motion. Figure 6.32 compares the measured and computed base shear and top displacement time histories using two different hysteretic models. As can be seen, the analysis using Pattern 1 tends to underestimate the responses, and this is particularly true for the top displacement for much of the response duration. Marked improvement is achieved with Pattern 2, and much better agreement between the computed and measured responses is achieved throughout

the entire duration of the responses. This confirms that in the present test frame, considerable stiffness degradation occurred during the strong motion responses. Figure 6.33 shows the base shear-top displacement hysteretic loops from the computed results.

(a) Hysteretic Pattern 1

(b) Hysteretic Pattern 2

Figure 6.32 Comparison between measured and predicted responses for frame FM

Figure 6.33 Computed base shear vs. top displacement relationships

Figure 6.34 Comparison among damage indices of different damage models

6.6.3 Assessment of cumulative damage in the test frames

Using the numerical model, a general assessment on the significance of cumulative damage in the total damage of the test frames under long duration earthquakes can be made. For this assessment, the structural damage in Frame FM is considered as an example. Figure 6.34 depicts the damage indices calculated using Park&Ang model with and without the cumulative energy term, respectively, during the CHL-046 test. An indication of the “actual” global damage status as evaluated on the basis of the measured natural frequencies (structural stiffness) using Eq. (6.5) is also shown in the figure. It can be seen that the damage index obtained from the P&A model with cumulative energy term agrees well with the “actual” damage at the end of the response. On the other hand, the calculated damage without considering the cumulative energy term considerably underestimates the actual damage. As can be seen from the figure, the cumulative damage accounts for almost 40% of the total damage in the present case.

The above observation confirms that cumulative damage plays an important part in the determination of the damage status, and this is particularly true in cases where ground motions with a long effective duration are anticipated, which is the case with low-moderate seismic zones subjected to long distance earthquakes.

6.7 Concluding remarks

A comprehensive shake table experiment has been performed on two companion frames subjected to a selected representative long duration ground motion. One test frame was designed with typical nonseismic reinforcing details (Frame FC) and another represents a limited modified version (Frame FM) in conformation to the Intermediate Requirements in ACI 318-02. The observed responses are presented and discussed. An associated nonlinear dynamic time history analysis was conducted to assist in the interpretation of the test results. Based on the experimental and analytical results, the following main conclusions may be drawn:

- (1) The nonseismically designed frame (Frame FC) is capable of resisting low-moderate earthquakes without major damage. Under a limited amplitude of cyclic deformation, the lap splices at undesired locations from strong seismic point of view does not seem to adversely affect the development of full lateral strength in low-to-medium rise frame structures where the axial force level is not very high.
- (2) Non-seismically designed frames with a regular configuration could possess an appreciable inherent ductility capacity. In the particular test frames, the global ductility capacity evaluated from the base shear-roof displacement relationships was in excess of 2.0 in both cases.
- (3) With the typical non-seismic detailing, the lap splice of column reinforcement, especially at column bottom region, tends to have an effect to increase the lateral stiffness. This could affect both the seismic demand as well as the response at low to moderate response levels. It is suggested that the potential effect of detailing on the lateral stiffness be considered in evaluation of seismic behaviour of non-seismic designs in the engineering practice for more accurate response predictions.
- (4) With a long duration ground excitation, the cumulative energy dissipation plays a more significant role in the process of structural degradation than in shorter duration excitation case. For the particular test cases under considerations, the contribution of cumulative damage accounted for more than 40% in the total damage during inelastic responses.

- (5) An appropriate nonlinear model for the analysis of non-seismic designed or low ductility frames needs to take into account the pronounced stiffness degradation and pinching. According to the nonlinear analysis, the strength degradation, stiffness degradation and pinching coefficients (α_s , β_e , and γ_p in the three-parameter hysteresis model) may be assigned to 0.3, 0.2 and 0.5, respectively. With the assistance of the numerical analysis, the damage in the test frames could be quantified using the damage indices. The damage prediction with the P&A damage model is found to agree well with the “actual” damage that is evaluated on the basis of the measured natural frequencies of the test frames, whereas a damage prediction without considering the cumulative energy tends to significantly underestimate the actual damage for structures under long-duration ground motions.

CHAPTER 7

FRP-REPAIRED FRAMES AND CHARACTERIZATION OF THEIR SEISMIC PERFORMANCE

In the preceding chapter, the seismic performance of two test frames designed with different reinforcement detailing schemes was investigated based on the shake table tests. It was shown that nonseismically designed frame structures according to the local design codes possess an inherent lateral load resistance and could resist low-to-moderate earthquakes without major damage. Under a strong ground excitation (CHL-046), it was observed that the damage was mainly concentrated at member ends along with slight cracks in the joints, showing a typical damage pattern of moment-resisting frames, but did not exhibit wide-spread material failure.

The above damaged frames then underwent a typical repairing procedure using FRP laminates. This chapter presents the test results obtained from the shake table tests of the repaired frames, and compares the seismic performance of the repaired frames with their original counterparts. The repairing and strengthening of structural components using FRP is a subject of extensive study in recent years; however, testing of FRP repaired structural systems under realistic seismic ground motions is scarce. Therefore, the present study is expected to provide useful insights on the effectiveness and potential drawbacks of the use of FRP for seismic repairing and retrofitting.

7.1 Introduction

Generally speaking, moderately damaged structures after an earthquake may still be repaired to enable continuing use. On the other hand, due to the enforcement of new design codes, many substandard structures need to be retrofitted and strengthened to upgrade their performances for different seismic hazard levels. For seismic upgrading and repairing of existing structures, different strategies may be considered depending on the structure type and the extent of damage. Typical strategies, as specified in FEMA 356 (BSSC 2000), include: 1) Local modification of components, 2) Removal or lessening of existing irregularities and discontinuities, 3) Global structural stiffening, 4) Global structural strengthening, 5) Mass reduction, 6) Seismic isolation, and 7) Supplemental energy dissipation. A suitable solution for a particular structure may be determined by investigating the potential structural deficiencies with respect to the seismic demands from representative earthquakes of interest. Apart from the technical considerations, the cost-benefits considerations imposed by the owners, as well as functional and architectural restrictions should also be taken into account in the development of a comprehensive repairing/upgrading strategy. As far as moment-resisting frames are concerned, retrofitting by local modification of components may be a suitable and economical solution.

Different techniques have been developed and widely used for repairing and strengthening structural components and connections. Generally, these include: (a) epoxy repair; (b) removal and replacement; (c) concrete jacketing; (d) concrete masonry unit jacketing; (e) steel jacketing and addition of external steel elements; and (f) strengthening with fiber-reinforced polymeric (FRP) composites. To repair a frame structure such as the current test frames, the FRP wrapping seems to be an obvious choice because of its advantages such as high strength-weight ratios, corrosion resistance, ease of application, low labor costs and no significant increase in member sizes. Other techniques may not be suitable. For example, epoxy repair approach may not be able to restore the strength of the damaged members or joints, while concrete and steel jacketing is difficult to implement in 3-dimensional cases

with floor members. A more detailed review and comparison among different techniques for beam-column joints can be found in Engindeniz et al. (2005).

Although the FRP wrapping technique has received much attention from researchers (e.g. Sheikh 2002; Antonopoulos and Triantafillou 2003; Balsamo et al. 2005), experimental data with regard to the performance of the retrofitted structures under realistic seismic loading are still limited. In the present study, the commonly used FRP wrapping technique was employed to repair the damaged frames for re-testing.

The main objectives of this study on FRP repaired frames include: (1) to investigate the effectiveness of a commonly adopted repairing scheme in restoring the structural properties in term of stiffness, strength and deformability for low to moderate seismic intensities; (2) to investigate the local damage pattern of the repaired frames at joints and member ends under simulated earthquake ground motions; and (3) to examine the potential impact of strengthening local regions on the inelastic response mechanisms and the final failure modes.

7.2 Repairing scheme and test programme

The primary interest here is to investigate a commonly used repairing scheme for moment-resisting frames to resist low-to-moderate seismic ground excitations. Thus, the uniaxial fiber texture was selected among the available fibers such as biaxial (0° - 90°) and quadriaxial (0° - 90° - $\pm 45^\circ$) fibers. The TYFO FIBRWRAP SYSTEM with uniaxial Glass Fabric Reinforced Polymer (GFRP), provided by a local supplier, was used for this repair study. The GFRP material is orientated in the 0° direction with additional yellow glass cross fibers at 90° direction. The properties of the selected GFRP are summarized in Table 7.1. Before the FRP wrapping was applied, some preparation work on the damaged frames was performed. The cracks were epoxy injected and the concrete surface at member ends was cleaned by grinding off all loose materials and paint. A layer of epoxy was applied to the

finished surface of the test frames. During the repair, the glass fabric was saturated with epoxy and then applied on the surface areas for repair.

Table 7.1 Composite gross laminate properties (SEH-51A)

Property	Typical Test value (ASTM D-3039)	Design value
Ultimate tensile strength in primary fiber direction	575 MPa	460 MPa
Elongation at break	2.2%	2.2%
Tensile modulus	26.1 GPa	20.9 GPa
Ultimate tensile strength 90 degrees to primary fiber	25.8 MPa	20.7 MPa
Laminate thickness	1.3 mm	1.3 mm

The arrangement of the glass fabric was made according to the damage patterns so that the fiber orientation is aligned along the direction of the principle stresses. Since the damage was mainly concentrated at the member end regions, the FRP wrapping was only applied to such damage regions. The procedure was completed in four steps, as demonstrated in Figure 7.1.

