Performance-oriented seismic reliability analysis of structural systems

Gu, Xiaoming

2005

Gu, X. M. (2005). Performance-oriented seismic reliability analysis of structural systems.
Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/12253>

<https://doi.org/10.32657/10356/12253>

Nanyang Technological University

Downloaded on 26 Apr 2023 19:24:39 SGT

PERFORMANCE-ORIENTED SEISMIC RELIABILITY ANALYSIS OF STRUCTURAL SYSTEMS

GU XIAOMING

**School of Civil & Environmental Engineering
NANYANG TECHNOLOGICAL UNIVERSITY**

2005

PERFORMANCE-ORIENTED SEISMIC RELIABILITY ANALYSIS OF STRUCTURAL SYSTEMS

GU XIAOMING

School of Civil & Environmental Engineering

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

2005

ACKNOWLEDGEMENTS

It is my great pleasure to thank all who have helped me to get this point.

I would like to express my sincere gratitude to my supervisor, Associate Professor Lu Yong, for his unreserved guidance, invaluable suggestions and enthusiastic encouragement during the study. I benefit a lot from his philosophical perspectives and visions. Outside research, I also learn a lot of things valuable from him in many other aspects.

I would like to thank fellow students, Mr. Wei Jianwu, Mr. Tu Zhenguo, Mr. Shao Yongbo and many others, who have spent their precious time on discussing with me.

The research scholarship provided by Nanyang Technological University is gratefully acknowledged.

Last but not least, my deepest gratitude goes to my family, to whom this thesis is dedicated, for their love and support in this long and winding road.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS.....	I
TABLE OF CONTENTS	IV
ABSTRACT	IX
LIST OF TABLES.....	XI
LIST OF FIGURES	XIII
LIST OF SYMBOLS.....	XVII
 CHAPTER 1 INTRODUCTION	 1
1.1 BACKGROUND.....	1
<i>1.1.1 Performance-based seismic design philosophy</i>	<i>1</i>
<i>1.1.2 Reliability-based seismic performance assessment</i>	<i>2</i>
1.2 RESEARCH OBJECTIVES	3
1.3 THESIS ORGANIZATION.....	4
 CHAPTER 2 LITERATURE REVIEW	 5
2.1 INTRODUCTION	5
2.2 PERFORMANCE-BASED SEISMIC DESIGN	5
2.3 DIRECT DISPLACEMENT-BASED DESIGN.....	7
2.4 UNCERTAINTIES INVOLVED IN THE RELIABILITY ANALYSIS	9
<i>2.4.1 Basic reliability model</i>	<i>9</i>
<i>2.4.2 Categorization of uncertainties.....</i>	<i>11</i>
<i>2.4.3 Uncertainties in material property.....</i>	<i>13</i>
<i>2.4.4 Uncertainty in modeling of confined concrete</i>	<i>16</i>

2.4.5 Modeling of uncertainty in seismic motions	22
2.5 PROBABILISTIC ASSESSMENT OF RC MEMBER CAPACITY	24
2.5.1 Strength capacity.....	24
2.5.2 Deformation capacity	26
2.6 ESTIMATION OF DISPLACEMENT DEMAND FOR RC STRUCTURES	28
2.6.1 Fajfar & Fischinger (1989) (N2 method)	28
2.6.2 Miranda (1999).....	29
2.6.3 Gupta & Krawinkler (2000)	30
2.6.4 Displacement spectrum.....	31
2.6.5 Hysteretic model	32
2.7 SEISMIC RELIABILITY ANALYSIS OF RC STRUCTURAL SYSTEMS	34
2.8 FUZZY RELIABILITY ANALYSIS	36
2.8.1 Overview of fuzzy set theory (based on Zimmerman 1996)	36
2.8.2 Application of Fuzzy Set Theory in reliability analysis	38
2.9 SUMMARY.....	42

CHAPTER 3 PROBABILISTIC ANALYSIS ON RC MEMBER

DEFORMATION LIMITS FOR DIFFERENT PERFORMANCE LEVELS.....44

3.1 INTRODUCTION	44
3.2 DEFINITION OF PERFORMANCE LEVELS	45
3.3 BASIC RANDOM VARIABLES INFLUENCING RC MEMBER BEHAVIOR.....	47
3.3.1 Variables related to concrete properties	48
3.3.2 VARIABLES RELATED TO REINFORCEMENT PROPERTIES	49
3.3.3 Uncertainty in modeling plastic hinge length.....	50
3.3.4 Proposed model for equivalent plastic hinge length.....	55
3.4 MONTE CARLO SIMULATION OF RC MEMBER BEHAVIOR	58
3.4.1 Sectional analysis	58
3.4.2 Representative specimens	59
3.5 PROBABILISTIC DISTRIBUTIONS OF RC MEMBER BEHAVIOR AT DIFFERENT	

PERFORMANCE LEVELS FROM SIMULATION RESULTS	61
3.5.1 Yield strength and yield curvature.....	61
3.5.2 Sectional response at Collapse level.....	62
3.5.3 Curvature at Functional performance level.....	65
3.5.4 Curvature at Damage Control performance level	68
3.6 DETERMINISTIC CALCULATIONS OF LIMIT CURVATURES AND ASSESSMENT OF THEIR RELIABILITY	71
3.7 MONTE CARLO SIMULATION ON RC MEMBER DRIFT	75
3.7.1 Probabilistic distribution of RC member drift and its deterministic calculation.....	75
3.7.2 Verification of ultimate state drift limit against experimental data	81
3.7.3 Comparison between simulated drift limits and code specified limits.....	82
3.8 SUMMARY AND CONCLUSIONS	84

CHAPTER 4 FUZZY-RANDOM RELIABILITY MODEL FOR COMPREHENSIVE PERFORMANCE ANALYSIS OF FRAMED STRUCTURES.....87

4.1 INTRODUCTION	87
4.2 NON-STRUCTURAL COMPONENTS.....	88
4.2.1 Classification of non-structural components	89
4.2.2 Non-structural damage and its effects	90
4.2.3 Probabilistic drift capacity for masonry infill walls at performance levels	92
4.3 DEFINING A PERFORMANCE LEVEL CONCERNING BOTH STRUCTURAL AND NON-STRUCTURAL COMPONENTS	98
4.4 THE FUZZY-RANDOM MODEL.....	99
4.4.1 Fuzzified definition of performance level.....	99
4.4.2 Membership function.....	100
4.5 PROPOSED FUZZY-RANDOM RELIABILITY MODEL	109
4.5.1 The classical reliability model	109

4.5.2 The proposed fuzzy-random reliability model.....	109
4.5.3 Example application	112
4.6 SUMMARY AND CONCLUSIONS.....	114

CHAPTER 5 PROBABILISTIC NONLINEAR CRITICAL INTER-STOREY

DRIFT DEMAND OF RC FRAME STRUCTURES 117

5.1 INTRODUCTION	117
5.2 GROUND MOTIONS.....	119
5.2.1 Consideration of uncertainties in seismic input	119
5.2.2 Scaling of ground motions	120
5.3 REPRESENTATIVE FRAMES	122
5.3.1 A generic regular 7-storey frame structure (7-R)	122
5.3.2 Static pushover analysis.....	125
5.3.3 New regularity index incorporating the effects of strength and stiffness... 126	
5.3.4 Irregular 7-storey frame structures (7-I-a and 7-I-b).....	132
5.3.5 Regular 12-storey frame structure 12-R	135
5.3.6 Irregular 12-storey frame structures (12-I-a and 12-I-b).....	137
5.4 NONLINEAR TIME HISTORY ANALYSIS	141
5.4.1 Component modeling	141
5.4.2 Structural modeling	142
5.4.3 Hysteretic modeling	142
5.5 DISCUSSION ON THE TIME HISTORY ANALYSIS RESULTS	143
5.5.1 Results for frame 7-R	143
5.5.2 Results for 7-I-a and 7-I-b	149
5.5.3 Results for 12-R	153
5.5.4 Results for 12-I-a and 12-I-b	155
5.6 PERFORMANCE-ORIENTED SEISMIC RELIABILITY ASSESSMENT BASED ON CRITICAL INTER-STOREY DRIFT	158
5.6.1 Probabilistic model of critical inter-storey drift ratio	158

5.6.2 Application example.....	160
5.7 SUMMARY AND CONCLUSIONS	167
CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS.....	169
6.1 CONCLUSIONS.....	169
6.2 RECOMMENDATIONS ON FUTURE RESEARCH	172
REFERENCE	174
APPENDIX A. EXPERIMENTAL LATERAL DEFORMATION CAPACITY OF MASONRY WALLS	195
APPENDIX B. SECTION DETAIL OF REPRESENTATIVE FRAMES	208
PUBLICATIONS.....	224

ABSTRACT

Past earthquake experiences indicate that most buildings designed in accordance with modern seismic design codes could survive moderate to strong earthquakes; however, the financial loss due to repairing cost and the subsequent business interruption can be unacceptable. One of the major developments in seismic design over the past 10 years in attempt to address the above concern is the so-called performance-based seismic design. There has been extensive research on the performance-based seismic design philosophy targeting a uniform seismic risk. However, a practical methodology of reliability-based seismic performance evaluation is far from mature. This thesis attempts to address some of the problems in performance-oriented seismic reliability analysis of structural systems. It contains four parts.

In the first part, an extensive literature review is performed on related issues including performance-based design philosophy, various uncertainties involved in seismic reliability analysis, probabilistic studies on RC element capacity, approximate methods to estimate the seismic displacement demand, reliability analysis procedures for RC structural systems and the potential application of the fuzzy set theory.

In the second part, probabilistic models defining three main performance levels are studied for both structural and non-structural components. The probabilistic limit state criteria for framed structures, particularly in terms of the inter-storey drift, are established by Monte Carlo Simulation incorporating the model uncertainties of both concrete confinement effect and equivalent plastic hinge length. The drift limits for non-structural damage is obtained from an extensive survey of relevant experimental data of masonry infill wall panels, which is considered as the representative of non-structural components. The obtained structural and non-structural drift ratio limits show good correlations with relevant experimental

data and code specified limit criteria. At the same time, considerable differences between the drift limits concerning structural and non-structural damages at similar performance levels are observed.

To incorporate both the structural and non-structural criteria into reliability analysis in a coherent manner, a fuzzy-random reliability model is developed as the third part of this study. The fuzziness comes from the difference between non-structural and structural limit criteria while the randomness is associated with the probabilistic properties of both limit criteria. The relative importance of structural and non-structural components in the building performance assessment is reflected by an appropriate cost function. Two fuzzy-random events are defined as Occurrence and Exceeding of a performance level and the corresponding probability of the fuzzy-random events can be calculated by the proposed model. Numerical application shows the proposed fuzzy-random model provides a more comprehensive reliability evaluation incorporating the influence of both non-structural and structural damage on the building performance, comparing with the results from classic reliability approach based on single-threshold limit criteria.

Finally, a framework for establishing a probabilistic model of the maximum inter-storey drift in a framed structure is proposed for the purposes of design applications and rapid performance assessment. A storey capacity factor is devised to account for the possible effects of both the storey strength and the storey stiffness on the distribution of the inter-storey drift in a frame structure. This factor is found to correlate well with the actual inter-storey displacement profile obtained from the detailed nonlinear time history analysis. A new regularity index is then defined based on the storey capacity factor. This new regularity index is found to have a good correlation with the drift concentration factor that defines the maximum inter-storey drift. An exponential equation is derived from regression analysis to describe the relationship between the concentration factor and the regularity index.

LIST OF TABLES

TABLE 2.1 SCATTER IN $\varepsilon_{CU,EXP}/\varepsilon_{CU,PRED}$ (DYMOTIS 1999).....	21
TABLE 3.1 EMPIRICAL FORMULAS OF L_p	51
TABLE 3.2 EXPERIMENTAL AND PREDICTED VALUES OF PLASTIC HINGE LENGTH L_p/D_s	52
TABLE 3.3 EXPERIMENTAL AND PREDICTED VALUE OF PLASTIC HINGE LENGTH L_p/Z_C	53
TABLE 3.4 MODEL UNCERTAINTY FACTOR $X_{M,Lp}$ FOR DIFFERENT EMPIRICAL MODEL OF L_p ...	56
TABLE 3.5 DETAILS OF SPECIMENS.....	60
TABLE 3.6 NORMALIZED SECTIONAL CURVATURE (MULTIPLIED BY SECTION DEPTH) AT DIFFERENT PERFORMANCE LEVELS	61
TABLE 3.7 PREDICTED SECTIONAL CURVATURES (MULTIPLIED BY SECTION DEPTH) AT DIFFERENT PERFORMANCE LEVELS USING DETERMINISTIC APPROACH.....	72
TABLE 3.8 LIMIT DRIFT RATIOS FOR DIFFERENT PERFORMANCE LEVELS.....	76
TABLE 3.9 DRIFT RATIO LIMITS BY DETERMINISTIC APPROACH AND ASSOCIATED PROBABILITY	80
TABLE 3.10 DRIFT LIMITS FROM DESIGN CODES	83
TABLE 3.11 DRIFT LIMITS CORRESPONDING TO PROBABILITY OF BEING EXCEEDED OF 95% ..	83
TABLE 4.1 REPRESENTATIVE CATEGORIES OF NON-STRUCTURAL COMPONENTS	90
TABLE 4.2 CATEGORIZATION OF SPECIMENS	95
TABLE 4.3 STATISTICAL PROPERTIES OF TEST DATA FROM LITERATURE SURVEY	96
TABLE 4.4 DETAILS OF ASSUMED INPUT DATA	113
TABLE 4.5 RESULTS FROM PROPOSED MODEL	113
TABLE 5.1 SUMMARY OF GROUND MOTIONS (SITE SOIL D, EPICENTRAL DIST. 0~35KM)	119
TABLE 5.2 GENERAL DESIGN INFORMATION FOR FRAME 7-R	123
TABLE 5.3 SECTION AND REINFORCEMENT DETAILS FOR FRAME 7-R	123
TABLE 5.4 FACTORS RELATED TO REGULARITY INDEX (7-R, FORCE STYLE-A)	131
TABLE 5.5 FACTORS RELATED TO REGULARITY INDEX (7-I-A, FORCE STYLE-A)	132

TABLE 5.6 FACTORS RELATED TO REGULARITY INDEX (7-I-B).....	134
TABLE 5.7 FACTORS RELATED TO REGULARITY INDEX (12-R)	136
TABLE 5.8 FACTORS RELATED TO REGULARITY INDEX (12-I-A).....	138
TABLE 5.9 FACTORS RELATED TO REGULARITY INDEX (12-I-B).....	140
TABLE 5.10 SUMMARY OF COV OF θ_{MAX} AND $\theta_{r,PEAK}$	159
TABLE 5.11 FACTORS RELATED TO REGULARITY INDEX (7-CS, FORCE STYLE-A)	160
TABLE 5.12 PROBABILISTIC DISPLACEMENT CAPACITY THE 5 TH STOREY OF FRAME 7-CS	161
TABLE 5.13 PROBABILISTIC MODEL OF NON-STRUCTURAL DISPLACEMENT CAPACITY	162
TABLE A.1 DETAILS OF THE TEST PROGRAMS	195
TABLE B.1 SECTION DETAILS FOR FRAME 7-I-A.....	208
TABLE B.2 SECTION DETAILS FOR FRAME 7-I-B.....	209
TABLE B.3 SECTION DETAILS FOR FRAME 12-R.....	210
TABLE B.4 SECTION DETAILS FOR FRAME 12-I-A.....	212
TABLE B.5 SECTION DETAILS FOR FRAME 12-I-B.....	214
TABLE B.6 DISPLACEMENT RESPONSE (7-R, A95=0.26G)	216
TABLE B.7 DISPLACEMENT RESPONSE (7-R, A95=0.20G)	217
TABLE B.8 DISPLACEMENT RESPONSE (7-I-A, A95=0.26G)	218
TABLE B.9 DISPLACEMENT RESPONSE (7-I-B, A95=0.26G)	219
TABLE B.10 DISPLACEMENT RESPONSE (12-R, A95=0.26G)	220
TABLE B.11 DISPLACEMENT RESPONSE (12-I-A, A95=0.26G)	221
TABLE B.12 DISPLACEMENT RESPONSE (12-I-B, A95=0.26G)	222
TABLE B.13 SECTION DETAILS OF FRAME 7-CS.....	223

LIST OF FIGURES

FIGURE 2.1 RELATIONSHIP BETWEEN EARTHQUAKE DESIGN LEVEL AND PERFORMANCE LEVEL (REPRODUCED FROM SEAOC 1995).....	6
FIGURE 2.2 FUNDAMENTALS OF DIRECT DISPLACEMENT-BASED DESIGN (PRIESTLEY 2000)	8
FIGURE 2.4 STRESS-STRAIN BEHAVIOR OF CONFINED AND UNCONFINED CONCRETE	14
FIGURE 2.5 CONFINED CONCRETE STRESS-STRAIN CURVE BY SHEIKH & UZUMERI (1982).....	18
FIGURE 2.6 THREE-PARAMETER MODEL: (PARK ET AL. 1987, VALLES ET AL. 1996).....	33
FIGURE 3.1 FIBER MODEL FOR RC SECTIONAL ANALYSIS	47
FIGURE 3.2 ESTIMATION OF L_p FROM SECTION HEIGHT D_s	53
FIGURE 3.3 ESTIMATION OF L_p FROM CONTRA-FLEXURE POINT HEIGHT Z_c	54
FIGURE 3.4 “EXACT” CURVATURE DISTRIBUTIONS FOR DEFLECTION CALCULATIONS	56
FIGURE 3.5 COMPARISON OF EXPERIMENTAL AND PREDICTED PLASTIC HINGE LENGTHS.....	57
FIGURE 3.6 TYPICAL HISTOGRAMS OF SECTIONAL CURVATURE FOR SPECIMENS WITH DIFFERENT GEOMETRY OR REINFORCEMENT DETAILS AT ULTIMATE STATE	64
FIGURE 3.7 VARIATION OF MEAN NORMALIZED SECTIONAL CURVATURE AT DIFFERENT PERFORMANCE LEVELS WITH REINFORCEMENT DETAILS AND AXIAL FORCE RATIOS	65
FIGURE 3.8 TYPICAL HISTOGRAMS OF SIMULATED FUNCTIONAL LEVEL CURVATURE.....	67
FIGURE 3.9 TYPICAL HISTOGRAMS OF SIMULATED DAMAGE CONTROL LEVEL CURVATURE ...	70
FIGURE 3.10 PROBABILITY OF ACTUAL CURVATURE LIMITS EXCEEDING THE VALUES PREDICTED BY DETERMINISTIC APPROACH AT DIFFERENT PERFORMANCE LEVELS	73
FIGURE 3.11 SCATTER PLOTS OF PROBABILITY OF ACTUAL CURVATURE LIMITS EXCEEDING THE VALUES PREDICTED BY DETERMINISTIC APPROACH	74
FIGURE 3.12 TYPICAL HISTOGRAMS OF SIMULATED DRIFT LIMITS FOR DIFFERENT PERFORMANCE LEVELS.....	78
FIGURE 3.13 VARIATION OF MEAN LIMIT DRIFT RATIOS FOR DIFFERENT PERFORMANCE LEVELS WITH REINFORCEMENT DETAILS AND AXIAL FORCE RATIOS	79
FIGURE 3.14 SCATTER PLOTS OF PROBABILITY OF ACTUAL DRIFT LIMITS EXCEEDING THE	

VALUES PREDICTED BY DETERMINISTIC APPROACH	80
FIGURE 3.15 COMPARISON BETWEEN SIMULATED AND EXPERIMENTAL RESULTS	82
FIGURE 4.1 DEFINITION OF PERFORMANCE DRIFT LIMITS FOR MASONRY WALLS	94
FIGURE 4.2 HISTOGRAM OF FUNCTIONAL DRIFTS FOR MASONRY WALL	97
FIGURE 4.3 AN EXAMPLE OF LOWER AND UPPER DRIFT LIMITS (DISTRIBUTIONS) DEFINING “DAMAGE CONTROL” PERFORMANCE LEVEL	99
FIGURE 4.4 PROBABLE SHAPE OF MEMBERSHIP FUNCTION CURVE OF EXCEEDING	103
FIGURE 4.5 FAILURE COST FUNCTION WITH VARIOUS γ , KANDA & SHAH (1997)	105
FIGURE 4.6 COMPARISON OF RELIABILITY RESULTS FROM DIFFERENT MODELS.....	113
FIGURE 5.1 ELASTIC ACCELERATION RESPONSE SPECTRUMS OF GROUPED GROUND MOTIONS.....	121
FIGURE 5.2 DISTRIBUTION OF OVERSTRENGTH FACTORS (7-R, COLUMNS, FORCE STYLE-A)	124
FIGURE 5.3 INTER-STOREY DRIFT RATIOS FROM PUSHOVER ANALYSIS (7-R, FORCE STYLE-A)	125
FIGURE 5.4 A TYPICAL BEAM-COLUMN JOINT	127
FIGURE 5.5 DISTRIBUTION OF OVERSTRENGTH FACTORS (7-R, JOINTS, FORCE STYLE-A)....	127
FIGURE 5.6 EVALUATION OF STOREY STIFFNESS (7-R).....	129
FIGURE 5.7 DISTRIBUTION OF NORMALIZED STOREY STIFFNESS FACTORS (7-R, FORCE STYLE-A).....	130
FIGURE 5.8 DISTRIBUTION OF STOREY CAPACITY FACTORS (7-R, FORCE STYLE-A).....	131
FIGURE 5.9 EVALUATION OF REGULARITY INDEX AND PUSHOVER RESULTS (7-I-A, FORCE STYLE-A).....	133
FIGURE 5.10 EVALUATION OF REGULARITY INDEX AND PUSHOVER RESULTS (7-I-B, FORCE STYLE-B)	135
FIGURE 5.11 EVALUATION OF REGULARITY INDEX AND PUSHOVER RESULTS (12-R, FORCE STYLE-B)	137
FIGURE 5.12 EVALUATION OF REGULARITY INDEX AND PUSHOVER RESULTS (12-I-A, FORCE STYLE-B)	139

FIGURE 5.13 EVALUATION OF REGULARITY INDEX AND PUSHOVER RESULTS (12-I-B, FORCE STYLE-B)	141
FIGURE 5.14 HYSTERETIC LOOPS OF TYPICAL COLUMNS AND STOREYS (7-R, GROUND MOTION 5028 H-E07140)	143
FIGURE 5.15 FINAL STATES OF FRAME UNDER CERTAIN GROUND MOTIONS (7-R, $A_{95}=0.26G$)	144
FIGURE 5.16 DISPLACEMENT PROFILES AT TIME AROUND MAXIMUM ROOF DISPLACEMENT (7-R)	145
FIGURE 5.17 DISTRIBUTION OF MEAN MAXIMUM INTER-STOREY DRIFT RATIOS (7-R, $A_{95}=0.26G$)	147
FIGURE 5.18 SCATTER PLOT OF θ_{MAX} AND $\theta_{R,PEAK}$ (7-R, $A_{95}=0.26G$)	147
FIGURE 5.19 DISPLACEMENT RESPONSE OF FRAME 7-R ($A_{95}=0.20G$)	148
FIGURE 5.20 HYSTERETIC LOOPS OF TYPICAL COLUMNS AND STOREYS (7-I-A & 7-I-B, GROUND MOTION 955 H-E04140)	150
FIGURE 5.21 FINAL STATES OF FRAME UNDER CERTAIN GROUND MOTIONS (7-I-A AND 7-I-B, GROUND MOTION 955 H-E04140)	151
FIGURE 5.22 DISTRIBUTION OF MEAN MAXIMUM INTER-STOREY DRIFT RATIOS	152
FIGURE 5.23 SCATTER PLOTS OF 7-STOREY FRAMES	152
FIGURE 5.24 PLOTS OF HYSTERETIC RESPONSE OF TYPICAL COLUMNS AND STOREYS (12-R, GROUND MOTION 5028 H-E07140)	153
FIGURE 5.25 DISPLACEMENT PROFILES AT TIME AROUND MAXIMUM ROOF DISPLACEMENT (12-R)	154
FIGURE 5.26 DISPLACEMENT RESPONSE OF FRAME 12-R	155
FIGURE 5.27 DISTRIBUTION OF MEAN MAXIMUM INTER-STOREY DRIFT RATIOS (12-I-A & 12-I-B)	157
FIGURE 5.28 SCATTER PLOTS OF 12-STOREY FRAMES AND ALL 6 FRAMES	157
FIGURE 5.29 FLOWCHART OF ESTABLISHING PROBABILISTIC MODEL OF CRITICAL INTER-STOREY DRIFT	159
FIGURE 5.30 DISTRIBUTION OF STOREY CAPACITY FACTORS (7-CS, FORCE STYLE-A)	160
FIGURE 5.31 ELASTIC DISPLACEMENT SPECTRUM OF 30 SELECTED GROUND MOTIONS	163
FIGURE 5.32 FLOWCHART FOR CONSTRUCTION OF PERFORMANCE CURVES FOR FRAME	

STRUCTURES	164
FIGURE 5.33 FUZZY-RANDOM FRAGILITY CURVES FOR THREE PERFORMANCE LEVELS	165
FIGURE 5.34 COMPARISON OF FRAGILITY CURVES GENERATED USING DIFFERENT CRITERIA	166
FIGURE A.1 LOAD-DEFLECTION CURVES FOR INFILL MASONRY WALLS AND BARE & INFILLED FRAMES	206

LIST OF SYMBOLS

A_{95}	Acceleration that contains 95% of the Arias Intensity
A/V	Ratio of the peak ground acceleration to the peak ground velocity
A_{sx}	Total area of transverse bars running in the x directions
A_{sy}	Total area of transverse bars running in the y directions
b	Width of cross section
b_c	Width of the confined concrete core
C	Displacement modification factor
COV	Coefficient of variation
C_u	Cost at ultimate limit state
d	Effective depth of cross section
d_b	Maximum diameter of longitudinal bars
d_c	Depth of the confined concrete core
D	Modified column lateral stiffness
D_n	Normalized storey stiffness
$D_R(x)$	Damage function
D_s	Section depth
E_c	Modulus of elasticity of concrete
E_h	Cumulative dissipated hysteretic energy
E_s	Modulus of elasticity of reinforcement
f_c	Cylinder compressive strength of concrete
f_{cc}	Confined concrete strength
f_{ck}	Characteristic cylinder compressive strength
f_l	Effective confining stress in any one of the two orthogonal directions
f_{le}	Lateral confining pressure
$f_R(r)$	Probability density function of R
$f_S(r)$	Probability density function of S
f_y	Yield strength of reinforcement

f_{yw}	Yield strength of transverse reinforcement
f_u	Ultimate strength of reinforcement
$g(R,S)$	Limit state function
h	Storey height
H	Height from base to roof of structure
i_c	Column linear stiffness
i_{ns}	Normalized storey stiffness factor
$(i_{ns})_{min}$	Minimum value of the normalized storey stiffness factor
$(i_{ns})_{ave}$	Average value of the normalized storey stiffness factor
i_{OS}	Storey overstrength factor
$(i_{OS})_{min}$	Minimum value of storey overstrength factor
$(i_{OS})_{ave}$	Mean value of storey overstrength factor
i_w	Storey weakness factor
$(i_w)_{min}$	Minimum value of i_w
$(i_w)_{ave}$	Average value of i_w
$I(t)$	Deterministic amplitude modulation function
K	Strength factor related to confined concrete strength
K_0	Initial lateral stiffness of SDOF system
K_c	Elastic lateral stiffness of column
K_e	Secant stiffness
K_s	Factor related to the stiffness ratio of beams and columns
L_p	Equivalent plastic hinge length
$L_{p, PL}$	Plastic hinge length at a particular performance level
m_e	Effective mass
m_i	Lumped mass in i^{th} storey
M_{cur}	Current moment
M_{max}	Maximum yield moment
M_{new}	Current yield moment
M_s	Surface wave magnitude of earthquake
M_y	Yield moment

M_w	Moment magnitude of earthquake
N	Axial force
N_s	Number of stories
P_E	Probability of Exceeding
P_f	Probability of failure
P_n	Normalized storey shear
P_O	Probability of Occurrence
P_s	Probability of safety
R	General structural capacity
s	Hoop spacing
$S(\tilde{X})$	Support of a fuzzy set $\tilde{X}$
S	General structural demand
S_{ae}	Elastic spectral acceleration
S_d	Displacement spectrum ordinate
S_{de}	Elastic spectral displacement
S_v	Spectral velocity
T_e	Effective period
V/D	Ratio of the peak ground velocity to the peak ground displacement
V_B	Design base shear force
V_c	Column shear strength considering the possible premature yielding in beam ends
$V_{c,y}$	Yield strength of the column
V_R	Available storey shear strength
V_S	Design storey shear force
V_c^l	Shear strength of the column corresponding to yielding of beams belong to its lower joint
V_c^u	Shear strength of the column corresponding to yielding of beams belong to its upper joint
x_0	Initial damage state in cost function
$\tilde{X}$	A fuzzy set

$ \tilde{X} $	Cardinality of $\tilde{X}$
Z	Limit state function
Z_c	Length measuring from the fixed end to the contra flexure of the member
Z_g	Gradient of the descending branch of stress-strain curve for confined concrete
α	Effectiveness factor of confinement
α_n	Hoop pattern factor
α_r	Regularity index based on overstrength factor
α_s	Hoop spacing factor
α_{sd}	Parameter about stiffness degradation in hysteretic model
α_{ns}	Regularity index corresponding to storey stiffness distribution
α_w	Regularity index
β	Reliability index
β_1	Mode participation factor
β_2	Amplification factor accounts for the concentrations of inter-storey drifts
β_3	Nonlinear amplification factor from the maximum inelastic displacement to maximum elastic displacement
β_4	Factor accounts for the increase of concentration of inter-storey drift ratio due to nonlinear response
β_c	Drift concentration factor
β_d	Parameter about strength deterioration in hysteretic model depends on the ductility
β_e	Parameter about strength deterioration in hysteretic model depends on the hysteretic energy
χ_m	Model uncertainty factor
$\chi_{m,Lp}$	Model uncertainty factor of equivalent plastic hinge
$\chi_{m,\varepsilon_{cu}}$	Model uncertainty factor of ultimate confined concrete strain
Δ_d	Design displacement
Δ_e	Elastic displacement response of SDOF system

Δ_{ie}	Inelastic displacement response of SDOF system
Δ_{roof}	Top displacement
ε_{cc1}	Characteristic strains corresponding to the peak stress of the confined concrete
ε_{c0}	Characteristic strains corresponding to the peak stress of unconfined concrete
$\varepsilon_{c,85}$	Strain of unconfined concrete correspond to the stress of 0.85 f_c on the descending branch of the unconfined stress-strain curve
ε_{cu}	Ultimate confined concrete strain
$\varepsilon_{cu,exp}$	Experimental values of the ultimate strain of confined concrete
$\varepsilon_{cu,pred}$	Predicted value of ultimate strain of confined concrete using confinement models
ε_{sm}	60% of longitudinal reinforcement strain at the maximum stress
ε_{su}	Ultimate tension strain of reinforcement
ϕ_{cur}	Current curvature
ϕ_m	Maximum curvature obtained so far
ϕ_{PL}	Section curvature at certain performance level
ϕ_y	Yield curvature
ϕ_u	Ultimate curvature
$\varphi(t)$	Transformation of the time scale intended to modulate frequencies
Φ	Standard normal distribution function
Φ_i	Assumed displacement of i^{th} storey
Γ	Mode participation factor
γ	Index of building category
γ_p	Parameter about pinching behavior in hysteretic model
η	Normalizing constant of membership function
μ_{max}	Maximum storey displacement ductility
$\mu(x)$	Membership function of a fuzzy set
$\mu_O(\theta)$	Membership function of fuzzy event Occurrence
$\mu_E(\theta)$	Membership function of fuzzy event Exceeding

μ_R	Mean of R
μ_S	Mean of S
μ_Z	Mean of Z
ν	Axial force ratio
θ_0	Drift ratio at initial damage state
θ_1	Lower limit of the fuzzy interval of drift ratios
θ_2	Upper limit of the fuzzy interval of drift ratios
$\underline{\theta}$	Abscissa of the point with full membership function
θ_{ci}	Critical inter-storey drift ratio
θ_{max}	Maximum inter-storey drift ratio
$\theta_{r,peak}$	Peak roof drift ratio
ρ_l	Longitudinal reinforcement ratio
ρ_s	Volume ratio of transverse reinforcement
σ_R	Standard deviation of R
σ_S	Standard deviation of S
σ_Z	Standard deviation of Z
ω_w	Mechanical volumetric ratio of transverse reinforcement
$\xi(t)$	Ground acceleration as a function of time t
ξ_0	Initial damping ratio of SDOF system
$\zeta(\phi(t))$	Unit-variance filtered white noise

Chapter 1 Introduction

1.1 Background

1.1.1 Performance-based seismic design philosophy

In 1994, the Olive View Medical Center in Sylmar (California), destroyed during San Fernando earthquake in 1971 and replaced in 1988, was attacked by the Northridge earthquake (moment magnitude $M_w = 6.7$). The new facility survived in the ground motion without any significant structural damage. However, the hospital had to evacuate all of its patients to other facilities for several days because of breakage of both sprinkler and chilled water lines (EERI 1994). Similarly, several major hospitals were forced to curtail their services and evacuate patients after the Northridge earthquake. None of the hospitals sustained serious structural damage; however, they were forced to close due to non-structural problems. These examples show clearly that the current design philosophy satisfying a life safety performance objective for rare earthquake ground motions does not necessarily imply that the facility will be operational after frequent earthquake ground motions.

One of the major developments in seismic design over the past 10 years for concerns described above is the so-called performance-based engineering (PBE), coupled with performance-based seismic design (PBSD). As described by Bertero (2002), PBE can be regarded as a process that consists of the selection of design criteria and structural systems proportioning, detailing, and the assurance/control of construction quality and long-term maintenance, for the purpose that at specified levels of excitations with defined levels of reliability, the building or facility will not be damaged beyond certain limit states. On the other hand, PBSD is a process involving identification of seismic hazards, selection of the performance levels and design objectives, various stages of design (conceptual, preliminary, and final), acceptability checks, specification of quality assurance during the construction as well as during the life of the building.

Because of its intended coverage of various desired performance targets, the performance-based design philosophy is widely considered to be the core of the next generation of seismic design codes. However, the entire design approach is still far from mature to be applied in real engineering practice. One of the key elements of this design philosophy that deserves continued research efforts is the definition of the performance levels. Due to the important role of cost for repair and business interruption in the consideration of building performance, the non-structural damage should be appropriately considered.

The lateral displacement or drift is generally recognized as a parameter that closely correlates with the degree of damage, especially in RC framed structures. Over the past twenty years, quite a lot of research efforts have been devoted to approximately estimate the displacement demands on RC structures for the purpose of structural performance evaluation. Requiring nonlinear pushover analysis (Fajfar & Fischinger 1989) or solution of differential equations (Miranda 1999), these methods are not easy to apply in design applications. More convenient alternative methods are desired.

1.1.2 Reliability-based seismic performance assessment

Uncertainties exist in almost all aspects of structural engineering, as well as the seismic engineering. The source of uncertainties can be classified into loading, material properties, structure geometric properties, material modeling in analysis, human errors in design and construction, and so on. Subsequently, the structural behavior will show considerable uncertainties. For this reason, deterministic approaches are not competent for structural safety evaluation. Although allowance is made at the design stage for the variability in material strengths through the use of lower fractile values and the additional introduction of partial safety factors, these are not the only variables affecting the structural behavior. The current seismic design codes are based on semi-probabilistic methods. However, it is widely recognized that the use of characteristic force-reduction or ductility factor could result in non-uniform risk (or reliability) designs, since ductility is a poor indicator of the damage potential.

Even in the performance-based design framework proposed by SEAOC (1995) and NEHRP (1997a), the uniform-risk target is not reached. The suggested procedure is simply a bounding concept of design for multiple pairs of motion intensities and damage states. As pointed out by Hadjian (2001), such procedure does not provide any useful information regarding failure probabilities, and thus falls short of providing a quantifiable basis for developing earthquake risk management strategies.

Furthermore, the performance-based seismic design philosophy emphasizes the non-structural damage by which extra uncertainty is introduced. It is widely observed that the occurrence of non-structural damage is not synchronous with structural damage in a building system excited by certain ground motions. In other words, there could be considerable difference between the deformation capacities of non-structural and structural components at a same performance level. Subsequently, a single-valued threshold is not capable to represent a performance level and as a result, the classical reliability analysis methods cannot fulfill a synthesized reliability analysis concerning coherent building performance.

1.2 Research objectives

To develop towards more rational and executable methodology for performance-based seismic design and reliability analysis, this research aims to achieve the following objective:

- to review relevant research works on performance-based design philosophy, seismic reliability analysis, determination of seismic demands (in particular the displacement demands) and the potential application of the fuzzy set theory in structural reliability analysis.
- to establish quantified definition of seismic performance levels (in terms of limit state parameters) on a probabilistic basis for both structural and non-structural components.
- to set up a fuzzy-random reliability model which can take into account both the randomness and the fuzziness involved in the performance assessment of building systems.

- to develop a simple and effective procedure through which the probabilistic model of the maximum inter-storey drift demand in a framed structure can be established.

1.3 Thesis organization

The problems related to performance-oriented seismic reliability analysis and the research objectives of this study are outlined in Chapter 1. Chapter 2 presents a literature review of recent research works on performance-based design, issues related to structural reliability analysis, the estimation of seismic displacement demand and the application of fuzzy set theory on structural reliability analysis. Using Monte Carlo Simulation, the probabilistic drift ratio limits for three main performance levels of typical RC columns are investigated in Chapter 3. The classification and the displacement limits of non-structural components at various levels of damages (performances) are discussed at the beginning of Chapter 4. Next, a fuzzy-random reliability model, which can be used to perform a comprehensive reliability analysis of building performance, is developed. Chapter 5 deals with a framework for estimating the probabilistic critical inter-storey drift ratio in framed structures under seismic loading. An illustrative application of performance-oriented seismic reliability analysis concerning the critical storey is provided. Finally, the main conclusions of this study and recommendations for future research are summarized in Chapter 6.

Chapter 2 Literature Review

2.1 Introduction

An extensive literature review has been conducted to cover a broad range of issues that relate to this study. In this chapter, the philosophy of performance-based seismic design and direct displacement-based design are briefly introduced first. Previous research works on the uncertainties in the material properties, behavior modeling and the seismic input are summarized next, followed by the review of probabilistic studies on RC element capacities. The approximate methods to estimate the seismic displacement demand and the reliability analysis procedures for RC structural systems are then discussed. Finally, the fuzzy-set theory is introduced briefly. The fuzzy-set theory will be used to deal with the problem arisen from inconsistent performance limit states of structural and non-structural components in a structural system.

2.2 Performance-based seismic design

Design for earthquake resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from “strength” to “performance”. It is gradually realized that increasing strength may not enhance safety, nor necessarily reduce damage. In the 1970’s, researchers in New Zealand developed the capacity design principles (Park & Pauley 1975) based on the understanding that the distribution of strength through a building was more important than the absolute value of the design base shear. It was recognized that the seismic performance would be better for a frame building under seismic attack if it could be assured that plastic hinges would occur in beams rather than in columns, which is well known as “weak-beam strong-column mechanism”, and if the shear strength of members exceeded the shear demand corresponding to the flexural strength. This may be considered as the true start to performance-based seismic design (Priestley 2000).

although frame buildings designed in accordance with the weak-beam strong-column philosophy survived the earthquake without collapse, the cost of repairing the many locations of inelastic response, and hence localized damage, was often excessive, and costly. Other structural systems with fewer locations of inelastic response, such as structural wall buildings, were more economical in terms of repair costs. Further, Poland & Hom (1997) suggested that it is necessary to include the business interruption costs in the consideration of repair cost for the definition of the performance level.

Kawashima (1997) noted the need to consider residual deformation in defining the Damage Control performance level, also from the lesson of the 1995 Kobe earthquake. The 1996 Japanese seismic design provisions for bridges (JRA 1996) explicitly requires considering the residual deformation. In this regard, the construction of residual displacement spectra can be useful.

Another aspect worth noting is the cumulative damage model. Cosenza & Manfredi (1997) noted that for poorly designed existing structures, cumulative damage criteria are very important. However, it is of lesser importance for well-designed modern structures. The documents of CEB (1997) also included similar considerations.

2.3 Direct displacement-based design

In the displacement-based design procedure, an engineer performs seismic design by specifying a target displacement rather than a displacement limit for the design. Strength and stiffness are not variables in the procedure but the end results.

Until today, the prevailing seismic design codes still use force-based procedures, in which an additional check on displacement is usually required to ensure that acceptable serviceability is achieved under moderate earthquakes. This is a perfectly acceptable design approach as far as the safety of the structures is concerned; however, it does not directly incorporate any performance criteria in the primary stage. If multiple performance targets are to be satisfied in this procedure,

it would require significantly increased design effort, since several iterations and adjustments could be needed. To tackle the problem, a new design procedure named “direct displacement-based design” has been proposed (Moehle 1992, Priestley 1992, Priestly & Calvi 1997 and Priestley & Kowalsky 2000).

The force-based design procedure characterizes a structure in terms of elastic properties, such as stiffness and damping, right at first yield. Differently, direct displacement-based design characterizes a structure by the secant stiffness K_e at maximum displacement Δ_d (refer to Figure 2.2b) and an equivalent viscous damping corresponding to the hysteretic energy absorbed during inelastic response (which can be very dependent on the detailing). Thus, as shown in Figure 2.2c, a steel structure with compact members will be assigned a higher level of equivalent viscous damping than a reinforced concrete structural wall building for a given level of ductility demand.

Figure 2.2 Fundamentals of direct displacement-based design (Priestley 2000)

Then, from a set of design displacement spectra (Figure 2.2d), the effective period T_e at maximum displacement response can be read, when the design displacement Δ_d and the damping are determined from the expected ductility demand. Applying a SDOF model shown in Figure 2.2a, the effective stiffness K_e at maximum response displacement can be obtained by

$$K_e = 4\pi^2 m_e / T_e^2 \quad (2.1)$$

where m_e is the effective mass. From Figure 2.2b, the design base shear force at maximum response is thus:

$$V_B = K_e \Delta_d \quad (2.2)$$

It should be noted that the above simple procedure is illustrative and is directly applied only for single performance target cases. For multi-target performance design, more sophisticated procedure may be derived on the above basis to enable the determination of multiple design parameters.

2.4 Uncertainties involved in the reliability analysis

Uncertainties exist in almost all areas of civil and structural engineering and rational design decisions cannot be made without modeling them and taking them into consideration. Structural reliability analysis is concerned with the rational treatment of uncertainties in structural engineering design and the associated problems of rational decision-making. In this section, some existing research work on various uncertainties and probabilistic assessment of RC member performance will be reviewed briefly.

2.4.1 Basic reliability model

Rigorous definition of a limit state is necessary to describe the working condition of a structure in structural reliability analysis. In general, the limit state can be expressed as the so-called limit state function

$$Z = g(R, S) = R - S \begin{cases} < 0 & \text{failure state} \\ = 0 & \text{limit state} \\ > 0 & \text{safe state} \end{cases} \quad (2.3)$$

where R denotes the performance level related to the supply or capacity of structural behavior, e.g. sectional resistance, inter-story drift capacity; while S denotes the demand of structural response, e.g. moment effect, lateral displacement, under a given loading condition. The failure probability of the structure with respect to a particular performance limit state can be derived from

$$P_f = P(Z < 0) = P((R - S) < 0) = \int_0^{+\infty} \left[\int_0^s f_R(r) dr \right] f_S(s) ds \quad (2.4)$$

or

$$P_f = P(Z < 0) = P((R - S) < 0) = \int_0^{+\infty} \left[\int_r^{+\infty} f_S(s) ds \right] f_R(r) dr \quad (2.5)$$

where P_f is the failure probability; $f_R(r)$ is the probability density function (PDF) of supply, R , and $f_S(s)$ is the PDF of the demand, S . Normally, R is a function of basic material properties, section geometry and other related structural properties. Therefore, to establish the probabilistic property of the supply R , it is necessary to understand the statistical characteristics of these basic variables.

For a fundamental case of structural reliability analysis, where both R and S are assumed as independent normally distributed variables, the limit state function Z is also normally distributed and $(Z - \mu_Z)/\sigma_Z$ is unit standard normal. Therefore, the failure probability can be obtained by

$$P_f = P(Z < 0) = \Phi\left(\frac{0 - \mu_Z}{\sigma_Z}\right) = \Phi\left(\frac{\mu_S - \mu_R}{(\sigma_S^2 + \sigma_R^2)^{\frac{1}{2}}}\right) \quad (2.6)$$

where Φ is the standard normal distribution function; μ_S & σ_S are the mean and standard deviation of S ; μ_R & σ_R are the mean and standard deviation of R .

The reliability index β may now be defined as the ratio of μ_Z/σ_Z or the number of standard deviations by which μ_Z exceeds zero, where μ_Z and σ_Z are the mean and standard deviation of the limit state function Z . Figure 2.3 graphically shows the meaning of β . Based on Equation 2.6 and Figure 2.3, the relationship between P_f and β can be expressed as follows:

$$P_f = \Phi\left(-\frac{\mu_z}{\sigma_z}\right) = \Phi(-\beta) \quad (2.7)$$

Figure 2.3 Illustration of the reliability index β

2.4.2 Categorization of uncertainties

There exist three major types of uncertainties, namely physical uncertainty, statistical uncertainty and model uncertainty.

- Physical uncertainty: Variability exists in all physical quantities related to structural engineering, such as loads, material properties and dimensions. This variability can be described in terms of probability distributions or stochastic processes.
- Statistical uncertainty: Physical variability can be quantified only by examining sample data. However, the sample size is always limited by practical and economical constraints. The statistical uncertainty, which depends on the amount of sample data and prior knowledge about pertinent probability distributions, arises as a result of lack of information.
- Model uncertainty: Structural design and analysis make use of mathematical models to obtain output quantities from a set of input quantities. With very few exceptions, it is almost impossible to make highly accurate predictions about the magnitude of the response of typical civil engineering structures

to loading even the governing input quantities are accurate enough. This additional source of uncertainty is termed as model uncertainty and occurs as a result of simplifying assumptions, unknown boundary conditions and as a result of the unknown effects of other variables and their interactions that are not included in the model. The model uncertainty associated with a particular mathematical model may be expressed in terms of the probabilistic distribution of a variable χ_m defined as

$$\chi_m = \frac{\text{actual response}}{\text{predicted response using model}} \quad (2.8)$$

To quantify uncertainties in the field of structural engineering and for subsequent reliability analysis, it is necessary to define a set of basic variables governing the static or dynamic response of the structure. It is well recognized that there are some uncertain variations in the material properties and section geometry, such as the compressive strength of concrete and the yield strength of reinforcing bars, which cannot be neglected in the engineering practice. They are basic in the sense that they are the most fundamental quantities normally recognized and used by designers and analysts in structural calculations and design.

