INFORMATION TO USERS

This manuscript has been reproduced from the microfilm master. UMI films the text directly from the original or copy submitted. Thus, some thesis and dissertation copies are in typewriter face, while others may be from any type of computer printer.

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleedthrough, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send UMI a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

Oversize materials (e.g., maps, drawings, charts) are reproduced by sectioning the original, beginning at the upper left-hand corner and continuing from left to right in equal sections with small overlaps. Each original is also photographed in one exposure and is included in reduced form at the back of the book.

Photographs included in the original manuscript have been reproduced xerographically in this copy. Higher quality 6" x 9" black and white photographic prints are available for any photographs or illustrations appearing in this copy for an additional charge. Contact UMI directly to order.

UMI

A Bell & Howell Information Company
300 North Zeeb Road, Ann Arbor MI 48106-1346 USA
313/761-4700 800/521-0600

Earthquake Analysis of Steel Moment Resisting Frame Structures

by

Jinbiao Liu

B.Sc. (Engineering Mechanics)
South China University of Technology, China, 1986

A thesis submitted to

The Faculty of Graduate Studies and Research
In partial fulfillment of the requirements

For the degree of
Master of Engineering

Department of Civil and Environmental Engineering

Carleton University, Ottawa

September 1997

The master of Engineering Program in Civil and Environmental
Engineering is a joint program with the University of Ottawa.
administered by the Ottawa-Carleton Institute for Civil Engineering

© Copyright

1997, Jinbiao Liu

National Library
of Canada

Acquisitions and
Bibliographic Services

395 Wellington Street
Ottawa ON K1A 0N4
Canada

Bibliothèque nationale
du Canada

Acquisitions et
services bibliographiques

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file Votre référence

Our file Notre référence

The author has granted a non-exclusive licence allowing the National Library of Canada to reproduce, loan, distribute or sell copies of this thesis in microform, paper or electronic formats.

The author retains ownership of the copyright in this thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without the author's permission.

L'auteur a accordé une licence non exclusive permettant à la Bibliothèque nationale du Canada de reproduire, prêter, distribuer ou vendre des copies de cette thèse sous la forme de microfiche/film, de reproduction sur papier ou sur format électronique.

L'auteur conserve la propriété du droit d'auteur qui protège cette thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

0-612-27000-9

Department of Civil and Environmental Engineering

The Undersigned recommend to the Faculty of Graduate
Studies and Research acceptance of the thesis

Earthquake Analysis of Steel Moment Resisting Frame Structures

submitted by

Jinbiao Liu, B.Sc.

in partial fulfillment of the requirements for
the degree of Master of Engineering

Chair, Department of Civil and
Environmental Engineering

Thesis Supervisor

Carleton University
Ottawa, Ontario, Canada
September 1997

To Jiaxi

Abstract

The thesis investigates the dynamic behaviour and performance of Steel Moment Resisting Frame (SMRF) structures subjected to horizontal and vertical ground motion components of selected earthquake records. Towards this goal, three SMRF buildings of 5-, 10-, and 20-storeys are designed as structural models based on the National Building Code of Canada (NBCC-95) and the standard CAN/CSA-S16.1-94 "Limit States Design of Steel Structures" in the seismic zone $Z_v = 6$. Six actual ground motion records from three recent earthquakes are used as input excitation. They are the 1988 Saguenay earthquake, 1994 Northridge earthquake and 1995 Kobe earthquake scaled to a peak horizontal ground velocity of 0.4 m/s or a peak horizontal ground acceleration of 0.4 g, which is equivalent to the velocity or acceleration seismic zone of Z_v or $Z_a = 6$. The vertical component of the ground motions is scaled by the same scale factor as the horizontal one to evaluate the effect of the vertical ground motions on the seismic behaviour and performance of the three SMRF buildings. The response parameters such as damage pattern, vibration period, top displacement, storey drift, member forces and storey forces are investigated and compared through time history analysis with emphasis on the behaviour in the inelastic range.

The results show that in the elastic range, the vertical ground motion has almost no influence on the seismic response of the SMRF structures except the axial forces. In the inelastic range, the vertical ground motion is a significant factor in contributing

to the extent and pattern of the structural damage. Also it increases the vibration periods, displacements and storey drifts at different storey heights, and decreases shear forces and bending moments of columns and storeys of the damaged structures. The difference of seismic response of the three SMRF structures between horizontal and horizontal plus vertical ground motions are all within 5% except the axial forces. Both in the elastic and inelastic ranges, the column and storey axial forces could be greatly increased when the effect of vertical ground motion is taken into account.

Acknowledgements

The student is deeply grateful to Dr. D.T. Lau of the Department of Civil and Environmental Engineering, Carleton University, for his valuable guidance and advice in the present study.

Sincere thanks to Professor J.L. Humar, Department of Civil and Environment Engineering, Carleton University, for his generous consultation and help offered to the student during his study.

Special thanks are due to my wife for her continuous encouragement, support, patience and understanding during the completion of this thesis.

Thanks to all my colleagues and friends for their help and encouragement.

Contents

Abstract	iv
Acknowledgements	vi
Contents	vii
List of Figures	x
List of Tables	xx
List of Symbols	xxi
1 Introduction	1
1.1 General	1
1.2 Description of Problem	3
1.3 Objectives and Scope	5
1.4 Organization	6
2 Literature Review	8
2.1 General	8
2.2 Seismic Response of SMRF Structure	8
2.2.1 Observed Past Performance	9
2.2.2 Previous Analytical Studies Based on Horizontal Ground Motion	13

2.2.3	Previous Analytical Studies Considering Vertical Ground Motion	15
2.2.4	Earthquake Motions	20
3	Analysis Models	24
3.1	General	24
3.2	Assumptions	25
3.3	Design Load and Procedures	26
3.4	Analysis Model	29
3.5	Description of the Program DRAIN-2DX	29
3.5.1	General Features of the Program	29
3.5.2	Equations of Motion and Step-by-Step Integration	30
3.5.3	Nonlinear Elements	32
4	Static Analysis	38
4.1	General	38
4.2	Analysis Procedure	39
4.3	The Calculation of NBCC (1995) Base Shear and Lateral Distributed forces	40
4.4	Results of Static Analysis	42
5	Dynamic Analysis	63
5.1	General	63
5.2	Analysis Procedure	63
5.3	Selection of Ground Motions	65

5.4	Numerical Results of Horizontal, Horizontal and Vertical Earthquake	
	Excitations	68
5.4.1	Damage Patterns	69
5.4.2	Vibration Periods	75
5.4.3	Lateral Displacements	77
5.4.4	Storey Drifts	81
5.4.5	Member Forces	83
5.4.6	Storey Forces	84
5.5	Summary	85
6	Conclusions and Recommendations	156
6.1	Conclusions	156
6.2	Suggestions for Future Work	158
	References	159

List of Figures

3.1	Floor plan of the 5-, 10-, and 20-storey steel frame buildings	34
3.2	Elevation and member size of 5-storey steel frame building	35
3.3	Elevation and member size of 10-storey steel frame building	36
3.4	Elevation and member size of 20-storey steel frame building	37
4.1	The distributed lateral forces over the frame height of the 5-storey steel frames	48
4.2	The distributed lateral forces over the frame height of the 10-storey steel frames	49
4.3	The distributed lateral forces over the frame height of the 20-storey steel frames	50
4.4	Storey drifts computed from code procedure for the 5-storey steel frame building	51
4.5	Storey drifts computed from code procedure for the 10-storey steel frame building	52
4.6	Storey drifts computed from code procedure for the 20-storey steel frame building	53

4.7	Base shear versus roof displacement responses of the 5-storey steel frames subjected to monotonically increasing lateral loading	54
4.8	Base shear versus roof displacement responses of the 10-storey steel frames subjected to monotonically increasing lateral loading	55
4.9	Base shear versus roof displacement responses of the 20-storey steel frames subjected to monotonically increasing lateral loading	56
4.10	Storey ductility ratios for the 5-storey steel frame building from monotonically increasing lateral loading	57
4.11	Storey ductility ratios for the 10-storey steel frame building from monotonically increasing lateral loading	58
4.12	Storey ductility ratios for the 20-storey steel frame building from monotonically increasing lateral loading	59
4.13	Sequence and location of plastic hinge formation for the 5-storey steel frame resulting from monotonically increasing lateral loading	60
4.14	Sequence and location of plastic hinge formation for the 10-storey steel frame resulting from monotonically increasing lateral loading	61
4.15	Sequence and location of plastic hinge formation for the 20-storey steel frame resulting from monotonically increasing lateral loading	62
5.1	Recorded acceleration time-history for the horizontal and vertical components of the 1988 Saguenay earthquake	89
5.2	Recorded acceleration time-history for the horizontal and vertical components of the 1988 Saguenay earthquake	90

5.3	Recorded acceleration time-history for the horizontal and vertical components of the 1994 Northridge earthquake	91
5.4	Recorded acceleration time-history for the horizontal and vertical components of the 1994 Northridge earthquake	92
5.5	Recorded acceleration time-history for the horizontal and vertical components of the 1995 Kobe earthquake	93
5.6	Recorded acceleration time-history for the horizontal and vertical components of the 1995 Kobe earthquake	94
5.7	Scaled response spectra for the horizontal and vertical components of the 1988 Saguenay earthquake (5 percent damping)	95
5.8	Scaled response spectra for the horizontal and vertical components of the 1988 Saguenay earthquake (5 percent damping)	96
5.9	Scaled response spectra for the horizontal and vertical components of the 1994 Northridge earthquake (5 percent damping)	97
5.10	Response spectra for the horizontal and vertical components of the 1994 Northridge earthquake (5 percent damping)	98
5.11	Scaled response spectra for the horizontal and vertical components of the 1995 Kobe earthquake (5 percent damping)	99
5.12	Response spectra for the horizontal and vertical components of the 1995 Kobe earthquake (5 percent damping)	100
5.13	Sequence and location of plastic hinge formation for the 5-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Horizontal; b) Horizontal and vertical	101

5.14	Sequence and location of plastic hinge formation for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Horizontal; b) Horizontal and vertical	102
5.15	Sequence and location of plastic hinge formation for the 5-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Horizontal; b) Horizontal and vertical	103
5.16	Sequence and location of plastic hinge formation for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Hori- zontal; b) Horizontal and vertical	104
5.17	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Newhall horizontal record of the 1994 Northridge earth- quake	105
5.18	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Newhall horizontal and vertical record of the 1994 Northridge earthquake	106
5.19	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Santa Monica horizontal record of the 1994 Northridge earthquake	107
5.20	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Santa Monica horizontal and vertical record of the 1994 Northridge earthquake	108

5.21	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Kobe university horizontal record of the 1995 Kobe earthquake	109
5.22	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Kobe university horizontal and vertical record of the 1995 Kobe earthquake	110
5.23	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Honzan horizontal record of the 1995 Kobe earthquake	111
5.24	Sequence and location of plastic hinge formation for the 10-storey frame resulting from Honzan horizontal and vertical record of the 1995 Kobe earthquake	112
5.25	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Newhall horizontal and vertical record of the 1994 Northridge earthquake	113
5.26	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Santa Monica horizontal record of the 1994 Northridge earthquake	114
5.27	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Santa Monica horizontal and vertical record of the 1994 Northridge earthquake	115
5.28	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Kobe university horizontal record of the 1995 Kobe earthquake	116

5.29	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Kobe university horizontal and vertical record of the 1995 Kobe earthquake	117
5.30	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Honzan horizontal record of the 1995 Kobe earthquake	118
5.31	Sequence and location of plastic hinge formation for the 20-storey frame resulting from Honzan horizontal and vertical record of the 1995 Kobe earthquake	119
5.32	Period time history of the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode	120
5.33	Period time history of the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode	121
5.34	Period time history of the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode	122
5.35	Period time history of the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode	123
5.36	Period time history of the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode	124
5.37	Period time history of the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode	125

5.38	Lateral displacement time history for the 5-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Top storey; b) 3rd storey; c) 1st storey	126
5.39	Lateral displacement time history for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 3rd storey; c) 1st storey	127
5.40	Lateral displacement time history for the 5-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Top storey; b) 3rd storey; c) 1st storey	128
5.41	Lateral displacement time history for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 3rd storey; c) 1st storey	129
5.42	Lateral displacement time history for the 10-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Top storey; b) 5th storey; c) 1st storey	130
5.43	Lateral displacement time history for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 5th storey; c) 1st storey	131
5.44	Lateral displacement time history for the 10-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Top storey; b) 5th storey; c) 1st storey	132

5.45 Lateral displacement time history for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 5th storey; c) 1st storey	133
5.46 Lateral displacement time history for the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Top storey; b) 10th storey; c) 1st storey	134
5.47 Lateral displacement time history for the 20-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 10th storey; c) 1st storey	135
5.48 Lateral displacement time history for the 20-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Top storey; b) 10th storey; c) 1st storey	136
5.49 Lateral displacement time history for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 10th storey; c) 1st storey	137
5.50 Storey drift time history for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 3rd storey; c) 1st storey	138
5.51 Storey drift time history for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 3rd storey; c) 1st storey	139

5.52	Storey drift time history for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 5th storey; c) 1st storey	140
5.53	Storey drift time history for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 5th storey; c) 1st storey	141
5.54	Storey drift time history for the 20-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 10th storey; c) 1st storey	142
5.55	Storey drift time history for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 10th storey; c) 1st storey	143
5.56	Member force time histories for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake	144
5.57	Member force time histories for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake	145
5.58	Member force time histories for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake	146
5.59	Member force time histories for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake	147
5.60	Member force time histories for the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake	148

5.61	Member force time histories for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake	149
5.62	Storey force time histories for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake	150
5.63	Storey force time histories for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake	151
5.64	Storey force time histories for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake	152
5.65	Storey force time histories for the 10-storey frame resulting from Hon- zan record of the 1995 Kobe earthquake	153
5.66	Storey force time histories for the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake	154
5.67	Storey force time histories for the 20-storey frame resulting from Hon- zan record of the 1995 Kobe earthquake	155

List of Tables

1.1	Major earthquakes during the past twenty years	7
4.1	Vibration periods	47
5.1	Peak ground motion parameters of the selected earthquake records . .	88
5.2	Maximum top displacement of the Saguenay earthquake records . . .	88

List of Symbols

a	Peak horizontal ground acceleration
C_f	Factored axial load
C_r	Factored axial compressive resistance of the member
C_s	Specified gravity load
C_y	Specified column yield axial load
$[C_t]$	Tangent values of the stiffness matrices for the structure at its current state
D	Dead load
E	Earthquake load
F	Foundation factor
F_t	Portion of V to be concentrated at the top of the structure
F_x	Lateral force applied to level x
h_i, h_n, h_x	The height above the base ($i=0$) to level i, n , or x respectively
h_s	Interstorey height
I	Seismic importance factor of the structure
$[K_g]$	Geometric stiffness matrix
L	Live load
$[M]$	Mass matrix

M_{pc}, M_{pb}	The plastic moment capacities of the columns and the beams respectively
M_{fx}	Maximum factored moments
M_{rx}	Factored moment resistances of the member
M_s	Surface wave magnitude of earthquake
M_w	Moment magnitude of earthquake
N	Total number of storeys of the building
n	Total number of floors of the structure
$[K_t]$	Tangent values of the stiffness matrices for the structure at its current state
$\{P\}$	Applied load vector
$\{\Delta P\}$	Increment form of the applied load
R	Force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour
S	Seismic response factor, for unit value of zonal velocity ratio
Δt	Time interval of the current time step
T	Fundamental period of vibration of the structure in seconds in the direction under consideration
U	Factor representing level of protection based on experience
v	Zonal velocity ratio = the specified zonal horizontal ground velocity expressed as a ratio to 1 m/s
$\{\ddot{u}\}, \{\dot{u}\}$ and $\{u\}$	Acceleration, velocity and displacement vector, respectively

$\{\Delta \vec{v}\}$, $\{\Delta \dot{v}\}$ and $\{\Delta v\}$	Increments of acceleration, velocity and displacement. respectively
$\Delta \vec{v}_0$, $\Delta \dot{v}_0$	Initial acceleration and velocity at the beginning of this time step
$\{\Delta \vec{v}_g\}$	Increment of ground acceleration
v	Peak horizontal ground velocity
V	Minimum lateral seismic force at the base of the structure
V_e	Equivalent lateral force at the base of the structure representing elastic response
W	Dead load plus 25% of the design snow load, plus 60% of the storage load for areas used for storage and the full contents of any tanks
W_i , W_x	The portion of W which is located at or is assigned to level i or x respectively
Z	Plastic modulus of the beam section
Z_v	Velocity-related seismic zone
Z_a	Acceleration-related seismic zone
σ_y	Yield stress
α and β	Mass-dependent and stiffness-dependent damping coefficients

Chapter 1

Introduction

1.1 General

Throughout recorded history, the natural hazard of earthquake has been a major cause of loss of human life. In modern society, earthquake hazards in seismically active regions are further exacerbated because of urban development. Collapse of buildings is the leading cause of loss of life in major earthquakes. Because of the rapid urban development in many parts of the world, catastrophic earthquakes seem to occur more frequently in the past twenty years. On July 28, 1976 a major earthquake of magnitude 7.8 occurred in Tangshan, China. The number of deaths resulting from this earthquake amounts to 240,000. In a matter of seconds the entire industrial city of a million people was reduced to rubble (Chen et al., 1976). Other major earthquakes in the past twenty years are listed in Table 1.1. It is worthwhile to note that from that list the Saguenay earthquake is the largest earthquake that occurred in eastern North America in 50 years. The catastrophic damage caused by recent major earthquakes has greatly increased public awareness of the seismic risks in building construction.

Since the 1994 Northridge earthquake in California, there have been new concerns about the safety and performance of steel moment resisting frame (SMRF) structures because of the extensive damage later found in many modern steel buildings as a result of that earthquake (Bertero et al., 1994; Tremblay et al., 1996). Seismic response of structures depends on many parameters including the effect of soil-structure interaction, material characteristics of the structures, and characteristics of the earthquake motion.