Step 1, installation of L-shaped FRP laminates

L-shaped FRP laminates with an extension length of 300mm (twice the column depth) were installed between members as shown in Figure 7.1(a) and Figure 7.2(a). They were used to improve the diffusion of stresses between members. At the column base, the L-shaped laminates were also adopted to enhance the resistance against large rotations at the column bottom ends

Step 2, installation of vertical laminates, as shown in Figure 7.1(b).

Step 3, installation of horizontal laminates

Overlaid laminates in orthogonal directions (vertical and horizontal) were employed to resist the shear stress in the joint. The vertical laminates were installed in the joint and extended for 300mm on one side of the column and terminated under the slab (Figure 7.1(b) and Figure 7.2(a)). The horizontal laminates were added on the

vertical laminates and also extended for the same length on both sides of the beam (Figure 7.1(c) and Figure 7.2(b)). The extended laminates along the beams and columns were used to resist flexural stress in the member ends and also to provide adequate anchorage for the laminates in the joint.

Step 4, installation of wrapping laminates at member ends

The extension regions mentioned above in both columns and beams were wrapped with FRP laminates, as shown in Figure 7.1(d) and Figure 7.2(b), to increase the lateral confinement of members and also enhance the bonding anchorage. Note that U-shaped wrapping laminates were applied to beams due to the restriction of slabs.

Figure 7.3 shows the repaired test frames mounted on the shake table. The instrumentation for displacements, accelerations and end rotations was identical to that of the original test frames. The test programme also followed that for the original frames (see Table 7.2) except that a failure test was added to investigate the final failure modes of the FRP repaired frames.

Figure 7.1 Repair scheme for test frames with GFRP composite

Figure 7.2 FRP wrapping at joint region

Figure 7.3 Repaired test frames mounted on shake table

Table 7.2 Test sequence for repaired test frames

Test number	Name	Description	Purpose
1	RND-AR	Random, PGA=0.02g	Initial structural identification
4	CHL-012R	Chile, PGA=0.12g, half intensity of original record	Minor shaking, elastic response
5	RND-BR	Random, PGA=0.02g	Structural identification
6	CHL-023R	Chile, PGA=0.23g, original record	moderate shaking, inelastic response
7	RND-CR	Random, PGA=0.02g	Structural identification
8	CHL-046R	Chile, PGA=0.46g double intensity of original record	Severe shaking, inelastic response
9	RND-DR	Random, PGA=0.02g	Structural identification
10	CHL-069R	Chile, PGA=0.69g triple intensity of original record	Failure test
11	RND-ER	Random, PGA=0.02g	Structural identification

7.3 Comparison between original and repaired frames

To investigate the effects of repairing/strengthening with FRP laminates on the structural performance, comparisons are made among the response quantities of the original and repaired frames with respect to stiffness, strength and deformability under the same earthquake ground motions. Meanwhile, the alteration of the final failure mode due to FRP wrapping is also evaluated with the failure test (CHL-069R).

7.3.1 Initial stiffness restoration and progressive damage during tests

Similar to the procedure presented in Chapter 6, random tests and dynamic identification analyses were performed on the repaired test frames after each earthquake test. Table 7.3 summarizes the dynamic characteristics including the

natural frequencies, equivalent damping ratios and mode shapes. For the present beam-column frames, the fundamental frequency is deemed a good indicator of the physical state of the structures. Figure 7.4 compares the fundamental frequencies of the original and repaired test frames at different test stages. It can be seen that the repaired frames generally have slightly larger fundamental frequencies than the original frames at comparable stages. In particular, the initial frequencies of the repaired frames are about 5% higher than those of the original frames. This indicates that the repairing scheme incorporating crack epoxy-injection and FRP wrapping is capable of restoring the stiffness of the moderately damaged frames from the previous earthquakes. It has been reported previously (Balsamo et al. 2005) that in case of severe damage, the recovered initial stiffness could be slightly lower than the initial stiffness of the undamaged frame. Obviously, the stiffness restoration will depend on the severity of the cumulative damage before repairing and also the repairing scheme.

Table 7.3 Summary of dynamic characteristics of repaired frames

Random test	FC-R				FM-R			
	Frequency	Damping	Modal shapes		Frequency	Damping	Modal shapes	
RND_A	4.82	2.22	1.00	-0.62	4.26	2.24	1.00	-0.63
	15.07	2.20	0.62	1.00	13.22	2.04	0.56	1.00
Chile012g								
RND_B	4.78	2.88	1.00	-0.61	4.11	2.68	1.00	-0.61
	15.01	2.17	0.63	1.00	12.85	2.34	0.60	1.00
Chile023g								
RND_C	4.60	3.49	1.00	-0.62	3.68	4.39	1.00	-0.58
	14.53	2.40	0.63	1.00	11.76	3.58	0.64	1.00
Chile046g								
RND_D	3.21	6.08	1.00	-0.73	2.91	6.37	1.00	0.68
	11.59	3.75	0.77	1.00	9.12	4.50	0.71	1.00
Chile069g								
RND_E	1.54	6.82	1.00	-0.83	1.24	7.11	1.00	-0.78
	10.45	4.03	0.87	1.00	8.82	4.92	0.84	1.00

Figure 7.4 Fundamental frequencies of test frames (original and repaired)

Figure 7.5 Global degradation indices of original and repaired frames

The development of damage in the consecutive tests may be represented by the stiffness degradation with respect to the initial state, i.e., the global degradation index, D_K , which can be calculated in terms of the change of the fundamental frequency as introduced in Chapter 6. Figure 7.5 compares the global degradation indices of all test frames obtained after each earthquake test. In general, the degradation indices of all the test frames increased as the damage cumulated during the tests. It is interesting to note that the repaired frames exhibit a lower degradation

rate than the original frames at comparable test stages up to CHL-046R (PGA=0.46g). For the CHL-046R test at which significant damage occurred, the degradation indices of the repaired frames are about 30% lower than those of the original frames. As presented in Chapter 6, the original frames sustained intensive cracks around the beam-column joints and the column base regions. In comparison, the damage in the repaired frame appeared to be lighter. Further discussion on the damage status follows.

Figure 7.6 depicts the damage distribution of the repaired frames after the test with PGA=0.46g (CHL-046R). The corresponding local damage patterns are shown in Figure 7.7. As compared with the intensive cracking in the original frames, the repaired frames exhibited almost no visible concrete cracking along the members, except that a few light cracks appeared at locations close to the edge of the wrapped regions in the first storey beams. The primary damage in the repaired frames appeared to associate with the debonding of FRP laminates at the connection between the column bottom ends and the base foundation. The first visible sign of such debonding was found in Frame FM-R during the moderate ground motion (CHL-023R) as shown in Figure 7.7(d). In the subsequent CHL-046R test, apparent debonding, as shown in Figure 7.7(c)-(d), developed at the column base ends of both repaired frames. No visible debonding was observed around the FRP wrapped beam-column joints except that minor epoxy cracks appeared at the corners between beams and columns, as shown Figure 7.7(a)-(b). Therefore, the current FRP repairing scheme can be regarded as effective in restoring the stiffness and strength capacities of the frames and reducing the structural damage.

Figure 7.6 Damage patterns of repaired frames after CHL-046R

Figure 7.7 Close-up of observed damage of repaired frames after CHL-046R

From Figure 7.5, the stiffness degradation indices of both original frames at each test stage are almost equal; however, in the repaired frames, Frame FM-R exhibits much larger stiffness degradation than Frame FC-R under low to moderate ground shaking (CHL-023R). This may be explained by the fact that in the repaired frames, the overall stiffness is largely dependent on the integrity of the FRP repaired regions, and it can be particularly sensitive to the column-base connections. Although no visible debonding was observed at this stage in either frame, the markedly larger stiffness reduction in frame FM-R after the CHL-023R test seems to indicate that appreciable debonding has actually initiated in the frame.

The stiffness degradation in both repaired frames became significant and was almost equal after test CHL-046R. Inspection of the damage (Figure 7.7c-d) revealed the debonding at the column-base connections occurred in a quite similar manner in the two frames. This further demonstrated that in the FRP repaired frames, the debonding at the column-base connections was a governing factor influencing the (stiffness) degradation of the repaired frames.

7.3.2 Displacement and strength

Table 7.4 summarizes the peak responses of the two repaired test frames. The response time histories can be found in Appendix B. In particular, Figure 7.8 compares the top displacements and the concentration factors among all the original and repaired test frames, where the concentration factor is defined as a ratio of the critical storey (first storey herein) drift ratio over the top displacement ratio. From Figure 7.8(a), it can be seen that the top displacements of the repaired frames are nearly the same as those of the corresponding original frames, especially for the tests with 0.46g (CHL-046/R) where all test frames exhibited marked inelastic behaviour. The concentration factors in Figure 7.8(b) reflect the damage concentration in the first storey and the response mode. The concentration factors of all test frames are almost the same at all comparable stages (CHL-012/R-CHL-046/R) except that 20% higher concentration factors are obtained for the original frames in the advanced inelastic stage (CHL-046/R). Referring to the damage

patterns presented in Chapter 6 in Figure 7.7, it is found that in the advanced inelastic stage the original frames were dominated by the soft storey mechanism, whereas the repaired frames maintained a global response mode and consequently achieved a lower damage concentration in the first storey.

In the repaired frames, the beam-column joint regions were wrapped by two overlaid plies of fibers. This effectively deterred the formation of plastic hinges at the upper ends of the first storey columns as in the original frames, and hence delayed the storey mechanism to occur. But with further increase of the ground motion intensity, a modified soft storey mechanism eventually occurred in the repaired frames and this will be discussed later.