These uncertainties may come from the imperfection of natural materials, production technology, degree of quality control and so on. The mechanical behavior of an RC member is affected by these variations more or less. Extensive research work has been done to investigate the uncertainty of the basic RC material properties and section geometry in the last several decades. Most of these investigations have been based on the database of industrial survey and laboratory tests. As a result of these previous works, most of the uncertainties can be described in terms of probability distributions. For example, Mirza et al. (1979) suggested that the cylinder compressive strength of concrete could be modeled as normal distribution or log-normal distribution corresponding to average and poor quality control levels, respectively.

2.4.3 Uncertainties in material property

In the following paragraphs, the uncertainties of the basic RC material properties and section geometry will be reviewed briefly. In recognition of the governing role of the ultimate compressive strain of confined concrete, ε_{cu} , in predicting the section deformation behavior of RC members, some existing models of confined concrete will be introduced and evaluated in the probabilistic context, taking into account the model uncertainty.

Cylinder compressive strength (f_c) and modulus of elasticity of concrete (E_c)

The major sources of variations in concrete strength are the variations in material properties and proportions of the concrete mix; the variations in mixing, transporting, placing and curing methods; the variations in testing procedures; and the variations due to concrete being in a structure rather than in control specimens.

According to the research work of Mirza et al. (1979), the COV (coefficient of variation) of compressive strength of concrete can be taken as roughly constant at 10%, 15%, and 20% for strength level below 4000psi (27.6MPa) for excellent, average, and poor quality control, respectively. For concrete with an average strength above 4000psi, the standard deviation remains approximately constant with values of 400psi (2.76MPa), 600psi (4.14MPa), and 800psi (5.52MPa) for the three levels of control listed previously. Mirza et al. (1979) also concluded based on extensive literature review that it is reasonable to assume a normal distribution for the compression strength of concrete, especially for concrete that has an average or higher control level.

The modulus of elasticity under consideration is the initial tangent modulus, which directly correlates to the cylinder compressive strength. The following relationship for the initial tangent modulus was obtained from Mirza et al. (1979) for normal weight concrete:

$$E_c = 60400 f_c^{1/2} \quad (\text{in psi}) \quad (2.9)$$

Ultimate compressive strain of unconfined and confined concrete

Typical stress-strain curves of unconfined and confined concrete are shown in Figure 2.4. ε_{c0} is the strain of unconfined concrete corresponding to the peak compressive stress f_c . ε_{cu} is the ultimate strain of confined concrete. The maximum strain $\varepsilon_{c,85}$ of unconfined concrete is defined to correspond to the stress of $0.85f_c$ on the descending branch of the unconfined stress-strain curve. $\varepsilon_{c,85}$ should be taken as a probabilistic distributed variable to account for its variability. According to Trezos (1997), the variation of $\varepsilon_{c,85}$, for normal weight concrete, is assumed to follow a normal distribution with the mean value equal to 0.0035 and the standard deviation equal to 0.0005.

Figure 2.4 Stress-strain behavior of confined and unconfined concrete

In practice, concrete may be confined by transverse reinforcement. At low levels of stress in the concrete, the transverse reinforcement is hardly stressed; hence the concrete is unconfined. The concrete becomes confined when at stresses approaching the uniaxial strength, at which stage the transverse strain becomes very high because of progressive internal cracking and the concrete bears out against the transverse reinforcement, which then applies a confining reaction to the concrete. Thus the transverse reinforcement provides passive confinement. Available test data also show that confinement by transverse reinforcement has little effect on the stress-strain curve until the uniaxial strength of the concrete is approached.

There are quite a lot of confinement models proposed by other researchers. Some predominant models will be reviewed in the later part about the model uncertainty.

Yield strength of reinforcement (f_y) and ultimate strength (f_u)

The sources of variation in steel yield strength include the variation in the strength of material itself, variation in the cross section area of the bar, the effect of loading rate, the effect of bar diameter and the effect of the strain at which yield is defined.

Mirza & Mac Gregor (1979a) suggested that Beta distribution was a good description for the statistical property of the steel yield strength. For Grade 40 (ACI code) reinforcing bars, the recommended value of mean and COV are 337MPa and 10.7% respectively, while for Grade 60 reinforcing bars, the mean and COV are 490MPa and 9.3%, respectively.

The sources of variations of ultimate strength of reinforcing bars are the same as those causing variations of the yield strength. Similar to the yield strength, the ultimate strength of reinforcing bars is also assumed to follow a Beta distribution (Mirza & Mac Gregor 1979a)

Modulus of elasticity of reinforcement (E_s)

The modulus of elasticity of the reinforcing bars is assumed to follow normal distribution with a mean value of 201,000Mpa and a COV of 3.3% according to Mirza & Mac Gregor (1979a).

Ultimate tension strain (ϵ_{su})

According to Kappos et al. (1991), the characteristic value for the ultimate strain of reinforcement is 8% at a 10% fractile and the COV is 9%. By assuming a normal distribution, a mean and standard deviation equal to 9% and 0.81% are obtained respectively.

Section geometry

Geometric imperfections in RC elements are caused by deviations from the specified values of the cross sectional shape and dimensions; the position of

reinforcing bars, ties and stirrups; the horizontality and verticality of concrete lines; the alignment of columns and beams, and so on. Among all the factors, the uncertainty of the section size is the most important one. Based on Mirza & Mac Gregor (1979b), for a rectangular section under consideration, the height and width of the cross section can be assumed to follow normal distribution with a mean value equal to the nominal value plus 1.6mm and a constant standard deviation equal to 6.4mm to represent the in-situ cast condition.

2.4.4 Uncertainty in modeling of confined concrete

The behavior of an RC cross section mainly depends on the material properties, the axial force and the confinement (quantity and pattern). Among these factors, the confinement effect plays a crucial role in determining the sectional behavior with regard to the deformation and ductility; yet it is most difficult to quantify. A number of experimental programs have been conducted to investigate the stress-strain behavior of confined concrete, leading to several proposed stress-strain laws. As these stress-strain laws differ from each other, the adoption of different analytical models for the confined concrete will result in different predictions. Thus, the issue with model uncertainty arises. Since the variation of the basic material properties is represented in the mechanical model relating the confined concrete behavior with the material parameters, it is reasonable to assume that the model uncertainty factor imposed on top of a mechanical model is independent of different test specimen details. Therefore, the experimental data from a variety of test programmes can be analyzed in a uniform manner to arrive at a representative and unbiased result. This section will examine the model uncertainties for confined concrete with regard to the RC sectional behavior.

As shown in Figure 2.4, the behavior of confined concrete may be characterized by the strength and the ultimate strain ε_{cu} . The confined concrete strength may be expressed as (Park & Priestley 1992)

$$f_{cc} = Kf_c \quad (2.10)$$

where K denotes the strength factor, obviously $K > 1$. The proposed values for K differ among the existing models. However, the uncertainty of K factor is much smaller as

compared to that of the post-peak behavior represented by the confined concrete strain ε_{cu} .

In a majority of RC section cases with adequate reinforcement, ε_{cu} governs the sectional ductility and hence its value constitutes a primary failure criterion. The value of ε_{cu} applied herein is corresponding to the point where σ_c drops to 85% of the unconfined concrete strength, f_c , in the descending branch of the stress-strain curve. Seven different models for confined concrete concerning ε_{cu} are examined in what follows.

Various confinement models

Tassios' model (EC8 1989)

$$\varepsilon_{cc1} = \varepsilon_{c0} K^2 \quad (2.11)$$

where ε_{cc1} and ε_{c0} are the characteristic strains corresponding to the peak stress of the confined and unconfined concrete, respectively. K is the strength factor. At the ultimate state, as described previously,

$$\varepsilon_{cu} = \varepsilon_{c,85} + 0.1\alpha\omega_w \quad (2.12)$$

where α is the factor representing the effectiveness of confinement, which is dependent on the hoop pattern and hoop spacing. ω_w is the mechanical volumetric ratio of transverse reinforcement, $\omega_w = \rho_w f_{yw} / f_c$, with ρ_w being the volumetric ratio of transverse reinforcement and f_{yw} , f_c the yield strength of transverse reinforcement and concrete, respectively.

Modified Kent & Park's model (Scott et al. 1982)

Scott et al. (1982) proposed a stress-strain curve for confined concrete as shown in Figure 2.4. This model is the modified version of a previous model by Kent & Park (1971). For low strain rate, ε_{cc1} can be well described by

$$\varepsilon_{cc1} = 0.002K \quad (2.13)$$

$$K = 1 + \frac{\rho_w f_{yw}}{f_c} \quad (2.14)$$

where ρ_w and f_{yw} are the volumetric ratio and the yield strength of the transverse reinforcement, respectively. f_c is the cylinder compressive strength of concrete.

The gradient of the descending branch is given by

$$Z_g = \frac{0.5}{\frac{3 + 0.29f_c}{145f_c - 1000} + 0.75\rho_w \sqrt{\frac{b_c}{s}} - \epsilon_{cc1}} \quad (2.15)$$

where b_c and s are the width of the confined core and spacing of the hoops, respectively. f_c is given in MPa.

Sheikh & Uzumeri's model (Sheikh & Uzumeri 1982)

As shown in Figure 2.5, this model differs from the more common shape shown in Figure 2.4. One of the main differences is that the ultimate strain, ϵ_{s85} , is defined as the strain at which the concrete stress decreases to 85% of f_{cc} not f_c , in the descending branch. A plateau was introduced at the maximum stress level instead of a single peak. The strain at which the peak stress is first reached is expressed as

$$\epsilon_{s1} = 80K_s f_c \times 10^{-6} \quad (2.16)$$

where f_c is in megapascals; the strength factor $K_s = f_{cc} / (0.85f_c)$, as well as the strain at the onset of the descending branch, ϵ_{s2} , is calculated from a set of parameters including the dimensions of the confined core, hoop spacing and the number of longitudinal bars.

Figure 2.5 Confined concrete stress-strain curve by Sheikh & Uzumeri (1982)

The gradient of the descending branch is given as

$$Z_g = \frac{0.5}{0.75\rho_w \sqrt{\frac{b_c}{s}}} \quad (2.17)$$

Paulay & Priestley's model (Mander et al. 1988)

The ultimate strain proposed by Paulay & Priestley (1992) takes the following form

$$\varepsilon_{cu} = 0.004 + 1.4\rho_w \frac{f_{yw}\varepsilon_{su}}{f_{cc}} \quad (2.18)$$

where ρ_w is the volumetric ratio of transverse reinforcement ratio, f_{yw} denotes the yield strength of transverse reinforcement, f_{cc} denotes the compressive strength of confined concrete and ε_{su} is the strain of transverse reinforcement corresponding to maximum stress.

The strength factor K proposed by Mander et al. (1988) is applied in this model,

$$K = -1.254 + 2.254 \sqrt{\left(1 + \frac{7.94f_l}{f_c}\right)} - \frac{2f_l}{f_c} \quad (2.19)$$

where f_l is the effective confining stress in any one of the two orthogonal directions. For rectangular transverse hoops, f_l can be expressed as

$$f_{lx} = \frac{A_{sx}}{sd_c} f_{yw} = \rho_x f_{yw} \quad (2.20)$$

and

$$f_{ly} = \frac{A_{sy}}{sb_c} f_{yw} = \rho_y f_{yw} \quad (2.21)$$

where A_{sx} and A_{sy} are the total area of transverse bars running in the x and y directions, respectively; b_c and d_c are the core dimensions measured between centerlines of the perimeter hoop in x and y directions, respectively; s is the center to center spacing or pitch of spiral or circular hoop; f_{yw} is the yield strength of the transverse reinforcement.

The strain corresponding to the peak stress f_{cc} is given by

$$\varepsilon_{cc1} = 0.002[1 + 5(K - 1)] \quad (2.22)$$

Kappos' model (Kappos 1991)

This model was established on the basis of a number of specimens tested in various investigations. The strength factor is given as

$$K = 1 + a\omega_w^b \quad (2.23)$$

where a and b are pre-determined constants depending on the hoop pattern. Equation 2.11 was adopted in this model for the strain at the peak stress. The slope of the descending branch is obtained by modifying Equation 2.15 so that f_c is substituted by f_c / K .

Saatcioglu & Razvi's model (Saatcioglu & Razvi 1992)

In this model, the peak confined concrete strength is predicted by

$$f_{cc} = f_c + 6.7 f_{le}^{0.83} \quad (2.24)$$

where f_{le} represents the lateral confining pressure. The strain corresponding to peak stress follows Equation 2.22 while the ultimate strain at $0.85 f_c$ is given by

$$\varepsilon_{cu} = 260\rho_l + 0.0038 \quad (2.25)$$

where ρ_l is longitudinal reinforcement ratio.

Hoshikuma's model (Hoshikuma et al. 1997)

Hoshikuma et al. (1997) proposed this model after investigating a number of previous models including those described above. For square sections the strength factor is calculated by

$$K = 1 + 0.73 \frac{\rho_w f_{yw}}{f_c} \quad (2.26)$$

and the strain at peak confined stress

$$\varepsilon_{cc1} = 0.00245 + 0.0122 \frac{\rho_w f_{yw}}{f_c} \quad (2.27)$$

The gradient of the descending branch of the stress-strain curve is given as

$$Z_g = \frac{11.2f_c^2}{\rho_w f_{yw}} \quad (2.28)$$

which is applicable for both square and circular sections.

Model uncertainty in the proposed constitutive models for confined concrete

Dymiotis (1999) investigated several prevailing models for the confined concrete stress-strain behavior and provided a probabilistic evaluation of the respective model uncertainty. Based on the results from various experimental programmes (Vallenas et al. 1977, Sheikh & Uzumeri 1980, Scott et al. 1982, Moehle & Cavanagh 1985, and Hoshikuma et al. 1997), 56 experimental values of the ultimate strain of confined concrete, $\varepsilon_{cu,exp}$, were collected as the actual values, while their analytically predicted values, $\varepsilon_{cu,pred}$, were obtained using several existing models. Subsequently, the 56 ratios of $\chi_m = \varepsilon_{cu,exp}/\varepsilon_{cu,pred}$ for each model were computed and their statistical properties were analyzed. Table 2.1 lists the evaluation results regarding the model uncertainty for the seven analytical confinement models.

Table 2.1 Scatter in $\varepsilon_{cu,exp}/\varepsilon_{cu,pred}$ (Dymiotis 1999)

Statistic	Confinement model						
	Tassios	MK&P	S&U	P&P	Kappos	S & R	HOS
Mean	1.492	1.14	1.074	1.643	1.061	1.563	2.226
COV (%)	47.3	38.5	47.0	81.4	39.0	44.0	45.9

Note: MK&P=Modified Kent & Park model, S&U=Sheikh & Uzumeri model, P&P=Paulay & Priestley model, S & R=Saatcioglu & Razvi's model, HOS=Hoshikuma et al. model.

In summary, several prevailing confinement models are briefly introduced in this section as well as a previous study on the corresponding model uncertainty. It is found that results obtained from these analytical confinement models show significant scatter around the experimental data. Therefore, if any of these confinement models is applied to predict the ultimate strain of confined concrete in a probabilistic context, the corresponding model uncertainty factor must be taken into account.

2.4.5 Modeling of uncertainty in seismic motions

The seismic motions are usually expressed in the form of different types of waves, usually as acceleration time-history records. Integration of the records enables velocities and displacements to be estimated. Earthquake loading on a structure is the consequence of the accelerograms passing by the structure's base. In a deterministic context, the earthquake loading is dependent on the ground motion's frequency contents and the structural properties as well. However, comparing to the structural properties, the seismic ground motions exhibit much more uncertainties. Therefore, the uncertainty involved in the seismic ground motion governs the uncertainty of earthquake loading and will be discussed briefly in the following paragraphs.

The uncertainty in seismic motions has been a subject of extensive study among the seismic reliability research community. Previous researches have shown that there are large variations existed in peak ground motion parameters, frequency contents, durations and so on. The processing of raw ground motions, e.g. filtering and scaling, may introduce additional uncertainties. Some researchers such as Tzavelis (1988), Arede & Pinto (1996), even assumed seismic motions the only source of uncertainty in their studies. Research efforts based on both real and artificial seismic accelerograms can be found in the relevant literature.

For real earthquake records, the criteria for selecting the accelerograms to be used and the number of accelerograms that is sufficient to represent the variability in seismic motion need to be determined first. For example, O'Connor & Ellingwood (1987) used 20 real records from different US earthquakes corresponding to different magnitudes, distances from fault and peak ground accelerations (PGA). Bazzurro & Cornell (1994) considered 43 real accelerograms involving a range of tectonic environments and seismic parameters and concluded that there is no significant functional dependence of average structural damage on magnitude and epicenter distance. Therefore, 5-7 recorded accelerograms were considered to be sufficient to account for the uncertainty of seismic motion. Cornell (1996a, b) also suggested that 4-5 real or artificial earthquake records would be enough for the calculation of average structural response.

The process of scaling the records was included in most of these studies in order to represent specific levels of seismic intensity. Chavez & De Leon (1984) suggested a method for randomizing real records in the scaling process. The proposed method includes the selection of 8 accelerograms corresponding to different PGAs and durations, and the definition of a Poisson distributed spectral velocity (S_v) at a deterministic fundamental structural period and for a given number of years (i.e. the return period). Randomly selecting one of the eight records and scaling this to achieve the randomly simulated values of S_v would finally obtain the population of records used.

Random artificial accelerograms are generated on the base of a number of assumptions. It may require more effort than that associated with the identification and scaling of real records. Research efforts to generate motions that comply with the code-specified response spectrum were usually carried out for the purpose of code calibration. Assuming a Gaussian stochastic process, Costa & Duarte (1990) obtained a sample of motions by which the average spectrum given fitted the design one. Four artificial motions were used by Arede & Pinto (1996) to fit the EC8 spectrum. All of the four artificial motions had a 10-second duration. Furthermore, Colangelo et al. (1996) obtained a sample of artificial motions based on a white-noise process assumption so that the resulted power spectra approximately correspond to the EC8 elastic response spectrum. Esteva et al. (2000) considered an earthquake accelerogram as the realization of a gaussian non-stationary stochastic process modulated in amplitude and frequency: $\xi(t) = I(t)\zeta(\varphi(t))$ (2.29)

where $\xi(t)$ is the ground acceleration as a function of time t ; $I(t)$ is a deterministic amplitude modulation function; $\varphi(t)$ is a transformation of the time scale, intended to modulate frequencies; and $\zeta(\varphi(t))$ is a unit-variance filtered white noise, stationary with respect to φ . The statistical properties of the model are determined by the forms and parameters of the amplitude and frequency modulation function.

Some of the artificial accelerograms were obtained on the basis of the real ones in several studies. In the study of Asano (1988), the root-mean square amplitude,

predominant angular frequency and shaping factor of spectral characteristics of real motions were considered so that the artificial accelerograms resembled these. Similarly, based on the characteristic of the El Centro and Mexico City motions, Esteva & Ruiz (1989) generated groups of 20 and 9 records, respectively.

Wen (1995) favored the use of artificial records generated in order to comply with the statistics of magnitude and epicentral distance of the real ones after considering a set of available real records from a specific region, and conforming to criteria such as $M_s \geq 5.5$ (M_s denotes the surface wave earthquake magnitude). Additionally, a log-normal model uncertainty parameter with a mean of 1.3 and a COV of 81% was introduced for the response spectra.

Artificial records have been found to lead to unrealistic outcomes. Naiem & Lew (1995) found that spectrum compatible artificial accelerograms, which are related to non-representative velocities, displacements and energy contents, results in unrealistic demands. Therefore, the use of real records is preferable. Anyway, it is necessary to be careful to ensure that the selection criteria lead to an effective representation of the uncertainty in the ground motion.

2.5 Probabilistic assessment of RC member capacity

2.5.1 Strength capacity

Numerous research works can be found concerning the probabilistic property of RC beams and columns concerning their strength. In these studies, essential material and geometrical parameters were considered as random variables. By considering the equilibrium of forces and compatibility of strains within the member, Ellingwood (1977) investigated the variability of points along the moment-axial load (M-N) interaction diagram. The values of concrete and steel strength, the steel area, cross-sectional dimensions and the ultimate concrete strain were all randomly sampled by Monte Carlo Simulation with the sample size equal to 100. Then, these random values were used as input data to the previously derived equations. A COV

of 17% was obtained for the column strength by further considering model uncertainty and errors in engineering judgment.

Grant et al. (1978) carried out a similar study as Ellingwood (1977). The Monte Carlo Simulation method was used with a larger sample size of 250. More variables were involved in this work including concrete compressive and rupture strength and elastic modulus, the steel yield strength and elastic modulus, as well as “geometric imperfection” and model uncertainty factors, based on a series research works on the probability properties of these variables. It was found that the results were only a little affected by the variability in column section size, steel strength and type of construction. The concrete strength and the steel ratio were found to have significant influences on the resulted distribution properties of the strength ratios.

The variability of the concrete strength was found to be the major contributing factor to the variability of cross-section strength in the compressive failure region, and the variability in the steel strength seems to be the major contributing factor to the variability of cross-section strength in the tension failure region.

Some researchers also applied First or Second Order Reliability Methods (FORM or SORM, respectively) rather than simulation methods for this purpose. Vismann & Zilch (1995) investigated the strength of RC beams subjected to static loads using FORM. The concrete compressive strength, the steel area and yield strength, section depth and width, the concrete cover, and dead and live loads were included as random variables in the study. Hong et al (1999) also carried out their research work on the strength reliability evaluation of RC columns based on FORM. The random variables considered were similar to the ones considered by Vismann & Zilch (1995). The resulted reliability indices were found to reasonably agree with those given by Monte Carlo Simulation.

In spite of significant differences in methodology applied, it is generally established that concrete properties govern the response above the balance point in the M-N interaction diagram, while below this point the steel properties appear to dominate.

2.5.2 Deformation capacity

The deformation capacity of an RC member can be measured by its ductility. In modern earthquake resistant design, the significance of structural ductility has been generally recognized as equally important as strength due to its close relationship with the energy dissipation capacity. However, much less research can be found in the literature concerning the member ductility than that concerning member strength, especially with regard to its probabilistic characteristics.

The ductility factor involves two limit states, yield and ultimate. For the latter, the member behavior is no longer elastic and the deformation capacity is affected by a large number of variables some of which involve great uncertainty. Trezos (1997) considered the material and geometry property as random variables in his study of curvature ductility of confined RC columns, based on simplified analytical models for the sectional response. The criteria used for ultimate limit state was based on the ultimate confined concrete strain corresponding to $0.85f_c$ (f_c being the unconfined concrete cylinder compressive strength) along the descending branch of the confined concrete stress-strain curve, following Tassios' model (EC8 1989). Random variables included the concrete strength, ultimate strain of unconfined concrete, yield and ultimate steel strengths, and concrete and steel areas. All these variables were assumed to follow normal distribution, while the confinement effectiveness coefficient and the mechanical ratio of transverse reinforcement were treated as either log-normal or normal variables. A sample size of 500 was applied for the yield curvature simulation. It was found that the yield curvature basically followed normal distribution for three axial force levels with very small COV. For the ultimate curvature, the sample size applied was increased to 1000 as more variables were involved in this stage response. The COV of the ultimate curvatures was found much larger than that for the yield curvature. Finally, the ductility factor, defined as the ratio of ultimate curvature and yield curvature, was found to follow a log-normal distribution. It must be pointed out that the ultimate limit state here was established based on the Tassios' model for the confined concrete strain and no model uncertainty factor were considered. This could affect the overall probabilistic description of the ductility factor.

Kappos et al. (1999) also carried out a similar study but using a different approach, in which the Response Surface Methodology was used to derive an approximate “limit state function” incorporating the random variables. The concrete strength, yield and ultimate strength of steel and ultimate strain of steel were considered as random variables. In addition, a model uncertainty factor for the ultimate strain of confined concrete was introduced to account for the uncertainties arising from the various models of confined concrete.

According to Equation 2.8, several prevailing models of confined concrete were examined in a probabilistic basis and their COV were obtained. The COV of the proposed model uncertainty factor $\chi_{m,con}$ was then incorporated into the COV of the simulation results without considering the model uncertainty by the following expression:

$$COV_{overall} = \sqrt{COV_{siml}^2 + COV_{\chi_{m,con}}^2} \quad (2.30)$$

where $COV_{overall}$ is the final coefficient of variations for the sectional behavior; COV_{siml} is that of the section analyzed without considering the model uncertainty; while, $COV_{\chi_{m,con}}$ is the coefficient of variations of model uncertainty factor χ_m , for confined concrete.

Obviously, the model uncertainty factor $\chi_{m,con}$ has a significant effect on the variation of inelastic sectional response because of the important role that the ultimate confined concrete strain plays in the inelastic sectional behavior. However, the contribution of the model uncertainty for confined concrete to the uncertainty of the overall sectional behavior may not be so straightforward as implied by Equation 2.30. A better way to account for the contribution of the model uncertainty is to involve the model uncertainty factor in simulation process the same way as that for other random variables such as the material and geometry properties. This approach is adopted in the present study as will be described in Chapter 3.

As a matter of fact, simulation of the section behavior based on the approximated limit state functions inevitably introduces some additional error in the results as a cost for the computational efficiency. Direct sectional analysis based on refined physical models is expected to avoid such additional error and hence increase the accuracy of result. This will also be considered in the present study.

2.6 Estimation of displacement demand for RC structures

Performance evaluation of structures requires the information of global (e.g. roof), local (e.g. storey), and elemental (e.g. columns and beams) deformation demands. Nonlinear time history analysis of a detailed analytical model subjected to a suit of representative site-specific ground motions, using a well calibrated analysis tool, is likely the best option to estimate these demands. However, the excessive complexity involved in the detailed structural model and ground motion selection make the time history analysis not feasible for design application and rapid performance assessment. Therefore, quite a lot of research work has been done to approximately estimate the displacement demand of RC structures. In the following paragraphs, some representative studies are reviewed briefly.

2.6.1 Fajfar & Fischinger (1989) (N2 method)

Fajfar & Fischinger (1989) proposed an approximate displacement demand estimating approach named as N2 method. The N2 method includes the application of capacity spectrum and displacement coefficients. Similar approaches are also suggested by ATC 40 (ATC 1996) and FEMA 273 (NEHRP 1997a).

All these methods incorporate the use of nonlinear pushover analysis and the response spectrum. These methods generally consist of the following steps:

1. Determine the base shear V_B versus top displacement Δ_{roof} relationship by pushover analysis with assumed vertical distribution of the lateral forces.
2. Convert the pushover curve to a capacity diagram. In this step, the Multi-Degree-Of-Freedom (MDOF) structural system is transformed to an equivalent Single-Degree-Of-Freedom (SDOF) system.

3. Determine the elastic response (or design) spectrum in Acceleration-Displacement form. The original elastic response spectrum ($S_{ae} - T$) is converted to elastic demand diagram ($S_{ae} - S_{de}$). Furthermore, the inelastic demand diagram for the specific performance level can be generated through strength reduction factors.
4. Plot the capacity diagram and demand diagram together and determine the displacement demand for the equivalent SDOF and convert the displacement demand to the global (top) displacement demand for MDOF structural system.
5. Perform the pushover analysis of the MDOF model up to the top displacement demand to determine the corresponding local quantities such as story drift and end rotations of components.
6. Compare the local and global demands with the capacities for the specified performance levels.

The difference among these methods exists in the lateral load pattern of the pushover analysis, transformation of MDOF system to SDOF system, simplification of the pushover curve and the generation of the inelastic response spectra. More detailed discussion on these methods can be found in some previous studies (Chopra & Goel 2000, Fajfar 1999, Fajfar 2000, Whittaker et al. 1998).

2.6.2 Miranda (1999)

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. In case of elastic response, the roof drift and the maximum inter-storey drift ratio can be expressed as

$$A_{roof} = \beta_1 S_d \quad (2.31)$$

$$\theta_{\max} = \beta_2 \frac{\Delta_{\text{roof}}}{H} \quad (2.32)$$

where S_d is the displacement spectrum ordinate; β_i is the mode participation factor which is denoted by Γ in the N2 method; β_2 is the amplification factor accounts for the concentration of inter-storey drifts. When the structure responses into a nonlinear range, the nonlinear roof drift ratio and maximum inter-storey drift ratio can be obtained by

$$\Delta_{\text{roof}} = \beta_1 \beta_3 S_d \quad (2.33)$$

$$\theta_{\max} = \beta_2 \beta_4 \frac{\Delta_{\text{roof}}}{H} \quad (2.34)$$

where β_3 is the nonlinear amplification factor from the maximum inelastic displacement to maximum elastic displacement; 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 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 of elastic and nonlinear response stages.

2.6.3 Gupta & Krawinkler (2000)

Based on the time history analysis of nine steel moment resisting frame structures, Gupta & Krawinkler (2000) evaluated the modification factors related to the estimation of seismic roof and inter-storey drift demands. The spectral displacement demands is related to the roof drift demand for MDOF structure using three modification factors, accounting 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 demands was found to strongly 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. Similar to the N2 method, a pushover analysis is required to provide details of structural behavior.

2.6.4 Displacement spectrum

All the approximate methods discussed above require the information of relevant response spectrum, for example the elastic displacement spectrum. The design spectrum approach is most commonly used in earthquake engineering to estimate the seismic demand in terms of acceleration, velocity and displacement. Elastic design spectra often given in seismic design codes or, in the case of important structures, are determined by a seismic hazard analysis at the site. Inelastic spectra depend not only on the characteristics of the expected ground motion at the site, but also on the non-linear characteristics of the structural system, which severely complicates the problem.

Recently, the displacement response spectrum (DRS) has attracted a great deal of research interest in connection with the implementation of the performance-based design philosophy. Bommer & Elnashai (1999), Calvi & Kingsley (1995) and Tolis & Faccioli (1999) developed displacement spectra based on ground motions recorded during a particular earthquake or a particular region. Bommer & Elnashai (1999) examined 22 seismic codes from around the world and found that the DRS derived directly from acceleration spectra increase indefinitely with periods nearly in all cases, either linearly or even parabolically. Tolis & Faccioli (1999) calculated the DRS from the attenuation relationships of spectral acceleration and spectral velocity. Comparing with EC8 displacement spectra (EC8 1996), the above two spectra have much smaller spectral displacement at period larger than 0.5s.

Guan et al. (2004) developed a series of elastic displacement spectra based on a larger number of ground motion records. More than 500 records are classified into 3 groups according to the ratio of the peak ground acceleration to the peak ground velocity (A/V) and/or the ratio of the peak ground velocity to the peak

ground displacement (V/D). However, the ground motion records grouped by A/V and V/D ratios do not show a satisfactory correlation.

For the nonlinear response analysis, inelastic displacement spectra are required. Generally speaking, there are four different approaches for the purpose of developing an inelastic spectrum:

- a) Non-linear time-history analysis.
- b) Random vibration analysis.
- c) Equivalent linearization method.
- d) Reduction of the elastic spectra applying reduction factors.

Obviously, method a) can give the most accurate results, provided that the site-specific ground motions and the appropriate hysteretic model of the SDOF model are available. Method b), random vibration analysis, can be used to estimate inelastic response in probabilistic terms (Barber & Wen 1979, Grossmeyer 1979, Lai 1978 and Sehayek 1976).

In method c), the maximum displacement response is estimated as the maximum deformation of a linear elastic system with lower lateral stiffness and with higher damping coefficient than those of the inelastic system (Rosenblueth & Herrera 1964, Gulkan & Sozen 1974, Iwan 1980, and Kowalsky 1994). Method d) is based on a displacement modification factor. The maximum response of the inelastic SDOF system, Δ_{ie} , is estimated as a product of the maximum deformation of a linear elastic system, Δ_e , with the same lateral stiffness and same damping coefficient as those of the inelastic system (i.e. k_0 and ξ_0) times a displacement modification factor. This type of methods has their origin in the study by Veletsos & Newmark (1960) and has been further developed by Veletsos et al. (1965), Newmark & Hall (1982), and Miranda (2000).

2.6.5 Hysteretic model

A proper hysteretic model is crucial in the time-history analysis of the inelastic response of a structure. Several hysteretic models have been developed in the past to describe the structure behavior during initial (or elastic), cracking, yielding, stiffness and strength degrading stages, and crack and gap closures. Some of the

models such as Clough's model and Takeda's model are suitable only for modeling particular components (columns, beams, etc.) or particular behavior patterns (stiffness degradation, slip, etc.). A more general three-parameter model, the so-called Park model, was first proposed by Park et al. (1987). This hysteretic model incorporates three parameters in conjunction with a nonsymmetric trilinear envelope to establish the rules under which inelastic loading reversals took place. The values of these three parameters determine the properties of the stiffness degradation, strength deterioration (hysteretic energy-induced) and pinching behavior. They are referred to as α_{sd} , β_e , γ_p , respectively. A variety of hysteretic patterns can be achieved through the combination of the tri-linear envelope and the three parameters. This hysteretic model was later extended to four parameters (α_{sd} , β_e , β_d , γ_p) by allowing the strength deterioration to depend on the hysteretic energy, the ductility, or both (Kunnath et al. 1992). Figure 2.6 depicts the four parameter hysteretic model. In the present study, the original Park's model including only the hysteretic energy-induced strength degradation is considered since it has been calibrated and widely accepted in the previous studies (Kunnath et al. 1992, Bracci et al. 1995, Kunnath et al. 1995, Valles et al. 1996).

Figure 2.6 Three-parameter model: (Park et al. 1987, Valles et al. 1996)

2.7 Seismic reliability analysis of RC structural systems

General steps involved in the structural reliability analysis suggested by Gianni et al. (1986) included (i) modeling of seismic motion; (ii) modeling of the structural system; (iii) identification of failure criteria at a member level; and (iv) formulation of a methodology for the estimation of the probability of global system failure as a result of any critical combination of member failures. There are great differences among the structural reliability studies concerning the structural modeling and thus the overall procedure for the estimation of failure probability. But the main consideration for such type of studies is all the same, i.e., the balance between adequately accurate modeling and sufficiently varying scenarios. Reliability predictions can be obtained from closed-form solutions or simulation methods, making use of structural models ranging from SDOF idealizations to discretised finite element models.

Colangelo et al. (1995) used the Equivalent Stochastic Linearization technique to represent MDOF systems by means of elastic elements with inelastic springs at end of nodes. Combined with the equation of motion and equilibrium considerations, this simplification led to the estimation of variances and covariances of the response parameters that included the plastic rotations in selected frame members. The unsymmetrical force-deformation relationship of structures was considered to account for the fact of non-zero mean structural response. The correlation existing among the mechanical properties of the structural elements of an RC frame was also included and the effect of different correlation patterns on the reliability analysis was investigated. It was found that the effect of varying the correlation factor ρ is not particularly significant. The same procedures were also employed by Colangelo et al. (1996) to investigate a 4-storey and an 8-storey RC frames with “setbacks”. The comparisons of results obtained using the deterministic systems with those obtained using the stochastic ones proved the appropriateness of such an approximation in reliability studies.

Han & Wen (1997) assumed different structural layouts of varying dynamic characteristics. 88 real earthquake records were used in dynamic analyses of these

structures in order to derive regression equations for some displacement-based factors to predict the maximum global and local response of the original systems. These empirical response modification factors were then used in order to ensure that the SDOF Equivalent systems exhibited the same dynamic characteristics for the first two modes and the same global yielding behavior as the original MDOF systems. This procedure was demonstrated for steel frames, but it was deemed to be equally applicable for RC structures.

Tzavelis & Shinozuka (1988) reported a structural reliability study using finite element analysis. The structural reliability is measured in terms of the probability P_s that its response will not violate a set of performance criteria selected to represent a safe state of structural behavior throughout the structure's lifetime. Then, the failure probability P_f can be obtained by $P_f = 1 - P_s$. Plane rigid frames with shear walls were considered and failure was assumed to occur at the onset of either flexural or shear failure, or at the attainment of an ultimate shear stress. A linear finite element analysis was carried out, with particular interest in the interaction between the shear walls and RC frames. Structural reliability was calculated using a closed-form solution that involved the derivation of a failure surface considering random vibration theory. The global safety region was assumed to be the intersection of safety regions for all members in the frame.

It is noted that due to the complexity in the dynamic response of inelastic structural systems, closed-form solution, such as FORM or SORM, are generally not suitable. A lot of research works were based on simulation methods.

Casciati & Faravelli (1984) examined RC frames consisting of members with finite plastic hinge lengths in order to account for the spread of plasticity and a mechanical model for this purpose was proposed and applied in the simulation.

A 4-storey 3-bay frame was analyzed as a case study.

Based on Monte Carlo Simulation method, Luo et al. (1995) carried out a study on the seismic reliability of 3- and 10-storey, 3-bay frames forming part of a flat-slab building designed to withstand gravity and wind loads only. Monte Carlo

Simulation method was used due to the simplicity of two-dimensional nonlinear models adopted, and the expected relatively high failure probabilities that would require a rather small sample size.

Dymiotis et al. (1999) presented a methodology for probabilistic assessment of RC frames subjected to seismic loading. Model uncertainty and randomness in member capacity and failure criteria were considered while the matter of variability in the seismic input was kept as simple as possible. It was suggested that the spatial distribution of the concrete strength (f_c) should also be considered in the reliability analysis because an RC frame would be constructed storey by storey. Various scenarios of independence (or dependence) of f_c among different storeys are examined in the case studies. By Latin Hypercube Simulation methods, which are expected to achieve a better accuracy with a smaller sample size than that required by Monte Carlo Simulation methods, it was found that assuming full dependence between stories might significantly underestimate the failure probabilities obtained.

There are also some other methodologies which may be employed for structure reliability analysis in certain cases including the Parametric Surveying Method as applied by Asano et al. (1992), Vulnerability Function Methodology applied by Arede & Pinto (1996) and Bento & Azevedo (1998), and so on.

2.8 Fuzzy reliability analysis

2.8.1 Overview of fuzzy set theory (based on Zimmerman 1996)

Most of the traditional tools for formal modeling, reasoning, and computing are crisp, deterministic, and precise in character. By crisp it means dichotomous, that is, yes-or-no type rather than more-or-less type. In conventional dual logic, for instance, a statement can be true or false and nothing in between. In crisp set theory, an element can either belong to a set or not; and in optimization, a solution is either feasible or not.

The concept of fuzzy set, also associated with the term “graded membership”, has been considered as a model for inexact, vague statements about the elements of an ordinary set. Particularly, in phrases like “large numbers”, “tall buildings”, “approximately equal to 10”, the attribute, although common in the natural language, is not tractable by the methods of classical set theory or probability theory. There are two important things worthy to be noted. First, there is indetermination about membership or non-membership in a collection of objects. Second, there is nothing random in the concepts in the question.

In the first publication about fuzzy set theory, Zadeh (1965) wrote, “The notion of a fuzzy set provides a convenient framework which parallels in many respects the framework used in the case of ordinary sets, but is more general than latter and, potentially, may prove to have a much wider scope of applicability, particularly in the fields of pattern classification and information processing. Essentially, such a framework provides a natural way of dealing with problems in which the source of imprecision is the absence of sharply defined criteria of class membership rather than the presence of random variables”.

“Imprecision” here is meant in the sense of vagueness rather than the lack of knowledge about the value of a parameter. Fuzzy set theory provides a strict mathematical framework in which vague conceptual phenomena can be precisely and rigorously studied. It can also be considered as a modeling language well suited for situations in which fuzzy relations, criteria, and phenomena exist.

The ordinary set or, more precisely, a subset X of a given, total set U , can always be identified with a binary valued function, $f_X : X \rightarrow \{0, 1\}$, called as its characteristic function, and defined as

$$f_X(x) = \begin{cases} 1 & \text{if } x \in X \\ 0 & \text{if } x \notin X \end{cases} \quad (2.35)$$

The basic idea of fuzzy set is quite simple. Since there is indetermination concerning membership and non-membership, one can consider, in a referential U , a function defined on X whose values are in the unit interval $[0, 1]$ instead of just 0

and I . Such a function measures the degree of membership and is called a membership function. More formally, the definition of a fuzzy subset of U can be expressed as a function:

$$\mu: \tilde{X} \rightarrow [0, 1] \quad (2.36)$$

For every $x \in \tilde{X}$, the value $\mu(x)$, $0 \leq \mu(x) \leq 1$, represents the membership degree of x in $\tilde{X}$ -that is, the degree to which x satisfies that imprecise statement.

Definition 1

The *support* of a fuzzy set $\tilde{X}$, $S(\tilde{X})$, is the crisp set of all $x \in U$ such that $\mu_{\tilde{X}}(x) > 0$.

Definition 2

The (crisp) set of elements that belongs to the fuzzy set $\tilde{X}$ at least to the degree α is called the α -level set:

$$X_{\alpha} = \{x \in U \mid \mu_{\tilde{X}}(x) \geq \alpha\}.$$

Definition 3

A fuzzy set $\tilde{X}$ is convex when

$$\mu_{\tilde{X}}(\lambda x_1 + (1 - \lambda)x_2) \geq \min\{\mu_{\tilde{X}}(x_1), \mu_{\tilde{X}}(x_2)\}, \quad x_1, x_2 \in U, \quad \lambda \in [0, 1]$$

Alternatively, a fuzzy set is convex if all α -level sets are convex.

Definition 4

For a finite fuzzy set $\tilde{X}$, the *cardinality* $|\tilde{X}|$ is defined as

$$|\tilde{X}| = \sum_{x \in U} \mu_{\tilde{X}}(x)$$

$$\|\tilde{X}\| = \frac{|\tilde{X}|}{|U|} \text{ is called the } \textit{relative cardinality} \text{ of } \tilde{X}.$$

2.8.2 Application of Fuzzy Set Theory in reliability analysis

Great achievements have been made on both theory development and engineering applications for the classical structural reliability for the past several decades. In the classical reliability theory, some of the basic variables are considered to be random

while others are deemed deterministic. The reliability of a structural system is evaluated based on probability and statistics theory. However, in most practical cases, some variables are not random but fuzzy, or more commonly, not only random but also fuzzy, for instance, loadings and failure criteria. Consequently, the structural response and failure events are also of fuzzy properties. In such cases, the classical reliability theory is not rigorous enough and a new methodology of reliability analysis in which both random and fuzzy properties can be taken into account is required.

Zadeh (1968) defined the probability of a fuzzy event A as the expectation of its membership function, i.e.,

$$P(A) = E[\mu_A(X)] = \int \mu_A(x) f_X(x) dx \quad (2.37)$$

where $\mu_A(x)$ is the membership function of the fuzzy event A and $f_X(x)$ is the probability density function of variable x . Kandel & Byatt (1980) extended such a concept into the time domain. For time-dependent processes with a density function $f_X(x, t)$, the probability of a fuzzy event A is given by

$$P[A(t)] = \int \mu_A(x) f_X(x, t) dx \quad (2.38)$$

In some pilot papers on the application of fuzzy set theory on structural engineering, Yao (1980, 1982) considered the probability of a structure being severely damaged in the next 20 years as the probability of the occurrence of a fuzzy event. An attempt was made by Yao (1980) using elementary fuzzy relations to explore the application of fuzzy sets theory in the damage assessment of existing structures. Subjective membership values were applied to translate linguistic terms such as “small” in terms of fuzzy sets. Yao (1980) also deemed the establishment of various membership functions as an essential step in the relevant studies.

Holicky (1998) pointed out that the performance requirements might be considerably affected by fuzziness in the definition of critical conditions. Two types of uncertainty of performance requirements are therefore identified: randomness, handled by commonly used methods of theory of probability and, fuzziness, described by basic tools of the recently developed theory of fuzzy sets. The

fundamental performance condition, $S < R$, between an action effect S (deflection, acceleration) and a relevant performance requirement R (acceptable deflection or acceleration) was analyzed assuming randomness of S and both randomness and fuzziness of R .

Holicky (1998) proposed a transition region of R , $\langle r_1, r_2 \rangle$, where a building is gradually losing its ability to perform adequately and the implied damage increases, may be rather broad depending on the nature of the performance requirement. Following the traditional probability concept, a damage function $D_R(x)$ was defined as the weighted average of damage probabilities reduced by the corresponding damage level

$$D_R(x) = \frac{1}{N} \int_0^1 \nu \left(\int_{-\infty}^x \varphi_R(x'|v) dx' \right) d\nu \quad (2.39)$$

where N denotes a factor normalizing the damage function $D_R(x)$ to the conventional interval $\langle 0, 1 \rangle$ and x' is the generic point of x .

The damage function $D_R(x)$ defined by Equation 2.39 may be used similarly as a traditional distribution function to specify characteristic or design value of performance requirements guaranteeing an acceptable level of the total expected damage. Note that $D_R(x)$ is more complex than probability, as it is dependent on membership function $\nu_R(x)$ defined on a transition region $\langle r_1, r_2 \rangle$. Further, a fuzzy probability of performance failure π is then defined as

$$\pi = \int_{-\infty}^{\infty} \varphi_S(x) D_R(x) dx \quad (2.40)$$

where $\varphi_S(x)$ is the probability density function of the action effect S . Holicky (1998) considered that Equation 2.40 enables formulation of various design criteria in terms of relevant randomness as well as fuzziness characteristics.

Hadipriono (1985) analyzed several causes contributing to a failure accident of falsework and classified these causes into three categories: 1) deterministic, 2) non-deterministic probabilistic, and 3) non-deterministic possibility. The approach

to identify and prevent the cause of the third type involved a subjective appraisal using qualitative data, i.e., some linguistic terms. Hadipriono considered three categories of linguistic variables, i.e., the enabling event-related performance, frequency of triggering events, and their consequence. For instance, the linguistic variables for Consequence includes Very light, Light, Moderate, Severe, and Very severe. Fuzzy sets based on the possibility approach were used to transform the linguistic terms into mathematical representations. Arbitrarily determined membership values were assigned to the linguistic variables based on subjective judgments.

Hadipriono & Lai (1986) used the fuzzy set concept to determine the urgency measure for the preventive care of concrete components based on subjective judgment. Four interrelated variables, i.e., concrete appearance, strength, performance, and urgency measure for failure prevention were considered not to be able to be quantified objectively and precisely, but rather, they can be estimated subjectively and imprecisely through the engineer's experience and knowledge. Max-Min Composite Fuzzy Relation was employed in the study based assumed discrete membership functions.

