A STUDY ON THE SEISMIC BEHAVIOUR OF A 52-STOREY STEEL FRAME BUILDING

by

YUMING DING

B.Sc., Shenzhen University, 1993

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF APPLIED SCIENCE in THE FACULTY OF GRADUATE STUDIES

Department of Civil Engineering

We accept this thesis as conforming to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

April 1999

© Yuming Ding, 1999
In presenting this thesis in partial fulfilment of the requirements for an advanced degree at the University of British Columbia, I agree that the Library shall make it freely available for reference and study. I further agree that permission for extensive copying of this thesis for scholarly purposes may be granted by the head of my department or by his or her representatives. It is understood that copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Department of Civil Engineering

The University of British Columbia
Vancouver, Canada

Date April 28, 1999

DE-6 (2/88)
ABSTRACT

Comparison of recorded structural earthquake response and theoretical dynamic analysis is vital to the study of earthquake engineering nowadays. This comparison could be either in the frequency domain by utilizing system identification algorithms or in the time domain by comparing response time histories. Not only had seldom previous studies done this comparison, but also less attention was paid to the three-dimensional nonlinear dynamic behaviour despite the fact that it has been generally admitted that three-dimensional response is important in inelastic behavior of buildings.

The purpose of this research is to study the seismic behaviour of a well-instrumented 52-storey steel frame building in Los Angeles, California. This building has been subjected to ground motions from several earthquakes among which the 1991 Sierra Madre earthquake and the 1994 Northridge earthquake were selected in this study. During both earthquakes the building responses appeared to remain in the linear range.

Detailed frequency domain analyses of the recorded motions from these two earthquakes were conducted to determine the dynamic characteristics of the structure. Three-dimensional nonlinear dynamic computer analyses were then employed to evaluate the response of the structure induced by other earthquake excitations. Dynamic characteristics and seismic responses from the above two approaches hence could be compared to yield valuable information.

The results of this study showed that by performing a linear three-dimensional analysis, the actual response of a building during past earthquakes could be reproduced with confidence. By further performing a nonlinear three-dimensional analysis, the state and sequence of damage could also be predicted.
Traditional Nonlinear Static Procedure (NSP, pushover analysis) has the deficiency of excluding the higher mode participation which becomes obvious for highrise buildings. Improvements to the NSP were explored. Because of the torsional response for highrise buildings, two-dimensional analysis is not feasible to reliably predict their nonlinear response during earthquakes.
# TABLE OF CONTENTS

**ABSTRACT** .................................................................................................................... ii  
**TABLE OF CONTENTS** ................................................................................................. iv  
**LIST OF TABLES** ......................................................................................................... vii  
**LIST OF FIGURES** ....................................................................................................... viii  
**ACKNOWLEDGEMENTS** ............................................................................................. x  

**Chapter 1 Introduction** ................................................................................................. 1  
1.1 General Remarks ........................................................................................................ 1  
1.2 Objectives and Scope ................................................................................................. 3  
1.3 Organization .............................................................................................................. 5  

**Chapter 2 Background of Nonlinear Analysis** ................................................................. 7  
2.1 Why Nonlinear Analysis ............................................................................................ 7  
2.2 Literature Review .................................................................................................... 9  

**Chapter 3 Description of the Figueroa at Wilshire Tower** ............................................... 14  
3.1 Introduction ................................................................................................................ 14  
3.2 Structural System .................................................................................................... 16  
3.3 Building Materials .................................................................................................. 21  
3.4 Instrumentation ....................................................................................................... 21  

**Chapter 4 Analyses of Recorded Data** ........................................................................ 24  
4.1 Recorded Earthquake Responses .......................................................................... 24  
4.2 Time Domain Analysis of Recorded Motions ......................................................... 25  
4.3 Frequency Domain Analysis .................................................................................. 30  
4.3.1 Theory .............................................................................................................. 30
Chapter 5 Description of the Nonlinear Analysis Program ........................................ 46
  5.1 Introduction .................................................................................. 46
  5.2 Elements ..................................................................................... 48
  5.3 CANNY Hysteresis Models .......................................................... 50
  5.4 More information about the program ............................................ 51

Chapter 6 Three Dimensional Linear Analysis of the Figueroa at Wilshire Tower ...... 54
  6.1 Introduction and theory ................................................................. 54
  6.2 Development of a linear model ................................................... 55
  6.3 Calibration of the linear model and period sensitivity .................. 58
  6.4 Comparison of predicted and recorded time histories ................. 61
    6.4.1 Comparison of predicted and recorded time histories during the Sierra Madre earthquake .............................................. 62
    6.4.2 Comparison of predicted and recorded time histories during the Northridge earthquake .............................................. 64
  6.5 Comparison of code design values .............................................. 66
  6.6 Summary .................................................................................... 70

Chapter 7 Three-Dimensional Nonlinear Analysis of the Figueroa at Wilshire Tower . 71
  7.1 Introduction ............................................................................... 71
  7.2 Description of the Nonlinear model ........................................... 72
  7.3 Time History Analysis ............................................................... 75
    7.3.1 Response of the building during the given earthquake excitations .... 80
LIST OF TABLES

Table 3.1 Comparative study of perimeter ductile tube versus spine structure ................. 17
Table 3.2 Properties of construction materials ................................................................. 21
Table 4.1 Peak accelerations recorded during two earthquakes ........................................... 28
Table 4.2 Periods of the first three modes in each direction ............................................... 39
Table 4.3 Damping ratios of the first three modes in each direction .................................... 39
Table 4.4 Scaling constants of residue matrix ..................................................................... 40
Table 4.5 Periods and dampings from the conditioned records ............................................ 43
Table 5.1 Uniaxial hysteresis model options and number of parameters ............................. 52
Table 6.1 Estimated weight and mass polar moment of inertia ............................................ 57
Table 6.2 Live loads for calibration of the models ............................................................... 58
Table 6.3 Periods of the first three modes in each direction ............................................... 59
Table 6.4 Damping ratios used for computer analyses ......................................................... 60
Table 7.1 Peak ground accelerations (PGA) of ground motions ......................................... 76
Table 7.2 Comparison of results ......................................................................................... 89
LIST OF FIGURES

Figure 3.1 The Figueroa at Wilshire Tower in Los Angeles ...................................... 15
Figure 3.2 Configuration of braced frames showing different failure mechanisms ........ 18
Figure 3.3 Typical floor plan and details of the columns of the FWT ............................... 20
Figure 3.4 Instrumentation plan of the FWT ................................................................. 23
Figure 4.1 Location of the FWT and epicenters of several earthquakes in the vicinity of the building site ................................................................. 24
Figure 4.2 Recorded accelerations during the Sierra Madre earthquake ......................... 26
Figure 4.3 Recorded accelerations during the Northridge earthquake .............................. 27
Figure 4.4 Response spectra of recorded ground floor motions during the Sierra Madre earthquake ................................................................. 29
Figure 4.5 Response spectra of recorded ground floor motions during the Northridge earthquake ................................................................. 29
Figure 4.6 FRFs from the Northridge earthquake ............................................................ 34
Figure 4.7 Comparison of torsional records' power spectra ........................................... 35
Figure 4.8 Mode shapes of the building from the Sierra Madre earthquake ...................... 37
Figure 4.9 Mode shapes of the building from the Northridge earthquake ......................... 38
Figure 4.10 Comparison of the coherence functions between recorded motions for “A” level during the Northridge earthquake ................................................................. 42
Figure 4.11 Conditioned records from the Northridge earthquake .................................... 43
Figure 5.1 3D multispring column model ..................................................................... 49
Figure 6.1 CANNY model of the FWT ........................................................................ 55
Figure 6.2 Mode shapes for the FWT in each direction .................................................... 59
Figure 6.3 Comparison of the responses using conventional damping ratios .................... 61
Figure 6.4 Comparison of the responses after damping calibration ........................................... 62

Figure 6.5 Comparison of recorded and computed roof absolute acceleration during the Sierra Madre earthquake ................................................................. 64

Figure 6.6 Comparison of recorded and computed roof absolute acceleration during the Northridge earthquake ................................................................. 65

Figure 6.7 Comparison of recorded and computed roof relative displacement during the Northridge earthquake ................................................................. 65

Figure 6.8 Comparison of recorded and computed 14th story absolute acceleration during the Northridge earthquake ................................................................. 67

Figure 6.9 Comparison of recorded and computed 14th story relative displacement during the Northridge earthquake ................................................................. 68

Figure 7.1 CANNY hysteresis model #14 ................................................................................. 73

Figure 7.2 Time history and response spectrum of the SVAC-A record (Niigata earthquake) ................................................................................................. 77

Figure 7.3 Time history and response spectrum of the Joshua Tree Fire station (Landers earthquake) ......................................................................................... 78

Figure 7.4 Time history and response spectrum of the Sylmar County Hospital station (Northridge earthquake) ........................................................................ 79

Figure 7.5 Maximum inelastic response of the FWT, displacement ........................................... 80

Figure 7.6 Maximum inelastic response of the FWT, interstory drift ........................................ 81

Figure 7.7 Maximum inelastic response of the FWT, story shear ............................................. 82

Figure 7.8 Maximum inelastic response of the FWT, overturning moment ................................ 83

Figure 7.9 Roof displacement time history responses, Sylmar County Hospital record ............... 84

Figure 7.10 Roof acceleration time history responses, Sylmar County Hospital record ............... 85

Figure 7.11 Base shear versus roof displacement, E-W direction ............................................... 86

Figure 7.12 Yielding propagation steps in the south perimeter frame in E-W direction ............... 88

Figure 7.13 Base shear versus roof displacement, cyclic test ..................................................... 92
ACKNOWLEDGEMENTS

I am deeply indebted to my supervisor, Dr. Carlos E. Ventura, for his guidance throughout this project, and all the time and effort he put in advising me.

Special thanks are due to Dr. Kang-Ning Li of CANNY Consultants Pte. Ltd., Singapore for providing his program, CANNY-E and valuable assistance on implementing its various features.

The financial support for this study was provided by a research grant awarded to Dr. C.E. Ventura from the Natural Science and Engineering Research Council of Canada.
Chapter 1 INTRODUCTION

1.1 General Remarks

Today, with the help of computers, buildings which have little or no historic precedent are being planned and designed. New structural systems are conceived and applied to extremely tall buildings in a practical demonstration of the engineer's confidence in the knowledge of structures, theory of analysis and efficiency of computer algorithms. It is therefore important to update the theory with the latest information obtained from recent research and practise. There are three major factors to consider in the design of all structures: strength, rigidity, and stability. In the design of tall buildings, all three these requirements play an important role. The strength requirement is the dominant factor in the design of low-rise structures. However, as height increases, the rigidity and stability requirements become more important, and they are often the dominant factors in the design. These requirements are forcing designers to change from the traditional linear static techniques to more advanced inelastic dynamic techniques.

In certain seismically active regions, severe earthquakes happen frequently, causing the loss of hundreds to thousands of lives. Also, property damage accounts for hundreds of millions of dollars. As a consequence of the 1994 Northridge earthquake in Los Angeles, significant research has been carried out in order to: 1) learn the seismic responses from analyzing a large amount of recorded data; and 2) re-evaluate the performance of buildings by detailed dynamic analysis.

To achieve the first goal of learning the seismic responses from analyzing the large amount of recorded data, instrumentation is a vital and prerequisite step. Due to the lack of instrumentation
in buildings in the first half of the 20th century, the original concept of spectrum design that dominates today's design codes is largely based on the recorded data from the 1940 El Centro earthquake. It is obvious that the knowledge from one earthquake event could be limited and misleading. Luckily the large amount of recorded strong motion data in recent years provide excellent opportunities to improve engineers' knowledge towards seismic response.

One of the objectives from analyzing these large amount of recorded strong motion data must be the better design of future structures. Thus the second goal of re-evaluating the performance of the buildings by detailed dynamic analysis comes to mind. Since the comparison of the recorded seismic response and the responses from computer dynamic analysis could bring improved understanding of seismic response of structures, it is of great interest to carry out a study including both of the purposes mentioned above. This study serves to reach this goal.

One of the most important factors which has been avoided in previous research is the three-dimensional nature of both the excitation and response of the structure during earthquakes. In fact, the three-dimensional behavior of structures during earthquakes has not been studied as extensively as that of planar or two-dimensional behavior (Rahmatian, 1997). The unavailability of computer programs capable of three-dimensional inelastic analyses was cited as the excuse of this simplification.(SAC95-04) Experimental studies, although limited, have demonstrated that there is a significant interaction between the responses of a non-symmetric structure along its two principal axes. It has been observed that even structures that appear symmetric often experienced non-uniform connection damage that could make the structures more sensitive to three-dimensional response. With the appearance of sophisticated computer programs and powerful
personal computers, a near future tendency is to utilize three-dimensional inelastic analyses for seismic design of structures.

1.2 Objectives and Scope

The goal of this thesis is to study and understand the seismic behavior of a steel high-rise building. In order to accomplish this, the following objectives are identified: 1) conduct a system identification of the recorded earthquake motions, 2) investigate the building behavior during past earthquakes through construction of a lumped plasticity computer model and calibrate the model using recorded motions, 3) evaluate the inelastic structural response of the building during severe earthquakes shaking, and 4) evaluate the inelastic static response of the building for three lateral load patterns.

The building in this study is modeled based on specifications and informations included in the design drawings and personal contacts between the designer of the building Dr. Banavalkar, P.E. of CBM Engineers Inc. of Houston, Texas and Dr. C.E. Ventura, P.Eng., Assoc. Prof. of the department of Civil Engineering in the University of British Columbia, Vancouver. The emphasis of the study conducted here is on the linear and non-linear global response and behavior of the structure, rather than on any of its elements. Foundation and the interaction between the foundation and the soil are not modeled in the computer analysis. The main emphasis of the inelastic response is on the evaluation of the effects of near field impulse-type ground motions.

In order to achieve the objectives of the study, the following tasks were performed:
1. The instrumentation scheme of the Figueroa at Wilshire Tower (FWT) is described, and the strong-motion response records obtained from the building during the 1991 Sierra Madre and the 1994 Northridge earthquakes are described and analyzed.

2. The modal frequencies are determined for the three directions: two translational and one rotational, by using system-identification techniques. Also, results are compared with those obtained from the computerized structural analysis conducted by the structural designers.

3. The damping ratios and contributions of each mode to the response are derived, and a number of dynamic characteristics and interesting features are further investigated for future dynamic analysis. (More attention has been paid to the frequency and damping variation between two earthquakes and the modal contributions.)

4. A structural model is set up using computer program CANNY (version E) and the linear dynamic characteristics such as frequencies and mode shapes are verified. The time history responses of the structural model are calculated using the recorded base motions as inputs to the model and are compared with recorded earthquake responses at different floors. Since the building responded linearly during the two earthquakes considered, the comparison of computer and recorded earthquake motions is used to verify the credibility of the computer program results.

5. Analysis is then conducted to understand the three-dimensional inelastic behavior of the building during severe shaking.
1.3 Organization

A short discussion about linear versus nonlinear analysis, together with a literature review on nonlinear modeling of steel frame buildings are covered in Chapter 2.

In Chapter 3 a complete description of the building studied (FWT) is given. Some of the topics discussed here are: the structural system and the reason it was chosen, construction materials, seismic instrumentation plan.

Detailed Strong Motion Data analysis results, including time domain analysis and frequency domain analysis, are presented in Chapter 4. Derived modal frequencies, mode shapes, modal dampings are shown in tables or figures.

A general review of the nonlinear analytical program (CANNY-E) used in this study and some of its unique features are presented in Chapter 5.

Chapter 6 is concerned with a three-dimensional linear-elastic modeling of the structure. Three-dimensional analyses of the structure are performed for two input ground motions: one from the Sierra Madre earthquake and the other from the Northridge earthquake. The purpose of these analyses is to calibrate and verify the computer model with the real structure.

The nonlinear three-dimensional analysis of the building is discussed in Chapter 7. The development of the model, the underlying assumptions, and finally the response of the structure during several severe earthquake excitations are presented here. In addition to the time history
analysis, pushover and cyclic analysis of the building are also conducted.

Finally, a summary of the results of this study followed by recommendations for further studies are given in Chapter 8.
Chapter 2 BACKGROUND OF NONLINEAR ANALYSIS

2.1 Why Nonlinear Analysis

In the 90s, many countries have experienced a number of major earthquake disasters such as the 1994 Northridge earthquake and the 1995 Kobe earthquake. These earthquakes have exposed many important problems that adversely impacted the urban areas around the world in terms of the well-being and safety of individual citizens, the operation of essential social services and life-line systems, and the productivity and economic vitality of the business, financial and industrial sectors.

Every earthquake provides many valuable lessons to the earthquake engineering profession. The event in Northridge was no exception to this. What surprised engineering professionals was the damage to modern steel buildings. Until this earthquake, steel buildings were thought to be among the most seismically resilient forms of construction possible. In Los Angeles area, however, welded steel frame buildings suffered many brittle fractures in their connections during the Northridge earthquake. And many of this type of structures collapsed during the motions experienced in Kobe.