In the past 30 years, dynamic response and failure mechanism of SMRF structures have been investigated both experimentally and analytically by many researchers. The findings from this earlier research have been applied to establish safety criteria for the seismic design of steel structures. These earlier investigations have also led to the development of new design procedures and appropriate design base shear factors for the seismic design of SMRF structures. In these investigations, various computational models are employed, and the experimental studies are carried out using scaled laboratory test models. Since many of the past investigations are limited to the case of structures being subjected to only one component of the three-dimensional earthquake motions, in the horizontal direction only, there is the question of whether the understanding and criteria derived from these past investigations on the seismic behaviour and performance of steel buildings are directly applicable to the case of modern steel structures subjected to ground motions with a substantial vertical component.

The existing seismic design provisions considered by the National Building Code of Canada (NBCC 95) assume that it is adequate to design for earthquake forces resulting from ground motions that act only in the horizontal direction. The code's seismic design procedure and base shear formula are developed based on the assumption that the ratio of vertical to horizontal ground motion is no more than $2/3$ to $3/4$, such that the effect of vertical ground motion can be safely ignored. However,

there is clear evidence from the records of recent earthquakes that the peak vertical ground acceleration exceeds this assumption (Saadeghvaziri and Foutch, 1991), as in the cases of the recent 1988 Saguenay, 1994 Northridge and 1995 Kobe earthquakes, where the maximum vertical acceleration recorded at several stations exceeds that in the horizontal direction. Furthermore, there have been suggestions from observations on the performance of SMRF structures in recent earthquakes that vertical ground acceleration may be a significant factor in contributing to the extent and pattern of the damage (Bertero et al., 1994). Consequently, a further investigation on the effect of vertical ground motion on the structural behaviour and performance of high-rise buildings, particularly SMRF buildings, is imperative.

1.2 Description of Problem

Up to now, rather limited attention has been paid to the effect of vertical ground motion on the structural responses, especially in the inelastic range. Among previous studies in which the effects of vertical ground motion have been considered are: Wirsching and Yao (1971), Iyengar and Shinozuka (1972), Gürpinar and Yao (1973), Anderson and Bertero (1973), Cheng and Oster (1976), Buckens (1977), Lin and Shin (1980), Lin and Shih (1982), Ariaratnam and Leung (1990), and Ger et al. (1993).

Wirsching and Yao (1971) adopt the theoretical results of Ariaratnam (1967) for asymptotic stochastic stability of a hinged column under the excitation of a vertical stationary random loading. Iyengar and Shinozuka (1972) apply the Monte Carlo approach where earthquake sample functions are generated by the computer and numerical integrations are performed to obtain the corresponding sample structural response functions in the time domain. Anderson and Bertero (1973) investigate the effect of vertical ground motion on inelastic multi-story buildings using numerical methods. Gürpinar and Yao (1973) adopt an approximate spectral analysis of columns

under seismic loads. Cheng and Oster (1976) apply a deterministic analysis in which highly irregular vertical ground motion was substituted by a sinusoidal motion.

Buckens (1977) analyzes a Timoshenko beam under the parametric excitation of the vertical component of an earthquake. The analysis, however, is incomplete. Lin and Shin (1980) use the theory of Markov process in a study of the response of a mass-less cantilever column supporting a concentrated mass at the top. In addition, Lin and Shih (1982) also conduct a study to analyze shear buildings subjected to both horizontal and vertical excitations, in which the earthquake models are treated as modulated white noise processes.

Ariaratnam and Leung (1990) study the effect of vertical ground acceleration on the earthquake response of elastic structures. The horizontal and vertical accelerations are represented by amplitude-modulated Gaussian random processes using the concept of Markov processes and Itô's stochastic differential equations. The results show that within the limit of linear elastic deformation, the vertical ground acceleration increases the lateral displacement of a moderately tall building by 0.08%. Ger et al. (1993) study the collapse behaviour of a tall steel building during the 1985 Mexico City earthquake by using the multi-component seismic input of actual Mexico City earthquake records.

While all these pioneering studies have provided some useful information, certain questions and limitations in the approaches point to the need for further research. Most of the earlier investigations are confined within the linear elastic range, and almost all of the previous studies consider only one earthquake record or adopt some simplified assumptions about the vertical ground motion. Therefore, the results may not be conclusive. Among the many conclusions and useful information obtained from the earlier studies, an important observation is that it is possible that the presence of the vertical ground motion may increase the damage potential of the horizontal ground motion on the structure. In view of the extent of damage suffered by many

modern steel frame buildings in the 1994 Northridge earthquake and the 1995 Kobe earthquake, the question of the extent of the influence of the vertical ground motion on the response parameters, such as the damage pattern, vibration period, top displacement, storey drift, member forces and storey forces, need to be investigated. Therefore, the objective of this thesis is to investigate the seismic behaviour and performance of SMRF structures subjected to simultaneous horizontal and vertical ground motion excitations, and to obtain information on the influence of the vertical ground motion on the seismic response of the SMRF structures. This thesis is primarily concerned with the interactive effect of the vertical ground motion with the horizontal ground motion on the dynamic behavior and performance of some typical SMRF structures.

1.3 Objectives and Scope

The primary objective of this study is to investigate the seismic behaviour and performance of SMRF structures subjected to both realistic horizontal and vertical earthquake ground motions, and on what extent the vertical ground motion will affect the response of the SMRF structures. The response parameters such as the damage pattern, vibration period, top displacement, storey drift, member forces and storey forces of the SMRF structures are studied.

In pursuing this objective, three SMRF buildings, respectively of 5-, 10-, and 20-storeys are designed, based on the National Building Code of Canada (NBCC-95) and the standard CAN/CSA-S16.1-94 "Limit States Design of Steel Structures" in the seismic zone $Z_v = 6$, and used as structural models in the numerical study. Six ground motion records from three recent earthquakes are selected as the excitation input in the study. These ground motion records are scaled to the appropriate level, which is Z_a or $Z_v = 6$, to evaluate the seismic response of the three SMRF buildings.

Emphasis is placed on the structural responses in the inelastic range.

1.4 Organization

The thesis is divided into six chapters. Chapter 1 introduces the problem, the motivation and objective of the study. Chapter 2 presents a detailed literature review. Findings from previous analytical studies on the SMRF structures are presented along with the advantages and disadvantages associated with the analytical methods and models used in previous studies. The performance of the SMRF structures during the earthquakes are also discussed in this chapter. Chapter 3 describes the analytical models of the present study, including the design procedures and assumptions of the three building models. An overview of the finite element computer program DRAIN-2DX used for the static and dynamic linear and non-linear analyses in the present study is presented at the end of this chapter. Chapter 4 presents the static analysis procedure and the analysis results of the three SMRF structures. Chapter 5 discusses the selection of the ground motions for the seismic response analysis of the three SMRF structures. The computed seismic responses of the three SMRF structures obtained separately from single horizontal ground motion input, and multiple simultaneous horizontal and vertical ground motion inputs, are evaluated and discussed in this chapter. Chapter 6 summarizes the findings of this study and conclusions from this research. Finally, recommendations are made for future work.

Table 1.1: Major earthquakes during the past twenty years

Name	Date	Magnitude	Reference
Chile earthquake	March 3, 1985	7.8	Thiel et al., 1986
Mexico earthquake	September 19, 1985	8.1	Esteva, 1988
Saguenay earthquake	November 25, 1988	5.7	Mitchell et al., 1990
Armenia earthquake	December 7, 1988	6.8	Beavers et al., 1989
Philippine earthquake	July 16, 1990	7.8	Wennerberg, 1991
Hokkaido earthquake	July 12, 1993	7.8	Chung, 1995
Guam earthquake	August 8, 1993	8.1	Moehle et al., 1995
Northridge earthquake	January 17, 1994	6.9	Heidebrecht, 1995
Kobe earthquake	January 17, 1995	7.2	Byrne et al., 1996

Chapter 2

Literature Review

2.1 General

Steel Moment Resisting Frame (SMRF) is a common structural system used in seismic zones. It carries both vertical and lateral loads. Lateral loads are resisted by deflection of the members and joints. The joints of the frame are designed as rigid connections that can resist moment. In this chapter, a literature review on the performance of SMRF buildings in past earthquakes is presented. Previous analytical studies on the seismic behaviour and performance of steel moment resisting frame structures are summarized. Here, emphasis is placed on the effect of the vertical ground motion.

2.2 Seismic Response of SMRF Structure

In this section, previous investigations on the seismic behaviour and performance of steel moment resisting frame structures are discussed. These investigations can be divided into two major categories: observed past performance and analytical study. The work done by earlier investigators in these two areas is discussed below.

2.2.1 Observed Past Performance

Earthquake damage to buildings can be attributed to three primary sources: earthquake-induced vibrations, liquefaction and landslides. Earthquake-induced vibration is a major cause of building damage and collapse and is a significant contributing factor to injuries and fatalities.

In the 1985 Mexico earthquake, SMRF structures performed very well with only a few cases of structural damage reported. The few cases of damage in steel buildings occurred at welded beam-to-column connections or were related to local buckling in columns (Osteraas and Krawinkler, 1989).

In another major earthquake that occurred in Northridge, California in January 1994, studies of the earthquake damage clearly show that SMRF buildings are susceptible to significant damage at welded beam-column joints as a result of welding defects. The observed damage in the Northridge earthquake was significantly different from the damage patterns that are expected in steel buildings, such as large plastic deformations in beams, columns, or joint panel zones, or significant local or lateral torsional buckling of columns. Instead, many steel structures suffered partial or complete fracturing failure in welded beam-to-column moment connections during the Northridge earthquake. In one observed case, about 90% of the connections in one storey of a damaged building were fractured. Overall, local failures at the welded beam-to-column connections have been detected in more than 100 buildings (Ventura et al., 1995). Other failures, such as column uplift and failure of the anchor bolts of the back ties of the steel structures, have also been observed. These failures indicate that the upper structure most likely experienced significant vertical acceleration during the earthquake (Tremblay and Filiatrault, 1994).

The performance of steel moment resisting frame structures during the Northridge earthquake has also been examined by Tremblay et al. (1995). The study confirms current knowledge and understanding of the inelastic response of the structural sys-

tems. The study also concludes that, although in general, adequate seismic performance can be expected from buildings designed according to the seismic provisions recently introduced in the Canadian building design code and the standard for steel structures, there are cases in which the observed damage raises concerns about the adequacy of the seismic design provisions in the current building design codes in considering the influence of the vertical ground motion.

The performance of steel buildings during the Northridge earthquake has also been investigated by Bertero et al. (1994). The observed local failures in modern steel buildings can be grouped in the following three categories: (1) Fracture of column base plates, some of which are as much as 4 inches thick, and associated failures in anchor bolts and reinforced concrete piers; (2) Overall buckling of lateral bracing members and local buckling that in some cases led to fracture of the member; and (3) Failure in welded beam-to-column connections of special moment resistant frames. Their subsequent research has focused on the behaviour of the beam-to-column connections. Research results show that the failure at the beam-column connection is the consequence of the combination of a series of factors, such as: (1) Selection of steel materials including chemical composition, thickness and manufacturing process by which the steel materials are produced; (2) Flaws in the structure related to the method and quality of fabrication, the base material and particularly that of the welds, the design and design details; (3) Magnitude of stress, which are functions not only of the loading and/or deformations to which the structure is subjected and the rate at which they are applied, but also of the internal stresses generated by the fabrication process, particularly at the connections, so that they depend on the structural systems, its design and design details, and the fabrication process; and stress concentrations originating from the welding processes and design details of the connections. Proposed solutions for upgrading the existing structures are suggested. These include: (1) Increasing, as much as economically feasible, the number of bays

with SMRF and the redundancy of the welded connections; (2) Adding stiffness or energy dissipation devices in a sufficiently large number of bays so that the existing welded connections are not required to dissipate more energy than the connections can reliably supply according to existing experimental data; and (3) Using semi-rigid connections as recently suggested by Nader and Astaneh (1992). In the recommendations for needed future research, they suggest that earthquake analyses should include the simultaneous effects of all the components of the earthquake ground motions that can occur at the building site, with particular attention to possible effects of the vertical ground motion.

During the January 17, 1995 Kobe earthquake in Japan, many steel structures were seriously damaged by the earthquake. According to the observations of the Architectural Institute of Japan (AIJ 1995b), which conducted an extensive damage survey in the Kobe area a few days after the event, a total of 3406 buildings were reported to have sustained some level of damage, of which 1247 were steel framed structures. Of the damaged steel buildings, 286 had collapsed or were sufficiently damaged to be considered in danger of collapsing during aftershocks. Numerous failures and examples of inadequate performance could be observed in buildings of various ages, sizes, heights, and braced with different structural systems.

In moment resisting frames, the observed damage patterns include failure of beams, columns, beam-to-column connections, and column bases. A large number of modern steel buildings designed to the latest code requirements have also suffered damage, similar to what has happened in California during the 1994 Northridge earthquake (Tremblay et al., 1995). The Kobe earthquake is an important event in that so many steel frame structures have been damaged. The damage of 988 modern steel buildings in an area extending from the western boundary of the city of Kobe to the eastern limit of the Hyogo prefecture has been meticulously surveyed (AIJ 1995d). Of the 988 modern steel buildings examined, 90 were rated as collapsed, 332

as severely damaged, 266 as moderately damaged, and 300 as slightly damaged. The ratios of these numbers are approximately in the order of 1:3:3:3 (Tremblay et al., 1996). Of the moment resisting frame structures, a large number of old and modern moment resisting frames suffered significant damage. Failure occurred at a number of locations. As in the case of the Northridge earthquake, a disconcerting number of beam-to-column connection failures were discovered following the earthquake.

The failures observed during this earthquake differ somewhat from those observed in the Northridge earthquake. Fracture failure at the beam-column connection was often found accompanied by yielding of beams. This inelastic behaviour was observed in many modern unbraced frames having square-tube columns and full penetration welds at the beams. In the majority of these cases, no sign of yielding was observed in the adjoining columns. Most of the fractures were observed to have occurred in the lower flange of the beams, and the beams exhibited clear signs of yielding as well as local buckling in many cases, although, in some cases, the level of visible yielding was modest. Typically, fracture initiated either from the corner of a weld access hole, near a run-off tab or a weld toe, or in the heat-affected zones on the beam flange or diaphragm. In many cases, the fracture progressed into the beam web and even, in some cases, progressed or initiated into the column flanges. In addition to the above, miscellaneous other forms of damage were also observed in individual buildings, such as local buckling of beams and columns, and failures in column base connections of steel frames.

The 1995 Kobe earthquake caused extensive damage to steel building structures in the Kobe area. Different types and sizes of steel frame buildings suffered severe damage from the earthquake and many different structural failure modes were observed. The observations are of particular interest to the Canadian engineering community, as the seismic setting for the Kobe region and parts of Canada are very similar and analogous design and construction practices are used in Japan and Canada. The seis-

mic setting for the Kobe area is very similar to that of the coastal region of British Columbia.

Recent studies have shown that near-source ground motions can be very demanding in terms of loads and ductility demands on building structures (Anderson and Bertero, 1987; Hall et al., 1995) and that design based on an elastic response spectrum approach as currently adopted in the National Building Code of Canada (NBCC 95) may not be sufficient to ensure adequate seismic performance of structures located in the near-field of important seismic events. Given the potential for large seismic events in urban regions of Canada, more research is needed to understand the characteristics of near-fault ground motions and their effects on building structures.

2.2.2 Previous Analytical Studies Based on Horizontal Ground Motion

Previous studies on earthquake analysis and response of building structures are mostly based on the horizontal earthquake excitation only. In order to better understand the state of seismic research on building structures, a brief literature review on seismic response of building structures based on the horizontal earthquake ground motion is presented here.

In regions of high seismicity, moment resisting frames are widely used as lateral force resisting systems for low-rise steel structures. In the past two decades, several studies have been conducted to investigate the cyclic response of components of the moment resisting frames such as columns and joint panel zones. Popov et al. (1975) investigate the cyclic response of beam-to-column sub-assemblages with weak columns. Results from the study show that columns can perform satisfactorily under cyclic loading conditions, provided they experience axial forces less than 50% of the column yield force. Bertero et al. (1972) and Ghobarah et al. (1992) examine the

cyclic behaviour of joint panel zones. They conclude that panel zones possess the characteristic of stable restoring force and the desirable ability to dissipate the earthquake energy. The implications of the different design philosophies and concepts in current North American Code requirements of strong column weak beam (SCWB), weak column strong beam (WCSB) and weak panel zone strong column (WPSC) on the seismic performance of low-rise steel moment resisting frames are investigated by Osman et al. (1995). The seismic behaviour from low-rise to high-rise moment resisting steel frames subjected to different earthquake accelerations are also investigated by Roder et al. (1993). In this research, the focus is mainly concentrated on comparing the behaviour of frames designed according to the strong column weak beam (SCWB) concept and frames with weak column strong beam (WCSB).

The seismic response of un-braced steel frames is also investigated by Sivakumaran (1991). In his research, two un-braced steel building frames, one 5-storey and the other 10-storey, are studied to determine the behaviour of beam-to-column connections. The static lateral load responses of frames designed based on the "simple construction" and "continuous construction" assumptions are compared. The non-linear dynamic responses of the frames subjected to the 1940 El Centro earthquake record and the 1952 Taft earthquake record scaled by a factor of 2 are also determined by step-by-step time-history analysis. The floor lateral displacement envelops, storey shear envelops and cumulative inelastic rotations of beams, columns and connections are presented. Results indicate that the "simple construction" frames experience larger lateral deflections while attracting lesser storey shears. When subjected to severe support excitations, yielding typically occurs in the columns and connections of "simple construction" frames, whereas in "continuous construction" frames yielding occurs in both beams and columns. The cyclic rotations in the connections and in the columns of "simple construction" frames are found to be considerably higher.