Wu et al. (1999) proposed a fuzzy failure criterion to describe the fuzzy nature of failure phenomenon in a rock mass. A transition region, $[D_L, D_U]$, on the axis of damage index of rock mass was defined between the safe and failure region, using some critical damage indices suggested by other researchers. It was commented that the construction of membership function was a difficult and debatable task, and in most cases the membership function may only be established on the basis of expert's experience and judgment due to lack of data. Several types of membership functions were suggested to represent the fuzziness of the material failure. Based on the fuzzy-random probability theory (Equation 2.37), a model including both the effects of randomness and fuzziness was recommended for the failure analysis of rock mass under explosive loads.

2.9 Summary

Past earthquake experiences indicate that the traditional force-based seismic design philosophy cannot ensure reliable structural performance during moderate to strong earthquakes. Although most of structures designed by current seismic design codes may survive strong ground motions without collapse, the cost for repair could be unacceptable even under moderate ground motions. Controlling the damage and ensuring adequate performances are the primary objectives of the emerging performance-based seismic design philosophy. To achieve a performance-based design or conduct performance reliability analysis, two major issues need to be resolved, one concerns the quantification of limit states for individual performance levels, another concerns the determination of the seismic demand, which in turn relates to the dynamic properties of the structures and the seismic inputs. The relevant research works on the above aspects and on the various uncertainties involved are reviewed in this chapter.

Regarding the performance levels, qualitative descriptions exist (SEAOC 1995, NEHRP 1997); however, rationally quantified definitions of the performance levels are not available yet. Due to the large number of uncertainties involved, the quantification of limit states for individual performance levels will be studied in probabilistic terms in the following chapters. Moreover, due to the vague nature in the description of damage at each performance level, the Fuzzy set theory may become a suitable choice to handle the vagueness in order to obtain more appropriate assessment of performance levels. Some applications are reviewed to show the potential application of the Fuzzy set theory in the structural reliability analysis.

The main uncertainties involved in both structural properties and the seismic input are reviewed and discussed. Particular attention is drawn to the model uncertainty in the use of some simplified formulas or models to describe complex behavioral parameters such as the confined concrete strain. Such model uncertainty factors could play an important role in modeling the actual uncertainties and hence will be appropriately considered in the probabilistic assessment in the next chapter.

The review is extended to the various approximate methods for estimating the seismic (displacement) demand of structures and the generation of probabilistic (displacement) response spectra in line with the performance-based design and performance reliability evaluation.

Chapter 3 Probabilistic Analysis on RC Member

Deformation limits for Different Performance Levels

3.1 Introduction

The behavior of RC members, especially the inelastic behavior, depends on various geometric and material parameters, most of which are of a random nature. Therefore, uncertainty exists in the RC member behavior concerning the strength, ductility, as well as the responses at other performance levels in the context of the performance-based design (Priestley 2000). To more realistically describe such capacities requires a probabilistic model. Specific values for design or performance verification can then be determined from the probabilistic model according to a desired percentage chance of being exceeded by the respective actual capacities. This chapter presents a systematic probabilistic evaluation of the member deformation limits, including the sectional curvature and member drift, for different performance levels. The reliability of commonly used deterministic calculations in predicting the curvature and drift capacity limits is also assessed. The establishment of the probability distributions of these limits and the reliability assessment of the corresponding deterministic calculations can be useful in the performance reliability analysis and the development towards a uniform reliability design.

The uncertainties of RC member behavior at the sectional level concerning the curvature ductility have been investigated by some researchers (Trezos 1997, Kappos et al. 1999), based on simplified relationships between the curvature and the influencing random variables. In the study by Kappos et al. (1999), the model uncertainty factor for the confined concrete strain was also considered. In the present study, a refined RC section analysis procedure (Zhang & Lu 1993) is employed to carry out direct Monte Carlo Simulation to obtain the probability distribution of the sectional response at the yield and ultimate stages, as well as at

two intermediate performance levels for which little information about their probabilistic properties is available in the existing literature. By applying a direct simulation using a refined computer model, the errors that might arise from simplifying the sectional responses can be minimized.

The probability distribution of the member lateral drift capacities depends not only on the probabilistic sectional curvature but also on the distribution of the inelastic deformation along the member length. The latter can be described using an equivalent plastic hinge length, although experimental evidences have shown that the determination of such plastic hinge length is highly uncertain. In this study, a unified plastic hinge length formula is proposed on the basis of statistic analysis of a large collection of relevant experimental data, while its uncertainty is described using a model uncertainty factor with a probability distribution. Using this probabilistic plastic hinge length model and the aforementioned sectional curvature model, the distributions of the drift limits at different performance levels are established using Monte Carlo Simulation.

The sectional curvature and the drift limits are also predicted using the deterministic approach whereby the characteristic material strengths and nominal limit strain values are adopted. By comparing to the probabilistic results, the reliability of these deterministic calculations is identified in terms of the probability that actual limit values exceed the deterministic predictions.

3.2 Definition of performance levels

In the context of performance-based design, SEAOC (1995) proposed five performance levels, namely *Fully Operational*, *Operational*, *Life Safety*, *Near Collapse* and *Collapse*. This definition provides an overall picture of the building performance satisfying different design demands in linguistic terms. However, when it comes to physical quantification it is found that the boundary between any two adjacent performance levels is not easily drawn. For this reason, it is deemed more practical at the present stage to combine some performance levels to allow for more explicit translation into physical quantities. Thus, three performance levels

emerge: a) “*Functional*” performance level, referring to the state that the normal function of the building is maintained; b) “*Damage Control*” level which is mainly concerned with controllable structural damage and the life safety of building occupants; and c) “*Collapse*” level. For individual members the limit state for “*Collapse*” can be interpreted to correspond to the ultimate capacities of the member. More detailed quantifications follow.

a) *Functional level* In terms of the material properties, crushing of compressive unconfined concrete and unacceptable residual crack widths could be used to define the limit state for Functional performance level for which easy repairing work is possible without major interruption to the normal function of the building. Priestley (2000) suggested that the Functional strain limit for compressive concrete and tensile reinforcing bars be assigned to 0.004 and 0.015, respectively. These two values correspond respectively to the onset of crushing of concrete cover and a state whereby residual crack width is about 1.0 mm. These values are employed in the present study as the nominal limit criteria for the Functional performance level.

b) *Damage Control level* The limit state for Damage Control (DC) level can also be defined by the material strain limits to restrict excessive structural damage. A rated ultimate compressive strain of confined concrete, ε_{cm} , may be used as the DC level concrete compressive strain limit below which the confined concrete may still be considered repairable. Based on a model uncertainty study related to the ultimate compressive strain of confined concrete by Dymiotis (1999), the confinement model originally suggested by Tassios (1983) and adopted in EC8 (1989), as will be described later, appears to be generally conservative by around 50%, which means the actual ultimate state will not occur until the compressive strain of confined concrete further increases from the predicted value by about 50%. Since this level of reservation is just adequate for controlling the damage to confined concrete, as considered also by Kowalsky (2000), the confined concrete strain calculated using the EC8 model is considered as the nominal DC level concrete strain limit in the present study. Hence, the limit concrete strain (mean value) will be a function of the transverse confining reinforcement. On the other hand, for the longitudinal reinforcement a limit tensile strain (ε_{sm}) equal to 60% of the strain at the maximum

stress is adopted to avoid buckling of compressive reinforcement and low cycle fatigue (Priestley 2000). According to Kappos et al. (1999), the mean ultimate strain of typical longitudinal reinforcement is about 0.09; thus, the mean limit strain of reinforcement at DC level is taken to be $0.6 \times 0.09 = 0.054$. It is noted that in a previous study, Kowalsky (2000) suggested that for damage control of bridge columns the limit compressive strain of confined concrete core be 0.018 and the limit tension reinforcement strain be 0.060.

c) Collapse level The Collapse level implies that partial to total collapse occurs. The limit state of the Collapse level can be considered equivalent with the ultimate state defined in current design codes. From the material strain point of view, the limit for Collapse level can be interpreted as the state whereby the confined concrete reaches the ultimate compressive strain, ε_{cu} , or the longitudinal reinforcement on the tension side reaches the ultimate tension strain, ε_{su} .

3.3 Basic random variables influencing RC member behavior

The RC member behavior at different response levels depends on different aspects and combination of material properties and other variables. Figure 3.1 shows a typical analytical model for the RC sectional response, from which it can be understood that the main random variables that contribute in the actual probabilistic performance of an RC member include the constitutive behavior of unconfined and confined concrete, of the reinforcement, as well as the distribution of the inelastic deformation along the member length (plastic hinge length). The probabilistic properties of the random variables affecting the variability of the RC member behavior are enumerated briefly in what follows.

Figure 3.1 Fiber model for RC sectional analysis

3.3.1 Variables related to concrete properties

Cylinder compressive strength and modulus of elasticity

Class C25 concrete is used in the present study. The C25 concrete has a characteristic value of cylinder compressive strength (f_{ck}) equal to 25MPa, corresponding to a 95% chance of being exceeded. Assuming an average quality control, the statistical property of its compressive strength can be described as normal distribution, and its mean compressive strength is equal to $(25+1.645 \times 4.14)=31.8$ MPa. The modulus of elasticity is defined as the initial tangent modulus, E_c , which may be related to the cylinder compressive strength as Mirza et al. (1979)

$$E_c = 5017 f_c^{1/2} \quad (\text{in MPa}) \quad (3.1)$$

Ultimate compressive strain of unconfined and confined concrete

The maximum strain $\varepsilon_{c,85}$ of unconfined concrete is defined to correspond to the stress of $0.85f_c$ on the descending branch of the unconfined stress-strain curve. In the present study, the variation of $\varepsilon_{c,85}$ is assumed to follow a normal distribution with the mean value equal to 0.0035 and a standard deviation equal to 0.0005, which corresponding to a COV of 15% (Trezos 1997).

There exist various models for the ultimate compressive strain of confined concrete, ε_{cu} . The diversity of predictions using these individual models testifies a considerable inherent variability of this crucial parameter, and a complete description of it thus requires an expressive model in conjunction with a model uncertainty factor as described in Chapter 2, Section 2.4. According to Table 2.1 (Dymiotis 1999), the mean values of model uncertainty factor χ_m using different confinement models range from 1.06 to 2.23, and in most cases the COV is around 40%. The distribution of this model uncertainty factor approximately follows the normal distribution. With such a model uncertainty factor, the probabilistic value of ε_{cu} can be obtained from the predicted value by

$$\varepsilon_{cu} = \varepsilon_{cu,pred} \times \chi_{m,\varepsilon_{cu}} \quad (3.2)$$

In this study, Equation 2.12 (Tassio's model, EC8 1989) is chosen as the reference confinement model. According to the study in Dymiotis (1999), the model uncertainty factor of this confined concrete strain model has a mean equal to 1.492 and COV equal to 47.3%.

3.3.2 Variables related to reinforcement properties

Tension strength (f_y and f_u) and modulus of elasticity (E_s) of reinforcement

In this study, the probability distribution of the steel yield strength (f_y) is assumed to follow the Beta-distribution according to Mirza & Mac Gregor (1979a). For Grade 40 (ACI standard) reinforcing bars, the mean and COV are equal to 337MPa and 10.7% respectively, while for Grade 60 reinforcing bars, the mean and COV are 490MPa and 9.3%, respectively (Mirza & Mac Gregor 1979a). The ultimate strength of reinforcing bars (f_u) is also assumed to follow the Beta-distribution. Equation 3.3 and 3.4 express the relevant probability density functions. The modulus of elasticity of the reinforcing bars follows a normal distribution with a mean equal to 201GPa and a COV equal to 3.3% (Mirza & Mac Gregor 1979a).

Grade 40

$$PDF = 3.721 \left(\frac{f_y - 248040}{220480} \right)^{2.21} \left(\frac{468520 - f_y}{220480} \right)^{3.82} \quad 248\text{MPa} < f_y < 468\text{Mpa}$$

(3.3a)

Grade 60

$$PDF = 3.721 \left(\frac{f_y - 392730}{351390} \right)^{2.02} \left(\frac{744120 - f_y}{351390} \right)^{6.95} \quad 392\text{MPa} < f_y < 744\text{Mpa}$$

(3.3b)

Grade 60

$$PDF = 4.61 \left(\frac{f_u - 606320}{544310} \right)^{2.02} \left(\frac{1150630 - f_u}{544310} \right)^{6.95} \quad 606\text{MPa} < f_u < 1151 \text{ MPa}$$

(3.4)

Ultimate tension strain of steel bars (ϵ_{su})

The rupture of tension bars is another criterion, additional to the ultimate compressive strain of confined concrete, for the ultimate limit state of RC members. The characteristic value of the ultimate strain, considering commonly used

reinforcing bars, is 0.08 at a 10% fractile and the COV is 9%. Assuming a normal distribution, these lead to a mean equal to 0.09 with a COV of 9%. (Kappos et al. 1999)

Concrete and reinforcement strain limits at intermediate performance levels

As mentioned before, the intermediate performance levels concerning RC components are defined in relation with the concrete and reinforcement limit strains. Since these strain limits effectively represent a particular degree of physical damage on RC components, they should also be considered as random variables.

Section geometry

For a rectangular section, the height and width of the cross section can be assumed to follow normal distribution with a mean value equal to the nominal value plus 1.6 mm and a constant standard deviation equal to 6.4 mm to represent the in-situ cast conditions. (Mirza & Mac Gregor 1979b)

3.3.3 Uncertainty in modeling plastic hinge length

When an RC member responds into inelastic stage, the inelastic rotation occurs over a length near the critical section where the bending moment exceeds the yield moment of the section. This inelastic rotation is customarily represented by the product of the inelastic curvature at the critical section and an equivalent plastic hinge length, L_p . Using elementary slope-deflection analysis, the member lateral drift ratio (in a sense representing the inter-storey drift ratio) can be expressed as

$$\theta = \frac{1}{3} \phi_y Z_c + (\phi - \phi_y) L_p \frac{Z_c - L_p/2}{Z_c} \quad (3.5)$$

where ϕ is the inelastic curvature at the response level concerned, ϕ_y is the yield curvature, and Z_c is the length measuring from the fixed end to the contra flexure of the member.

Most of the available data in the literature regarding L_p refers to the plastic hinge length at the ultimate stage. Studies (Sheikh et al. 1994 and Priestley & Park 1987) tend to suggest that the equivalent plastic hinge length is almost independent of variables such as the steel configuration, axial load level, amount of longitudinal

and confining reinforcement, yield stress of longitudinal reinforcement, and concrete strength. Most of existing models for the calculation of the plastic hinge length relate L_p to the effective depth of cross section, d (Sheikh et al. 1994, and Oguzhan 1999), or in a combined manner with the contra-flexure point height, Z_c (Sawyer 1964, Corley 1966, Mattock 1967 and Priestley & Park 1987). Priestley & Park (1987) also suggested that L_p be a function of Z_c and the maximum diameter of longitudinal bars d_b , which to some extent represents the effect of slip of longitudinal reinforcement relative to the member fixed end (fixed-end rotation). Some empirical expressions for L_p are tabulated in Table 3.1. Table 3.2 summarizes some experimental data for L_p obtained from 9 test programmes mainly on seismic behaviour of RC columns and the predicted values (all normalized by the section depth D_s) corresponding to various empirical expressions.

By statistical analysis, the mean and COV of L_p/D_s corresponding to the experimental results of Oguzhan (1999) and Sheikh et al. (1994) are found to be 1.031 and 11.3%, respectively. In contrast, the mean and COV of L_p/D_s corresponding to the rest of the test results shown in Table 3.2 are 0.466 and 23.1%, respectively. For the complete test results listed in Table 3.2, the mean and COV are equal to 0.791 and 38.5%, respectively. Such discrepancy clearly indicates that the value of L_p is not very well correlated only with section depth, D_s (or the effective section depth, d). This is further observed from the scatter plot for all 47 experimental results in Figure 3.2. Neither the approximation of L_p equal to $0.5D_s$, suggested by Priestley & Park (1987), nor L_p equal to $1.0D_s$, suggested by Oguzhan (1999) and Sheikh et al. (1994), does provide an adequate prediction.

Table 3.1 Empirical formulas of L_p

Reference	Proposed formula
Sawyer (1964)	$L_p = 0.25d + 0.075Z_c$
Corley (1966)	$L_p = 0.5d + d^{0.5}(Z_c/d)$
Mattock (1967)	$L_p = 0.5d + 0.05Z_c$
Priestley & Park (1987)	$L_p = 0.08Z_c + 6d_b$

Table 3.2 Experimental and predicted values of plastic hinge length L_p/D_s

Reference:	D_s	Z_c/D_s	L_p/D_s				
			Exp values	Corley	Mattock	Sawyer	Priestly& Park
Davey (1987)	500	3.50	0.540	0.598	0.625	0.488	0.436
	500	5.50	0.580	0.683	0.725	0.638	0.596
Munro (1984)	500	5.50	0.450	0.683	0.725	0.638	0.596
Ng (1978)	250	4.00	0.580	0.626	0.600	0.500	0.632
Gill (1982)	550	2.18	0.440	0.543	0.564	0.391	0.436
	550	2.18	0.340	0.543	0.564	0.391	0.436
	550	2.18	0.400	0.543	0.564	0.391	0.436
	550	2.18	0.500	0.543	0.564	0.391	0.436
	600	2.00	0.350	0.537	0.558	0.379	0.400
Potangaroa (1981)	600	2.00	0.350	0.537	0.558	0.379	0.400
	600	2.00	0.370	0.537	0.558	0.379	0.400
	600	2.00	0.420	0.537	0.558	0.379	0.400
	600	2.00	0.420	0.537	0.558	0.379	0.400
Ang (1981)	400	4.00	0.540	0.625	0.638	0.519	0.560
	400	4.00	0.610	0.625	0.638	0.519	0.560
	400	4.00	0.730	0.625	0.638	0.519	0.560
	400	4.00	0.550	0.625	0.638	0.519	0.560
Mander (1984)	750	4.27	0.370	0.617	0.680	0.554	0.422
	750	4.27	0.380	0.617	0.680	0.554	0.422
	750	4.27	0.400	0.617	0.680	0.554	0.422
	750	4.27	0.410	0.617	0.680	0.554	0.422
Oguzhan (1999)	305	6.04	1.150	0.810	0.740	0.670	0.860
	305	6.04	0.980	0.810	0.740	0.670	0.860
	305	6.04	0.910	0.810	0.740	0.670	0.860
	305	6.04	0.890	0.810	0.740	0.670	0.860
	305	6.04	0.860	0.810	0.740	0.670	0.860
	305	6.04	1.050	0.810	0.740	0.670	0.860
	305	6.04	0.880	0.810	0.740	0.670	0.860
	305	6.04	1.260	0.810	0.740	0.670	0.860
	350	5.26	1.020	0.750	0.710	0.610	0.750
	350	5.26	1.270	0.750	0.710	0.610	0.750
	350	5.26	1.130	0.750	0.710	0.610	0.750
	350	5.26	1.190	0.750	0.710	0.610	0.750
	350	5.26	0.850	0.750	0.710	0.610	0.750
	350	5.26	0.910	0.750	0.710	0.610	0.750
	350	5.26	0.980	0.750	0.710	0.610	0.750
	350	5.26	1.000	0.750	0.710	0.610	0.750
	350	5.26	1.120	0.750	0.710	0.610	0.750
	350	5.26	1.110	0.750	0.710	0.610	0.750
	350	5.26	1.030	0.750	0.710	0.610	0.750
	350	5.26	0.990	0.750	0.710	0.610	0.750
	250	7.37	1.120	0.936	0.803	0.774	1.047
	250	7.37	1.110	0.936	0.803	0.774	1.047
	250	7.37	1.020	0.936	0.803	0.774	1.047
	250	7.37	0.990	0.936	0.803	0.774	1.047
Sheikh (1994)	305	4.84	1.060	0.702	0.687	0.586	0.760
	305	4.84	0.890	0.702	0.687	0.586	0.760
	305	4.84	1.080	0.702	0.687	0.586	0.760

Figure 3.2 Estimation of L_p from section height D_s

In the theory of Priestley & Park (1987), the equivalent plastic hinge length, L_p , is mainly dependent on the contra-flexure point height, Z_c . The test values of L_p normalized with respect to Z_c , as well as the ratios between test and predicted values using different models involving Z_c are listed in Table 3.3. The mean value and COV of L_p/Z_c for all the 47 experimental results are equal to 0.168 and 24.1%, respectively. This result tends to indicate that L_p is much better correlated to Z_c than to D_s . Figure 3.3 shows the scatter plot of experimental value of L_p against contra-flexure point height Z_c . Except a few data points, the relationship $L_p=0.168Z_c$ appears to represent well the variation trend from a statistical point of.

Table 3.3 Experimental and predicted value of plastic hinge length L_p/Z_c

Reference:	D_s	Z_c/D_s	$L_{p,exp}/Z_c$	$(L_{p,exp}/Z_c)/(L_{p,anal}/Z_c)$			
				Corley	Mattock	Sawyer	Priestly & Park
Davey (1987)	500	3.50	0.154	0.901	0.856	1.100	1.222
	500	5.50	0.105	0.847	0.789	0.905	0.963
Munro (1984)	500	5.50	0.082	0.661	0.617	0.707	0.752
Ng (1978)	250	4.00	0.145	0.873	0.918	1.124	0.918
	550	2.18	0.201	0.810	0.782	1.058	0.995
Gill (1982)	550	2.18	0.156	0.629	0.607	0.821	0.772
	550	2.18	0.183	0.738	0.712	0.963	0.906
	550	2.18	0.229	0.923	0.891	1.205	1.134
	600	2.00	0.175	0.648	0.625	0.921	0.875
Potangaroa (1981)	600	2.00	0.175	0.648	0.625	0.921	0.875
	600	2.00	0.185	0.685	0.661	0.974	0.925
	600	2.00	0.210	0.778	0.750	1.105	1.050
	400	4.00	0.135	0.854	0.844	1.038	0.964
Ang (1981)	400	4.00	0.150	0.949	0.938	1.154	1.071

Mander (1984)	400	4.00	0.183	1.158	1.144	1.408	1.307
	400	4.00	0.138	0.873	0.863	1.062	0.986
	750	4.27	0.087	0.600	0.547	0.674	0.888
	750	4.27	0.089	0.614	0.560	0.690	0.908
	750	4.27	0.094	0.648	0.591	0.729	0.959
Oguzhan (1999)	305	6.04	0.190	1.496	1.545	1.696	1.338
	305	6.04	0.162	1.276	1.317	1.446	1.141
	305	6.04	0.151	1.189	1.228	1.348	1.063
	305	6.04	0.147	1.157	1.195	1.313	1.035
	305	6.04	0.142	1.118	1.154	1.268	1.000
	305	6.04	0.174	1.370	1.415	1.554	1.225
	305	6.04	0.146	1.150	1.187	1.304	1.028
	305	6.04	0.209	1.646	1.699	1.866	1.472
	350	5.26	0.194	1.426	1.426	1.644	1.366
	350	5.26	0.241	1.772	1.772	2.042	1.697
	350	5.26	0.215	1.581	1.581	1.822	1.514
	350	5.26	0.226	1.662	1.662	1.915	1.592
	350	5.26	0.161	1.184	1.184	1.364	1.134
	350	5.26	0.173	1.272	0.272	1.466	1.218
	350	5.26	0.186	1.368	1.368	1.576	1.310
	350	5.26	0.190	1.397	1.397	1.610	1.338
	350	5.26	0.213	1.566	1.566	1.805	1.500
	350	5.26	0.211	1.551	1.551	1.788	1.486
	350	5.26	0.196	1.441	1.441	1.661	1.380
	350	5.26	0.188	1.382	1.382	1.593	1.324
	250	7.37	0.152	1.197	1.394	1.448	1.070
	250	7.37	0.151	1.189	0.385	1.438	1.063
	250	7.37	0.139	1.094	1.275	1.324	0.979
	250	7.37	0.134	1.055	1.229	1.276	0.944
Sheikh (1994)	305	4.84	0.219	1.510	1.542	1.810	1.395
	305	4.84	0.184	1.269	1.296	1.521	1.172
	305	4.84	0.224	1.545	1.577	1.851	1.427
Mean			0.168	1.114	1.127	1.320	1.142
COV (%)			24.1	30.8	32.8	28.3	20.2

Figure 3.3 Estimation of L_p from contra-flexure point height Z_c

3.3.4 Proposed model for equivalent plastic hinge length

Priestly & Park (1987) provided two reasons to support their argument that longitudinal bar diameter should be considered in formulating L_p . The first reason is associated with the slip of reinforcement relative to the concrete. A classical moment-curvature analysis is based on the assumption of perfectly rigid concrete base. It means that the curvature distribution decreases from a maximum to zero over an infinitely small depth in the column base. In fact, there will be relative slip between the reinforcement and concrete in the upper part of the column base, and hence additional rotation. In the plastic hinge region, slippage of reinforcement can also result in higher strain in longitudinal bars at sections next to the base than predicted. The extent of yield penetration will clearly be independent of column height, Z_c , and would apparently depend mainly on the diameter of longitudinal bars. The second reason involves the shear effect on the crack pattern. When flexural cracks are inclined under the shear stress, the assumption of “plane-sections-remaining plane” is no longer strictly applicable and there will be an increase in steel stress and strain above the levels predicted on the plane-sections assumption. This will tend to spread the plasticity, and increase the plastic hinge length. Assuming a 45-degree crack direction, the extent of this influence would be proportional to the section depth.

Figure 3.4 schematically illustrates how the influence of steel slippage and shear spreading plasticity are included in the formulation of L_p . Accordingly, a general expression for L_p can be written as:

$$L_p = C_1 Z_c + C_2 d_b + C_3 D_s \quad (3.6)$$

where, Z_c denotes the distance between the contra-flexure point of the column and the section of maximum moment, d_b is the diameter of longitudinal bars, D_s is the depth of section, while C_1 , C_2 and C_3 are constants to be determined. Based on the available test results, Priestly & Park (1987) suggested that $C_1=0.08$, $C_2=6$ and $C_3=0$. The zero value of C_3 implies that there is no significant dependence of plastic hinge length on section depth. Equation 3.6 then becomes

$$L_p = 0.08 Z_c + 6 d_b \quad (3.7)$$

Figure 3.4 “Exact” curvature distributions for deflection calculations

In the present study, it is proposed that Equation 3.6 be adopted for its sound physical mechanism background. To get a more appropriate estimation of the coefficients, a regression analysis is carried out based on all 47 test results included in Table 3.4. The following two expressions are obtained, based on consideration of two variables, Z_c and d_b , and one variable, Z_c , respectively:

$$L_p = 0.077Z_c + 8.16d_b \quad (3.8)$$

$$L_p = 0.15Z_c \quad (3.9)$$

Table 3.4 lists the mean and COV of the 47 values of $(L_{p,exp}/Z_c)/(L_{p,anl}/Z_c)$ corresponding to Equation 3.8 and Equation 3.9 as compared to other existing models.

Table 3.4 Model uncertainty factor $X_{m,Lp}$ for different empirical model of L_p

	$(L_{p,exp}/Z_c)/(L_{p,anl}/Z_c)$					
	Corley	Mattock	Sawyer	Priestley	Equ. 3.8	Equ. 3.9
Mean	1.114	1.127	1.32	1.142	1.000	1.119
COV	30.8%	32.8%	28.3%	20.2%	19.8%	24.1%

It can be found that the model uncertainty factor $\chi_{m,Lp}$ corresponding to Equation 3.8 has a mean value equal to unity and a COV of 19.8% that is smallest among all

existing empirical formulas. Without considering the effect of d_b on the value of L_p , Equation 3.9 gives $\chi_{m,Lp}$ with a mean equal to 1.119 and a COV of 24.1%, which are less satisfactory than the results from Equation 3.8. By hypothesis test at a significance level of 0.05, the 47 values of $(L_{p,exp}/Z_c)/(L_{p,anl}/Z_c)$ corresponding to Equation 3.8 can be described as normal distribution.

Figure 3.5 shows a comparison between the predicted L_p using Equation 3.8 and the experimental data. Considerable scatters which still can be observed from Figure 3.5 are deemed as inherent with the test data. Even for specimens with similar geometric and sectional details, there are still significant differences among the measured values of equivalent plastic hinge length L_p . Although from the basic mechanics point of view, there could be more accurate prediction model for L_p , but as attempted by some other researchers mentioned above, they may not show any advantage over the simple model from statistical point of view. This phenomenon is largely attributable to the complex mechanisms involved in the critical regions, and hence the high uncertainties. The lack of a uniform and rigorous way in determining the L_p from experiments is another source of the large scatter. For these reasons, it has become more advantageous to adopt simple prediction models for the L_p . In the subsequent simulation analysis, each sample of L_p is taken to be the value calculated from Equation 3.8 multiplied by a randomly sampled $\chi_{m,Lp}$ according to the above distribution.

Figure 3.5 Comparison of experimental and predicted plastic hinge lengths

To obtain drift limits for intermediate performance levels according to Equation 3.5, it is necessary to determine the plastic hinge length at these performance levels. Considering the fact that the plastic hinge zone develops with the increase of the level of the inelastic deformation, it is deemed appropriate to use a variable plastic hinge length for different performance levels. Because of lacking pertinent experimental data, and taking into account the “fixed-end rotation” contribution which starts to cumulate immediately following the occurrence of cracking at the “fixed end”, an equivalent plastic hinge length of $0.5L_p$ is assumed for the yielding stage. A linear interpolation then applies between the yield and ultimate stages,

$$L_{p,PL} = \frac{1}{2}L_p + \left(\frac{\phi_{PL} - \phi_y}{\phi_u - \phi_y} \right) \frac{1}{2}L_p = \frac{0.5(\phi_u + \phi_{PL}) - \phi_y}{\phi_u - \phi_y} L_p \quad (3.10)$$

where $L_{p,PL}$ denotes the plastic hinge length at a particular performance level; ϕ_{PL} is the corresponding section curvature.

3.4 Monte Carlo Simulation of RC member behavior

The behavior of RC members can be predicted by means of structural analysis based on the material properties and member geometry. As the main variables are random in nature, the RC member behavioral parameters are probabilistic and their probabilistic distributions can be simulated by means of Monte Carlo Simulation. For randomly sampled values of the member properties, the sectional analysis is carried out using a computer program to obtain the response limits at different performance levels. Subsequently, the statistical property of the system response can be derived based on the simulation results.

3.4.1 Sectional analysis

A computer program (Zhang & Lu 1993) for cyclic and monotonic analysis of RC sections is adapted to carry out the analysis in the simulation. The program is based on the fibre-element method, which is schematically shown in Figure 3.1. An RC cross section is divided into 5 groups (regions), in which the first and second groups are the bottom and top cover areas respectively, the 3rd group is the confined concrete area, the 4th group includes the remaining side cover areas, and the 5th group consists of all steel layers. For a given curvature, iteration is carried out until

equilibrium is satisfied considering individual material stress-strain constitutive relations. Different stress-strain models are applied for the confined concrete core and the unconfined concrete cover of the section.

3.4.2 Representative specimens

Three groups of RC column specimens are selected to represent typical design details seen in practice. Table 3.5 summarizes the details of the specimens. Group-1 consists of specimens *A*, *B* and *C*, which have the same longitudinal reinforcement ratio and effective confinement (represented by $\alpha\omega_w$) but varying section dimensions; Group-2 includes specimens *B*, *D* and *E* which have the same confining reinforcement (0.81%) but different longitudinal reinforcement ratios ranging from 0.95% to 3.8%. Group-3 includes specimens *D*, *F* and *G* which have the same longitudinal reinforcement (2.15%) but different confining reinforcement ratios ranging from 0.81% to 2.09%.

Three axial force levels, with the axial force ratio (ν) equal to 0.1, 0.2 and 0.4, respectively, are considered. The axial force ratio is defined as $\nu = N / f_c b h$, where N is the axial force, f_c is the concrete compressive strength, and $b h$ denotes the sectional area. It should be pointed out that different axial load levels considered here are to reflect their effect on the concrete compressive strain from the demand point of view. Although it is recognized that the axial load level could also affect the effectiveness of the confinement (especially through the influence on the compression zone area), there is still a lack of systematic data to allow for quantification of such effect, so that effect is not considered in the present study.

A sample size of 1200 is found appropriate for the simulation by trial computations. Hence, 1200 simulation runs are performed for each specimen under a given axial force level. The simulation results are summarized and discussed in what follows.

Table 3.5 Details of specimens**Group 1: varying geometry**

Specimen	A	B	C
Dimensions (mm)	300×300	400×400	500×500
Concrete grade	C25	C25	C25
Rebars	8D12.7	12D12.7	12D15.9
Rebar grade	Grade 60	Grade 60	Grade 60
ρ_ℓ (%)	1.00	0.95	0.96
Hoops	D9.5@150	D9.5@150	D9.5@140
Hoop grade	Grade 40	Grade 40	Grade 40
ρ_s (%)	1.23	0.81	0.78
$\alpha\omega_w$	0.042	0.043	0.044
Confinement pattern

Group 2 (varying longi. reinforcement) & Group 3 (varying trans. reinforcement)

Specimens	B	D	E	F	G
Section (mm)	400×400	400×400	400×400	400×400	400×400
Concrete grade	C25	C25	C25	C25	C25
Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
Rebars	12D12.7	12D19.1	12D25.4	12D19.1	12D19.1
ρ_ℓ (%)	0.95	2.15	3.80	2.15	2.15
Hoops (mm)	D9.5@150	D9.5@150	D9.5@150	D9.5@90	D12.7@90
Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
ρ_s (%)	0.81	0.81	0.81	1.34	2.09
$\alpha\omega_w$	0.043	0.043	0.043	0.084	0.131
Confinement pattern

* ρ is the reinforcement ratio while $\alpha\omega_w$ is the mechanical confinement ratio. Bar diameter of 9.5mm, 12.7mm, 15.9mm and 19.1mm corresponds respectively to No.3, No.4, No.5 and No.6 steel in ACI standard.

3.5 Probabilistic distributions of RC member behavior at different performance levels from simulation results

3.5.1 Yield strength and yield curvature

The yield state is defined when the longitudinal reinforcement first reaches the yield strain. The statistics of the simulated yield curvature are summarized in Table 3.6 (Column 2~3). It can be observed that the yield curvature exhibits a small variability, with a COV ranging from 7% to 10%. The COV of the yield moment is slightly higher, in the range of 10~15%. The low variability of the yield level response apparently is attributable to the low variability in the yield strain and stress of the longitudinal reinforcing bars which play a governing role on the yield level response. The distribution of the yield curvature can be described as normal distribution.

Table 3.6 Normalized sectional curvature (multiplied by section depth) at different performance levels

Specimen & Axial force (v)		Yield		Functional		Damage Control		Collapse	
		Mean	COV	Mean	COV	Mean	COV	Mean	COV
A	0.1	0.0044	8.8%	0.0217	17.0%	0.0627	33.4%	0.1022	31.8%
	0.2	0.0049	8.3%	0.0181	26.3%	0.0413	43.9%	0.0599	49.3%
	0.4	0.0058	9.1%	0.0107	21.0%	0.0226	46.4%	0.0337	44.2%
B	0.1	0.0045	8.2%	0.0195	14.5%	0.0640	31.0%	0.1044	28.9%
	0.2	0.0049	7.9%	0.0168	20.7%	0.0503	43.3%	0.0769	43.2%
	0.4	0.0059	8.9%	0.0105	21.1%	0.0302	49.6%	0.0470	46.3%
C	0.1	0.0047	7.5%	0.0195	14.5%	0.0636	31.4%	0.1032	29.9%
	0.2	0.0051	7.6%	0.0174	20.3%	0.0511	44.5%	0.0789	42.3%
	0.4	0.0060	8.4%	0.0114	19.9%	0.0295	50.8%	0.0466	45.9%
D	0.1	0.0045	7.9%	0.0190	18.1%	0.0545	41.2%	0.0838	41.4%
	0.2	0.0052	8.2%	0.0146	20.1%	0.0403	44.9%	0.0597	44.1%
	0.4	0.0060	9.0%	0.0101	20.0%	0.0291	50.3%	0.0452	47.6%
E	0.1	0.0052	10.7%	0.0157	18.3%	0.0418	43.7%	0.0607	43.1%
	0.2	0.0055	9.1%	0.0132	19.0%	0.0366	47.2%	0.0549	45.8%
	0.4	0.0062	9.0%	0.0098	18.0%	0.0281	51.1%	0.0438	50.9%
F	0.2	0.0053	8.5%	0.0147	19.9%	0.0611	40.3%	0.0937	41.3%
	0.4	0.0061	8.7%	0.0103	20.0%	0.0486	48.0%	0.0754	46.7%
G	0.2	0.0053	8.3%	0.0148	19.4%	0.0751	30.4%	0.1189	31.0%
	0.4	0.0061	8.7%	0.0104	19.9%	0.0706	38.7%	0.1114	39.3%

3.5.2 Sectional response at Collapse level

The limit state for Collapse level (i.e., ultimate state) is considered to have been reached when either the concrete compressive strain at the extreme edge of the confined core reaches the ultimate strain, ε_{cu} , or the outmost tension bars reach the ultimate strain, ε_{su} . Typical histograms of the simulated ultimate curvature are shown in Figure 3.6. A summary of the mean and COV of the ultimate curvature for different specimens and under different axial force ratios is given in Table 3.6.

The Collapse curvature exhibits a large variability with COV on an order around 50%. This is attributable to the high variability of the ultimate strain of confined concrete which is affected by several basic random variables, especially its model uncertainty factor. It is noteworthy that the COV for low axial force cases is reduced to the order of 30~35% due to increased chances for the reinforcement ultimate strain, which has a relatively low variability, to govern the ultimate state. The histograms of ultimate curvature shown in Figure 3.6 suggest that most of the ultimate curvatures, especially in high axial force levels, follow normal distribution which is also proven by a skewness-kurtosis test.

From Table 3.6, it can be observed that the cross-section dimension (comparing A, B and C) has limited influence on the normalized section curvatures (actual curvature multiplied by the section depth) as well as their probabilistic distributions under a given axial force level. The general effects of the longitudinal and transverse reinforcement are illustrated in Figure 3.7(a). Note that concerning the confinement effect only the axial force ratios of 0.2 and 0.4 are shown, as at lower axial force the normal confining effect of the transverse reinforcement calculated with Equation 2.12 may become questionable due to a small compression zone. From the figures it can be seen that the effect of varying longitudinal reinforcement on the Collapse curvature becomes less significant with increase of the axial force ratio. This is because with higher axial forces the neutral axis depth, which directly affects the ultimate curvature, becomes less sensitive to the longitudinal reinforcement. On the other hand, increasing the transverse reinforcement exhibits a continuous enhancement on the ultimate curvature, due to increased ultimate confined concrete strain which governs the ultimate curvature in most cases.

(a) Specimens A, B, C (different cross section dimensions)

(b) Specimens D, F, G (different confining reinforcement)

Figure 3.6 Typical histograms of sectional curvature for specimens with different geometry or reinforcement details at ultimate state

Figure 3.7 Variation of mean normalized sectional curvature at different performance levels with reinforcement details and axial force ratios

3.5.3 Curvature at Functional performance level

Crushing of compressive unconfined concrete and unacceptable residual crack widths are used to define the Functional performance level for which easy repairing work is possible. In the present study, a concrete compressive strain limit of 0.004 and a tensile reinforcement strain limit of 0.015 are used as the Functional performance limit criteria. The uncertainties of these limit strains are described by a

normal distribution with a COV equal respectively to 15% and 9%. The COV values are selected such that they are consistent with those of the unconfined concrete compressive strength and the reinforcement yield strength with which the uncertainty of the Functional limit most closely correlate.

With the above probabilistic properties, the probabilistic sectional curvature at the Functional performance level can be simulated. Figure 3.8 shows the typical histograms of the Functional performance level curvatures. The statistics of the simulation results are included in Table 3.6. According to the hypothesis test, almost all the simulation results of the Functional section curvature under different axial force levels satisfy the normal distribution. From Table 3.6 it can be observed that, despite a wide variation of the mean Functional curvature for different specimens, the COV is relatively stable and in a majority of cases it falls between 15~20%, which is slightly higher than the assumed COV for the limit concrete compressive strain. Examining the section response details confirms that most of the samples are governed by the limit concrete strain at this performance level. As shown in Figure 3.7(b), the nominal Functional curvature practically is not affected by the confinement but decreases slightly with the increase of the longitudinal reinforcement.

Figure 3.8 Typical histograms of simulated Functional level curvature

3.5.4 Curvature at Damage Control performance level

For DC performance level, the limit tensile strain of longitudinal reinforcement is taken as 60% of the ultimate reinforcement strain, and it has a mean equal to 0.054 and COV equal to 9%. The limit compressive strain of confined concrete, $(\varepsilon_c)_{\text{lim},DC}$, obtained directly from Equation 2.12, has a model uncertainty factor of 1 with COV equal to 47.3%. With these probabilistic DC limit criteria, the simulation is carried out and the statistics of the simulated DC level curvature are listed in Table 3.6. Figure 3.9 shows some typical histograms. Hypothesis tests show that the DC level curvature fits well the normal distribution. The effect of longitudinal and transverse reinforcement on the nominal DC level curvature, plotted in Figure 3.7(c), shows similar trends as observed at the Collapse level.

Axial force ratio = 0.1

Axial force ratio = 0.2

Axial force ratio = 0.4

Figure 3.9 Typical histograms of simulated Damage Control level curvature

3.6 Deterministic calculations of limit curvatures and assessment of their reliability

Having established the probabilistic distributions of the limit curvature for the specimens at different performance levels, it is now possible to assess the reliability of the deterministic limit curvatures calculated using deterministic values of the material properties and other variables involved.

A common deterministic calculation of the RC member deformation capacities for design verifications is to use nominal (mean) values of the limit strain properties in conjunction with the design material strengths without safety factors (i.e., the characteristic strength) for concrete and reinforcement. Since at different performance levels (depending also on the reinforcement details and the axial force ratios) the relative importance of the various influencing parameters could be different, it can be expected that the reliability of the deterministic calculations could also vary.

The deterministic limit curvature values calculated using the above-mentioned strain and strength properties for the various specimens are tabulated in Table 3.7. By fitting the curvature values onto the corresponding probability distributions as illustrated in Figure 3.10, the probability of the deterministic predictions being exceeded by actual curvature limits can be obtained and the results are summarized in Table 3.7. Figure 3.11 depicts such probability results in the form of scatter plots.

Table 3.7 Predicted sectional curvatures (multiplied by section depth) at different performance levels using deterministic approach

Specimen & Axial force (v)	<i>Functional</i>		<i>Damage Control</i>		<i>Collapse</i>		
	Curvature (×Section d)	Probability of being exceeded (%)	Curvature (×Section d)	Probability of being exceeded (%)	Curvature (×Section d)	Probability of being exceeded (%)	
A	0.1	0.0234	32.3	0.0762	26.3	0.1236	26.2
	0.2	0.0145	77.5	0.0351	65.2	0.0492	68.1
	0.4	0.0090	76.3	0.0214	56.9	0.0298	61.3
B	0.1	0.0207	33.3	0.0752	28.7	0.1216	29.1
	0.2	0.0152	72.3	0.0456	60.3	0.0608	69.1
	0.4	0.0093	77.1	0.0280	57.5	0.0397	64.1
C	0.1	0.0207	33.0	0.0750	28.9	0.1210	27.9
	0.2	0.0151	73.8	0.0467	59.5	0.0655	66.0
	0.4	0.0094	79.9	0.0241	65.8	0.0393	64.7
D	0.1	0.0173	62.1	0.0520	55.9	0.0772	58.2
	0.2	0.0134	69.3	0.0399	52.8	0.0572	56.8
	0.4	0.0091	74.4	0.0241	64.6	0.0386	63.2
E	0.1	0.0148	59.8	0.0432	48.2	0.0604	51.3
	0.2	0.0121	66.9	0.0379	48.0	0.0556	49.3
	0.4	0.0090	71.4	0.0235	63.9	0.0380	62.4
F	0.2	0.0134	71.5	0.0624	43.3	0.0908	54.0
	0.4	0.0089	78.8	0.0460	56.2	0.0644	62.9
G	0.2	0.0134	72.7	0.0924	21.8	0.1432	25.7
	0.4	0.0092	76.3	0.0744	45.8	0.1084	54.0

Specimen D (Functional, Damage Control, Collapse)

Specimen G (Functional, Damage Control, Collapse)

Figure 3.10 Probability of actual curvature limits exceeding the values predicted by deterministic approach at different performance levels

Figure 3.11 Scatter plots of probability of actual curvature limits exceeding the values predicted by deterministic approach

As can be seen, the probability of the deterministic predictions being exceeded by the actual curvature limits is not uniform. Whereas in most cases such probability is in the range of 50%~70%, there are cases whereby the probability is only on the order of about 30% for all performance levels. Further examination of the results reveals that these situations occur mainly at low axial force ratio and with low longitudinal reinforcement, or at medium axial force ratio ($\nu=0.2$) and with a high confinement. These are cases where the limit state is more likely to be governed by the reinforcement limit strain. As the reduction ratio of the characteristic concrete strength from the mean strength is markedly higher than that in the reinforcement strength due to a higher COV, a larger compression zone depth is obtained in the deterministic calculation as compared to that corresponding to the mean material strength which should represent the average situation in the simulation.

Consequently, when the limit criteria turn to be governed by the reinforcement tensile strain, the deterministic calculation of the curvature becomes higher than the average curvature and hence a probability of being exceeded to be lower than 50%.

3.7 Monte Carlo Simulation on RC member drift

3.7.1 Probabilistic distribution of RC member drift and its deterministic calculation

With the establishment of the probabilistic models for the RC section curvature and the model uncertainty for the plastic hinge length as described in the preceding sections, the probabilistic evaluation of the limit member drift ratios at different performance levels becomes straightforward using Monte Carlo Simulation based on Equations 3.5, 3.8 and 3.10. For simplicity and without losing generality, the member length to the contra-flexure point, Z_c , is assumed to be half of a typical storey height (representing a frame column) and equal to 1600 mm.

Figure 3.12 plots the typical histograms of the limit drift ratios at the three performance levels. All the probability distributions are tested to satisfy the normal distribution. A summary of the mean and COV of the limit drift ratios for all specimens are given in Table 3.8. Figure 3.13 shows the variation of the mean drift ratios with the reinforcement details at different performance levels.