It is therefore necessary to study the behavior of buildings during these earthquakes in order to improve the knowledge of seismic design. In past design practise, results from linear elastic analysis performed using computer programs were dominantly used to determine member strength demands. The extensive damage and collapse of structures in these two earthquakes proved the deficiency of this approach to predict the building response. The implementation of a perfor-
mance-based design procedure necessitates reliable analytical predictions of important seismic demands (strength, deformation, and energy dissipation demands). Research needs to be performed to develop reliable analytical techniques and tools that permit performance evaluation of three-dimensional systems and its components at all levels of performance, ranging from elastic behavior to degradation leading to collapse. Therefore it is of critical importance to accurately model three-dimensional inelastic building responses.

The focus of these type of research need to cover the following important issues:

- 3-D modeling procedures
- Reliable modeling of strength and stiffness irregularities in plan and elevation
- Modeling of deteriorating systems

Nonlinear analysis, attempting to represent more realistically the behavior of a building structure, has its own shortcomings. Traditionally it is labour intensive and expensive to implement. Furthermore, nonlinear analysis requires a qualitative interpretation of many inelastic parameters in order to capture the response of the structure, while linear analysis could provide reasonable member strength demands by assigning appropriate values to the member stiffness.

The nonlinear models expected to capture the true behavior of structural members are difficult to construct. A complete realistic relation between the complex mechanics of material nonlinearities and the three-dimensional structural response of a system have not been well established through analytical work and experimental testing. Therefore, decisions must be made on which aspects of nonlinearities are significant, and how to capture them in the analytical model.
Despite these long time difficulties faced by both practicing engineers and academic researchers, recent enhancements in computational techniques and the development of more sophisticated models allow structural analysis to have integrated tools, based on components' behavior, which are capable of determining the behavior of three-dimensional structures in a realistic manner.

2.2 Literature Review

Literature on three-dimensional nonlinear modeling of tall steel frame structures is limited. One of the most comprehensive books on the structural analysis of tall buildings (Taranath, 1988) included only the geometric nonlinearities of tall buildings (such as P-delta effect) but not their material nonlinearities. SAC 95-04 report on steel moment frame buildings are the most detailed studies after the Northridge earthquake. However, all of the inelastic analyses in these reports were limited to two-dimensions. The unavailability of computer programs capable of three-dimensional analyses was quoted as the reason.

Henceforth the literature review on two-dimensional inelastic analyses at the member level are discussed briefly below. The determination of the inelastic member properties is an essential step in the evaluation of earthquake response. Early studies of the response of multistory buildings were based on shear-beam idealizations (Penzien, 1960). The entire structure was modeled as a series of single lateral degree of freedom systems, one at each floor, having the global hysteretic characteristics representative of the cyclic inelastic response of the corresponding floor. A parallel component element was introduced by Clough et al. (1965) and allowed for a bilinear moment-rotation relation. The element consists of two components: a fully elastic component to
represent strain hardening and an elasto-perfectly plastic component to represent yielding. The element stiffness matrix is the summation of the stiffness matrix of both components.

Kanaan and Powell (1973) used a modification of Clough's parallel component model for their beam column element. The reduction of section plastic moment capacity due to an axial force was introduced by using an interaction formula. The one component series model was formally introduced by Giberson (1969). This element consists of a linear elastic element with one equivalent nonlinear rotational spring attached to each end. The inelastic deformations of the member are lumped into the end springs.

Chen and Powell (1982) developed a lumped plasticity beam-column element, where it is assumed that inelastic behavior is concentrated in plastic hinges at the element end and the remaining part of the element remains elastic. In this model it is difficult to establish the general parameters required for the force-deformation relationships and the four dimensional yield surfaces, which are governed by the cross-sectional dimensions and the hysteretic force-deformation characteristic of the member material. Hsu (1974) and Takayanagi and Schnobrich (1979) proposed a multiple spring model for analyzing wall members. The member is divided into several subelements along its axis, each represented by a nonlinear spring.

A few analytical models on inelastic element behavior have been developed in recent years. One study provided comprehensive information was by Kim (1995). In his study Kim divided analytical models developed in the past two decades into three categories in accordance with an increasing level of refinement and complexity. These models are:
1. Global models. In these models the nonlinear response of a structure is concentrated at selected degrees of freedom. Each degree of freedom has the hysteretic characteristics of the inter story response. Such models are used in the preliminary design phase. Member forces and local inelastic deformation demands are not available.

2. Discrete element models. In these models the structure is modeled as an assembly of interconnected elements that describe the hysteretic behavior of structural members. Correspondingly, two types of element formulation are possible: lumped plasticity models and distributed plasticity models.

3. Microscopic finite element models. In these models members and joints are discretized into a large number of finite elements.

Ideally, analytical models should be based on an accurate representation of material behavior taking into account the controlling states of stress or strain and identifying the main factors which influence the hysteretic behavior of each critical region in any structural component during the earthquake response. At the same time these models should be computationally efficient. In practice, it is very difficult to find an analytical model satisfying both an accurate representation of material behavior and computational efficiency. Although lumped plasticity models sacrifice accuracy to some degree, these models have the capability to reasonably compromise between simplicity and accuracy in nonlinear seismic response studies. Kim’s study concentrates on this second class of models. He developed lumped plasticity models to better represent the experimental data. Then he used distributed plasticity models (fibre models) to check the results against lumped plasticity models. His study included lumped plasticity models for bare steel beams and columns, composite beams, and column panel zones.
Since Kim’s study was on steel moment frames, models for bracing element were not included. However, Jain and Goel (1978) denoted a multi-linear hysteretic model EL9 for computer program DRAIN-2D based on empirical data by Jain (1978). In Jain’s study small square tube and angle specimens, with or without gusset plate connections, were subjected to large cyclic static and dynamic axial displacements. A new hysteresis model was proposed for steel tubular members which includes reduction in compressive strength and increase in member length with number of cycles.

It was concluded that in concentric braced frames end moments and axial forces are equally important for bracing members having slenderness ratio of 60, whereas, elastic buckling dominates for slenderness ratio of more than 120. In eccentric frames, end moments dominate over axial forces for bracing members with slenderness ratio of 60 or less. End moments and axial forces are equally important for those with slenderness ratio between 60 to 120 and elastic buckling dominates for slenderness ratio more than 120. Effective slenderness ratio seems to be the most influential parameter in determining the hysteresis behavior of bracing members.

In FEMA-274, a typical force versus axial deformation response of a steel brace is shown and the residual compression force for the brace is about 20% of the buckling load. It also states that this percentage is about the same for many brace configurations. Therefore, it further recommends that for a more simplified Nonlinear Static Procedure analysis, the axial force-deformation behavior of a brace in compression could be modeled as an elasto-plastic element with the yield force equal to the residual force. However, an elastic analysis will also need to be done to
determine the maximum axial force delivered to the column, the beam, and the beam-column connections.

Since structures designed according to current building codes are expected to deform well into the inelastic range during severe earthquake ground motions, constructing a sophisticated model for structural analysis has become increasingly important. Due to the complex interactions between the various components of real structures, their dynamic characteristics up to failure cannot be identified solely from dynamic tests of scaled models. Moreover, the cost of such tests is often substantial, particularly for large scaled specimens. Historically these difficulties have been overcome by static tests on components and on down-scaled subassemblies of structures under cyclic load reversals. Results from these tests are used in the development and calibration of hysteretic models that permit the extrapolation of the limited test data to other cases and to the dynamic response of complete structures.

In this study computer program CANNY-E (Li, 1996) is chosen because of its ability of performing both three-dimensional time history elastic and inelastic analysis. Lumped plasticity models were constructed using this program. Detail introduction of the program is given in Chapter 5.
Chapter 3 DESCRIPTION OF THE FIGUEROA AT WILSHIRE TOWER

3.1 Introduction

An innovative steel structural system that emerged in the 1980s is the so-called spine or outrigger structure, which is considered to be an especially appropriate structural system for tall buildings in high seismic areas. The spine consists of vertical or inclined elements, and shear membranes in the form of braced frames, walls or vierendeel girders. The Figueroa at Wilshire Tower (FWT) is a good example of this type of structure in which the spine is the lateral load resisting system that provides stiffness and stability (Banavalkar, 1991).

The FWT is a 52-story steel frame building in Los Angeles, California. The building was designed in 1988, constructed in 1988-90 and instrumented to measure seismic motions after construction. Acceleration sensors recorded the building’s motions during many earthquakes among which the 1991 Sierra Madre earthquake and the 1994 Northridge earthquake were chosen for this study. These records provided detailed and important information about the building behavior during both earthquakes.

The FWT consists of a 52-story steel frame office tower and five levels of enlarged basement as underground parking. The floor plans of the tower are not perfectly square. On each floor, the tip of every corner was clipped and the middle third of each side was notched. Above the 36th story, at clusters of about five stories, the corners of the floors were clipped further to provide a stepback view to the exterior of the building. An overview of the building is shown in Fig. 3.1 and typical floor plan is shown in Fig. 3.3.
Figure 3.1 The Figueroa at Wilshire Tower in Los Angeles
3.2 Structural System

A comparative study of perimeter ductile tube (popular) versus spine structure (innovative) is shown in Table 3.1. The structural designer of the FWT considered three different structural schemes (two of which are perimeter tubular structure and the other is spine structure) for the building and the spine structure was finally chosen because its many advantages.

The structural system of the FWT has three main components: a braced core, twelve columns (8 on the perimeter and 4 in the core), and eight 914-mm (36-inch) deep outrigger beams at each floor connecting the inner and outer columns. The core, about 17.37 m (57 feet) by 21.34 m (70 feet), is concentrically braced between the “A” level (the level just below the ground level) and the 50th story (between 1st story and 2nd story is an exception due to architectural reason). Moment resisting connections are used at the intersection of beams and columns. The outrigger beams, about 12.19 m (40 feet) long, link the four core columns to the eight perimeter columns to form a ductile moment resisting frame. The outrigger beams are laterally braced to prevent lateral torsional buckling and are effectively connected to the floor diaphragm by shear studs to transmit the horizontal shear force to the frame.

The interior core is concentrically braced. The sizes of mechanical ducts and door openings into the core dictate the configuration and sizes of braces (Fig. 3.2). In order to achieve redundancy in carrying gravity loads, the most important design criteria of braces is that in the event of buckling of one of the compression diagonals, the horizontal members supporting the floor loads do not form a mechanism resulting in catastrophic failure of the floor. For bracing types II and III,
Table 3.1: COMPARATIVE STUDY OF PERIMETER DUCTILE TUBE VERSUS SPINE STRUCTURE (Banavalkar, 1992)

<table>
<thead>
<tr>
<th>CRITERIA</th>
<th>PERIMETER TUBE</th>
<th>SPINE STRUCTURE</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>I. Structural Performance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Wind load.</td>
<td>a) 100 year storm.</td>
<td>a) 100 year storm.</td>
</tr>
<tr>
<td>b) Seismic event.</td>
<td>b) Maximum probable (lower level event).</td>
<td>b) Maximum credible (upper level event).</td>
</tr>
<tr>
<td>c) Redundancy &amp; ductility.</td>
<td>c) Very ductile &amp; redundant.</td>
<td>c) Ductile &amp; redundant.</td>
</tr>
<tr>
<td>d) Torsional stiffness.</td>
<td>d) Torsionally stiff.</td>
<td>d) Depends upon composition of the spine.</td>
</tr>
<tr>
<td>e) Stiffness &amp; strength.</td>
<td>e) Very good.</td>
<td>e) Very good.</td>
</tr>
<tr>
<td><strong>II. Planning</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Flexibility in core &amp; lease space.</td>
<td>a) Complete flexibility.</td>
<td>a) Disciplined layout of core &amp; mechanical system.</td>
</tr>
<tr>
<td>b) Perimeter planning.</td>
<td>b) Too many columns &amp; restrictive in planning.</td>
<td>b) Complete freedom for open spaces along perimeter.</td>
</tr>
<tr>
<td>c) Articulation of the tower.</td>
<td>c) Restrictive.</td>
<td>c) Very flexible.</td>
</tr>
<tr>
<td><strong>III. Cost</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Quantity of steel.</td>
<td>a) Higher steel tonnage.</td>
<td>a) Less steel tonnage.</td>
</tr>
<tr>
<td>b) Cost/ton.</td>
<td>b) May be lower.</td>
<td>b) Competitive.</td>
</tr>
<tr>
<td>c) Erection time.</td>
<td>c) Longer because more erection pieces.</td>
<td>c) Shorter time to erect. Few pieces to erect.</td>
</tr>
<tr>
<td><strong>IV. Innovations</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>b) Up &amp; down construction.</td>
<td>b) Impractical.</td>
<td>b) Very plausible.</td>
</tr>
</tbody>
</table>
different failure mechanisms shown in Figure 3.2 are considered. The bracing members are sized in such a way that the gravity load carrying capacity of the system is not impaired. For bracing type I, in which case the bracing is in tension under gravity load, no failure mechanism needs to be checked.

Figure 3.2 Configuration of braced frames showing different failure mechanisms

The outrigger beams have to perform three functions:

1) they have to support the design floor loads;

2) the outrigger beams along with core and perimeter columns have to act as a ductile moment resisting frame to carry a minimum of 25% design code level forces without
the presence of interior core bracing;

3) the stiffness of the beam should be such so as to create effective linkage between the interior core and the perimeter columns to provide effective overturning resistance to the seismic loads (Banavalkar, 1991).

In high-rise buildings, it is always cost effective to minimize floor-to-floor height. The depth of outrigger beams is therefore dictated by the restriction on the ceiling cavity. To achieve these goals, the beams were offset into the floor and were notched at midspan to allow for the passage of the mechanical ducts. These features eventually have effects on the inelastic modelling of the structure.

Heavily loaded perimeter and core columns provide overturning resistance to the entire structure generated by lateral loads. The perimeter column capacity was checked against full plastification of outrigger beams. Perimeter columns are standard I-sections, while core columns are built-up sections with crucifix shape.

The floors, acting as diaphragms, are connected to the frames by shear studs to transfer lateral shear forces to the frame. The long span floor framing is structured in such a way that the main columns participating in the lateral load resisting system are heavily loaded by gravity load. Composite galvanized floor deck is 51 mm deep, except at ground floor and level “A” where it is 76 mm deep. In this way, all lateral forces were expected to be resisted by the braced core and the four moment resisting frames. Typical floor plan are in square shape with 47.45 m sides. Typical story heights are 3.96 m. A typical story plan and column details of the building are shown in Figs. 3.3. Selected elevations and details of the building are presented in Appendix A.
Figure 3.3 Typical floor plan and details of the columns of the FWT
Basement of the building is enlarged and has irregular floor plan. Columns in basement are steel section encased in concrete. A 610-mm-thick (2-ft.) concrete wall supporting the parking ramps is not part of the lateral force resisting system for the tower, but it does provide lateral stiffness.

The building foundation is concrete spread footings (2.74 to 3.35 m thick), supporting the steel columns with 127mm thick concrete slab on grade. The type of soil is very stiff shale or sandstone and has allowable bearing pressure of 718.2 kpa (15000 lbs/sqft).

3.3 Building Materials

From the general notes on design plans, all structural steel framing including columns use ASTM A-572 (grade 50). For properties of structural steel and other materials please refer to Table 3.2.

**Table 3.2: Properties of construction materials**

<table>
<thead>
<tr>
<th>Member</th>
<th>Nominal Strength (MPa)</th>
<th>Expected Strength (MPa)</th>
<th>Modulus of Elasticity E(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM A-572 (GRADE 50)</td>
<td>fy=345</td>
<td>380</td>
<td>200,000</td>
</tr>
<tr>
<td>Large tower footings and water tank</td>
<td>fc ‘=34.5</td>
<td>NA</td>
<td>28,960</td>
</tr>
<tr>
<td>Slab on grade</td>
<td>fc ‘=20.7</td>
<td>NA</td>
<td>22,753</td>
</tr>
<tr>
<td>All other concrete</td>
<td>fc ‘=27.6</td>
<td>NA</td>
<td>25,510</td>
</tr>
</tbody>
</table>

3.4 Instrumentation

The Figueroa at Wilshire Tower was instrumented by the California Strong Motion Instrumentation Program (CSMIP) in 1990. Twenty sensors in total were installed on different levels of the building to measure the global building response to earthquakes, including the translational, tor-
sional, and vertical motions. For the locations of the sensors, please see Fig. 3.4.

The purpose of this instrumentation is to measure the global building response to earthquakes, specially the vertical motion of the building at the base, torsional, and translational motions at the base and upper floors. The mode shapes provided by the structural engineers are used for optimizing the sensor locations in order to determine the antinodes of: a) the first three modes of lateral motion in each direction, and b) the first two modes of torsional motion. It is expected that the contributions of the higher modes to the total response of this building are significant.