Table 7.4 Peak responses of repaired frames during earthquake tests

Earthquake test	FC-R			FM-R		
	Peak top displ. (%H)	Peak inter-storey drift ratio (%)	Peak storey shear (kN)	Peak top displ. (%H)	Peak inter-storey drift ratio (%)	Peak storey shear (kN)
CHL -012R	0.10	0.15	3.96	0.21	0.26	5.67
CHL -023R	0.22	0.30	7.23	0.77	1.02	12.53
CHL -046R	1.66	2.30	18.27	1.84	2.21	18.72
CHL -069R	3.35	5.95	16.35	3.95	5.94	17.97

Figure 7.8 Maximum displacements of test frames

Figure 7.9 plots the combined major hysteresis loops of base shear vs. top displacement from all the tests for the repaired Frame FC-R and FM-R, respectively. The envelopes of the hysteresis loops are also plotted in thick lines in the same figures. Averaging the envelope values in both directions, the envelope base shear vs. top displacement relationship of the test frames can be obtained and they are plotted in Figure 7.10 together with those of the original frames. It is observed that the maximum base shear forces of the repaired frames are slightly lower than those of the original frames. This indicates that the current repairing scheme generally restored the base shear strength of the frames, however no further strength enhancement was gained. This apparently was attributable to the debonding at the column base ends. Due to the debonding as shown in Figure 7.7(c)-(d), the strength of fiber could not fully develop during the severe ground motion, which finally affected the potential strength enhancement of the test frames. However, the debonding was a progressive process as the local deformation increased during the earthquakes, resulting in a gradual strength degradation and consequently an improved overall ductility, as can be observed from Figure 7.10

Figure 7.9 Base shear vs. top displacement relationships

Figure 7.10 Base shear vs. top displacement envelopes of test frames

Similar to the procedure introduced in Chapter 6, the overall displacement ductility factors of the repaired Frame FC-R and FM-R are found to be 3.0 and 2.9 as compared to 2.4 and 2.2 of the original frames, respectively. The above observations are generally consistent with a previous study on a frame-wall structure (Balsamo et al. 2005). At a heavily damaged state, the repaired frame-wall structure had slightly larger top displacement and almost the same base shear strength as compared with the original structure.

7.3.3 Final failure modes of repaired frames

After the main tests up to 0.46g were completed, the failure test with PGA=0.69g (CHL-069R) was conducted to expose the final failure mode. Figure 7.11 depicts the failure patterns of both repaired frames after the final test. The corresponding local damage details are shown with the photos in Figure 7.12.

Figure 7.11 Damage patterns of repaired frames after CHL-069R

(a) Column failure (FC-R)

(b) Column failure (FM-R)

(c) Debonding at the base interface (FC-R)

(d) Debonding at the base interface (FM-R)

Figure 7.12 Close-up of observed damage of repaired frames after CHL-069R

The damage patterns of both repaired frames were quite similar and they can be characterized by major cracks across the upper wrapped edge of the columns and plastic hinges associated with debonding at the column base ends. The FRP wrapped joint regions with two overlaid plies of fibers appeared to have effectively strengthened the flexural/shear strength of the joint regions, such that the whole wrapped region around a joint appeared to behave like an expanded rigid zone. Consequently, the column section immediately below the wrapped zone became a critical location along the column. As a result, plastic hinges occurred at this location when the base motion intensity increased to $\text{PGA}=0.69\text{g}$.

Figure 7.13 gives a schematic comparison of the final failure modes between the original and repaired frames. Despite a similar soft storey mechanism at failure, the expanded rigid zones around beam-column joints of the repaired frames tend to result in a shorter clear height of the first-storey columns. Given a certain rotational capacity at the section level, this reduction of the clear column height would potentially result in a reduction of the inter-storey displacement capacity, although such an effect was not evident in the present tests, which is likely due to the large slenderness of the columns.

Figure 7.13 Schematic of final failure modes of test frames

Additional cracks also developed in the column segment below the wrapped edge in the repaired Frame FM-R, as shown in Figure 7.11(b). Besides the increased moment and shear effects in the middle portion of the columns due to the expanded rigid zones, as shown schematically in the moment diagrams in Figure 7.14, this was also attributable to the fact that a weak link was present at about the cracked section as a result of termination of the longitudinal reinforcing bars at the lap splices, as indicated in Figure 7.14. Frame FC-R does not have such a weak link at this location (see Chapter 6) and therefore no additional cracks appear there.

The above observations bring up a significant aspect that one should take into account in the design and evaluation of FRP repaired frames. The creation of some essentially rigid zones could alter the configuration of the moment and shear force distributions, and hence shift the critical regions to locations which were not regarded and designed as critical regions. In the present frames, due to the strengthening effect in the extended joint regions, the lap splice in the middle portion of the column could potentially be detrimental to the structural safety under severe ground motions although it is not a problem for the original frame (FM).

Figure 7.14 Schematic of moment diagrams of first storey column in original and repaired frames

7.3.4 Debonding at column base ends

Debonding is a primary factor that can cause large stiffness degradation and eventually the neutralization of the structural repairing effect. As shown in Figure 7.7 debonding in the present frames was mainly concentrated at the column base ends and no apparent debonding was observed at the wrapped joint regions. In fact, because of the difficulty in applying the FRP in a more effective way than surface bonding, the debonding of FRP laminates at the interface between a column or wall bottom end and the top surface of the foundation (same problem could arise at 3-D joints) is generally a critical issue in the application of FRP laminates for seismic repair and strengthening. The present test results demonstrated clearly such a drawback. The solution will lie on the development of more effective and robust bonding-integration technique at such interfaces, for instance using appropriate bolts or anchors, to allow sustained resistance against large deformation reversals..

7.4 Concluding remarks

In this chapter, the repair of the damaged frames with FRP laminates is presented and the seismic performance of the repaired frames during the shake table tests was discussed in comparison with the original frames. Based on the test results, the following conclusions may be drawn:

- (1) The application of FRP laminates proved to be an effective and efficient approach for repairing RC frame structures. The frames repaired with the commonly-used scheme (uniaxial and multi-ply) in conjunction with the crack epoxy injection can restore the initial stiffness for moderately damaged frame structures.
- (2) For comparable ground motions, the top displacements of all test frames appeared to be equal, while the inter-storey drifts of the repaired frames became more uniformly distributed than the original frames. The base shear strengths of the repaired frames were about the same as the original frames. The FRP repairing slowed down the strength degradation; as a result, the ductility capacity was increased from about 2.0 to about 3.0.

- (3) Due to effective wrapping of FRP around all the beam-column connections and in the lower portion of the first storey columns, the failure mechanism of the repaired frames exhibited distinctive features as compared with the original frames. The plastic hinges in the upper part of the first storey columns shifted down from the beam-column joint to the level immediately below the wrapped region, rendering high flexural and shear demand in the middle portion of the columns. Although no marked adverse effect was observed in the present test frames (due likely to slender columns), in a more general sense one would need to take into account the complications thus induced on the behaviour of the columns in the design and evaluation of FRP repaired frames.
- (4) Debonding is crucial issue affecting the performance of FRP repaired frames at advanced inelastic stage with large deformation reversals. In the present planner frames, debonding did not occur at the joint regions thanks to the planner configuration which allowed a rather effective wrapping around the joint regions. However, at the interface between column bottom ends and the top surface of the base foundation, where only surface bonding was possible, pronounced debonding occurred under strong ground motions, which eventually neutralized the repair effect on the overall response of the frames. To resolve such problems, more effective FRP installation technique at interfaces where wrapping is not possible needs to be developed to allow for a sustained integrity of the repaired connections at such interfaces.

CHAPTER 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This thesis presents the research work focusing on damage based methodologies for performance-based seismic design and evaluation. Both analytical and experimental studies are conducted. The thesis covers five main aspects. The first one concerns the damage-based structural design and the generation of damage-based inelastic spectra. The second one deals with the correlation between the damage spectra of SDOF systems and the actual damage in multi-storey frame structures, and an implementation procedure to apply the damage spectra for the estimation of damage and its distribution in a multi-storey RC frame. This is followed by the prediction of inelastic deformation, particularly inter-storey drift, in multi-storey frames with a storey capacity factor, taking into account the varying influences of strength and stiffness at different ductility levels. The next one addresses the issue concerning a realistic assessment of the potential seismic performance of structures with nonseismic details, and this involves a comparative experimental study using shake table tests. Finally, as supplementary to the experimental programme, the effectiveness of the commonly-used FRP repairing technique for damaged RC frames in restoring the structural mechanical properties is investigated with shake table tests.

The originality of this study encompasses the development of a unified methodology covering the generation of inelastic demand spectra, which specify the overall seismic demand for a given level of damage, to the application in the

damage-based design and evaluation in actual multi-storey frame systems. The physical influences of the storey strength and stiffness on the inelastic response distribution (in particular the inter-storey drift) are explicitly expressed in the proposed storey capacity factor evaluation with consideration their varying significance with increase of inelastic response (ductility) levels. The emphasis on assessing the likely seismic performance for non-seismically designed structures with respect to realistic (low-to-moderate) seismic demands brings new insights on the potential problems that might be caused by undesired detailing in such structures.

Detailed conclusions on each aspect covered in this study have been presented in the individual chapters of the thesis. The following summarizes the main conclusions and observations.

A. Damage-Based Inelastic Spectra

With the development of performance-based design, the structural damage, especially the cumulative damage, has become a subject of increasing attention in recent years. As far as structural performance is concerned, the construction of damage-based strength reduction factor (R_D) is deemed to be an effective way for evaluating the overall seismic demand for specific levels of structural damage.