At the limit state for Functional performance level, the drift ratios are dominated by the elastic deformation while the plastic rotation is still insignificant; hence, the drift ratios exhibit a small COV ($\leq 13.6\%$) and the mean value varies in a fairly narrow range from 1.03% to 1.34%. Thus, it appears to be reasonable to combine all results into a single group. The overall mean Functional limit drift ratio is found equal to 1.18% with a COV of 8.7%.

Table 3.8 Limit drift ratios for different performance levels

Specimens	Axial force ratio	Functional		Damage Control		Collapse	
		Mean	COV	Mean	COV	Mean	COV
B	0.1	0.0108	10.9%	0.0327	31.8%	0.0581	31.0%
	0.2	0.0106	12.2%	0.0262	36.7%	0.0438	42.8%
	0.4	0.0093	8.1%	0.0174	32.9%	0.0300	40.6%
D	0.1	0.0113	13.3%	0.0314	38.0%	0.0557	40.7%
	0.2	0.0107	12.0%	0.0250	37.4%	0.0409	43.1%
	0.4	0.0096	8.4%	0.0191	35.2%	0.0336	42.5%
E	0.1	0.0117	12.5%	0.0292	38.9%	0.0473	41.3%
	0.2	0.0109	11.3%	0.0265	39.4%	0.0432	44.6%
	0.4	0.0099	8.3%	0.0206	37.3%	0.0370	45.4%
F	0.2	0.0106	11.5%	0.0367	37.2%	0.0622	42.8%
	0.4	0.0097	8.3%	0.0298	39.3%	0.0534	44.5%
G	0.2	0.0106	11.1%	0.0440	30.9%	0.0782	34.9%
	0.4	0.0097	8.1%	0.0410	35.4%	0.0767	40.9%

Due to large inelastic deformation, the limit drift ratios at DC performance level vary considerably among different specimens, with mean values in a range of 1.7% ~ 4.4% for axial force ratio equal to 0.2~0.4, while the COV vary between 30 to 40% in most cases. In general, the limit drift ratio decreases with the increase of the axial force, and it increases progressively with the increase of the confining reinforcement. On the other hand, the variation of the longitudinal reinforcement ratio appears to have less effect on the drift ratio, especially for higher axial force cases, for similar reasons as explained with the sectional curvature.

At the ultimate state for Collapse performance level, the COV of the ultimate drift ratios slightly further increased from the DC level to a range of 30~45%. The mean ultimate drift ratios vary in the range of 3.0% to 8.0% for different specimens.

Functional level (Axial force ratio = 0.2)

Functional level (Axial force ratio = 0.4)

Damage Control level (Axial force ratio = 0.4)

Collapse level (Axial force ratio = 0.4)

Figure 3.12 Typical histograms of simulated drift limits for different performance levels

Figure 3.13 Variation of mean limit drift ratios for different performance levels with reinforcement details and axial force ratios

The drift ratios are also predicted using the aforementioned deterministic approach, with a plastic hinge length equal to the value calculated according to Equations 3.8 and 3.10 without considering the model uncertainty. Table 3.9 summarizes the deterministic calculation results and the corresponding probability of being exceeded by the actual limit drift ratios. The above probability results are also plotted in Figure 3.14 to show their scatter.

Table 3.9 Drift ratio limits by deterministic approach and associated probability

Specimens	Axial force ratio	Functional level	Prob. of being exceeded	Damage Control	Prob. of being exceeded	Collapse level	Prob. of being exceeded
B	0.1	0.0113	31.5%	0.0379	31.0%	0.0679	29.5%
	0.2	0.0101	63.1%	0.0240	60.3%	0.0363	64.1%
	0.4	0.0095	42.0%	0.0173	52.7%	0.0262	63.5%
D	0.1	0.0114	55.9%	0.0305	55.4%	0.0527	56.1%
	0.2	0.0103	62.2%	0.0246	53.4%	0.0402	52.2%
	0.4	0.0097	45.3%	0.0173	61.7%	0.0292	63.3%
E	0.1	0.0114	56.5%	0.0298	48.4%	0.0479	49.3%
	0.2	0.0105	61.6%	0.0270	49.1%	0.0446	47.8%
	0.4	0.0099	47.7%	0.0186	61.8%	0.0321	62.9%
F	0.2	0.0102	65.0%	0.0370	50.3%	0.0616	52.0%
	0.4	0.0097	52.2%	0.0292	53.7%	0.0456	63.3%
G	0.2	0.0106	65.8%	0.0528	25.7%	0.0950	27.4%
	0.4	0.0098	48.2%	0.0437	43.6%	0.0737	54.8%

(a) Functional level

(b) Damage Control level

(c) Collapse level

Figure 3.14 Scatter plots of probability of actual drift limits exceeding the values predicted by deterministic approach

Similar to the observation from the curvature calculations, the probability of the deterministic limit drift ratios being exceeded by the actual limit values are not uniform under different design conditions, even at the same performance level. In most cases, such probability falls in the range of 45%~65%; however, for cases where the performance limit is more likely to be governed by the reinforcement strain, the above probability drops to only about 25% for reasons explained before.

3.7.2 Verification of ultimate state drift limit against experimental data

Unlike other performance levels for which few experimental drift data are available in the literature, experimental data on the Collapse limit (usually referred as ultimate state) drifts have been widely reported, although the actual attainment of “ultimate” state may sometimes be subjective. Dymiotis et al. (1999) have conducted a rather thorough survey of experimental data pertaining to the ultimate state inter-storey drift, and the distribution of 76 test results for axial force $v \leq 0.5$ are plotted in Figure 3.15. Because of the diverse nature of the various test structures, it is not possible to divide the data into specific groups with respect to the axial force and reinforcement details. Hence, to enable a comparison, the current simulation results for the various specimens and under axial force ratios of $v=0.2\sim0.4$ (excluding $v=0.1$ which is considered very low for actual columns) are also combined, with two combination schemes. One consists of all specimens and another consists of specimens *B*, *D* and *E* to represent more evenly sampled transverse reinforcement (in the range of 0.81~2.09%). The overall distributions of the combined simulation results are compared with the experimental distribution in Figure 3.15. In view of the diversity of the experimental methods and standards involved, the comparison can be regarded as very reasonable. Similar to the reported log-normal distribution of experimental data Dymiotis et al. (1999), the combined simulation results were found to follow log-normal distribution as well.

Figure 3.15 Comparison between simulated and experimental results

3.7.3 Comparison between simulated drift limits and code specified limits

Bertero et al. (1991) carried out a review on the drift limits for both serviceability and ultimate state specified by various design codes. EC8 (1996) and SEAOC (1995) also suggested drift limits for performance levels. The relevant information is summarized in Table 3.10. To make a comparison, the drift limit values with a probability of being exceeded as 95% are tabulated in Table 3.11 based on corresponding probabilistic models.

From Table 3.11, it is found that the Collapse drift limits range from 0.94% to 3.33%, which is close to the range of 1.0%~3.0% for ultimate state from Bertero (1991) and also cover the drift limit interval of Near Collapse (1.5%~2.5%) and Collapse (>2.5%) suggested by SEAOC (1995). With such a favorable agreement with both the experimental and code-specified ultimate state drifts, the simulated probabilistic model of Collapse drift limits from the column member analysis is deemed to represent reasonably the actual ultimate inter-storey drift limits in RC frames.

Table 3.10 Drift limits from design codes

Limit state	Reference	Critical inter-story drift (%)	Notes
Serviceability	EC8 (1996)	0.4	Non-structural elements of brittle materials attached to the structure
		0.6	Non-structural elements fixed so that they do not affect structural deformations
	Bertero et al. (1991)	0.06~0.6	State-of-the-art range considering various seismic codes
		0.3	Based on field observations
Ultimate	Bertero et al. (1991)	1.0~3.0	State-of-the-art range considering various seismic codes
Fully Operational		<0.2%	-
Operational		0.2%~0.5%	-
Life safety	SEAOC (1995)	0.5%~1.5%	-
Near collapse		1.5%~2.5%	-
Collapse		>2.5%	-

Table 3.11 Drift limits corresponding to probability of being exceeded of 95%

Specimens	Axial force ratio	Functional	Damage control	Collapse
B	0.1	0.0089	0.0156	0.0285
	0.2	0.0085	0.0104	0.0130
	0.4	0.0081	0.0080	0.0100
D	0.1	0.0088	0.0118	0.0184
	0.2	0.0086	0.0096	0.0119
	0.4	0.0083	0.0080	0.0101
E	0.1	0.0093	0.0105	0.0152
	0.2	0.0089	0.0093	0.0115
	0.4	0.0085	0.0080	0.0094
F	0.2	0.0086	0.0142	0.0184
	0.4	0.0084	0.0105	0.0143
G	0.2	0.0087	0.0216	0.0333
	0.4	0.0084	0.0171	0.0251

However, the Functional drift limits in Table 3.11, ranging from 0.81% to 0.93%, are generally two times the code specified values for serviceability (Bertero 1991, EC8 1996), or Operational (SEAOC 1995). In fact, for such intermediate

performance levels, the damage to typical non-structural components incurs at a much lower level of deformation than that could induce structural damage, and the non-structural damage could play an important role in the whole performance of buildings. This may explain why the code specified drift limits tend to be much lower than those obtained in the present study based on structural components only.

It is noted from Table 3.11 that for specimens B, D and E at an axial force ratio equal to 0.4, Functional drift limits corresponding to probability of being exceeded of 95% are larger than Damage control ones. Numerically, this is because in high axial load cases, the mean values of Functional and Damage control drift are relatively close but the COV of Functional drift is much lower than that of Damage control drift. This is more a statistical phenomenon.

Quantitative definition of performance levels requires the relevant information of non-structural components. In the next chapter, a fuzzy random model incorporating non-structural damage in the performance evaluation will be introduced.

3.8 Summary and conclusions

In this chapter, three performance levels, rather than the five levels proposed in SEAOC (1995), are defined and applied in the evaluation of column deformation capacity. It is deemed to be more useful for practitioners. Taking into account the random material properties and model uncertainty factor of ultimate strain of confined concrete, the probabilistic sectional curvature of typical RC column sections at the three performance levels is evaluated through a rigorous statistical simulation study, which has not been done systematically in the literature. A statistically more reliable expression of equivalent plastic hinge length is also proposed on the base of a quite extensive test data survey, along with a probability model for the uncertainty factor that has not been available before. Subsequently, the probabilistic inter-storey drift limits of RC columns at different performance levels are established based on the probabilistic model of sectional curvature and the plastic hinge length. The drift limits may be directly applicable to frame cases

with a storey height on an order of 3~4m. For frame storeys whose dimensions are significantly different from the above range, the absolute drift limits may vary; however, the probabilistic distribution from the above analysis should still apply. Finally, the reliability of the deterministic calculations of the respective drift limits is assessed. Based on the results obtained in this chapter, the following conclusions may be drawn:

- a) The RC member behavior at the Yield level has a small variability. The *COV* of the yield curvature and yield moment is in the range of 7~10% and 10~15%, respectively. The *COV* of the limit curvature at the Functional performance level slightly increases to about 15~20%. The Functional limit drift ratio, after combining the results from all specimens, gives a mean of 1.18% and a low *COV* equal to 8.7%.
- b) Due to the involvement of confined concrete, the variability of the sectional curvature and member drift at the limit for Damage Control level increase considerably. The *COV* is in the range of 40~45% for higher axial force cases ($v=0.2\sim0.4$), and it reduces to the order of 30% at low axial force ratio ($v=0.1$). The variability of the limit curvature and drift increases slightly further at the Collapse level. The limit curvature and drift at both DC and Collapse levels can be approximated by the normal distribution.
- c) The predicted limit curvature and drift values using the deterministic approach with characteristic material strengths and nominal limit strain values are shown to have a probability of being exceeded by the respective actual capacities in a range of 50%~70% for most cases at all performance levels. However, care should be exercised in predicting the limit curvature and drift using the above deterministic approach for cases where the state is likely to be governed by the reinforcement strain, for example with a low longitudinal reinforcement ratio (e.g., 0.81%) and at low axial force level (e.g., $v=0.1$), or at relatively lower axial force (e.g., $v=0.2$) but the confining reinforcement ratio is high. The results indicate that the deterministic calculations in these cases could result in only 25~30% probability of being exceeded by the actual limit capacities. In such cases, a reduction factor may have to be considered on the deterministic predictions in order to maintain a consistent reliability level.

- d) By comparison, it is found that the obtained Collapse drift limits correlate well with the relevant experimental data and design code specifications. However, the code specified drift limits for serviceability and Operational level appear to be much less than those obtained in this study based on structural performance alone. This may be explained by the fact that non-structural damage, which is somehow given consideration of at the Code level, usually could incur at lower deformation than structural damage concerning service level performance. To describe the performance of entire building systems appropriately, the non-structural performance should be included in the quantitative definition of performance levels, as will be discussed in the next chapter.

Chapter 4 Fuzzy-random Reliability Model for Comprehensive Performance Analysis of Framed Structures

4.1 Introduction

A modern building system is usually composed of basic structural system which constitutes the mechanical skeleton of the building, and non-structural system which provides partitioning, utility and functions to building occupants. Attacked by a ground shock, both the structural and non-structural systems would suffer damage to certain extents. Therefore, the seismic performance of a building system is a combined outcome of the performance of both categories of building components.

In many moderate to strong earthquakes that have struck the world since the beginning of the twentieth century, the cost for damage to non-structural building components has exceeded that of structural damage in most affected buildings. Moreover, damage to non-structural components may also pose danger to life safety as well as functional and monetary impairment. Even if the structural components of a building achieve an immediate occupancy performance level during a seismic event, non-structural damage to the building can lower the performance level of the entire building system. In line with the development of performance-based earthquake engineering, harmonization of the performance levels between structural and non-structural components is necessary.

In the first part of this chapter, some general information about the categorization of non-structural components, non-structural damage and the corresponding performances will be discussed briefly. Due to the limited information about the general non-structural behavior, the masonry infill wall, which constitutes a significant source of dollar losses in earthquakes, is deemed as the representative of

general non-structural components in the present study. The probabilistic models for the lateral drift limits for the performance of masonry infill walls at three levels are established based in a statistic analysis of the collected experimental data.

Taking into account the damage to both structural components and non-structural components, a performance limit state has two limit criteria corresponding respectively to the random structural and non-structural deformation limits. Applying the classical reliability theory, the failure probability against both structural and non-structural criteria can be calculated when a probabilistic displacement demand is given. However, due to the significant difference between the structural and non-structural drift limits, one has to weigh the relative significance of these two aspects and it has to be done in a consistent manner. Even when one limit is more important than the other one, it is still more appropriate to give consideration to the other aspect than simply adopting a single-limit dominating case. From this point of view, the pure probabilistic methods are not sufficient for the purpose of performance evaluation and thus the fuzzy set theory is introduced.

Enlightened by the fuzzy set theory, this study proposes to define each performance limit state as an interval of inter-storey drift ratios concerning respectively the non-structural and structural performances with a membership function, while the relative importance of the two aspects can be reflected through the use of an appropriate cost function. By a fuzzy-random reliability model proposed in this chapter, both randomness and fuzziness involved in the definition of performance levels of whole building system are taken into account and a synthesized reliability evaluation is feasible.

4.2 Non-structural components

Non-structural components of buildings play a crucial role in performance-based earthquake engineering. In most types of buildings, non-structural components represent a major part of total cost, as well as the potential losses to owners, occupants and insurance companies. For example, in the case of commercial

buildings, non-structural components typically account for 65% to 85% of the total cost of building (Taghavi & Miranda 2003). From the engineering point of view, damage to most types of non-structural components is triggered at a much lower level of deformation than that could induce structural damage. For example, damage to brittle infill walls often begins at low inter-storey drift level at which no structural damage could happen. Furthermore, significant monetary losses may take place due to the interruption of function of the building as a result of substantial non-structural damage. Therefore, information regarding the performance of non-structural components is required in order to implement the performance-based seismic engineering.

4.2.1 Classification of non-structural components

The performance of non-structural components can be studied from different perspectives such as their functionality in buildings, their effects on the building performance, their mechanism to get damaged, their sensitizations to different structural responses and consequences resulting from their damage. However, a systematic classification of non-structural components based on various criteria is out of the scope of the present study. In the following paragraphs, a basic classification is described to provide a general understanding of non-structural components based on Lagorio (1990).

A building can be identified as having three principle components critical to the seismic performance: (1) the basic structural system, (2) its non-structural elements, and (3) its contents. The basic structural system consists of a) vertical support components (columns, piers, bearing walls, foundations, etc); b) horizontal span members (floors slabs, beams, girders, rafters, truss, space frames, etc.); and c) any other structural element used for supporting the building's basic live loads and dead loads. Non-structural components of a building include all those parts of the total building system that are not part of the basic structural system. The basic structural system of a building is designed to withstand all static live and dead loads, as well as all dynamic loads such as winds and earthquakes, while the non-structural components are predominately inserted into the building during the final stages of construction.

More specifically, non-structural components include all the architectural components (e.g., cladding, ceilings, partitions, doors/windows, stairs, furnishings and equipments, contents, parapets, canopies, etc.), in addition to all mechanical, electrical, and plumbing components (e.g., elevators, lights, pipings, ducts, HVAC systems, escalators, security systems, fire protection systems, telephone and communication systems, computer equipment, etc.), whether on the exterior or interior. A list of the principal categories of non-structural elements is given in Table 4.1. (based on Lagorio 1990)

Table 4.1 Representative categories of non-structural components

Categories	Details
Exterior Elements	Cladding, veneers, glazing, infill walls, canopies, parapets, cornices, appendages, ornamentation, roofing, louvers, doors, signs, detached planters, etc.
Interior Elements	Partitioning, ceilings, stairways, storage racks, shelves, doors, glass, furnishings (file cabinets, book cases, library stacks, display cases, desks, chairs, tables, lockers, etc.), ornamentation, detached planters, artwork, etc.
Mechanical/Electrical/Plumbing elements	HVAC equipment, elevators, piping, ducts, electric panel boards, life-support systems, fire protection systems, telephone/communication systems, motors/power control systems, emergency generators, tanks, pumps, escalators, boilers, chillers, fire extinguishers, controls, light fixtures, etc.
Contents	Electronic equipment, data-processing facilities, medical supplies, blood bank inventories, high-tech equipment, hazardous and toxic materials, antique/fine arts (museums and art galleries), office equipment, radios, life-support equipment, etc.

4.2.2 Non-structural damage and its effects

Damage to non-structural elements of a building is generally caused in two ways (Lagorio 1990): 1) By differential movement and distortion of the primary structures, and 2) By the shaking and overstressing of the elements themselves, either in-plane or out-of-plane.

Distortion-related damage may occur to any non-structural component when it is not able to withstand to loads caused by the deformations of the basic structure. For example, brittle infill walls, continuous stairways between several floors, or inflexible pipe risers between two or more floors are at high risk to damage caused

by extreme building movements during a major earthquake. Furthermore, a full-height partition element may be crushed between floors due to excessive inter-storey drift.

Shaking-related damage is basically caused by the inability of a non-structural component to respond well to overall shaking or the vibratory motions induced by the basic structure. Failure will occur in a non-structural component when overstressed due to vibrating internally, overturning, sliding, oscillating, and/or swinging back and forth.

Certain extents of damage to various non-structural components will result in different effects on the whole building systems. According to Lagorio (1990), these effects can be divided into two main categories, i.e. issues on life safety and functional impairment.

Representative examples of life safety hazard due to non-structural damage may include: (1) collapse of suspended light fixtures; (2) falling shards of shattered glass; (3) falling exterior cladding materials; (4) overturning of heavy equipment; (5) falling parapets; (6) collapse of hung ceilings; (7) overturning of book shelves, storage racks, and library stacks; (8) collapse of infill walls; (9) falling wall-mounted ornamentation/artwork; and (10) rupture of toxic chemical containers. In an indirect manner, failure of life-support systems in hospitals will delay the treatment on injuries and may consequently claim more life.

Damage to non-structural components may also lead to interruption of function. Moreover, there are economic concerns. The potential of having several buildings in an investment portfolio functionally impaired and out of service for a long period of time, with extended loss of income, owing to major non-structural damage is also unacceptable to clients.

Furthermore, it is widely known that relatively rigid non-structural components can adversely affect the dynamic characteristics and seismic response of the basic structure when they are placed into the building system without any regard to their

influence on structural performance. For instance, different damage states of solid infill masonry walls will significantly affect the lateral stiffness of RC frame structures, and the corresponding dynamic characteristic as well.

4.2.3 Probabilistic drift capacity for masonry infill walls at performance levels

So far, the discussions on the characteristics of non-structural components are limited to qualitative terms. To take the non-structural performance into account in a reliability assessment, a quantitative description of the non-structural performance is required. There are two possible ways to quantify non-structural performance, one is to use experimental data from relevant experimental studies and another based on statistic data from post-event damage investigations. However, due to the general lack of attention to the non-structural damage in the past, there are very limited data available in the literature. An exception to this situation is that, as an essential part of the non-structural components, masonry infill walls have been studied extensively in terms of its strength and deformation capacities under in-plane lateral load. Since the infill and partition masonry walls constitute a significant source of dollar losses in earthquakes (Taghavi & Miranda 2003), it is rational to use these components as representative for the performance and loss modeling of general non-structural components.

Definition of performance levels of masonry infill walls

As discussed in Chapter 3, three damage levels, namely Functional, Damage Control and Collapse, are deemed to be adequate in defining the performances with significant physical implications. Similar distinctions may be applied to masonry walls and the limit states may be identified according to the damage states in terms of the drift from experimental data, as follows. The definitions are also graphically shown in Figure 4.1.

1) Functional level: Minor cracks are observed and small pieces of plaster fall.

Based on experimental results, most of the “First (noticeable) crack” points correspond to the first noticeable deviation from the initial increasing portion of the

curve. In most cases, the “First crack” drift is very small and the damage to the masonry walls at this stage is generally too slight to be considered as a limit concerning the functionality of the building. On the other hand, when the peak lateral load is reached, many noticeable cracks with limited length and width along the diagonal and around the foot corner occur. Although generally speaking the damage to masonry walls at this state is still not so significant, some local repair work may be necessary. However, the basic building function is maintained and the economic loss should be small. For these reasons, the point corresponding to the peak lateral load is chosen as the Functional limit state and the corresponding drift value is taken as the Functional drift limit.

2) Damage Control level: Significant cracking and flaking are observed but stability is retained.

The key feature of Damage Control level is “repairable”. Following the first crack in the masonry walls, cracks intensify gradually under the increasing racking force. But the cracked masonry wall is deemed repairable until a major diagonal crack comes into sight. Therefore, the occurrence of continual cracks along the diagonal is considered to be the physical sign of this performance level.

However, few experimental studies have identified the above state on the lateral load - displacement curves, although it seems clear that this particular state falls into the descending portion of a load-deflection curve. In the present study, the Damage Control drift for masonry infill panels is assigned to correspond to 70% of the peak load on the descending branch.

3) Collapse: The wall panel loses its stability.

The Collapse level is defined as the drift at which the wall panel loses its stability, implying that the wall approaches collapse state. For wall panels with a boundary frame, the ultimate drift for collapse may be defined by a substantial reduction of the lateral resistance with however an appreciable residual strength. Herein the ultimate drift is defined by 70% reduction (or 30% retention) of the peak strength

on the descending portion, similar to that adopted by some other researchers (Algan 1982).

Figure 4.1 Definition of performance drift limits for masonry walls

Survey and statistic analysis of experimental data

A survey on available experimental data concerning the in-plane behavior of masonry walls was performed. The sources of the test data are broad. As a result, there are sometimes remarkable differences in the details of these test programs, such as the material types (masonry units and mortar), specimen scales, specimen dimensions, boundary conditions (with or without a boundary frame), reinforced or not reinforced and reinforcement details, loading programs, and workmanship. Therefore, it is necessary to categorize all the collected data before a statistical analysis is carried out.

Since few test programs used small scale models for masonry walls, the difference in scales of specimens is neglected. For material types, the types of masonry units, e.g, clay bricks, concrete blocks, are considered in the grouping; however, the differences in mortar are not considered. The reinforcement condition is only divided into unreinforced and reinforced. The variation in the details of reinforcement was not considered. For boundary conditions, it is grouped into “no frame”, “RC frame” and “steel frame”. Different sectional details of boundary frames are not taken into account. The loading scheme is also a very important

factor. For vertical loading, specimens with vertical stress lower than 0.1 times of masonry compressive strength is grouped into “no or low vertical load”. Otherwise they are grouped into “higher vertical load”. The lateral load scheme is divided into “monotonic, static”, “cyclic, static” and “dynamic”.

According to the above grouping scheme, 15 groups of specimens are obtained as summarized in Table 4.2. The number of specimens for which the test data are available for identification of the limit drifts for individual performance levels are also shown in the table.

Table 4.2 Categorization of specimens

Group	Description	Number			
		Specimens	Func.	DC	Ultimate
1	Clay bricks, un-reinforced, no frame, monotonic loading, no or low vertical load	7	7	0	0
2-a	Clay bricks, un-reinforced, steel frame, monotonic loading, no or low vertical load	8	8	4	0
2-b	Clay bricks, un-reinforced, RC frame, monotonic loading, no or low vertical load	47	47	6	17
3	Clay bricks, un-reinforced, no frame, cyclic loading, no or low vertical load	15	13	5	5
4	Clay bricks, un-reinforced, frame, cyclic loading, no or low vertical load	8	8	4	2
5	Clay bricks, reinforced, no frame, monotonic loading, no or low vertical loading	6	6	0	0
6	Clay bricks, reinforced, no frame, monotonic loading, high vertical loading	3	3	0	0
7	Clay bricks, reinforced, frame, monotonic loading, no or low vertical loading	1	1	0	0
8	Clay bricks, reinforced, no frame, cyclic loading, no or low vertical loading	13	13	1	0
9	Clay bricks, reinforced, no frame, cyclic loading, high vertical loading	11	11	5	0
10	Clay bricks, reinforced, frame, cyclic loading, no or low vertical loading	3	3	1	0
11	Concrete blocks, un-reinforced, no frame, monotonic loading, no or low vertical loading	6	6	0	0
12	Concrete blocks, un-reinforced, no frame, monotonic loading, high vertical loading	7	7	0	0
13	Concrete blocks, reinforced, no frame, monotonic loading, no or low vertical loading	6	6	0	0
14	Concrete blocks, reinforced, no frame, monotonic loading, high vertical loading	2	2	0	0
15	Concrete blocks, reinforced, no frame, cyclic loading, no or low vertical loading	6	6	0	0
Total number		148	146	26	24

The details of all the test programs are also tabulated in Table A.1 (see Appendix A). It is noted that some of the data are cited from Algan (1982), who had done a survey on masonry walls concerning damage at the maximum loading and ultimate stages.

Totally 148 sample specimens are collected. However, the descending portion after peak load was not provided in many cases. For those cases, it is not possible to identify the limit drifts for Damage Control and Collapse levels from the load-displacement curves. Consequently, the sample sizes for the Damage Control and Collapse limits are much smaller than that for Functional limit. To overcome this problem and expand the sample size as much as possible, several groups with properties closer to each other, such as groups 1, 2-b, 3, and 4, are merged. The merged group represents un-reinforced specimens made by clay bricks without or with only low vertical loading. The effect of bounding frame is neglected as it is partly taken into account by the different peak load reduction factor for frame with/without infill masonry (see Figure A.1, Appendix A). Different lateral loading programs are also combined in this category. Despite the merging, the sample size of data for Damage Control and Collapse levels are still only around 20. The statistical properties of the so obtained limit state drifts for the three performance levels are summarized in Table 4.3.

Table 4.3 Statistical properties of test data from literature survey

Performance levels	Composing categories	Number of specimens	Total number	Mean value	COV	Probability distribution
Functional	1	7	74	0.254%	82.8%	Log-normal
	2-b	46				
	3	13				
	4	8				
Damage Control	2-b	6	15	0.541%	74.8%	normal/ log-normal
	3	5				
	4	4				
Collapse	2-b	17	24	0.801%	43.1%	Normal
	3	5				
	4	2				

The variability of both Functional and Damage Control drifts is quite large which manifests by itself the great uncertainty in the definition of intermediate performance levels for masonry infill walls, as well as the uncertainties in material

properties, loading schemes and workmanships in the various test programs considered. Log-normal distribution is found to be suitable for describing the probabilistic properties of the drift limits for Functional and Damage Control levels. For Damage Control and Collapse limits, normal distribution is also a good alternative. The histogram of Functional drift limits is plotted in Figure 4.2 as an example.

Figure 4.2 Histogram of Functional drifts for masonry wall

Comparing with the drift limits of RC columns as described in Chapter 3, the drift limits of the masonry infill walls are considerably lower for a comparable level of performance. This implies that if a performance level is stipulated according to the limit state of structural components only, damage to the non-structural components would be unacceptably severe. Therefore, some compromises are inevitable. This further explains why the code specified drift limits appear to be much lower than the drift limits obtained for RC structural components, as mentioned in Chapter 3. In this regard, it is more desirable that some rigorous procedure be devised to take into account the differences in the limit states for structural and non-structural components concerning the performance of a building system. This prompts the development of the fuzzy-random model for performance reliability analysis, as will be presented in the sections that follow.

4.3 Defining a performance level concerning both structural and non-structural components

The performance of a building system is rated depending on the damage state of the entire building. In the context of the performance-based design, the non-structural building components are emphasized especially in the performance levels mainly concerning the building function after a medium ground shaking. Generally speaking, the non-structural components are more sensitive to building deformation than the structural components. For example, the partition wall may undergo considerable damage even before the reinforcement of column yields. Therefore, a single-valued threshold damage index is not capable to symbolize certain performance level.

Horizontal displacement response of a building subjected to earthquake motions is generally referred to as “drift”. The drift value Δ or the drift ratio θ (defined as the ratio of Δ to the vertical distance between measuring points) are the observable response quantities that relate to both the structural and non-structural performance of buildings subjected to earthquake motions. Hence, the drift ratio is chosen as the damage index of different performance levels in this study.

As mentioned previously, the masonry wall is considered as the representative of non-structural building components. Based on the extensive literature survey on the deformation behavior of masonry wall panel under lateral testing load, a fairly large set of drift ratio values corresponding to three main performance levels were extracted and the corresponding probabilistic properties were also evaluated. Comparing to the corresponding drift ratio limits from the Monte Carlo Simulation for RC members (Chapter 3), there is significant deviation between these two sets of limit drift ratios for non-structural and structural components at each performance level, as depicted in Figure 4.3. It is therefore suggested in this study to define a performance limit state as an interval of drift ratio which has a lower bound and a higher bound corresponding respectively to the specific non-structural and structural drift ratio limits. In probabilistic context, both the non-structural and

structural drift ratio limits are obviously random variables, which in turn will affect the random property of the proposed model.

Figure 4.3 An example of lower and upper drift limits (distributions) defining “Damage Control” performance level

4.4 The fuzzy-random model

4.4.1 Fuzzified definition of performance level

So far, the performance level defined as an interval of drift ratio is a crisp set of drift ratios in mathematical language. In the conventional crisp set theory, the membership of an element can be either 1, which means the element fully belongs to the set, or 0, which means the element does not belong to the set and nothing in between. However, can all drift ratio values fall in the interval of a performance level be considered with the same full degree of membership to a set, such as occurrence of the performance level? The answer is negative because of the varying significance of each pertinent aspect (say, non-structural and structural damage) contributing to a particular performance target. The interval employed here to define a performance level should not be a conventional crisp interval but a fuzzy interval upon which definitions of various fuzzy events or sets could be based. In a fuzzy set, various degrees of membership from 0 to 1 for the elements are allowed to account for the situations in which vagueness or imprecision exists.

By combining the fuzzy and random properties of a performance level, a fuzzy-random model can be developed. The first step to introduce fuzzy set theory in the intending reliability model is to define a fuzzy-random event (or a fuzzy-random set) relevant to drift ratio and performance levels. As previously mentioned, the random property of this model can be attributed to the random drift limits of non-structural and structural components. The fuzzy feature can be attributed to the considerable difference between the drift limits of non-structural and structural components for a performance level. In terms of drift ratio, two types of fuzzy-random events are defined as

1. **Occurrence** of the limit state for a performance level; and
2. **Exceeding** of a performance level.

According to the fuzzy set theory, different elements in any fuzzy set defined above may have different degrees of membership. The relationship between these two fuzzy sets will be discussed later.

4.4.2 Membership function

Zadeh (1965) has given in his famous paper the following definition: *A fuzzy set is a class with a continuum of membership grades.* Therefore, a fuzzy set F in a referential U is characterized by a membership function which associates each element u in U with a real number in the interval $[0, 1]$. The value of the membership function at element u represents the “grade of membership” of u in F . The membership function of a fuzzy set F will be denoted as $\mu_F(u)$ in this study.

The evaluation of membership values is of critical importance in applications of fuzzy set theory to engineering area. In most cases the most representative members in a set are assigned the value of 1 while non-members the value of 0. Consequently, in-between members are assigned values from the open interval $(0, 1)$.

Membership of an element in a fuzzy set reflects the relative degree that the element belongs to the set. According to relevant knowledge and information, membership of a finite subset of elements can be specified. However, for a common real-valued variable, the remaining elements must have their membership determined by some means of interpolation. In this study, the evaluation of

membership function is divided into two steps. The first step is to determine the elements with membership of 0 and 1. Subsequently, the second step is to evaluate the grade of membership of other elements in-between the 0-valued and 1-valued points. Before proceeding to evaluate the membership function, it is necessary to discuss its general property first.

General property of the membership function

In this section, the general properties and some basic assumptions of the membership functions are discussed briefly based on Chen & Otto (1995). This provides some useful preliminary information about the evaluation of membership function. Details of the relevant fuzzy set theory can be found from Zimmermann (1996) or other standard texts.

Axiom 1. *A membership function is a real function bounded in $[0, 1]$.*

Axiom 2. *Among all points in a fuzzy set, at least one will be a most representative point with membership 1, and at least one will be an ineligible point with membership 0.*

The method to elicit membership function values is based on these two axioms. The best and worst elements are assigned membership values 1 and 0, respectively. The remaining are assigned membership values between these two extremum.

Assumption 1. *The rate of the membership values vary in a domain is continuous.*

There are two implications from this assumption. The first one is the membership function itself is continuous and differentiable. The second one is that its first order derivative is also continuous. And accordingly, it is found that:

Proposition 1. *If $\mu(x)$ is 0 or 1, then $\mu'(x) = 0$.*

Proof: There are two probable cases corresponding to the situation that $\mu(x)$ equals to 0 or 1:

1. x is a local maximum or minimum, or
2. Either points in the right- or left-hand neighborhood of x have the same value of $\mu(x)$.

In case 1, it is obvious that $\mu'(x) = 0$. In case 2, $\mu'(x)$ must be zero since μ is differential and μ' is zero in the neighborhood of x .

Assumption 2. *The membership function $\mu(x)$ is convex, i.e.*

$$\mu(\lambda x_i + (1 - \lambda)x_j) \geq \min\{\mu(x_i), \mu(x_j)\}, x_i, x_j \in X, \lambda \in [0, 1]$$

General shape of the membership function curve

Once the fuzzy interval of drift ratio, $[\theta_l, \theta_2]$, is determined for the limit state of a performance level, there may be numerous fuzzy sets defined on it for different purposes by assigning different membership function to their elements. As defined previously, there are two types of fuzzy sets to be involved in this study. Subsequently, there will be two corresponding membership functions, respectively. In this section, the graphic feature of these two functions, as well as the relationship between them, will be discussed.

The first type of fuzzy set under consideration is **Occurrence** of a performance limit state. It is believed that after a ground shaking, the occurrence of any drift value which falls into the interval represents the occurrence of a performance limit state to a certain degree (with the membership values between 1 and 0) and any drift value located outside is not considered as a member of the performance level (with membership value of 0). And it is also postulated that there will be one and only one most representative point $\underline{\theta}$ with full membership value of 1 (the assumption of convex). Therefore, the membership function $\mu_o(\theta)$ curve may have a shape similar to the one plotted in Figure 4.4a.

The second type of fuzzy set is **Exceeding** of a performance level. The corresponding membership function is supposed to be monotonously increasing in (θ_l, θ_2) . Any drift value greater than θ_2 is considered to fully exceed the performance level (with membership value of 1) and any drift value less than θ_l is considered to have no possibility of exceeding the performance level (with membership value of 0). The event of any drift value falls into the interval represents to a certain degree the exceeding of the performance level (with the

membership values between 0 and 1). Therefore, the membership function $\mu_E(\theta)$ curve may have a general shape as shown in Figure 4.4b.

a) Membership function for Occurrence

b) Membership function for Exceeding

Figure 4.4 Probable shape of membership function curve of Exceeding

Comparing the membership function curves to those of probability density function and cumulative distribution, it is found that there are some similarities between these two sorts of curves, such as the monotonous property and convex shape. Enlightened by the relationship between the probability density function and cumulative distribution, the membership function of **Exceeding** of a performance

level is defined as the normalized integral of the membership function of **Occurrence** of the performance limit state and the point $\underline{\theta}$ on $\mu_o(\theta)$ with membership value of 1 is supposed to correspond to the zero curvature of $\mu_E(\theta)$. Thus, the membership function $\mu_o(\theta)$ has to be constructed first.

Determination of the 1-valued membership point in $\mu_o(\theta)$

The first step to evaluate the membership function is to determine the point with non- and full-membership. According to the definition of the fuzzy event **Occurrence**, it is obvious that any point with drift ratio θ falls outside of the open interval (θ_l, θ_2) should be assigned 0-valued membership. Determination of the full-membership point requires an elaboration of the possible meaning of this point.

The lower and upper bounds of the fuzzy interval, θ_l and θ_2 , are related respectively to the random drift ratio limits of non-structural and structural components. For a certain performance level, a drift ratio value with membership of 1 implies it is the most representative point of this performance level. It is reasonable to set the point at the middle of the drift ratio interval if the non-structural and structural components are considered equally important for the performance level concerned. However, if the non-structural part is considered to be more important than the structural part at a particular performance level, the point with 1-valued membership should be closer to the non-structural drift ratio limit than to the structural side, and vice versa. The relative importance of non-structural and structural components varies for different performance levels because there are different focuses on the damage state at different performance levels and it also depends on the building category and the corresponding function. Therefore, the most representative points vary for different kinds of buildings and at different performance levels.

Failure cost may be used to measure the damage state of a building system. Kanda & Shah (1997) suggested a simple continuous failure cost function as follows:

$$y = \left(\frac{x - x_0}{1 - x_0} \right)^\gamma C_u \quad (4.1)$$

where y is the cost at damage state x , x_0 denotes the initial damage state, C_u is the cost at ultimate limit state at which x equals to 1, and γ is a constant which affects the shape of the cost function curve. Figure 4.5 illustrates the cost function curves with various values of γ . It is recommended by Kanda & Shah (1997) that small γ values be used for cases where high failure costs are expected even at low damage levels, and high γ values be used if low failure costs are expected at lower damage levels. Accordingly, it is suggested in this study that for buildings with essential functions, delicate decoration and expensive contents, the value of γ be less than 1 to emphasize the non-structural components, especially at early performance levels. On the other hand, γ should be greater than 1 to emphasize the structural components for normal buildings.

Once a continuous cost function is determined, the cost corresponding to the lower and upper bound of a performance level can be obtained. It is reasonable to assume that the average of these two cost values is the most representative cost value, and therefore, the drift value corresponding to this average cost on the cost function curve is considered as the most representative point for the performance level. It is obvious that for a cost function with γ less than 1, this most representative point will shift toward the non-structural side, while for a cost function with γ greater than 1, this most representative point will shift toward the structural side at all performance levels.

Figure 4.5 Failure cost function with various γ , Kanda & Shah (1997)

Evaluation of membership function $\mu_o(\theta)$ by constrained interpolation

According to its general properties discussed previously, the intended membership function should satisfy the following restrictions:

1. $\mu_o(\theta) \in [0, 1] \quad \forall \theta$.
2. $\mu_o(\theta)$ is differentiable.
3. $\mu_o(\theta)$ is convex.

Traditional least squares and simple splines may not satisfy the requirements about the boundary condition, convexity and local monotonicity. McAllister & Roulier (1981a, 1981b) developed an algorithm which produces a boundedness, monotonicity and convexity preserving second-degree Bernstein polynomial. Furthermore, it is fast and efficient to implement.

As mentioned previously, the lower and upper limit of the fuzzy interval, θ_1 and θ_2 , are all random variables. Therefore, these two parameters will also appear in the membership function. Let $\mu_o(\theta, \theta_1, \theta_2)$ denote the membership function of the fuzzy random event **Occurence**. According to the algorithm by McAllister & Roulier (1981a, 1981b),

$$\mu(\theta, \theta_1, \theta_2) = \begin{cases} 0 & \theta < \theta_1 \\ \mu_{o,1}(\theta, \theta_1, \theta_2) & \theta \in [\theta_1, t_1) \\ \mu_{o,2}(\theta, \theta_1, \theta_2) & \theta \in [t_1, \underline{\theta}) \\ \mu_{o,3}(\theta, \theta_1, \theta_2) & \theta \in [\underline{\theta}, t_2) \\ \mu_{o,4}(\theta, \theta_1, \theta_2) & \theta \in [t_2, \theta_2] \\ 0 & \theta > \theta_2 \end{cases} \quad (4.2)$$

where, $\underline{\theta}$ is the abscissa of the point with full membership function, 1, which can be derived from Equation 4.1 as:

$$\underline{\theta} = \theta_0 + \sqrt[\gamma]{\frac{(\theta_1 - \theta_0)^\gamma + (\theta_2 - \theta_0)^\gamma}{2}} \quad (4.3)$$

and,

$$t_1 = \frac{\theta_1 + \underline{\theta}}{2}, \quad t_2 = \frac{\theta_2 + \underline{\theta}}{2} \quad (4.4)$$

Details of the four parts of $\mu_o(\theta, \theta_1, \theta_2)$ are as follows:

$$\mu_{o,1}(\theta, \theta_1, \theta_2) = \frac{(\theta - \theta_1)^2}{(t_1 - \theta_1)(\underline{\theta} - \theta_1)} \quad (4.5)$$

$$\mu_{o,2}(\theta, \theta_1, \theta_2) = \frac{\frac{t_1 - \theta_1}{\underline{\theta} - \theta_1}(\underline{\theta} - \theta)^2 + 2(\theta - t_1)(\underline{\theta} - \theta) + (\theta - t_1)^2}{(\underline{\theta} - t_1)^2} \quad (4.6)$$

$$\mu_{o,3}(\theta, \theta_1, \theta_2) = \frac{(t_2 - \theta)^2 + 2(\theta - \underline{\theta})(t_2 - \theta) + \frac{\theta_2 - t_2}{\theta_2 - \underline{\theta}}(\theta - \underline{\theta})^2}{(t_2 - \underline{\theta})^2} \quad (4.7)$$

$$\mu_{o,4}(\theta, \theta_1, \theta_2) = \frac{(\theta_2 - \theta)^2}{(\theta_2 - \underline{\theta})(\theta_2 - t_2)} \quad (4.8)$$

Let $\mu_E(\theta, \theta_1, \theta_2)$ be the membership function of the fuzzy-random event **Exceeding**. $\mu_E(\theta, \theta_1, \theta_2)$ can be expressed as the normalized integral of $\mu_o(\theta, \theta_1, \theta_2)$ as:

$$\mu_E(\theta, \theta_1, \theta_2) = \eta \int_{\theta_1}^{\theta} \mu_o(t, \theta_1, \theta_2) dt \quad (4.9)$$

where, η is the normalizing constant which satisfies that $\mu_E(\theta_2, \theta_1, \theta_2) = 1$.

Similar to $\mu_o(\theta, \theta_1, \theta_2)$,

$$\mu_E(\theta, \theta_1, \theta_2) = \begin{cases} 0 & \theta < \theta_1 \\ \mu_{E,1}(\theta, \theta_1, \theta_2) & \theta \in [\theta_1, t_1) \\ \mu_{E,2}(\theta, \theta_1, \theta_2) & \theta \in [t_1, \underline{\theta}) \\ \mu_{E,3}(\theta, \theta_1, \theta_2) & \theta \in [\underline{\theta}, t_2) \\ \mu_{E,4}(\theta, \theta_1, \theta_2) & \theta \in [t_2, \theta_2] \\ 1 & \theta > \theta_2 \end{cases} \quad (4.10)$$

and

$$\mu_{E,1}(\theta, \theta_1, \theta_2) = \eta \int_{\theta_1}^{\theta} \mu_{o,1}(t, \theta_1, \theta_2) dt \quad (4.11)$$

$$\mu_{E,2}(\theta, \theta_1, \theta_2) = \eta \left(\int_{\theta_1}^{t_1} \mu_{o,1}(t, \theta_1, \theta_2) dt + \int_{t_1}^{\theta} \mu_{o,2}(t, \theta_1, \theta_2) dt \right) \quad (4.12)$$

$$\mu_{E,3}(\theta, \theta_1, \theta_2) = \eta \left(\int_{\theta_1}^{t_1} \mu_{O,1}(t, \theta_1, \theta_2) dt + \int_{t_1}^{\theta} \mu_{O,2}(t, \theta_1, \theta_2) dt + \int_{\theta}^{\theta} \mu_{O,3}(t, \theta_1, \theta_2) dt \right) \quad (4.13)$$

$$\mu_{E,4}(\theta, \theta_1, \theta_2) = \eta \left(\int_{\theta_1}^{t_1} \mu_{O,1}(t, \theta_1, \theta_2) dt + \int_{t_1}^{\theta} \mu_{O,2}(t, \theta_1, \theta_2) dt + \int_{\theta}^{t_2} \mu_{O,3}(t, \theta_1, \theta_2) dt + \int_{t_2}^{\theta} \mu_{O,4}(t, \theta_1, \theta_2) dt \right) \quad (4.14)$$

The final expressions for these four parts of $\mu_E(\theta, \theta_1, \theta_2)$ are listed below:

$$\mu_{E,1}(\theta, \theta_1, \theta_2) = \eta \left(\frac{(\theta - \theta_1)^3}{3(t_1 - \theta_1)(\theta - \theta_1)} \right) \quad (4.15)$$

$$\begin{aligned} \mu_{E,2}(\theta, \theta_1, \theta_2) = \eta \left(\frac{(t_1 - \theta_1)^2}{3(\theta - \theta_1)} + \frac{1}{(\theta - t_1)^2} \left(\frac{t_1 - \theta_1}{\theta - \theta_1} \frac{(\theta - \theta)^3 - (t_1 - \theta)^3}{3} \right. \right. \\ \left. \left. + 2 \left(-\frac{\theta^3}{3} + \frac{\theta + t_1}{2} \theta^2 - t_1 \theta \theta + \frac{\theta}{2} t_1^2 - \frac{t_1^3}{6} \right) + \frac{(\theta - t_1)^3}{3} \right) \right) \end{aligned} \quad (4.16)$$

$$\begin{aligned} \mu_{E,3}(\theta, \theta_1, \theta_2) = \eta \left(\frac{(t_1 - \theta_1)^2}{3(\theta - \theta_1)} + \frac{1}{(\theta - t_1)^2} \left(\frac{t_1 + \theta - 2\theta_1}{\theta - \theta_1} \frac{(\theta - t_1)^3}{3} + \frac{\theta^3}{3} - t_1 \theta^2 + t_1^2 \theta - \frac{t_1^3}{3} \right) \right. \\ \left. + \frac{1}{(t_2 - \theta)^2} \left(\frac{(\theta - t_2)^3 - (\theta - t_2)^3}{3} + 2 \left(-\frac{\theta^3}{3} + \frac{t_2 + \theta}{2} \theta^2 - \theta t_2 \theta + \frac{t_2}{2} \theta^2 - \frac{\theta^3}{6} \right) \right. \right. \\ \left. \left. + \frac{\theta_2 - t_2}{\theta_2 - \theta} \frac{(\theta - \theta)^3}{3} \right) \right) \end{aligned} \quad (4.17)$$

$$\begin{aligned} \mu_{E,4}(\theta, \theta_1, \theta_2) = \eta \left(\frac{(t_1 - \theta_1)^2}{3(\theta - \theta_1)} + \frac{1}{(\theta - t_1)^2} \left(\frac{t_1 + \theta - 2\theta_1}{\theta - \theta_1} \frac{(\theta - t_1)^3}{3} + \frac{\theta^3}{3} - t_1 \theta^2 + t_1^2 \theta - \frac{t_1^3}{3} \right) \right. \\ \left. + \frac{1}{(t_2 - \theta)^2} \left(-\frac{(\theta - t_2)^3}{3} + \frac{t_2^3}{3} - \theta t_2^2 + \theta^2 t_2 - \frac{\theta^3}{3} + \frac{\theta_2 - t_2}{\theta_2 - \theta} \frac{(t_2 - \theta)^3}{3} \right) \right. \\ \left. + \frac{(\theta - \theta_2)^3 - (t_2 - \theta_2)^3}{3(\theta_2 - \theta)(\theta_2 - t_2)} \right) \end{aligned} \quad (4.18)$$

4.5 Proposed fuzzy-random reliability model

4.5.1 The classical reliability model

The classical limit state function of a certain performance level is

$$Z = R - S \quad (4.19)$$

where R and S are the general resistance and general response of the structural system at certain performance level, respectively.