To measure torsional motion of the floor, pairs of sensors are placed in the N-S direction rather than the E-W direction because the eccentricity is larger in the E-W direction. The floor locations of the sensors are based on the mode shapes provided by the structural designer according to the computer analysis.

It is worth noting that an initial sensor layout proposal included two vertical sensors to measure the rocking motion at the “A” level (CSMIP, 1990), but these sensors were dropped because: 1) the four columns at the core are continuous from the base to the roof; 2) the soil is very stiff and the tower is very flexible, the rocking contribution to the total motion is expected to be very small; and 3) the rocking motion would be too small to be measured by accelerometers. No additional sensor to measure torsion is placed at the roof level because the lever arm between sensors is small; the distance from the center to the east side of the core is less than 10 m.
Figure 3.4 Instrumentation plan of the FWT
4.1 Recorded Earthquake Responses

The two earthquakes considered in this study are the 1991 Sierra Madre earthquake and the 1994 Northridge earthquake. The building experienced its first strong ground motion during the Sierra Madre earthquake, which had its epicenter about 25 km north-east of the building. No damage was reported after this event. The epicenter of the 1994 Northridge earthquake was located about 30 km north-west of the building. Again, no damage was reported during this earthquake.

Figure 4.1 Location of the FWT and epicenters of several earthquakes in the vicinity of the building site (SCEC_DC, 1999)
The locations of the FWT and epicenters of several earthquakes including those happened before the construction of the FWT are shown in Fig. 4.1. The epicenters of the Sierra Madre earthquake and the Northridge earthquake have similar distance to the location of the FWT. However, their directions have 90 degree difference.

The acceleration response records obtained during the Sierra Madre earthquake and the Northridge earthquake from the instrumentation scheme are shown in Fig. 4.2 and Fig. 4.3. Recorded peak acceleration values are included at the right side of figures above each record.

### 4.2 Time Domain Analysis of Recorded Motions

The acceleration records are shown in Fig. 4.2 and Fig. 4.3. The peak acceleration derived from the processed data are summarized in Table 4.1. In this table SM represents the Sierra Madre earthquake and NR represents the Northridge earthquake.

At the basement, the recorded peak horizontal accelerations are 0.08g in E-W direction and 0.09g in N-S direction during the Sierra Madre earthquake. The peak horizontal acceleration for the superstructure is 0.23g recorded by the N-S accelerometer at the roof (53rd floor). Note that the peak horizontal acceleration in the N-S direction at the 14th floor is 0.14g which equals to that at the 49th floor. These acceleration values are higher than at any other recording stories, specially the 35th floor. This is a clear evidence of participation of higher modes in the building response.

In the Northridge earthquake, the recorded peak horizontal acceleration at the basement are
<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Location</th>
<th>Max. Accel. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level E:</td>
<td>Center of Tower - Up</td>
<td>0.04</td>
</tr>
<tr>
<td>E. End of</td>
<td>Tower - N</td>
<td>0.07</td>
</tr>
<tr>
<td>Level A:</td>
<td>Center of Tower - E</td>
<td>0.10</td>
</tr>
<tr>
<td>E. End of</td>
<td>Tower - N</td>
<td>0.11</td>
</tr>
<tr>
<td>14th Floor:</td>
<td>Center - E</td>
<td>0.11</td>
</tr>
<tr>
<td>E. Wall - N</td>
<td></td>
<td>0.14</td>
</tr>
<tr>
<td>22nd Floor:</td>
<td>E - E</td>
<td>0.10</td>
</tr>
<tr>
<td>E. Wall - N</td>
<td></td>
<td>0.12</td>
</tr>
<tr>
<td>35th Floor:</td>
<td>Center - E</td>
<td>0.08</td>
</tr>
<tr>
<td>E. Wall - N</td>
<td></td>
<td>0.11</td>
</tr>
<tr>
<td>49th Floor:</td>
<td>Center - E</td>
<td>0.09</td>
</tr>
<tr>
<td>E. Wall - N</td>
<td></td>
<td>0.12</td>
</tr>
<tr>
<td>Roof:</td>
<td>Center - E</td>
<td>0.17</td>
</tr>
<tr>
<td>E. Wall - N</td>
<td></td>
<td>0.23</td>
</tr>
</tbody>
</table>

Trigger Time: 14:43:59.5 GMT  
Structure Reference Orientation N=355°

Figure 4.2 Recorded accelerations during the Sierra Madre earthquake (Huang, et al., 1991)
<table>
<thead>
<tr>
<th>Level</th>
<th>Location</th>
<th>Max. Accel.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level E</td>
<td>Center of Tower - Up</td>
<td>0.114 g</td>
</tr>
<tr>
<td>Level E</td>
<td>Center of Tower - E</td>
<td>0.112 g</td>
</tr>
<tr>
<td>Level E</td>
<td>Center of Tower - N</td>
<td>0.140 g</td>
</tr>
<tr>
<td>Level E</td>
<td>E. End of Tower - N</td>
<td>0.154 g</td>
</tr>
<tr>
<td>Level A</td>
<td>Center of Tower - E</td>
<td>0.125 g</td>
</tr>
<tr>
<td>Level A</td>
<td>Center of Tower - N</td>
<td>0.171 g</td>
</tr>
<tr>
<td>Level A</td>
<td>E. End of Tower - N</td>
<td>0.183 g</td>
</tr>
<tr>
<td>14th Floor</td>
<td>Center - E</td>
<td>0.124 g</td>
</tr>
<tr>
<td>14th Floor</td>
<td>Center - N</td>
<td>0.187 g</td>
</tr>
<tr>
<td>22nd Floor</td>
<td>Center - E</td>
<td>0.124 g</td>
</tr>
<tr>
<td>22nd Floor</td>
<td>Center - N</td>
<td>0.186 g</td>
</tr>
<tr>
<td>22nd Floor</td>
<td>E. Wall - N</td>
<td>0.188 g</td>
</tr>
<tr>
<td>35th Floor</td>
<td>Center - E</td>
<td>0.108 g</td>
</tr>
<tr>
<td>35th Floor</td>
<td>Center - N</td>
<td>0.217 g</td>
</tr>
<tr>
<td>35th Floor</td>
<td>E. Wall - N</td>
<td>0.230 g</td>
</tr>
<tr>
<td>49th Floor</td>
<td>Center - E</td>
<td>0.134 g</td>
</tr>
<tr>
<td>49th Floor</td>
<td>Center - N</td>
<td>0.127 g</td>
</tr>
<tr>
<td>49th Floor</td>
<td>E. Wall - N</td>
<td>0.150 g</td>
</tr>
<tr>
<td>Roof</td>
<td>Center - E</td>
<td>0.227 g</td>
</tr>
<tr>
<td>Roof</td>
<td>Center - N</td>
<td>0.406 g</td>
</tr>
</tbody>
</table>

Figure 4.3 Recorded accelerations during the Northridge earthquake (Shakal, et al., 1994)
Table 4.1: Peak accelerations (g) recorded during two earthquakes

<table>
<thead>
<tr>
<th>Story</th>
<th>SM N-S</th>
<th>SM E-W</th>
<th>SM Torsional</th>
<th>NR N-S</th>
<th>NR E-W</th>
<th>NR Torsional</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>0.28</td>
<td>0.20</td>
<td>NA</td>
<td>0.40</td>
<td>0.23</td>
<td>NA</td>
</tr>
<tr>
<td>49th</td>
<td>0.15</td>
<td>0.12</td>
<td>0.04</td>
<td>0.13</td>
<td>0.13</td>
<td>0.06</td>
</tr>
<tr>
<td>35th</td>
<td>0.13</td>
<td>0.10</td>
<td>0.03</td>
<td>0.21</td>
<td>0.11</td>
<td>0.03</td>
</tr>
<tr>
<td>22nd</td>
<td>0.15</td>
<td>0.13</td>
<td>0.05</td>
<td>0.19</td>
<td>0.12</td>
<td>0.06</td>
</tr>
<tr>
<td>14th</td>
<td>0.17</td>
<td>0.14</td>
<td>NA</td>
<td>0.19</td>
<td>0.12</td>
<td>NA</td>
</tr>
<tr>
<td>“A” Level</td>
<td>0.14</td>
<td>0.12</td>
<td>0.02</td>
<td>0.17</td>
<td>0.13</td>
<td>0.04</td>
</tr>
<tr>
<td>“E” Level</td>
<td>0.11</td>
<td>0.09</td>
<td>0.05</td>
<td>0.14</td>
<td>0.11</td>
<td>0.08</td>
</tr>
</tbody>
</table>

0.11g in E-W direction and 0.14g in N-S direction. The peak horizontal acceleration for the superstructure is 0.41g recorded by the N-S accelerometer at the roof (53rd floor).

Linear-elastic response spectra of horizontal ground motions recorded at the ground floor of the building (5% damping) are provided in Fig. 4.4 and Fig. 4.5. These response spectra will be helpful in understanding the frequency range of energy input for each ground motion. The response spectra of the Sierra Madre earthquake show the peak of energy input between 0.2 sec to 0.6 sec. The response spectra of the Northridge earthquake show the peak of energy input between 0.5 sec to 5 sec.
Figure 4.4 Response spectra of recorded ground floor motions during the Sierra Madre earthquake

Figure 4.5 Response spectra of recorded ground floor motions during the Northridge earthquake
4.3 Frequency Domain Analysis

Detailed frequency domain analyses of recorded responses were performed to determine the dynamics characteristics of the building. The probable natural frequencies, damping ratios and mode shapes of the FWT were estimated based on a Fourier Transform analysis of recorded motions. The results are presented in section 4.3.2.

4.3.1 Theory

In this study, the natural periods, damping ratios and mode shapes were identified using computer program ME'scope (The ME'scope, 1998). The strong motion records were used to calculate Frequency Response Functions (FRF's) and coherence functions between recorded accelerations. The accelerations frequency response function is the Fourier transform of the unit-impulse response function which is usually used to describe the dynamic properties of physical systems. From the frequency response functions the natural frequencies and the mode shapes are derived.

The physical interpretation of the frequency response function is straightforward. For a harmonic excitation at a particular frequency, it is intuitively obvious that the ratio of output motion to input force would be greater at the resonant frequency than at other frequencies. Based on the same reasoning, when looking at the FRFs of the building response, the peaks may indicate possible modes of the structure. The frequency response functions are calculated by taking the record at the base as the input into the structure and all other measurements as the outputs.

In ME'scope, a MIMO(Multi-Input-Multi-Output) model is constructed as a frequency domain
model, where Linear Spectra (Fourier Transforms) of multiple inputs are multiplied by elements of a FRF matrix to yield the Linear Spectra of multiple outputs. It can be written as:

\[ \{ Y(\omega) \} = [H(\omega)] \{ X(\omega) \} \]  (4.1)

\( \{ Y(\omega) \} \) is the vector of linear spectra of outputs; \([H(\omega)]\) is the FRF matrix; \( \{ X(\omega) \} \) is the vector of linear spectra of inputs and \( \omega \) is an independent frequency variable (in rad/sec).

Each FRF in the MIMO model defines the dynamic properties between a single input DOF and a single output DOF. FRF measurements are usually made under controlled conditions, where the test structure is artificially excited using one or more shakers. A FRF measurement is typically formed as the ratio of output motion over input force, where the measured output motion can be acceleration, velocity, or displacement.

Strictly speaking, when an acceleration record is used as the input rather than a force time history, the result is not a frequency response function but rather a dimensionless ratio of input to output motion. If the mass of the building is known, the dimensionless FRF could be divided by the mass to yield the standard FRF, since they both are output motion divided by input base shear.

To obtain FRFs, estimates of the Cross and Input Auto Spectrum matrices are calculated first. Time domain windows and overlap signal processing can be used during the calculation of these matrix estimates. Next, the FRF matrix is formed by multiplying the Cross Spectrum matrix by the inverse of the input Auto Spectrum matrix. This is done by solving the equation below:

\[ [H(\omega)] = [\{ Y(\omega) \} \times \{ X(\omega) \}^T] \times [[\{ X(\omega) \} \times \{ X(\omega) \}^T]^{-1} \]  (4.2)
where the superscript "T" denotes "transpose of the vector".

Modal parameters are identified by curve-fitting the FRFs. Curve-fitting is a process of matching a parametric model of a FRF to the measured data. The unknown parameters of the parametric model are the modal frequencies, modal damping and modal residues (mode shape components) such as shown in equation 4.3.

\[
[H(\omega)] = \sum_{k=1}^{N} \left[ \frac{[R(k)]}{j \cdot \omega - p(k)} + \frac{[R^*(k)]}{j \cdot \omega - p^*(k)} \right]
\]  

(4.3)

where: $H(\omega)$ = FRF matrix (n by n)

$R(k)$ = Residue matrix for the kth mode (n by n)

$p(k)$ = pole location for the kth mode = $\sigma(k) + j \omega(k)$

$\sigma(k)$ = modal damping for the kth mode (rad/sec, or Hz)

$\omega(k)$ = modal frequency for the kth mode (rad/sec, or Hz)

$n$= number of DOFs of the FRF model

* denotes the complex conjugate

j- denotes the imaginary axis in the complex plane (= $\sqrt{-1}$)

$N$ = number of modes considered in the analysis

The outcome of curve-fitting is a set of modal parameters for each mode that is identified in the frequency range of the measurements. Modal parameter estimation is done in the following steps: 1) Determine the number of modes in a frequency band by visual inspection of the FRFs,

2) Estimate modal frequency and damping for the modes in a frequency band,
3) Estimate modal residues for the modes with frequency and damping estimates in a band,

4) Derive mode shapes from the modal parameters.

When analyzing the response of buildings to earthquakes, we always have to deal with the noise problem. The recorded signals are not as “clean” as those generated by the computer simulation or even from a lab experiment. Hence the observation of frequencies could be difficult. Past research experience provides us with a number of useful tools. We know from existing theory that when there is a natural frequency, the data will exhibit the following features:

1. the real component of the FRF will be zero;

2. the imaginary component of the FRF will peak;

3. the magnitude of the FRF will peak;

4. the phase of the FRF will be -90 degrees;

5. the coherence will usually drop;

6. the vibration shape will exhibit properties of the mode shape.

4.3.2 Dynamic Behavior from Seismic Response

Selected amplitudes of FRFs derived from the Northridge records are shown in Fig. 4.6. These FRFs are between the acceleration records in the E-W direction of the building. The amplitude of the FRFs does not have unit since the FRFs are derived by \( \frac{\text{output acceleration}}{\text{input acceleration}} \). The FRFs from the real earthquake records are different from those derived from lab testing or theoretical input. However, at least three peaks are clearly observed from these FRFs therefore implies three possible natural frequencies for the structure. A MIMO system identification procedure has been implemented and the following observations are pertinent to the find-
Figure 4.6 FRFs from the Northridge earthquake
(Note: m stands for milli)

1. The acceleration records recorded during both the Sierra Madre earthquake and the Northridge earthquake have been given in Fig. 4.2 and 4.3. The torsional acceleration record is then obtained by subtracting the centre location signal from the edge location signal on the same level (e.g. channel 4 - channel 3). The comparisons of the torsional acceleration records and their power spectra between the “A” level and the “E” level (see Fig 4.7) show that the magnitude of the “E” level torsional record is much larger than the magnitude of the “A” level torsional record. This observation goes against the usual dynamic assumption of a rigid body basement for ordinary building design (Schueller, 1976). However for high-rise build-
ings the rigid body assumption may not hold because of the complexity of the basement. For the FWT this discrepancy could be caused by: 1) the isolated column foundation which made the slab at the “E” level be more flexible than that at the “A” level therefore the signals are much more prominent at the “E” level and 2) the long concrete wall (shown in Fig 3.4) supporting the parking ramp which may significantly reduce the torsional response.

Furthermore, the power spectra show high energy level at the “E” level between 6Hz to 7Hz and between 8.9Hz to 9.2Hz. These high frequencies do not contribute significantly to high-rise building analyses, which require low frequencies (long periods). Hence the modal analysis will be based on the records from the “A” level as the input record. This in effect isolates the superstructure as the system to be identified and excludes the effect of any soil-structure interaction. The translational acceleration records do not show such behavior as of the tor-
2. The building periods derived from the ME'scope program are shown in Table 4.2. For reference, the periods used for design of the building (Banavalkar, 1992) are also included in the table. The last column in the table shows the percentage change of derived periods between the Sierra Madre earthquake to the Northridge earthquake. The maximum change is less than 13 percent. While the natural periods in the E-W direction between two earthquakes remain almost constant (the -1.7% change for N-S(1) is negligible), the natural periods in the N-S direction during the Northridge earthquake are longer than those during the Sierra Madre earthquake.

There are several possible reasons for the observed tendency such as an increase of building mass and/or a decrease of building stiffness. Previous studies (Safak and Celebi, 1991) also showed that larger excitation tends to lengthen the periods of the structure. In this case, the Northridge earthquake has a relatively larger amplitude of excitation than the Sierra Madre earthquake.