The non-linear response of tall steel moment resisting frame buildings subjected

to earthquake ground motions have been studied by many researchers (Anderson and Bertero, 1969; Meyer, 1974; Tsai and Popov, 1988). The non-linear response of tall planar moment-resistant steel frames considering the P-delta effect under severe earthquake ground motions has been investigated by Challa and Hall (1994).

In all of the aforementioned studies, the ground motion is assumed to occur in one horizontal direction only. Although significant advances have been made in the earthquake resistant design of steel frame structures as a result of this previous research, the effects of multiple directional ground motions such as the significant vertical ground motion components recorded in the 1988 Saguenay earthquake, 1994 Northridge earthquake and the 1995 Kobe earthquake are ignored.

2.2.3 Previous Analytical Studies Considering Vertical Ground Motion

In traditional earthquake analysis of a building, the deflection, bending moment, and shear force of the building model subjected to horizontal ground motion are determined. The effect of vertical ground motion is generally ignored in the evaluation of these structural response parameters.

It is often argued that most structures are quite stiff in the vertical direction. As a result of the design for gravity loads, typical structures have a much greater factor of safety for vertical loads than for horizontal loads. Therefore, it would appear that the additional vertical ground acceleration should not be very important because it rarely exceeds $1g$.

Until recently, little attention was paid to the effect of vertical ground motion on seismic structural responses, partly because of the lack of actual strong motion earthquake records with both horizontal and vertical components recorded simultaneously. Consequently, previous analytical studies on the effect of vertical ground

motion have adopted some simplified assumptions on the characteristics of the vertical ground motion by the deterministic method, the Monte-Carlo method, and the stochastic method. An understanding on the seismic behaviour and performance of building structures subjected to horizontal and vertical ground motions is still very much lacking.

Early research on the effect of vertical ground motion began in the 1960's. Stratonouich (1966) studies the effect of vertical ground motion by assuming the vertical ground motion as a stationary stochastic process. In another study, Wirsching and Yao (1971) use the theoretical results of Ariaratnam (1967) for the asymptotic stochastic stability of a hinged column under the excitation of a vertical stationary random loading. Iyengar and Shinozuka (1972) study the effect of self-weight and vertical acceleration on the behaviour of tall structures that are idealized as uniform cantilevers. It is found that the presence of self-weight and vertical ground excitation can significantly affect the top displacement, base moment and shear force and may either increase or decrease the peak responses. In their research, the effect of the vertical acceleration is considered as change to the weight of the structure, and the horizontal and vertical accelerations are represented jointly by stationary Gaussian random processes, digitally simulated on a computer. Gürpınar and Yao (1973) study the design of columns for seismic loads. They apply spectral analysis to obtain the structural response spectrum at successive short time intervals. Results indicate that the vertical earthquake excitation of earthquake motions should not be ignored in the seismic design of building structures.

In the deterministic approach, Anderson and Bertero (1973) investigate the effect of vertical ground motion on the inelastic response of multi-storey buildings using numerical methods. The analysis model is a ten-storey unbraced steel frame. The results show that the inclusion of gravity load results in a significant increase in the ductility requirements of the upper storey girders and of the lower storey columns.

The inclusion of the vertical component of ground motion can result in a further increase in the ductility requirements of these elements and also increase significantly the ductility requirement of the upper storey columns.

Cheng and Oster (1976) study the effect of coupling earthquake motions on inelastic structural models by deterministic analysis in which the vertical ground motion is represented by a sinusoidal function. The responses of two structural models subjected to the coupling earthquake motions of the vertical ground motion together with the horizontal component of the 1940 El Centro earthquake are studied using a bilinear material model in the analysis. The influences of P-delta effect and linear viscous damping are considered in the investigation. The structural models have lumped masses. In one structural model masses are lumped at the structural joints, whereas the second model has an additional node with lumped mass at the center of an individual girder. Two structures are analyzed: a 4-storey building with 3 bays and a 10-storey single-bay rigid frame building. The results show that the model containing nodes at the girder centers can realistically determine the effect of coupling earthquake motions on structural systems. The influence of the vertical earthquake component depends on the structural response considered. The response parameters considered in the comparison are the maximum horizontal floor displacement, maximum relative horizontal floor displacements, maximum vertical floor displacements, energy absorption, and ductility and excursion ratios.

Cheng and Oster (1976) also explore the technique for dynamic instability analysis of structural systems subjected to vertical earthquakes from which the P-delta effects on the response and structural stability are examined. It is concluded that the vertical earthquake motions can excite some structures having certain natural frequencies to become dynamically unstable due to large deflection. However, the vertical earthquake may not always have an adverse effect on the dynamic response, as it may result in certain structures having smaller deflections in comparison to those

without consideration of the P-delta effect.

Buckens (1977) analyzes the response of a multi-storey building that is modelled as a Timoshenko beam subjected to horizontal and vertical ground motions. The conclusion shows that the effect of vertical accelerations leads to problems of instability.

Cheng and Oster (1977) study the ductility and excursion ratios of inelastic frame structures subjected to earthquake ground motions of simultaneous horizontal and vertical components. The numerical results show the significant effect of coupling earthquake motions on ductility requirements. When a structure is subjected to combined horizontal and vertical earthquake ground motions, the ductility requirement is greater than that for a system subjected to horizontal motion only (Anderson and Bertero, 1973; Cheng and Oster, 1976; Cheng and Oster, 1977).

Cheng and Srfuengfung (1978) study the impact on optimum structural design of simultaneous multi-component static and dynamic inputs based on the Uniform Building Code (UBC 1973, 1976). The results show that the P-delta effect, caused by vertical ground motion and the gravity load, must be considered.

Lin and Shih (1980) study the column response subjected to horizontal and vertical earthquakes. They use a simple single-degree-of-freedom linear system structural model, which consisted of a mass-less column supporting a concentrated mass at the top, and the theory of Markov random process in the analysis. The structural response is treated as a Markov vector in the phase plane. The results show that the vertical ground motion can increase the response of mean-square displacement and velocity.

Lin and Shih (1982) extend the analysis to a multiple degree of freedom system. The structural model considered in the investigation represents a six-storey building with linear elastic and linear viscous damping properties. The vertical and horizontal ground accelerations are modeled by delta-correlated Gaussian random processes us-

ing the concept of Markov's process model and Itô's stochastic differential equations. The restoring force in each storey of the structural model is assumed to be induced from the bending deformation of the columns whose rigidities are subjected to a general reduction due to gravitational forces and to a random variation due to vertical ground accelerations. An analytical procedure is presented for the calculation of the statistical properties of the response of a linear elastic tall building under earthquake excitations. The results show that within the limit of linear elastic deformation, the vertical ground acceleration is shown to have the effect of an increase in the column shear response by 10% in a moderately tall building. The increase is expected to be larger in a taller building and much larger under more intense earthquakes when the deformation exceeds the elastic limit.

Cheng and Kitipitayangkul (1980) study the inelastic behaviour of building systems subjected to three-dimensional earthquake motions. The effect of the simultaneous excitations of the N-S, E-W, and vertical components from the 1940 El Centro earthquake on the response of an eight-storey un-braced steel building with an L-shaped plan is investigated. In their research, an analytical study is carried out to investigate the interactive effect of three dimensional ground motions on the response behaviour of elastic and inelastic building systems consisting of steel and concrete members. The horizontal earthquake inputs are applied in two arbitrary perpendicular directions to the plan of the building. The P-delta effect of the second-order moment resulting from the gravity load and the vertical ground motion is considered. Numerical results show that interacting horizontal components can significantly increase the internal forces and the lateral displacements of the structure resulting in significant permanent deformations. The amount of the influence depends on the geometric condition of the structural plan and elevation. The results of considering vertical ground motion show that there is a significant increase in axial forces but only a slight increase in bending moments of columns.

Ariaratnam and Leung (1990) study the effect of vertical ground acceleration on the earthquake response of elastic structures, using the same procedures and assumptions as Lin and Shih (1982). The statistical response of a linear elastic six-storey building is presented. The results show that within the limit of linear elastic deformation, the vertical ground acceleration increases the lateral displacement of a moderately tall building by 0.08%.

Ger et al. (1993) study the collapse behaviour of a tall steel building during the 1985 Mexico City earthquake by using the multi-component seismic input of actual Mexico City earthquake records. The structural system of the analyzed building is moment resisting steel frames. The results show that the structural response exceeds the original design ductility of the building. Many girders in the building suffer severe inelastic deformation. Due to load redistribution that results from ductile girder failure, local buckling occurs in many columns in floors 2, 3, and 4. Therefore, most columns on floors 2 to 4 lose their load-carrying capabilities and rigidities, which then causes the building to tilt and rotate.

2.2.4 Earthquake Motions

The NBCC 95 seismic design procedure and base shear formula are developed based on the assumption that the ratio of vertical to horizontal ground motion is $2/3$ to $3/4$. However, contrary to this assumption, observations from recent earthquakes indicate that the vertical acceleration is not only significant (Bertero et al., 1994; Tremblay and Filiatrault, 1994; Tremblay, 1995), but there are even earthquake records where the maximum vertical acceleration exceeds that in the horizontal direction.

The maximum vertical acceleration of the San Fernando earthquake recorded at Pacoima Dam is 0.7 g. The maximum displacement observed across the fault trace is reported to have been about six feet in the vertical direction where it is only five feet in the lateral direction. The El Asnam earthquake of Oct. 10, 1980 is considered ex-

traordinary because of the strong movement in the vertical direction. Unfortunately, no acceleration record of this earthquake is available. However, there are indications that the vertical acceleration may have reached a maximum very close to 1.0 g, while the maximum horizontal acceleration is estimated to have been about 0.25 g. During the Loma Prieta earthquake of Oct. 17, 1989, the peak vertical acceleration recorded at several locations is greater than the peak horizontal acceleration, and the peak vertical acceleration was 0.7 g (Saadeghvaziri and Foutch, 1991).

During the 1988 Saguenay earthquake a maximum vertical acceleration of 0.23 g was recorded at Les Eboulements, where in comparison the maximum horizontal acceleration was only 0.12 g. The ratio of vertical to horizontal ground motion is almost 2. During the Northridge earthquake of January 17, 1994, a maximum vertical acceleration recorded at Arleta was 0.55 g, while the maximum horizontal acceleration was only 0.34 g. Again, the vertical ground motion was much higher than the horizontal one. During the January 17, 1995 Kobe earthquake, the peak vertical acceleration recorded at several locations was greater than the peak horizontal acceleration. A maximum vertical acceleration of 0.45 g was recorded at a rock site of Kobe university, where the maximum horizontal acceleration was only 0.30 g. The ratio of vertical to horizontal ground motion is 1.5.

The ratio of vertical to horizontal ground motion varies widely depending on site conditions. In the National Building Code of Canada (NBCC 1995), an average value for this ratio is assumed to be between $2/3$ to $3/4$. As noted, the observed ratio of vertical to horizontal ground motion in recent earthquakes exceeds the assumed limit.

Concerning the influence of vertical ground motion on seismic design of multi-storey buildings, the NBCC considers that columns at upper floors, especially at the roof level, could be adversely affected by abnormally high vertical accelerations. However, the NBCC assumes that there is usually sufficient reserve strength in vertical load-carrying members so that vertical accelerations can be safely neglected. In view

of the extensive damage and collapse suffered by steel buildings and the apparently different damage pattern exhibited by the damaged buildings in recent earthquakes in California and Japan, a systematic investigation on the effect of vertical ground motion on the seismic behaviour of multi-storey buildings is needed.

Since strong earthquake ground motion records generally last only a short time and are highly irregular, earlier investigations on the effect of vertical ground motion have the following shortcomings: First, modeling such motions either as deterministic sinusoidal functions or even stationary stochastic processes is not appropriate. Second, the asymptotic stability solution has little practical significance when the duration of the excitation is limited. Third, the vertical acceleration causes the structural stiffness to vary with time, thus making the spectral technique inapplicable. Fourth, the Monte Carlo method as a tool for random vibration analysis has proved to be valuable, providing qualitative results, and sometimes providing the only results when other methods are not available. But the amount of computing time required for meaningful Monte Carlo solutions escalates astronomically as the structural model becomes more complex.

Compared to the progress in the analytical studies of structures subjected to horizontal ground motion, the research on the effect of multiple directional ground motions on structural response significantly lags behind. Although some researchers have investigated the effect of vertical ground motion on the response of building structures, unfortunately, they did not consider the effect of deformations exceeding the elastic limit and/or their analytical model was not a practical building, and usually only limited earthquake records were considered.

To better understand the effect of multiple ground motions, including the vertical ground motions, on the seismic behaviour and performance of typical SMRF structures, a detailed parametric study should be carried out. In comparison to previous analytical studies of vertical ground motion effects, the present study considers the

following: (1) Selection of earthquake records with different characteristics, (2) Studied structures designed according to the provisions of the NBCC 95 and the standard CAN/CSA-S16.1-94 "Limit States Design of Steel Structures", (3) Consideration of plastic deformation in the structures.

Chapter 3

Analysis Models

3.1 General

The objective of the earthquake-resistant design requirements in the National Building Code of Canada (NBCC 1995) is to provide an acceptable level of public safety in engineered buildings. This is achieved by a design philosophy that prevents major failure and loss of life. Structures designed in conformance with the NBCC seismic provisions should be able to resist moderate earthquakes without significant damage and major earthquakes without collapse. The objective of the NBCC 1995 provisions is to reduce the probability of fatalities to an appropriately small level while the designed structures suffer damage but do not collapse in major earthquakes. To design an earthquake-resistant structure, one needs to know the characteristics of the “design” seismic ground motion, the characteristics of the structure and the foundation, the allowable stresses in the construction materials, including the foundation soil, and the amount of damage that is tolerable. The design must provide not only sufficient structural strength to resist the ground motion, but also the proper stiffness to limit the lateral deflection or drift. The design earthquake of the NBCC 1995 is defined as

an event that has a probability of exceedance of 10% in 50 years.

In this chapter, the assumptions of the analysis models, the design loading and design procedures of the studied buildings are presented. Three steel moment resistant frames respectively of 5-, 10-, and 20-storeys, are designed for use in the numerical study. The computer program used in the analysis is DRAIN-2DX (Prakash et al., 1993), capable of calculating the nonlinear static and dynamic response of 2-dimensional planar structures; it is described at the end of this chapter.

3.2 Assumptions

In the analysis and design of steel building frames to meet the serviceability and fatigue requirements under specified loads and the strength and overturning requirements under factored loads, simplified assumptions are usually made in regards to the behaviour of beam-to-column connections and column-to-base connections in the determination of the structural response to the considered loading. The standard CAN/CSA-S16.1-94 "Limit States Design of Steel Structures" published by the Canadian Standards Association (1994) recognizes various types of construction and the associated design assumptions. Accordingly, in the analysis and design of steel building frames subjected to gravity, wind or earthquake loads, and a combination of the gravity and lateral loads, the designer may use one of the following concepts:

- (1) Continuous construction (clause 8.2), commonly known as a rigid frame.
- (2) Simple construction (clause 8.3), commonly known as a simple frame.

Based on the aforementioned provisions, the following design practices and assumptions are followed in the design process. The sample buildings are designed following the continuous construction concept of 5-, 10-, and 20-storey SMRF structures. That means the beam-to-column connections have sufficient rigidity that the original angles between the interconnecting members at the joints remain unchanged during

the dynamic response while the moment due to both gravity and lateral loads are transmitted through the connections to the columns. A non-composite floor system is assumed in the design of the three SMRF structures. The three SMRF structures are un-braced. The floor diaphragms are assumed to be rigid, and the base of the columns are assumed to be fixed at the foundation level in the analysis models.

The contribution and effect of the non-structural elements of the buildings are neglected in the dynamic analysis. Although in some instances, the contribution of non-structural elements to the lateral load resistance of a building can be significant, especially in the early phase of an earthquake for relatively flexible systems, their effects on the inelastic behaviour under large deformations of the buildings are usually of less significance. A numerical quantification of the contribution of non-structural elements to the strength of the structure is very difficult, therefore the contribution of the non-structural elements to the stiffness property of the examined buildings is ignored.

3.3 Design Load and Procedures

Building codes and standards have, for many years, incorporated seismic design provisions with the design philosophy that buildings are expected to undergo significant inelastic response without collapse during severe earthquakes. As a consequence of this design philosophy, typical buildings are designed to resist inelastic seismic design forces smaller than those expected if the structure is to respond elastically.

The three SMRF building structures under consideration are subjected to two types of applied forces:

- (1) Stationary vertical gravity forces due to the dead weight, resulting from occupancy and use, and equipment loads; and
- (2) Transient horizontal lateral forces caused by earthquake or wind loads.

According to the NBCC 1995, the factored load combinations, including earthquakes are taken as follows:

a) $1.0D + 1.0E$ and,

b) $1.0D + 0.5L + 1.0E$,

where D is the dead load, L is the live load, and E is the earthquake load. Since the buildings are taken as office buildings, the dead load is assumed to be 3.3 KN/m^2 at the roof level. 3.0 KN/m^2 at other office floors and 4.0 KN/m^2 at the retail level. The live load is taken as 2.4 KN/m^2 according to the live load provision of office buildings in NBCC 95. A uniform live load reduction factor of 0.93 is used for the design of both the beams and columns.

Torsional moments are ignored by assuming the buildings are regular and symmetric with respect to the resistance. As is customary, the column sections are changed every two floors and the girders in the same floor have the same size. The analysis and design of the frames are based on the bare steel frame.