If both R and S are all random variables and proved to follow some probability distribution, the probability of certain performance level being violated or the probability of random event, $R - S < 0$, can be expressed as follows:

$$\begin{aligned} P(Z < 0) &= P(R < S) = \int_0^{+\infty} \left[\int_0^s f_R(r) dr \right] f_S(s) ds \\ &= \int_0^{+\infty} \left[\int_r^{+\infty} f_S(s) ds \right] f_R(r) dr \end{aligned} \quad (4.20)$$

4.5.2 The proposed fuzzy-random reliability model

Zadeh (1968) defined the probability of a fuzzy-random event A as the expectation of its membership function $\mu_A(X)$, i.e.,

$$P(A) = E[\mu_A(X)] = \int \mu_A(x) f_X(x) dx \quad (4.21)$$

where $f_X(x)$ is the probability density function of X . According to the fuzzy-random events defined in this study, Equation 4.21 needs to be further implemented.

According to the definition of a performance level in the beginning of this section, the general resistance R is a fuzzy-random variable. The general response (demand) S is considered as a normal random variable. Let R_1 and R_2 denote respectively the drift limit corresponding to non-structural and structural components (i.e., $R_1 = \theta_1$, $R_2 = \theta_2$), and their probability density function are $f_{R_1}(r_1)$ and $f_{R_2}(r_2)$, respectively. The probability density function of S is $f_S(s)$. Let $\mu_E(s, r_1, r_2)$ and $\mu_O(s, r_1, r_2)$ denote the membership functions of the fuzzy event of **Exceeding** and **Occurrence**.

The fuzzy-random event of **Occurrence** and **Exceeding** can be expressed as

$$R_1 \leq S \leq R_2 \parallel \mu_O(s, r_1, r_2) \quad (4.22)$$

and

$$S \geq R_1 \cap S \geq R_2 \parallel \mu_E(s, r_1, r_2) \quad (4.23)$$

where, $\parallel \mu(\)$ means following a certain membership function $\mu(\)$.

Assuming random variables, R_1 , R_2 and S , are independent to each other, the probability density function of the fuzzy-random event, $f_X(x)$, is that of the joint probability distribution of R_1 , R_2 and S , i.e.,

$$f_X(x) = f_{R_1}(r_1)f_{R_2}(r_2)f_S(s) \quad (4.24)$$

Thus, Equation 4.21 changes into

$$P(A) = \iiint \mu_E(s, r_1, r_2) f_{R_1}(r_1) f_{R_2}(r_2) f_S(s) ds dr_1 dr_2 \quad (4.25)$$

For fuzzy-random event **Exceeding** and **Occurrence**, the corresponding probability can be respectively expressed as

$$\begin{aligned} P_E &= P(R < S \parallel \mu_E(s, r_1, r_2)) \\ &= \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_0^{+\infty} \mu_E(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \end{aligned} \quad (4.26)$$

and

$$\begin{aligned} P_O &= P(R_1 \leq S \leq R_2 \parallel \mu_O(s, r_1, r_2)) \\ &= \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_0^{+\infty} \mu_O(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \end{aligned} \quad (4.27)$$

Considering the non-zero range of the membership functions, Equation 4.26 and 4.27 can also be rewritten as:

$$P_E = \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_1}^{+\infty} \mu_E(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \quad (4.28)$$

and

$$P_O = \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_1}^{r_2} \mu_O(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \quad (4.29)$$

According to Equation 4.10, the membership function $\mu_E(s, r_1, r_2)$ is a piecewise function which equals to 0 when S is less than R_1 or 1 when S is greater than R_2 . Therefore, Equation 4.28 can be expressed as:

$$\begin{aligned} P_E &= \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_1}^{r_2} \mu_E(s, r_1, r_2) f_S(s) ds + \int_{r_2}^{+\infty} f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \\ &= \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_1}^{r_2} \mu_E(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \\ &\quad + \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_2}^{+\infty} f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \end{aligned}$$

Consequently,

$$\begin{aligned} P_E &= \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_1}^{r_2} \mu_E(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \\ &+ \int_0^{+\infty} f_{R_1}(r_1) dr_1 \cdot \int_0^{+\infty} \left[\int_{r_2}^{+\infty} f_S(s) ds \right] f_{R_2}(r_2) dr_2 \quad (4.30) \end{aligned}$$

where, $\int_0^{+\infty} f_{R_1}(r_1) dr_1$ is easily to be obtained provided that $f_{R_1}(r_1)$ is known. If

$f_{R_1}(r_1)$ is defined in $(0, +\infty)$, for instance log-normal, $\int_0^{+\infty} f_{R_1}(r_1) dr_1$ equals to 1.

Equation 4.30 then reduces into

$$\begin{aligned} P_E &= \int_0^{+\infty} \left[\int_0^{+\infty} \left[\int_{r_1}^{r_2} \mu_E(s, r_1, r_2) f_S(s) ds \right] f_{R_2}(r_2) dr_2 \right] f_{R_1}(r_1) dr_1 \\ &\quad + \int_0^{+\infty} \left[\int_{r_2}^{+\infty} f_S(s) ds \right] f_{R_2}(r_2) dr_2 \quad (4.31) \end{aligned}$$

It can be seen that P_E as expressed in Equation 4.31 consists of two parts. The membership function is included in the first part which has the same integral domain as that of P_O . It is obvious that this integral is the probability of S falling into the interval of $[R_1, R_2]$ following the membership function $\mu_E(s, r_1, r_2)$. The second part of Equation 4.31 is the failure probability of demand S against capacity R_2 , i.e. $P(R_2 < S)$, based on classical reliability theory. Thus, it is obtained that

Inference 1: P_E is always greater than $P(R_2 < S)$ since the first part of Equation 4.31 is always greater than zero.

4.5.3 Example application

So far, a fuzzy-random reliability model is established to comprehensively evaluate the performance of a building system. In this section, a simple numerical application of the proposed fuzzy-random model is carried out to examine its quantitative features with various sets of limit state and demand data. It is noted that for purpose of simplicity in the illustrative application, all the drift demands from seismic inputs are arbitrarily assumed.

The drift ratio value corresponding to the initial damage state in Equation 4.1 is assumed as 0.05% at which the very first minor damage may take place. Two values of γ , 0.1 and 10, are applied to check the effect of building category. A performance level Functional is defined based on the probabilistic model of structural and non-structural deformation capacity established in previous chapters. The lower bound, Functional drift limit of masonry infill wall, and the upper bound Functional drift limit of Column D ($v=0.2$), are tabulated in Table 4.4.

Obviously, the mean of S , implying the relative position of PDF curve $f_S(s)$ to the performance interval, and the standard deviation of S , indicating the evenness of $f_S(s)$, jointly determine the value of P_O and P_E . Thus according to different relative positions to the capacity interval, three different PDFs of demand S , named **Less**, **Between** and **Beyond**, which may represent the seismic demand due to relative low, medium and high intensity earthquakes, are specified. For each case, two more PDFs for $f_S(s)$ with the same mean but different COV are also considered to examine the effect of COV on the result of reliability evaluation. The details of the assumed input data are also listed in Table 4.4.

By numerical integral, Equation 4.28 and 4.29 are calculated based on the input data in Table 4.4. For the purpose of comparison, the failure probability of S against R_1 and R_2 , $P_{f,1}$ and $P_{f,2}$, are also calculated based on classical reliability theory. The results are shown in Table 4.5. Figure 4.6 depicts the results of $P_{f,1}$, $P_{f,2}$ and P_E for high COV group.

Table 4.4 Details of assumed input data

Input Variables (drift ratio)		Prob. distribution	Mean	COV
Capacity	Lower limit	Log-normal	0.00254	82.8%
	Upper limit	Normal	0.0107	12.0%
Demand	Low COV	Less-L	0.0012	15.7%
		Between-L	0.007	15.7%
		Beyond-L	0.013	15.7%
	Medium COV	Less-M	0.0012	31.4%
		Between-M	0.007	31.4%
		Beyond-M	0.013	31.4%
	High COV	Less-H	0.0012	47.1%
		Between-H	0.007	47.1%
		Beyond-H	0.013	47.1%

Table 4.5 Results from proposed model

Cases		$\gamma=0.1$		$\gamma=10$		$P_{f,1}$	$P_{f,2}$
		P_O	P_E	P_O	P_E		
Low COV	Less-L	5.60E-2	3.33E-3	1.83E-3	9.34E-5	0.283	1.29E-13
	Between-L	0.530	0.653	0.602	0.314	0.945	1.52E-2
	Beyond-L	2.93E-2	0.977	0.132	0.955	0.991	0.836
Medium COV	Less-M	6.23E-2	4.67E-3	2.58E-3	1.66E-4	0.281	6.60E-13
	Between-M	0.455	0.621	0.507	0.356	0.922	7.57E-2
	Beyond-M	9.73E-2	0.922	0.203	0.857	0.980	0.709
High COV	Less-H	7.31E-2	6.92E-3	3.85E-3	3.12E-4	0.282	6.75E-12
	Between-H	0.375	0.594	0.408	0.389	0.874	0.151
	Beyond-H	0.128	0.849	0.191	0.774	0.942	0.647

Figure 4.6 Comparison of reliability results from different models

Clearly, the position of S relative to R_1 and R_2 , is the most influencing factor among all parameters affecting the values of P_O and P_E . The COV value also plays a considerable role in determining P_O and P_E . Consistent to *Inference 1*, P_E is always greater than $P_{f,2}$ from Table 4.5. The value of γ , which controls the shape of the membership function adopted in this study, also has a substantial effect on P_O and P_E . In all cases, larger γ results in lower P_E . In fact, if γ becomes infinite, the influence of lower bound of a performance level would be nil and P_E tends to be equal to $P_{f,2}$. The most significant effect of γ value is found in case of **Between** which may correspond to relative medium-to-high intensity ground motions.

The capacity data in Table 4.4 are real data selected to represent the Functional level of a generic masonry infilled RC frame. Subsequently, several sets of drift demands are assumed to cover different scenarios (in terms of the relative magnitude as compared to the capacity, and the variance). Changes to the input data would cause changes to the numerical results, but the general trend of effects of different γ values and the inferences of the fuzzy-random model shown by using the assumed input data in Table 4.4 will remain the same.

Furthermore, in the context of performance-based design, the goal is to make a structure achieve, rather than be bounded by, a given performance limit state under a given seismic intensity. The information provided by P_O is useful with regard to fulfilling a performance target with a consistent risk level.

4.6 Summary and conclusions

The performance of buildings involves structural as well as non-structural components. From the cost and loss point of view, the non-structural components is worthy of even more attention than the structural components concerning the performance of an entire building system, especially at earlier performance levels.

Because of the important role it plays as a type of non-structural components, and due to the availability of data, the masonry infill walls are chosen as the representative of non-structural components for quantified performance analysis in

this study. Based on an extensive survey of experimental data on the lateral force - deformation behavior of masonry wall panels, the limit state drifts for the three performance levels are extracted and the corresponding probabilistic models are obtained within this chapter. Before more comprehensive data on non-structural performance becomes available the proposed probabilistic drift limits for masonry walls may be used in a reliability analysis procedure where non-structural damage is to be considered.

It is observed that the limit drifts governing the structural and non-structural performances at a certain target level are very different. The classic reliability methods based on single-threshold limit criteria are not capable of taking both structural and non-structural damage into account for a comprehensive performance reliability evaluation. To tackle the problem, a novel fuzzy-random reliability model is proposed in the present study. Each performance limit state is defined as a fuzzy-random interval bounded by the limit drifts, θ_1 and θ_2 , respectively for non-structural and structural performance. The fuzziness comes from the difference between θ_1 and θ_2 while the randomness is associated with the probabilistic properties of θ_1 and θ_2 . Two membership functions are devised to represent the Occurrence and the Exceeding of a performance level, respectively. In this way, the different limit criteria for non-structural and structural components can be considered in a comprehensive and computable manner for performance-based seismic reliability analysis of a building system. The relative importance of the non-structural and structural damage at a target performance state can be reflected by assigning appropriate value to the γ -coefficient in the cost function.

An illustrative application example of using the proposed fuzzy-random reliability model is given at the end of this chapter. It is demonstrated that using different values of γ in the cost function, different emphasis of the structural and non-structural damage can be achieved effectively. Comparing with the results from classic reliability approach based on single-threshold limit criteria, the proposed fuzzy-random model provides a more comprehensive reliability evaluation incorporating the influence of both non-structural and structural damage on the building performance.

The proposed fuzzy-random model establishes a robust platform for a comprehensive evaluation of building performance taking various influencing factors into account. The evaluation of the “Occurrence” probability of a certain performance limit state provides a useful measure which may be applied in an optimization procedure in achieving multiple performance targets.

Further examples of applying the fuzzy-random reliability model will be given in the next chapter in association with the analysis of the critical storey in a frame structure.

Chapter 5 Probabilistic Nonlinear Critical Inter-storey Drift Demand of RC Frame Structures

5.1 Introduction

In the previous chapters of this thesis, research efforts have been made to establish probabilistic models of displacement (drift) limits of structural and non-structural components of RC frame structures for different performance levels, and a fuzzy-random reliability model is also developed which can incorporate structural and non-structural performance in a rigorous manner. To apply these models in the analysis of the entire frame systems, it is necessary to establish a probabilistic model describing the critical seismic displacement demand on RC frame structures.

The response spectrum approach is most commonly used in earthquake engineering to estimate the seismic demand on a structure. Elastic response spectra of SDOF systems are dependent on the ground motion characteristics, which in turn depend on the type of the earthquake, epicenter distance, site conditions and so on. Inelastic spectra depend not only on the characteristics of the expected ground motion at the site, but also on the non-linear behavior of the structural system. From the response spectrum, the mean and variance of the seismic demand of SDOF systems can be obtained. However, to transform the SDOF spectral response to the critical drift demand on a frame system is not a simple task, especially when inelastic response and structural irregularity are involved. Usually, the roof displacement demand is first evaluated through the modal participation factor which relates the MDOF response to the SDOF model. The roof displacement is then distributed to get the inter-storey drifts.

To obtain the maximum inter-storey drift, various methods have been proposed in the past. In the well-known N2 method (Fajfar & Fischinger 1989), a nonlinear pushover analysis is performed to determine local displacement quantities including the storey

drift. However, for a rapid performance assessment, a detailed nonlinear pushover analysis doesn't seem to be practical. Miranda (1997) proposed a series of factors to get the maximum inter-storey drift from the elastic displacement spectrum ordinates. However, in inelastic systems, the location where the maximum inter-storey drift occurs is not determined. Furthermore, both the above methods provide no information about the variation of maximum inter-storey drift, which is an essential issue for a probabilistic model.

In this chapter, a framework for establishing a probabilistic model of the maximum inter-storey drift ratio in a framed structure is proposed. A new storey capacity factor and the corresponding regularity index are introduced to relate the distribution of the strength and stiffness of both columns and beams to the distribution of the inter-storey displacements. By comparing with the pushover results, it is found that the distribution of the proposed storey capacity factor correlates with the storey displacement profile in a much improved manner than the storey overstrength factor based on column strength only.

Subsequently, nonlinear time history analyses are carried out on several representative frames subject to a selected group of ground motions. The mean displacement profiles show satisfactory correlation with the distribution of the storey capacity factors. The maximum storey drift generally takes place at the storey with the lowest capacity factor. Defining a drift concentration factor as the ratio between the maximum inter-storey drift ratio and the roof drift, the relationships between the drift concentration factor and the nonlinear response level, the number of storeys and the regularity index are evaluated. Furthermore, by statistic analysis, the probabilistic properties of the drift concentration factor are obtained. Finally, a regression equation is proposed to describe the relationship between the drift concentration factor and the proposed regularity index.

At the end of this chapter, an example application is carried out to illustrate the application of the fuzzy-random model in performance-oriented reliability analysis of the critical storey of a 7-storey RC frame. The probabilistic drift limits of both

structural and non-structural components are employed as the capacity input. The framework proposed in this chapter is used to obtain the displacement demand of the critical storey.

5.2 Ground motions

5.2.1 Consideration of uncertainties in seismic input

The characteristics of the earthquake ground motions significantly affect the seismic response of RC frame structures. As the focus of the present study is to establish a framework to estimate the critical inter-storey drift in an RC frame (essentially a matter of drift distribution), it is not intended to deal with the uncertainties in the seismic input extensively. The influence of the ground motions on the drift distribution mainly depends upon their frequency characteristics that affect the higher mode contributions. It is generally accepted that a group of 20~30 ground motions taken from a similar site condition should be able to represent the variability of ground motion characteristics for the site condition. Thus, a set of 30 ground motions occurred in California are selected here which corresponds to a stiff soil class D (NEHRP classification) and earthquakes with epicentral distance in the range of 0~35km. The ground motion records are obtained from the PEER Strong Motion Database. Table 5.1 summaries the selected 30 ground motions.

Table 5.1 Summary of ground motions (Site soil D, Epicentral dist. 0~35km)

Earthquake	Station name	Epic. dist. (km)	Components	PGA (g)	PGV (cm/s)	PGD (cm)	A ₉₅ (g)
Imperial Valley 1979/10/15	5054 Bonds Corner	9.0	IMPVALL/H-BCR140	0.588	45.2	16.78	0.316
			IMPVALL/H-BCR230	0.775	45.9	14.89	0.381
Imperial Valley 1979/10/15	5028 El Centro Array #7	29.4	IMPVALL/H-E07140	0.338	47.6	24.68	0.163
			IMPVALL/H-E07230	0.463	109.3	44.74	0.219
Imperial Valley 1979/10/15	5053 Calexico Fire Station	17.4	IMPVALL/H-CXO225	0.275	21.2	9.02	0.162
			IMPVALL/H-CXO315	0.202	16.0	9.2	0.151
Imperial Valley 1979/10/15	952 El Centro Array #5	30.5	IMPVALL/H-E05140	0.519	46.9	35.35	0.216
			IMPVALL/H-E05230	0.379	90.5	63.03	0.217
Imperial Valley 1979/10/15	955 El Centro Array #4	29.7	IMPVALL/H-E04140	0.485	37.4	20.23	0.197
			IMPVALL/H-E04230	0.36	76.6	59.02	0.169
Imperial Valley 1979/10/15	412 El Centro Array #10	29.8	IMPVALL/H-E10050	0.171	47.5	31.1	0.136
			IMPVALL/H-E10320	0.224	41.0	19.38	0.146
Imperial Valley 1979/10/15	5058 El Centro Array #11	30.1	IMPVALL/H-E11140	0.364	34.5	16.07	0.234
			IMPVALL/H-E11230	0.38	42.1	18.59	0.214
Imperial Valley	931 El Centro Array	32.1	IMPVALL/H-E12140	0.143	17.6	11.3	0.114

1979/10/15	#12		IMPVALL/H-E12230	0.116	21.8	12.06	0.106
San Fernando 1971/02/09 14:00	135 LA - Hollywood Stor Lot	24.0	SFERN/PEL090	0.21	18.9	12.4	0.144
			SFERN/PEL180	0.174	14.9	6.25	0.122
Loma Prieta 1989/10/18 00:05	47380 Gilroy Array #2	29.5	LOMAP/G02000	0.367	32.9	7.15	0.188
			LOMAP/G02090	0.322	39.1	12.07	0.189
Loma Prieta 1989/10/18 00:05	47381 Gilroy Array #3	31.1	LOMAP/G03000	0.555	35.7	8.21	0.240
			LOMAP/G03090	0.367	44.7	19.25	0.198
Northridge 1994/01/17 12:31	24303 LA - Hollywood Stor FF	22.9	NORTHHR/HOL090	0.231	18.3	4.81	0.169
			NORTHHR/HOL360	0.358	27.5	3.04	0.236
Whittier Narrows 1987/10/01 14:42	24303 LA - Hollywood Stor FF	25.0	WHITTIER/A-HOL000	0.221	9.0	1.43	0.096
			WHITTIER/A-HOL090	0.124	6.9	1.12	0.078
Whittier Narrows 1987/10/01 14:42	23525 Pomona – 4th & Locust FF	30.4	WHITTIER/A-PMN012	0.067	3.4	0.35	0.040
			WHITTIER/A-PMN102	0.056	2.5	0.22	0.039
Whittier Narrows 1987/10/01 14:42	14368 Downey – Co Maint Bldg	17.0	WHITTIER/A-DWN180	0.221	28.8	3.95	0.121
			WHITTIER/A-DWN270	0.141	13.4	1.6	0.092

5.2.2 Scaling of ground motions

To evaluate the statistical properties of the structural response under a given type of earthquake the ground motion records must be scaled to a specified level of seismic intensity. Various ground motion parameters may be used as the scaling factor, for example:

- 1) Peak ground acceleration (PGA)
- 2) Effective peak acceleration (EPA). Proposed by NEHRP (1994), EPA is calculated as the mean linear-elastic 5%-damped spectral acceleration for the period range of 0.1 to 0.5s divided by 2.5.
- 3) Arias intensity-based parameter (A_{95}). Sarma & Yang (1987) defined the A_{95} as the acceleration that contains 95% of the Arias Intensity, $E_s = \int_0^t a^2(t)dt$, where $a(t)$ is the acceleration time-history of the ground motion. Based on a linear regression study (Sarma & Yang 1987), A_{95} can be approximately determined as $A_{95} = 0.764E_s^{0.438}$.
- 4) Maximum incremental velocity (MIV). The IV is the incremental velocity which is defined as the area under the acceleration time-history of a ground motion between two consecutive zero acceleration crossings.
- 5) Effective peak velocity (EPV). According to NEHRP (1994), EPV is defined as the linear-elastic 5%-damped spectral pseudo-velocity at period, $T=1s$.

- 6) Spectral response acceleration at the structure fundamental period ($S_a(T_0)$).
- 7) Spectral acceleration over a range of structure periods ($\bar{S}_a(T_0 \rightarrow T_\mu)$). T_μ is the elongated fundamental period taking into account the elongation of all the structural periods due to non-linear behavior.

Some recent studies have demonstrated that certain scaling methods can introduce a large scatter in the estimation of seismic demands. Kurama & Farrow (2003) investigated the effectiveness of the above listed seven ground motion scaling methods in terms of the scatter in the estimated peak lateral displacement demands. It was found that certain scaling methods can be effective in reducing the scatter only for certain site conditions and structural characteristics. According to the above study, the Arias intensity-based parameter A_{95} , which was observed to perform well for stiff soil site and near field earthquakes, is selected as the scaling factor in the present study. Since it is actually the ratio between the critical inter-storey drift and the nominal drift that is of interest, different scaling methods should not affect significantly the results. Figure 5.1 shows the mean elastic acceleration spectrum (5% damping ratio) of the selected 30 ground motion records scaled by $A_{95}=0.17g$, which results in a mean PGA of 0.3g, and the NEHRP97 (NEHRP 1997) spectrum.

Figure 5.1 Elastic acceleration response spectrums of grouped ground motions

5.3 Representative frames

Besides the strong dependence on ground motions, the characteristics of the structures also affect the seismic displacement demand. The main structural parameters include the vertical distribution of storey stiffness and strength (regularity), number of storeys and the maximum storey ductility (or the nonlinear level). Totally 6 frames are considered in this study to evaluate the effects of different structural parameters on the seismic displacement demand. A generic 7-storey frame will be discussed first.

5.3.1 A generic regular 7-storey frame structure (7-R)

As a benchmark structure, a generic planar 7-storey frame structure, designated as 7-R (“7” indicates the number of storeys and “R” means regular frame), is designed to satisfy the following requirements:

- Weak-beam strong-column
- No abrupt change in vertical distribution of storey stiffness and strength (as reflected by the regularity index, α_r)
- Design PGA=0.3g on medium stiff soil site
- Medium design ductility

The regularity index, α_r , is defined according to EC8 (1996) as:

$$\alpha_r = (i_{OS})_{\min} / (i_{OS})_{ave} \quad (5.1)$$

where $(i_{OS})_{\min}$ and $(i_{OS})_{ave}$ are the minimum and mean values of storey overstrength factor, respectively. Generally, α_r is not allowed to be smaller than 0.55 while $\alpha_r \geq 0.85$ is classified as regular. The storey overstrength factor, i_{OS} , is given by:

$$i_{OS} = V_R / V_S \quad (5.2)$$

where V_R and V_S are the available storey shear strength and design storey shear force, respectively. Normally, V_R is evaluated as a sum of shear strength of all columns in a storey. In the static analysis performed in this study, two static lateral force distributions, styles A and B, are applied to reflect different modal effects on the dynamic response of the frame. Style-A is an inverted triangle lateral load distribution which mainly represents the first-mode effect on the dynamic response of regular and low-to-medium rise frame structures. Style-B is an inverted triangle distribution with a

concentrated force equals to 20% of the total base shear on the top storey, which reflects an appreciable contribution of higher-mode effect on irregular and high rise frame structures. The general design information and sectional details are given in Table 5.2 and 5.3.

Table 5.2 General design information for frame 7-R

		Span to adjacent frames: 6.0m
		Thickness of slab: 110mm
		Live load: 3.0kN/m ²
		Concrete: C25, $f_c=25\text{MPa}$
		Longitudinal reinforcement: Grade 60, $f_y=415\text{MPa}$
		Transverse reinforcement: Grade 40, $f_y=277\text{MPa}$
		Site soil: NEHRP Class D, stiff soil
		Design ground acceleration: 0.3g
		Response modification factor: 3
		Response spectrum: NEHRP 97
		Fundamental period: 0.86s

Table 5.3 Section and reinforcement details for frame 7-R

Storey		1	2	3	4	5	6	7
Column	Section (mm)	600×600	600×600	500×500	500×500	450×450	450×450	400×400
	Rebar	12D22.3	12D22.3/ 12D19.1	12D22.3/ 12D19.1	12D22.3/ 12D15.9	12D19.1	12D19.1/ 12D15.9	8D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	1.30/0.95	1.87/1.37	1.87/0.95	1.69	1.69/1.17	0.99
	Hoops	D9.5@150	D9.5@150	D9.5@200	D9.5@200	D9.5@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.50	0.50	0.38	0.38	0.49	0.49	0.53
	$\alpha\omega_v$	0.032	0.032	0.020	0.020	0.026	0.026	0.020
	Hoop pattern							

Overstrength factor by Storey shear (column)		3.65	4.58	3.86	3.62	4.02	4.44	4.86
Beam	Storey	Section (mm)	Rebar	Rebar grade	Hoops	Hoop bar grade		
	1	500×300		Grade 60	4D9.5@150	Grade 40		
	2	500×300		Grade 60	4D9.5@150	Grade 40		
	3	500×300		Grade 60	4D9.5@150	Grade 40		
	4	500×300		Grade 60	4D9.5@150	Grade 40		
	5	400×300		Grade 60	4D9.5@150	Grade 40		
	6	400×300		Grade 60	4D9.5@150	Grade 40		
	7	400×300		Grade 60	2D9.5@150	Grade 40		

Note: Beam longitudinal reinforcement: a-4D22.3, b-4D19.1, c-3D19.1, d-3D15.9

Table 5.3 also gives the storey overstrength factors, i_{OS} , which are calculated according to column shear strength and a lateral load distribution of style-A. According to Equation 5.1, the regularity index α_r is calculated to be 0.87 and thus the frame can be classified as regular. As illustrated in Figure 5.2, the distribution of i_{OS} shows an abrupt increase at the 2nd storey. This is attributed to the abrupt change of the storey height from 5m to 3.5m.

Figure 5.2 Distribution of overstrength factors (7-R, columns, force style-A)

5.3.2 Static pushover analysis

To obtain some preliminary knowledge about the lateral displacement response of the frame 7-R, a pushover analysis is carried out using the program IDARC 2D 4.0 (Kunnath et al. 1992). IDARC 2D 4.0 is an inelastic damage analysis program developed in University at Buffalo, NY. More discussion on the program will be given later.

In the pushover analysis of frame 7-R, lateral force distribution style-A is applied and the first element failure is taken as the criteria to terminate the analysis. The distribution of the inter-storey drift ratios obtained from the pushover analysis is plotted in Figure 5.3.

Figure 5.3 Inter-storey drift ratios from pushover analysis (7-R, force style-A)

From Figure 5.3, it is observed that the inter-storey drift ratio of the first storey is less than those of the second and third storeys. This is not really consistent with the distribution of overstrength factors shown in Figure 5.2, because generally speaking, higher overstrength factor results in lower inter-storey drift ratio.

By tracking the damage history of the pushover analysis, it is found that plastic hinges occur extensively in beams before the first yielding in columns. This phenomenon is reasonable because the frame is designed according to the weak-beam strong-column philosophy. Consequently, the failure mechanism of the frame will involve extensive yielding in beams rather than the soft-storey type of yielding of only columns. Besides,

the storey stiffness can also be a significant factor affecting the distribution of the inter-storey displacements. To take into account the effects of beam yielding and storey stiffness in the structural regularity evaluation, it is necessary to modify the overstrength factor and the regularity index for frame structures.

5.3.3 New regularity index incorporating the effects of strength and stiffness

Overstrength factor accounting for yielding of beams

Figure 5.4 shows a typical beam-column joint. According to the weak-beam strong-column design philosophy, earlier occurrence of yielding at beam-ends than in column-ends implies that the column yield moment strength may not be reached. Therefore, the shear force in the column at the formation of the mechanism needs to be reduced from that corresponding to yielding of the column to reflect the actual storey strength. Approximately, the shear force in column corresponding to yielding of beams connecting to its upper end can be expressed as:

$$V_c^u = \begin{cases} V_{c,y} \times \frac{\sum M_{b,y}}{\sum M_{c,y}}, & \frac{\sum M_{b,y}}{\sum M_{c,y}} \leq 1 \\ V_{c,y}, & \frac{\sum M_{b,y}}{\sum M_{c,y}} > 1 \end{cases} \quad (5.3)$$

where, $V_{c,y}$ denotes the shear force corresponding to yielding at two column ends; $\sum M_{b,y}$ and $\sum M_{c,y}$ are the summation of the yield moments of beams and columns connecting to the joint, respectively. When $\frac{\sum M_{b,y}}{\sum M_{c,y}} > 1$, the beams are collectively stronger than the columns at the joint and no reduction on $V_{c,y}$ is needed. In a similar way, the shear strength of the column corresponding to yielding of beams connecting to its bottom end, V_c^l , can be calculated. Thus, the representative column shear strength considering the possible yielding in beam ends can be obtained as the average value of V_c^u and V_c^l :

$$V_c = \frac{V_c^u + V_c^l}{2} \quad (5.4)$$

Subsequently, the storey shear strength can be evaluated by summing up the representative shear strength of all columns belonging to the storey.

Figure 5.4 A typical beam-column joint

For frame 7-R, the new overstrength factor for each storey is evaluated and the results are plotted in Figure 5.5. Without exception, the overstrength factors are reduced significantly as compared to Figure 5.2 due to the incorporation of yielding of beams. The lowest overstrength values now occur at 2nd and 3rd storeys, which correlate very well with the distributions of inter-storey drift ratios in Figure 5.3. The regularity index according to the new overstrength factors is found to be 0.83, so frame 7-R can still be regarded as regular.

Figure 5.5 Distribution of overstrength factors (7-R, joints, force style-A)

Normalized stiffness factor

The initial storey lateral stiffness has direct effect on the distribution of the storey drift in the elastic range. Such effect will understandably extend into low-to-medium nonlinear response range and could become less significant at more advanced inelastic stage. Therefore, it is rational to include the effect of elastic storey lateral stiffness in the evaluation of structural regularity.

The elastic lateral stiffness of a column with presumably rigid floors can be expressed as:

$$K_c = \frac{12i_c}{h^2} = \frac{12EI}{h^3} \quad (5.5)$$

where, i_c is the linear stiffness which equal to EI/h . EI is the bending stiffness and h is the storey height. However, in the weak-beam and strong-column design, the linear stiffness of beams are usually on the same numerical order as that of columns. Therefore, Equation 5.5 can lead to significant overestimation of the actual storey lateral stiffness.

The D-value method (Muto 1974) provides a simple way to modify the storey lateral stiffness incorporating the effect of linear stiffness of beams. In the D-value method, the modified lateral stiffness of an internal column at j^{th} storey ($j > 1$) can be expressed as:

$$D = \alpha \frac{12i_c}{h^2} = \alpha \frac{12EI}{h^3} \quad (5.6)$$

$$\alpha = \frac{K_s}{2 + K_s} \quad (5.7)$$

$$K_s = \frac{i_1 + i_2 + i_3 + i_4}{2i_c} \quad (5.8)$$

where, D is the modified column lateral stiffness and α is the modification factor; i_1 , i_2 , i_3 and i_4 are linear stiffness of beams (see Figure 5.6a) and i_c is the column linear stiffness.

a. Linear stiffness of beams and columns

b. Normalized storey shear force and storey stiffness (7-R, force style-A)

Figure 5.6 Evaluation of storey stiffness (7-R)

The uniformity in the distribution of the storey stiffness is actually a relative measure and it has to be viewed with respect to the distribution of the storey shear distribution. For frame 7-R, the distributions of storey shear corresponding to inverted triangle distribution of lateral forces, and modified storey stiffness D (both are normalized to 1 in the first storey), as shown in Figure 5.6b, are similar. This indicates a good uniformity in terms of the storey lateral stiffness.

In a general way, a normalized storey stiffness factor, i_{ns} , may be defined as:

$$i_{ns} = D_n / V_n \quad (5.9)$$

where, D_n and V_n are the normalized storey stiffness and normalized storey shear, respectively. The i_{ns} values reflect the relative stiffness distribution associated with a

certain lateral force distribution. Similarly, a regularity index α_{ns} corresponding to storey stiffness distribution can be defined as:

$$\alpha_{ns} = (i_{ns})_{\min} / (i_{ns})_{ave} \quad (5.10)$$

where, $(i_{ns})_{\min}$ and $(i_{ns})_{ave}$ are the minimum and average values of the normalized storey stiffness factor, i_{ns} , respectively.

Figure 5.7 plots the distribution of the normalized storey stiffness factor, i_{ns} , of frame 7-R. A quite uniform distribution (except the 7th storey) is observed.

Figure 5.7 Distribution of normalized storey stiffness factors (7-R, force style-A)

Storey capacity factor

To take into account the possible effects of both the storey strength and the storey stiffness on the distribution of the inter-storey drift in a frame structure, a storey capacity factor, i_{sc} , is defined as the product of the storey overstrength factor, i_{os} , and the normalized storey stiffness factor, i_{ns} :

$$i_{sc} = i_{os} \times i_{ns} \quad (5.11)$$

Accordingly, a comprehensive regularity index, α_{sc} , is defined as:

$$\alpha_{sc} = (i_{sc})_{\min} / (i_{sc})_{ave} \quad (5.12)$$

where, $(i_{sc})_{\min}$ and $(i_{sc})_{ave}$ are the minimum and average values of i_{sc} , respectively. Generally speaking, larger value of i_{sc} results in smaller inter-storey displacement for the particular lateral force distribution considered. Similar to α_r , the regularity factor associated with storey strength, α_{sc} reflects the regularity of the distribution of storey strength and storey stiffness along the height of a frame structure.

For 7-R, Figure 5.8 plots the distribution of the storey capacity factor. The 2nd and 3rd storey become the weakest storeys in this 7-storey frame, while the 7th storey exhibits the largest strength and stiffness margin. Comparing to Figure 5.3, it is found that the distribution of capacity factor matches the pushover results very well.

The corresponding regularity index calculated according to Equation 5.12 is equal to 0.64. This value of regularity index is quite low which is mainly due to the extraordinary large capacity factor at the 7th storey. Normally, the behavior of the top storey would not significantly affect the general response of the structural system. Therefore, it appears to be more reasonable to exclude the capacity factor of top storey in the regularity index evaluation. In the case of frame 7-R, the regularity index corresponding to storey 1-6 equals to 0.84. Table 5.4 summarizes all related parameters, i.e. strength, stiffness and their capacity factors for 7-R.

Table 5.4 Factors related to regularity index (7-R, force style-A)

Storey	1	2	3	4	5	6	7	$\frac{(i)_{\min}}{(i)_{ave}}$
Overstrength factor (columns)	3.65	4.58	3.86	3.62	4.02	4.44	4.86	0.87
Overstrength factor (joints)	2.57	2.14	2.10	2.32	2.30	2.66	3.98	0.83
Normalized stiffness factor	1.00	0.99	0.96	1.11	1.02	1.07	1.89	0.89
Storey capacity factor	2.57	2.12	2.02	2.59	2.35	2.85	7.53	All 7 storeys: 0.64 Storeys 1 to 6: 0.84

Note: $(i)_{\min}$ and $(i)_{ave}$ are the minimum and mean value of the corresponding factors, respectively.

Figure 5.8 Distribution of storey capacity factors (7-R, force style-A)

5.3.4 Irregular 7-storey frame structures (7-I-a and 7-I-b)

Based on the topology of frame 7-R, two irregular frames, 7-I-a and 7-I-b, are designed to have different regularity indices by reducing the sectional dimensions and longitudinal reinforcement of columns in the 5th to 7th storeys. The purpose of introducing these two irregular frames is to evaluate the effect of the regularity index on the critical inter-storey displacement response.

Frame 7-I-a

The sectional details of frame 7-I-a are tabulated in Table B.1 (see Appendix B). The results from regularity index evaluation and the pushover analysis, based on style-A lateral force distribution, are summarized in Table 5.5 and also shown in Figure 5.9.

Abrupt reductions of i_{os} , i_{ns} and i_{sc} are found at the 5th storey. Comparing with the pushover results (Figure 5.9e), it is found again that the distribution correlates well with the distribution of inter-storey drift ratios from the pushover analysis. Based on i_{sc} , the regularity index for 7-I-a is equal to 0.57 when considering all 7 storeys and 0.77 if only storeys 1 to 6 are accounted for. This indicates a considerable irregularity of frame 7-I-a as compared with frame 7-R.

Table 5.5 Factors related to regularity index (7-I-a, force style-A)

Storey	1	2	3	4	5	6	7	$\frac{(i)_{\min}}{(i)_{ave}}$
Overstrength factor (column)	3.68	4.61	3.89	3.70	2.70	3.48	4.85	0.70
Overstrength factor (joints)	2.60	2.17	2.13	2.60	1.99	2.51	4.28	0.76
Normalized stiffness factor	1.00	0.99	0.96	1.12	0.89	0.96	1.86	0.80
Storey capacity factor	2.60	2.15	2.05	2.92	1.77	2.40	7.98	All 7 storey: 0.57 Storeys 1~6: 0.77

Figure 5.9 Evaluation of regularity index and pushover results (7-I-a, force style-A)

Frame 7-I-b

Table B.2 (see Appendix B) lists the sectional details of frame 7-I-b. It is suggested by Lu (2002) that for frames with significant irregularity like 7-I-b, a lateral force distribution of style-B be considered in the static analysis concerning the critical dynamic response of the frame. Table 5.6 tabulates the results from the regularity index evaluation for 7-I-b corresponding to both styles of lateral force distributions (A and B). As expected, more severe irregularity than 7-I-a is observed from the distribution of storey capacity factors and the regularity indices with or without 7th storey.

Figure 5.10 plots the results based on style-B lateral force distribution. With style-B, the distribution of storey capacity factors (Figure 5.10d) shows a good correlation with the pushover results (Figure 5.10e).

Table 5.6 Factors related to regularity index (7-I-b)

Lateral force	Storey	1	2	3	4	5	6	7	$\frac{(i)_{\min}}{(i)_{ave}}$
Style-A	Overstrength factor (column)	3.68	4.61	3.89	3.70	2.21	2.79	4.85	0.60
	Overstrength factor (joints)	2.60	2.17	2.13	2.70	1.83	2.25	4.28	0.71
	Normalized stiffness factor	1.00	0.99	0.96	1.12	0.89	0.96	1.86	0.80
	Storey capacity factor	2.60	2.15	2.05	3.03	1.63	2.15	7.99	0.53
Style-B	Overstrength factor (column)	3.68	4.56	3.76	3.44	1.94	2.17	2.78	0.61
	Overstrength factor (joints)	2.60	2.15	2.06	2.52	1.60	1.75	2.45	0.74
	Normalized stiffness factor	1.00	0.98	0.93	1.04	0.78	0.75	1.07	0.80
	Storey capacity factor	2.60	2.10	1.91	2.63	1.24	1.30	2.62	0.60

Figure 5.10 Evaluation of regularity index and pushover results (7-I-b, force style-B)

5.3.5 Regular 12-storey frame structure 12-R

To examine the effect of the number of storeys on the drift distribution, a 2-bay 12-storey frame structure, designated as 12-R, is designed following the same general requirements as frame 7-R. The geometry of 12-R is similar to that of 7-R. The storey height is 5m for the base storey and 3.5m for the remaining storeys. The span length is 5m for both spans. The fundamental period of 12-R is 1.1s. The sectional details for 12-R are tabulated in Table B.3 (see Appendix B).

Due to the larger number of storeys, the high-mode effect is expected to become more significant in the dynamic response of 12-R. Therefore, besides the style-A lateral force distribution applied to 7-R, style-B is also used to represent the possible high-mode effect in the pushover analysis as well as the regularity evaluation of 12-R.

Table 5.7 summaries the regularity evaluation results. It can be seen that the regularity index of frame 12-R corresponding to style-A is close to that of frame 7-R, and can be classified as regular. However, much lower regularity index is obtained based on lateral force style-B. This implies that frame 12-R could show a rather irregular dynamic behavior due to the possible high-mode effect. Figure 5.11 plots the results related to the evaluation of regularity index based on the lateral force distribution of style-B. Good correlation between the distribution of storey capacity factors and the pushover results is again observed.

Table 5.7 Factors related to regularity index (12-R)

Lateral force	Storey	1	2	3	4	5	6	7	8	9	10	11	12	$\frac{(i)_{\min}}{(i)_{ave}}$
Style-A	Overstrength factor (column)	3.79	4.57	4.16	4.02	3.42	3.63	3.89	3.47	3.45	3.90	3.46	5.62	0.87
	Overstrength factor (joints)	2.57	1.78	1.89	2.07	1.83	1.89	2.05	2.04	1.97	2.10	2.16	3.94	0.81
	Normalized stiffness factor	1.00	0.71	0.74	0.78	0.63	0.55	0.55	0.50	0.48	0.46	0.56	1.13	0.84
	Storey capacity factor	2.57	1.26	1.39	1.61	1.14	1.04	1.14	1.02	0.96	0.96	1.22	4.46	0.78
Style-B	Overstrength factor (column)	3.79	4.54	4.10	3.92	3.28	3.40	3.53	3.02	2.81	2.87	2.13	2.29	0.64
	Overstrength factor (joints)	2.57	1.77	1.86	2.02	1.75	1.77	1.86	1.77	1.61	1.55	1.33	1.60	0.74
	Normalized stiffness factor	1.00	0.70	0.73	0.76	0.60	0.51	0.50	0.43	0.40	0.34	0.35	0.46	0.60
	Storey capacity factor	2.57	1.25	1.35	1.53	1.05	0.91	0.94	0.77	0.63	0.52	0.46	0.74	0.44

Figure 5.11 Evaluation of regularity index and pushover results (12-R, force style-B)

5.3.6 Irregular 12-storey frame structures (12-I-a and 12-I-b)

Two irregular 12-storey frame structures, designated as 12-I-a and 12-I-b, are also designed according to the similar design requirements as for other frames to supplement the observations on the inter-storey drift distributions obtained from the group of 7-storey frames. All the three 12-storey frames have the same geometry. Frame 12-I-a differs from 12-R in column sections only for storeys 8-12 while the member dimension of frame 12-I-b are largely rearranged to result in a more different strength/stiffness profile.

Frame 12-I-a

For frame 12-I-a, the sectional details are presented in Table B.4 (see Appendix B). The results from the regularity evaluation based on both styles A and B lateral force distributions are shown in Table 5.8. Severe irregularity can be observed from the values of i_{os} , i_{ns} and i_{sc} comparing with the frame 12-R. The distributions of i_{os} , i_{ns} and i_{sc} and the pushover results based on style-B lateral force distribution are illustrated in Figure 5.12. A good correlation between the distribution of i_{sc} and the drift profile from the pushover analysis is observed.