The first two mode shapes in each direction of the building from the Sierra Madre earthquake and the Northridge earthquake are shown in Fig. 4.8 and Fig. 4.9.

3. The natural periods of this high-rise building in both translational directions are around 6 seconds. Therefore a low frequency of 0.171Hz(5.85sec) must be detectable in the FRFs. In order to use the FRFs to identify the fundamental natural frequency, the frequency step for the
computed FRFs must be less than 0.015Hz. However signal processing theory states that the frequency step should be the inverse of the total time of the time domain signal. To reach a frequency step of 0.0125Hz, a minimum span of 80 seconds must be considered for the time-history records.

4. The trend of damping ratios in both earthquakes is that the lowest frequency has the highest damping ratio, which can be explained by that the first mode of each direction bears the major part of energy dissipation. Damping ratios are shown in Table 4.3.
Signal processing theory using Fourier Transform depends on the assumption of linear system response, which has traditional viscous damping. However, the actual damping mechanism is far more complicated and the viscous damping has a serious deficiency in that the energy loss per cycle at a fixed displacement amplitude is dependent upon the response frequency. Experiments on structural metals indicate that the energy dissipated internally in cyclic straining of the material is essentially independent of the cyclic frequency (Chopra, 1995).
Table 4.2: Periods (sec) of the first three modes in each direction

<table>
<thead>
<tr>
<th>Direction</th>
<th>Design value</th>
<th>Sierra Madre</th>
<th>Northridge</th>
<th>Percent change</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-W(1)</td>
<td>6.73</td>
<td>5.47</td>
<td>6.06</td>
<td>+10.8</td>
</tr>
<tr>
<td>N-S(1)</td>
<td>6.59</td>
<td>5.85</td>
<td>5.75</td>
<td>-1.7</td>
</tr>
<tr>
<td>Torsional(1)</td>
<td>6.36</td>
<td>4.55</td>
<td>4.82</td>
<td>+5.9</td>
</tr>
<tr>
<td>E-W(2)</td>
<td>2.08</td>
<td>1.71</td>
<td>1.86</td>
<td>+8.8</td>
</tr>
<tr>
<td>N-S(2)</td>
<td>2.11</td>
<td>1.74</td>
<td>1.74</td>
<td>0</td>
</tr>
<tr>
<td>Torsional(2)</td>
<td>N.A.</td>
<td>1.62</td>
<td>1.82</td>
<td>+12.4</td>
</tr>
<tr>
<td>E-W(3)</td>
<td>1.13</td>
<td>0.92</td>
<td>0.96</td>
<td>+4.3</td>
</tr>
<tr>
<td>N-S(3)</td>
<td>1.18</td>
<td>0.91</td>
<td>0.91</td>
<td>0</td>
</tr>
<tr>
<td>Torsional(3)</td>
<td>N.A.</td>
<td>0.94</td>
<td>1.04</td>
<td>+10.6</td>
</tr>
</tbody>
</table>

Table 4.3: Damping ratios of the first three modes in each direction

<table>
<thead>
<tr>
<th>Direction</th>
<th>Sierra Madre</th>
<th>Northridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-W(1)</td>
<td>6.7%</td>
<td>1.6%</td>
</tr>
<tr>
<td>N-S(1)</td>
<td>4.4%</td>
<td>5.5%</td>
</tr>
<tr>
<td>E-W(2)</td>
<td>0.01%</td>
<td>2.0%</td>
</tr>
<tr>
<td>N-S(2)</td>
<td>3.2%</td>
<td>0.5%</td>
</tr>
<tr>
<td>E-W(3)</td>
<td>0.14%</td>
<td>1.2%</td>
</tr>
<tr>
<td>N-S(3)</td>
<td>1.9%</td>
<td>1.2%</td>
</tr>
</tbody>
</table>

When the records of Northridge earthquake are used to compute the FRFs, discrepancies between the imaginary part and the magnitude of the FRFs are observed. This implies the existence of complex damping. Therefore the damping ratios derived here can only be treated as approximation of damping effects evaluated as equivalent viscous damping. The negative damping ratios derived from torsional FRFs are not within the normal range (viscous damping is always positive in order to dissipate energy) therefore have been eliminated.
5. The FRFs can also be expressed in terms of modal pole locations and mode shapes:

\[
H(\omega) = \sum_{k=1}^{\text{modes}} \left[ \frac{A(k)\{u(k)\}^T}{j\omega - p(k)} \right] + \frac{A^*(k)\{u^*(k)\}^T}{j\omega - p^*(k)}
\]  \hspace{1cm} (4.4)

A(k) in the equation is the scaling constant for mode k and \{u(k)\} is the “kth” modal vector. A(k) can be used to resemble the modal participation factor times the dynamic amplification factor. It accounts for both the structural system characteristics and the excitation information. For each earthquake the frequency components are unique therefore the dynamic amplification factor is unique.

Scaling constants derived by this MIMO model are shown in Table 4.4.

**Table 4.4: Scaling Constants of Residue Matrix**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Sierra Madre</th>
<th></th>
<th>Northridge</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E-W</td>
<td>N-S</td>
<td>TORSION</td>
<td>E-W</td>
</tr>
<tr>
<td>First</td>
<td>2.9</td>
<td>6.8</td>
<td>0.018</td>
<td>0.5</td>
</tr>
<tr>
<td>Second</td>
<td>6.5</td>
<td>8.3</td>
<td>0.016</td>
<td>5.0</td>
</tr>
<tr>
<td>Third</td>
<td>0.5</td>
<td>6.2</td>
<td>0.025</td>
<td>3.0</td>
</tr>
</tbody>
</table>

The magnitudes are compared in each direction. During both earthquakes the second modes in the E-W direction have larger magnitude than that of the other modes, which implies bigger contribution to the building response.
4.4 Conditioning of Raw Records

During the system identification process the coherence functions between any two of the input record signals are observed to be close to 1. (A coherence function of unity between pairs of input records implies complete linear dependence.) Hence the records from the two translational directions are highly correlated. This correlation might affect the results of the modal analysis. To reduce the correlation, computer program DADiSP (DADiSP 4.1, 1997) was used to implement the conditioning algorithm developed by Bendat and Piersol (1993) to obtain conditioned records from the original ones. Details of this conditioning algorithm are presented in Chapter 10 of their book "Engineering Applications of Correlation and Spectral Analysis.

To obtain conditioned records, the equivalent of least-squares prediction operations are used to remove the linear effects of one or more input records from the data. In physical terms, system identification using conditioned records is equivalent to determining the dependence between one of the inputs and the output when the other input is turned off, assuming all correlated effects between the two inputs originated from the other input are turned off.

The coherence functions are reduced significantly. Comparison of the coherence between the original and the conditioned input records is shown in Fig. 4.10. The dotted line is for the coherence function of the original records, while the solid line is the coherence function of the conditioned records. It is clear that the conditioning process reduces the dependence of two input records significantly.

Frequency domain system identification theory require that the input records are not correlated.
The conditioning process performed above will help to justify the use of the frequency domain system identification theory. However, since the magnitude of the input record signals are also reduced after the conditioning, these records are difficult to be utilized on the analyses of modal dampings and mode shapes. The conditioned records are shown in Fig. 4.11. After removing the correlations from the second and the third acceleration records, it is clear that the general magnitude is reduced to less than half of the original records. This may cause difficulty when deriving modal damping and mode shapes.

Conditioned records are also analyzed for the modal frequencies and damping ratios. Results are shown in Table 4.5. There are small variations between the modal frequencies derived from the
Table 4.5: Periods (sec) and dampings (%) from the conditioned records

<table>
<thead>
<tr>
<th>Direction</th>
<th>Periods</th>
<th>Damping Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sierra Madre</td>
<td>Northridge</td>
</tr>
<tr>
<td>E-W(1)</td>
<td>5.62</td>
<td>5.94</td>
</tr>
<tr>
<td>N-S(1)</td>
<td>5.88</td>
<td>5.79</td>
</tr>
<tr>
<td>Torsional(1)</td>
<td>4.55</td>
<td>4.40</td>
</tr>
<tr>
<td>E-W(2)</td>
<td>1.72</td>
<td>1.81</td>
</tr>
<tr>
<td>N-S(2)</td>
<td>1.69</td>
<td>1.87</td>
</tr>
<tr>
<td>Torsional(2)</td>
<td>1.58</td>
<td>1.74</td>
</tr>
<tr>
<td>E-W(3)</td>
<td>0.92</td>
<td>1.00</td>
</tr>
<tr>
<td>N-S(3)</td>
<td>0.90</td>
<td>0.96</td>
</tr>
<tr>
<td>Torsional(3)</td>
<td>0.97</td>
<td>0.97</td>
</tr>
</tbody>
</table>
original records and those from the conditioned records. These variations are acceptable in this study. However most of the damping ratios from the conditioned records are negative. Therefore improvements on the method of conditioning shall be investigated in the future.

4.5 Conclusions

The study of the dynamic behavior of the Figueroa at Wilshire Tower during the two earthquakes yields valuable information. Using signal processing techniques, the periods, damping ratios and mode shapes of the building were found. Values are close to the analytical results obtained by the structural designer.

The tendency of frequency variation between two earthquakes has been observed. This variation will be considered in later linear time history analysis of the building. The damping ratio has long been an argument to the theoretical analysis of real structures. From this study, it is shown that damping ratio can be identified from the real earthquake records.

The modal participation factor and the dynamic amplification factor are very important in deciding the dynamic response of the structure for certain earthquake excitations. The magnitude of the Residue function is shown to represent the modal participation factor times the dynamic amplification factor in this study. During both earthquakes the second modes in the E-W direction have larger magnitude than that of the other modes, which implies a bigger contribution of the second mode to the building response. Since the dynamic amplification factor heavily depends on the excitation, the lack of certain frequency components in the earthquake excitation may cause the related mode shape to disappear in the building response. In this study no lack of fre-
quency components is observed.

Conditioning of original records was performed to remove correlation between input excitations. Coherence functions between any two input excitations are significantly reduced. These conditioned records are the signals that are more appropriate to be used to compute FRFs. However the magnitudes of input excitations were also reduced, causing problems in deriving mode shapes and modal damping ratios. Improvement of this conditioning technique needs to be investigated.

The estimated parameters will be used to assess the reliability of parameters in computer models for the building studied. Details will be presented in Chapter 6.
Chapter 5 DESCRIPTION OF THE NONLINEAR ANALYSIS PROGRAM

5.1 Introduction

CANNY-E (Li, 1996) is a general purpose computer program for 3-dimensional nonlinear static and dynamic analyses of reinforced concrete and steel building structures. The program is based on a lumped plasticity model and has the capacity for analyzing large structures. Some general features of the program are summarized in the following paragraphs.

The structure is idealized as a number of nodes, or joints, connected by a number of deformable elements. The program has both rigid and flexible floor modeling capabilities. Geometric nonlinearities, i.e., large deformation, are not included. Although the analysis is limited to small deformations, P-delta effects can be considered. In the analysis every member in the structure is treated as a massless straight linear element. The mass of structure is lumped at structural joints or concentrated at the center of gravity of every floor level if a rigid floor slab is assumed. Initial conditions can be defined before the start of static and dynamic analyses.

The nonlinear dynamic analysis is conducted step by step in a specified small time interval. The time interval is selected considering the characteristics of the input motion and taking into consideration the following: (1) the nonlinear relations between forces and resultant displacements; (2) the compatibility relation of displacements at each structural joint; (3) the equilibrium of forces at each structural joint; and (4) differential relation among time response functions. The current version (CANNY-E, 1996) of the program used for this study does not iterate during time
steps to account for the stiffness changes, but the unbalanced force at the end of each time step is calculated and added to the next time step.

The equation of motion includes all degrees of freedom with or without mass of the structure. The program does not implement any static condensation technique. The skyline matrix technique is used in the program to assemble and memorize the structural matrices. Matrix decomposition due to the change of stiffnesses is carried out by the Square-root-free Choleski method. An internal optimum renumbering system was developed and utilized in the program to minimize the total size of the skyline matrix. Such approach reduces the required size of computer core memory and speeds up the computation. The incremental equation of motion is expressed as:

\[
[M]\{\Delta \dot{X}\} + [C]\{\Delta \dot{X}\} + [K]\{\Delta X\} = \{\Delta F_e\} - \{F_u\} + \{\Delta F_t\} \tag{5.1}
\]

where, \([M]\) is the diagonal mass matrix of the system,

\([C] = a_k[K] + a_m[M] + a_0[K_0]\) is a Rayleigh-type viscous damping matrix,

\(\{\Delta X\}\) is the vector of displacement increments of the structure relative to the base,

\(\{\Delta \dot{X}\}\) is the increment of the velocity vector,

\(\{\Delta \ddot{X}\}\) is the increment of acceleration vector,

\([K]\) is the instantaneous stiffness matrix,

\([K_0]\) is the initial stiffness matrix,

\(\{\Delta F_e\}\) is the increment of earthquake inertia force vector,

\(\{F_u\}\) is the unbalanced force vector at the beginning of the present time interval,

\(\{\Delta F_t\}\) is the incremental time-varying external load vector.
Two numerical procedures, the Newmark’s Beta-Method and the Wilson’s Theta-Method are employed and available in the program to solve the equations of motion. Linear variation of acceleration is assumed in the numeric integration of the equations of motion during a specified time interval. Newmark’s Beta-Method was used in this study because of its stability on convergence.

5.2 Elements

The elements used in the program to represent the structural members are beams, columns, shear panels, trusses, cables and constraint spring elements.

A beam element is limited to uniaxial bending with optional shear and axial deformation. The inelastic flexural deformation of the beam element is assumed to be concentrated at its ends, and represented by the rotation of two nonlinear bending springs. The shear and axial deformations of beam are approximated by independent shear and axial springs placed at midspan. The beam element does not include the interactions among the bending, shear and axial forces. The beam axial deformation option can not be included if it is in a rigid floor slab.

Columns are idealized by one of the three models described here: a one-component model for uniaxial bending, a biaxial bending model, or a multi-spring model. Interaction between the axial load and bending moment can be taken into account by only the multi-spring model. Shear and torsional deformation of the column element can be included in the analysis.
One of the important features of this program is its multispring column model (Fig. 5.1). It was

3D ELEMENT MODEL

3D COLUMN MODEL

Figure 5.1 3D multispring column model (after Li, 1996)
developed based on a modification of the model originally proposed by Lai et al. (1984) to simulate the flexural behavior of reinforced concrete columns under varying axial load and bi-directional lateral load reversal. The column was idealized to be a linear element with its length equal to the column clear height and two multi-spring elements with zero length at the base and the top. The spring properties were determined based on the material properties and section geometries. The number of steel and concrete springs were selected according to the section shape and location of reinforcing bars.

A link element connects two nodes and subjects them to tension and compression with no bending. The link element can be defined in a three-dimensional space or a two-dimensional plane. The cable element has a start node and a terminal node. Between the start node and the terminal node, middle nodes can be specified to change its direction in space. The cable has stiffness in tension only.

The constraint spring element is a simple one-component element. It is used to change any one of the displacement components at a node. It can be used to analyze a substructure or partial frames without changing the properties of structural members.

5.3 CANNY Hysteresis Models

The program includes a number of hysteresis models expressing nonlinear force-displacement relationships. Some are used for one-component models to simulate the inelastic behavior of uniaxial bending, shear and axial deformation, and some used for multi-spring model to represent the behavior of biaxial-bending and axial force interaction.
The uniaxial hysteresis models included in the program are summarized in Table 5.1. A detailed description of each hysteresis model is presented in the CANNY-E users' manual.

5.4 More Information about the Program

Program CANNY-E is comprised of three programs running in DOS interface: “PRECANNY”, “CANNY”, and “PSCANNY”.

PRECANNY is a pre-processing program for reading the input data file and preparing it for the main program. It reads the free format text data file and performs memory allocation, automatic renumbering, and initialization of element and structural matrices. It generates a binary data file, called “CANNY data file”. CANNY is the main program where all the numerical computations are carried out. It stores the calculated results into a binary file, called “Binary result file”. PSCANNY is a post-processing program which reads the calculated results from the binary result file and transforms them into text-format file.

An additional graphical interface and animation module for the program called VCANNY is used to check the contents of the CANNY data file and show peak values of forces for each element.