With the proposed procedure, a series of R_D spectra are generated for a large set of ground motions with different groupings to examine the effects of site conditions, earthquake magnitudes, source-to-site distances, as well as hysteretic patterns and design ductility levels. As far as firm sites (S_{AB} , S_C and S_D) are concerned, it is observed that the local site condition does not appear to have significant effect on the mean R_D spectra. The distances and structural hysteretic patterns also have negligible effects. The influence of earthquake magnitudes is relatively more significant; higher magnitude earthquakes tend to give rise to lower R_D factors for

the same damage level.

A unified overall mean R_D spectrum is proposed for all firm sites, with a modification to take into account the influence of earthquake magnitude. By means of regression analysis, empirical formulas are constructed for the mean R_D spectra as a function of the natural period of the system, the damage level and the ductility class (capacity) for moderate, low and high ductility designs. The empirical formulas provide a convenient means for the construction and application of the damage-based spectra in actual design and performance evaluation.

B. Seismic Damage Evaluation of Multi-Storey Frame Structures

In a performance-based design era, a damage measure may be used as a performance criterion as well as a basis for decision-making with regard to repairing and retrofitting. For this purpose, a methodology is developed to estimate the seismic damage of multi-storey RC frames in terms of both the overall damage and the damage distribution. The overall damage of an actual frame can be deduced from the damage of its equivalent SDOF system, which can be conveniently obtained from the above damage-based R_D spectra in conjunction with an elastic response spectrum. The distribution of damage in the frame can be established taking into account the strength and stiffness characteristics of the structure.

To implement the proposed methodology, two alternative methods have been investigated through nonlinear static and seismic time-history analyses, one based on the modal pushover analysis, and the other based on the characterization using the storey capacity factor. With a model pushover analysis considering the first two modes, the first method can provide a reasonable prediction of the damage distribution. On the other hand, the storey capacity factor is also found to correlate well with the damage distribution in regular as well as irregular frames. This

provides a convenient means for a rapid assessment of damage distribution along the frame height in a design environment.

C. Prediction of Inter-Storey Drift with Storey Capacity Factors

The performance of a building depends not only on the damage state of structural members, but also on the damage of non-structural components and building contents. To a large extent, the non-structural damage can be associated with the maximum inter-storey drift. The inter-storey drift distributions are dependent upon the storey overstrength and stiffness, and their relative effects can vary with the inelastic response (ductility) levels. A sound procedure for the prediction of the inter-storey drifts at different response levels need to take all the above factors into account.

The proposed method stems from a modified storey capacity factor as a product of an overstrength term that increases with the response ductility and a stiffness term which decreases with the ductility in an exponential form. Results from the nonlinear time history analysis on representative frames demonstrate that the modified storey capacity factor has good correlation with the inter-storey drifts in a consistent manner at different ductility levels.

Further refinement of the approach to predict the inter-storey drift concentration factor as a function of the structural regularity index has also been carried out. Statistical results from the non-linear time history analyses show that the inter-storey drift concentration factor is not sensitive to the level of inelastic response. Based on the enlarged dataset, an improved empirical formula is proposed for the prediction of the critical inter-storey drift.

D. Comparative Shake Table Tests on Nonseismically Designed Frame Models

A comparative shake table experiment has been conducted on two companion frames of different detailing arrangements with a selected representative long duration ground motion of increasing intensities.

It is found that the nonseismically detailed frame (Frame FC) is generally capable of resisting low-to-moderate earthquakes without major damage, and under a limited amplitude of cyclic deformation the nonseismic details, such as lap splices at undesired locations, do not appear to induce adverse effects on the lateral strength. The global ductility capacity evaluated from the base shear-roof displacement relationships was in excess of 2.0 in both cases. The results suggest that non-seismically designed frames with a regular configuration could possess an appreciable inherent ductility capacity.

It is interesting to find out that, in the typical non-seismic detailing with the lap splice of column reinforcement at column bottom region, the frame tends to have an appreciate increase in its lateral stiffness. This could affect both the seismic demand as well as the response at low to moderate response levels, and therefore should be considered in the evaluation of seismic behaviour of non-seismic designs for more accurate response predictions.

Non-seismic designed or low ductility frames tend to have pronounced stiffness degradation and pinching; therefore, an appropriate determination of the model parameters pertaining to these effects is essential in the nonlinear analysis of this category of structure. According to the present study, the strength degradation, stiffness degradation and pinching coefficients (α_s , β_e , and γ_p in the three-parameter hysteresis model) may be assigned to 0.3, 0.2 and 0.5, respectively.

The cumulative energy is found to contribute in the actual damage of non-seismically designed frames under long-duration ground motions. For the particular

test cases under considerations, the contribution of cumulative damage accounted for more than 40% in the total damage during inelastic responses.

E. Effectiveness of commonly used FRP- Repairing technique for seismic repairing of RC Frames

Generally speaking, the current FRP repairing technique is found to be effective in repairing moderately damaged frames in terms of restoration of the initial stiffness and lateral strength. The adopted repairing scheme also effects to delay the strength degradation and consequently increase the ductility capacity (2.0 to about 3.0 in the particular cases studies).

The failure mechanism of repaired frames can be significantly modified due to strengthening of extended joint regions. The plastic hinges could shift away from joint interfaces, thus reducing the net length of the column and beam members. In general, this could induce some adverse effect on the deformability of beam-column members and hence reduce to a certain extent the overall ductility of the structure.

Debonding is crucial issue affecting the performance of FRP repaired frames at advanced inelastic stage with large deformation reversals. In the case studied, pronounced debonding occurred at the interface between column bottom ends and the top surface of the base foundation, which eventually neutralized the repair effect on the overall response of the frames. To resolve such problems, more effective FRP installation technique at interfaces where wrapping is not possible needs to be developed to allow for a sustained integrity of the repaired connections at such interfaces.

8.2 Recommendations on Future Research

Some important aspects are worthy further study in order to improve the understanding of the seismic performance of building structures in terms of damage, and advance towards a systematic implementation of the damage-based seismic design and evaluation in real practice.

The R_D spectra for soft soil sites are expected to have some different characteristics from those for the firm sites, due partly to the wider variation of the predominant period of the ground motions. The R_D spectra for soft soil sites should be constructed when enough data about the pertinent ground motions become available.

The proposed procedure for damage estimation is expected to be generally applicable in seismic performance evaluation concerning structural damage. However, the correlation between SDOF damage and the actual damage in multi-storey frames may vary for ground motions with markedly different frequency characteristics, such as near-field ground motions, and for structures with different complexity. In this regard, further parametric investigations need to be carried out to expose such differences and refine the empirical expressions accordingly.

To improve the understanding of seismic performance of nonseismically designed structures, further experimental study needs to be conducted on frame models with more realistic configuration, for example to include more storeys and multiple spans. This would enhance the understanding of the global system behaviour, the behaviour of interior joints and the effect of redundancy on the seismic performance. Tests of structural models with representative nonstructural elements such as masonry walls and windows will be helpful for the development of more comprehensive performance and damage criteria.

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APPENDIX A

SELECTED EARTHQUAKE GROUND MOTIONS

Appendix A-1 Earthquake Ground Motions Selected on NEHRP Site A+B

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Anza (Horse Canyon) 1980/02/25 10:47	4.7	5160 Anza Fire Station	12.1	225	0.07	3.30	0.36	5.9
				315	0.07	2.60	0.16	6.8
Anza (Horse Canyon) 1980/02/25 10:47	4.7	5044 Anza - Pinyon Flat	12.0	45	0.11	2.50	0.11	2.1
				135	0.13	5.10	0.49	1.8
Anza (Horse Canyon) 1980/02/25 10:47	4.7	5045 Anza - Terwilliger Valley	5.8	45	0.13	3.90	0.17	1.4
				135	0.08	1.70	0.06	2.4
Chi-Chi, Taiwan 1999/09/20	7.6	HWA056	44.5	NS	0.11	10.80	10.36	16.3
				WE	0.11	11.70	17.64	16.9
Chi-Chi, Taiwan 1999/09/20	7.6	TCU046	14.3	NS	0.12	30.90	23.18	18.2
				WE	0.13	39.80	37.37	18.5
Chi-Chi, Taiwan 1999/09/20	7.6	HWA026	55.3	NS	0.06	9.10	9.74	18.4
				WE	0.07	11.20	18.17	19.7
Chi-Chi, Taiwan 1999/09/20	7.6	ILA031	118.1	NS	0.08	9.10	10.68	20.2
				WE	0.06	10.00	9.94	19.9
Chi-Chi, Taiwan 1999/09/20	7.6	ILA050	75.1	NS	0.06	9.90	16.41	22.1
				WE	0.07	7.30	6.69	24.4
Chi-Chi, Taiwan 1999/09/20	7.6	ILA051	88.5	WE	0.08	12.30	9.66	29.7
				NS	0.03	7.30	9.19	15.2
Chi-Chi, Taiwan 1999/09/20	7.6	TAP035	96.7	NS	0.09	8.30	8.00	18.5
				WE	0.07	8.40	12.78	22.5
Chi-Chi, Taiwan 1999/09/20	7.6	NSK	64.0	EW	0.07	6.90	4.23	18.4
				NS	0.07	5.10	1.16	21.7
Chi-Chi, Taiwan 1999/09/20	7.6	TAP051	105.1	WE	0.11	16.60	21.52	25.0
				NS	0.06	14.90	13.39	26.0
Chi-Chi, Taiwan 1999/09/20	7.6	TAP103	125.1	NS	0.18	21.70	8.93	15.6
				WE	0.12	22.70	8.32	10.9
Chi-Chi, Taiwan 1999/09/20	7.6	TCU025	54.4	NS	0.06	10.50	10.17	15.2
				WE	0.08	19.00	22.00	17.9
Chi-Chi, Taiwan 1999/09/20	7.6	TCU085	64.0	NS	0.05	6.40	7.38	22.7
				WE	0.06	7.50	13.88	20.0