Table 5.8 Factors related to regularity index (12-I-a)

Lateral force	Storey	1	2	3	4	5	6	7	8	9	10	11	12	$\frac{(i)_{\min}}{(i)_{ave}}$
Style-A	Overstrength factor (column)	3.84	4.64	4.21	4.07	3.51	3.69	3.98	2.16	2.47	2.51	3.43	5.57	0.59
	Overstrength factor (joints)	2.61	1.83	1.92	2.11	1.88	1.94	2.03	1.34	1.55	1.38	2.05	3.91	0.65
	Normalized stiffness factor	1.00	0.71	0.74	0.78	0.63	0.55	0.56	0.38	0.38	0.38	0.55	1.10	0.59
	Storey capacity factor	2.61	1.29	1.42	1.65	1.19	1.07	1.14	0.51	0.60	0.52	1.13	4.30	0.35
Style-B	Overstrength factor (column)	3.84	4.62	4.16	3.97	3.36	3.45	3.60	1.87	2.01	1.86	2.14	2.31	0.60
	Overstrength factor (joints)	2.61	1.82	1.90	2.06	1.80	1.82	1.83	1.16	1.26	1.02	1.28	1.62	0.61
	Normalized stiffness factor	1.00	0.70	0.73	0.76	0.60	0.52	0.51	0.33	0.31	0.28	0.34	0.46	0.60
	Storey capacity factor	2.61	1.28	1.38	1.57	1.09	0.94	0.93	0.38	0.39	0.28	0.44	0.74	0.38

Figure 5.12 Evaluation of regularity index and pushover results (12-I-a, force style-B)

Frame 12-I-b

The sectional details of frame 12-I-b are shown in Table B.5 (see Appendix B). Results from the regularity index evaluation and the pushover analysis are presented in Table 5.9 and in Figure 5.13. It can be seen from Table 5.9 that the strength and stiffness profiles of 12-I-b are much more regular than those of 12-I-a in cases of both Style-A and Style-B. 12-I-b also shows better regularity than 12-R corresponding to Style-B which can be observed from a comparison between Figure 5.11e and 5.13e. Generally, a good correlation between the distribution of i_{sc} and the drift profile from the pushover analysis is observed.

Table 5.9 Factors related to regularity index (12-I-b)

Lateral force	Storey	1	2	3	4	5	6	7	8	9	10	11	12	$\frac{(i)_{\min}}{(i)_{ave}}$
Style-A	Overstrength factor (column)	3.21	4.55	4.56	4.61	3.39	3.60	3.83	4.20	3.14	3.77	3.87	5.62	0.80
	Overstrength factor (joints)	2.15	1.57	1.63	1.95	1.42	1.44	1.61	2.07	1.62	2.00	1.97	3.66	0.74
	Normalized stiffness factor	1.00	0.90	0.94	0.99	0.80	0.67	0.75	0.88	0.75	0.94	1.38	2.82	0.70
	Storey capacity factor	2.15	1.41	1.51	1.88	1.09	0.91	1.11	1.59	1.00	1.41	1.71	4.27	0.60
Style-B	Overstrength factor (column)	3.21	4.53	4.50	4.50	3.26	3.38	3.50	3.68	2.58	2.82	2.43	2.32	0.72
	Overstrength factor (joints)	2.15	1.56	1.61	1.90	1.36	1.36	1.47	1.81	1.33	1.50	1.24	1.52	0.79
	Normalized stiffness factor	1.00	0.90	0.93	0.97	0.77	0.63	0.69	0.77	0.62	0.52	0.64	0.87	0.67
	Storey capacity factor	2.15	1.40	1.49	1.75	1.05	0.86	1.02	1.39	0.82	0.78	0.80	1.31	0.63

Figure 5.13 Evaluation of regularity index and pushover results (12-I-b, force style-B)

5.4 Nonlinear time history analysis

The computer program IDARC 2D 4.0 (Kunnath et al. 1992) is used to perform nonlinear static pushover analysis and dynamic time history analysis. In this section, a brief introduction of IDARC 2D 4.0 on issues relevant to this study is provided.

5.4.1 Component modeling

RC column elements are modeled using macro models with inelastic flexural deformations, and elastic shear and axial deformations. RC beam elements are

modeled using a nonlinear stiffness model, considering linear elastic shear deformations. For shear walls, inelastic shear and bending deformations are considered, with a de-coupled elastic axial component.

5.4.2 Structural modeling

The component models described above are used to assemble a three-dimensional structural system possibly composed of columns, beams, shear walls, slabs and transverse elements. It also allows the integration of ductile moment-resisting frames with shear-wall models and out-of-plane transverse elements.

Two classes of structures can be modeled by IDARC. The first one includes the structures where the floors may be treated as rigid components within their plane. For structures with long narrow rectangular plans, where in-plane floor deformations may have to be explicitly considered in the analysis, a floor slab elements is incorporated. For both classes of structures, the beam-column elements are modeled as continuous equivalent shear-flexure springs.

5.4.3 Hysteretic modeling

A robust hysteretic model is available in which a non-symmetric trilinear curve is utilized in conjunction with three user-specified parameters, α_{sd} , β_e , γ_p , to produce the effects of stiffness degradation, strength deterioration and pinching, respectively. Stiffness degradation is achieved by reducing the unloading stiffness as a function of ductility. Strength deterioration is introduced as a function of dissipated hysteretic energy. Pinching or slip behavior (caused by crack-closing) is incorporated by lowering the target point of the force-deformation curve upon crossing the force-axis. Since the release of IDARC 3.0 (Kunnath et al. 1992), the hysteretic model has been extended to four parameters (α_{sd} , β_e , β_d and γ_p) by allowing the strength deterioration to depend on the hysteretic energy, the ductility, or both. More details can be found in the relevant sections in Chapter 2.

The variation of the parameters enables the simulation of different hysteretic models. In this study, the general Takeda's hysteretic model (Takeda et al. 1970) is adopted to simulate the hysteretic characteristics of beams and columns.

5.5 Discussion on the time history analysis results

5.5.1 Results for frame 7-R

Frame 7-R is analyzed for the 30 ground motions described in Section 5.2. All these ground motions are scaled to a uniform A_{95} of 0.26g, corresponding to a mean PGA of 0.496g, to induce sufficient nonlinear response.

Hysteretic behavior

The hysteretic response reflects the energy dissipation as well as the inelastic stability of a structure. Figure 5.14 plots the hysteretic loops of some typical columns and inter-storey units under one of the ground motions. It is observed that frame 7-R maintains a stable hysteretic response throughout the time history.

**Figure 5.14 Hysteretic loops of typical columns and storeys
(7-R, ground motion 5028 H-E07140)**

Modal effects

Figure 5.15 depicts some typical plastic hinge patterns. Figure 5.15a exhibits more a first-mode dominated response, as the plastic hinges occur in the base storey columns and in almost all beam-ends at upper storeys. High mode effects can be observed from the pattern shown in Figure 5.15b, where extensive nonlinear response occurs in the upper storeys. Figure 5.16 illustrates the displacement profiles of the frame at the time around the maximum roof displacement response. In general, the first mode dominance can be observed.

Figure 5.15 Final states of frame under certain ground motions (7-R, $A_{95}=0.26g$)

Ground motion: 135 PEL090

Time range: 6.4s~8.4s; step length: 0.1s

Time of maximum roof displacement: 7.45s

Time of maximum first floor displacement: 7.50s

Figure 5.16 Displacement profiles at time around maximum roof displacement (7-R)

Distribution of inter-storey drifts

Table B.6 (see Appendix B) summaries the maximum inter-storey displacements of frame 7-R subject to the 30 selected ground motions. It is observed that in nearly half of the 30 cases the largest inter-storey drift ratios occur at the second storey, and some occurred at the fifth and sixth storeys. Figure 5.17 shows the distribution of the mean maximum inter-storey drift ratios from the 30 analyses. The mean maximum of inter-storey drift ratio of the first storey is less than those of the second and third storeys. The results correlate well with the distribution of storey capacity factor shown in Figure 5.8.

Maximum inter-storey drift ratios θ_{max} and peak roof drift ratios $\theta_{r, peak}$

For each ground motion case, there is a maximum inter-storey drift ratio θ_{max} among the 7 storeys and a peak roof drift ratio. The relationship between these two parameters reflects the deformation concentration of the nonlinear behavior of the structures under the particular ground motion. Figure 5.18 shows the scatter plot of θ_{max} and $\theta_{r, peak}$. From the plots, roughly a linear relationship between these two quantities can be observed. By regression analysis, a simple regression equation is derived as follows:

$$\theta_{max} = 1.566\theta_{r, peak} \quad (5.13)$$

Probabilistic properties of drift data

For each storey, the probabilistic properties (mean value, COV and probability distributions) of the maximum inter-storey drift $\theta_{max, i}$ are evaluated. The results are given in Table B.6. It is found that the log-normal distribution can fit the distribution of $\theta_{max, i}$ very well.

The COV of θ_{max} and $\theta_{r, peak}$ are almost the same at approximately 63%. The high value of COV implies that significant uncertainties exist in estimating these two parameters from a group of earthquake records. The ratio of $\theta_{max}/\theta_{r, peak}$ has a mean equal to 1.628, which is close to the regression parameter 1.566 in Equation 5.13, and a relatively low COV of 22.0%. Such a low COV manifests a good correlation between θ_{max} and $\theta_{r, peak}$.

Figure 5.17 Distribution of mean maximum inter-storey drift ratios (7-R, $A_{95}=0.26g$)

Figure 5.18 Scatter plot of θ_{max} and $\theta_{r,peak}$ (7-R, $A_{95}=0.26g$)

Results of 7-R from ground motions with lower intensities

To evaluate the effect of nonlinear response level on the relationship between θ_{max} and $\theta_{r,peak}$, the seismic response of the same frame (7-R) under lower intensity ground motions are also calculated. The same group of 30 ground motions is used, but the intensity is scaled to a lower level of A_{95} equal to 0.20g, corresponding to a mean PGA of 0.353g. The computed displacement results are tabulated in Table B.7 (see Appendix B).

Similar to the results from the ground motions with a higher intensity, the maximum inter-storey drift occurs in most cases at storey 2 and 6. The vertical distribution of mean maximum inter-storey drift ratios, plotted in Figure 5.19a, shows a similar pattern as in Figure 5.17, especially in the first four storeys. A good correlation with the storey capacity profile shown in Figure 5.8 is also observed.

The scatter plot of θ_{max} and $\theta_{r,peak}$, shown in Figure 5.19b, also exhibits a rough linear relationship between these two parameters. A regression analysis yields a regression equation as:

$$\theta_{max} = 1.539\theta_{r,peak} \quad (5.14)$$

In this case, the COV values of θ_{max} and $\theta_{r,peak}$ are 51.6% and 52.7% respectively, which are smaller than those from ground motions of a higher intensity. This is reasonable because higher uncertainty is expected to occur in case of higher levels of nonlinear response due to the non-linear behavior as well as the formation of various storey mechanisms.

Figure 5.19 Displacement response of frame 7-R ($A_{95}=0.20g$)

Effects of nonlinear response level

Miranda (1999) proposed an empirical equation to estimate θ_{max} from $\theta_{r,peak}$ as:

$$\theta_{max} = \beta_2 \beta_4 \theta_{r,peak} \quad (5.15)$$

$$\beta_2 = \max \left[\frac{du(z_j)}{dz} \frac{H}{u(H)} \right] \quad (5.16)$$

$$\beta_4 = 1 + \frac{\mu_{max}}{30} + \frac{N_s}{200} \quad (5.17)$$

where z_j = height of the j^{th} floor (measured from the ground level); H = total height of the building; $u(z_j)$ and $u(H)$ are the lateral displacements at the j^{th} floor level and the roof; $\mu_{\max}$ = maximum storey displacement ductility ratio; and N_s = number of storeys. β_2 takes into account non-uniformity of inter-storey drifts along the height of the building. While β_4 reflects the effect of nonlinear response on the displacement profile and the concentration of the inter-storey drifts, in relation to the maximum storey ductility ratio and the number of storeys for frames.

In the present case, the mean maximum storey displacement ductility produced by ground motions with $A_{95}=0.26g$ and $A_{95}=0.20g$ are equal to 2.97 and 1.77, respectively. From Equation 5.15~5.17, increasing μ by a factor of 1.2 will increase β_4 by only 0.04, which means the nonlinear response level affects the ratio of $\theta_{\max}$ to $\theta_{r,peak}$ very slightly. This actually echoes the results from the present study as one compares Equation 5.13 and 5.14. The concentration factor changes from 1.566 to 1.539 when A_{95} is reduced from 0.26g to 0.20g. Based on these observations, it may be concluded that the nonlinear response level affects the ratio of $\theta_{\max}$ to $\theta_{r,peak}$ only slightly and this influence may be neglected in a design application.

Equation 5.17 also suggests an even slighter effect of number of storeys on the ratio of $\theta_{\max}$ to $\theta_{r,peak}$ in nonlinear response range. This will be examined in the later part of this chapter.

5.5.2 Results for 7-I-a and 7-I-b

To evaluate the effect of regularity index, α_{sc} , on the seismic displacement response of frame structures, frames 7-I-a and 7-I-b with lower values of α_{sc} are analyzed with a similar procedure as for frame 7-R under the 30 ground motions scaled to $A_{95}=0.26g$. The maximum inter-storey drift ratios for 7-I-a and 7-I-b are tabulated in Table B.8 and B.9 (see Appendix B), respectively. Figure 5.20 plots the hysteretic loops of some columns and storeys for 7-I-a and 7-I-b. Stable hysteretic behavior can be observed. Figure 5.21 compares the final plastic hinge patterns of 7-I-a and 7-I-b under the same ground motion. As a result of less longitudinal reinforcement, there are more plastic hinges at the 5th storey in 7-I-b than in 7-I-a.

Figure 5.20 Hysteretic loops of typical columns and storeys (7-I-a & 7-I-b, ground motion 955 H-E04140)

Figure 5.21 Final states of frame under certain ground motions (7-I-a and 7-I-b, ground motion 955 H-E04140)

Due to the abrupt reduction in storey shear strength at the 5th storey, for more than 80% of the ground motions the maximum inter-storey drift ratios occur at the 5th or 6th storey for both frames 7-I-a and 7-I-b. This is in fact expected according to the storey capacity factors shown in Figure 5.9d and 5.10d. Figure 5.22 illustrates the profiles of the mean maximum inter-storey drift ratios of 7-I-a and 7-I-b. The COV of θ_{max} and $\theta_{r,peak}$ for 7-I-a are 57.6% and 63.1%, respectively. Similar COV values of θ_{max} and $\theta_{r,peak}$ for 7-I-b are obtained (57.0% and 63.8%).

Figure 5.23a & b show the scatter plots of the maximum inter-storey drift ratios θ_{max} and peak roof drift ratios $\theta_{r,peak}$ for 7-I-a and 7-I-b. The relationship between θ_{max} and $\theta_{r,peak}$ also appears to be roughly linear, and can be expressed from the regression analysis as

$$\theta_{max} = 1.888\theta_{r,peak} \quad \text{for 7-I-a} \quad (5.18)$$

and

$$\theta_{max} = 2.255\theta_{r,peak} \quad \text{for 7-I-b} \quad (5.19)$$

For the three 7-storey frame cases with varying regularity, Figure 5.23c depicts the scatter plot of the drift concentration factor ($\theta_{\max}/\theta_{r,peak}$) versus the regularity factor α_{sc} . As can be clearly seen, the increase of irregularity (decrease of α_{sc}) results in an increase in the concentration factor, apparently due to increased concentration of the nonlinear deformation. By a regression analysis, the following equation is proposed:

$$\theta_{\max}/\theta_{r,peak} = \alpha_{sc}^{-1.165} \quad (5.20)$$

The trend line according to the above equation is also shown in Figure 5.23c. The choice of the exponential form is to satisfy the condition that the ratio would only approach 1.0 when α_{sc} approaches 1.0.

Figure 5.22 Distribution of mean maximum inter-storey drift ratios

Figure 5.23 Scatter plots of 7-storey frames

5.5.3 Results for 12-R

Figure 5.24 shows the hysteretic loops of typical column sections and storeys for frame 12-R. Stable hysteretic behavior can be observed. Figure 5.25 illustrates the displacement profiles around the time when the maximum roof displacement occurs. The high-mode contribution is clearly observed.

Table B.10 (see Appendix B) summarizes the results of the maximum inter-storey drift ratio for frame 12-R. The distribution of the mean maximum inter-storey drifts shown in Figure 5.26a is seen to correlate very well with the distribution of the storey capacity factors shown in Figure 5.11d. This to some extent also indicates that the static analysis based on style-B lateral force distribution can better reflect the actual dynamic response of such high-rise frames.

Figure 5.24 Plots of hysteretic response of typical columns and storeys (12-R, ground motion 5028 H-E07140)

Ground motion: 135 PEL090

Time range: 6.45s~8.35s; step length: 0.1s

Time of maximum roof displacement: 7.53s

Time of maximum first floor displacement: 7.27s

**Figure 5.25 Displacement profiles at time around maximum roof displacement
(12-R)**

The COV values for θ_{max} and $\theta_{r,peak}$ are 38.7% and 58.7%, respectively. The higher COV value for $\theta_{r,peak}$ may be attributed to the multiple modes effect on the dynamic response of this 12-storey frame. Figure 5.26b shows the scatter plot of θ_{max} and $\theta_{r,peak}$ for 12-R. A rough linear regression formula can be obtained as:

$$\theta_{max} = 2.170\theta_{r,peak} \quad (5.21)$$

Figure 5.26 Displacement response of frame 12-R

5.5.4 Results for 12-I-a and 12-I-b

For these relatively more irregular frames, the maximum inter-storey drift ratios from the dynamic analysis are tabulated in Table B.11 and B.12 (see Appendix B). For 80% of the ground motions the maximum inter-storey drift ratios occur at the 10th storey for 12-I-a and at 10th~12th storey for 12-I-b. The distributions of mean maximum inter-storey drift ratios are shown in Figure 5.27.

Abrupt increase in the inter-storey drift ratios occurs at storeys near the top for 12-I-a, which is in fact predictable by the regularity evaluation shown in Figure 5.12. For frame 12-I-b, which is less irregular as predicted by the regularity index evaluation ($\alpha_{sc}=0.62$), the actual distribution of the inter-storey drift ratios is indeed more uniform than frame 12-I-a. Good correlation is again observed between the distributions of storey capacity factors (Figure 5.12d and 5.13d) and the profiles of the mean maximum inter-storey drift ratios (Figure 5.27).

The COV of θ_{max} and $\theta_{r,peak}$ are found to be 63.5% and 71.7% respectively for frame 12-I-a, and 41.0% and 65.4% for frame 12-I-b. The higher COV in $\theta_{r,peak}$ may be attributed to the high-mode effect on the dynamic response of these 12-storey frames.

Figure 5.28a & b show the scatter plots of θ_{max} and $\theta_{r,peak}$ for 12-I-a and 12-I-b, respectively. The following linear regression equations are obtained:

$$\theta_{max} = 3.245\theta_{r,peak} \quad \text{for frame 12-I-a} \quad (5.22)$$

and

$$\theta_{max} = 1.789\theta_{r,peak} \quad \text{for frame 12-I-b} \quad (5.23)$$

With regard to the important relationship between the drift concentration factor ($\theta_{max}/\theta_{r,peak}$) and the regularity factor α_{sc} , Figure 5.28c shows the scatter plots of $\theta_{max}/\theta_{r,peak}$ vs. α_{sc} for three 12-storey frames. The trend can be approximated by an exponential function in a similar fashion as for the 7-storey frame as

$$\theta_{max}/\theta_{r,peak} = \alpha_w^{-1.111} \quad (5.24)$$

It is noted that the values of α_{sc} used in the regression analysis for the 12-storey frames are based on the style-B lateral force distribution to reflect the high-mode effect. Comparing Equation 5.24 with Equation 5.21, the difference is quite small and may be neglected. Figure 5.28d combines the scatter plots of $\theta_{max}/\theta_{r,peak}$ and α_{sc} from both the 7-storey and 12-storey groups of frames. The trend is very consistent and this tends to suggest that the drift concentration factor $\theta_{max}/\theta_{r,peak}$ is rather independent of the number of storeys and may be evaluated from the appropriate regularity index α_{sc} only. In other words, the effect of the number of storeys has been somehow incorporated in the evaluation of the regularity index. Based on the results as shown in Figure 5.28d, a unified regression equation is obtained as

$$\theta_{max}/\theta_{r,peak} = \alpha_w^{-1.129} \quad (5.25)$$

Due to the limited number of frame cases investigated in this study, Equation 5.25 is recommended to be applied to medium-to-high-rise frame structures.

Figure 5.27 Distribution of mean maximum inter-storey drift ratios (12-I-a & 12-I-b)

Figure 5.28 Scatter plots of 12-storey frames and all 6 frames

5.6 Performance-oriented seismic reliability assessment based on critical inter-storey drift

5.6.1 Probabilistic model of critical inter-storey drift ratio

Based on the results regarding the inter-storey drift concentration factor and the variance of the drift response under a given type of earthquakes, the probabilistic critical inter-storey drift demand for a given RC frame can be established through an approximate procedure as follows.

First, the distribution of the storey capacity factors and the corresponding regularity index are evaluated using the procedure proposed in Section 5.3.3. At the same time, the weakest storey (or storeys) can be identified. Since the maximum inter-storey drift may also occur in storeys whose capacity factors are close to that of the weakest storey, it is suggested that those storeys with the storey capacity factor within 1.0~1.1 times of the lowest value are also considered in the evaluation.

Assuming a fundamental mode dominated response, the roof displacement of the frame system can be related to the corresponding spectral displacement of the SDOF oscillator having the period equal to the fundamental period of the frame, as

$$\Delta_{roof} = \Gamma S_d \quad (5.26)$$

where Γ is the first-mode participation factor when the mode shape is normalized to one at the roof; S_d is the mean spectral displacement for the targeted earthquake intensity. The critical inter-storey drift can be estimated as

$$\theta_{ci} = \frac{\Delta_{ci}}{h_{ci}} = \beta_c \frac{\Delta_{roof}}{H} = \beta_c \frac{\Gamma S_d}{H} \quad (5.27)$$

where H is the total height of the frame; h_i is the storey height of the weakest storey; β_c is the concentration factor concerning the maximum inter-storey drift, $\beta_c = \theta_{\max} / \theta_{r,peak}$. The β_c value can be estimated from Equation 5.25 as a function of the regularity index.

The resulted value of critical inter-storey drift ratio θ_{ci} can be treated as the mean value. According to the probabilistic analysis described in Section 5.5, the log-normal

distribution is suitable to model the probabilistic property of the maximum inter-storey drift ratios. Based on the results about the COV values of θ_{max} and $\theta_{r,peak}$ as is summarized in Table 5.10, it is found that the COV of $\theta_{r,peak}$ is rather stable while the COV of θ_{max} exhibits certain fluctuation. Nevertheless, in general the COV of θ_{max} and $\theta_{r,peak}$ can be considered as stable. Using the arithmetic mean of all COV values shown in Table 5.10 to represent a unified COV, it has $COV \approx 60\%$.

Table 5.10 Summary of COV of θ_{max} and $\theta_{r,peak}$

	7-R ($A_{95}=0.26g$)	7-R ($A_{95}=0.20g$)	7-I-a	7-I-b	12-R	12-I-a	12-I-b
COV of θ_{max}	62.9%	51.6%	57.6%	57.0%	38.7%	63.5%	41.0%
COV of $\theta_{r,peak}$	63.3%	52.7%	63.1%	63.8%	58.7%	71.7%	65.4%

The flowchart of the above procedure is illustrated in Figure 5.29.

Figure 5.29 Flowchart of establishing probabilistic model of critical inter-storey drift

5.6.2 Application example

7-storey frame: 7-cs

A simple case study to evaluate the performance fragility curves for the critical storey of an RC frame building is carried out in this section. A 7-storey RC frame designated as 7-cs (Table B.13, Appendix B), with the same geometry and section dimensions but different sectional reinforcement of frame 7-I-a is employed for this purpose. The fundamental period of 7-cs is equal to 1.0s.

Based on the procedures proposed in this chapter, the storey capacity factors and the corresponding regularity index of the frame are evaluated. The results are summarized in Table 5.11 and Figure 5.30. It is found that the absolute values of the overstrength factors and storey capacity factors are generally smaller than those of 7-I-a due to the increased lateral force demand. However, the distribution shape and the regularity index changes slightly. Using Equation 5.26, the critical drift concentration factor is found to be 1.85 for the regularity index of 0.58.

Table 5.11 Factors related to regularity index (7-cs, force style-A)

Storey	1	2	3	4	5	6	7	$\frac{(i)_{\min}}{(i)_{\text{ave}}}$
Overstrength factor (column)	3.07	3.85	3.19	3.03	2.19	2.79	3.77	0.70
Overstrength factor (joints)	2.17	1.80	1.69	1.98	1.65	2.22	3.38	0.78
Normalized stiffness factor	1.00	0.99	0.96	1.11	0.88	0.94	1.81	0.80
Storey capacity factor	2.17	1.78	1.61	2.19	1.44	2.08	6.10	0.58

Figure 5.30 Distribution of storey capacity factors (7-cs, force style-A)

From the distribution of the storey capacity factors, the 5th storey appears to be the weakest storey in this frame. Given the details of columns at this storey, the probabilistic model of column's displacement capacity at different performance levels can be established based on the procedure proposed in Chapter 3. Table 5.12 tabulates the relevant statistic information and the deterministic yield displacement capacity for these columns. Since the drift properties do not differ significantly among the columns of this storey, the maximum mean drift capacity and the average value of COV of the three columns are used to represent the probabilistic properties of the entire storey.

Table 5.12 Probabilistic displacement capacity the 5th storey of frame 7-cs

	Axial force ratio	Performance level	Inter-storey drift ratio		Probability distribution
			mean	COV	
External columns	0.062	yield	0.0064	-	-
		Functional	0.012	12.2%	Normal
		Damage Control	0.032	33.5%	Normal
		Ultimate	0.059	35.8%	Normal
Internal column	0.115	yield	0.0066	-	-
		Functional	0.012	13.0%	Normal
		Damage Control	0.029	39.7%	Normal
		Ultimate	0.047	42.7%	Normal
Storey		yield	0.0066	-	-
		Functional	0.012	12.5%	Normal
		Damage Control	0.032	35.6%	Normal
		Ultimate	0.059	38.1%	Normal

Using the column drift capacity to represent the storey drift capacity is based on an assumption of rigid floor which is not theoretically valid for a weak-beam strong-column frame. However, it is believed that the column drift capacity at some performance level can rationally represent the corresponding storey drift capacity in number. The reason is presented in the following paragraph.

The drift capacity of a whole storey at any performance level (related to structural components) is constituted by the deformation of all columns and beams. The storey drift capacity at a performance level only based on column capacity overestimates the column deformation while the effect of beam-end rotation is neglected. The overestimation of column deformation and underestimation of beam deformation may

result in a storey drift close to the real value. Based on the comparison of inter-storey drift ratios at ultimate state (Collapse level) between the simulated and experimental results (Chapter 3), it is found that the distribution of simulated Collapse inter-storey drift ratio matches that of the experimental results very well. Therefore, the statistic property of column drift capacity is applied to establish the probabilistic model of the storey drift capacity in the present performance reliability analysis.

For the non-structural performance, the probabilistic models of drift limits of infill masonry wall established in Chapter 4 are employed here as the representative of non-structural drift capacity. The details of the probabilistic model of non-structural drift limits at three performance levels are listed in Table 5.13.

Table 5.13 Probabilistic model of non-structural displacement capacity

Performance level	Non-structural (infill masonry wall)		
	Mean	COV	Distribution
Functional	0.254%	82.8%	Log-normal
Damage Control	0.541%	74.8%	Log-normal
Ultimate	0.801%	43.1%	Normal

Estimation of displacement demands from displacement response spectra

The roof displacement demand can be evaluated from the displacement response spectra according to Equation 5.26. The first-mode participation factor, Γ , can be expressed as

$$\Gamma = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i^2} \quad (5.28)$$

where m_i is the lumped mass at the i^{th} storey and Φ is the assumed displacement shape. The Γ value of frame 7-cs is obtained to be 1.43.

To keep the consistency in the seismic input, the same group of ground motions as used in the earlier part of this chapter are used to evaluate the elastic displacement spectrum. The program of NSPECTRA (Reinhorn et al. 1998) is employed for the computation of the displacement response spectrum. The generated elastic

displacement spectra for 5%-damped systems are presented in terms of mean spectrum in Figure 5.31.

In determining the inelastic spectral displacement S_d , the substituted system approach is employed in which the secant stiffness and an equivalent damping representing the inelastic hysteresis effect are considered. For each level of earthquake intensity, a few iterations need to be performed until results converge.

Figure 5.31 Elastic displacement spectrum of 30 selected ground motions

Performance reliability evaluation

With the establishment of the random drift intervals for the three performance levels and the random drift demand, the performance reliability of the frame concerning the critical storey can be evaluated in terms of the performance occurrence probability P_O and exceeding probability P_E using the fuzzy-random reliability model proposed in Chapter 4. For different levels of ground motion intensity (herein represented by A_{95}), the P_O and P_E can be calculated and subsequently two sets of performance curves, i.e., P_O vs. A_{95} and P_E vs. A_{95} (P_E is also called fragility) can be produced. A more general procedure for the generation of the performance curves for framed structures is summarized in Figure 5.32.

Figure 5.32 Flowchart for construction of performance curves for frame structures

For the particular example case herein, the P_O and P_E corresponding to the three performance levels (assuming $\gamma = 0.1$) are shown in Figure 5.33. Figure 5.34 compares the fragility curves (P_E) for performance levels using respectively the non-structural drift limit and structural drift limit separately with the classical reliability model, and the combined criteria with the fuzzy-random model. As expected, there is a quite significant difference between the fragility curves obtained using separately the non-structural and structural limit criteria. For Damage Control level, at an A_{95} level about 0.3g, the failure probability based on the non-structural criteria exceeds 90% while that based on structural criteria is still at a lower value of less than 10%. The fragility curve obtained using the fuzzy-random criteria falls in-between the non-structural and structural criteria governed curves. Through the use of different values of γ , the relative importance of the non-structural and structural components can be emphasized. With the increase of γ than 1, the fragility curve is shifted towards the structural side.

a) Exceeding probability (P_E) b) Probability of occurrence (P_O)**Figure 5.33 Fuzzy-random fragility curves for three performance levels**

a) Functional level

b) Damage Control level

c) Collapse level

Figure 5.34 Comparison of fragility curves generated using different criteria

5.7 Summary and conclusions

In this chapter, the relationship between the critical inter-storey drift and the roof drift in framed structures is investigated. This is an important parameter in a simplified analysis procedure using SDOF analogy. A modified scheme is proposed here to evaluate the regularity of the storey strength and stiffness distribution along the structure height. In this regard, a new storey “capacity” factor is introduced in this study based on which the improved regularity index of a frame structure can be computed. The distributions of the storey capacity factor are found to correlate well with the displacement profiles of frames obtained by nonlinear pushover analysis.

Six RC frame structures are designed to represent medium-to-high rise frames with different regularity profiles, and they are subjected to a group of 30 ground motions. The effects of the regularity index, nonlinear response level and the number of storeys on the inter-storey drift concentration factor, $\theta_{\max}/\theta_{r,peak}$, are examined by using the results from nonlinear time history analysis performed in this chapter. It is found that the nonlinear response level affects the concentration factor very slightly. However, the concentration factor depends very much on the regularity index by which the effect of number of storeys is somehow incorporated. Based on a regression analysis, an empirical relationship is proposed in the present study to describe the relationship between the drift concentration factor and the regularity index. With the relationship, the critical drift demand on a framed structure can be established more reliably using a SDOF analogy and the displacement response spectrum.

The ground motions selected here represent a stiff site under near field earthquakes. The focus of this chapter is the establishment of an improved framework for estimating the concentration of inelastic response at a critical storey in an RC frame. The method is generally applicable but the specific results from using the selected group of ground motions is suitable for similar site conditions. For other site conditions, especially the soft soil site where ground motions may have lower frequency contents, the concentration coefficient ($\theta_{\max}/\theta_{r,peak}$) needs to be evaluated accordingly under the proposed framework.

Subsequently, the performance reliability of the frame structure can be evaluated. An example application is given with a 7-storey RC frame to demonstrate the proposed procedure for generating the performance reliability curves as the final part of this chapter.

Chapter 6 Conclusions and recommendations

6.1 Conclusions

This thesis presents the research work on two important aspects in the performance-based design and performance reliability evaluation of RC framed structures, one concerns the quantification of the limit states for the various performance levels, another concerns the prediction of seismic demands in terms of critical inter-storey drift. Various uncertainties are considered to come up with the corresponding probabilistic models for describing the demands and the limit state capacities. In the quantification of the performance limit states, the fuzzy set theory is adopted to deal with the vagueness in balancing the structural and non-structural damages at a particular performance level. Subsequently, a fuzzy-random reliability model is developed for the seismic performance reliability analysis taking into account structural and non-structural damages.

The following summarizes the main conclusions in four aspects.

a) On the drift limits concerning RC components for different performance levels

Acknowledging the unclear boundary between some performance levels suggested by SEAOC (1995), three distinctive performance levels, namely Functional, Damage Control and Collapse, are proposed to allow for more explicit translation into physical quantities.

By Monte Carlo Simulation, the probabilistic deformation limits for RC members at both sectional and member (“inter-storey”) levels are investigated. Results show that in general, the drift limits follow the normal distribution at all three performance levels. For all cases studied, the overall mean Functional drift limit is about 1.08%, with a low COV equal to 10.5%.

The drift limit at Damage Control performance level involves considerable inelastic deformation at the plastic hinge region and hence is affected significantly by the reinforcement details and axial force ratios. The mean drift limits at DC level vary from 2.0% to 4.5%, with COV ranging from 30% to 40%. The characteristic DC level drift limits (with 95% probability of being exceeded) range from 0.8% to 2.16%.

At the ultimate state for Collapse, the mean limits for different columns vary in 3.0~7.8% and COV ranges from 30% to 45%. The characteristic ultimate drift limits range from 1.13% to 3.33%. The combined distribution of the simulated ultimate drifts for several design cases shows a good correlation with the experimental data on the ultimate inter-storey drifts of RC frames, and both the experimental and the combined simulation results for the ultimate drift limits appear to follow a log-normal distribution.

b) On the drift limits concerning non-structural masonry infill walls for different performance levels

Due to the limited information about the general non-structural behavior, the masonry infill wall, which constitutes a significant source of dollar losses in earthquakes, is deemed as the representative of general non-structural components.

Based on the analysis of a large number of masonry wall specimens, the drift limit at Functional level can be described approximately by log-normal distribution with a mean equal to 0.254% and COV equal to 82.8%. The Damage Control drift limit also follows a log-normal distribution with a mean of 0.541% and COV of 74.8%. Normal distribution is found to well fit the Collapse drift limit which has a mean equal to 0.801% and COV equal to 43.1%.

Significant deviation between these two sets of limit drift ratios for non-structural and structural components at each performance level is observed. Some compromises are required to take into account both the structural and non-structural damage in a comprehensive performance assessment.

c) On the fuzzy-random model for the performance limit states and the performance reliability analysis

The Fuzzy set theory is deemed a suitable approach for a rigorous definition of the performance levels taking into account both the non-structural and structural damages. In this respect, each performance level can be defined as a fuzzy-random interval bounded by the limit drifts respectively representing non-structural and structural performances. With two membership functions, the Occurrence and the Exceeding of a performance level can be appropriately described. The evaluation of the “Occurrence” probability of a certain performance level provides a useful measure which may be applied in an optimization procedure in achieving multiple performance targets.

With the formulation of the fuzzy random reliability model, the performance reliability of a building system can be evaluated in a more comprehensive manner than the classic reliability approach based on single-threshold limit criteria. The relative importance of the non-structural and structural damage at a target performance level can be readily reflected by adjusting the cost function curve.

d) On the estimation of seismic demand on a framed system in terms of critical drift

The new storey capacity factor incorporates the possible effects of both the storey strength and the storey stiffness on the distribution of the inter-storey drift in a frame structure, or more generally a shear type structure. Comparison with the results from detailed nonlinear time history analysis indicates that the distribution of storey capacity factors correlates well with the actual distribution of the inter-storey drifts. The weak storeys in a frame can be identified rather accurately by the storey capacity factor.

A modified regularity index defined on the basis of the new storey capacity factor is found to indicate the actual regularity more adequately than that based on the classical storey shear overstrength factor.

Because of the improved correlation using the modified regularity index, the inter-storey drift concentration factor, which is defined as the ratio of the maximum inter-storey drift to the roof drift, can be described as a function of the regularity index in a simple and straightforward manner, without the need of incorporating other correction coefficients. The effect of nonlinear response level on the amplification factor is found to be insignificant.

The proposed inter-storey drift concentration factor enables the rapid estimation of the critical drift demand on a frame structure from displacement spectra using a simple procedure, hence allowing for a more convenient implementation of a performance-based design or performance-oriented reliability analysis.

6.2 Recommendations on future research

Several important aspects are also identified to be worthy of further study and improvements in line with the performance-based seismic design and reliability analysis of building systems.

Systematic experimental data regarding the performance limit states of building systems, especially concerning the service level performances, are needed. These experiments should cover both structural and pertinent non-structural contents and have adequate representation of the system behavior. A thorough survey of non-structural damage from existing earthquake damage records is also needed to fully make use of past earthquake experiences for the determination of appropriate limit states for non-structural performances. In the present study, the non-structural limit is represented by infill walls for which the limit is still behavior-oriented. When other non-structural aspects come into play, the determination of the corresponding drift limit may become more of a matter of cost than behavior. In that case, the determination of the drift limits itself reflects the actual cost due to building damage.

The adequacy of using drift limits derived at component (column) level to represent the storey performance concerning structural damage needs to be verified, and modified where appropriate, through experimental and numerical studies.

The construction of the cost function in the fuzzy-random performance reliability model needs to be further explored. With the form of the cost function as used in this study, it is useful to establish an explicit relationship between the γ -coefficient and different building categories with clearer physical implications.

The framework on the estimation of the critical drift demand in a frame structure in relation with the storey capacity factor is expected to generally apply. However, the precise relationship between the drift concentration factor and the regularity index defined from the storey capacity factor may vary for different types of earthquakes and for structures of significantly different complexities. In this regard, further parametric investigations are required to expose such differences and establish suitable formulas accordingly.

Reference

Abrams D. P. (1992), “Strength and Behavior of Unreinforced Masonry Elements”, Proceedings, Tenth World Conference on Earthquake Engineering, Madrid, pp. 3475-3480.

ACI 318 (1995), “Building Code Requirements for Structural Concrete (ACI318-95) and Commentary (ACI318R-95)”, American Concrete Institute.

Algan B. B. (1982), “Drift and Damage Considerations in Earthquake-resistant Design of Reinforced Concrete Buildings”, PhD thesis, University of Illinois at Urbana-Champaign.

Ang B. G., Priestley M. J. N and Park R. (1981), “Ductility of Reinforced Concrete Bridge Piers Under Seismic Loading”, Report No. 81-3, Department of Civil Engineering, University of Canterbury, Christchurch.

Anthoine A., Magonette G. and Magenes G. (1994), “Shear-compression Testing and Analysis of Brick Masonry Walls”, Proceedings, Tenth European Conference on Earthquake Engineering, Vienna, pp. 1657-1662.

Arede A. and Pinto A. V. (1996), “Performance of RC Buildings Designed for Different Ductility Classes”, Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, Paper No. 1877.

Asano K. (1988), “Probabilistic Approach to Idealization of Earthquake Excitation and Aseismic Safety Analysis of Structural Systems”, Proceedings of the Ninth World Conference on Earthquake Engineering, Tokyo-Kyoto, 8, pp.735-740.

Asano K. and Tsuda N. (1992), “Structural Design Based on the Optimum Seismic Reliability”, Proceedings of the Tenth World Conference on Earthquake Engineering, Madrid, July, 7, pp.3683-3688.

ATC 40, (1996), “Recommended methodology for seismic evaluation and retrofit of buildings”, Rep. No. ATC-40, Applied Technology Council, Redwood City, California.

Banon H. and Veneziano D. (1982), “Seismic Safety of Reinforced Concrete Members and Structures”, Earthquake Engineering and Structural Dynamics, 10, pp. 179-193.

Barber, T. T. and Wen, T. K. (1979), “Random Vibration of Hysteretic Degrading Systems”, Proceedings, Third ASCE Engineering Mechanics Division Specialty Conference, pp. 51-54.

Bazzurro P. and Cornell C. A. (1994), “Seismic Hazard Analysis of Nonlinear Structures. I: Methodology”, ASCE, Journal of Structural Engineering, 120(11), pp. 3320-3344.

Beck J. L., Irfanoglu A., Papadimitriou C. and Siu K. A. (2000), “A Performance-based optimal Design Methodology Incorporating Multiple Criteria”, Proceedings of Twelfth World Conference of Earthquake Engineering, No. 344.

Benjamin J. R. and Williams A. M. (1958), “The Behavior of One-story Brick Shear Walls”, ASCE, Journal of the Structural Division, ST4, paper-1723.

Bento R. and Azevedo J. (1998), “Probabilistic Assessment of q Factors for Irregular Structures”, Proceedings of the Sixth SECED Conference on Seismic Design Practice into the Next Century, Oxford, March, pp. 435-442.

Bertero, V. V., Anderson, J. C., Krawinkler, H. and Miranda, E. (1991), “Design Guidelines for Ductility and Drift Limits”, Report No. UCB/EERC-91/15, University of California, Berkeley.

Bertero R. D. and Bertero V. V. (2002), “Performance-Based Seismic Engineering: The Need for a Reliable Conceptual Comprehensive Approach”, Earthquake Engineering and Structural Dynamics, 31, pp. 627-652.

Bommer J. J. and Elnashai A. S (1999), “Displacement Spectra for Seismic Design”, Journal of Earthquake Engineering, 3(1), pp. 1-32.

Bracci J. M. (1989), “Deterministic Model for Seismic Damage Evaluation of Reinforced Concrete Structures”, Techn. Rep. NCEER, State University of New York at Buffalo, 89-0033.

Bracci J. M., Reinhorn, A. M., and Mander, J. B. (1995), “Seismic resistance of reinforced concrete frame structures designed for gravity loads: performance of structural system”, ACI Structural Journal, 92(5), 597-609.

Calvi G. M. and Kingsley G. R. (1995), “Displacement-based Seismic Design of Multi-degree-of-freedom Bridge Structure”, Earthquake Engineering and Structural Dynamics, 24, pp. 1247-1266.

Casciati F. and Faravelli L. (1984), “Plastic Zone Spread and Seismic Reliability of RC Frames”, Proceedings of Eighth World Conference on Earthquake Engineering, San Francisco, July, 4, pp. 567-574.

CEB, 1997, Bulletin 279, “Seismic Design of Structures”, CEB, Paris.

Chavez M. and De Leon D. (1984), “Reliability of Nonlinear Systems with Uncertain Parameters and Random Seismic Excitation”, Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, July, 4, pp. 435-441.

Chen J. E. and Otto K. N. (1995), “Constructing Membership Functions Using Interpolation and Measurement Theory”, Fuzzy Sets and Systems, 73, pp. 313-327.

Chopra, A. K. and Goel, R. K. (2000), “Evaluation of NSP to estimate seismic deformation SDF systems”, ASCE, Journal of Structural Engineering, 126(4), pp. 482-490.

Chung Y. S. Meyer C. and Shinozuka M. (1989), “Modeling of Concrete Damage”, ACI Structural Journal, 86(3), pp. 259-271.

Colangelo F., Gianni R. and Pinto P. E. (1995), “Stochastic Linearization for the Assessment of Behavior Factors and Capacity Design Provisions in Eurocode 8”, Proceedings of the Tenth European Conference on Earthquake Engineering, Vienna, August-September 1994, 2, pp. 947-952.

Colangelo F., Gianni R. and Pinto P. E. (1996), “Seismic Reliability of Reinforced Concrete structures with Stochastic Properties”, Structural Safety, 18(2/3), pp. 151-168.

Corley W.G. (1966), “Rotational Capacity of Reinforcing Concrete Beams”, ASCE, Journal of Structural Division, 92, ST5, October, pp. 121-146.

Cornell C. A. (1996a), “Calculating Building Seismic Reliability: A Basis for Multi-Level Design Norms”, Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, Paper No. 2122.

Cornell C. A. (1996b), “Reliability-Based Earthquake-Resistant Design; the Future”, Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, Paper No. 2166.

Cosenza E., Manfredi G and Ramasco K (1990), “An Evaluation of the Use of Damage Functional in Earthquake-resistant Design”, Proceeding, Ninth European Conference On Earthquake Engrgineering, The Kucherenko Tsniisk of the USSR, Gosstroy, Moscow, 9, pp. 303-312.

Cosenza E. and Manfredi G. (1997), “The Improvement of the Seismic-Resistant Design for existing and New Structures Using Damage Concept” in Fajar P. and Krawinkler H. (Eds.), “Seismic Design Methodologies for the Next Generation of Codes”, Proceedings of International Conference at Bled. Slovenia, A. A. Balkema, Rotterdam/Brookfield, pp.119-130.

Costa A. C. and Duarte R. T. (1990), “Earthquake Behavior of 4 Storeys Reinforced Concrete Frame Structures Infilled with Brick Masonry Panels”, Proceedings of the Ninth European Conference on Earthquake Engineering, Moscow, 2, pp. 144-153.

Davey B.E. and Park R. (1987), “Reinforced Concrete Piers Under Seismic Loading”, Report No. 75-3, Department of Civil Engineering, University of Canterbury, Christchurch.

De Leon D. and Ang A. H. S. (1994), “A Damage Model for Reinforced Concrete Buildings: Further Study with the 1985 Mexico City Earthquake”, Proceeding, Sixth International Conference on Structural Safety and Reliability, A. A. Balkema, Rotterdam, The Netherlands, 3, pp. 2081-2087.

Dipasquale E. and Cakmak A. S. (1990), “Seismic Damage Assessment Using Linear Models”, Soil Dynamics and Earthquake Engineering, 9(4), pp. 194-215.

Dymiotis Christina, Kappos A. J. and Chryssanthopoulos M. K. (1999), “Seismic Reliability of RC Frames with Uncertain Drift and Member Capacity”, ASCE, Journal of Structural Engineering, 125(9), September, pp. 1038~1047.