The program uses the computer core memory and part of the hard disk as virtual memory if required to store the stiffness matrices and conduct the analysis. It does not have any limitation
Table 5.1: Uniaxial hysteresis model options and number of parameters

<table>
<thead>
<tr>
<th>Hysteresis Number (HN)</th>
<th>Hysteresis Model</th>
<th>Bilinear</th>
<th>Trilinear</th>
<th>Tension/compression only</th>
<th>Stiffness degradation</th>
<th>Strength deterioration</th>
<th>Slip</th>
<th>Softening</th>
<th>Number of Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Linear elastic model</td>
<td>x</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>1T</td>
<td>Tension only elastic model</td>
<td>x</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>1C</td>
<td>Compression only elastic model</td>
<td>x</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>degrading bilinear model</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>Modified Clough model</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>Degrading bi/tri-linear model</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>Cross-peak bilinear model</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>Cross-peak bi/tri-linear model</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>Cross-peak pinching model</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>3</td>
</tr>
<tr>
<td>8</td>
<td>Origin-oriented model</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>Peak-oriented model</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>Bilinear slip model</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>2</td>
</tr>
<tr>
<td>11</td>
<td>Bi/tri-linear slip model</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>2</td>
</tr>
<tr>
<td>12</td>
<td>Bilinear elastic model</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>Bi/tri-linear elastic model</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>CANNY simple model</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>2</td>
</tr>
<tr>
<td>15</td>
<td>CANNY sophisticated model</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>7</td>
</tr>
<tr>
<td>16</td>
<td>Maeda bi/tri-linear model</td>
<td>0</td>
<td>0</td>
<td>x</td>
<td>0</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>2</td>
</tr>
<tr>
<td>17</td>
<td>Axial stiffness model 1</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1</td>
</tr>
<tr>
<td>18</td>
<td>Axial stiffness model 2</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1</td>
</tr>
</tbody>
</table>

0, functional    x, not functional    --, not applicable
regarding to the size of the structure, provided that sufficient core memory in hard drive space is available.

The program is very efficient regarding to the computational time for an analysis. A time history analysis of the building studied subjected to 180 seconds (18,000 time steps) of shaking takes about 3 hours on a Pentium 266 PC.

The CANNY program has been applied in a number of structural analyses of actual buildings and research studies. It has been used in the three-dimensional nonlinear dynamic seismic analyses of a seven story reinforced concrete building (Rahmatian 1998) in Los Angeles. A computer model was set up by Khoshnevissan (Khoshnevissan 1998) using CANNY for the analysis of a bent at Oak Street bridge in Vancouver. Sewell (Sewell 1999) used a CANNY model to study the behavior of reinforced concrete shear wall. CANNY program has been used to analyze single and multi-story steel plate shear walls under simulated seismic loading by Rezai (Rezai 1999). Currently a low-rise steel frame buildings in Burbank, California is being investigated by Bakhtavar (1999) using CANNY. Several new commercial buildings in Japan have been analyzed and designed using CANNY (Li, 1999).
Chapter 6 THREE-DIMENSIONAL LINEAR ANALYSIS OF
THE FIGUEROA AT WILSHIRE TOWER

6.1 Introduction and Theory

The primary objective of performing a three-dimensional linear analysis is to understand the behavior of the FWT during the Sierra Madre and Northridge earthquakes, where the building is believed to have responded as a linear elastic system. The results from this analysis also help to determine the extent of contribution of the different structural components as well as the non-structural components to the overall response of the building. The linear model and the related analytical studies are used to form a basis for the development of the nonlinear model.

The mode superposition combined with the response spectrum method is one of the most popular techniques for the elastic analysis of structures. However, it is not directly applicable to inelastic analysis because the principle of superposition is no longer applicable. Direct integration time history analysis overcomes this drawback. This method consists of a step-by-step direct integration in which the time domain is discretized into a number of small time increments $\delta t$, and for each time interval the equations of motion are solved with the displacements and velocities of the previous step serving as initial functions.

Direct integration time history analysis is not normally employed as an analysis tool in practical design of high-rise buildings because it requires large computational effort. However, recent enhancements in computational techniques and the development of more sophisticated models al-
low popular application of time history analyses, based on components' behavior, which are capable of determining the behavior of three-dimensional structures in a more realistic manner.

6.2 Development of a linear model

Computer program CANNY is used for the three-dimensional linear analysis of the FWT. The program is chosen because of its ability to perform both linear and non-linear time-history analyses. The building is modeled as a combination of braced frame and moment frame consisting of 14 main column lines. The wireframe model of the building is shown in Fig. 6.1.

![Figure 6.1 CANNY model of the FWT](image)

The in-plane stiffness of the floor system was considered to be high (the thickness of the composite slabs at each floor is 50mm), and therefore, the floor diaphragms were assumed to be rigid. Under this constraint, each diaphragm can be shown to consist of three in-plane degrees of free-
dom, one rotational and two translational.

The building was assumed to be fixed at the ground level. This assumption is based on the information obtained from the analysis of the recorded motions in Chapter 4. All the beam and column centre lines were considered to coincide with column line coordinates. The masses of the beam and column members associated with each floor were calculated from the specified geometric dimensions of elements and combined with the mass of slab to determine the total story mass at corresponding floor. The estimated final mass and mass moment of inertia were lumped at the centre of mass at each floor. The estimated weight of the structure at each story level is shown in Table 6.1. The total mass of the building was estimated to be about 50927 tonnes. This information was provided by the structural designer (Banavalkar 1991).

Beams, columns and bracing members were modeled in CANNY by beam element, column element and link element. In beam and column elements, properties of flexural rotation in both ends, shear deformation and axial deformation were numbered. These numbered properties were defined later by different hysteresis models. Properties of built-up section were calculated according to steel design handbook (handbook, 1985). For elastic analysis, all the hysteresis models chosen are simple linear models. For link element, only property of axial deformation need to be defined. Contribution of panel zone of beam column connection was accounted by the rigid zone length at the element start and end node.

To evaluate the contribution of gravity load frames, an additional model was constructed taking into account the perimeter gravity load columns. Comparison of periods (periods derived by the
<table>
<thead>
<tr>
<th>Floor</th>
<th>Weight (kN)</th>
<th>Rotational Inertia (kN \cdot m^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2nd</td>
<td>16808</td>
<td>8362769</td>
</tr>
<tr>
<td>3rd</td>
<td>11938</td>
<td>4774000</td>
</tr>
<tr>
<td>4th</td>
<td>10894</td>
<td>4218800</td>
</tr>
<tr>
<td>5th-6th</td>
<td>10471</td>
<td>3994320</td>
</tr>
<tr>
<td>7th-9th</td>
<td>10204</td>
<td>3801671</td>
</tr>
<tr>
<td>10th-13th</td>
<td>10007</td>
<td>3733875</td>
</tr>
<tr>
<td>14th-17th</td>
<td>9808</td>
<td>3665661</td>
</tr>
<tr>
<td>18th-19th</td>
<td>9710</td>
<td>3631765</td>
</tr>
<tr>
<td>20th</td>
<td>10646</td>
<td>3690051</td>
</tr>
<tr>
<td>21st-30th</td>
<td>9599</td>
<td>3570164</td>
</tr>
<tr>
<td>31st</td>
<td>10354</td>
<td>3568513</td>
</tr>
<tr>
<td>32nd-33rd</td>
<td>9657</td>
<td>3517663</td>
</tr>
<tr>
<td>34th-35th</td>
<td>9486</td>
<td>3447384</td>
</tr>
<tr>
<td>36th</td>
<td>10369</td>
<td>3749583</td>
</tr>
<tr>
<td>37th-42nd</td>
<td>9078</td>
<td>3104672</td>
</tr>
<tr>
<td>43rd</td>
<td>8505</td>
<td>2693336</td>
</tr>
<tr>
<td>44th</td>
<td>8433</td>
<td>2657367</td>
</tr>
<tr>
<td>45th</td>
<td>8589</td>
<td>2670596</td>
</tr>
<tr>
<td>46th</td>
<td>8348</td>
<td>2657780</td>
</tr>
<tr>
<td>47th-48th</td>
<td>7703</td>
<td>2204690</td>
</tr>
<tr>
<td>49th</td>
<td>8316</td>
<td>2504407</td>
</tr>
<tr>
<td>50th</td>
<td>9550</td>
<td>2907479</td>
</tr>
<tr>
<td>51st</td>
<td>10845</td>
<td>2200801</td>
</tr>
<tr>
<td>52nd</td>
<td>8050</td>
<td>1391108</td>
</tr>
<tr>
<td>53rd/Roof</td>
<td>3055</td>
<td>234813</td>
</tr>
</tbody>
</table>
two models are within 5 percent difference), mode shapes and responses showed that effect of accounting gravity load frames is negligible on changing the dynamic characteristics of the computer model. Therefore the computer model shown in Fig. 6.1 does not include gravity load frames.

6.3 Calibration of the linear model and period sensitivity.

The live load contributions to the dynamic mass used to calibrate the models to study the response of the building during the Sierra Madre and the Northridge earthquakes are shown in Table 6.2. The corresponding computed fundamental periods are also included for reference.

<table>
<thead>
<tr>
<th>Model</th>
<th>Description</th>
<th>Fundamental Periods(sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sierra Madre</td>
<td>Self Weight + 50% Live Load</td>
<td>5.74/5.38/4.63</td>
</tr>
<tr>
<td>Northridge</td>
<td>Self Weight + 80% Live Load</td>
<td>6.00/5.61/4.75</td>
</tr>
</tbody>
</table>

The periods obtained from the CANNY model are compared with those from the analysis of the recorded earthquake response in Table 6.3. The computer model yields periods that are in good agreement (within 10%) with the observed building periods from the Northridge earthquake.

Views of mode shapes for the first six modes of the Northridge model are presented in Fig. 6.2. The first mode shapes in both X and Y direction show the shape of shear type structure. The second mode shapes in both X and Y direction are typical and have an inflection point at about 2/3 height of the structure. The second torsional mode shape shows a large translational curve below the mid-height of the structure.
Figure 6.2 Modes shapes for the FWT in three directions

Table 6.3: Periods (sec) of the first three modes in each direction

<table>
<thead>
<tr>
<th>Direction</th>
<th>CANNY model (SM)</th>
<th>Sierra Madre</th>
<th>CANNY model (NR)</th>
<th>Northridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-W(1)</td>
<td>5.74</td>
<td>5.85</td>
<td>6.00</td>
<td>6.06</td>
</tr>
<tr>
<td>N-S(1)</td>
<td>5.38</td>
<td>5.47</td>
<td>5.61</td>
<td>5.75</td>
</tr>
<tr>
<td>Torsional(1)</td>
<td>4.63</td>
<td>4.55</td>
<td>4.75</td>
<td>4.82</td>
</tr>
<tr>
<td>E-W(2)</td>
<td>1.65</td>
<td>1.71</td>
<td>1.85</td>
<td>1.86</td>
</tr>
<tr>
<td>N-S(2)</td>
<td>1.67</td>
<td>1.74</td>
<td>1.77</td>
<td>1.74</td>
</tr>
<tr>
<td>Torsional(2)</td>
<td>1.60</td>
<td>1.62</td>
<td>1.71</td>
<td>1.82</td>
</tr>
<tr>
<td>E-W(3)</td>
<td>0.93</td>
<td>0.92</td>
<td>0.97</td>
<td>0.96</td>
</tr>
<tr>
<td>N-S(3)</td>
<td>0.91</td>
<td>0.91</td>
<td>0.97</td>
<td>0.91</td>
</tr>
<tr>
<td>Torsional(3)</td>
<td>0.92</td>
<td>0.94</td>
<td>1.04</td>
<td>1.04</td>
</tr>
</tbody>
</table>

Damping ratios (Rayleigh damping) are critical to the analytical response of the building. For low-rise building analysis, 5 percent critical damping is usually applied to the first mode and the
second mode. If the same 5 percent damping ratios for the first mode and the second mode are applied to this high-rise building, the analytical response from the computer model is significantly different from the recorded response. Comparison of the responses using the Northridge records in the E-W direction on the 49th story is presented in Fig. 6.3. Dotted line is the recorded acceleration time history, while solid line is the computed acceleration time history. Because of the high damping ratios for the first mode and the second mode, response caused by the higher modes was significantly reduced and the amplitude of the response was significantly lower than the recorded response. Maximum response of the computer model was only about 1/5 of the maximum recorded response.

After careful comparison of the analytical responses due to different damping ratios, 5 percent critical damping for mode 1 and 2 percent critical damping for mode 9 were used for the computer model. Comparison of the analytical response and recorded response is shown in Fig. 6.4. The recorded motion used is the same as in the previous comparison. The analytical response can capture high frequencies of the recorded response very well. The maximum amplitude of the analytical response is very close to the recorded maximum amplitude.

Damping ratios for mode 1 to mode 12 are thus derived and summarized in Table 6.4.

<table>
<thead>
<tr>
<th>Mode</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period (sec)</td>
<td>6.00</td>
<td>5.61</td>
<td>4.75</td>
<td>1.85</td>
<td>1.77</td>
<td>1.71</td>
<td>1.04</td>
<td>0.98</td>
<td>0.97</td>
<td>0.74</td>
<td>0.68</td>
<td>0.66</td>
</tr>
<tr>
<td>Damping Ratio</td>
<td>0.05</td>
<td>0.047</td>
<td>0.041</td>
<td>0.021</td>
<td>0.021</td>
<td>0.021</td>
<td>0.020</td>
<td>0.020</td>
<td>0.020</td>
<td>0.022</td>
<td>0.023</td>
<td>0.023</td>
</tr>
</tbody>
</table>
6.4 Comparison of predicted and recorded time histories.

The three-dimensional responses of the building during the Sierra Madre earthquake and the Northridge earthquake were computed analytically. The purpose of such an analysis was to correlate the recorded responses with the analytical results, and to assess the effects of various members on the overall three-dimensional behavior of the structure for small to medium levels of excitation.

The response of the building was computed for the 60 seconds of the Sierra Madre record and the 180 seconds of the Northridge record. During both analyses two translational and one for torsional ground excitations were used as inputs.
6.4.1 Comparison of predicted and recorded time histories during the Sierra Madre earthquake

Figure 6.5 shows the recorded and computed absolute acceleration and relative displacement at the 49th floor during the Sierra Madre earthquake. Acceleration were chosen at the 49th floor instead of the roof because the top three floors consist only the core of the building. A good match between the two responses implies that the model is able to predict the frequency content of the actual response of the structure.
Figure 6.5 Comparison of recorded and computed roof absolute acceleration during the Sierra Madre earthquake
6.4.2 Comparison of predicted and recorded time histories during the Northridge earthquake

Figure 6.6 and Figure 6.7 show the recorded and predicted absolute accelerations and relative displacements at the 49th floor during the Northridge earthquake. The same as that for the Sierra Madre earthquake, acceleration were chosen at the 49th floor.

The correlation between predicted and recorded accelerations are better than that for displacements. This difference may be due to the way in which displacements are inferred from recorded accelerations. The integration routine may not produce the same results as those from the direct time history analyses.

Figure 6.8 and Figure 6.9 show the first 35 seconds (for comparison clarity) of recorded and predicted absolute accelerations and relative displacements at the 14th floor during the Northridge earthquake. This second comparison of motions also provides good match between recorded time histories and computed time histories.

A good match between the two responses implies that the model is able to reproduce the frequency content of the actual response of the structure. And since the time history responses of this computer model fit well with the responses from two major earthquakes that the building experienced, the computer model can be treated as a reliable representation of the structure system of the building during typical earthquake excitations. This observation sets the base of next step of the analysis.
Figure 6.6 Comparison of recorded and computed roof absolute acceleration during the Northridge earthquake
6.5 Comparison of code design values

In order to achieve a better understanding of the code design values in comparison with the dynamic analysis values, two code design procedures are followed to derive the elastic base shear for the building. The building code used in this building’s structural design is the City of Los Angeles Building Code, 1985 edition. The structural design also complied with the 1987 SEAOC recommended force. These codes conform to Uniform Building Code (UBC) 82 section 91.2312. Earthquake Regulations. These design regulations were used to evaluate the elastic static seismic force.
According to UBC82, minimum earthquake forces for structures should be calculated as:

\[ V = ZIKCSW \]  \hspace{1cm} (6.1)

\( Z \) is the numerical coefficient dependent upon the zone. \( I \) is the occupancy importance factor. \( K \) is the numerical coefficient for structural type. \( C \) is the numerical coefficient derived from the natural period. \( S \) is the numerical coefficient for site-structure resonance. \( W \) is the total dead load of the structure.
For Los Angeles located in Zone No. 4, numerical coefficient $Z = 1$. Occupancy Importance Factor $I$ for this building was chosen as 1. For a dual bracing system consisting of a ductile moment-resisting space frame and braced frames, the horizontal force factor is $K = 0.80$. Since the natural period of the building is about 5.65 seconds, the numerical coefficient $C$ is $C = \frac{1}{15 \cdot \sqrt{T}} = 0.0281$. 
From the general notes on the site plans, the type of soil under this building is very stiff shale or sandstone with bearing value of 15000 lbs/sq.ft. In such case numerical coefficient for site-structure resonance S was chosen as 1. The total weight of the building was estimated to be 50927 tons. Therefore the total seismic force for the building is:

\[ V = 1 \times 1 \times 0.80 \times 0.0281 \times 1 \times 50927 \times 9.8 = 11198 \text{kN} \quad (6.2) \]

NBCC1990 as a reference was also used to evaluate the seismic lateral force in order to compare design guidelines. In NBCC1990, the equivalent lateral seismic force representing elastic response shall be calculated as:

\[ V_e = v \cdot S \cdot I \cdot F \cdot W \quad (6.3) \]

Since the zonal velocity ratio v for Los Angeles is not available, v = 0.3 (for Victoria) is used. The seismic response factor S is

\[ S = \frac{1.5}{\sqrt{T}} = \frac{1.5}{\sqrt{5.65}} = 0.63. \]

The seismic importance factor I was chosen as 1. The foundation factor F was also chosen as 1. Therefore the elastic lateral seismic force \( V_e = 0.3 \times 0.63 \times 1 \times 1 \times 50927 \times 9.8 = 94328 \text{kN}. \)

For ductile braced frame force modification factor R should be chosen as 3.0. Hence the minimum lateral seismic force is

\[ V = \frac{V_e}{R} \cdot U = \frac{94328}{3.0} \cdot 0.6 = 18866 \text{kN}. \]

The two design codes are from different countries and different period of time. Design base shear from NBCC is shown to be higher than UBC (in about 40% reduction). However, it should be
noticed that the NBCC used an empirical load reduction factor (R) of 3 to account for the ductility of the steel braced frame. The base shear derived is not the maximum base shear that the building can resist during earthquakes.