The design philosophy of strong column - weak beam (SCWB) is used for designing the buildings. In this design philosophy, the beams are detailed to be weaker than the adjoining columns. In the Canadian Steel Design Standard (CAN/CSA-S16.1-94), which is based on the limit states design concept, this criterion is satisfied by ensuring that the strength of the columns at each joint is higher than the over-strength of the adjoining beams using the relationship

$$\Sigma M_{pc}(1 - C_s/C_y)/0.85 > 1.2\Sigma M_{pb} \quad (3.1)$$

where 1.2 is the over-strength factor, M_{pc} and M_{pb} are respectively the plastic moment capacities of the columns and the beams, while C_s and C_y are respectively the specified gravity load and column yield axial load. The factor 0.85 is the inverse of 1.18, which is an empirical curve fitting parameter for the ultimate strength interaction relationship for the case where that instability is of no concern.

The beam yield strength is limited to $Z\sigma_y$, where Z is the plastic modulus of

the beam section and σ_y is the yield stress. For the columns, a yield interaction relationship involving both axial force and bending moment is prescribed by the expression

$$C_f/C_r + 0.85U_{1x}M_{fx}/M_{rx} \leq 1.0 \quad (3.2)$$

where C_f is the factored axial load, C_r is the factored axial compressive resistance of the member, $U_{1x} = 1.0$, M_{fx} is the maximum factored moments, M_{rx} is the factored moment resistances of the member, and the coefficient 0.85 reflects the increased bending resistance of Class 1 sections. The flexibility effects of the joint panel zone response are neglected in the analysis.

The lateral design base shear, V , for each frame is determined by the base shear formula of NBCC 1995. The three SMRF structures are assumed to be located in regions with high seismicity, namely, $Z_v = 6$. Accordingly, v is taken as 0.4. A storey drift limitation is also imposed by the NBCC 1995, which is $0.01h_s$ for post-disaster buildings and $0.02h_s$ for all other buildings where h_s is the storey height. The calculated inter-storey drift to be limited by this value is the maximum expected drift during the inelastic response of the structure, which is specified to be R times the elastic value obtained from the given seismic load.

The seismic design of the buildings is carried out by an iterative process as follows:

1. Preliminary selection of member sizes.
2. Seismic design loads are calculated based on the building system and a load reduction factor $R = 4$.
3. Horizontal forces at each storey are calculated based on the lateral force distribution formula of NBCC 95.
4. The equivalent static lateral forces are applied to the structure together with other loads in appropriate combinations, to give the first design of the building.
5. Lateral deflections of the designed buildings are calculated as R times the elastic deflections under the seismic load. These are compared with the seismic drift

limitation of $0.02h_s$.

6. If the frame is considered too stiff or too weak to meet the drift requirement, the design process in step 2 is repeated by modifying some member sizes until the storey drift limitation set by NBCC is satisfied.

3.4 Analysis Model

The buildings considered in the present study are 5-, 10- and 20-storey steel office buildings assumed to be located in regions of high seismicity, namely, Zone 6 with $v = 0.40$. The three SMRF structures have the same floor plan. Figure 3.1 shows the typical floor plan of the three designed SMRF structures. The seismic action is assumed to be in the N-S direction. A typical interior frame for each of the three buildings in the N-S direction is designed and analyzed. Figures 3.2, 3.3 and 3.4 show the structural layout and member size for the interior frame elevations of the 5-, 10- and 20-storey buildings, respectively. To achieve an optimum design, the beam and column sizes are changed over the frame height with the variation occurring every two storeys. This ensures a smooth transition in member stiffness throughout the height of each frame. The buildings are designed in accordance with the National Building Code of Canada (NBCC 1995) and the standard CAN/CSA-S16.1-94 "Limit States Design of Steel Structures."

3.5 Description of the Program DRAIN-2DX

3.5.1 General Features of the Program

The computer program DRAIN-2DX (Dynamic Response Analysis of INelastic 2-Dimensional Structures) is employed to perform inelastic static and dynamic analysis

of the three SMRF structures. The version of the computer program DRAIN-2DX used in the present study was released in 1993 (Prakash et al., 1993). This computer program provides a number of features that are important for nonlinear static and dynamic seismic analysis. When DRAIN-2DX performs dynamic analysis of structures, dynamic loading may be specified as support acceleration with all supports moving in phase or ground displacements with the supports moving out of phase. imposed dynamic loads, and specified initial velocities. Static and dynamic loads can be applied in any sequence.

The static nonlinear analysis is performed by an event-to-event scheme, where each event corresponds to a significant change in stiffness. In the nonlinear (material and geometric) dynamic analysis, the equations of motion are solved using a step-by-step numerical integration procedure in which the acceleration during a small time increment is assumed to be constant.

3.5.2 Equations of Motion and Step-by-Step Integration

In this part, the procedure to solve the governing equations of motion is briefly described for the completeness of the thesis. Details can be found in the theoretical manual of the program by Kannan and Powell (1973) and other standard references (Humar, 1990).

At any instant of time, the general form of the equations of motion for multi-degree-of-freedom systems can be expressed as follows:

$$[M]\{\ddot{v}\} + [C_t]\{\dot{v}\} + [K_t]\{v\} = \{P\} \quad (3.3)$$

where $[M]$ is the mass matrix; $[C_t]$ and $[K_t]$ are tangent values of the damping and stiffness matrices for the structure at its current state; $\{P\}$ is the applied load vector; and $\{\ddot{v}\}$, $\{\dot{v}\}$ and $\{v\}$ are the acceleration, velocity and displacement vector, respectively.

In the non-linear range, the general form of the equations of motion for the multi-degree-of-freedom systems cannot be solved directly. The incremental form of the equations of motion is usually used instead in the non-linear numerical analysis. For a very short time interval, the general form of the equations of motion for multi-degree-of-freedom systems can be rewritten in the incremental form for the time step from time t_i to time t_{i+1} as follows:

$$[M]\{\Delta\ddot{v}\} + [C_t]\{\Delta\dot{v}\} + [K_t]\{\Delta v\} = \{\Delta P\} \quad (3.4)$$

where $\{\Delta\ddot{v}\}$, $\{\Delta\dot{v}\}$ and $\{\Delta v\}$ are the increments of acceleration, velocity and displacement, respectively; and $\{\Delta P\}$ is the increment form of the applied load. In the present study, the average (constant) acceleration method within each time step is adopted to solve Equation 3.4. The advantage of this method is that it is numerically stable for all vibration periods and time step intervals, and does not introduce damping errors into the system. Thus, with the average acceleration method (Humar, 1990),

$$\Delta\dot{v} = -2\dot{v}_0 + \Delta v \frac{2}{\Delta t} \quad (3.5)$$

$$\Delta\ddot{v} = -2\ddot{v}_0 - \dot{v}_0 \frac{4}{\Delta t} + \Delta v \frac{4}{\Delta t^2} \quad (3.6)$$

where $\Delta\ddot{v}_0$ and $\Delta\dot{v}_0$ are the initial acceleration and velocity at the beginning of this time step; and Δt is the time interval of the current time step. Substituting Equation 3.5 and 3.6 into Equation 3.4 yields

$$\left[\frac{4}{\Delta t^2}[M] + \frac{2}{\Delta t}[C_t] + [K_t]\right]\{\Delta v\} = \{\Delta P\} + [M]\{2\ddot{v}_0 + \dot{v}_0 \frac{4}{\Delta t}\} + [C_t]\{2\dot{v}_0\} \quad (3.7)$$

In the analysis, it is assumed that the damping property of the buildings can be expressed as a combination of the mass and stiffness matrices, as follows

$$[C_t] = \alpha[M] + \beta[K_t] \quad (3.8)$$

where α and β are the mass-dependent and stiffness-dependent damping coefficients. Thus, substituting Equation 3.8 into Equation 3.7 yields

$$\begin{aligned} \left[\left(\frac{4}{\Delta t^2} + \frac{2\alpha}{\Delta t}\right)[M] + \left(\frac{2\beta}{\Delta t} + 1\right)[K_t]\right]\{\Delta v\} &= \{\Delta P\} + [M]\{2\ddot{v}_0 + \dot{v}_0 \frac{4}{\Delta t} + \\ &\quad 2\alpha\dot{v}_0\} + \beta[K_t]\{2\dot{v}_0\} \end{aligned} \quad (3.9)$$

which can be rewritten in the following form

$$[K_t^*]\{\Delta v\} = \{\Delta P^*\} \quad (3.10)$$

where

$$[K_t^*] = \left(\frac{4}{\Delta t^2} + \frac{2\alpha}{\Delta t}\right)[M] + \left(\frac{2\beta}{\Delta t} + 1\right)[K_t] \quad (3.11)$$

$$\{\Delta P^*\} = \{\Delta P\} + [M]\{2\ddot{v}_0 + \dot{v}_0 \frac{4}{\Delta t} + 2\alpha\dot{v}_0\} + \beta[K_t]\{2\dot{v}_0\} \quad (3.12)$$

The incremental displacement vector Δv is obtained by solving Equation 3.10. Knowing the displacement vector Δv , the incremental velocity vector $\Delta \dot{v}$ can be obtained using Equation 3.5. Finally, the acceleration response can be calculated by solving with the known current displacement and velocity responses, which have been updated with the calculated corresponding increment values from Equation 3.3.

For earthquake excitation, $\{\Delta P\}$ is replaced by $-[M]\{\Delta \ddot{v}_g\}$, where $\{\Delta \ddot{v}_g\}$ is the increment of ground acceleration. The calculated displacements, velocities and accelerations from Equation 3.10 are relative to the ground support.

3.5.3 Nonlinear Elements

The inelastic static and dynamic responses of the building frames are analyzed using the DRAIN-2DX computer program (Prakash et al., 1993). Equivalent viscous damping of 5% of critical damping in the first and second modes is assumed in the analysis models of 5- and 10-storey buildings, respectively. For the analysis model of 20-storey building, the equivalent viscous damping of 5% of critical damping in the first and

fifth modes is assumed. A nonlinear beam-column element is used in the analysis. This is a simple inelastic element for modeling beams and columns of steel and reinforced concrete. The element consists essentially of an elastic beam, two rigid-plastic hinges at the ends of this beam, and optional rigid end zones. Yielding takes place only in the plastic hinges. The moment-rotation relationship is bilinear. Unloading and reloading will proceed linearly with stiffness similar to the initial stiffness. Strain hardening in bending is modelled by assuming that the element consists of elastic and inelastic components in parallel. Plastic hinges form in the inelastic component. The moment continues to increase in the elastic component for simulating the strain hardening effect. The P-delta effects are considered based on the linear approximation of the geometric stiffness of a truss bar element. The influences of panel zone yielding and deformation are not considered in the present study. Further, the stiffness and resistance of the deck slab are not included.

Figure 3.1: Floor plan of the 5-, 10-, and 20-storey steel frame buildings

Figure 3.2: Elevation and member size of 5-storey steel frame building

Figure 3.3: Elevation and member size of 10-storey steel frame building

Figure 3.4: Elevation and member size of 20-storey steel frame building

Chapter 4

Static Analysis

4.1 General

To evaluate the seismic behaviour and performance of SMRF structures, the inelastic behaviour of the designed frames subjected to monotonically increased lateral load is studied first. The static push-over analysis can provide some useful insight for the evaluation of the inelastic dynamic responses of the frames subjected to earthquake excitations. Estimates on the member forces and the global and local deformations the structure is likely to experience can be obtained, which then can be used to assess the integrity of the structural system. The purposes of the push-over analysis are to permit an approximate evaluation of the deformation demands in critical elements, expose undesirable characteristics such as strength and stiffness discontinuities and overloads on potentially brittle elements, expose regions of large deformation demands requiring proper detailing, and assess global stability of the structural system (Lawson et al., 1994). Another reason for conducting the static analysis is that seismic design procedure in the NBCC-95 is based on the equivalent static load approach, so that the static results obtained here can be used to compare with the design load specified

by the code.

Details of the three SMRF structures considered in the present study are presented in Chapter 3. The three building structures of 5-, 10-, and 20-storeys are designed to resist a combination of gravity, live load and equivalent lateral seismic forces as specified in NBCC 1995. Full gravity loads without modification by load factors and one half of the live loads are applied on the frames before the application of the lateral load. The analysis procedure and results of the static push-over analysis are presented in this chapter.

4.2 Analysis Procedure

The structural analysis models of the 5-, 10-, and 20-storey high SMRF structures used for this study are shown in Figures 3.1 to 3.4. A typical interior frame in the N-S direction of each building is analyzed for incremental static push-over considering gravity and earthquake forces. The computer program DRAIN-2DX (Prakash et al., 1993) is employed for the analysis. In the analysis, the plastic hinge beam-column element is employed to model the beams and columns of the building frames. In this plastic hinge beam-column element model, a structural member is represented by an inelastic component. The inelastic deformation is associated with the inelastic component and is assumed to be concentrated at the two ends of the member. The strain hardening in bending can be considered by modelling the element that it consists of an elastic component in parallel with an inelastic component, where the elastic component is included to account for the strain hardening effect. The moment-rotation relationship of any potential plastic hinge is taken to be bilinear. The effect of axial force and bending moment interaction (P-M interaction) is considered for each column by a yield moment-axial force interaction curve specified by the Canadian Steel Design Standard (CAN/CSA-S16.1-94). The P-M interaction is not considered

for beams because the axial deformation of beams are constrained to be zero. The P-delta effect is taken into account only for columns by adding the geometric stiffness to the column stiffness. The element length is taken as the distance between the joint centers. The strain hardening is assumed to be zero.

In the static analysis, the full gravity loads without modification by the load combination factor and one half of the live loads are applied on the frame before the application of the lateral earthquake loads that are distributed over the frame height in accordance with the NBCC 1995 lateral force distribution formula. The lateral earthquake loads are applied incrementally until a mechanism is formed or the overall drift of the structure reaches the 2% limit, whichever comes first.

4.3 The Calculation of NBCC (1995) Base Shear and Lateral Distributed forces

The lateral design base shear, V , for each frame is determined by the base shear formula specified in the NBCC 1995

$$V = \frac{V_e}{R} U \quad (4.1)$$

where V_e is the equivalent lateral seismic force representing elastic response, R is the force modification factor, and U is a calibration factor equal to 0.6. For the ductile moment resistant frame (DMRF) buildings considered, R is taken as 4, which implies that the frame is capable of sustaining repeated cyclic lateral deflections of about 4 times the deflection corresponding to initial significant yield. The elastic design base shear, V_e is given as follows:

$$V_e = v S I F W \quad (4.2)$$

where v is the zonal velocity ratio, S is the seismic response factor, I is the importance factor, F is the foundation factor, and W is the dead load plus 25% of the design snow load, plus 60% of the storage load for areas used for storage and the full contents of any tanks. The three frames are assumed to be located in regions with high seismicity, namely, $Z_v = 6$. Accordingly, v is taken as 0.4. I and F are set equal to 1.0.

In order to calculate the seismic response factor, S , the vibration periods of the analysis models are determined first by the DRAIN-2DX computer program. The calculated vibration periods of the analysis model for the 5-, 10-, and 20-storey buildings are shown in Table 4.1. The fundamental periods for the 5-, 10-, and 20-storey buildings are 1.367 seconds, 2.194 seconds, and 3.656 seconds, respectively. According to Clause (a) of Sentence 4.1.9.1.(7) of NBCC 95, where the fundamental period, T , shall be determined by the formula $0.1 N$ for any moment resisting frame, where N is the total number of storeys of the building, the corresponding code specified vibration periods for the three structures are 0.5 seconds, 1.0 seconds, and 2.0 seconds, respectively. It is clear that the calculated vibration periods are larger than the code-specified values. However, these results agree with the results from other comparative studies on the fundamental periods of vibration obtained from modal analyses and code formulae (Osman et al., 1995; Redwood et al., 1990; Naman and Goodno, 1986; Montgomery and Hall, 1979). Since the fundamental periods of the three SMRF structures are all greater than 0.5s, according to Sentence 4.1.9.1.(6) of NBCC 95, the seismic response factor, S , shall be taken as $1.5/T^{1/2}$, where T is the fundamental period of vibration of the structure in seconds in the direction under consideration. According to Clause (c) of Sentence 4.1.9.1.(7) of NBCC 95 the value of V_e used for design shall be not less than 0.80 of the value computed using the period calculated in Clause (a) of Sentence 4.1.9.1.(7) of NBCC 95. Therefore, the 1.2 times code specified vibration periods of the three SMRF structures are used to determine the seismic response factors of the three SMRF structures.

The calculated seismic response factors for the 5-, 10-, and 20-storey buildings are 1.94, 1.37 and 0.97, respectively. Correspondingly, the calculated base shears for the 5-, 10-, and 20-storey buildings are 335.32 KN, 467.46 KN and 647.4 KN, respectively.

According to Clause (a) of Sentence 4.1.9.1.(13) of NBCC 95, a portion, F_t , of the total lateral seismic force, V , contributed from the higher vibration modes of the structures is distributed at the top of the frame. It is equal to $0.07 TV$, except that F_t need not exceed $0.25 V$ and may be considered as zero where T does not exceed 0.7 seconds. The remainder of the lateral design base shear, $V - F_t$, is distributed along the height of the building, including the top level, in accordance to the formula

$$F_x = \frac{(V - F_t)W_x h_x}{(\sum_{i=1}^n W_i h_i)} \quad (4.3)$$

where F_x represents the lateral seismic force at floor x ; n is the total number of floors of the structure; W_i and W_x represent the portion of W which is located at or is assigned to level i or x respectively; h_i and h_x are the height above the base ($i=0$) to level i or x respectively, where the base of the structure is that level at which horizontal earthquake motions are considered to be imparted to the structure.

The equivalent earthquake lateral forces distributed over the frame height in accordance with the NBCC 1995 lateral force distribution formula for the 5-, 10- and 20-storey frames are shown in Figures 4.1, 4.2 and 4.3, respectively.

4.4 Results of Static Analysis

In this section, the three designed SMRF frames designed to resist the earthquake loads specified in the NBCC 95 are analyzed. The response parameters of storey drift, yield resistance, ultimate resistance, overall displacement ductility and inter-storey displacement ductility are evaluated.