Dymiotis Christina (1999), “Probability Seismic Assessment of Reinforced Concrete Buildings With and Without Masonry Infills”, Ph.D. thesis, Imperial College of Science, Technology and Medicine, London.

EC8 (1989), “Background Document: Justification note No. 13: Confined concrete constitutive law”, CEC DG III/8076/89 EN, 104-16.

EC8 (1996), “Earthquake Resistant Design of Structures, Part 1.1: General Rules”, ENV 1998-1, CEN.

EERI (1994), “Northridge Earthquake”, Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, 94-01.

Ellingwood B. (1977), “Statistical Analysis of RC Beam-Column Interaction”, ASCE, Journal of the Structural Division, 103(7), pp. 1377-1388.

Elwood K. J. and Wen Y. K. (1996), “Evaluation of a Dual-Level Design Approach for the earthquake Resistant Design of Buildings”, Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, pp. 609.

Esteva L. and Ruiz S. E. (1989), “Seismic Failure Rates of Multistorey Frames”, ASCE, Journal of Structural Engineering, 115(2), pp. 268-284.

Esteva L., Diaz-Lopez, O. J. and Mendoza E. (2000), “Reliability Indices in Earthquake Resistant Design of Multi-storey Frame Buildings”, Proceedings of the Twelfth World Conference on Earthquake Engineering, pp. 1662.

Fajar Peter (1999), “Capacity spectrum method based on inelastic demand spectra”, Earthquake Engineering Structure Dynamics, 28(9), pp. 979-993.

Fajfar Peter (2000), “A Nonlinear Analysis Method for Performance Based Seismic Design”, Earthquake Spectra, Vol.16, No.3, pp. 573-592.

Fajfar Peter and Fischinger M. (1989), “N2 - a method for non-linear seismic analysis of regular buildings”, Proceedings, 9th World Conference of Earthquake Engineering, Vol. V, Tokyo, Kyoto, Maruzen, Tokyo, 1989, pp 111-116.

Fajfar Peter & Gaspersic Peter (1996), “The N2 Method for the Seismic Damage Analysis of RC Buildings”, Earthquake Engineering and Structural Dynamics, Vol. 25, pp. 31-46.

Fardis M. N. (1995), “Damage Measures and Failure Criteria for Reinforced Concrete Members”, Proceedings of the Tenth European Conference on Earthquake Engineering, Vienna, Austria. Balkema, Rotterdam, 1, pp. 121-128.

Feghali H. L. (1999), “Seismic Performance of Flexible Concrete Structures”, Ph.D. thesis, University of Texas at Austin.

Frangopol D.M., Yutuka I., Spacone E. and Iwaki I. (1996), “A New Look at Reliability of Reinforced Concrete Columns”, Structural Safety, 18(2/3), pp. 123-150.

Gianni R., Nuti C. and Pinto P. E. (1986), “System Reliability Analysis under Earthquake Excitation”, Proceedings of the Eighth European Conference on Earthquake Engineering, Lisbon, 1, pp. 2.4/71-78.

Gianni R., Aly N. M. and El-Attar M. (1997), “Performance Level Criteria and Evaluation”, Proceedings of the International Workshop on Seismic Design Methodologies for the Next Generation of Codes, Bled Slovenia, pp. 207-215.

Gill Wayne D., Park R. and Priestley M. J. N. (1982), “Ductility of Square-Confined Concrete Columns”, ASCE Proceedings, 108, ST4, Apr., pp. 929-950.

Grant L. H., Mirza S. A. and MacGregor J. G. (1978), “Monte Carlo Study of Strength of Concrete Columns”, ACI Journal, 75(8), pp. 348-357.

Grossmeyer, R. L. (1979), “A Seismic Approach for Elasto-Plastic Systems Under Stationary Random Excitation”, Proceedings, Third ASCE Engineering Mechanics Division Specialty Conference, pp. 55-59.

Gulkan, P. and Sozen, M. (1974), “Inelastic Response of Reinforced Concrete Structures to Earthquake Motions”, ACI Journal, 71, pp. 604-610.

Gupta A. and Krawinkler H. (2000), “Estimation of Seismic Drift Demands for Frame Structures”, Earthquake Engineering and Structural Dynamics, 29, pp. 1287-1305.

Hadjian A. H. (2001), “A General Framework for Risk-Consistent Seismic Design”, Earthquake Engineering and Structural Dynamics, 31, pp. 601-626.

Hadipriono, F. C. (1985), “Assessment of Falsework Performance Using Fuzzy Set Concepts”,

Structural Safety, 3, pp. 47-57.

Hadipriono, F. C. and Lai, J. J. Y (1986), "Assessment of Urgency Measure to Prevent Further Damage of Concrete Components", Structural Safety, 4, pp. 49-62.

Han S. W. and Wen Y. K. (1997), "Method of Reliability-Based Seismic Design, I: Equivalent Nonlinear Systems", ASCE, Journal of Structural Engineering, 123(3), pp. 256-263.

Hidalgo P. A. & Luders C. (1984), "Earthquake-resistant Design of Reinforced Masonry Buildings", Proceedings, Eighth World Conference on Earthquake Engineering, San Francisco, pp. 815-822.

Holicky, M., (1998), "Fuzziness of Performance Requirements in Reliability Analysis", Proceedings, Structural Safety and Reliability, Balkema, Rotterdam, pp. 333-338.

Hong H. P. and Zhou W. (1999), "Reliability Evaluation of RC Columns", ASCE Journal of Structural Engineering, 125(7), pp. 784-790.

Hoshikuma J., Kawashima K., Nagaya K. and Taylor A.W. (1997), "Stress-Strain Model for Confined Reinforced Concrete in Bridge Piers", ASCE, Journal of Structural Engineering, 123(5), pp. 624-633.

Iwan W. D. (1980), "Estimating Inelastic Response Spectra from Elastic Spectra", Earthquake Engineering and Structural Dynamics, 8, pp. 375-388.

JRA (1996), "Design Specifications of Highway Bridges, part I common part, part II steel bridges, part III concrete bridges, part IV foundations and part V seismic design", Japan Road Association.

Kanaan A. and Powell G. H. (1973), "DRAIN-2D: A General Purpose Computer Program for Dynamic Analysis of Inelastic Plane Structures", Rep. No. EERC-73/22, Earthquake Engineering Research Center, University of California at Berkeley.

Kanda J. and Shah H. (1997), "Engineering Role in Failure Cost Evaluation for Buildings", Structural Safety, Vol. 19, No. 1, pp. 79-90.

Kandel, A. and Byatt, W. J. (1980), "Fuzzy Processes", Fuzzy Sets and Systems, 4, No.2, 117.

Kappos A.J. (1991), "Analytical Prediction of the Collapse Earthquake for RC Buildings: Suggested Methodology", Earthquake Engineering and Structural Dynamics, 20(2), pp. 167-176.

Kappos A. J., Chryssanthopoulos M. K. and Dymiotis C. (1999), "Uncertainty Analysis of Strength and Ductility of Confined Reinforced Concrete Members", Engineering Structures, 21, pp.195-208.

Kato H., Goto T. and Mizuno H. (1992), "Cyclic Loading Tests of Confined Masonry Wall Elements for Structural Design Development of Apartment Houses in The Third World", Proceedings, Tenth World Conference on Earthquake Engineering, Madrid, pp. 3539-3544.

Kawashima K. (1997), "The 1996 Japanese Seismic Design Specification of Highway Bridges and the Performance Based Design", Proceedings of International Conference at Bled., Slovenia, A. A. Balkema, Rotterdam/ Brookfield, pp. 371-382.

Kent D. C. and Park R. (1971), "Inelastic Behavior of Reinforced Concrete Members with Cyclic Loading", Bulletin of the New Zealand Society for Earthquake Engineering, 4(1), pp. 108-125.

Kowalsky M. J. (1994), "Displacement-based Design-A Methodology for Seismic Design Applied to RC Bridge Columns", Master's thesis, University of California at San Diego, La Jolla, California.

Kowalsky M. J., Priestley M. J. N. and Macrae G. A. (1995), "Displacement-Based Design of RC Bridge Columns in Seismic Regions", Earthquake Engineering and Structural Dynamics, 24, pp. 1623-1643.

Kowalsky M. J. (2000), “Deformation Limit States of Circular Reinforced Concrete Bridge Columns”, ASCE, Journal of Structural Engineering, 126, No.8, pp. 869-878.

Kramer S. L. (1996), “Geotechnical Earthquake Engineering”, Prentice-Hall International Series in Civil Engineering and Engineering Mechanics.

Kunnath, S. K., Reinhorn, A. M. and Park, Y. J. (1990), “Analytical modeling of inelastic seismic response of RC structures”, ASCE, Journal of Structural Engineering, 116(4), pp. 996-1017.

Kunnath S. K., Reinhorn A. M. and Abel J. F. (1991), “A Computational Tool for Evaluation of Seismic Performance of Reinforced Concrete Buildings”, Computer & Structures, Vol. 41, No.1, pp. 157-173.

Kunnath, S. K., Reinhorn, A. M., and Lobo, R. F. (1992), “IDARC Version 3.0: A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures,” Technical Report No. NCEER-92-0022, National Center for Earthquake Engineering Research, State University of New York at Buffalo.

Kunnath, S. K., Hoffmann, G., Reinhorn, A. M., and Mander, J. B. (1995), “Gravity load-designed reinforced concrete buildings—part II: evaluation of detailing enhancements”, ACI Structural Journal, 92(4), pp. 470-478.

Kurama Y. C. & Farrow K. T. (2003), “Ground Motion Scaling Methods for Different Site Conditions and Structure Characteristics”, Earthquake Engineering and Structural Dynamics, 32, pp. 2425-2450.

Lagorio H. J (1990), “Earthquake, An Architects Guide to Nonstructural Seismic Hazard”, Wiley.

Lai, S. P. (1978), “On Inelastic Response Spectra for Aseismic Design”, Publication R78-18, Department of Civil Engineering, MIT, Cambridge, Massachusetts.

Lu Y (2002), “Comparative Study of Seismic Behavior of Multistory Reinforced Concrete Framed Structures”, ASCE, Journal of Structural Engineering, 128(2), pp. 169-178.

Lu Y and Gu X. (2004), “Probability analysis of RC member deformation limits for different performance levels and reliability of their deterministic calculations”, Structural Safety, Vol. 26, 4, pp. 367-389.

Luo Y. H., Durrani A and Conte J. (1995), “Seismic Reliability Assessment of Existing RC Flat-slab Buildings”, ASCE, Journal of Structural Engineering, 121(10), pp. 1522-1530.

Magenes G. & Calvi G. M. (1992), “Cyclic Behavior of Brick Masonry Walls”, Proceedings, Tenth World Conference on Earthquake Engineering, Madrid, pp. 2419-2424.

Magenes G. & Calvi G. M. (1994), “Shaking Table Tests on Brick Masonry Walls”, Proceedings, Tenth European Conference on Earthquake Engineering, Vienna, pp. 3517-3522.

Mainstone & Weeks (1970), “The Influence of a Bounding Frame on the Racking Stiffness and Strengths of Brick Walls”, Proceedings, 2nd International Brick Masonry Conference, Stoke-on-Trent, England.

Mander J. B., Priestley M. J. N and Park R. (1984), “Seismic Design of Bridge Piers”, Report No. 84-2, Department of Civil Engineering, University of Canterbury, Christchurch.

Mander J. B., Priestley M. J. N. and Park R. (1988), “Theoretical Stress-strain Model for confined concrete”, ASCE, Journal of Structural Engineering, 114(8), pp.1804-1826.

Mattock A. H. (1967), “Discussion of ‘Rotational Capacity of Reinforcing Concrete Beams’ by Corley W. G.”, ASCE, Journal of Structural Division, 93(2), April, pp. 519-522.

McAllister D. F. and Roulier (1981a), “An Algorithm for Computing a Shape-preserving Osculatory Quadratic Spline”, ACM Trans. of Math. Software, 7, pp. 331-347.

McAllister D. F. and Roulier (1981b), "Shape-preserving Osculatory Quadratic Splines", ACM Trans. of Math. Software, 7, pp. 384-386.

Meli R. (1973), "Behavior of Masonry Walls under Lateral Loads", Proceedings, Fifth World Conference on Earthquake Engineering, Rome, pp. 853-862.

Meli R. (1975), "Seismic Behavior of Masonry Walls", Report No. 352, Institute de Ingenieria, Universidad Nacional Autonoma de Mexico.

Miranda E. (1999), "Approximate Seismic Lateral Deformation Demands in Multistory Buildings", ASCE, Journal of Structural Engineering, 125, pp. 417-425.

Miranda E. (2000), "Inelastic Displacement Ratios for Structures on Firm Sites", ASCE, Journal of Structural Engineering, 126, pp. 1150-1159.

Miranda E., and Ruiz-Garcia Jorge (2002), "Evaluation of Approximate Methods to Estimate Maximum Inelastic Displacement Demands", Earthquake Engineering and Structural Dynamics, 31, pp. 539-560.

Mirza S. A., Hatzinkolas M., and Mac Gregor J. G. (1979), "Statistical Description of the Strength of Concrete", ASCE, Journal of Structural Division, 105(6), June.

Mirza S. A., and Mac Gregor J. G. (1979a), "Variability of the Mechanical Properties of Reinforcing Bars", ASCE, Journal of Structural Division, 105(5), May.

Mirza S. A., and Mac Gregor J. G. (1979b), "Variation in Dimensions of Reinforced Concrete", ASCE, Journal of Structural Division, 105(4), 4, April.

Mizuhata K. and Maeda Y. (1989), "Study on Evaluation of Cumulative Damage Earthquake Response of Structures Based on Computer-Actuator on-line Test", Proceedings of the Ninth World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan, Aug. IV, 83-88.

Moehle J. P. and Cavanagh T. (1985), "Confinement Effectiveness of Crossties in RC", ASCE, Journal of Structural Engineering, 111(10), pp. 2105-2120.

Moehle J. P. (1992), “Displacement-based Design of RC Structures Subjected to Earthquakes”, Earthquake Spectra, 8(3), pp. 403-428.

Munro I. R. M., Priestley M. J. N. and Park R. (1984), “Seismic Behavior of Reinforced Concrete Bridge Piers”, Report No. 76-7, Department of Civil Engineering, University of Canterbury, Christchurch.

Muto K. (1974), “A Seismic Design Analysis of Buildings”, Maruzen, Tokyo.

Naiem F. and Lew M. (1995), “On the Use of Design Spectrum Compatible Time Histories”, Earthquake Spectra, 11(1), pp. 111-127.

NEHRP (1994), “Recommended Provisions for the Development of Seismic Regulations for New Buildings”, 1994 Edition, The Building Seismic Safety Council, Washington, D.C.

NEHRP (1997a), “NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA273) and Commentary (FEMA274)”, The Building Seismic Safety Council Washington, D.C.

NEHRP (1997b), “Recommended Provisions for Seismic Regulations for New Buildings and Other Structures”, 1997 Edition, The Building Seismic Safety Council, Washington, D.C.

Newmark, N. M. and Hall W. J. (1982), “Earthquake Spectra and Design”, Earthquake Engineering Research Institute, Berkeley, California.

Ng K. H., Priestley M. J. N. and Park R. (1978), “Seismic Behavior of Circular Reinforced Concrete Bridge Piers”, Report No. 78-14, Department of Civil Engineering, University of Canterbury, Christchurch.

Nilsson & Losberg (1970), “The Strength of Horizontally Loaded Prefabricated Brick Panel Walls”, Proceedings, 2nd International Brick Masonry Conference, Stoke-on-Trent, England.

O'Connor J. M. and Ellingwood B. (1987), "Reliability of Nonlinear Structures with Seismic Loading", ASCE, Journal of Structural Engineering, 113(5), pp. 1011-1028.

Oguzhan B. (1999), "Seismic Performance of Rectilinearly Confined High Strength Concrete Columns", Ph.D. Thesis, University of Toronto.

Otani S. (1997), "Development of Performance-based Design Methodology in Japan" in Fajfar P. and Krawinkler H. (Eds) "Seismic Design Methodologies for the Next Generation of Codes", Proceedings of International Conference at Bled, Slovenia, A. A. Balkema, Rotterdam/Brookfield, pp. 59-68.

Page A. W. and Huizer A. (1988), "Racking Test on Reinforced and Prestressed Hollow Clay Masonry Walls", Proceedings, 8th International Brick and Block Masonry Conference, Dublin, Ireland.

Park R. and Paulay T. (1975), "Reinforced Concrete Structures", J. Wiley & Sons, New York.

Park Y. J., Ang A. H. S. and Wen Y. K. (1984), "Seismic Damage analysis and Damage-Limiting Design of RC Buildings", Struct. Res. Ser. Rep. No. UILU-ENG-84-2007, Univ. of Illinois at Urbana-Champaign, Urbana, I11.

Park Y. J. and Ang A. H. S. (1985a), "Mechanics Seismic Damage Model for Reinforced Concrete", ASCE, Journal of Structural Engineering, 111(4), pp. 722-739.

Park Y. J. and Ang A. H. S. (1985b), "Seismic Damage Analysis of Reinforced Concrete Buildings", ASCE, Journal of Structural Engineering, 111(4), pp. 740-757.

Park Y. J., Reinhorn A. M. and Kunnath S. K. (1987), "IDARC: Inelastic Dynamic Analysis of Reinforced Concrete Frame-Shear-Wall Structures", Tech. Rep. NCEER-87-0008, Nat. Ctr. For Earthquake Engrg. Res., State Univ. of New York, Buffalo, N. Y.

Pauly T. and Priestley M. J. N. (1992), “Seismic Design of Reinforced Concrete and Masonry Buildings”, J. Wiley & Sons, New York.

Poland C. D. and Hom D. B. (1997), “Opportunities and Pitfalls of Performance-based Seismic Engineering” in Fajar P. and Krawinkler H. (Eds.), “Seismic Design Methodologies for the Next Generation of Codes”, Proceedings of International Conference at Bled, Slovenia, A. A. Balkema, Rotterdam/Brookfield, pp.69-78.

Potangaroa R. T., Priestley M. J. N. and Park R. (1981), “Ductility of Spirally-Confined Reinforced Concrete Columns”, Proceedings, ASCE, 107(1), Jan., pp.181-202.

Priestley M. J. N and Park R. (1987), “Strength and Ductility of Concrete Bridge Columns Under Seismic Loading”, ACI Structural Journal, January-February, pp.61-76.

Priestley M. J. N (1996), “Seismic Design and Retrofit of Bridges”, J. Wiley & Sons, New York.

Priestley M. J. N. (2000), “Performance Based Seismic Design”, Proceedings of twelfth World Conference of Earthquake Engineering, Auckland, New-Zealand.

Reinhorn A. M., Barron R., Valles R. E. and Sivaselvan M. V. (1998), “USER MANUAL: NSPECTRA Ver. 2.0”, University at Buffalo, State University of New York.

Rosenblueth E. and Herrera I. (1964), “On a kind of Hysteretic Damping”, ASCE, Journal of Engineering Mechanics Division, 90, pp. 37-48.

Saatcioglu M. and Razvi S.R. (1992), “Strength and Ductility of Confined Concrete”, ASCE, Journal of Structural Engineering, 118(6), pp. 1590-1607.

Sakai Koji and Sheikh Shamim A. (1989), “What Do We Know Confinement In Reinforced Concrete Columns?”, ACI Structural Journal, March-April, pp. 192-207.

Saito T., Abe S. and Shibata A. (1996), “Reliability Based Seismic Design Criteria of Reinforced Concrete Buildings in Japan”, Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, Paper No. 758.

Sarma S. K. & Yang K. S. (1987), “An Evaluation of Strong Motion Records and a New Parameter A_{95} ”, Earthquake Engineering and Structural Dynamics, 15, pp. 119-132.

Sawyer H. A. (1964), “Design of Concrete Frames for Two Failure States”, Proceedings of the International Symposium on the Flexural Mechanics of Reinforced Concrete, ASCE-ACI, Miami, November 1964, pp. 439-456.

Scott B. D., Park R. and Priestley M. J. N. (1982), “Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High strain rates”, ACI Journal, Vol. 79, January-February, pp. 13-27.

SEAOC (1989), “Recommended Lateral Force Requirements and Commentary”, San Francisco, California.

SEAOC (1995), “VISION 2000, Performance Based Seismic Engineering of Buildings”, Sacramento, California, April.

Sehayek S. (1976), “Effect of Ductility on Response Spectra for Elasto-Plastic Systems”, Publication R76-42, Dept. of Civ. Engrg., MIT, Cambridge, Massachusetts.

Sheikh Shamim A., Shah Dharmendra V., Khoury Shafik S. (1994), “Confinement of High-Strength Concrete Columns”, ACI Structural Journal, 91 (1), January-February, pp. 100-111.

Sheikh S. A and Uzumeri S. M (1980), “Strength and Ductility of Tied Concrete Columns”, ASCE, Journal of the Structural Division, 106 (5), pp. 1079-1102.

Sheikh S.A and Uzumeri S.M. (1982), “Analytical Model for Concrete Confinement in Tied Concrete Columns”, ASCE, Journal of the Structural Division, 108(12), pp. 2703-2722.

Singhal A and Kiremidjian A. S. (1996), “Method for Probabilistic Evaluation of Seismic Structural Damage”, ASCE, Journal of Structural Engineering, 122(12), pp. 1459-1467.

Sozen M. A. (1989), “A Frame of Reference”, The Art and Science of Geotechnical Engineering at the Dawn of the Twenty-First Century. A Volume honoring Ralph B. Peck. Editors: E. J. Cording, W. J. Hall, J.D. Halmi, A. J. Hendron Jr., G. Mesri. Prentice Hall, pp. 240-247.

Stone W. C. and Taylor A. W. (1994), “ISDP: Integrated Approach to Seismic Design of Reinforced Concrete Structures”, ASCE, Journal of Structural Engineering, 120(12), pp. 3548-3566.

Stylianidis K. C. (1988), “Cyclic Behavior of Infilled RC Frames”, Proceedings, Eighth International Brick and Block Masonry Conference, Dublin, Ireland.

Taghavi S. and Miranda E. (2003), “Response Assessment of Nonstructural Building Elements”, PEER report 2003/05, University of California Berkeley.

Takeda T., Sozen M. A. and Nielsen N. N. (1970), “Reinforced Concrete Response to Simulated Earthquakes”, ASCE, Journal of the Structural Division, 96(12), pp. 2557-2573.

Tassios T. P. (1983), “Response of RC Critical Regions Under Large Amplitude Reversed Actions”, Bulletin D’Information, N°161, COMETI EURO-INTERNATIONAL DU BETON.

Tassios T. P. (1989), “Specific Rules for Concrete Structures-Justification Note No.13: Confined Concrete Constitutive Law”, Background Document for Eurocode 8-Part 1, Vol.2-Design Rules, CEC DG III/8076/89 EN, pp. 104-116.

Tolis S. V. and Faccioli E. (1999), “Displacement Design Spectra”, Journal of Earthquake Engineering, 3(1), pp. 107-125.

Tomazevic M. & Zarnic R. (1984), “Shaking-table Study of A Four-storeyed Masonry Building Model”, Proceedings, Eighth World Conference on Earthquake Engineering, San Francisco, pp. 847-854.

Trezos C. G. (1997), “Reliability Considerations on the Confinement of RC Columns for Ductility”, Soil Dynamics and Earthquake Engineering, 16, pp.1-8.

Tzavelis C. and Shinozuka M. (1988), “Seismic Reliability of Rigid Frame Structures”, ASCE, Journal of Engineering Mechanics, 114(11), pp. 1953-1977.

Vallena J, Bertero V. V. and Popov E. P. (1977), “Concrete Confined by Rectangular Hoops and Subjected to Axial Loads”, Report No. UCB/EERC 72/13, University of California at Berkeley, August.

Valles, R. E., Reinhorn, A. M., Kunnath, S. K., and Madan, A. (1996), “IDARC 2D Version 4.0: a program for the inelastic damage analysis of buildings.” Technical Report NCEER-96-0010, National Center for Earthquake Engineering Research, Buffalo, N. Y.

Veletsos, A. S. and Newmark, N. M. (1960), “Effect of inelastic Behavior on the Response of Simple Systems to Earthquake Motions”, Proceedings of the World Conference on Earthquake Engineering, Japan, Vol.2. pp. 895-912.

Veletsos, A. S., Newmark, N. M. and Chepalati, C. V. (1965), “Deformation Spectra for Elastic and Elastoplastic Systems Subjected to Ground Shock and Earthquake Motion”, Proceedings of

the 3rd World Conference on Earthquake Engineering, New Zealand, Vol. II, pp. 663-682.

Vismann U. and Zilch K. (1995), "Non-Linear Analysis and Safety Evaluation by Finite-Element-Reliability-Method", CEB Bulletin d'Information 229, pp. 49-73.

Wen Y. K. (1995), "Building Reliability and Code Calibration", Earthquake Spectra, 11(2), pp. 269-296.

Whittaker, A., Constantinou, M., and Tsoelas, P. (1998), "Displacement estimates for performance-based seismic design" ASCE, Journal of Structural Engineering, 124(8), pp. 905-912.

Williams D. and Scrivener J. C. (1973), "Response of Reinforced Masonry Shear Walls to static and Dynamic Cyclic Loading", Proceedings, Fifth World Conference on Earthquake Engineering, Rome, pp. 1491-1494.

Wu C. Q., Hao H. and Zhou Y. X. (1999), "Fuzzy-random Probabilistic Analysis of Rock Mass Responses to Explosive Loads", Computers and Geotechnics, 25, pp. 205-225.

Yao J. T. P. (1980), "Damage Assessment of Existing Structures", ASCE, Journal of the Engineering Mechanics Division, 106 (4), pp. 785-799

Yao J. T. P. (1982), "Probabilistic Methods for the Evaluation of Seismic Damage of Existing Structures", Soil Dynamics and Earthquake Engineering, 1(3), pp. 91-96.

Zadeh, L. A. (1965), "Fuzzy Sets", Information Control, 8, pp. 338-353.

Zadeh L. A. (1968), "Probability Measures of Fuzzy Events", Journal of Mathematical Analysis and Application, 23, 421.

Zarnic R. & Tomazevic M. (1984), "The Behavior of Masonry Infilled Reinforced Concrete Frames Subjected to Cyclic Lateral Loading", Proceedings, Eighth World Conference on Earthquake Engineering, San Francisco, pp. 863-870.

Zhang G. and Lu Y. (1993), “CABAC v3.00: Program for Cyclic Analysis of Beam and Column: Users’ manual”, RC Frame Project No.6, National Technological University of Athens, Greece.

Zimmermann H. J. (1996), “Fuzzy Set Theory and Its Applications”, 3rd edition. Kluwer Academic Publishers.

Appendix A. Experimental lateral deformation capacity of masonry walls

Table A.1 Details of the test programs

Reference	Material	Designation	Panel Dimensions H*W (m)	Aspect ratio	Frame	Reinforce ment	Vertical loading	Loading pattern	Drift limits (%)		
									Functional	Damage Control	group
Mainstone & Weeks, 1970	Clay brick	1	2.77*3.36	1.21	Light steel frame, concrete encased	No	No	Monotonic, static	1.34	-	2-a
		2	2.77*3.36	1.21	Light steel frame, concrete encased	No	No	Monotonic, static	1.59	-	2-a
		3	2.77*3.36	1.21	Light steel frame, concrete encased	No	No	Monotonic, static	1.23	3.94	2-a
		4	2.77*3.36	1.21	Light steel frame, concrete encased	No	No	Monotonic, static	0.54	2.74	2-a
		5	2.72*3.31	1.22	Heavy steel frame, concrete encased	No	No	Monotonic, static	1.29	4.85	2-a
		6	2.74*3.36	1.23	Stiffened steel frame, concrete encased	No	No	Monotonic, static	1.82	2.92	2-a
Nilsson & Losberg, 1970	Clay brick	1	2.80*1.96	0.70	No	No	No	Monotonic, static	0.06	-	1
		2	2.80*1.96	0.70	No	Yes	No	Monotonic, static	1.00	-	5

Chapter 6

		4	2.80*1.96	0.70	No	Yes	No	Monotonic, static	1.07	-	-	5
		6	2.80*1.96	0.70	No	Yes	No	Monotonic, static	0.536	-	-	5
Williams & Scrivener, 1973	Clay brick	B1	1.19*0.66	0.55	No	Yes	Yes, $\sigma_m=1.72\text{MPa}$	Dynamic	1.06	1.94	-	9
		B1*	1.19*0.66	0.55	No	Yes	Yes, $\sigma_m=1.72\text{MPa}$	Cyclic, static	1.10	-	-	9
		B2	1.19*0.66	0.55	No	Yes	No	Dynamic	0.19	2.05	-	8
		B3	1.19*0.66	0.55	No	Yes	Yes, $\sigma_m=0.86\text{MPa}$	Cyclic, static	0.75	-	-	9
		B4	1.19*0.66	0.55	No	Yes	Yes, $\sigma_m=1.72\text{MPa}$	Dynamic	1.17	2.03	-	9
Meli, 1972	Concrete block	-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.18	-	-	13
		-	2.0*2.0	1.0	No	Yes	Yes, $\sigma_m=0.34\text{MPa}$	Monotonic, static	0.26	-	-	13
		-	2.0*2.0	1.0	No	Yes	Yes, $\sigma_m=0.98\text{MPa}$	Monotonic, static	0.47	-	-	14
		-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.39	-	-	13
		-	2.0*2.0	1.0	No	Yes	Yes, $\sigma_m=0.34\text{MPa}$	Monotonic, static	0.42	-	-	13
		-	2.0*2.0	1.0	No	Yes	Yes, $\sigma_m=0.98\text{MPa}$	Monotonic, static	0.47	-	-	14
		-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.35	-	-	13
		-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.56	-	-	13
		-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.21	-	-	5
		-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.21	-	-	5

brick	-	2.0*2.0	1.0	No	Yes	Yes, $\sigma_m=0.59\text{MPa}$	Monotonic, static	0.19	-	-	6
	-	2.0*2.0	1.0	No	Yes	Yes, $\sigma_m=1.23\text{MPa}$	Monotonic, static	0.34	-	-	6
	-	2.0*2.0	1.0	No	Yes	No	Monotonic, static	0.33	-	-	5
	-	2.0*2.0	1.0	Tie columns and bond beams	No	No	Monotonic, static	0.38	-	-	2-b
Solid brick	-	2.0*2.0	1.0	Tie columns and bond beams	Yes	No	Monotonic, static	0.63	-	-	7
	-	2.0*2.0	1.0	Tie columns and bond beams	No	No	Monotonic, static	0.32	-	-	2-b
	-	2.0*2.0	1.0	Tie columns and bond beams	No	Yes, $\sigma_m=0.29\text{MPa}$	Monotonic, static	0.27	-	-	2-b
	-	2.0*2.0	1.0	Tie columns and bond beams	No	No	Monotonic, static	0.54	-	-	2-b
Perforate d brick	-	2.0*2.0	1.0	Tie columns and bond beams	No	Yes, $\sigma_m=0.29\text{MPa}$	Monotonic, static	0.32	-	-	2-b
	-	2.0*2.0	1.0	Tie columns and bond beams	No	No	Monotonic, static	0.28	-	-	2-b

		-	2.0*2.0	1.0	Tie columns and bond beams	No	Yes, $\sigma_m=0.29\text{MPa}$	Monotonic, static	0.18	-	-	2-b
	Sand lime brick	-	2.0*2.0	1.0	Tie columns and bond beams	No	No	Monotonic, static	0.36	-	-	2-b
		-	2.0*2.0	1.0	Tie columns and bond beams	No	Yes, $\sigma_m=0.29\text{MPa}$	Monotonic, static	0.25	-	-	2-b
Benjamin & Williams, 1958	Clay brick	BW-1	0.84*1.26	1.5	Concrete frame	No	No	Monotonic, static	0.20	-	-	2-b
		BW-2	0.84*1.26	1.5	Concrete frame	No	No	Monotonic, static	0.136	-	-	2-b
		BW-3	0.84*1.26	1.5	Concrete frame	No	No	Monotonic, static	0.118	-	-	2-b
		BW-4	0.84*1.26	1.5	Concrete frame	No	No	Monotonic, static	0.113	-	-	2-b
	Concrete frame	BW-5	0.84*1.26	1.5	Concrete frame	No	No	Monotonic, static	0.164	-	-	2-b
		BW-6	1.83*2.74	1.5	Concrete frame	No	No	Monotonic, static	0.138	-	-	2-b
		BW-7	1.83*2.74	1.5	Concrete frame	No	No	Monotonic, static	0.213	-	-	2-b
		BW-8	2.44*3.66	1.5	Concrete frame	No	No	Monotonic, static	0.239	-	-	2-b
		3b2-2	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.028	0.140	-	2-b
		3b2-2a	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.090	0.447	-	2-b
		3b2-3	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.033	0.226	-	2-b

Kato H. et. al, 1992	Clay brick	3b2-3a	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.100	0.149	-	2-b
		3a3-1	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.070	0.157	-	2-b
		3a3-2	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.096	0.195	-	2-b
		3a3-3	0.91*1.6	1.76	Concrete frame	No	No	Monotonic, static	0.126	-	-	2-b
		4a-1	0.56*0.81	1.45	Concrete frame	No	No	Monotonic, static	0.127	-	-	2-b
		4a-2	0.56*1.12	2.0	Concrete frame	No	No	Monotonic, static	0.114	-	-	2-b
		4a-3	0.56*1.68	3.0	Concrete frame	No	No	Monotonic, static	0.029	-	-	2-b
		4a-4	0.56*1.12	2.0	Concrete frame	No	No	Monotonic, static	0.136	-	-	2-b
		4a-5	0.56*0.51	0.91	Concrete frame	No	No	Monotonic, static	0.174	-	-	2-b
		SF-1	2.40*3.57	1.49	Steel frame	No	No	Monotonic, static	3.44	-	-	2-a
		SF-3	2.40*3.57	1.49	Steel frame	No	No	Monotonic, static	2.65	-	-	2-a
		A	1.33*0.87	0.65	RC frame	No	Yes	Cyclic, static	0.25	1.48	-	4
		B	1.33*0.87	0.65	RC frame	No	Yes	Cyclic, static	0.23	0.54	-	4
		C	1.33*0.87	0.65	RC frame	No	Yes	Cyclic, static	0.35	0.89	-	4
		D	1.33*0.87	0.65	RC frame	No	Yes	Cyclic, static	0.16	0.41	-	4

Magenes G. & Calvi G. M., 1992	Clay brick	MI1	2.0*1.5	0.75	No	No	Yes, $\sigma_m=1.2\text{MPa}$	Cyclic, static	0.19	0.56	-	3
		MI1m	2.0*1.5	0.75	No	No	Yes, $\sigma_m=1.2\text{MPa}$	Cyclic, static	0.25	-	-	3
		MI2	2.0*1.5	0.75	No	No	Yes, $\sigma_m=0.4\text{MPa}$	Cyclic, static	0.40	-	-	3
		MI3	3.0*1.5	0.50	No	No	Yes, $\sigma_m=1.2\text{MPa}$	Cyclic, static	0.22	0.53	-	3
		MI4	3.0*1.5	0.50	No	No	Yes, $\sigma_m=0.4\text{MPa}$	Cyclic, static	0.39	0.55	-	3
Magenes G. & Calvi G. M., 1994	Clay brick	5	1.35*1.0	0.74	No	No	Yes, $\sigma_m=0.63\text{MPa}$	Shaking table	0.58	-	-	3
		8	2.0*1.0	0.5	No	No	Yes, $\sigma_m=1.11\text{MPa}$	Shaking table	1.01	1.35	-	3
Anthoine A and Magonette 1994	Clay brick	Low wall	1.35*1.0	0.74	No	No	Yes, $\sigma_m=0.6\text{MPa}$	Quasi-static , cyclic	0.22	0.53	-	3
		High wall	2.0*1.0	0.5	No	No	Yes, $\sigma_m=0.8\text{MPa}$	Quasi-static , cyclic	0.629	-	-	3
		Low wall	1.35*1.0	0.74	No	No	Yes, $\sigma_m=1.08\text{MPa}$	Pseudo-dyn amic	0.148	-	-	3
Tomazevi C M. & Zarnic R., 1984	Clay brick	S2	0.173*0.142	0.82	No	Yes	Yes, $\sigma_m=0.19\text{MPa}$	Cyclic, static	0.20	1.15	-	9
		S3	0.173*0.142	0.82	No	Yes	Yes, $\sigma_m=0.19\text{MPa}$	Cyclic, static	0.373	1.45	-	9
		S4	0.173*0.142	0.82	No	Yes	Yes, $\sigma_m=0.19\text{MPa}$	Cyclic, static	0.81	1.40	-	9
		S5	0.173*0.142	0.82	No	Yes	Yes, $\sigma_m=0.19\text{MPa}$	Cyclic, static	0.289	-	-	9

		F2	0.318*0.142	0.45	No	Yes	Yes, $\sigma_m=0.24\text{MPa}$	Cyclic, static	0.85	-	-	9
		F3	0.318*0.142	0.45	No	Yes	Yes, $\sigma_m=0.24\text{MPa}$	Cyclic, static	0.98	-	-	9
		F4	0.318*0.142	0.45	No	Yes	Yes, $\sigma_m=0.24\text{MPa}$	Cyclic, static	0.833	-	-	9
Zarnic R. & Tomaze vic M., 1984	Clay brick	M2	1.50*2.08	1.39	RC frame	No	No	Cyclic, static	1.00	-	-	4
		M3	1.50*2.08	1.39	RC frame	Yes	No	Cyclic, static	1.00	-	-	10
		M4	1.50*2.08	1.39	RC frame	Yes	No	Cyclic, static	2.00	-	-	10
Hidalgo P. A. & Luders C., 1984	Clay brick	TC-1-01	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.64	-	-	8
		TC-1-02	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.44	-	-	8
		TC-1-03	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.74	-	-	8
		TC-1-04	1.02*1.04	1.02	No	Yes	No	Cyclic, static	1.20	-	-	8
		TC-1-05	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.69	-	-	8
		TC-1-06	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.59	-	-	8
		TC-1-07	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.55	-	-	8
		TC-1-08	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.34	-	-	8

Page A. W. and Huizer A., 1988	Hollow clay brick	TC-1-09	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.98	-	-	8
		TC-1-10	1.02*1.04	1.02	No	Yes	No	Cyclic, static	1.67	-	-	8
		TC-1-11	1.02*1.04	1.02	No	Yes	No	Cyclic, static	0.62	-	-	8
		TC-1-12	1.02*1.04	1.02	No	Yes	No	Cyclic, static	1.67	-	-	8
	Hollow clay brick	B	3.0*2.5	0.83	No	Yes	Yes, $\sigma_m=2.0\text{MPa}$	Monotonic, static	0.589	-	-	6
		C	3.0*2.5	0.83	No	Yes	No	Monotonic, static	0.698	-	-	5
	Clay brick	F1	1.0*1.5	1.5	RC frame	Yes	No	Cyclic, static	0.60	2.40	-	10
	Clay brick	1	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.17	-	1.05	2-b
		2	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.16	-	1.05	2-b
		3	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	1.03	-	1.06	2-b
		4	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.04	-	1.17	2-b
		5	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.52	-	0.53	2-b
		6	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.13	-	1.05	2-b
		7	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.17	-	1.05	2-b
		9	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.11	-	1.11	2-b

		10	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.09	-	1.08	2-b
		11	2.0*2.0	1.0	RC frame	Yes	No	Monotonic, static	0.07	-	1.10	2-b
		15	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.13	-	1.10	2-b
		18	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.03	-	1.21	2-b
		20	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.52	-	0.83	2-b
		22	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.12	-	0.71	2-b
		23	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.10	-	0.76	2-b
		24	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.20	-	0.68	2-b
		25	2.0*2.0	1.0	RC frame	No	No	Monotonic, static	0.20	-	0.60	2-b
	Hollow clay brick	HCBR-11-1	1.42*1.21	0.85	No	No	Yes $\sigma_m=2.3\text{MPa}$	Cyclic, static	0.44	-	0.84	3
	Solid clay brick	HCBR-11-2	1.42*1.21	0.85	No	No	Yes $\sigma_m=2.3\text{MPa}$	Cyclic, static	0.28	-	0.39	3
		CBCR-11-1	1.42*1.21	0.85	No	No	Yes $\sigma_m=2.0\text{MPa}$	Cyclic, static	0.52	-	0.96	3
	Hollow clay brick	-	3.0*3.0	1.0	No	No	Yes $\sigma_m=0.4\text{MPa}$	Cyclic, static	-	-	0.20	3
		-	3.0*3.0	1.0	No	No	Yes $\sigma_m=0.4\text{MPa}$	Cyclic, static	-	-	0.20	3
	Solid clay	-	3.0*3.0	1.0	Tie columns	No	Yes $\sigma_m=0.4\text{MPa}$	Cyclic, static	0.1	-	0.3	4

Chen et al.
1978Meli 1973
(cited
from
Algan B.
B., 1982)

	brick	-	3.0*3.0	1.0	Tie columns	No	Yes $\sigma_m=0.4\text{MPa}$	Cyclic, static	0.25	-	-	4
		-	3.0*3.0	1.0	RC frame	No	Yes $\sigma_m=0.4\text{MPa}$	Cyclic, static	0.25	-	0.15	4
Priestley 1977 (cited from Algan B. B., 1982)	Concrete block	A1	1.65*2.43	1.47	No	Yes	No	Cyclic, static	0.85	-	-	15
		A2	1.65*2.43	1.47	No	Yes	No	Cyclic, static	0.73	-	-	15
		A3	1.65*2.43	1.47	No	Yes	No	Cyclic, static	1.28	-	-	15
		A4	1.65*2.43	1.47	No	Yes	No	Cyclic, static	0.84	-	-	15
		A5	1.65*2.43	1.47	No	Yes	Yes $\sigma_m=0.7\text{MPa}$	Cyclic, static	1.36	-	-	15
		A6	1.65*2.43	1.47	No	Yes	Yes $\sigma_m=0.7\text{MPa}$	Cyclic, static	1.34	-	-	15
Jolley 1979	Concrete block	M-3-C	2.44*2.44	1.0	No	No	No	Monotonic, static	1.28	-	-	11
		M-4-C	2.44*2.44	1.0	No	No	No	Monotonic, static	0.69	-	-	11
		M-1-C	2.44*2.44	1.0	No	No	Yes $\sigma_m=0.8\text{MPa}$	Monotonic, static	0.14	-	-	12
		M-2-C	2.44*2.44	1.0	No	No	Yes $\sigma_m=1.6\text{MPa}$	Monotonic, static	0.41	-	-	12
		M-5-C	2.44*2.44	1.0	No	No	Yes $\sigma_m=0.8\text{MPa}$	Monotonic, static	0.74	-	-	12
		M-6-C	2.44*2.44	1.0	No	No	Yes $\sigma_m=1.6\text{MPa}$	Monotonic, static	1.80	-	-	12
		M-7-CR	2.44*2.44	1.0	No	No	Yes $\sigma_m=0.8\text{MPa}$	Monotonic, static	0.38	-	-	12

		M-8-CR	2.44*2.44	1.0	No	No	Yes $\sigma_m=1.8\text{MPa}$	Monotonic, static	0.36	-	-	12
		8M-1-C	1.22*2.44	2.0	No	No	Yes $\sigma_m=0.7\text{MPa}$	Monotonic, static	0.15	-	-	11
		8M-2-C	1.22*2.44	2.0	No	No	Yes $\sigma_m=1.5\text{MPa}$	Monotonic, static	1.13	-	-	12
		8M-3-C	1.22*2.44	2.0	No	No	Yes $\sigma_m=0.4\text{MPa}$	Monotonic, static	0.31	-	-	11
		8M-4-C	1.22*2.44	2.0	No	No	Yes $\sigma_m=0.7\text{MPa}$	Monotonic, static	0.23	-	-	11
		8M-5-C	1.22*2.44	2.0	No	No	Yes $\sigma_m=0.4\text{MPa}$	Monotonic, static	0.78	-	-	11
		R-1-C	2.44*2.44	1.0	No	No	No	Monotonic, static	0.18	-	-	1
		R-2-C	2.44*2.44	1.0	No	No	No	Monotonic, static	0.33	-	-	1
		R-5-H	2.44*2.44	1.0	No	No	No	Monotonic, static	0.32	-	-	1
		R-6-H	2.44*2.44	1.0	No	No	No	Monotonic, static	0.36	-	-	1
	Clay brick	D-2-C	2.44*2.44	1.0	No	No	No	Monotonic, static	0.16	-	-	1
		D-2-H	2.44*2.44	1.0	No	No	No	Monotonic, static	0.39	-	-	1

Damage Control level for masonry wall panel without frame

From some separated test data of bare frames and infilled frames (Mainstone & Weeks 1970, Zarnic & Tomazevic 1984), it is found that the shape of the descending portion of load-displacement curve of an infilled frame is determined by the masonry infill wall because after yield, the load-displacement curve of the bare frame is basically a horizontal line (see Figure A.1). In other words, the difference in terms of lateral load between load-displacement curves of a bare and an infilled frame is more or less a constant on the descending portion. It is also noted that the peak strength of bare frames normally takes about 25%~35% of the peak load of the infilled frames. Therefore, for a rough estimation, the reduction percentile of the peak load on the load-displacement curve of masonry wall at a displacement level equivalent to 70% of peak load in an infilled frame can be determined as follows.

Figure A.1 Load-deflection curves for infill masonry walls and bare & infilled frames

It is assumed that in an infilled frame, the peak lateral load of the infill masonry wall panel, the bare frame and the entire infilled frame is A , B and C , respectively, which satisfy

$$C = A + B \quad (\text{A.1})$$

$$B = 0.3C \quad (\text{A.2})$$

The corresponding Damage Control displacement is assumed to be D .

Thus, the lateral load of the infilled frame at Damage Control level is

$$0.7C = 0.7A + 0.7B \quad (\text{A.3})$$

and according to the relationship between the descending portions of load-displacement curves of bare and infilled frame, the lateral load of infill masonry, X , corresponding to the same displacement level of D , can be expressed as

$$X = 0.7C - B \quad (\text{A.4})$$

From Equation 1, 2, 3 and 4, it is obtained that

$$X = 0.571A \quad (\text{A.5})$$

which means a reduction factor of 57.1% applied to the peak load of an masonry wall corresponds approximately to the similar state as that of the infilled panel in an infilled frame at 70% of the peak load of the entire infilled frame.