6.6 Summary

In this chapter it has been shown that a linear model and linear analysis have reasonably captured the recorded seismic response of the FWT building during past small to medium level earthquakes. The building was modeled as a combination of braced frame and moment frame consisting of 14 main column lines. After live load and damping ratio calibration of the model, the periods derived from the computer model were shown to be in good agreement (within 10%) with the observed building periods from the Northridge earthquake.

The response of the building was then computed for the 60 seconds of the Sierra Madre record and the 180 seconds of the Northridge record. During both analyses two translational and one for torsional ground excitations were used as inputs. A good match between the computed responses and the recorded responses implied that the model was able to reproduce the frequency content of the actual response of the structure. Therefore, the computer model can be treated as a reliable representation of the structure system of the building during typical earthquake excitations.

In doing this analytical study confidence is gained toward the computer model based on recorded seismic responses. Hence the linear model and the related analytical studies will be used to form a basis for the development of the nonlinear model.
7.1 Introduction

This chapter describes the nonlinear analyses of the Figueroa at Wilshire Tower (FWT). Planar pushover and cyclic analysis of the three-dimensional model of the building were performed first. Then, three-dimensional nonlinear time history analyses of the building were conducted to simulate the building response during three earthquake excitations. Finally, comparison between the prediction from pushover analysis and responses from time history analyses was used to modify the traditional method of pushover analysis in order to obtain better prediction from pushover analysis for the ground motions considered.

The purpose of performing nonlinear analyses is to determine the response and behavior of the building during severe shaking. Therefore it is extremely important to choose the variables defining the inelastic properties of independent elements. In-depth research on previous studies and national/local guidelines was carried out to verify the credibility of these variables.

On the other hand, there is no intention of simulating any pre-defined behavior of the building by artificially changing the properties and variables of the structure model. The model is an as-built model and all the responses and conclusions discussed later are not manipulated by any form of number-picking.
7.2 Description of the Nonlinear model.

A detailed finite element model is developed based on structural drawings of the building. It has the same basic features as the linear model. The building is modeled as a combination of braced frames and moment frames consisting of 14 main column lines. The model of the building has been shown in Fig. 6.1. Floors are modeled as rigid diaphragms with three degrees of freedom for each floor. The building is assumed to be fixed at the base. The estimated mass of each floor is lumped at the center of mass of the corresponding diaphragm (mass of columns is included in floor masses), and all members are assumed to be massless elements.

Additional nonlinear model features are discussed below. Hysteresis behavior of each member is simulated by a hysteresis model expressing the nonlinear force-displacement relationship. All flexural members are modeled by a simple bilinear model, shown in Fig. 7.1 (CANNY hysteresis model #14). A more sophisticated model is available in CANNY (model #15) but its use significantly increases the computational effort. Also the study of general responses of tall buildings does not require the most sophisticated model as discussed in Chapter 2. Yielding strengths are calculated based on the expected steel yielding strength. From the general notes on the building drawings, all steel framing including columns are ASTM A-572 (grade 50). Nominal yielding strength for A-572 (grade 50) is 345MPa (50ksi). Expected steel yielding strength is assumed based on average values such as those cited in the Interim Guidelines (FEMA 1995) in accordance with the SAC reports: \( F_y = 380 \text{MPa (55ksi)} \).

The post-yielding stiffness factor is chosen as 0.02 in reference to the SAC reports. Beyond the
initial yield strength, strain hardening can increase member and connection strengths, although this increase will depend upon the member's resistance to local or lateral/torsional buckling or other modes of failure. The increase in strength due to strain hardening is generally limited to a small value that only causes a modest increase in member strength.
Strain hardening is sometimes specified to help maintain numerical convergence of nonlinear analyses, but in such cases it is important to keep the value to a minimum to avoid masking true (physical) instabilities. The strain hardening factor for the compression side of bracing elements was chosen as 0.001.

The unloading stiffness degradation factor $\gamma$ was chosen as 0.3. The unloading stiffness was determined as following:

$$K_u = K_0 \cdot \left( \frac{d_y - d'_y}{d_{\text{max}} - d'_{\text{max}}} \right)^\gamma$$  \hspace{1cm} (7.1)

where, $d_{\text{max}} \geq d_y$, $d'_{\text{max}} \leq d'_y$. The strength deterioration factor $\lambda_u$ was chosen as 0.75. The reloading yielding strength was evaluated considering the attained ductility as following:

$$\bar{F}_y = F_y \cdot \left(1 - \lambda_u \cdot \left(1 - \frac{1}{u}\right)\right)$$  \hspace{1cm} (7.2)

where, $u$ is the ductility. When $\lambda_u = 0.75$ was chosen, there would be 50% strength remained at ductility $u = 3$.

The axial force-deformation behavior of bracing members in compression are modeled as an elasto-plastic element with the yield force equal to the residual force (FEMA 274). However, an elastic analysis is necessary to determine the maximum axial force delivered to the column, the beam, and the beam-column connections.
The shear stiffness of beams and the axial stiffness of columns are assumed to be linear elastic and are modeled by linear elastic hysteresis model (CANNY model #1).

There is no panel zone element in computer program CANNY. Though it is feasible to use other software to analyze panel zone after using CANNY to compute global response of the building structure, this was not carried out during this study. First, the difficulty in defining panel zone behavior makes it hard to construct a realistic panel zone element. Dr. Krawinkler of Stanford university provided a report in SAC 95-04 with similar conclusion. Please refer to Literature Review for further information. Secondly, this research is focused on global response of the building structure therefore the detail modeling of panel zone elements is not necessary (contribution of rigid end caused by the panel zone is accounted in the computer model by defining the length of the rigid end in both beam and column elements).

Convergence was not a problem for this lumped plasticity model during time history analysis. The small time step (typically 0.01 or 0.005 second) of input acceleration records may be the reason. However, during pushover and cyclic analyses the program crashed frequently when a significant number of elements started to yield and the stiffness matrix changed significantly. Therefore, during certain critical pushover stages, a smaller time step of 0.001 second was used.

7.3 Time History Analysis

Since this building remained in the elastic range during the past earthquakes, there is no real nonlinear response records available. In order to observe the nonlinear response of the building in possible stronger earthquakes, three recorded earthquake motions from elsewhere were used to
compute the building responses.

First, a set of records from the SMAC-A station during the 1964 Niigata (Japan) earthquake was used as input excitation. These records show a major energy input around the first mode period (4.5 to 6 seconds). The second set of recorded motions used as input excitation was the Joshua Tree Fire station records during the 1992 Landers Earthquake in California. These records have a duration of at least 40 seconds of strong shaking. This set was selected because it may be of practical interest to observe the responses of high-rise buildings during this type of long duration shaking. Last, the Sylmar County Hospital station record obtained during the 1994 Northridge earthquake was used as input excitation. This is a short duration, strong impulse, near-source ground motion that may be very damaging to tall buildings.

Table 7.1 shows the peak ground accelerations of the three excitation records used in the time history analyses. Figure 7.2, 7.3, 7.4 show the time history plots of the ground motions used and their response spectra in comparison with the UBC 94 (as reference) design spectrum.

<table>
<thead>
<tr>
<th>Motion</th>
<th>Niigata(x)</th>
<th>Niigata(y)</th>
<th>Landers(x)</th>
<th>Landers(y)</th>
<th>Sylmar(x)</th>
<th>Sylmar(y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA cm/sec*sec</td>
<td>129</td>
<td>171</td>
<td>268</td>
<td>278</td>
<td>578</td>
<td>319</td>
</tr>
<tr>
<td>g</td>
<td>0.13</td>
<td>0.18</td>
<td>0.27</td>
<td>0.28</td>
<td>0.59</td>
<td>0.33</td>
</tr>
<tr>
<td>PGV (cm/sec)</td>
<td>46</td>
<td>61</td>
<td>27</td>
<td>28</td>
<td>78</td>
<td>42</td>
</tr>
<tr>
<td>PGD (cm)</td>
<td>33</td>
<td>44</td>
<td>8</td>
<td>8</td>
<td>16</td>
<td>9</td>
</tr>
</tbody>
</table>

Note: PGA is Peak Ground Acceleration, PGV is Peak Ground Velocity, and PGD is Peak Ground Displacement.
Figure 7.2 Time history and response spectrum of the SMAC-A record (Niigata earthquake)
Figure 7.4 Time history and response spectrum of the Sylmar County Hospital (Northridge earthquake)
7.3.1 Responses of the building during the selected earthquake excitations

Results of the inelastic, time-history analyses are shown in Fig. 7.5, 7.6, 7.7 and 7.8 in terms of lateral displacement, interstory drift, story shear and overturning moment. In these plots x-direction corresponds to the East-West direction of the building (see Fig. 3.4) while y-direction corresponds to the North-South direction of the building.

![Figure 7.5 Maximum inelastic responses of the FWT, displacement](image)

Figure 7.5 Maximum inelastic responses of the FWT, displacement

The plot of maximum lateral displacements shows the significant effect of the Niigata motion on the response of this building. Maximum displacement at roof level in the N-S direction reach 1.8 m. The maximum displacement plot also shows clearly the combination of at least two mode shapes in the responses of this building for the Landers and Northridge motions.
The input motions selected produce different envelopes of interstory drift in each direction of the building as shown in Fig. 7.6. The largest drifts occur for the Niigata motion in the North-South direction (y-direction). The maximum value for the interstory drift is 1.5% and it occurs at the 23rd story. Since interstory drift ratio is a major factor on damage, damages of the building will be expected the most severe on the 23rd story in the case of the Niigata type of ground excitations. Except for the y-direction (N-S) of the Niigata response, the FWT generally behaves well. The consistent shapes of the interstory drift eliminate any weak story mechanism for the building.

It can be observed that in contrast with the variation of the story displacements, with uniform
shape the maximum story shear increases down from the roof story to the 49th story (similar to the expected distribution for low-rise buildings). For the next 20 stories below there is no significant change of maximum story shear. This contradicts the code-defined invert-triangular lateral load distribution along the height of the building. It is also a clear sign of higher modes effects. The maximum story shears in the lower part of the building show the usual increase towards the ground floor. However, one still could notice that in some cases the tip of the curve is growing outward instead of growing inward. This can only be explained by a lateral load pattern which is nearly rectangular or trapezoidal.

The overturning moment plots for the Landers and the Northridge motions have peculiar onion-like shapes. The floors between the 20 to 25 story experienced the largest overturning moment.
Figure 7.8 Maximum inelastic response of the FWT, overturning moment during these two earthquakes. The overturning moment for the Niigata motion in the x (East-West) direction show similar behavior, but in the y (North-South) direction the moment distribution is as generally expected.

Roof displacement time history responses from the Sylmar County Hospital excitation are plotted in Fig. 7.9. The pulse effect of the Sylmar ground motion is apparent in the displacement time history show, where the maximum amplitude reached 0.66m on one cycle. A permanent displacement of more than 0.09m is also indicated. The displacement time history for the other two excitations does not appear to be any permanent displacement at the end of the time history even though many beam and bracing members have experienced large yielding. Comparing the time
Figure 7.9 Roof displacement time history responses, Sylmar County Hospital

history plot of input ground excitations, it infers to the strong impulse, short duration of the Sylmar record as the major reason for residue displacement. Roof acceleration time history responses are plotted in Fig. 7.10.

7.4 Pushover Analysis

In order to estimate the lateral resistance of the building at ultimate load, static nonlinear (push-over) analyses were conducted. The results of pushover analysis are sensitive to the choice of
load patterns, particularly for this case of high-rise building. A predetermined lateral force pattern from building code was applied incrementally in a step-wise manner in each direction independently. This lateral force pattern from building code assumes the lateral force distribution as an inverted-triangular pattern which is in accordance with the first mode shape of multi-degree-of-freedom building structures. Later on, other lateral force patterns used to perform pushover analyses will be presented to achieve better responses when comparing to the time history analyses.

The building was analyzed based on applying the selected lateral force pattern until a specified
target roof displacement was reached. A target roof displacement of 4.5m (3 percent of the height of building) and an increment of 0.1m to 0.001m (depend on the condition of convergence) was selected for this study.

The base shear versus roof displacement for pushover analysis of the building in the east-west direction is shown in Fig. 7.11. Significant yielding in the structure occurs at a roof deflection of approximately 1.5m. First yielding members are the outrigger beams between the 23rd to the 26th floors when the roof displacement reached 0.883m. The first group of yielding bracing members are those between the 22rd to the 26th floors when the roof displacement reached 1.13 m. Maximum base shear is $4.71 \times 10^4$ kN, while maximum overturning moment is
6.72 \times 10^6 \text{ kN} \cdot \text{m}.

Figure 7.12(b) shows the yielding propagation in the south perimeter frame in the east-west direction. Outrigger beams start yielding at around the 22nd floor and propagate upward. Bracing members start yielding at around 4th story and propagate upward. The north perimeter frame has similar steps.

The same target displacement and increment were selected for the analysis in the north-south direction. The yielding propagation steps in the south perimeter frame in the north-south direction are similar with those in the east-west direction which shows that this building has similar strength in both lateral direction.

After completing the analysis for the code-defined lateral load pattern, a second lateral pattern defined in FEMA-273 was adopted. This pattern, often termed the uniform pattern, shall be based on lateral forces that are proportional to the total mass at each floor level. FEMA-273 is the first guideline to introduce the uniform pattern as one of the necessary load patterns for pushover analysis.

The same target displacement as that used for code-defined lateral pattern was selected for the uniform load pattern. Figure 7.12c shows the yielding propagation steps in the south perimeter frame in the north-south direction. Bracing members start yielding at around 15th story. The yielding sequence of the building when subjected to a near-field type time history motion, such as the Sylmar record, is presented in Figure 7.12a. It is clear that when comparing results from
Figure 7.12 Yielding propagation steps in the south perimeter frame in E-W direction: a) time history, Sylmar motion,

b) Pushover, triangular load pattern, c) Pushover, Uniform load pattern, d) Pushover, trapezoidal load pattern.
triangular and uniform lateral load pattern analyses, a closer yielding sequence to that obtained from the Sylmar motion analysis is obtained by the uniform load pattern.

A third lateral pattern was investigated in order to make the response from the pushover analysis better match the response from the time history analysis. This proposed load pattern has a trapezoidal shape. Long side of the trapezoid doubles the short side. The yielding propagation (bracing members start yielding at around 18th story) in Fig. 7.12d clearly show that the best comparison of yielding steps is between this load pattern and the time history response subjected to a near-field type input motion.

Table 7.2 is a summary of the maximum base shear and overturning moments for the FWT based

<table>
<thead>
<tr>
<th>Event</th>
<th>Base Shear (kN)</th>
<th>Overturning Moment (value x 10^6 kN • m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>V</td>
<td>V/Vd</td>
</tr>
<tr>
<td>Design</td>
<td>32716</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Northridge</td>
<td>14182</td>
</tr>
<tr>
<td></td>
<td>Niigata</td>
<td>50961</td>
</tr>
<tr>
<td></td>
<td>Joshua</td>
<td>39623</td>
</tr>
<tr>
<td></td>
<td>Sylmar</td>
<td>68121</td>
</tr>
<tr>
<td></td>
<td>Triangular</td>
<td>14325</td>
</tr>
<tr>
<td></td>
<td>Uniform</td>
<td>73283</td>
</tr>
<tr>
<td></td>
<td>Trapezoidal</td>
<td>56043</td>
</tr>
</tbody>
</table>

Note: Vd = design base shear = 32716 kN, Md = design overturning moment = 2.92 x 10^6 kN*m.
on design values, four time history analyses and three pushover analyses. In the table it is clear that the Northridge (elastic response based on the FWT's recorded motion during the Northridge earthquake) event pushed the structure well within the design range. Base shear during this earthquake is only 40 percent of the design value. Different types of earthquakes have produced greatly different base shear and overturning moment demands, ranging from 70 percent to 220 percent of the design value. While the Sylmar excitation caused the largest base shear for the structure, the Niigata excitation demanded the highest overturning moment resistance.