Figures 4.4, 4.5, and 4.6 show the inter-storey drifts obtained from the static analysis by the code-specified equivalent static lateral loads multiplying the resulting

lateral deflection by force modification factor R to give the real estimates of the anticipated deflections during inelastic response of the structure. The code limits of $0.02 h_s$ for the 5-, 10-, and 20-storey frames are also shown in these figures for comparison, where h_s is inter-storey height. From these figures, it is clearly shown that all the storey drifts are within the code limit, and there is no big difference in storey drift between adjacent floors, which indicates that the structural stiffness is well distributed over the building height and the building is well designed.

The base shear versus roof displacement curves for the 5-, 10- and 20-storey frames are shown in Figures 4.7, 4.8, and 4.9, respectively. The points corresponding to the first beam and first column hinge formations are also shown in the figures. In order to compare the overall lateral strength of the frame relative to the seismic design force, and to give an indication of the overall stiffness of the frame in the post-elastic range, the design base shear and the two points corresponding to the overall drifts of 1.0% and 2.0% are also shown in the figures.

The base shear roof displacement response curve is nearly linear up to the formation of the first column hinge. Thereafter, the overall stiffness of the frame decreases. For all the cases considered, the actual lateral strength of the frame is higher than the design base shear. The overstrength factors for the three SMRF structures can be easily determined from Figures 4.7, 4.8 and 4.9. According to Osman et al. (1995), two kinds of overstrength factors can be defined: the local overstrength factor and the global overstrength factor, where the local overstrength factor, $1/U_l$, is defined as the ratio between the base shear corresponding to the formation of the first plastic hinge to the code-specified design base shear; the global overstrength factor, $1/U_g$, is defined as the ratio between the base shear corresponding to the first significant yielding in the structure determined from the base shear-roof deflection relationship by a bilinear curve to the code specified design base shear. The calculated local overstrength factors for the 5-, 10-, and 20-storey frames are 1.12, 1.35 and 1.29, respectively, and the

calculated global overstrength factors for the 5-, 10-, and 20-storey frames are 2.12, 2.07 and 1.81, respectively. The results show that all the global overstrength factors for the three designed SMRF structures are higher than the code-specified value of 1.67 that is applicable to all structural systems with all periods, and the overstrength factors decrease with the increase of the number of storeys. This result agrees well with the results of Osteraas and Krawinkler (1989, 1990). Osteraas and Krawinkler (1989) studied the behaviour of steel frame structures in Mexico City during the 1985 Mexico earthquake and found an overstrength which increases from near 2 for long period structures to about 13 for very short period structures. The global overstrength factors for steel frames (Osteraas and Krawinkler, 1990) range from greater than 3 for a two-storey building to under 2 for tall buildings. The overstrength of the structure is influenced by many factors such as the higher material strength than the nominal values specified in the design, the code limit on storey drift, the minimum available member size, the redistribution of internal forces and the strain hardening effect in the inelastic range.

The observed behaviour on the overall displacement ductilities for the three SMRF structures can also be described and divided into two categories. First, the overall displacement ductility is defined as the ratio between the roof displacement at ultimate state to the roof displacement at first yielding. The ultimate state of the frame is defined as the point where the inter-storey drift exceeds $0.02h_s$, where h_s is inter-storey height. In the second category, the overall displacement ductility is defined as the ratio between the roof displacement at ultimate state to the roof displacement corresponding to the significant yielding of the frame. The results of the first overall displacement ductility for the 5-, 10- and 20-storey frames are 3.89, 3.46 and 3.77, respectively, whereas the second overall displacement ductility for the 5-, 10- and 20-storey frames are 2.18, 2.25 and 2.28, respectively. These calculated ductilities are significantly less than the code-specified value of 4 for the ductile moment resisting

frames, but they correlate well with the results of Osman et al. (1995).

To evaluate the displacement ductility of individual storeys at the ultimate state, where the ultimate state is defined as the formation of a collapse mechanism at a storey or the top displacement reaches a 2% overall drift, the storey ductility is defined as the ratio between the maximum relative storey displacement to the relative storey displacement at first yield. The storey ductility ratios for the 5-, 10- and 20-storey frames are shown in Figures 4.10, 4.11 and 4.12, respectively. The results show that, in general, the storey ductility ratio is lower in the middle storeys of the building, and the ductility ratio is largest in the first storey.

Figures 4.13 to 4.15 show the sequence and location of plastic hinge formation in the 5-, 10-, and 20-storey frames subjected to monotonically increased lateral loading. The open circles denote plastic hinges formed in the beams, whereas the solid circles represent plastic hinges formed in the columns. The number besides each circle indicates the order of formation in the sequence. The hinge pattern corresponds to an overall drift of 2% in the studied frame. The results show that in general plastic hinges form in the beam first. After a number of plastic hinges are formed in the beams, plastic hinges start to form at the base of the first storey columns due to the large overturning moment caused by the lateral loading and the assumption of perfect fixity at the frame base. Further increase in the lateral loading results in more plastic hinges formed in the beams and columns. For the 5-storey frame, the first three plastic hinges are all formed in the beams of the first storey. This is related to the significantly higher ductility demand on the first storey in comparison to that on other floors of the 5-storey building, as presented in Figure 4.10. For the 10-storey frame, the first plastic hinge forms at a first storey beam, and then the subsequent hinges occur at higher storeys. For the 20-storey frame, the first plastic hinge occurs at a middle storey beam followed by additional hinges at the beams of lower storeys. This behaviour is due to the higher contribution of higher vibration modes to the

dynamic response. In comparison to the pattern of the column plastic hinges, they are concentrated on the first, middle and top storeys for the three SMRF structures. In the next chapter, the actual dynamic behaviour of the three SMRF structures subjected to appropriate recorded ground motions are analyzed and compared to the static push-over analysis results.

Table 4.1: Vibration periods

No. of storeys	Periods (second)				
	1st	2nd	3rd	4th	5th
5	1.367	0.513	0.304	0.194	0.138
10	2.194	0.810	0.479	0.330	0.247
20	3.656	1.311	0.785	0.555	0.424

Figure 4.1: The distributed lateral forces over the frame height of the 5-storey steel frames

Figure 4.2: The distributed lateral forces over the frame height of the 10-storey steel frames

Figure 4.3: The distributed lateral forces over the frame height of the 20-storey steel frames

Figure 4.4: Storey drifts computed from code procedure for the 5-storey steel frame building

Figure 4.5: Storey drifts computed from code procedure for the 10-storey steel frame building

Figure 4.6: Storey drifts computed from code procedure for the 20-storey steel frame building

Figure 4.7: Base shear versus roof displacement responses of the 5-storey steel frames subjected to monotonically increasing lateral loading

Figure 4.8: Base shear versus roof displacement responses of the 10-storey steel frames subjected to monotonically increasing lateral loading

Figure 4.9: Base shear versus roof displacement responses of the 20-storey steel frames subjected to monotonically increasing lateral loading

Figure 4.10: Storey ductility ratios for the 5-storey steel frame building from monotonically increasing lateral loading

Figure 4.11: Storey ductility ratios for the 10-storey steel frame building from monotonically increasing lateral loading

Figure 4.12: Storey ductility ratios for the 20-storey steel frame building from monotonically increasing lateral loading

Figure 4.13: Sequence and location of plastic hinge formation for the 5-storey steel frame resulting from monotonically increasing lateral loading

Figure 4.14: Sequence and location of plastic hinge formation for the 10-storey steel frame resulting from monotonically increasing lateral loading

Figure 4.15: Sequence and location of plastic hinge formation for the 20-storey steel frame resulting from monotonically increasing lateral loading

Chapter 5

Dynamic Analysis

5.1 General

In the previous chapter, the inelastic behaviour and performance of the three SMRF structures subjected to monotonically increased lateral loading were presented. The objective of the present chapter is to evaluate the seismic performance of the three SMRF structures under simultaneous horizontal and vertical ground motion excitations. This chapter first describes the analysis procedure, then selection of the ground motions, and finally the detailed analysis results under the horizontal and vertical ground motions. The response parameters such as the damage pattern, vibration periods of undamaged and damaged structures, the lateral displacements, storey drifts, member forces and storey forces are discussed at the end of the chapter.

5.2 Analysis Procedure

The analysis models of the 5-, 10-, and 20-storey SMRF structures used in this study are shown in Figures 3.1 to 3.4. The buildings are designed to resist gravity and

earthquake forces in the N-S direction according to the provisions of NBCC 95 and the standard CAN/CSA-S16.1-94 "Limit States Design of Steel Structures." The load combination of $1.0D + 0.5L + 1.0E$ is used for the horizontal, and horizontal plus vertical ground motion analyses in the horizontal direction. The load combination of $1.0D + 1.0L + 1.0E$ is used for the simultaneous horizontal and vertical ground motion analysis in the vertical direction. This is because, in reality, the live loads in the buildings are likely to slide on the floor when subjected to severe horizontal ground motions especially in the inelastic range. Therefore, in the analysis of the three designed SMRF structures under horizontal ground motions, only one half of the live load is included in the analysis. But in the analysis with multi-directional earthquake inputs with both horizontal and vertical ground motion components, the whole live loads are considered by assuming that they are attached to the floors in the vertical direction. The inertial effects from the other half of the live loads are ignored in the horizontal direction, but this is consistent with the load combination specified by the NBCC-95 in the seismic analysis under the horizontal direction. In the present analysis, the live load is considered with a reduction factor of 0.93 based on the provisions of NBCC 95. The live load is taken as 2.4 KN/m^2 according to the provisions for office buildings of NBCC 95. A typical interior frame in the N-S direction of each building is analyzed subjected to earthquake excitations. The computer program DRAIN-2DX (Prakash et al., 1993) is used to perform elastic and inelastic dynamic analyses.

In the dynamic analysis, the damping property of the structure is assumed to be of the Rayleigh type. The Rayleigh damping can be expressed as a linear combination of the mass and stiffness matrices. The combination coefficients are selected to give 5% of the critical damping in the first two modes of vibration for the 5- and 10-storey SMRF structures. For the 20-storey SMRF structure, the higher modes' contribution to the dynamic response of the structures is important, therefore the coefficients are selected

to give 5% of the critical damping in the first and fifth modes of vibration. The other modelling parameters, such as the moment-rotation relationship, the moment-axial force interaction and the P-delta effect, have the same characteristics as discussed in the static push-over analysis. It is noted here that the effect of soil-structure interaction is not considered in the earthquake excitation analysis.

5.3 Selection of Ground Motions

The seismic zoning maps of Canada specified in the NBCC (1995) are characterized by two ground motion parameters, the peak horizontal ground acceleration, a , and the peak horizontal ground velocity, v , both with a probability of exceedance of 10% in 50 years. In a seismic response analysis of building structures, the response and behaviour of the analyzed structures are affected by the characteristics of the input earthquake ground motion. To understand the seismic response characteristics of building structures, realistic ground motions with different frequency characteristics should be used to assess the dynamic behaviour and performance of structures.

In this study, three sets of strong ground motion records from three recent earthquakes are used as input. The first set is obtained from the Saguenay earthquake of magnitude $M_s = 5.7$ (Mitchell et al., 1990), which occurred in the province of Quebec, Canada on November 25, 1988. Ten records were recorded on rock sites during this earthquake at an epicentral distance between 36 to 177 km. The peak horizontal acceleration ranges from 2.14 cm/s^2 to 170.6 cm/s^2 , and the peak vertical acceleration ranges from 19.49 cm/s^2 to 229.9 cm/s^2 . The maximum recorded vertical acceleration occurs at Les Eboulements. For the velocity record, the peak horizontal velocity ranges from 0.136 cm/s to 5.431 cm/s . The records have relatively high a/v ratios ranging from 1.57 to 9.5. The 1988 Saguenay earthquake contains relatively high energy in the frequency range of relatively stiff structural systems (with periods

smaller than 0.3 seconds) such as low-rise buildings (Leger and Romano, 1992). Although the Saguenay earthquake did not cause extensive damage, it has raised serious concerns for the seismic performance of existing and new low-rise structures (Tso and Zhu, 1991).

In this analysis, the records from Chicoutimi-Nord and Les Eboulements are selected for the seismic response analysis of the present study. Chicoutimi-Nord is the nearest station to the earthquake epicentre, and the record of Les Eboulements has the maximum recorded vertical acceleration. Both stations are located on bedrock.

The second set of ground motion input is obtained from the Northridge, California earthquake of magnitude $M_w = 6.9$ on January 17, 1994 (Heidebrecht, 1995). In this earthquake, the peak horizontal acceleration ranges from 139.2 cm/s^2 to 866.0 cm/s^2 , while the peak vertical acceleration ranges from 56.0 cm/s^2 to 541.4 cm/s^2 . The maximum recorded vertical acceleration occurs at Arleta where the recorded vertical acceleration is higher than the recorded horizontal acceleration. The peak horizontal ground velocity ranges from 2.79 cm/s to 128.9 cm/s . The records have intermediate a/v ratios ranging from 0.61 to 3.09.

In this analysis, the records from Newhall and Santa Monica are selected for the seismic response analysis. The record at Newhall has the lowest a/v ratio, whereas record at the Santa Monica has the maximum recorded horizontal acceleration. The peak horizontal velocity is 41.751 cm/s , which is nearly the same as the design value of $v = 0.4$ for the three SMRF structures. Both stations are located on bedrock.

The third set is obtained from the Kobe, Japan earthquake of a Richter magnitude of 7.2 and a moment magnitude of 6.8 on January 17, 1995 (Cherry, 1996; Byrne et al., 1996). The earthquake epicentre is located just west of downtown Kobe at a depth of 14 km (Architectural Institute of Japan 1995). High accelerations in the range of 0.4 g to 0.8 g are recorded in the epicentral region. Peak ground velocities are generally in the range of 35 to 55 cm/s in the heavily damaged region near Kobe.

Such high accelerations are to be expected at a close distance to the epicenter for earthquakes of magnitude $M = 7$. Because of the high population and infrastructure density, the high accelerations caused catastrophic damage to buildings, highways, railways, and port facilities resulting in about 27,000 injuries, and more than 5000 deaths.

In the present study, the records from Kobe university and Honzan, Kobe are selected for the seismic response analysis. The record of the Kobe university recorded on rock has the maximum recorded vertical acceleration. The record of Honzan has the peak velocity both in horizontal and vertical directions of 40 cm/s, which is the same as the design value of $v = 0.4$ for the three SMRF structures. This ground motion was recorded at a diluvial deposit.

In this study, since the zonal velocity ratio, v , is taken as 0.4 in the design load calculation for all the three SMRF structures, the horizontal input ground motion records except the Saguenay earthquake are scaled to a peak horizontal ground velocity of 0.4 m/s, which corresponds to the requirement of sites located in zone 6 of NBCC 1995 seismic map. The vertical ground motion records are scaled by the same scale factors as the horizontal ones. The Saguenay earthquake records have a very high frequency content, which is more detrimental to the dynamic response of stiffer structures with shorter vibration periods. The peak horizontal velocity ranges from 0.136 cm/s to 5.431 cm/s, which is very low. As the design of stiff structures falls within the acceleration controlled region of the earthquake response spectra, the Saguenay earthquake records are scaled to a peak horizontal ground acceleration of 0.4 g. It is obvious that the key assumption of this scaling procedure is that the frequency content and the duration of the ground motion are independent of the shaking intensity. This assumption may be questionable especially when soft soil layers are present, because the intensity of the ground shaking will certainly affect the severity of the nonlinear response of the foundation material and thus modify the expected

free-field motion characteristics. The peak acceleration, the peak velocity and the a/v ratio of the six earthquake records from the three selected earthquakes are shown in Table 5.1. The selected earthquake records cover a wide range of a/v ratios from the very low a/v ratio of 0.76 of the Newhall, Northridge earthquake to the very high a/v ratio of 6.91 of the Chicoutimi-Nord, Saguenay earthquake. The selected horizontal and vertical acceleration time-histories from the Saguenay, Northridge and Kobe earthquakes are shown in Figures 5.1 to 5.6. The calculated horizontal and vertical acceleration response spectra of the three earthquakes, which corresponds to a peak horizontal ground velocity of 0.4 m/s or a peak horizontal ground acceleration of 0.4 g, are shown in Figures 5.7 to 5.12. It is noted here that the vertical acceleration response spectra is calculated by applying the vertical ground motion in the vertical direction. It is evident from Figures 5.7 to 5.12 that the earthquake ground motions selected for this study cover a wide range of spectral response, from the very high frequency content of the Saguenay earthquake to the longer period Kobe earthquake. The spectral analysis of the selected earthquake records clearly indicate that the high a/v ground motions have higher energy content in the high frequency range, whereas the low a/v ground motions have higher seismic energy in the low frequency range.

5.4 Numerical Results of Horizontal, Horizontal and Vertical Earthquake Excitations

In this section, a detailed comparison of the seismic response of the three SMRF structures under the horizontal, and horizontal plus vertical ground motion excitations is presented. It is noted here that the typical structural degradation after repeated excursions into the inelastic range is not considered in the seismic response analyses of the three designed SMRF structures. Furthermore the torsional effect of the stud-

ied buildings introduced by the inelastic deformations after yielding of the structural members is not considered in the present study.

5.4.1 Damage Patterns

To assess the effect of the frequency content of the ground motion on the inelastic seismic response of the SMRF structures, the sequence and location of the plastic hinge formation of the three SMRF structures are investigated. Figures 5.13 to 5.31 show the sequence and location of the plastic hinge formation of the three SMRF structures under horizontal, and horizontal plus vertical ground motion excitations. The open circles denote the plastic hinges formed in the beams, whereas the solid circles represent the plastic hinges formed in the columns. The number besides each circle indicates the order in the formation sequence. The analysis results show that there is no plastic hinge formed in the 5-, 10-, and 20-storey frames under the Saguenay earthquake excitations. This is not surprising because there is almost no seismic energy in the period range greater than 1 second of the analyzed buildings from the scaled acceleration response spectra of the Saguenay earthquake presented in Figures 5.7 and 5.8.