It is believed that the assumptions made in the above derivation introduce negligible uncertainties as compared to the uncertainty involved in the various nature of specimens. Therefore, for masonry walls without frames, it is assumed that the reduction factor on peak load of infill masonry for Damage Control displacement equals to 60%, which is deemed consistent with the reduction factor of 70% applied for infilled frames.

Appendix B. Section detail of representative frames

Table B.1 Section details for frame 7-I-a

Storey		1	2	3	4	5	6	7
Column	Section (mm)	600×600	600×600	500×500	500×500	400×400	400×400	400×400
	Rebar	12D22.3	12D22.3/ 12D19.1	12D22.3/ 12D19.1	12D22.3/ 12D15.9	12D15.9	12D15.9	8D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	1.30/0.95	1.87/1.37	1.87/0.95	1.49	1.49	0.99
	Hoops	D9.5@150	D9.5@150	D9.5@200	D9.5@200	D9.5@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.50	0.50	0.38	0.38	0.53	0.53	0.50
	$\alpha\omega_w$	0.032	0.032	0.020	0.020	0.026	0.026	0.021
	Hoop pattern
Beam	Storey	Section (mm)	Rebar			Rebar grade	Hoops	Hoop bar grade
	1	500×300				Grade 60	4D9.5@150	Grade 40
	2	500×300				Grade 60	4D9.5@150	Grade 40
	3	500×300				Grade 60	4D9.5@150	Grade 40
	4	500×300				Grade 60	4D9.5@150	Grade 40
	5	400×300				Grade 60	4D9.5@150	Grade 40
	6	400×300				Grade 60	4D9.5@150	Grade 40
	7	400×300				Grade 60	2D9.5@150	Grade 40

Note: Beam longitudinal reinforcement: a-4D22.3, b-4D19.1, c-3D19.1, d-3D15.9

Table B.2 Section details for frame 7-I-b

Storey		1	2	3	4	5	6	7
Column	Section (mm)	600×600	600×600	500×500	500×500	400×400	400×400	400×400
	Rebar	12D22.3	12D22.3/ 12D19.1	12D22.3/ 12D19.1	12D22.3/ 12D15.9	8D15.9	8D15.9	8D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	1.30/0.95	1.87/1.37	1.87/0.95	0.99	0.99	0.99
	Hoops	D9.5@150	D9.5@150	D9.5@200	D9.5@200	D9.5@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.50	0.50	0.38	0.38	0.50	0.50	0.50
	$\alpha\omega_v$	0.032	0.032	0.020	0.020	0.021	0.021	0.021
	Hoop pattern
Beam	Storey	Section (mm)	Rebar			Rebar grade	Hoops	Hoop bar grade
	1	500×300				Grade 60	4D9.5@150	Grade 40
	2	500×300				Grade 60	4D9.5@150	Grade 40
	3	500×300				Grade 60	4D9.5@150	Grade 40
	4	500×300				Grade 60	4D9.5@150	Grade 40
	5	400×300				Grade 60	4D9.5@150	Grade 40
	6	400×300				Grade 60	4D9.5@150	Grade 40
	7	400×300				Grade 60	2D9.5@150	Grade 40

Note: Beam longitudinal reinforcement: a-4D22.3, b-4D19.1, c-3D19.1, d-3D15.9

Table B.3 Section details for frame 12-R

Storey		1	2	3	4	5	6
Column	Section (mm)	800×800	700×700	700×700	700×700	600×600	600×600
	Rebar	16D22.3	16D22.3	16D22.3 / 12D22.3	12D22.3	12D22.3	12D22.3
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	0.98	1.27	1.27/0.96	0.96	1.30	1.30
	Hoops	D12.6@150	D12.6@150	D12.6@150	D12.6@150	D12.6@200	D12.6@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.68	0.76	0.76	0.76	0.68	0.68
	$\alpha\omega_w$	0.048	0.052	0.052	0.052	0.041	0.041
	Hoop pattern
Storey		7	8	9	10	11	12
Column	Section (mm)	600×600	500×500	500×500	500×500	400×400	400×400
	Rebar	12D22.3	12D22.3	12D22.3/ 12D19.1	12D19.1	12D19.1/ 12D15.9	12D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	1.87	1.87/1.37	1.37	2.15/1.45	1.45
	Hoops	D12.6@200	D12.6@200	D12.6@200	D12.6@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.68	0.79	0.79	0.79	0.53	0.53
	$\alpha\omega_w$	0.041	0.044	0.044	0.044	0.026	0.026
	Hoop pattern
Beam	Storey	Section (mm)	Rebar		Rebar grade	Hoops	Hoop bar grade
	1	650×300			Grade 60	4D9.5@150	Grade 40
	2	650×300			Grade 60	4D9.5@150	Grade 40
	3	650×300			Grade 60	4D9.5@150	Grade 40
	4	650×300			Grade 60	4D9.5@150	Grade 40

5	550×300		Grade 60	4D9.5@150	Grade 40
6	550×300		Grade 60	4D9.5@150	Grade 40
7	500×300		Grade 60	4D9.5@150	Grade 40
8	500×300		Grade 60	4D9.5@150	Grade 40
9	400×300		Grade 60	4D9.5@150	Grade 40
10	400×300		Grade 60	4D9.5@200	Grade 40
11	400×300		Grade 60	4D9.5@200	Grade 40
12	400×300		Grade 60	2D9.5@200	Grade 40

Note: Beam longitudinal reinforcement: a-4D25.5, b-2D25.5+2D22.3, c-4D22.3, d-4D19.1, e-4D15.9, f-3D15.9

Table B.4 Section details for frame 12-I-a

Storey		1	2	3	4	5	6
Column	Section (mm)	800×800	700×700	700×700	700×700	600×600	600×600
	Rebar	16D22.3	16D22.3	16D22.3 / 12D22.3	12D22.3	12D22.3	12D22.3
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	0.98	1.27	1.27/0.96	0.96	1.30	1.30
	Hoops	D12.6@150	D12.6@150	D12.6@150	D12.6@150	D12.6@200	D12.6@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.68	0.76	0.76	0.76	0.68	0.68
	$\alpha\omega_w$	0.048	0.052	0.052	0.052	0.041	0.041
	Hoop pattern
Storey		7	8	9	10	11	12
Column	Section (mm)	600×600	400×400	400×400	400×400	400×400	400×400
	Rebar	12D22.3	12D19.1	12D19.1	12D19.1 / 12D15.9	12D19.1 / 12D15.9	12D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	2.15	2.15	2.15/1.45	2.15/1.45	1.45
	Hoops	D12.6@200	D12.6@200	D12.6@200	D12.6@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.68	0.96	0.96	0.96	0.53	0.53
	$\alpha\omega_w$	0.041	0.047	0.047	0.047	0.026	0.026
	Hoop pattern
Beam	Storey	Section (mm)	Rebar		Rebar grade	Hoops	Hoop bar grade
	1	650×300			Grade 60	4D9.5@150	Grade 40
	2	650×300			Grade 60	4D9.5@150	Grade 40
	3	650×300			Grade 60	4D9.5@150	Grade 40

Chapter 6

4	650×300		Grade 60	4D9.5@150	Grade 40
5	550×300		Grade 60	4D9.5@150	Grade 40
6	550×300		Grade 60	4D9.5@150	Grade 40
7	500×300		Grade 60	4D9.5@150	Grade 40
8	500×300		Grade 60	4D9.5@150	Grade 40
9	400×300		Grade 60	4D9.5@150	Grade 40
10	400×300		Grade 60	4D9.5@200	Grade 40
11	400×300		Grade 60	4D9.5@200	Grade 40
12	400×300		Grade 60	2D9.5@200	Grade 40

Note: Beam longitudinal reinforcement: a-4D25.5, b-2D25.5+2D22.3, c-4D22.3, d-4D19.1, e-4D15.9, f-3D15.9

Table B.5 Section details for frame 12-I-b

Storey		1	2	3	4	5	6
Column	Section (mm)	700×700	700×700	700×700	700×700	600×600	600×600
	Rebar	16D22.3	16D22.3	16D22.3	16D22.3	12D22.3	12D22.3
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.27	1.27	1.27	1.27	1.30	1.30
	Hoops	D12.6@150	D12.6@150	D12.6@150	D12.6@150	D12.6@200	D12.6@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.76	0.76	0.76	0.76	0.68	0.68
	$\alpha\omega_w$	0.052	0.052	0.052	0.052	0.041	0.041
	Hoop pattern
Storey		7	8	9	10	11	12
Column	Section (mm)	600×600	600×600	500×500	500×500	500×500	500×500
	Rebar	12D22.3	12D22.3	12D19.1	12D19.1	12D15.9	8D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	1.30	1.37	1.37	0.95	0.64
	Hoops	D12.6@200	D12.6@200	D12.6@200	D12.6@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.68	0.68	0.79	0.79	0.38	0.38
	$\alpha\omega_w$	0.041	0.041	0.044	0.044	0.020	0.020
	Hoop pattern
Beam	Storey	Section (mm)	Rebar		Rebar grade	Hoops	Hoop bar grade
	1	600×300			Grade 60	4D9.5@150	Grade 40
	2	600×300			Grade 60	4D9.5@150	Grade 40
	3	600×300			Grade 60	4D9.5@150	Grade 40

Chapter 6

4	600×300		Grade 60	4D9.5@150	Grade 40
5	500×300		Grade 60	4D9.5@150	Grade 40
6	500×300		Grade 60	4D9.5@150	Grade 40
7	500×300		Grade 60	4D9.5@150	Grade 40
8	500×300		Grade 60	4D9.5@150	Grade 40
9	400×300		Grade 60	4D9.5@150	Grade 40
10	400×300		Grade 60	4D9.5@200	Grade 40
11	400×300		Grade 60	4D9.5@200	Grade 40
12	400×300		Grade 60	2D9.5@200	Grade 40

Note: Beam longitudinal reinforcement: a-4D25.5, b-2D25.5+2D22.3, c-4D22.3, d-4D19.1, e-4D15.9, f-3D15.9

Table B.6 Displacement response (7-R, A95=0.26g)

Ground motions		PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ_i (%)							θ_{max} (%)	Peak roof drift ratio, θ_r , peak (%)	θ_{max}/θ_r , peak	
			1	2	3	4	5	6	7				
135	PEL090	0.419	0.75	0.88	0.77	0.55	0.54		0.51	0.36	0.88	0.59	1.49
	PEL180	0.419	0.35	0.36	0.48	0.54	0.57		0.49	0.35	0.57	0.38	1.52
412	H-E10050	0.365	1.10	1.12	0.88	0.50	0.40		0.43	0.37	1.12	0.63	1.77
	H-E10320	0.440	2.57	2.72	2.58	2.65	1.24		0.93	0.51	2.72	1.75	1.56
931	H-E12140	0.373	0.75	0.78	0.62	0.46	0.60		0.63	0.49	0.78	0.49	1.60
	H-E12230	0.326	0.51	0.54	0.47	0.39	0.41		0.39	0.29	0.54	0.38	1.43
952	H-E05140	0.650	0.72	0.73	0.67	0.69	0.76		0.71	0.56	0.76	0.60	1.27
	H-E05230	0.474	1.45	1.57	1.42	1.16	1.05		1.08	1.01	1.57	1.11	1.41
955	H-E04140	0.676	1.03	1.15	1.25	1.27	1.24		1.11	0.60	1.27	1.03	1.23
	H-E04230	0.600	2.92	3.13	2.75	1.79	1.01		0.77	0.55	3.13	1.84	1.70
5028	H-E07140	0.587	1.46	1.63	1.32	1.20	1.19		1.06	0.63	1.63	1.08	1.50
	H-E07230	0.572	2.01	2.27	2.20	1.81	1.07		0.94	0.57	2.27	1.50	1.51
5053	H-CX0225	0.480	0.27	0.27	0.26	0.28	0.52		0.78	0.69	0.78	0.28	2.74
	H-CX0315	0.381	0.37	0.37	0.31	0.27	0.41		0.48	0.42	0.48	0.30	1.60
5054	H-BCR140	0.479	0.34	0.33	0.30	0.32	0.85		1.27	1.19	1.27	0.49	2.61
	H-BCR230	0.510	0.31	0.36	0.39	0.40	0.71		0.89	0.75	0.89	0.51	1.75
5058	H-E11140	0.419	0.41	0.42	0.36	0.29	0.37		0.44	0.41	0.44	0.29	1.50
	H-E11230	0.482	0.41	0.44	0.44	0.36	0.46		0.53	0.48	0.53	0.35	1.50
14368	A-DWN180	0.541	0.87	1.07	1.26	1.36	1.38		1.26	0.88	1.38	0.92	1.49
	A-DWN270	0.471	0.65	0.77	0.76	0.70	0.79		0.85	0.57	0.85	0.60	1.43
23525	A-PMN012	0.581	0.17	0.18	0.27	0.29	0.37		0.37	0.40	0.40	0.24	1.66
	A-PMN102	0.490	0.27	0.24	0.26	0.27	0.41		0.51	0.46	0.51	0.23	2.17
24303	HOL090	0.384	0.46	0.56	0.56	0.45	0.46		0.45	0.45	0.56	0.45	1.25
	HOL360	0.407	0.59	0.64	0.52	0.49	0.52		0.43	0.44	0.64	0.44	1.45
24303	A-HOL000	0.702	0.40	0.46	0.49	0.51	0.65		0.66	0.69	0.69	0.51	1.37
	A-HOL090	0.500	0.51	0.60	0.65	0.58	0.57		0.57	0.46	0.65	0.52	1.26
47380	G02000	0.540	0.56	0.61	0.61	0.68	0.93		0.91	0.86	0.93	0.56	1.66
	G02090	0.471	1.62	1.86	1.66	1.08	0.88		0.81	0.59	1.86	1.12	1.66
47381	G03000	0.619	0.31	0.28	0.29	0.39	0.62		0.86	0.96	0.96	0.45	2.15
	G03090	0.509	1.18	1.27	1.07	0.72	0.58		0.45	0.32	1.27	0.79	1.60
Mean		0.496	0.84	0.92	0.86	0.75	0.72		0.72	0.58	1.08	0.68	1.628
COV		0.194	81.7%	81.8%	78.7%	75.5%	41.5%		37.4%	38.1%	62.9%	63.3%	22.0%
Distribution		N/L	L	L	L	L	N/L		L	L	L	L	-
Times of max. drift		-	0	13	1	1	4		8	3	-	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.7 Displacement response (7-R, A95=0.20g)

Ground motions		PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ_i (%)							θ_{max} (%)	Peak roof drift ratio, $\theta_{r, peak}$ (%)	$\theta_{max}/\theta_{r, peak}$
			1	2	3	4	5	6	7			
135	PEL090	0.299	0.48	0.51	0.39	0.34	0.35		0.34	0.29		1.68
	PEL180	0.299	0.24	0.25	0.33	0.36	0.41		0.38	0.28	0.51	1.52
412	H-E10050	0.260	0.43	0.42	0.37	0.30	0.33		0.37	0.32	0.43	1.36
	H-E10320	0.314	1.26	1.40	1.18	0.73	0.51		0.39	0.25	1.40	1.74
931	H-E12140	0.266	0.31	0.28	0.29	0.29	0.29		0.42	0.37	0.42	1.42
	H-E12230	0.232	0.35	0.33	0.29	0.24	0.29		0.30	0.27	0.35	1.45
952	H-E05140	0.463	0.45	0.45	0.46	0.40	0.45		0.58	0.49	0.58	1.53
	H-E05230	0.338	1.10	1.17	0.95	0.60	0.46		0.49	0.50	1.17	1.64
955	H-E04140	0.481	0.62	0.69	0.76	0.85	0.91		0.78	0.46	0.91	1.38
	H-E04230	0.428	1.30	1.33	1.10	0.82	0.76		0.64	0.44	1.33	1.58
5028	H-E07140	0.418	0.86	1.02	1.03	0.90	0.75		0.60	0.33	1.03	1.34
	H-E07230	0.408	1.06	1.31	1.29	1.04	0.70		0.63	0.41	1.31	1.49
5053	H-CX0225	0.342	0.19	0.21	0.21	0.25	0.43		0.55	0.44	0.55	2.41
	H-CX0315	0.271	0.24	0.23	0.22	0.20	0.25		0.28	0.29	0.29	1.51
5054	H-BCR140	0.341	0.24	0.26	0.26	0.30	0.61		0.73	0.55	0.73	2.15
	H-BCR230	0.363	0.29	0.30	0.30	0.27	0.37		0.47	0.50	0.50	1.87
5058	H-E11140	0.298	0.29	0.26	0.26	0.21	0.25		0.29	0.30	0.30	1.44
	H-E11230	0.343	0.26	0.29	0.29	0.22	0.31		0.45	0.45	0.45	1.91
14368	A-DWN180	0.385	0.81	0.97	1.07	1.08	0.94		0.69	0.49	1.08	1.41
	A-DWN270	0.335	0.50	0.55	0.62	0.55	0.50		0.47	0.28	0.62	1.36
23525	A-PMN012	0.414	0.14	0.14	0.22	0.21	0.25		0.30	0.29	0.30	1.84
	A-PMN102	0.349	0.17	0.17	0.19	0.19	0.30		0.36	0.38	0.38	2.27
24303	HOL090	0.274	0.28	0.26	0.25	0.25	0.35		0.38	0.36	0.38	1.45
	HOL360	0.290	0.41	0.41	0.40	0.35	0.41		0.36	0.36	0.41	1.19
24303	A-HOL000	0.500	0.35	0.39	0.44	0.41	0.45		0.44	0.44	0.45	1.17
	A-HOL090	0.356	0.44	0.47	0.45	0.39	0.44		0.43	0.31	0.47	1.18
47380	G02000	0.385	0.48	0.48	0.38	0.39	0.63		0.65	0.48	0.65	1.74
	G02090	0.336	1.01	1.13	0.92	0.54	0.54		0.52	0.36	1.13	1.72
47381	G03000	0.441	0.23	0.19	0.22	0.28	0.43		0.63	0.64	0.64	2.09
	G03090	0.363	0.57	0.61	0.55	0.44	0.44		0.37	0.24	0.61	1.38
Mean		0.353	0.51	0.55	0.52	0.45	0.47		0.48	0.39	0.66	1.61
COV		19.4%	65.9%	70.8%	65.0%	58.2%	39.0%		29.6%	26.1%	51.6%	19.7%
Distribution		N/L	L	L	L	L	L		N/L	N/L	L	N/L
Times of max. drift		-	3	8	2	1	3		8	5	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.8 Displacement response (7-I-a, A95=0.26g)

Ground motions	PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ_i (%)							θ_{max} (%)	Peak roof drift ratio, θ_r peak (%)	θ_{max}/θ_r peak
		1	2	3	4	5	6	7			
135	PEL090	0.61	0.74	0.67	0.51	0.84	0.68	0.36	0.84	0.61	1.38
	PEL180	0.38	0.36	0.41	0.42	0.82	0.70	0.38	0.82	0.38	2.17
	H-E10050	1.10	1.13	0.89	0.52	0.69	0.73	0.45	1.13	0.66	1.71
412	H-E10320	2.68	2.83	2.59	1.91	1.35	0.92	0.39	2.83	1.68	1.69
	H-E12140	0.53	0.59	0.51	0.40	1.06	1.06	0.36	1.06	0.52	2.05
	H-E12230	0.46	0.45	0.40	0.38	0.65	0.57	0.34	0.65	0.39	1.65
952	H-E05140	0.75	0.76	0.65	0.51	1.36	1.30	0.47	1.36	0.65	2.09
	H-E05230	1.08	1.17	1.16	1.16	2.60	2.18	0.66	2.60	1.25	2.08
	H-E04140	0.90	1.01	0.97	0.92	1.98	1.69	0.46	1.98	1.02	1.94
955	H-E04230	2.81	3.01	2.69	2.10	1.31	0.86	0.42	3.01	1.86	1.62
	H-E07140	1.01	1.09	1.11	0.97	2.07	1.89	0.47	2.07	1.12	1.86
	H-E07230	1.86	2.14	2.12	1.85	3.65	2.60	0.55	3.65	1.70	2.14
5053	H-CXO225	0.29	0.29	0.25	0.21	0.65	0.88	0.60	0.88	0.30	2.92
	H-CXO315	0.40	0.40	0.33	0.27	0.58	0.63	0.38	0.63	0.32	1.96
	H-BCR140	0.39	0.38	0.28	0.26	1.21	1.48	0.47	1.48	0.43	3.46
5054	H-BCR230	0.30	0.29	0.33	0.33	1.37	1.41	0.59	1.41	0.54	2.60
	H-E11140	0.35	0.37	0.35	0.31	0.43	0.51	0.39	0.51	0.31	1.62
	H-E11230	0.38	0.36	0.44	0.40	0.70	0.72	0.37	0.72	0.39	1.86
14368	A-DWN180	0.86	1.02	1.12	1.30	2.16	1.76	0.47	2.16	0.98	2.20
	A-DWN270	0.54	0.64	0.67	0.65	1.36	1.54	0.62	1.54	0.62	2.49
	A-PMN012	0.16	0.17	0.22	0.25	0.50	0.43	0.37	0.50	0.25	2.04
23525	A-PMN102	0.30	0.29	0.24	0.23	0.50	0.72	0.38	0.72	0.23	3.08
	HOL090	0.46	0.52	0.49	0.43	0.81	0.79	0.39	0.81	0.49	1.64
	HOL360	0.43	0.46	0.45	0.44	0.86	0.66	0.40	0.86	0.47	1.82
24303	A-HOL000	0.33	0.36	0.41	0.43	0.95	0.82	0.64	0.95	0.48	1.96
	A-HOL090	0.41	0.48	0.51	0.46	1.04	1.01	0.45	1.04	0.53	1.98
	G02000	0.52	0.56	0.57	0.49	1.48	1.63	0.64	1.63	0.61	2.66
47380	G02090	1.36	1.62	1.65	1.36	1.31	1.09	0.54	1.65	1.17	1.41
	G03000	0.32	0.30	0.28	0.31	0.91	1.00	0.76	1.00	0.44	2.29
	G03090	1.11	1.20	1.03	0.69	0.80	0.61	0.35	1.20	0.81	1.49
Mean	COV	0.77	0.83	0.79	0.68	1.20	1.10	0.47	1.39	0.71	2.062
	COV	85.6%	86.6%	84.1%	77.6%	58.8%	49.1%	24.0%	57.6%	63.1%	23.9%
	Distribution	N/L	L	L	L	L	L	N/L	L	L	L
Times of max. drift		0	4	1	0	15	10	0	-	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.9 Displacement response (7-I-b, A95=0.26g)

Ground motions	PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ_i (%)							θ_{max} (%)	Peak roof drift ratio, $\theta_{r, peak}$ (%)	$\theta_{max}/\theta_{r, peak}$
		1	2	3	4	5	6	7			
135	PEL090	0.58	0.60	0.53	0.43	1.44	1.09	0.32	1.44	0.62	2.33
	PEL180	0.35	0.36	0.38	0.38	1.12	0.73	0.28	1.12	0.38	2.94
	H-E10050	1.10	1.12	0.90	0.52	0.81	0.90	0.35	1.12	0.67	1.68
412	H-E10320	2.52	2.67	2.43	1.83	2.00	0.93	0.24	2.67	1.68	1.59
	H-E12140	0.43	0.48	0.51	0.47	1.28	1.21	0.30	1.28	0.52	2.46
	H-E12230	0.39	0.39	0.37	0.31	0.86	0.71	0.31	0.86	0.39	2.20
952	H-E05140	0.70	0.78	0.64	0.44	1.82	1.58	0.36	1.82	0.67	2.73
	H-E05230	1.12	1.19	1.06	0.91	2.76	2.13	0.38	2.76	1.20	2.29
	H-E04140	0.86	0.94	0.85	0.68	2.59	1.89	0.44	2.59	1.04	2.49
955	H-E04230	0.600	2.67	2.56	2.04	1.81	1.00	0.36	2.85	1.90	1.50
	H-E07140	0.587	1.06	1.00	0.79	3.04	2.36	0.36	3.04	1.14	2.66
	H-E07230	0.572	2.03	2.11	2.00	4.99	4.72	0.42	4.99	1.83	2.73
5053	H-CX0225	0.480	0.32	0.26	0.25	0.74	1.15	0.42	1.15	0.32	3.61
	H-CX0315	0.381	0.39	0.33	0.27	0.68	0.71	0.32	0.71	0.33	2.17
5054	H-BCR140	0.479	0.45	0.44	0.32	1.39	1.62	0.40	1.62	0.43	3.80
	H-BCR230	0.510	0.33	0.29	0.29	1.61	1.70	0.41	1.70	0.53	3.20
	H-E11140	0.419	0.34	0.35	0.31	0.51	0.57	0.35	0.57	0.33	1.73
5058	H-E11230	0.482	0.34	0.43	0.38	1.12	1.17	0.35	1.17	0.46	2.57
	A-DWN180	0.541	0.84	0.93	0.87	3.43	2.88	0.39	3.43	1.02	3.37
	A-DWN270	0.471	0.45	0.57	0.56	1.93	1.97	0.37	1.97	0.60	3.29
23525	A-PMN012	0.581	0.17	0.21	0.23	0.58	0.48	0.34	0.58	0.25	2.32
	A-PMN102	0.490	0.30	0.24	0.21	0.51	0.72	0.33	0.72	0.23	3.16
24303	HOL090	0.384	0.43	0.38	0.34	1.24	1.22	0.38	1.24	0.50	2.48
	HOL360	0.407	0.38	0.38	0.36	1.26	0.97	0.30	1.26	0.44	2.89
	A-HOL000	0.702	0.30	0.37	0.36	1.25	0.89	0.60	1.25	0.47	2.65
W	A-HOL090	0.500	0.38	0.48	0.39	1.32	1.10	0.38	1.32	0.54	2.44
	G02000	0.540	0.56	0.53	0.42	1.86	2.15	0.40	2.15	0.65	3.30
	G02090	0.471	1.21	1.43	1.30	2.40	1.59	0.35	2.40	1.23	1.95
47380	G03000	0.619	0.32	0.27	0.26	1.21	1.17	0.49	1.21	0.42	2.85
	G03090	0.509	1.02	0.95	0.67	1.16	0.79	0.31	1.16	0.84	1.38
	Mean	0.496	0.73	0.74	0.62	1.62	1.40	0.37	1.74	0.72	2.559
Distribution	COV	0.194	84.7%	85.7%	83.9%	60.4%	61.0%	18.4%	57.0%	63.8%	24.5%
	Times of max. drift	N/L	L	L	L	L	L	L	L	L	N/L
		-	0	0	0	18	9	0	-	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.10 Displacement response (12-R, A95=0.26g)

Ground motions	PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ_i (%)												θ_{max} (%)	Peak roof drift ratio, θ_r , peak (%)	θ_{max}/θ_r , peak
		1	2	3	4	5	6	7	8	9	10	11	12			
135	PEL090	0.32	0.39	0.38	0.39	0.48	0.55	0.57	0.62	0.92	1.24	1.37	1.00	1.37	0.58	2.35
	PEL180	0.13	0.16	0.17	0.19	0.23	0.25	0.26	0.35	0.62	0.72	0.85	0.72	0.85	0.26	3.30
412	H-E10050	0.91	1.00	0.88	0.72	0.68	0.60	0.51	0.59	0.68	0.72	0.66	0.42	1.00	0.59	1.70
	H-E10320	1.30	1.59	1.62	1.62	1.70	1.70	1.54	1.29	1.07	1.04	1.00	0.53	1.70	1.22	1.39
931	H-E12140	0.25	0.31	0.32	0.32	0.41	0.50	0.57	0.65	0.81	0.98	1.07	1.11	1.11	0.42	2.66
	H-E12230	0.20	0.24	0.24	0.24	0.32	0.39	0.42	0.43	0.56	0.67	0.75	0.67	0.75	0.33	2.25
952	H-E05140	0.31	0.39	0.41	0.41	0.46	0.50	0.52	0.60	0.83	1.04	1.27	1.30	1.30	0.49	2.65
	H-E05230	1.02	1.22	1.15	1.02	1.02	0.98	0.89	0.96	1.27	1.38	1.40	1.36	1.40	1.02	1.37
955	H-E04140	0.676	0.54	0.55	0.48	0.58	0.74	0.81	0.99	1.49	1.78	2.03	1.70	2.03	0.83	2.45
	H-E04230	0.600	1.74	2.06	1.97	1.85	1.57	1.21	1.02	1.08	1.20	1.23	0.87	2.06	1.40	1.48
5028	H-E07140	0.88	1.05	1.00	0.87	0.82	0.74	0.86	1.24	1.64	1.92	1.85	1.32	1.92	0.77	2.49
	H-E07230	1.03	1.27	1.29	1.24	1.31	1.45	1.53	1.79	2.86	3.23	3.04	1.84	3.23	1.57	2.06
5053	H-CXO225	0.17	0.22	0.22	0.19	0.22	0.25	0.24	0.32	0.47	0.60	1.18	1.56	1.56	0.31	4.98
	H-CXO315	0.25	0.27	0.28	0.26	0.31	0.35	0.34	0.36	0.59	0.90	1.27	1.02	1.27	0.37	3.47
5054	H-BCR140	0.23	0.27	0.24	0.22	0.30	0.36	0.37	0.41	0.54	0.96	1.71	2.11	2.11	0.33	6.43
	H-BCR230	0.27	0.33	0.30	0.25	0.26	0.30	0.35	0.46	0.83	1.26	2.28	2.08	2.28	0.49	4.69
5058	H-E11140	0.23	0.28	0.27	0.26	0.29	0.28	0.30	0.37	0.46	0.53	0.84	1.05	1.05	0.31	3.39
	H-E11230	0.34	0.41	0.36	0.32	0.31	0.32	0.37	0.56	1.02	1.29	1.26	1.06	1.29	0.48	2.71
14368	A-DWN180	0.42	0.52	0.50	0.49	0.59	0.73	0.80	1.06	1.86	2.39	2.56	1.68	2.56	0.67	3.80
	A-DWN270	0.17	0.22	0.23	0.21	0.28	0.37	0.44	0.62	1.24	1.79	1.84	1.53	1.84	0.43	4.27
23525	A-PMN012	0.19	0.25	0.25	0.24	0.25	0.28	0.29	0.41	0.70	0.82	0.91	0.67	0.91	0.35	2.57
	A-PMN102	0.11	0.14	0.15	0.18	0.24	0.27	0.25	0.26	0.30	0.42	0.63	0.53	0.63	0.17	3.74
24303	HOL090	0.384	0.24	0.28	0.25	0.29	0.31	0.33	0.39	0.70	1.02	1.60	1.75	1.75	0.43	4.07
	HOL360	0.407	0.19	0.22	0.20	0.24	0.28	0.33	0.41	0.65	0.82	0.93	0.80	0.93	0.34	2.72
24303	A-HOL000	0.702	0.21	0.24	0.18	0.21	0.28	0.38	0.48	0.80	1.02	1.03	0.88	1.03	0.36	2.84
	A-HOL090	0.500	0.17	0.22	0.25	0.34	0.41	0.42	0.53	0.89	1.15	1.22	1.20	1.22	0.44	2.76
47380	G02000	0.540	0.38	0.51	0.53	0.49	0.57	0.57	0.68	1.19	1.51	1.65	1.51	1.65	0.60	2.77
	G02090	0.471	0.76	0.93	0.91	0.92	1.18	1.32	1.38	1.31	1.31	1.21	0.91	1.38	0.89	1.55
47381	G03000	0.619	0.21	0.25	0.23	0.28	0.28	0.27	0.36	0.65	0.93	1.55	1.31	1.55	0.33	4.69
	G03090	0.509	0.59	0.76	0.78	0.87	0.93	0.94	0.97	1.14	1.14	0.88	0.58	1.14	0.82	1.39
Mean		0.496	0.46	0.55	0.54	0.51	0.56	0.60	0.69	0.97	1.19	1.37	1.17	1.50	0.59	2.97
COV		87.2%	86.1%	87.7%	88.0%	78.5%	69.7%	63.4%	55.2%	52.6%	48.6%	41.4%	40.3%	38.7%	58.7%	41.0%
Distribution	N/L	L	L	L	L	L	L	L	L	L	L	L	N/L	L	L	N/L
Times of max. drift	-	0	2	0	0	1	0	0	1	1	3	16	6	-	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.11 Displacement response (12-I-a, A95=0.26g)

Ground motions	PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ (%)												θ_{max} (%)	Peak roof drift ratio, $\theta_{r, peak}$ (%)	$\theta_{max}/\theta_{r, peak}$
		1	2	3	4	5	6	7	8	9	10	11	12			
135	PEL090	0.25	0.31	0.28	0.24	0.27	0.30	0.33	0.69	1.54	2.34	2.24	1.47	2.34	0.67	3.47
	PEL180	0.20	0.23	0.21	0.19	0.22	0.23	0.25	0.37	0.79	1.11	0.78	0.50	1.11	0.24	4.55
412	H-E10050	0.52	0.61	0.58	0.53	0.54	0.55	0.68	1.32	1.84	1.94	1.07	0.53	1.94	0.81	2.39
	H-E10320	0.69	0.82	0.80	0.75	0.79	0.92	1.15	2.74	4.51	4.70	2.57	0.56	4.70	1.42	3.30
931	H-E12140	0.28	0.31	0.30	0.28	0.30	0.29	0.29	0.62	1.59	1.95	1.33	0.75	1.95	0.47	4.17
	H-E12230	0.19	0.23	0.20	0.18	0.22	0.31	0.43	0.80	1.08	1.18	0.82	0.47	1.18	0.40	2.93
952	H-E05140	0.28	0.36	0.34	0.30	0.35	0.40	0.47	1.11	1.77	1.95	1.00	0.62	1.95	0.49	4.01
	H-E05230	1.33	1.55	1.38	1.13	0.97	0.86	0.88	1.73	3.65	3.88	1.42	0.78	3.88	1.32	2.95
955	H-E04140	0.56	0.69	0.66	0.59	0.52	0.46	0.58	1.35	2.72	3.26	2.19	0.85	3.26	0.93	3.51
	H-E04230	1.74	2.06	2.06	1.97	1.85	1.57	1.21	1.02	1.08	1.20	1.23	0.87	1.23	1.40	0.88
5028	H-E07140	1.14	1.24	1.01	0.80	0.73	0.68	0.75	1.38	2.99	3.51	2.04	1.14	3.51	1.10	3.18
	H-E07230	1.04	1.24	1.21	1.08	1.04	1.04	1.13	3.17	7.41	7.61	2.55	0.68	7.61	2.09	3.64
5053	H-CXO225	0.25	0.28	0.26	0.25	0.28	0.27	0.22	0.41	0.66	1.14	1.08	0.61	1.14	0.26	4.40
	H-CXO315	0.22	0.27	0.26	0.24	0.27	0.30	0.34	0.55	1.09	1.39	1.32	0.65	1.39	0.41	3.41
5054	H-BCR140	0.27	0.32	0.32	0.29	0.29	0.28	0.32	0.46	0.84	1.33	1.36	0.60	1.36	0.31	4.35
	H-BCR230	0.35	0.41	0.36	0.30	0.28	0.29	0.30	0.41	0.91	1.76	1.91	0.92	1.91	0.34	5.54
5058	H-E11140	0.25	0.28	0.27	0.27	0.34	0.35	0.34	0.59	1.12	1.15	0.74	0.72	1.15	0.41	2.79
	H-E11230	0.31	0.37	0.33	0.27	0.28	0.27	0.25	0.48	1.36	1.72	1.22	1.12	1.72	0.41	4.16
14368	A-DWN180	0.70	0.80	0.69	0.55	0.44	0.41	0.60	1.38	2.08	3.72	1.84	0.62	3.72	0.52	7.18
	A-DWN270	0.51	0.62	0.56	0.43	0.34	0.25	0.26	0.65	1.47	2.27	2.01	0.78	2.27	0.42	5.41
23525	A-PMN012	0.18	0.20	0.20	0.18	0.22	0.21	0.22	0.49	1.20	1.61	0.93	0.52	1.61	0.33	4.88
	A-PMN102	0.10	0.13	0.13	0.13	0.18	0.24	0.28	0.42	0.38	0.57	0.60	0.50	0.60	0.17	3.60
24303	HOL090	0.33	0.40	0.36	0.29	0.27	0.27	0.30	0.45	1.04	1.56	1.29	0.76	1.56	0.38	4.12
	HOL360	0.19	0.22	0.20	0.20	0.24	0.28	0.33	0.41	0.65	0.82	0.93	0.80	0.93	0.34	2.72
24303	A-HOL000	0.702	0.19	0.23	0.21	0.17	0.19	0.25	0.56	0.90	1.34	1.10	0.70	1.34	0.29	4.65
	A-HOL090	0.500	0.17	0.21	0.20	0.22	0.23	0.24	0.44	1.04	1.43	1.10	0.64	1.43	0.31	4.68
47380	G02000	0.540	0.34	0.45	0.44	0.43	0.41	0.39	0.81	1.68	1.97	1.43	1.12	1.97	0.55	3.60
	G02090	0.471	0.49	0.58	0.50	0.43	0.48	0.69	2.22	3.58	3.87	1.60	0.86	3.87	1.13	3.43
47381	G03000	0.619	0.22	0.27	0.25	0.31	0.31	0.27	0.44	0.98	1.56	1.25	0.85	1.56	0.30	5.27
	G03090	0.509	0.42	0.51	0.49	0.50	0.58	0.74	2.27	3.49	3.64	1.28	0.55	3.64	1.07	3.40
Mean		0.46	0.54	0.50	0.45	0.44	0.44	0.48	0.99	1.85	2.25	1.41	0.75	2.29	0.64	3.91
COV		84.4%	82.9%	84.4%	86.4%	78.2%	69.0%	61.0%	75.9%	80.0%	65.5%	37.8%	30.4%	63.5%	71.7%	28.4
Distribution	N/L	L	L	L	L	-	-	L	L	L	L	N/L	L	L	L	N
Times of max. drift	-	0	1	0	0	0	0	0	0	0	25	4	-	-	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.12 Displacement response (12-I-b, A95=0.26g)

Ground motions	PGA scaled (g)	Maximum inter-storey drift ratio ever occurred, θ_i (%)												Max. θ_{max} (%)	Peak roof drift ratio, θ_r , peak (%)	θ_{max}/θ_r , peak
		1	2	3	4	5	6	7	8	9	10	11	12			
135	PEI090	0.36	0.38	0.34	0.35	0.53	0.72	0.74	0.69	0.95	1.11	1.13	1.05	1.13	0.63	1.80
	PEI180	0.419	0.19	0.18	0.18	0.23	0.27	0.27	0.32	0.47	0.63	0.69	0.74	0.74	0.26	2.90
	H-EI0050	0.365	1.16	1.30	1.20	1.00	0.87	0.63	0.51	0.48	0.46	0.39	0.34	1.30	0.74	1.75
412	H-EI0320	0.440	1.14	1.54	1.64	1.94	2.05	1.87	1.55	1.40	1.08	0.75	0.71	2.05	1.33	1.54
	H-EI2140	0.373	0.39	0.42	0.38	0.43	0.53	0.52	0.51	0.69	0.86	1.00	0.89	1.00	0.42	2.37
	H-EI2230	0.326	0.37	0.42	0.41	0.40	0.47	0.57	0.53	0.44	0.51	0.52	0.47	0.53	0.42	1.27
952	H-E05140	0.650	0.38	0.49	0.51	0.57	0.67	0.72	0.67	0.78	0.91	0.87	0.77	0.91	0.52	1.76
	H-E05230	0.474	1.78	1.92	1.78	1.64	1.65	1.61	1.38	1.09	1.00	1.26	1.18	1.92	1.40	1.38
	H-E04140	0.676	0.76	0.89	0.81	0.70	0.98	1.03	1.03	1.46	1.53	1.38	1.25	1.53	0.88	1.75
955	H-E04230	0.600	2.40	2.70	2.68	2.57	2.43	2.12	1.58	1.14	0.91	0.82	0.69	2.70	1.71	1.58
	H-E07140	0.587	1.03	1.24	1.24	1.20	1.20	1.08	0.94	0.99	1.32	1.55	1.36	1.55	0.98	1.58
	H-E07230	0.572	1.83	1.93	1.78	1.75	1.89	1.81	1.71	2.05	2.09	2.00	1.64	2.09	1.57	1.33
5053	H-CXO225	0.480	0.24	0.24	0.22	0.21	0.25	0.31	0.32	0.30	0.44	0.59	1.15	1.15	0.30	3.79
	H-CXO315	0.381	0.30	0.34	0.32	0.32	0.37	0.45	0.46	0.45	0.55	0.77	1.07	1.07	0.43	2.50
	H-BCR140	0.479	0.26	0.30	0.28	0.27	0.34	0.38	0.34	0.33	0.52	0.72	1.58	1.90	0.32	5.94
5054	H-BCR230	0.510	0.29	0.32	0.29	0.26	0.28	0.35	0.34	0.64	1.07	1.76	1.79	1.79	0.40	4.49
	H-EI1140	0.419	0.43	0.48	0.48	0.44	0.41	0.41	0.36	0.42	0.46	0.73	0.80	0.80	0.39	2.06
	H-EI1230	0.482	0.37	0.41	0.37	0.32	0.34	0.37	0.40	0.45	0.83	1.13	1.17	1.17	0.48	2.45
14368	A-DWN180	0.541	0.36	0.42	0.42	0.45	0.66	0.88	0.90	0.84	1.19	1.78	2.46	2.57	0.54	4.73
	A-DWN270	0.471	0.21	0.25	0.25	0.23	0.32	0.44	0.52	0.36	1.03	1.32	1.73	1.78	0.44	4.09
	A-PMN012	0.581	0.25	0.29	0.27	0.23	0.27	0.33	0.35	0.40	0.63	0.68	0.65	0.68	0.31	2.20
23525	A-PMN102	0.490	0.13	0.13	0.14	0.17	0.25	0.31	0.28	0.24	0.28	0.40	0.59	0.62	0.18	3.50
	HOL090	0.384	0.29	0.33	0.32	0.33	0.41	0.45	0.43	0.40	0.59	0.90	1.28	1.29	0.44	2.90
	HOL360	0.407	0.24	0.27	0.25	0.22	0.26	0.32	0.32	0.36	0.63	0.75	0.88	0.88	0.32	2.75
24303	A-HOL000	0.702	0.23	0.25	0.22	0.19	0.28	0.37	0.39	0.41	0.59	0.77	0.91	1.04	0.31	3.32
	A-HOL090	0.500	0.24	0.25	0.22	0.24	0.33	0.40	0.41	0.44	0.68	0.85	0.83	0.95	0.36	2.63
	G02000	0.540	0.42	0.54	0.57	0.57	0.58	0.60	0.59	0.64	0.91	1.04	1.18	1.57	0.58	2.71
47380	G02090	0.471	0.92	1.13	1.20	1.25	1.34	1.29	1.23	1.05	1.02	1.09	1.11	1.34	0.93	1.43
	G03000	0.619	0.24	0.27	0.25	0.22	0.26	0.29	0.29	0.32	0.62	1.02	1.10	1.14	0.31	3.70
	G03090	0.509	0.82	1.03	1.10	1.14	1.26	1.26	1.15	0.96	0.95	0.93	0.54	1.26	0.91	1.38
47381	Mean	0.496	0.60	0.69	0.67	0.65	0.71	0.76	0.71	0.65	0.81	0.96	1.12	1.35	0.63	2.59
	COV	93.7%	92.3%	93.4%	93.5%	83.5%	72.8%	66.2%	58.6%	45.9%	39.7%	42.3%	44.3%	41.0%	65.4%	45.4%
	Distribution	N/L	L	L	L	L	L	L	L	L	L	L	L	N/L	L	L
Times of max. drift		0	3	0	0	2	1	0	0	0	4	11	9	-	-	-

Note: the shaded cell is the maximum inter-storey drift ratio in each ground motion case

Table B.13 Section details of frame 7-cs

Storey		1	2	3	4	5	6	7
Column	Section (mm)	600×600	600×600	500×500	500×500	400×400	400×400	400×400
	Rebar	12D22.3	12D22.3/ 12D19.1	12D22.3/ 12D19.1	12D22.3/ 12D15.9	12D15.9	12D15.9	8D15.9
	Rebar grade	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60	Grade 60
	ρ_ℓ (%)	1.30	1.30/0.95	1.87/1.37	1.87/0.95	1.49	1.49	0.99
	Hoops	D9.5@150	D9.5@150	D9.5@200	D9.5@200	D9.5@200	D9.5@200	D9.5@200
	Hoop bar grade	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40	Grade 40
	ρ_s (%)	0.50	0.50	0.38	0.38	0.53	0.53	0.50
	$\alpha\omega_w$	0.032	0.032	0.020	0.020	0.026	0.026	0.021
	Hoop pattern
Beam	Storey	Section (mm)	Rebar		Rebar grade	Hoops	Hoop bar grade	
	1	500×300			Grade 60	4D9.5@150	Grade 40	
	2	500×300			Grade 60	4D9.5@150	Grade 40	
	3	500×300			Grade 60	4D9.5@150	Grade 40	
	4	500×300			Grade 60	4D9.5@150	Grade 40	
	5	400×300			Grade 60	4D9.5@150	Grade 40	
	6	400×300			Grade 60	4D9.5@150	Grade 40	
	7	400×300			Grade 60	2D9.5@150	Grade 40	

Note: Beam longitudinal reinforcement: a-2D25.5+2D22.3, b-4D22.3, c-3D19.1, d-3D15.9

Publications

Journal papers

Y. Lu and **X. Gu**, “Probability analysis of RC member deformation limits for different performance levels and reliability of their deterministic calculations”. Structural Safety, Vol. 26, 4, pp. 367~389, Oct. 2004.

Y. Lu, **X. Gu** and J. Guan, “Probabilistic drift limits and performance evaluation of RC columns”. ASCE, Journal of Structural Engineering, Vol. 131, 6, pp. 966~978, Jun. 2005.

X. Gu and Y. Lu, “A fuzzy-random reliability model for seismic performance analysis of frame buildings incorporating both structural and non-structural damage”. Earthquake Engineering & Structural Dynamics, Vol. 34, 10, pp. 1305~1321, Aug. 2005.

Conference paper

Y. Lu and **X. Gu**, “Performance based seismic reliability evaluation of RC frames.” Proceedings, The Eighth Int. Symposium on Structural Engineering for Young Experts, 20-22 Aug 2004, Xian, China, pp. 530-536.

X. Gu and Y. Lu, “Fuzzy-random reliability analysis of building performance.” Proceedings, The Tenth International Conference on Civil, Structural and Environmental Engineering Computing, 30 Aug.-2 Sep. 2005, Rome, Italy. (In press)