For pushover analysis, a trapezoidal lateral load pattern is a compromise between the traditional invert-triangular pattern and the FEMA defined uniform pattern. Uniform pattern provides the upper bound but may be too conservative. Its base shear and overturning moment demands are more than four times of those by invert-triangular pattern. When comparing with the time history results the proposed trapezoidal lateral load pattern remains in close range of both base shear and overturning moment demands for the types of ground motions considered.

7.5 Cyclic Analysis

Cyclic analysis, a static nonlinear analysis under load reversal, was also conducted. The model was subjected to several load reversals, so that it would capture the dynamic characteristics of the member elements as well as the whole structure. A predetermined lateral load pattern was applied incrementally in a step-wise manner. In this analysis, a lateral load pattern of invert-triangle was assumed in each direction.
The analysis was performed based on displacement control. The first target displacement was 1 m at roof. Then the structure model was unloaded and pushed back to -1 m roof displacement. In this range the structure is within linear response. No hysteresis loop energy dissipation was observed during the load reversal. The second target displacement of 2 m pushed the structure into yielding. Displacement increases rapidly while base shear generally remained at the 45,000 kN level. After unloading the structure to its initial position, a -2 m roof displacement was used for the cyclic analysis. It is obvious that the stiffness of the structure has changed due to the element unloading stiffness degradation. A third target displacement of 4 m roof displacement pushed the structure well into inelastic response. The building’s base shear capacity has been reached. The total strength of the structure was seen deteriorated. Large area of hysteresis loop showed the advantage of steel structure in dissipating energy.

The base shear versus roof displacement for the cyclic response of the building in the east-west direction is shown in Fig. 7.13. The maximum base shear capacity for this building is shown to be about 47,000 kN at roof displacement of around 2 m. Cyclic analysis in the north-south direction shows similar base shear / roof displacement curve. This observation is consistent with the uni-directional pushover analyses.

### 7.6 Conclusions

Based on the design specifications, a three-dimensional nonlinear model of the building has been developed and analyzed using different analytical techniques. Time history analyses using three distinguish types of severe shakings recorded in past earthquakes were conducted to study the
inelastic behavior of the building. Afterwards, the pushover analysis using traditional inverted-triangular lateral load pattern shows the possible sequence of damage greatly different from that predicted by the three-dimensional nonlinear time history analysis under the Sylmar excitation. An improved lateral load pattern is therefore derived and yields better fit with the dynamic responses for the type of ground motion considered.

By applying recent enhancements in computational techniques and the development of more sophisticated models this chapter provides valuable information about the dynamic behavior of high-rise steel buildings. Envelop plots of lateral displacement, interstory drift, story shear and overturning moments present interesting features that have not been shown in previous research.
And the importance of choosing ground excitations for dynamic analyses is clearly demonstrated by the responses of the FWT under the three distinctive types of excitations considered in this study.
Chapter 8 SUMMARY AND RECOMMENDATIONS

8.1 Summary

The goal of this thesis was to study and understand the seismic behavior of a steel high-rise building. The primary objectives of the study have been achieved. These include: 1) system identification of the recorded earthquake motions, 2) investigation of the building behavior during past earthquakes through construction of a lumped plasticity computer model and calibration of the model using recorded motions, 3) evaluation of the inelastic structural response during severe shaking, and 4) evaluation of the inelastic static response of the building for three lateral load patterns.

The subject of this study is an instrumented 52-story steel braced and moment frame building in Los Angeles which has experienced several earthquakes after construction was finished in 1990. Among the recorded building seismic responses those from the 1991 Sierra Madre earthquake and the 1994 Northridge earthquake are investigated.

The dynamic characteristics of the structure were determined by utilizing system identification algorithm on the recorded responses during both earthquakes. Afterwards three-dimensional linear elastic time history analyses of the building are conducted using information obtained from the system identification analyses. The linear computer model was calibrated to better represent the dynamic behavior of the building during past earthquakes.
Nonlinear computer model was constructed to predict structural responses during future severe earthquakes. First, three distinctive types of earthquake excitations were used in the nonlinear time history analyses for the prediction of building behavior. Next, pushover and cyclic analyses were considered. Three lateral load patterns were discussed to improve the yielding sequence obtained from the pushover analyses.

8.2 Recorded Behavior of the FWT and Prediction

The following conclusions are drawn regarding the behavior of the building during the earthquakes. These are based on the discussion of the building responses and on the analytical studies of the structure.

• Linear analysis is able to reproduce the response of the building when the dynamic characteristics of the structure are well defined and the computer model calibrated. The correlation between the computed and recorded responses of the building during the 1991 Sierra Madre earthquake and the 1994 Northridge earthquake is a clear evidence for the observation. The good match implies that the calibrated linear model is a good representation of the dynamic behavior of the building during past earthquakes.

• Pushover analyses and time history analyses performed on the nonlinear lumped plasticity model disclose interesting features of the structure during future possible earthquakes. Three distinctive types of earthquake excitations were used. The first set of records from the 1964 Niigata (Japan) earthquake have a major energy input around the first mode period of the
studied building (4.5 to 6 seconds). The second set of records are the Joshua Tree Fire station records during the 1992 Landers Earthquake which have a long duration of at least 40 seconds of strong shaking. The last set of records are from the Sylmar County Hospital station during the 1994 Northridge earthquake. This is a short duration, strong impulse, near-source ground motion that may be very damaging to tall buildings.

• Envelope of the maximum displacement shows the significant effect of the Niigata motion on the response of this building. The Niigata earthquake has the largest magnitude among the three records considered. Because of the major energy input around the first mode period in this record the maximum displacement plot has the conventional inverted-triangular shape which is the typical first mode shape of buildings. The maximum displacement plot also shows clearly the combination of at least two mode shapes in the building responses for the other two excitations.

• The input motions selected produced different envelopes of interstory drift in each direction of the building as shown in Chapter 7. The largest drifts occur for the Niigata motion in the mid-height of the building. Therefore severe damage may be expected to happen in these area if Niigata type of ground motions are experienced.

• Maximum story shear increases down from the roof to the 49th story. For the next 20 stories below there is no significant change of maximum story shear resulting from the Sylmar and Joshua Tree motions. This again contradicts the code-defined inverted-triangular lateral load distribution along the height of the building. It is also a clear sign of higher modes effects.
The maximum story shears in the lower part of the building show the usual increase towards the ground floor. However, it can still be noticed that in several cases the tip of the curve is growing outward instead of growing inward. This can only be explained by a lateral load pattern which is nearly rectangular or trapezoidal.

- The overturning moment plots for the Sylmar and Joshua Tree motions have peculiar onion-like shapes. The floors between the 20th to 25th story experienced the largest overturning moment for these two excitations. Since the overturning moment resistance of the building is designed to be taken by the outrigger beams, it may suggest the outrigger beams between these stories will reach their yield strength first.

- A proposed trapezoidal lateral load pattern provides the best comparison of yielding propagation steps for the building between the pushover analyses and the time history analyses under the Sylmar type of excitation considered.

### 8.3 Recommendations for Further Research

- In the future, the development of more sophisticated models for steel members may be employed into the 3-D nonlinear computer analysis, accounting for better representative of the steel element properties. The capacity and behavior of each element, especially in the inelastic range, are determined by its expected material strength. An analysis based on the actual material strengths obtained from experimental tests of samples taken from the building, rather than the specified values, will probably result in a better correlation between predicted and
future possible earthquakes responses.

- A more detailed computer model including the expanded basement foundation and the interaction between the foundation and the soil would provide additional insight into the performance of the building.

- Due to the limitation of the computer program CANNY, damping used for the dynamic analysis was restricted to Rayleigh-type damping. When features of the program include other types of damping, it is recommended that other types of damping be implemented into the computer model to simulate more realistically the energy dissipation in the structure.

- Effect of gravitational frames was investigated for linear dynamic analysis of the structure. Including the gravitational frames for the nonlinear dynamic time history analysis may provide more useful information to assess the contribution of the gravitational frames.

- Ductility demand and collapse mechanism of the structure were not covered in this study. It would be interesting to carry out further research on these two aspects.

- Conditioning of input records are important to apply system identification algorithm. However magnitude of the conditioned records were reduced, caused difficulty in the system identification process. Therefore improvements on the method of conditioning shall be investigated in the future.
REFERENCES


Banavalkar, P.V., 1992, Personal communication with Dr. C.E. Ventura, June.


Li, K.N., 1999, personal communication with Y. Ding.


Appendix A

Elevations and Details of the Building
Typical Floor Plan

DESIGN CRITERIA

ABS(Mi-Mg) < Mpn
ABS(M2+Mg2) < Mpn
Mg3 < Mpn

Vu = Vp +/- Vg1 or Vg2 or Vg3

Mpn = plastic moment capacity of notched section
Mg or Vg = factored moment or shear

Figure a.1. FWT, typical floor plan and details
Figure a.2. FWT, Frame A Elevation
Figure a.3. FWT, Frame B Elevation
Figure a.4. FWT, Frame C Elevation
Figure a.5. FWT, Frame D Elevation
Appendix B

Photos of the Building
Figure b.5 Close-up view on lower stories I (Jan. 1990)

Figure b.6 Close-up view on lower stories II (Jan. 1990)
Appendix C

CANNY input data file
1998.6, dynamic analysis of Figueroa at Wilshire
a steel frame highrise in L.A.

// analysis assumptions and output options

title = DYNAMIC ANALYSIS OF FIGUEROA AT WILSHIRE
force unit = kN
length unit = m
time unit = sec

gravity acceleration is 9.805
including floor rotation of rigid floor level
including P-Delta effect
output of overall response at floor levels
/*output of period time history

/*static analysis
// control data of static response
response limit 10.0
master DOFs for analysis control: 49F XY-translation
binary format output of response results at every 0-step

/*destination base shear factor 0.20 by increment 0.01
/*destination base shear factor 0.21 by increment 0.001

destination displacement 1.5 by increment 0.01
destination displacement 2.5 by increment 0.001
destination displacement 4.5 by increment 0.005

/*destination base shear factor 0.20 by increment 0.01
/*destination base shear factor 0.21 by increment 0.001

/*destination displacement 1.0 by increment 0.1
/*destination displacement -1.0 by increment 0.1

/*destination displacement 1.0 by increment 0.1
/*destination displacement 2.0 by increment 0.01
/*destination displacement -1.5 by increment 0.01
/*destination displacement -2.0 by increment 0.005

/*destination displacement 4.0 by increment 0.001
/*destination displacement -4.0 by increment 0.001

/*destination displacement 18.0 by increment 0.2
/*destination displacement -18.0 by increment 0.2
/*destination displacement 0 by increment 0.001

// control data of dynamic response
integration 2-step for one acceleration data
start time 0.0, end time 180.0
check peak displacement 0.05
response limit 5.0
master DOFs for analysis control: 49F XY-translation
binary format output of response results at every 0-step
Newmark method using Beta-value 0.25
/* damping constant 0.053 to mass matrix [M]
/* damping constant 0.00168 to stiffness matrix [K]
damping coefficient 0.1012 proportional to mass matrix [M]
damping coefficient 0.00321 proportional to stiffness matrix [K]
scale factor 0.01 to X-EQ file = e:\ding\fw-c\nr2.txt
scale factor 0.01 to Y-EQ file = e:\ding\fw-c\nr2.txt
scale factor 0.01 to R-EQ file = e:\ding\fw-c\nr-r22.txt

// node locations
X1 and X7, Y1 and Y3, 1F to 53F
X8 and X9, Y1 and Y3, 1F to 50F
X1 and X7, Y4 and Y5, 1F to 50F
Y1, X4, 1F to 17F
Y1, X4, 32F to 43F
Y1, X2, 19F to 32F
Y1, X3, 44F to 49F
Y1, X5, 18F to 31F
Y1, X5, 50F to 53F
Y1, X3, 50F to 53F
Y1, X6, 45F to 49F
Y2, X1 and X7, 2F to 48F
Y2, X1 and X7, 50F to 53F
Y3, X2, 3F to 4F
Y3, X2, 31F to 44F
Y3, X3, 4F to 29F
Y3, X3, 44F to 49F
Y3, X4, 1F to 3F
Y3, X5, 30F to 43F
Y3, X5, 50F to 53F
Y3, X3, 50F to 53F
Y3, X6, 3F to 49F
/* node X8 and X9, Y4 and Y5, 1F to 50F

// node degrees of freedom
node Y1, X4, 2F to 17F, eliminate Y-Z rot	node Y1, X4, 32F to 43F, eliminate Y-Z rot	node Y1, X2, 19F to 32F, eliminate Y-Z rot	node Y1, X3, 44F to 49F, eliminate Y-Z rot	node Y1, X5, 18F to 31F, eliminate Y-Z rot	node Y1, X5, 50F, eliminate Y-Z rot	node Y1, X6, 45F to 49F, eliminate Y-Z rot	node Y2, X1 and X7, 2F to 48F, eliminate X-Z rot	node Y2, X1 and X7, 50F, eliminate X-Z rot	node Y3, X2, 3F to 4F, eliminate Y-Z rot	node Y3, X2, 31F to 44F, eliminate Y-Z rot	node Y3, X3, 4F to 29F, eliminate Y-Z rot	node Y3, X3, 44F to 49F, eliminate Y-Z rot	node Y3, X3, 44F to 49F, eliminate Y-Z rot	node Y3, X4, 2F to 3F, eliminate Y-Z rot	node Y3, X5, 30F to 43F, eliminate Y-Z rot	node Y3, X5, 50F, eliminate Y-Z rot	node Y3, X6, 3F to 49F, eliminate Y-Z rot
node Y3, X6, 3F to 49F, eliminate Y-Z rot
/*node X1 to X9, Y1 to Y5, 1F, eliminate all component

// floor level

53F (rigid floor, above 52F): Z=218.0 G(8.7,10.5), W=3360 Rj=234813,
52F (rigid floor, above 51F): Z=213.0 G(8.6,10.4), W=8855 Rj=1391108,
51F (rigid floor, above 50F): Z=207.0 G(9.0,10.0), W=11930 Rj=2200801,
50F (rigid floor, above 49F): Z=202.0 G(8.7,10.7), W=10555 Rj=2907479,
49F (rigid floor, above 48F): Z=194.0 G(8.7,10.7), W=9147 Rj=2504407,
48F (rigid floor, above 47F): Z=190.1 G(8.7,10.7), W=8473 Rj=2204690,
47F (rigid floor, above 46F): Z=186.1 G(8.7,10.7), W=8473 Rj=2204690,
46F (rigid floor, above 45F): Z=182.2 G(8.7,10.7), W=9178 Rj=2657780,
45F (rigid floor, above 44F): Z=178.2 G(8.6,10.84), W=9439 Rj=2670596,
44F (rigid floor, above 43F): Z=174.2 G(8.7,10.7), W=9273 Rj=2657367,
43F (rigid floor, above 42F): Z=170.2 G(8.7,10.7), W=9355 Rj=2693336,
42F (rigid floor, above 41F): Z=166.3 G(8.7,10.7), W=9988 Rj=3104672,
41F (rigid floor, above 40F): Z=162.3 G(8.7,10.7), W=9988 Rj=3104672,
40F (rigid floor, above 39F): Z=158.3 G(8.7,10.7), W=9988 Rj=3104672,
39F (rigid floor, above 38F): Z=154.3 G(8.7,10.7), W=9988 Rj=3104672,
38F (rigid floor, above 37F): Z=150.4 G(8.7,10.7), W=9988 Rj=3104672,
37F (rigid floor, above 36F): Z=146.5 G(8.7,10.7), W=9988 Rj=3104672,
36F (rigid floor, above 35F): Z=142.5 G(8.7,10.7), W=11399 Rj=3749583,
35F (rigid floor, above 34F): Z=138.5 G(8.7,10.7), W=10346 Rj=3447384,
34F (rigid floor, above 33F): Z=134.5 G(8.7,10.7), W=10346 Rj=3447384,
33F (rigid floor, above 32F): Z=130.6 G(8.7,10.7), W=10626 Rj=3517663,
32F (rigid floor, above 31F): Z=126.7 G(8.7,10.7), W=10626 Rj=3517663,
31F (rigid floor, above 30F): Z=122.7 G(8.4,10.2), W=11398 Rj=3568513,
30F (rigid floor, above 29F): Z=118.7 G(8.7,10.7), W=10559 Rj=3570164,
29F (rigid floor, above 28F): Z=114.7 G(8.7,10.7), W=10559 Rj=3570164,
28F (rigid floor, above 27F): Z=110.8 G(8.7,10.7), W=10559 Rj=3570164,
27F (rigid floor, above 26F): Z=106.9 G(8.7,10.7), W=10559 Rj=3570164,
26F (rigid floor, above 25F): Z=102.9 G(8.7,10.7), W=10559 Rj=3570164,
25F (rigid floor, above 24F): Z=98.9 G(8.7,10.7), W=10559 Rj=3570164,
24F (rigid floor, above 23F): Z=95.0 G(8.7,10.7), W=10559 Rj=3570164,
23F (rigid floor, above 22F): Z=91.0 G(8.7,10.7), W=10559 Rj=3570164,
22F (rigid floor, above 21F): Z=87.0 G(8.7,10.7), W=10559 Rj=3570164,
21F (rigid floor, above 20F): Z=83.0 G(8.7,10.7), W=10559 Rj=3570164,
20F (rigid floor, above 19F): Z=79.0 G(9.0,11.2), W=11706 Rj=3690051,
19F (rigid floor, above 18F): Z=75.0 G(8.7,10.7), W=10680 Rj=3631765,
18F (rigid floor, above 17F): Z=71.1 G(8.7,10.7), W=10680 Rj=3631765,
17F (rigid floor, above 16F): Z=67.2 G(8.7,10.7), W=10780 Rj=3665661,
16F (rigid floor, above 15F): Z=63.2 G(8.7,10.7), W=10780 Rj=3665661,
15F (rigid floor, above 14F): Z=59.2 G(8.7,10.7), W=10780 Rj=3665661,
14F (rigid floor, above 13F): Z=55.2 G(8.7,10.7), W=10780 Rj=3665661,
13F (rigid floor, above 12F): Z=51.3 G(8.7,10.7), W=11006.5 Rj=3733875,
12F (rigid floor, above 11F): Z=47.4 G(8.7,10.7), W=11006.5 Rj=3733875,
11F (rigid floor, above 10F): Z=43.4 G(8.7,10.7), W=11006.5 Rj=3733875,
10F (rigid floor, above 9F): Z=39.4 G(8.7,10.7), W=11006.5 Rj=3733875,
9F (rigid floor, above 8F): Z=35.4 G(8.7,10.7), W=11224 Rj=3801671,
8F (rigid floor, above 7F): Z=31.4 G(8.7,10.7), W=11224 Rj=3801671,
3F (rigid floor, above 2F): Z=10.4 G(9.2,13.0), W=13138 Rj=4774000,
2F (rigid floor, above 1F): Z=6.10 G(7.5,11.7), W=18488 Rj=8362769,
1F (footing floor, fixed support): Z=0