Appendix A-1 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Coyote Lake 1979/08/06 17:05	5.6	47379 Gilroy Array #1	9.1	230	0.10	3.40	0.48	7.3
				320	0.13	8.30	1.52	5.8
Hollister 1974/11/28 23:01	4.5	47379 Gilroy Array #1	10.0	157	0.11	2.70	0.13	2.7
				247	0.13	4.00	0.17	2.5
Kocaeli, Turkey 1999/08/17	7.8	Gebze	17.0	0	0.24	50.30	42.74	7.4
				270	0.14	29.70	27.54	7.7
Kocaeli, Turkey 1999/08/17	7.8	Izmit	4.8	180	0.15	22.60	9.81	13.2
				90	0.22	29.80	17.12	15.0
Landers 1992/06/28 11:58	7.5	21081 Amboy	68.3	0	0.12	18.30	11.16	29.8
				90	0.15	20.00	7.38	25.2
Landers 1992/06/28 11:58	7.5	12206 Silent Valley - Poppet Flat	51.3	0	0.05	3.80	2.03	30.0
				90	0.04	5.10	3.88	31.7
Landers 1992/06/28 11:58	7.5	22161 Twentynine Palms	41.9	0	0.08	3.70	2.34	30.7
				90	0.06	4.90	4.30	30.9
Loma Prieta 1989/10/18 00:05	7.1	47379 Gilroy Array #1	10.5	0	0.41	31.60	6.38	6.5
				90	0.47	33.90	8.03	3.7
Loma Prieta 1989/10/18 00:05	7.1	1032 Hollister - SAGO Vault	29.9	270	0.04	7.10	4.55	16.3
				360	0.06	8.40	4.89	15.4
Loma Prieta 1989/10/18 00:05	7.1	47377 Monterey City Hall	42.7	0	0.07	3.50	1.41	13.2
				90	0.06	5.80	2.89	11.9
Loma Prieta 1989/10/18 00:05	7.1	58338 Piedmont Jr High	77.2	45	0.08	8.20	2.94	10.6
				315	0.07	9.10	3.35	11.1
Loma Prieta 1989/10/18 00:05	7.1	58043 Point Bonita	88.1	207	0.07	11.40	3.98	10.1
				297	0.07	12.90	3.93	8.4
Loma Prieta 1989/10/18	7.1	58131 SF - Pacific Heights	80.5	270	0.06	12.80	3.45	9.7
				360	0.05	9.20	2.92	10.8
Loma Prieta 1989/10/18 00:05	7.1	58151 SF - Rincon Hill	78.5	0	0.08	6.70	2.58	13.7
				90	0.09	10.40	3.91	11.4
Loma Prieta 1989/10/18 00:05	7.1	58539 So. San Francisco, Sierra Pt.	67.6	115	0.06	7.10	5.18	12.2
				205	0.11	8.80	4.59	9.5
Loma Prieta 1989/10/18 00:05	7.1	58163 Yerba Buena Island	79.5	0	0.03	4.20	1.45	14.5
				90	0.07	13.40	3.26	8.0

Appendix A-1 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Morgan Hill 1984/04/24 21:15	6.1	47379 Gilroy Array #1	16.0	230	0.07	2.90	1.26	9.6
				320	0.10	2.90	1.02	8.9
Northridge 1994/01/17 12:31	6.7	127 Lake Hughes #9	28.9	0	0.17	8.40	4.54	9.3
				90	0.22	10.10	2.77	7.9
Northridge 1994/01/17 12:31	6.7	90019 San Gabriel - E. Grand Ave.	39.5	180	0.14	9.60	2.21	14.5
				270	0.26	9.80	2.79	13.1
Northridge 1994/01/17 12:31	6.7	23595 Littlerock - Brainard Can	49.7	90	0.07	6.00	1.35	11.9
				180	0.06	6.30	1.25	17.7
Northridge 1994/01/17 12:31	6.7	23572 Mt Baldy - Elementary Sch	74.0	90	0.08	3.80	0.56	14.0
				180	0.07	4.30	0.39	12.9
Northridge 1994/01/17 12:31	6.7	24399 Mt Wilson - CIT Seis Sta	37.8	0	0.23	7.40	0.70	8.8
				90	0.13	5.80	0.45	10.8
Northridge 1994/01/17 12:31	6.7	23590 Wrightwood - Jackson Flat	67.7	90	0.06	10.00	2.92	11.8
				180	0.04	7.00	2.84	18.5
Northridge 1994/01/17 12:31	6.7	24310 Antelope Buttes	48.4	0	0.05	3.60	2.26	15.1
				90	0.07	4.30	2.23	13.9
N. Palm Springs 1986/07/08 09:20	6	13198 Murrieta Hot Springs	61.0	0	0.05	1.80	0.30	9.6
				90	0.05	1.30	0.32	7.0
N. Palm Springs 1986/07/08 09:20	6	12206 Silent Valley - Poppet F	23.7	0	0.14	3.90	0.55	6.9
				90	0.11	4.00	0.80	7.0
N. Palm Springs 1986/07/08 09:20	6	13199 Winchester Bergman Ran	55.3	0	0.07	1.90	0.19	9.4
				90	0.09	1.80	0.29	8.7
San Fernando 1971/02/09 14:00	6.6	127 Lake Hughes #9	20.2	21	0.16	4.50	1.28	9.4
				291	0.13	3.90	1.12	11.8
San Francisco 1957/03/22 14:00	5.3	1117 Golden Gate Park	8.0	10	0.10	3.90	0.19	4.1
				100	0.11	4.60	0.43	3.7
Whittier Narrows 1987/10/01 14:42	6.1	24399 Mt Wilson - CIT Seis Sta	22.1	0	0.12	3.30	0.37	10.1
				90	0.19	4.60	0.21	8.3

Appendix A-2 Earthquake Ground Motions Selected on NEHRP Site C

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Anza (Horse Canyon) 1980/02/25	4.7	5047 Rancho De Anza	20.6	45	0.10	6.70	0.55	5.2
				135	0.09	6.00	0.45	5.5
Anza (Horse Canyon) 1980/02/25	4.7	5049 Borrego Air Ranch	41.4	225	0.05	2.60	0.28	3.3
				315	0.04	1.20	0.10	3.6
Cape Mendocino 1992/04/25	7.1	89324 Rio Dell Overpass - FF	12.3	270	0.39	43.90	22.03	15.3
				360	0.55	42.10	18.62	10.9
Cape Mendocino 1992/04/25	7.1	89486 Fortuna - Fortuna Blvd	13.7	0	0.12	30.00	27.59	18.7
				90	0.11	21.70	12.79	18.2
Cape Mendocino 1992/04/25	7.1	89530 Shelter Cove Airport	32.6	0	0.23	7.10	0.39	16.1
				90	0.19	6.60	0.57	4.4
Cape Mendocino 1992/04/25	7.1	89509 Eureka - Myrtle & West	35.8	0	0.15	20.20	5.89	20.8
				90	0.18	28.30	11.41	19.8
Chi-Chi, Taiwan 1999/09/20	7.6	TCU136	9.0	EW	0.17	55.80	66.48	19.8
				NS	0.18	47.50	44.82	23.9
Chi-Chi, Taiwan 1999/09/20	7.6	TCU104	13.6	NS	0.09	47.20	52.70	30.5
				WE	0.11	36.60	51.97	28.2
Chi-Chi, Taiwan 1999/09/20	7.6	TCU048	14.4	NS	0.18	48.30	53.55	34.1
				WE	0.12	32.60	52.18	27.4
Chi-Chi, Taiwan 1999/09/20	7.6	TCU039	16.7	NS	0.15	54.00	44.54	23.5
				WE	0.21	50.00	76.78	26.3
Chi-Chi, Taiwan 1999/09/20	7.6	TCU105	18.1	NS	0.13	38.90	45.59	27.2
				WE	0.11	34.60	48.59	27.7
Chi-Chi, Taiwan 1999/09/20	7.6	CHY087	34.5	NS	0.13	11.90	8.11	31.1
				WE	0.14	10.20	7.18	31.9
Chi-Chi, Taiwan 1999/09/20	7.6	CHY086	35.4	WE	0.12	14.20	6.66	31.4
				NS	0.20	17.80	7.89	25.2
Chi-Chi, Taiwan 1999/09/20	7.6	TCU095	43.4	NS	0.71	49.10	24.45	10.2
				WE	0.38	62.00	51.75	13.2