The sequence and location of plastic hinge formation of the 5-storey frame resulting from the Northridge earthquake excitations are shown in Figures 5.13 and 5.14, and from the Kobe earthquake excitations, in Figures 5.15 and 5.16. For the 5-storey frame subjected to the Northridge horizontal earthquake motions, the plastic hinge begins to form in the first storey at 5.18 seconds for the Newhall record and 13.36 seconds for the Santa Monica earthquake record. In comparison, under the horizontal and vertical ground motions, there are more plastic hinges and they form earlier at 4.42 seconds and 12.66 seconds respectively for the two earthquake records. It is also evident that the sequence and location of the plastic hinge formation can be affected by the vertical ground motion component. Under the Santa Monica earthquake mo-

tions after the initial plastic hinge in a first storey beam, additional plastic hinges occur at upper storeys of the frame. The reason that more plastic hinges formed under the Santa Monica earthquake excitations is because it contains higher seismic energy than the Newhall earthquake record in the fundamental period range of 1.367 seconds of the 5-storey frame. In comparison with the static push-over analysis results, the dynamic results from the Santa Monica earthquake records on the damage pattern of the plastic hinges in the beams on the first, second and fourth storeys of the 5-storey SMRF structure correlate well with the static push-over results.

When subjected to the Kobe horizontal earthquake records, which have a relatively low frequency content, plastic hinges initially occur in the beams of the first storey and then further develop at the upper storey beams of the frame. Different from records of the Northridge earthquake, there are some plastic hinges formed at the base of the first storey columns. This is because as the Kobe earthquake record contains higher seismic energy than the Northridge earthquake record in the fundamental period range of 1.367 seconds of the 5-storey frame, the ductility demand at the first storey of the frame during the Kobe earthquake is also higher, as can be observed from the results shown in Figure 4.10. In comparison, under the horizontal and vertical ground motion excitations, similar behaviour in terms of the earlier formation of plastic hinges can also be observed in the 5-storey frame. Plastic hinges in the column occurring on the first and top storeys of the frame are due to the relatively high ductility demand at the top and bottom storeys as shown in Figure 4.10. The analysis results obtained here generally agree with the extensive damage suffered by many low-rise steel structures during the Kobe earthquake.

The sequence and location of the plastic hinge formation for the 10-storey frame resulting from the Northridge and Kobe earthquakes are shown in Figures 5.17 to 5.24. For the 10-storey frame subjected to the Newhall, Northridge horizontal earthquake input, plastic hinges develop only in the beams in the upper storeys of the frame.

This is due to the fact that higher mode contributions to the overall response of the system is more significant in the 10-storey building than in a low-rise structure. In comparison, under the horizontal and vertical ground motion excitations, plastic hinges develop in the beams at the first storey, and then additional plastic hinges occur at the beams in the upper storeys of the frame. Similar behaviour in terms of the earlier formation of plastic hinges in the beams of the first storey in this 10-storey frame is due to the same reason as discussed in the case of the 5-storey frame when subjected to the Kobe earthquake excitation. On the other hand, using the Santa Monica, Northridge horizontal earthquake record as input, the first plastic hinge develops in the middle storey beam. After a number of additional plastic hinges form in the beams in the upper and lower floors, plastic hinges start to form at the base of the first storey columns and then continue to form in the upper storey beams and columns. In comparison, under the horizontal and vertical ground motion excitations, the first three plastic hinges occur in the first storey beams. Similar behaviour in terms of the earlier formation of plastic hinges can also be observed in the 10-storey frame. In comparison with the static push-over analysis results, the locations of plastic hinges in the beams and columns during the Santa Monica earthquake correlate very well with the results of static push-over analysis by comparing the damage pattern shown in Figure 4.14 with that in Figures 5.19 and 5.20. Another interesting observation for the 10-storey frame during the Northridge earthquake is that, with the vertical ground motion included, the first two plastic hinges are formed in the first storey beams instead of forming in the upper storey beams as in the case of the horizontal ground motion excitation only. This behaviour needs further investigation.

Under the Kobe university horizontal earthquake motion, the plastic hinges develop in the upper storey beams at 3.73 seconds. The plastic hinges in the column are located only at the base of the first-storey columns. In comparison, under the horizontal and vertical ground motion excitations, the plastic hinges occur in the

bottom storey beams at 3.43 seconds. After a number of plastic hinges have formed in the beams, plastic hinges start to form at the base of the first storey columns and then continue to form in the beams and columns at other floors.

When subjected to the Honzan horizontal earthquake record, which has an intermediate frequency content, the first plastic hinge forms in the first storey beams. After a couple of plastic hinges have formed in the first storey beams, plastic hinges start to form at the base of the first storey columns and then additional plastic hinges occur at the upper storey beams and columns. In comparison, under the horizontal and vertical ground motions, similar behaviour can also be observed with the horizontal earthquake excitation. The earlier formation of plastic hinges at the bottom of the first storey columns is due to the relatively high seismic energy of this record at the fundamental period of 2.19 seconds of the 10-storey SMRF structure. Thus it causes a large overturning moment at the base of the first storey, and according to the axial force-bending moment relationship, a plastic hinge is formed even though the column axial force is relatively small. As can be seen from Figures 5.23 and 5.24, heavy damage is concentrated at the middle height of the 10-storey building. This damage pattern indicates that collapse of the frame at mid-height is likely to occur, which correlates with much of the observed damage to high-rise buildings during the Kobe earthquake. It is noted here that although the extent of damage under the Honzan record of the Kobe earthquake excitation is more severe than the results of the static push-over analysis, the top displacement under this earthquake record excitation is far less than that of the static push-over analysis results.

For the 10-storey frame, it is obvious that both the results of the static push-over and seismic response analyses of the Northridge and Kobe earthquake records indicate that the beams in the 1st, and 6th to 8th storeys are damaged earlier than the beams in other storeys by examining the results of the earlier formation of plastic hinges in the beams in those storeys, as shown in Figure 4.14 and Figures 5.17 to 5.24.

The sequence and location of the plastic hinge formation for the 20-storey frame resulting from the Northridge and Kobe earthquakes are shown in Figures 5.25 to 5.31. For the 20-storey frame, using the Newhall horizontal earthquake record as input, there is no plastic hinge formed because the Newhall earthquake record has little seismic energy around the fundamental period of 3.66 seconds of the 20-storey frame. In comparison, under the horizontal and vertical ground motion excitations, only two plastic hinges develop at the base of the first storey columns. These two plastic hinges were formed because the Newhall earthquake record has a relatively high vertical acceleration, thus it causes a high axial force in the bottom storey column. According to the axial force-bending moment interaction relationship, a plastic hinge is formed even though the column moment is relatively small. When subjected to the Santa Monica, Northridge horizontal earthquake record, there is only one plastic hinge developed in the tenth storey beam. In comparison, under the horizontal and vertical ground motion excitations, there are more plastic hinges formed in the beams of middle storeys, similar to the Northridge earthquake record. This is also because the Santa Monica earthquake record has little energy over a fundamental period of 3.66 seconds of the 20-storey frame.

Using the Kobe university horizontal earthquake record as input, the first plastic hinge develops in the beam of middle storeys and then additional plastic hinges form at the beams of upper and lower storeys of the 20-storey frame. In comparison, under the horizontal and vertical ground motion excitations, the first two plastic hinges occur at the base of the first storey columns, and then additional plastic hinges continue to form at beams throughout the whole frame. The earlier formation of column hinges at the base of the first storey columns is due to the large axial force caused by the vertical ground acceleration and gravity load, and the large overturning moment caused by the horizontal earthquake. The high concentration of plastic hinges in the lower storeys under the Kobe university earthquake excitations is due to the

relatively high ductility demand at the lower storeys of the 20-storey frame as shown in Figure 4.12.

When subjected to the Honzan horizontal earthquake record as input, the first plastic hinge develops in the medium storey beam, and then additional plastic hinges form at the upper and lower storey beams and columns. In comparison, under the horizontal and vertical ground motion excitations, the first couple of plastic hinges develop at the base of the first storey columns, then additional plastic hinges occur at the beams throughout the frame. The plastic hinges are concentrated on the lower and upper storeys under the Honzan earthquake excitations. This is due to the higher mode contribution. Compared to the Northridge earthquake, the Kobe earthquake would have caused more damage to the studied 20-storey frame. The earlier formation of plastic hinges at the base of first storey column of the 20-storey frame can explain why so many of the new designed high-rise steel buildings suffer dramatic damage at the base of first storey columns. Similar to the 5- and 10-storey frames, the behaviour of the 20-storey frame in terms of earlier formation of plastic hinges is also observed. In comparison with the results of static push-over analysis of the 20-storey frame, only the result from the Kobe university earthquake excitation bears little relationship with the result of static push-over analysis corresponding to the concentration of plastic hinges at lower storeys.

The general observations from this section can be summarized as follows:

(1) The low a/v ratio horizontal and vertical ground motions are more damaging to the 5-, 10-, and 20-storey SMRF frames than high a/v ratio horizontal and vertical ground motions.

(2) Under the horizontal and vertical ground motion excitations, there are more plastic hinges formed in the beams and columns, the sequence and location of plastic hinge formation can be greatly influenced, and the formation of the first plastic hinge occurs earlier in comparison with the results of horizontal earthquake excitation only.

(3) For the 20-storey SMRF building, the vertical ground motion can easily cause the base storey columns to develop a plastic hinge first. This is due to the axial force-bending moment interaction effect. That is, a large axial force requires a relatively small bending moment to cause the column to yield. This issue should be emphasized in the design of new high-rise steel buildings in the future edition of the NBCC.

(4) In comparison to the damage patterns of static push-over analysis, to some extent, the results of seismic analysis correlate well with those of static push-over analysis for the 5- and 10-storey frames under some earthquake excitations. For the 20-storey frame, the sequence and/or location of plastic hinge formation of seismic analysis are different compared to that of static push-over analysis. This is because all the selected earthquake records contain little seismic energy around the fundamental period of 3.66 seconds of the 20-storey frame.

5.4.2 Vibration Periods

In order to examine the effect of the damage on the dynamic behaviour of the three SMRF structures, the variation of the vibration periods of the three SMRF structures are determined under the Northridge and Kobe earthquake excitations. Because all three SMRF structures remain within the elastic range throughout the response under the Saguenay earthquake excitations, this means that the vibration periods of the structures do not change. Therefore, the vibration period time histories of the three SMRF structures under the Saguenay earthquake are omitted. Figures 5.32 to 5.37 show the vibration period time histories of the three SMRF structures under the Northridge and Kobe earthquake excitations. The solid line represents the vibration period of the building considering only horizontal ground motion, whereas the dotted line denotes the vibration period for simultaneous horizontal and vertical ground motion excitations.

The vibration period of the first three modes of the 5-storey frame when subjected

to the Santa Monica earthquake record of the 1994 Northridge earthquake are shown in Figure 5.32. From this figure, it is evident that the vibration period of the damaged structure changes greatly from 13.36 seconds to 14.56 seconds under the horizontal ground motion excitation, and from 12.66 seconds to 14.62 seconds under the horizontal and vertical ground motion excitations, which correspond to the time intervals during which all the plastic hinges are formed. This coincides with the time intervals of strong ground shaking for the corresponding earthquake record, as can be seen by examining the acceleration time histories of the ground motion record shown in Figure 5.4. Figure 5.32 shows that the vertical ground motion causes the vibration period of the structure to change earlier and more extensively than the horizontal ground motion only. This means that the vertical ground motion can cause the structure to be damaged earlier and more severely than the horizontal ground motion.

The vibration period of the first three modes of the 5-storey frame resulting from the Honzan record of the 1995 Kobe earthquake are shown in Figure 5.33. Similar behaviour as the Santa Monica earthquake record is observed.

The vibration periods of the first three modes of the 10- and 20-storey frames resulting from the Northridge and Kobe earthquakes are shown in Figures 5.34 to 5.37. Similar behaviour as the 5-storey frame is observed.

It is noted here that, during and after the strong motion period, the vibration periods of the damaged structures wave and back to the vibration periods of the undamaged structure. This is because the model of plastic hinge beam-column element of the DRAIN-2DX computer program is perfectly elasto-plastic. Another thing to be mentioned is that there is a small difference between the vibration periods under horizontal, and horizontal plus vertical ground motion excitations. This is caused by the different masses considered between horizontal, and horizontal plus vertical ground motion excitations in the vertical direction.

The observations from this section can be summarized as follows:

(1) The vertical ground motion can increase the vibration periods of the damaged structures at a specific time. This means that the vertical ground motion causes more damage to the structure than horizontal ground motion only.

(2) The vertical ground motion can cause the structure to be damaged earlier and longer than horizontal ground motion.

(3) The change rate of the fundamental vibration period of the structure under the Kobe earthquake record is greater than that under the Northridge earthquake record. This is because the Kobe earthquake record causes more damage than the Northridge record.

5.4.3 Lateral Displacements

In an earlier section, the damage patterns of the three SMRF buildings under the horizontal, and horizontal plus vertical ground motion excitations have been discussed. The main purpose of this study is to investigate the inelastic behaviour and performance of the SMRF structures under horizontal, and horizontal plus vertical ground motion excitations. The three SMRF structures remain in the linear elastic range under the Saguenay earthquake excitations because no plastic hinge is formed. Therefore, in this section, the comparison of lateral displacements considers only the Northridge and Kobe earthquake records. However, in order to give a picture of the lateral displacement behaviour under the Saguenay earthquake excitations, the maximum top displacements of the 5-, 10- and 20-storey frames resulting from the Saguenay earthquake are shown in Table 5.2. As can be seen from this Table, the top displacement of the 5-, 10- and 20-storey frames under the horizontal, and horizontal plus vertical, ground motion excitations of the Saguenay earthquake are exactly the same.

The lateral displacement time histories for the 5-storey frame resulting from the Northridge earthquake record are shown in Figures 5.38 and 5.39. The solid line

denotes the lateral displacement of horizontal ground motion excitation, whereas the dotted line represents the lateral displacement of horizontal and vertical ground motion excitations. For the 5-storey steel frame subjected to the Newhall earthquake input, within the limit of linear elastic deformation, the lateral displacements at the top, third, and first floor levels are almost exactly the same under the horizontal, and the horizontal plus vertical, ground motion excitations. This result agrees well with the results obtained by Ariaratnam and Leung (1990), which show that within the limit of linear elastic deformation the vertical ground acceleration increases the lateral displacement of a moderately tall building by 0.08%. In the inelastic range beginning at 5.18 seconds for the Newhall horizontal earthquake excitation, and 4.42 seconds for the Newhall horizontal and vertical earthquake excitations, there is a noticeable increment of lateral displacements at different floor levels under the effect of vertical ground motion. This is because under the Newhall earthquake there are only a couple of plastic hinges formed at the beams of the 5-storey frame, which means that the frame behaves mainly in the linear elastic range. Figure 5.38 shows that the lateral displacements at different floor levels have a big increment from 4.5 to 6.5 seconds. This is because the scaled Newhall record has a strong ground shaking around 3 seconds, as can be seen by examining the acceleration time history of the ground motion record shown in Figure 5.3. The maximum top displacement under the Newhall earthquake record is 0.1 meter, which is less than 0.5% of the building height. Therefore, after the strong ground shaking period, almost no permanent deformation results after the earthquake. Under the Santa Monica earthquake record excitations, similar behaviour as in the previous case is observed. Because there are more plastic hinges formed at the beams of the 5-storey frame during this earthquake than the Newhall earthquake, the lateral displacements at different storey levels under the Santa Monica earthquake excitations are larger than these under the Newhall earthquake excitation. But for the lateral displacement, even in the inelastic range,

there is no obvious difference in the behaviour between the horizontal, and horizontal plus vertical, ground motion excitations. This is because first, the plastic hinge beam-column element of the computer program DRAIN-2DX uses a perfectly elasto-plastic model. Second, the more damaged structure under horizontal and vertical ground motion excitations (with a larger vibration period) attracts less base shear, resulting in similar deflections in comparison to the less damaged structure under horizontal earthquake excitation (with a smaller vibration period).

The lateral displacement time histories for the 5-storey frame resulting from the Kobe earthquake records are shown in Figures 5.40 and 5.41. For the 5-storey steel frame, within the limit of linear elastic deformation, similar behaviour as in the Northridge earthquake is observed. In the inelastic range, the lateral displacements at different storey levels under the Kobe earthquake excitations are larger than these under the Northridge earthquake excitations. The maximum top displacement that occurred during the Honzan earthquake record reaches 0.23 meter, which is greater than 1% of the building height. This is because under the Kobe earthquake excitations there are more plastic hinges formed in the beams and columns than under the Northridge earthquake excitations, which means that the 5-storey frame suffers relatively heavier damage during this earthquake. Figures 5.40 and 5.41 also show that there are permanent plastic deformations left after the strong shaking period of the ground motions. The maximum permanent plastic deformation at the top of the 5-storey frame reaches 5 centimeters under the Honzan earthquake record excitations, which correlates well with the more severe damage pattern of the 5-storey frame when subjected to the Honzan earthquake record of the 1995 Kobe earthquake.

The lateral displacement time histories for the 10-storey frame resulting from the Northridge earthquake records are shown in Figures 5.42 and 5.43. For the 10-storey steel frame, within the limit of linear elastic deformation, similar behaviours as in the 5-storey frame are observed. In the inelastic range, it is obvious that the lateral

permanent plastic deformation at the top of the 10-storey frame under horizontal and vertical ground motion excitations of the Newhall earthquake record is larger than that under the horizontal ground motion excitation of the Newhall earthquake record. Also, in terms of the maximum top lateral displacement of the 10-storey frame under the Newhall earthquake excitations, similar behaviour is observed as in the case of the 5-storey frame. Under the Santa Monica earthquake record excitations, similar to the case of the 5-storey frame, the maximum top lateral displacement of the 10-storey frame reaches 1% of the total height of the building.