Appendix A-2 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Chi-Chi, Taiwan 1999/09/20	7.6	TCU018	63.8	NS	0.06	22.30	28.27	35.0
				WE	0.05	34.50	52.36	36.0
Chi-Chi, Taiwan 1999/09/20	7.6	TTN046	71.8	NS	0.07	7.40	3.18	30.1
				WE	0.11	11.20	6.13	25.0
Chi-Chi, Taiwan 1999/09/20	7.6	TCU083	78.9	NS	0.11	23.60	13.27	32.0
				WE	0.09	31.90	48.44	37.3
Chi-Chi, Taiwan 1999/09/20	7.6	TCU010	80.4	NS	0.07	19.30	23.89	32.6
				WE	0.09	31.80	46.68	39.4
Chi-Chi, Taiwan 1999/09/20	7.6	TCU008	83.7	NS	0.06	17.50	13.38	31.2
				WE	0.07	29.80	42.50	40.4
Chi-Chi, Taiwan 1999/09/20	7.6	TAP052	99.9	NS	0.13	23.60	14.06	18.4
				WE	0.07	16.60	25.29	25.6
Chi-Chi, Taiwan 1999/09/20	7.6	TAP066	117.5	NS	0.07	12.70	7.78	18.5
				WE	0.05	9.10	15.81	22.1
Chi-Chi, Taiwan 1999/09/20	7.6	CHY042	34.9	NS	0.07	12.30	7.97	37.2
				WE	0.10	15.50	6.50	30.5
Chi-Chi, Taiwan 1999/09/20	7.6	CHY050	50.1	NS	0.07	8.30	7.73	39.5
				WE	0.11	9.80	4.51	28.0
Chi-Chi, Taiwan 1999/09/20	7.6	CHY052	45.0	EW	0.09	9.60	6.91	34.5
				NS	0.15	12.10	9.40	39.4
Coyote Lake 1979/08/06 17:05	5.6	57383 Gilroy Array #6	1.2	230	0.43	49.20	7.77	3.2
				320	0.32	24.50	3.85	3.6
Coyote Lake 1979/08/06 17:05	5.6	1377 San Juan Bautista	17.9	213	0.11	7.60	0.95	11.0
				303	0.11	7.50	1.02	15.1
Coyote Lake 1979/08/06 17:05	5.6	1492 SJB Overpass, Bent 3 g.l.	19.2	67	0.10	5.90	0.55	7.9
				337	0.12	7.60	1.07	6.9
Imperial Valley 1979/10/15 23:16	6.9	5051 Parachute Test Site	14.2	225	0.11	17.80	12.35	17.9
				315	0.20	16.10	9.94	16.9
Imperial Valley 1979/10/15 23:16	6.9	6604 Cerro Prieto	23.5	147	0.17	11.60	4.25	29.7
				237	0.16	18.60	7.95	36.2

Appendix A-2 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Imperial Valley 1979/10/15 23:16	6.9	286 Superstition Mtn Camera	26.0	45	0.11	5.20	2.21	11.6
				135	0.20	8.80	2.78	6.7
Kern County 1952/07/21 11:53	7.7	1095 Taft Lincoln School	42.0	21	0.16	15.30	9.25	30.3
				111	0.18	17.50	8.99	28.8
Kern County 1952/07/21 11:53	7.7	283 Santa Barbara Courthouse	85.0	42	0.09	12.10	2.81	27.6
				132	0.13	15.50	4.06	31.9
Kern County 1952/07/21 11:53	7.7	80053 Pasadena - CIT Athenaeum	109.0	180	0.05	5.60	1.25	28.0
				270	0.05	9.20	2.53	29.2
Landers 1992/06/28 11:58	7.4	22170 Joshua Tree	7.1	0	0.27	27.50	9.82	27.2
				90	0.28	43.20	14.51	26.1
Livermore 1980/01/27 02:33	5.5	57T02 Livermore - Morgan Terr Park	10.1	265	0.20	11.70	1.02	4.2
				355	0.25	9.80	1.30	2.4
Livermore 1980/01/27 02:33	5.5	67070 Antioch - 510 G St	30.9	270	0.11	5.80	0.62	10.6
				360	0.05	2.70	0.32	12.2
Loma Prieta 1989/10/18 00:05	7.1	58219 APEEL 3E Hayward CSUH	56.0	0	0.08	5.60	3.93	14.7
				90	0.08	6.40	3.57	16.0
Loma Prieta 1989/10/18 00:05	7.1	58498 Hayward - BART Sta	57.7	220	0.16	15.10	3.72	10.9
				310	0.16	10.60	3.33	13.2
Loma Prieta 1989/10/18 00:05	7.1	57007 Corralitos	0.0	0	0.64	55.20	10.88	6.9
				90	0.48	45.20	11.37	7.9
Loma Prieta 1989/10/18 00:05	7.1	47006 Gilroy - Gavilan Coll.	10.9	67	0.36	28.60	6.35	5.0
				337	0.33	22.30	4.59	4.8
Loma Prieta 1989/10/18 00:05	7.1	58065 Saratoga - Aloha Ave	11.7	0	0.51	41.20	16.21	9.4
				90	0.32	42.60	27.53	8.3
Loma Prieta 1989/10/18 00:05	7.1	58235 Saratoga - W Valley Coll.	12.0	0	0.26	42.40	19.55	11.1
				270	0.33	61.50	36.40	10.7
Loma Prieta 1989/10/18 00:05	7.1	57383 Gilroy Array #6	19.9	0	0.13	12.80	4.74	12.9
				90	0.17	14.20	3.79	12.6
Loma Prieta 1989/10/18 00:05	7.1	57504 Coyote Lake Dam (Downst)	21.7	195	0.16	13.00	6.11	13.4
				285	0.18	22.60	13.20	11.4

Appendix A-2 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Loma Prieta 1989/10/18 00:05	7.1	47189 SAGO South - Surface	34.1	261	0.07	10.50	6.40	14.4
				351	0.07	9.60	6.42	73.1
Loma Prieta 1989/10/18 00:05	7.1	58127 Woodside	38.7	0	0.08	13.70	8.47	16.8
				90	0.08	16.70	8.89	15.4
Loma Prieta 1989/10/18 00:05	7.1	57064 Fremont - Mission San Jose	42.0	0	0.12	11.50	5.43	18.2
				90	0.11	8.80	4.36	16.7
Loma Prieta 1989/10/18 00:05	7.1	58378 APEEL 7 - Pulgas	46.5	0	0.16	16.10	7.75	17.4
				90	0.09	15.70	8.41	18.4
Loma Prieta 1989/10/18 00:05	7.1	58262 Belmont - Envirotech	48.7	0	0.11	11.80	3.30	12.2
				90	0.11	16.20	5.71	10.3
Loma Prieta 1989/10/18 00:05	7.1	58130 SF - Diamond Heights	75.9	0	0.10	10.00	2.10	8.8
				90	0.11	13.10	3.36	9.3
Loma Prieta 1989/10/18 00:05	7.1	58471 Berkeley LBL	83.9	0	0.06	9.20	1.78	16.9
				90	0.12	20.90	4.44	8.0
Lytle Creek 1970/09/12 14:30	5.3	290 Wrightwood - 6074 Park Dr	13.0	115	0.16	10.10	1.02	3.3
				205	0.20	10.50	0.62	2.7
Morgan Hill 1984/04/24 21:15	6.1	57383 Gilroy Array #6	11.5	0	0.22	11.40	2.45	7.3
				90	0.29	36.70	6.12	6.5
Morgan Hill 1984/04/24 21:15	6.1	47006 Gilroy Gavilan Coll.	16.0	67	0.11	3.60	0.87	8.6
				337	0.10	2.90	0.93	8.2
Northridge 1994/01/17 12:31	6.7	5108 Santa Susana Ground	3.7	0	0.28	19.40	4.11	7.3
				90	0.29	19.70	7.45	8.1
Northridge 1994/01/17 12:31	6.7	5081 Topanga - Fire Sta	12.6	0	0.36	17.60	2.87	8.7
				90	0.27	12.90	1.34	9.4
Northridge 1994/01/17 12:31	6.7	90058 Sunland - Mt Gleason Ave	14.7	170	0.13	13.80	5.54	15.9
				260	0.16	14.50	4.29	11.8
Northridge 1994/01/17 12:31	6.7	24688 LA - UCLA Grounds	16.8	90	0.28	22.00	4.32	11.3
				360	0.47	22.20	7.38	10.2
Northridge 1994/01/17 12:31	6.7	24389 LA - Century City CC North	18.3	90	0.26	21.10	6.68	13.2
				360	0.22	25.20	5.70	13.6

Appendix A-2 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Northridge 1994/01/17 12:31	6.7	24278 Castaic - Old Ridge Route	25.4	90	0.57	52.10	4.21	9.1
				360	0.51	52.20	2.41	8.6
Northridge 1994/01/17 12:31	6.7	24401 San Marino, SW Academy	35.2	90	0.12	7.30	1.10	11.8
				360	0.15	7.40	0.75	12.4
Northridge 1994/01/17 12:31	6.7	24461 Alhambra - Fremont School	36.1	90	0.10	10.90	2.53	21.2
				360	0.08	5.00	1.43	18.5
Northridge 1994/01/17 12:31	6.7	90067 Duarte - Mel Canyon Rd.	50.2	90	0.08	3.40	1.84	16.6
				180	0.03	2.40	0.46	15.8
Northridge 1994/01/17 12:31	6.7	14578 Seal Beach - Office Bldg	63.9	0	0.06	5.80	1.99	18.9
				90	0.08	6.90	1.34	16.0
Northridge 1994/01/17 12:31	6.7	13122 Featherly Park - Pk Maint Bldg	82.5	0	0.10	7.70	0.81	10.4
				90	0.10	5.80	0.51	10.9
Northridge 1994/01/17 12:31	6.7	13123 Riverside - Airport	100.4	180	0.06	2.70	0.28	12.3
				270	0.06	3.10	0.50	9.9
Parkfield 1966/06/28 04:26	6.0	1438 Temblor pre- 1969	16.1	205	0.36	21.50	3.87	4.4
				295	0.27	15.00	3.40	5.5
Parkfield 1966/06/28 04:26	6.0	1016 Cholame #12	17.3	50	0.06	5.80	2.56	28.5
				320	0.06	6.80	3.55	26.6
San Fernando 1971/02/09 14:00	6.6	128 Lake Hughes #12	17.0	21	0.37	17.00	1.65	10.7
				291	0.28	12.70	2.97	11.9
San Fernando 1971/02/09 14:00	6.6	126 Lake Hughes #4	19.6	111	0.19	5.60	0.92	12.7
				201	0.15	8.40	1.85	12.9
San Fernando 1971/02/09 14:00	6.6	24278 Castaic - Old Ridge Route	24.2	21	0.32	15.60	2.31	14.5
				291	0.27	25.90	4.67	15.4
San Fernando 1971/02/09 14:00	6.6	80053 Pasadena - CIT Athenaeum	25.7	0	0.09	6.40	1.36	13.6
				90	0.11	13.30	7.78	12.3
San Fernando 1971/02/09 14:00	6.6	262 Palmdale Fire Station	28.6	120	0.12	12.30	2.65	17.5
				210	0.15	8.10	1.85	17.3
San Fernando 1971/02/09 14:00	6.6	269 Pearblossom Pump	37.4	0	0.10	4.70	1.53	13.7
				270	0.14	5.60	1.61	13.7