The lateral displacement time histories for the 10-storey frame resulting from the Kobe earthquake records are shown in Figures 5.44 and 5.45. For the 10-storey steel frame, using the Kobe university earthquake records as input, both in the elastic and inelastic ranges, similar behaviour as in the Santa Monica earthquake excitations is observed. It is noted here that the difference of lateral displacements between the horizontal, and horizontal plus vertical, ground motion excitations is larger at the fifth floor level. This is because one column was damaged at this floor level under horizontal and vertical ground motion excitations. When subjected to the Honzan earthquake excitations, both in the elastic and inelastic ranges, similar behaviour as in the previous case is observed. As can be seen from Figure 5.45, the maximum lateral displacement at the top of the 10-storey frame under the Honzan earthquake record reaches 1.4% of the total height of the 10-storey building. This is because this earthquake record contains relatively high seismic energy in the fundamental period of 2.19 seconds of the 10-storey frame, thus causing more severe damage than other earthquake records.

The lateral displacement time histories for the 20-storey frame resulting from the Northridge and Kobe earthquake records are shown in Figures 5.46 to 5.49. For the 20-storey steel frame, both in the elastic and inelastic ranges, similar behaviour as in the 5- and 10-storey frames is observed. Figures 5.46 to 5.49 show that, even in

the inelastic range, the maximum top displacements of the 20-storey frame under the Northridge earthquake and the Honzan earthquake record of the Kobe earthquake excitations are less than those of the 10-storey frame under the same earthquake excitations. This is because all the earthquake records have little seismic energy over the fundamental period of 3.66 seconds of the 20-storey frame, thus causing little damage for the 20-storey frame, and resulting in relatively small top displacement.

The observations from this section can be summarized as follows:

(1) Within the limit of linear elastic deformation, the lateral displacements at the top, middle, and 1st floor levels are almost the same for the three SMRF structures under horizontal, and horizontal plus vertical, ground motion excitations. In the inelastic range, the lateral displacements at different floor levels could be increased under the horizontal and vertical ground motion excitations, but it is within a limit of 5% corresponding to the lateral displacements under the horizontal ground motion excitation only.

(2) The more damage to the structure, the bigger the top displacement of the structure.

(3) The lower a/v ratio ground motions cause large lateral displacements for the three SMRF structures.

(4) The magnitude of the maximum lateral displacement of the structure depends on the amount of seismic energy of the earthquake record at the fundamental period of the structure. That is, the more seismic energy of the earthquake record contained in the fundamental period of the structure, the larger the lateral displacement of the structure.

5.4.4 Storey Drifts

In this section, the storey drift time histories of the three SMRF structures are compared under the Santa Monica earthquake record of the Northridge earthquake and

the Honzan earthquake record of the Kobe earthquake.

The storey drift time histories of the three SMRF structures resulting from the Santa Monica earthquake excitations and the Honzan earthquake excitations are shown in Figures 5.50 to 5.55. The solid line represents the result of horizontal ground motion excitation, whereas the dotted line denotes the result of horizontal and vertical ground motion excitations.

For the 5-storey frame, the maximum storey drift occurs at the first storey under the Honzan earthquake record excitation. This is because the damage in the first storey under the Honzan earthquake record excitation is heavier than in the other storeys under the other earthquake excitations, as can be seen by examining the damage pattern of the 5-storey frame shown in Figures 5.13 to 5.16. It is clear from Figure 5.51 that there is a 2.2 centimeters permanent plastic deformation at the first storey level under the Honzan earthquake record excitations. For the 10- and 20-storey frames, the maximum storey drift occurs at the fifth storey of the 10-storey frame under the Honzan earthquake record excitation due to a similar reason. For all three frames, the maximum storey drift occurs at the fifth storey of the 10-storey frame during the Honzan earthquake record excitation, which reaches nearly 2% of the storey height. This means that the collapse of the 10-storey frame is likely to occur according to the limit of maximum storey drift of 2% in the NBCC 95.

The observation from this section is that, within the limit of linear elastic deformation, the storey drifts at the top, middle, and first floor levels are almost exactly the same under horizontal, and horizontal plus vertical, ground motion excitations. In the inelastic range, the storey drifts could be increased under the horizontal and vertical ground motion excitations, but it is within a limit of 5% range corresponding to the storey drifts under the horizontal ground motion excitation only.

5.4.5 Member Forces

In this section, the column shear, bending moment and axial force are compared under the horizontal, and horizontal plus vertical ground motion excitations in each of the Northridge and the Kobe earthquake records. For the 5-storey structure, the column forces of the first storey are compared. For the 10-storey frame, the column forces of the middle storey are compared. For the 20-storey building, the column forces of the top storey are compared.

The member force time histories of the three frames resulting from the Northridge and the Kobe earthquake excitations are shown in Figures 5.56 to 5.61. The solid line represents the result of horizontal ground motion excitation, whereas the dotted line denotes the result of horizontal and vertical ground motion excitations.

The member force time histories of the bottom storey interior column of the 5-storey frame resulting from the Northridge and Kobe earthquake excitations are shown in Figures 5.56 and 5.57. Figures 5.56 and 5.57 show that the vertical ground motion can decrease the column shear and bending moment of the bottom storey column in the inelastic range. This is because the vertical ground motion increases the extent of damage to the structure. This results in a higher vibration period of the structure, since the long vibration period of a structure causes less lateral inertia force in comparison with a structure with a short vibration period. The column axial forces can be greatly increased under the effect of vertical ground motion. It is noted here that the difference of the column axial force is caused by the higher live load considered in the vertical direction than that considered in the horizontal direction between the horizontal, and horizontal plus vertical, ground motion excitations.

The member force time histories of the fifth storey interior column of the 10-storey frame resulting from the Northridge and the Kobe earthquake excitations are shown in Figures 5.58 and 5.59. Similar behaviour as in the 5-storey frame is observed.

The member force time histories of the top storey interior column of the 20-storey

frame resulting from the Northridge earthquake excitations are shown in Figure 5.60. It is clearly indicated that the vertical ground motion can increase the axial force, shear and bending moment of the top storey column in the inelastic range. This behaviour is different from the column behaviour in the 5- and 10-storey frames; it needs further investigation.

The member force time histories of the top storey interior column of the 20-storey frame resulting from the Kobe earthquake excitation are shown in Figure 5.61. Similar behaviour in terms of the column axial force is observed as in the previous cases. The shear and bending moment of the top storey column caused by the horizontal, and horizontal plus vertical, ground motion excitations are almost the same in the inelastic range.

The observations from this section can be summarized as follows:

- (1) In general, the vertical ground motion can decrease the shear and bending moment of lower storey columns, but it can increase the shear and bending moment of top storey columns.
- (2) The vertical ground motion can increase the column axial force for all three buildings.
- (3) The more damage to the structure, the higher the force of the column.

5.4.6 Storey Forces

In this section, the base shear, overturning moment and overall axial forces of the 5-, 10- and 20-storey SMRF structures under the horizontal, and horizontal plus vertical, ground motion excitations are compared in each of the Northridge and Kobe earthquake records. The bottom storey force time histories of the 5-storey frame resulting from the Santa Monica earthquake record and the Honzan earthquake record excitations are shown in Figures 5.62 and 5.63, respectively. The solid line represents the result of horizontal ground motion excitation, whereas the dotted line denotes

the result of horizontal and vertical ground motion excitations. From these figures, it can be seen that the vertical ground motion can decrease the maximum base shear and overturning moment of the damaged structure in the inelastic range. This is due to the similar reason as discussed in the last section. The storey axial forces can be greatly increased under the effect of vertical ground motion.

The bottom storey force time histories of the 10-storey frame resulting from the Santa Monica earthquake record and the Honzan earthquake record excitations are shown in Figures 5.64 and 5.65, respectively. Similar behaviour is observed as in the previous case.

The bottom storey force time histories of the 20-storey frame resulting from the Newhall earthquake record and the Honzan earthquake record excitations are shown in Figures 5.66 and 5.67, respectively. The maximum base shear and overturning moment of the damaged structure under horizontal, and horizontal plus vertical, ground motion excitations are almost same. But the storey axial forces can be greatly increased under the effect of vertical ground motion.

The observations from this section clearly show that:

- (1) The vertical ground motion can decrease base shear and overturning moment for the low-rise and medium high-rise buildings, but it has almost no influence on the maximum base shear and overturning moment for the super high-rise building.
- (2) The vertical ground motion can greatly increase the overall axial force for all three buildings; the increment depends on the maximum vertical acceleration.

5.5 Summary

The seismic responses of the 5-, 10-, and 20-storey SMRF structures subjected to three different earthquake records are investigated. The response parameters such as the damage patterns, vibration periods of undamaged and damaged structures, the

lateral displacements, storey drifts, member forces and storey forces are compared under horizontal, and horizontal plus vertical, ground motion excitations. Based on the above observations, the findings can be summarized as follows:

(1) The low a/v ratio ground motions are more damaging to the three 5-, 10-, and 20-storey SMRF structures than high a/v ratio ground motions even when the frames are designed based on the same peak ground velocity. Second, under the horizontal and vertical ground motions, there are more plastic hinges formed in the beams and columns, the sequence and location of plastic hinge formation can be greatly influenced, and the formation of the first plastic hinge is earlier.

(2) The vertical ground motion can increase the vibration periods of the damaged structures at a specific time. This means that the vertical ground motion causes more damage to the structure than that caused by horizontal ground motion only. Besides, the the vertical ground motion can cause the structure to be damaged earlier and longer than horizontal ground motion only.

(3) Within the limit of linear elastic deformation, the lateral displacements and storey drifts at the top, middle, and 1st floor levels of the three SMRF structures are almost exactly the same under horizontal, and horizontal plus vertical ground motion excitations. In the inelastic range, the lateral displacements and storey drifts at different floor levels of the three SMRF structures could be increased under the horizontal and vertical ground motion excitations, but it is within a limit of 5% corresponding to the lateral displacements or storey drifts under the horizontal ground motion excitation only.

(4) The vertical ground motion can decrease the shear and bending moment of the lower storey columns for the 5-, and 10-storey buildings in the inelastic range, but it can increase the shear and bending moment of the top storey columns for the 20-storey building in the inelastic range. Also, the vertical ground motion can increase the column axial force for all three buildings.

(5) The vertical ground motion can decrease base shear and overturning moment for the 5-, and 10-storey buildings, but it has almost no influence on the maximum base shear and overturning moment for the 20-storey building. Also, the vertical ground motion can increase the overall axial force for all three buildings.

Table 5.1: Peak ground motion parameters of the selected earthquake records

Name	Component	PGA(cm/s^2)	PGV(cm/s)	a/v
Chicoutimi-Nord	H	104.48	1.51	6.91
	V	100.5	1.86	5.42
Les Eboulements	H	100.3	2.65	3.80
	V	229.89	5.01	4.59
Newhall	H	571.62	74.84	0.76
	V	537.35	30.74	1.75
Santa Monica	H	865.97	41.75	2.07
	V	227.67	14.02	1.63
Kobe University	H	305.3	31.0	0.99
	V	446.5	33.2	1.34
Honzan	H	421.0	40.0	1.05
	V	379.3	40.0	0.95

Table 5.2: Maximum top displacement of the Saguenay earthquake records

Number of storeys	Maximum top displacement (cm)			
	Chicoutimi-Nord		Les Eboulements	
	H	H+V	H	H+V
5	0.63	0.63	2.07	2.07
10	0.60	0.60	1.73	1.73
20	0.71	0.71	1.27	1.27

Figure 5.1: Recorded acceleration time-history for the horizontal and vertical components of the 1988 Saguenay earthquake

Figure 5.2: Recorded acceleration time-history for the horizontal and vertical components of the 1988 Saguenay earthquake

Figure 5.3: Recorded acceleration time-history for the horizontal and vertical components of the 1994 Northridge earthquake

Figure 5.4: Recorded acceleration time-history for the horizontal and vertical components of the 1994 Northridge earthquake

Figure 5.5: Recorded acceleration time-history for the horizontal and vertical components of the 1995 Kobe earthquake

Figure 5.6: Recorded acceleration time-history for the horizontal and vertical components of the 1995 Kobe earthquake

Figure 5.7: Scaled response spectra for the horizontal and vertical components of the 1988 Saguenay earthquake (5 percent damping)

Figure 5.8: Scaled response spectra for the horizontal and vertical components of the 1988 Saguenay earthquake (5 percent damping)

Figure 5.9: Scaled response spectra for the horizontal and vertical components of the 1994 Northridge earthquake (5 percent damping)

Figure 5.10: Response spectra for the horizontal and vertical components of the 1994 Northridge earthquake (5 percent damping)

Figure 5.11: Scaled response spectra for the horizontal and vertical components of the 1995 Kobe earthquake (5 percent damping)

Figure 5.12: Response spectra for the horizontal and vertical components of the 1995 Kobe earthquake (5 percent damping)

a) Horizontal

b) Horizontal and vertical

Figure 5.13: Sequence and location of plastic hinge formation for the 5-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Horizontal: b) Horizontal and vertical

a) Horizontal

b) Horizontal and vertical

Figure 5.14: Sequence and location of plastic hinge formation for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Horizontal; b) Horizontal and vertical

a) Horizontal

b) Horizontal and vertical

Figure 5.15: Sequence and location of plastic hinge formation for the 5-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Horizontal: b) Horizontal and vertical

a) Horizontal

b) Horizontal and vertical

Figure 5.16: Sequence and location of plastic hinge formation for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Horizontal; b) Horizontal and vertical

Figure 5.17: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Newhall horizontal record of the 1994 Northridge earthquake

Figure 5.18: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Newhall horizontal and vertical record of the 1994 Northridge earthquake

Figure 5.19: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Santa Monica horizontal record of the 1994 Northridge earthquake

Figure 5.20: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Santa Monica horizontal and vertical record of the 1994 Northridge earthquake

Figure 5.21: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Kobe university horizontal record of the 1995 Kobe earthquake

Figure 5.22: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Kobe university horizontal and vertical record of the 1995 Kobe earthquake

Figure 5.23: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Honzan horizontal record of the 1995 Kobe earthquake

Figure 5.24: Sequence and location of plastic hinge formation for the 10-storey frame resulting from Honzan horizontal and vertical record of the 1995 Kobe earthquake

Figure 5.25: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Newhall horizontal and vertical record of the 1994 Northridge earthquake

Figure 5.26: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Santa Monica horizontal record of the 1994 Northridge earthquake

Figure 5.27: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Santa Monica horizontal and vertical record of the 1994 Northridge earthquake

Figure 5.28: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Kobe university horizontal record of the 1995 Kobe earthquake

Figure 5.29: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Kobe university horizontal and vertical record of the 1995 Kobe earthquake

Figure 5.30: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Honzan horizontal record of the 1995 Kobe earthquake

Figure 5.31: Sequence and location of plastic hinge formation for the 20-storey frame resulting from Honzan horizontal and vertical record of the 1995 Kobe earthquake

Figure 5.32: Period time history of the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode

Figure 5.33: Period time history of the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode

Figure 5.34: Period time history of the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode

Figure 5.35: Period time history of the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode

Figure 5.36: Period time history of the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode

Figure 5.37: Period time history of the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) 1st mode; b) 2nd mode; c) 3rd mode

Figure 5.38: Lateral displacement time history for the 5-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Top storey; b) 3rd storey; c) 1st storey

Figure 5.39: Lateral displacement time history for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 3rd storey; c) 1st storey

Figure 5.40: Lateral displacement time history for the 5-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Top storey; b) 3rd storey; c) 1st storey

Figure 5.41: Lateral displacement time history for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 3rd storey; c) 1st storey

Figure 5.42: Lateral displacement time history for the 10-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Top storey; b) 5th storey; c) 1st storey

Figure 5.43: Lateral displacement time history for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 5th storey; c) 1st storey

Figure 5.44: Lateral displacement time history for the 10-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Top storey; b) 5th storey; c) 1st storey

Figure 5.45: Lateral displacement time history for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 5th storey; c) 1st storey

Figure 5.46: Lateral displacement time history for the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake: a) Top storey; b) 10th storey; c) 1st storey

Figure 5.47: Lateral displacement time history for the 20-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 10th storey; c) 1st storey

Figure 5.48: Lateral displacement time history for the 20-storey frame resulting from Kobe university record of the 1995 Kobe earthquake: a) Top storey; b) 10th storey; c) 1st storey

Figure 5.49: Lateral displacement time history for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 10th storey; c) 1st storey

Figure 5.50: Storey drift time history for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 3rd storey; c) 1st storey

Figure 5.51: Storey drift time history for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 3rd storey; c) 1st storey

Figure 5.52: Storey drift time history for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 5th storey; c) 1st storey

Figure 5.53: Storey drift time history for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 5th storey; c) 1st storey

Figure 5.54: Storey drift time history for the 20-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake: a) Top storey; b) 10th storey; c) 1st storey

Figure 5.55: Storey drift time history for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake: a) Top storey; b) 10th storey; c) 1st storey

Figure 5.56: Member force time histories for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake

Figure 5.57: Member force time histories for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake

Figure 5.58: Member force time histories for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake

Figure 5.59: Member force time histories for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake

Figure 5.60: Member force time histories for the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake

Figure 5.61: Member force time histories for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake

Figure 5.62: Storey force time histories for the 5-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake

Figure 5.63: Storey force time histories for the 5-storey frame resulting from Honzan record of the 1995 Kobe earthquake

Figure 5.64: Storey force time histories for the 10-storey frame resulting from Santa Monica record of the 1994 Northridge earthquake

Figure 5.65: Storey force time histories for the 10-storey frame resulting from Honzan record of the 1995 Kobe earthquake

Figure 5.66: Storey force time histories for the 20-storey frame resulting from Newhall record of the 1994 Northridge earthquake

Figure 5.67: Storey force time histories for the 20-storey frame resulting from Honzan record of the 1995 Kobe earthquake

Chapter 6

Conclusions and Recommendations

6.1 Conclusions

Three SMRF buildings of 5-, 10- and 20-storeys are used to investigate the influence of vertical ground motion on the seismic response of the SMRF structures. The behaviour of rigid beam-to-column connections is assumed. The response parameters such as the damage pattern, vibration periods of undamaged and damaged structures, lateral displacements, storey drifts, member forces and storey forces are assessed and discussed. The results have revealed some differences of structural response under horizontal, and horizontal plus vertical, ground motion excitations in the inelastic range.