Appendix A-2 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
San Fernando 1971/02/09 14:00	6.6	290 Wrightwood - 6074 Park Dr	60.7	25	0.06	2.60	0.47	11.2
				295	0.04	3.30	0.68	11.5
Whittier Narrows 1987/10/01 14:42	5.7	24461 Alhambra, Fremont Sch	6.7	180	0.33	22.00	2.42	5.3
				270	0.41	16.30	2.32	5.8
Whittier Narrows 1987/10/01 14:42	5.7	24401 San Marino, SW Academy	7.7	270	0.13	5.60	0.58	11.4
				360	0.20	12.80	2.60	7.4
Whittier Narrows 1987/10/01 14:42	5.7	14196 Inglewood - Union Oil	22.5	0	0.30	8.90	0.78	7.7
				90	0.25	18.10	1.92	11.3
Whittier Narrows 1987/10/01 14:42	5.7	24278 Castaic - Old Ridge Route	77.3	0	0.07	4.40	0.40	13.2
				90	0.07	4.50	0.38	11.7

Appendix A-3 Earthquake Ground Motions Selected on NEHRP Site D

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Borrego Mtn 1968/04/09 02:30	6.5	117 El Centro Array #9	45.0	180	0.19	11.2	2.41	16.0
				270	0.20	12.7	3.45	20.3
Chi-Chi, Taiwan 1999/09/20	7.6	TCU060	9.5	NS	0.19	7.4	2.06	9.4
Chi-Chi, Taiwan 1999/09/20	7.6	TCU068	0.5	WE	0.11	6.0	1.79	10.7
				NS	0.58	75.5	17.57	5.9
Chi-Chi, Taiwan 1999/09/20	7.6	CHY101	11.1	WE	0.59	97.2	38.05	5.5
				NS	0.18	12.5	1.14	9.7
Chi-Chi, Taiwan 1999/09/20	7.6	TCU036	16.7	WE	0.16	11.3	3.00	11.0
				NS	0.08	7.9	1.27	7.2
Chi-Chi, Taiwan 1999/09/20	7.6	CHY010	25.4	WE	0.11	6.8	1.76	7.1
				EW	0.21	18.9	12.40	10.5
Chi-Chi, Taiwan 1999/09/20	7.6	CHY046	29.5	NS	0.17	14.9	6.25	11.2
				NS	0.11	14.8	2.59	10.8
Chi-Chi, Taiwan 1999/09/20	7.6	HWA005	39.0	WE	0.08	4.6	0.80	13.7
				EW	0.10	12.2	3.71	20.2
Chi-Chi, Taiwan 1999/09/20	7.6	HWA006	39.2	NS	0.19	14.3	2.38	14.8
				EW	0.07	12.8	6.95	23.1
Chi-Chi, Taiwan 1999/09/20	7.6	CHY015	43.5	NS	0.04	8.6	3.52	28.4
				EW	0.08	10.9	6.95	39.4
Chi-Chi, Taiwan 1999/09/20	7.6	HWA007	56.4	NS	0.09	13.8	5.29	35.8
				NS	0.10	7.8	2.23	21.5
Chi-Chi, Taiwan 1999/09/20	7.6	CHY063	78.1	WE	0.09	7.7	3.08	24.4
				NS	0.12	17.3	2.59	12.0
Chi-Chi, Taiwan 1999/09/20	7.6	TAP047	85.8	WE	0.11	14.2	3.91	13.7
				NS	0.30	22.1	7.84	12.4
Chi-Chi, Taiwan 1999/09/20	7.6	CHY065	90.2	WE	0.44	38.2	10.04	17.0
				NS	0.61	117.4	53.47	15.1
Chi-Chi, Taiwan 1999/09/20	7.6	TAP041	111.2	WE	0.90	102.8	46.99	7.5
				NS	0.19	37.5	9.96	12.5
				WE	0.28	29.3	9.72	11.6

Appendix A-3 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Chi-Chi, Taiwan 1999/09/20	7.6	TAP046	127.0	NS	0.06	16.6	12.13	28.8
				WE	0.09	21.6	28.19	29.6
Coyote Lake 1979/08/06 17:05	5.6	47380 Gilroy Array #2	7.4	50	0.05	6.6	4.93	28.7
				140	0.08	12.6	7.08	26.5
Coyote Lake 1979/08/06 17:05	5.6	47381 Gilroy Array #3	5.3	50	0.05	17.4	12.23	25.1
				140	0.05	16.9	29.53	28.4
Coyote Lake 1979/08/06 17:05	5.6	57382 Gilroy Array #4	3.7	270	0.13	50.2	42.17	27.0
				360	0.14	59.6	63.60	23.4
Imperial Valley 1979/10/15 23:16	6.9	412 El Centro Array #10	8.5	50	0.17	13.7	3.89	12.8
				320	0.14	11.5	5.65	12.9
Imperial Valley 1979/10/15 23:16	6.9	5052 Plaster City	32.0	45	0.22	28.8	3.95	9.3
				135	0.14	13.4	1.60	12.3
Imperial Valley 1979/10/15 23:16	6.9	5053 Calexico Fire Station	10.6	225	0.13	26.3	12.18	25.6
				315	0.06	13.2	10.03	28.6
Imperial Valley 1979/10/15 23:16	6.9	5058 El Centro Array #11	12.6	230	0.17	26.0	12.64	18.4
				140	0.16	17.6	9.75	21.4
Imperial Valley 1979/10/15 23:16	6.9	5059 El Centro Array #13	22.0	140	0.04	1.4	0.13	20.1
				230	0.03	1.6	0.10	17.8
Imperial Valley 1979/10/15 23:16	6.9	5066 Coachella Canal #4	49.0	45	0.22	9.0	1.43	11.5
				135	0.12	6.9	1.12	13.8
Imperial Valley 1979/10/15 23:16	6.9	6610 Victoria	43.5	75	0.15	18.9	6.13	10.3
				345	0.08	6.1	1.69	14.2
Imperial Valley 1979/10/15 23:16	6.9	931 El Centro Array #12	18.0	140	0.04	2.5	0.19	13.64
				230	0.03	1.6	0.18	16.6
Imperial Valley 1979/10/15 23:16	6.9	952 El Centro Array #5	4.0	140	0.05	2.1	0.14	13.9
				230	0.06	2.7	0.19	13.8
Kern County 1952/07/21 11:53	7.7	135 LA - Hollywood Stor Lot	107.0	90	0.16	5.1	1.42	16.4
				180	0.21	12.6	2.10	11.8
Landers 1992/06/28 11:58	7.5	12331 Hemet Fire Station	69.1	0	0.04	1.6	0.10	14.1
				90	0.05	1.9	0.11	14.4
Landers 1992/06/28 11:58	7.5	12026 Indio - Coachella Canal	54.9	0	0.04	2.9	0.31	17.1
				90	0.06	2.9	0.41	14.6

Appendix A-3 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Landers 1992/06/28 11:58	7.5	12025 Palm Springs Airport	36.7	0	0.07	3.4	0.35	15.4
				90	0.06	2.5	0.22	14.3
Landers 1992/06/28 11:58	7.5	14368 Downey - Co Maint Bldg	156.0	0	0.05	7.5	2.35	19.6
				90	0.07	7.6	1.81	20.2
Landers 1992/06/28 11:58	7.5	90002 Fountain Valley - Euclid	148.8	22	0.05	10.0	3.84	24.0
				292	0.05	9.5	3.04	24.8
Landers 1992/06/28 11:58	7.5	90074 La Habra - Briarcliff	142.8	0	0.44	24.7	5.15	6.5
				90	0.37	21.8	3.83	7.4
Landers 1992/06/28 11:58	7.5	90084 Lakewood - Del Amo Blvd	155.8	0	0.25	10.2	3.60	13.1
				90	0.27	11.3	3.20	10.6
Landers 1992/06/28 11:58	7.5	90078 Compton - Castlegate	161.2	0	0.53	36.5	9.11	11.9
				270	0.44	29.3	5.50	13.2
Landers 1992/06/28 11:58	7.5	90083 Huntington Bch - Waikiki	153.3	200	0.07	12.2	4.99	22.3
				290	0.06	13.1	4.05	21.7
Landers 1992/06/28 11:58	7.5	23525 Pomona - 4th & Locust	117.6	0	0.23	19.2	7.26	26.4
				90	0.17	21.9	11.07	29.8
Landers 1992/06/28 11:58	7.5	23542 San Bernardino-E & Hospitalit	79.9	90	0.15	22.5	9.83	37.5
				180	0.16	25.7	13.17	38.8
Landers 1992/06/28 11:58	7.5	22074 Yermo Fire Station	26.3	270	0.18	21.0	11.90	32.2
				360	0.14	20.6	10.28	33.0
Livermore 1980/01/24 19:00	5.8	57187 San Ramon - Eastman Kodak	15.7	180	0.06	9.5	5.09	25.0
				270	0.06	10.8	4.07	22.4
Livermore 1980/01/24 19:00	5.8	57063 Tracy - Sewage Treatm Plant	28.5	93	0.25	51.5	43.81	17.6
				183	0.15	29.7	24.69	18.9
Loma Prieta 1989/10/18 00:05	7.1	47380 Gilroy Array #2	12.1	0	0.07	9.4	8.27	43.4
				90	0.06	7.9	6.92	40.4
Loma Prieta 1989/10/18 00:05	7.1	57191 Halls Valley	29.3	0	0.10	12.5	8.25	37.9
				90	0.12	15.8	8.44	37.9
Loma Prieta 1989/10/18 00:05	7.1	57066 Agnews State Hospital	27.0	0	0.44	115.0	68.75	26.5
				90	0.35	70.6	45.28	30.4
Loma Prieta 1989/10/18 00:05	7.1	47125 Capitola	8.6	0	0.05	14.1	4.88	28.0
				90	0.05	12.8	5.14	27.4

Appendix A-3 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Loma Prieta 1989/10/18 00:05	7.1	57425 Gilroy Array #7	24.3	0	0.16	13.8	2.28	17.3
				90	0.23	11.3	2.14	14.4
Loma Prieta 1989/10/18 00:05	7.1	58393 APEEL 2E Hayward Muir Sch	56.3	0	0.07	14.7	7.87	27.8
				90	0.06	10.3	4.70	26.2
Loma Prieta 1989/10/18 00:05	7.1	58224 Oakland - Title & Trust	76.3	180	0.37	32.9	7.15	11.0
				270	0.21	10.9	2.29	7.5
Loma Prieta 1989/10/18 00:05	7.1	1601 Palo Alto - SLAC Lab	35.0	270	0.32	39.1	12.07	9.5
				360	0.34	24.9	5.81	4.2
Loma Prieta 1989/10/18 00:05	7.1	58505 Richmond City Hall	92.7	190	0.27	18.7	3.42	8.9
				280	0.23	28.8	4.87	8.7
Morgan Hill 1984/04/24 21:15	6.1	47380 Gilroy Array #2	1.0	0	0.25	23.1	2.60	8.4
				90	0.27	26.3	4.78	11.4
Morgan Hill 1984/04/24 21:15	6.1	47381 Gilroy Array #3	14.4	0	0.23	16.4	2.52	11.5
				90	0.32	16.6	3.26	9.1
Morgan Hill 1984/04/24 21:15	6.1	57425 Gilroy Array #7	13.7	0	0.08	5.6	1.36	33.2
				90	0.10	5.7	2.27	31.9
Northridge 1994/01/17 12:31	6.7	90057 Canyon Country - W Lost Cany	12.2	0	0.12	12.5	2.33	11.1
				270	0.13	15.6	2.95	10.0
Northridge 1994/01/17 12:31	6.7	90060 La Crescenta - New York	17.9	90	0.28	21.2	9.02	11.0
				180	0.20	16.0	9.20	14.6
Northridge 1994/01/17 12:31	6.7	90006 Sun Valley - Roscoe Blvd	6.1	0	0.52	46.9	35.35	8.3
				90	0.38	90.5	63.03	9.4
Northridge 1994/01/17 12:31	6.7	74 Sylmar - Converter Sta	0.2	52	0.17	47.5	31.10	12.8
				142	0.22	41.0	19.38	12.0
Northridge 1994/01/17 12:31	6.7	24279 Newhall - Fire Sta	4.5	90	0.36	34.5	16.07	8.7
				360	0.38	42.1	18.59	7.9
Northridge 1994/01/17 12:31	6.7	24303 LA - Hollywood Stor FF	20.8	90	0.14	17.6	11.30	19.1
				360	0.12	21.8	12.06	19.4
Northridge 1994/01/17 12:31	6.7	24612 LA - Pico & Sentous	29.0	90	0.12	14.7	7.33	21.4
				180	0.14	13.0	5.84	21.2
Northridge 1994/01/17 12:31	6.7	24271 Lake Hughes #1	37.7	0	0.09	10.1	1.13	27.8
				90	0.07	13.9	1.49	28.0

Appendix A-3 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Northridge 1994/01/17 12:31	6.7	14368 Downey - Co Maint Bldg	45.1	90	0.23	18.3	4.81	12.0
				360	0.36	27.5	3.04	10.7
Northridge 1994/01/17 12:31	6.7	23542 San Bernardino - E & Hosp	110.4	90	0.09	5.9	0.97	17.0
				180	0.10	6.5	1.34	16.2
Northridge 1994/01/17 12:31	6.7	24281 Port Hueneme - Naval Lab.	50.0	90	0.04	3.2	1.34	10.8
				180	0.06	5.4	1.94	10.7
Northridge 1994/01/17 12:31	6.7	13197 Huntington Beach - Lake St	68.9	0	0.12	6.4	2.09	19.4
				90	0.17	8.3	1.05	17.1
Parkfield 1966/06/28 04:26	6.0	1014 Cholame #5	9.3	85	0.11	45.3	45.56	26.7
				355	0.20	36.3	51.89	21.8
Parkfield 1966/06/28 04:26	6.0	1015 Cholame #8	13.0	50	0.46	263.1	430.00	13.1
				320	0.57	176.6	324.11	12.5
Point Mugu 1973/02/21 14:45	5.2	272 Port Hueneme	16.0	180	0.20	19.9	3.54	11.6
				270	0.24	36.1	7.20	12.5
San Fernando 1971/02/09 14:00	6.6	135 LA - Hollywood Stor Lot	24.6	90	0.13	15.4	3.30	16.2
				180	0.10	13.5	5.46	13.7
San Fernando 1971/02/09 14:00	6.6	994 Gormon - Oso Pump Plant	46.7	0	0.15	12.5	8.22	23.7
				270	0.14	16.6	9.18	27.4
Whittier Narrows 1987/10/01 14:42	5.7	14368 Downey - Co Maint Bldg	16.2	180	0.09	9.2	6.11	18.1
				270	0.08	7.3	5.89	20.3
Whittier Narrows 1987/10/01 14:42	5.7	24303 LA - Hollywood Stor FF	23.8	0	0.09	15.2	13.95	24.0
				90	0.07	18.4	26.27	35.9
Whittier Narrows 1987/10/01 14:42	5.7	23525 Pomona - 4th & Locust FF	29.9	12	0.10	9.6	5.05	35.9
				102	0.11	15.2	9.69	37.9
Whittier Narrows 1987/10/01 14:42	5.7	12331 Hemet Fire Station	105.0	270	0.04	7.5	4.79	32.4
				360	0.06	6.2	1.86	28.9
Whittier Narrows 1987/10/01 14:42	5.7	24271 Lake Hughes #1	74.7	0	0.05	18.3	24.03	44.8
				90	0.04	11.3	10.32	49.9
Whittier Narrows 1987/10/01 14:42	5.7	24055 Leona Valley #5 - Ritter	61.3	0	0.08	19.8	10.49	71.0
				90	0.09	14.6	7.63	77.1
Whittier Narrows 1987/10/01 14:42	5.7	24309 Leona Valley #6	64.8	0	0.09	9.4	3.70	13.9
				90	0.08	9.5	2.40	16.1

Appendix A-3 (Continued)

Earthquake	Ms	Station	Dist.	Comp.	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (s)
Whittier Narrows 1987/10/01 14:42	5.7	24279 Newhall - Fire Sta	55.2	180	0.41	43.0	11.75	6.3
				270	0.48	45.1	12.58	5.6

APPENDIX B

STOREY DISPLACEMENT AND STOREY SHEAR TIME HISTORIES OF FRP-REPAIRED FRAMES

Figure B-1 Storey shear time histories for CHL-012R

Figure B-2 Storey shear time histories for CHL-023R

Figure B-3 Storey shear time histories for CHL-046R

Figure B-4 Storey shear time histories for CHL-069R

Figure B-5 Storey displacement time histories for CHL-012R

(a) Frame FC

(b) Frame FM

Figure B-6 Storey displacement time histories for CHL-023R

(a) Frame FC

(b) Frame FM

Figure B-7 Storey displacement time histories for CHL-046R

(a) Frame FC

(b) Frame FM

Figure B-8 Storey displacement time histories for CHL-069R

APPENDIX C

SCALING FACTORS FOR MODEL RESPONSE QUANTITIES

(Based on Harris and Sabnis 1999)

Category	Quantity	Dimension	Scale factor
Loading	Force	F	λ^2
	Pressure	FL^{-2}	1
	Acceleration	LT^{-2}	1
	Gravitational acceleration	LT^{-2}	1
	Velocity	LT^{-1}	$\lambda^{1/2}$
	Time	T	$\lambda^{1/2}$
Geometry	Linear dimension,	L	λ
	Displacement,	L	λ^2
	Frequency,	T^{-1}	$\lambda^{-1/2}$
Material properties	Modulus,	FL^{-2}	1
	Stress,	FL^{-2}	1
	Strain,	—	1
	Poisson's ratio,	—	1
	Mass	$FL^{-1}T^2$	λ^2
	Energy,	FL	λ^3

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