Based on the findings, the conclusions can be summarized as follows:

(1) In the elastic range, the vertical ground motion has almost no influence on vibration periods, displacements and storey drifts at different storey heights, and bending moments and shear forces of columns and storeys of the SMRF structures.

(2) In the inelastic range, the vertical ground motion is a significant factor in contributing to the extent and pattern of the structural damage, that is, the formation of the first plastic hinge under horizontal plus vertical ground motion excitations is earlier than under horizontal earthquake excitation only. The sequence and location of the plastic hinge formation could be greatly influenced by the vertical ground motion and there are more plastic hinges formed under horizontal and vertical ground motion excitations.

(3) In the inelastic range, the vertical ground motion increases the vibration periods, displacements and storey drifts at different storey heights, and decreases shear forces and bending moments of columns and storeys of the damaged structures.

(4) Both in the elastic and inelastic ranges, the column and storey axial forces could be greatly increased when the effect of vertical ground motion is taken into account.

(5) Although the ratios of vertical to horizontal ground motion far exceed the range of $2/3$ to $3/4$ assumed by the NBCC 95 in the recent earthquakes of the Saguenay 1988, Northridge 1994 and Kobe 1995, the differences of structural seismic responses of the three SMRF structures between horizontal, and horizontal plus vertical ground motions are all within 5% limit except the axial forces.

(6) The low a/v ratio horizontal and vertical ground motions cause more severe damage to the three 5-, 10-, and 20-storey frames than the high a/v ratio horizontal and vertical ground motions even when the frames are designed based on the same peak ground velocity.

(7) In comparison to the damage patterns of the static push-over analysis, to some extent, the results of seismic analysis correlate well with those of static push-over analysis for the 5- and 10-storey frames under some earthquake excitations. For the 20-storey frame, the damage patterns of the seismic response are different compared to that of static push-over analysis. This is because all the selected earthquake

records contain little seismic energy around the fundamental period of 3.66 seconds of the 20-storey frame.

(8) Although DRAIN-2DX is a good computer program for the seismic response analysis of building structures, the deficiencies of using a perfect elasto-plastic model in the plastic hinge beam-column element in the inelastic range cannot produce exact results of structural analysis. The stiffness degrading model should be used to further evaluate the inelastic response of the SMRF structures, and also the case of constant geometric stiffness matrix $[K_g]$ based on static gravity load to determine the nonlinear axial load effect is inadequate.

6.2 Suggestions for Future Work

Based on the findings of this study, further research is still required as follows:

(1) Modifications to the perfect elasto-plastic model in the plastic hinge beam-column element of the DRAIN-2DX program are required in order to model the structure degrading in the inelastic range properly. That means the stiffness degrading model should be used in the inelastic range.

(2) The geometric stiffness matrix of the DRAIN-2DX program is not changed with time under the vertical ground motions. This is another major factor for predicting the exact structural seismic response in the elastic and inelastic range, therefore, modifications to the geometric stiffness in each time step are required for further studies.

(3) Additional studies are required to better understand the effect of soil-structure interaction under the simultaneous input of horizontal and vertical ground motions.

(4) Further studies are still needed to investigate the effect of vertical ground motion on other kinds of buildings such as weak column strong beam (WCSB) structural systems and reinforced concrete structures.

References

- AIJ 1995b, "Investigation of the disaster due to the Hyogo-Ken Nanbu earthquake - The urgent meeting report". *Architectural Institute of Japan*, Tokyo, Japan
- AIJ 1995d, "Performance of steel buildings during the 1995 Hyogo-Ken Nanbu earthquake". *Architectural Institute of Japan*, Tokyo, Japan. (In Japanese with English summary)
- Anderson, J.C., and Bertero, V.V., 1969, "Seismic behavior of multistorey frames designed by different philosophies". *Earthquake Engineering Research Center, University of California at Berkeley*. Report No. EERC 69-11
- Anderson, J.C. and Bertero, V.V., 1973, "Effect of Gravity loads and vertical ground acceleration on the seismic response of multi-storey frames". *Proceedings of Fifth World Conference on Earthquake Engineering*, Rome, Italy, 2914-2923
- Anderson, J.C., and Bertero, V.V., 1987. "Uncertainties in establishing design earthquakes". *Journal of Structural Engineering, ASCE*, No. 8. 1709-1724
- Ariaratnam, S.T., 1967, "Dynamic stability of a column under random loading". *Dynamic Stability of Structures*, G. Herrman ed., Pergamon Press Inc., New York
- Ariaratnam, S.T. and Leung, K.C.K., 1990, "Parametric effect of vertical ground acceleration on the earthquake response of elastic structures". *Canadian Journal of Civil Engineering*, Vol. 17, 209-217
- Beavers et al., 1989, "Strong motion records". *Earthquake Spectra-Armenia Earthquake Reconnaissance Report*, 43-53
- Bertero, V.V., Popov, E.P., and Krawinkler, H., 1972, "Beam-column subassemblages under repeated loading". *Journal of Structural Division, ASCE*, Vol. 98, 1137-1159

Bertero, V.V., Anderson, J.C., and Krawinkler, H., 1994, "Performance of steel building structures during the Northridge earthquake". *Earthquake Engineering Research Center, University of California at Berkely*, REPORT NO. UCB/EERC-94/09

Biggs, J.M., Hansen, R.J., and Holley, M.J., 1977, "On methods of structural analysis and design for earthquakes. In structural and geotechnical mechanics: A volume honoring Nathan M. Newmark. Edited by W.J. Hall". *Prentice-Hall Inc., Englewood Cliffs, NJ*, 91–101

Buckens, F., 1977, "Analysis of the damped response of a multi-story building , considered as a heavy Timoshenko beam, to a bidimensional shaking motion of the ground". *Proceedings of Sixth World Conference on Earthquake Engineering*, India. 1382–1387

Byrne, P. M., Finn, W. D. L., and Ventura, C. E., 1996, "The Hyogo-Ken Nanbu (Kobe) earthquake of 17 January 1995: seismological aspects and ground motions". *Canadian Journal of Civil Engineering*, Vol. 23, 771–777

Canadian Institute of Steel Construction. 1995. "Handbook of Steel Construction. Sixth Edition"

Challa, V. R.M., and Hall, J.F., 1994, "Earthquake collapse analysis of Steel frames". *Earthquake Engineering and Structural Dynamics*, Vol. 23, 1199–1218

Chen, Y., Tsoi, K.L., Chen, F., Gao, Z., Zou, Q., and Chen, Z., 1976, "The Great Tangshan Earthquake of 1976"

Cheng, F.Y., and Oster, K.B., 1976, "Effect of coupling earthquake motions on inelastic structural models". *International symposium on earthquake structural engineering*, St. Louis, Missiuri, USA, Vol. 1, 107–125

- Cheng, F.Y., and Oster, K.B., 1976, "Dynamic instability and ultimate capacity of inelastic systems parametrically excited by earthquakes-part II". *Civil Engineering Study 76-1, structural series, Dept. of Civil Engineering, Univ. of Missouri, Rolla*
- Cheng, F.Y., and Oster, K.B., 1977, "Ductility studies of parametrically excited systems". *Proceedings of Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. 2, 1118-1123
- Cheng, F.Y., and Srifuengfung, D., 1978, "Optimum structural design for simultaneous multicomponent static and dynamic inputs". *International Journal for Numerical Methods in Engineering*, Vol. 13, No. 2, 353-371
- Cheng, F.Y., and Kitipitayangkul, P., 1980, "Inelastic behavior of building systems subjected to three-dimensional earthquake motions". *Proceedings of Seventh World Conference on Earthquake Engineering*, Istanbul, Turkey, Vol. 7, 495-502
- Cherry, S., 1996, "Introduction to papers on the 1995 Hyogo-ken Nanbu (Kobe) earthquake". *Canadian Journal of Civil Engineering*, Vol. 23, 650-651
- Chung, R.M., 1995, "introduction". *Earthquake Spectra-Hokkaido Earthquake Reconnaissance Report*, Supplement A to Volume 11, 1-7
- Esteva, L., 1988, "The Mexico earthquake of September 19, 1985-consequences, lessons, and impact on research and practice ". *Earthquake Spectra*, Vol. 4, No. 3, 413-426
- Ger, J.F., Cheng, F.Y., and Lu, L.W., 1993, "Collapse behavior of Pino Suarez building during 1985 Mexico city earthquake". *Journal of Structural Engineering*, 119, 3, 852-870
- Ger, J.F. and Cheng, F.Y., 1992, "Collapse assessment of a tall steel building damaged by 1985 Mexico earthquake". *Proceedings of the tenth world conference on earthquake engineering*, Vol. 1, 51-56

- Ghobarah, A., Korol, R.M., and Osman, A., 1992, "Cyclic behaviour of extended end-plate joints". *Journal of Structural Engineering, ASCE*, Vol. 118, 1333–1353
- Gürpınar, A., and Yao, J.T.P., 1973, "Design of columns for seismic loads". *Journal of Structural Division, ASCE*, Vol. 99, No. ST9, 1875–1889
- Hall, J.F., Heaton, T.H., Halling, M.W., and Wald, D.J., 1995, "Near-source ground motion and its effects on flexible buildings". *Earthquake Spectra*, No. 8, 569–605
- Heidebrecht, A.C., 1995, "Introduction to papers on the 1994 Northridge earthquake". *Canadian Journal of Civil Engineering*, Vol. 22, 298–299
- Humar, J.L., 1990, "Dynamics of Structures". *Prentice Hall, Englewood Cliffs, New Jersey*, 622–625
- Iyengar, R.N. and Shinozuka, M., 1972, "Effect of self-weight and vertical acceleration on the behavior of tall structures during earthquakes". *Earthquake Engineering and Structural Dynamics*, Vol. 1, 69–78
- Kannan, A.E., and Powell, G.H., 1973, "DRAIN - 2D". *Earthquake Engineering Research Center, University of California at Berkeley*, REPORT NO. UCB/EERC 73-6 and EERC 73-22
- Kulak, G.L., Adams, P.F., and Gilmor, M.I., 1995, "Limit States Design in Structural Steel". *Canadian Institute of Steel Construction*
- Lawson, R.S., Vance, V., and Krawinkler, H., 1994, "Nonlinear static push-over analysis - why, when, and how". *Proceedings of fifth U.S. national conference on earthquake engineering*, Vol. 1, 283–292
- Leger, P. and Romano A., 1992, "Seismic analysis of short-period structures subjected to the 1988 Saguenay earthquake". *Canadian Journal of Civil Engineering*, Vol. 19, 510–520

- Lin, Y.K. and Shih, T.Y., 1980, "Column response to horizontal-vertical earthquakes". *Journal of the Engineering Mechanics Division, ASCE*, 106(EM6), 1099–1109
- Lin, Y.K. and Shih, T.Y., 1982, "Vertical seismic load effect on building response". *Journal of the Engineering Mechanics Division, ASCE*, 107(EM2), 331–343
- Meyer, C., 1974, "Inelastic dynamic analysis of tall buildings". *Earthquake Engineering and Structural Dynamics*, Vol. 2, 325–342
- Mitchell, D., Tinawi, R., and Law, T., 1990, "Damage caused by the November 25, 1988, Saguenay earthquake". *Canadian Journal of Civil Engineering*, Vol. 17, 338–365
- Moehle et al., 1995, "Seismology and tectonics". *Earthquake Spectra-Guam Earthquake Reconnaissance Report*, , Supplement B to Volume 11, 11–24
- Montgomery, C.J., and Hall, W.J., 1979, "Seismic design of low rise steel buildings". *Journal of the Structural Division, ASCE*, 105, 1917–1933
- Nader, N.N., and Astanteh, A., 1992. "Seismic behavior and design of semi-rigid steel frames". *Earthquake Engineering Research Center, University of California at Berkeley*. Report No. UCB/EERC-92/06
- Naman, S.K., and Goodno, B.J., 1986, "Seismic evaluation of low-rise steel buildings". *Engineering Structures*, No. 8, 9–16
- NRC-CNRC, 1995 "National Building Code of Canada 1995"
- Osman, A., Ghobarah, A. and Korol, R.M., 1995, "Implications of design philosophies for seismic response of steel moment frames". *Earthquake Engineering and Structural Dynamics*, Vol. 24, 127–143
- Osteraas, J. and Krawinkler, H., 1989, "The Mexico earthquake of September 19, 1985—behavior of steel buildings". *Earthquake Spectra*, Vol. 5, No. 1, 51–88

Osteraas, J, and Krawinkler, H., 1990, "Seismic design based on strength of structures". *Proceedings of 4th U.S. national conference on earthquake engineering*, Vol. 2, 955–964

Popov, E.P., Bertero, V.V., and Chandramouli, S., 1975, "Hysteretic behavior of steel columns". *Proceedings of U.S. National Conference on Earthquake Engineering*, Ann Arbor, Michigan, 245–254

Powell, G.H., and Row, D.G., 1976, "Influence of design and analysis assumptions on computed inelastic response of moderately tall frames". *Earthquake Engineering Research Center, University of California at Berkely*, REPORT NO. UCB/EERC-76/11

Prakash. V., Powell. G.H., and Campbell, S., 1993. "DRAIN-2DX base program description and user guide". *University of California at Berkeley*, Report No. UCB/SEMM-93/17

Redwood. R.G., Lefki, L. and Amar, G., 1990. "Earthquake resistant design of steel moment resisting frames". *Canadian Journal of Civil Engineering*, Vol. 17, 659–667

Roeder, C.W., Schneider, S.P., and Carpenter, J.E., 1993, "Seismic behavior of moment-resisting steel frames: Analytical study". *Journal of Structural Engineering, ASCE*, Vol. 119, 1866–1883

Saadeghvaziri, M.A. and Foutch, D.A., 1991, "Dynamic behaviour of R/C highway bridges under the combined effect of vertical and horizontal earthquake motions". *Earthquake Engineering and Structural Dynamics*, Vol. 20, 535–549

Shih, T.Y., and Lin Y.K., 1982, "Vertical seismic load effect on hysteretic columns". *Journal of Engineering Mechanics Division, ASCE*, Vol. 108(EM2), 242–254

Sivakumaran, K. S., 1990, "Lateral load response of unbraced steel building frames". *Canadian Journal of Civil Engineering*, Vol. 17, 974–986

- Sivakumaran, K.S., 1991, "Seismic response of unbraced steel frames". *Earthquake Engineering and Structural Dynamics*, Vol. 20, 1029–1043
- Stratonovich, R.L., 1963, "Topics in the theory of random noise, Vol. 1, Translation by R.A. Silverman". *Gordon and Breach Science Publishers*, New York
- Thiel et al., 1986, "The Chile earthquake of March 3, 1985–introduction ". *Earthquake Spectra*, Vol. 2, No. 2, 249–252
- Tremblay, R., and Filiatrault, A., 1994, "Preliminary observations on damage to steel structures". *Preliminary report on the Northridge, California, earthquake of January 17, 1994*, Canadian Association for Earthquake Engineering, 159–165
- Tremblay, R., Timler, P., Bruneau, M., and Filiatraut, A., 1995. "Performance of steel structures during the 1994 Northridge earthquake". *Canadian Journal of Civil Engineering*, Vol. 22, 338–360
- Tremblay, R., Bruneau, M., Prion, H.G.L., Filiatraut, A., and Devall, R., 1995, "Preliminary observations on the performance of steel buildings during the January 17, 1995 Hyogo-Ken Nanbu earthquake". *The Hyogo-Ken Nanbu (Kobe) Earthquake of 17 January 1995*, Vancouver, Canada, 46–70
- Tremblay, R., Bruneau, M., Nakashima, M., Prion, H.G.L., Filiatraut, A., and Devall R., 1996. "Seismic design of steel buildings: lessons from the 1995 Hyogo-ken Nanbu earthquake". *Canadian Journal of Civil Engineering*, Vol. 23, 727–756
- Tsai, K.C., and Popov, E.P., 1988, "Steel beam-column joints in seismic moment resisting frames". *Earthquake Engineering Research Center, University of California at Berkeley*. Report No. UCB/EERC 88-19
- Tso, W.K., and Guru, B.P., 1976, "A statistical study on artificially generated earthquake records". *Canadian Journal of Civil Engineering*, Vol. 3, No. 1, 11–19

Tso, W.K., and Zhu, T.J., 1991, "Implications of the 1988 Saguenay earthquake on Canadian seismic strength specification". *Canadian Journal of Civil Engineering*, Vol. 18, 130-139

Ventura, C.E., Finn, W.D.L., and Schuster, N.D., 1995, "Seismic response of instrumented structures during the 1994 Northridge, California, earthquake". *Canadian Journal of Civil Engineering*, Vol. 22, 316-337

Wennerberg, L., 1991, "Preliminary interpretation of aftershock data". *Earthquake Spectra-Philippines Earthquake Reconnaissance Report*, Supplement A to Volume 7, 17-26

Wirsching, P.H., and Yao, J.T.P., 1971, "Random behavior of columns". *Journal of Engineering Mechanics Division, ASCE*, Vol. 97, No. EM3, 605-618

Zhu, T.J., Tso, W.K., and Heidebrecht, A.C., 1992, "Seismic performance of reinforced concrete ductile moment-resisting frame buildings located in different seismic regions". *Canadian Journal of Civil Engineering*, Vol. 19, 688-710

IMAGE EVALUATION TEST TARGET (QA-3)

APPLIED IMAGE, Inc.
1653 East Main Street
Rochester, NY 14609 USA
Phone: 716/482-0300
Fax: 716/288-5989

© 1993, Applied Image, Inc., All Rights Reserved