// frame locations
X8: 31.21, X9: -13.84
Y1: 0, Y2: 10.82, Y3: 21.23, Y4: 33.35, Y5: -11.71

// element data: beam
frame X1 and X7, Y1 to Y3, 2F LU06 RU06 SU806 .46 .46
frame X1 and X7, Y1 to Y3, 3F to 6F LU08 RU08 SU808 .46 .46
frame X1 and X7, Y1 to Y3, 7F to 8F LU09 RU09 SU809 .46 .46
frame X1 and X7, Y1 to Y3, 9F to 12F LU10 RU10 SU810 .46 .46
frame X1 and X7, Y1 to Y3, 13F to 16F LU11 RU11 SU811 .46 .46
frame X1 and X7, Y1 to Y3, 17F to 19F LU12 RU12 SU812 .46 .46
frame X1 and X7, Y1 to Y3, 20F LU07 RU07 SU807 .46 .46
frame X7 , Y1 to Y3, 21F LU11 RU11 SU811 .46 .46
frame X7 , Y1 to Y3, 22F to 25F LU12 RU12 SU812 .46 .46
frame X7 , Y1 to Y3, 26F to 35F LU13 RU13 SU813 .46 .46
frame X7 , Y1 to Y3, 36F to 47F LU14 RU14 SU814 .46 .46
frame X1 and X7, Y1 to Y3, 48F to 49F LU15 RU15 SU815 .46 .46
frame X1 and X7, Y1 to Y3, 50F LU88 RU88 SU888 .46 .46
frame X1 and X7, Y1 to Y3, 51F to 53F LU02 RU02 SU802 .46 .46
frame X1 , Y1 to Y3, 20F to 22F LU12 RU12 SU812 .46 .46
frame X1 , Y1 to Y3, 23F to 30F LU13 RU13 SU813 .46 .46
frame X1 , Y1 to Y3, 31F LU09 RU09 SU809 .46 .46
frame X1 , Y1 to Y3, 32F LU12 RU12 SU812 .46 .46
frame X1 , Y1 to Y3, 33F to 35F LU13 RU13 SU813 .46 .46
frame X1 , Y1 to Y3, 36F to 44F LU14 RU14 SU814 .46 .46
frame X1 , Y1 to Y3, 45F LU09 RU09 SU809 .46 .46
frame X1 , Y1 to Y3, 46F to 47F LU14 RU14 SU814 .46 .46
frame Y3 , X1 to X7, 2F LU87 RU87 SU887 .46 .46
frame Y3 , X1 to X7, 3F LU15 RU15 SU815 .46 .46
frame Y3 , X1 to X7, 4F LU13 RU13 SU813 .46 .46
frame Y3 , X1 to X7, 5F to 6F LU15 RU15 SU815 .46 .46
frame Y3 , X1 to X7, 7F to 10F LU16 RU16 SU816 .46 .46
frame Y3 , X1 to X7, 11F to 14F LU17 RU17 SU817 .46 .46
frame Y3 , X1 to X7, 15F to 18F LU18 RU18 SU818 .46 .46
frame Y3 , X1 to X7, 19F LU19 RU19 SU819 .46 .46
frame Y3 , X1 to X7, 20F LU14 RU14 SU814 .46 .46
frame Y3 , X1 to X7, 21F to 24F LU19 RU19 SU819 .46 .46
frame Y3 , X1 to X7, 25F to 30F LU20 RU20 SU820 .46 .46
frame Y3 , X1 to X7, 31F to 43F LU16 RU16 SU816 .46 .46
frame Y3 , X1 to X7, 44F to 49F LU20 RU20 SU820 .46 .46
frame Y1 and Y3, X1 to X7, 50F LU88 RU88 SU888 .46 .46
frame Y1 and Y3, X1 to X7, 51F to 52F LU89 RU89 SU889 .46 .46
frame Y1 and Y3, X1 to X7, 53F LU02 RU02 SU802 .46 .46
frame Y1 , X1 to X7, 2F LU86 RU86 SU886 .46 .46
frame Y1 , X1 to X7, 3F to 8F LU12 RU12 SU812 .46 .46
frame Y1 , X1 to X7, 9F to 11F LU13 RU13 SU813 .46 .46
frame Y1 , X1 to X7, 12F to 18F LU14 RU14 SU814 .46 .46
frame Y1 , X1 to X7, 19F to 24F  LU19 RU19 SU819 .46 .46
frame Y1 , X1 to X7, 25F to 30F  LU20 RU20 SU820 .46 .46
frame Y1 , X1 to X7, 31F  LU17 RU17 SU817 .46 .46
frame Y1 , X1 to X7, 32F to 35F  LU16 RU16 SU816 .46 .46
frame Y1 , X1 to X7, 36F to 41F  LU16 RU16 SU816 .46 .46
frame Y1 , X1 to X7, 42F to 44F  LU18 RU18 SU818 .46 .46
frame Y1 , X1 to X7, 45F to 49F  LU20 RU20 SU820 .46 .46
frame X7 , Y5-Y1, 2F  LU92 RU92 SU892 .46 .46
frame X1 and X7, Y5-Y1, 3F to 4F  LU03 RU03 SU803 .46 .46
frame X1 and X7, Y5-Y1, 5F to 49F  LU05 RU05 SU805 .46 .46
frame X1 and X7, Y5-Y1, 50F  LU94 RU94 SU894 .46 .46
frame X1 , Y5-Y1, 2F  LU92 RU92 SU892 .46 .46
frame X7 , Y3 to Y4, 2F  LU01 RU01 SU801 .46 .46
frame X1 and X7, Y3 to Y4, 3F to 4F  LU03 RU03 SU803 .46 .46
frame X1 and X7, Y3 to Y4, 5F to 49F  LU05 RU05 SU805 .46 .46
frame X1 and X7, Y3 to Y4, 50F  LU94 RU94 SU894 .46 .46
frame X1 , Y3 to Y4, 2F  LU01 RU01 SU801 .46 .46
frame Y1 and Y3, X9-X1, 2F  LU02 RU02 SU802 .46 .46
frame Y1 and Y3, X9-X1, 3F to 4F  LU03 RU03 SU803 .46 .46
frame Y1 and Y3, X9-X1, 5F to 36F  LU04 RU04 SU804 .46 .46
frame Y1 and Y3, X9-X1, 37F to 49F  LU05 RU05 SU805 .46 .46
frame Y1 and Y3, X9-X1, 50F  LU94 RU94 SU894 .46 .46
frame Y1 and Y3, X7 to X8, 2F  LU02 RU02 SU802 .46 .46
frame Y1 and Y3, X7 to X8, 3F to 4F  LU03 RU03 SU803 .46 .46
frame Y1 and Y3, X7 to X8, 5F to 36F  LU04 RU04 SU804 .46 .46
frame Y1 and Y3, X7 to X8, 37F to 49F  LU05 RU05 SU805 .46 .46
frame Y1 and Y3, X7 to X8, 50F  LU94 RU94 SU894 .46 .46
/frame X8 and X9, Y1 to Y3, 2F to 50F  LU06 RU06 SU806 0 0
/frame Y4 and Y5, X1 to X7, 2F to 50F  LU06 RU06 SU806 0 0
/frame X8 and X9, Y3 to Y4, 2F to 50F  LU25 RU25 SU825 0 0
/frame X8 and X9, Y5-Y1, 2F to 50F  LU25 RU25 SU825 0 0
/frame Y4 and Y5, X7 to X8, 2F to 50F  LU25 RU25 SU825 0 0
/frame Y4 and Y5, X9-X1, 2F to 50F  LU25 RU25 SU825 0 0

// element data : link

/*X4 to X4 Y1 and Y3 1F to 2F  AU968
X4-X7 Y1 2F~3F  AU907
X4-X7 Y1 3F~6F  AU910
X4-X7 Y1 6F~10F  AU911
X4-X7 Y1 10F~14F  AU912
X4-X7 Y1 14F~16F  AU913
X4-X7 Y1 16F~18F  AU914
X4-X7 Y1 32F~34F  AU917
X4-X7 Y1 34F~39F  AU918
X4-X7 Y1 39F~42F  AU919
X4-X7 Y1 42F~44F  AU920
X4-X1 Y1 2F–3F AU907
X4-X1 Y1 3F–6F AU910
X4-X1 Y1 6F–10F AU911
X4-X1 Y1 10F–14F AU912
X4-X1 Y1 14F–16F AU913
X4-X1 Y1 16F–18F AU914
X4-X1 Y1 32F–34F AU917
X4-X1 Y1 34F–39F AU918
X4-X1 Y1 39F–42F AU919
X4-X1 Y1 42F–44F AU920
X1-X2 Y1 18F–21F AU921
X1-X2 Y1 21F–24F AU922
X1-X2 Y1 24F–30F AU923
X1-X2 Y1 30F–31F AU920
X1-X2 Y1 31F–32F AU923
X5-X2 Y1 18F–19F AU917
X5-X2 Y1 19F–24F AU918
X5-X2 Y1 24F–28F AU919
X5-X2 Y1 28F–30F AU920
X5-X2 Y1 30F–31F AU918
X5-X2 Y1 31F–32F AU919
X5-X7 Y1 18F–19F AU917
X5-X7 Y1 19F–24F AU920
X5-X7 Y1 24F–28F AU919
X5-X7 Y1 28F–30F AU920
X5-X7 Y1 30F–31F AU918
X5-X7 Y1 31F–32F AU923
X3-X1 Y1 44F–49F AU923
X3-X1 Y1 49F–50F AU920
X3-X6 Y1 14F–45F AU922
X3-X6 Y1 45F–49F AU923
X3-X5 Y1 49F–50F AU919
X7-X6 Y1 44F–45F AU931
X7-X6 Y1 45F–47F AU932
X7-X6 Y1 47F–49F AU933
X7-X5 Y1 49F–50F AU916

X1-X2 Y3 2F–3F AU916
X1-X2 Y3 3F–4F AU913
X4-X2 Y3 2F–4F AU916
X4-X6 Y3 2F–4F AU916
X7-X6 Y3 2F–4F AU916
X3-X1 Y3 4F–5F AU912
X3-X1 Y3 5F–8F AU915
X3-X1 Y3 8F–12F AU916
X3-X1 Y3 12F–16F AU917
X3-X1 Y3 16F–21F AU918
X3-X1 Y3 21F–22F AU919
X3-X1 Y3 22F–29F AU920
X3-X1 Y3 29F–30F AU921
X3-X1 Y3 44F–46F AU922
X3-X1 Y3 46F–49F AU923
X3-X1 Y3 49F–50F AU920
X3-X6 Y3 4F–5F AU998
X3-X6 Y3 5F–8F AU940

118
X3-X6 Y3 8F–12F AU941
X3-X6 Y3 12F–14F AU942
X3-X6 Y3 14F–16F AU917
X3-X6 Y3 16F–24F AU918
X3-X6 Y3 24F–28F AU919
X3-X6 Y3 28F–30F AU920
X3-X6 Y3 44F–46F AU920
X3-X6 Y3 46F–49F AU923
X3-X5 Y3 49F–50F AU919
X7-X6 Y3 4F–6F AU917
X7-X6 Y3 6F–10F AU918
X7-X6 Y3 10F–14F AU919
X7-X6 Y3 14F–16F AU920
X7-X6 Y3 16F–19F AU921
X7-X6 Y3 19F–20F AU919
X7-X6 Y3 20F–24F AU922
X7-X6 Y3 24F–30F AU923
X7-X6 Y3 44F–45F AU923
X7-X6 Y3 45F–46F AU931
X7-X6 Y3 46F–47F AU932
X7-X6 Y3 47F–49F AU933
X7-X5 Y3 49F–50F AU916
X1-X2 Y3 30F–41F AU923
X1-X2 Y3 41F–44F AU931
X5-X2 Y3 30F–31F AU920
X5-X2 Y3 31F–34F AU921
X5-X2 Y3 34F–44F AU922
X5-X6 Y3 30F–37F AU933
X5-X6 Y3 37F–44F AU934
X7-X6 Y3 30F–31F AU920
X7-X6 Y3 31F–33F AU921
X7-X6 Y3 33F–35F AU922
X7-X6 Y3 35F–44F AU923
X1 Y1–Y2 1F–2F AU996
X1 Y2–Y1 1F–2F AU996
X1 Y2–Y1 2F–5F AU997
X1 Y2–Y1 5F–10F AU998
X1 Y2–Y1 10F–13F AU943
X1 Y2–Y1 13F–14F AU944
X1 Y2–Y1 14F–16F AU945
X1 Y2–Y1 16F–19F AU940
X1 Y2–Y1 19F–22F AU941
X1 Y2–Y1 22F–23F AU946
X1 Y2–Y1 23F–31F AU942
X1 Y2–Y1 31F–49F AU947
X1 Y2–Y3 2F–5F AU997
X1 Y2–Y3 5F–10F AU998
X1 Y2–Y3 10F–13F AU943
X1 Y2–Y3 13F–14F AU944
X1 Y2–Y3 14F–16F AU945
X1 Y2–Y3 16F–19F AU940
X1 Y2–Y3 19F–22F AU941
X1 Y2–Y3 22F–23F AU946
X1 Y2-Y3 23F-31F AU942
X1 Y2-Y3 31F-49F AU947
X1 Y1-Y2 49F-50F AU999
X1 Y3-Y2 49F-50F AU999
X7 Y1-Y2 1F-2F AU996
X7 Y3-Y2 1F-2F AU996
X7 Y2-Y1 2F-5F AU997
X7 Y2-Y1 5F-10F AU998
X7 Y2-Y1 10F-13F AU943
X7 Y2-Y1 13F-14F AU944
X7 Y2-Y1 14F-16F AU945
X7 Y2-Y1 16F-19F AU940
X7 Y2-Y1 19F-22F AU941
X7 Y2-Y1 22F-23F AU946
X7 Y2-Y1 23F-31F AU942
X7 Y2-Y1 31F-49F AU947
X7 Y2-Y3 2F-5F AU997
X7 Y2-Y3 5F-10F AU998
X7 Y2-Y3 10F-13F AU943
X7 Y2-Y3 13F-14F AU944
X7 Y2-Y3 14F-16F AU945
X7 Y2-Y3 16F-19F AU940
X7 Y2-Y3 19F-22F AU941
X7 Y2-Y3 22F-23F AU946
X7 Y2-Y3 23F-31F AU942
X7 Y2-Y3 31F-49F AU947
X7 Y1-Y2 49F-50F AU999
X7 Y3-Y2 49F-50F AU999

// element data : column

/* 4-core-corner columns
X1 and X7, Y1 and Y3, 1F to 2F BU90 TU90 SU890 AU990 0.0 0.42
X1 and X7, Y1 and Y3, 2F to 4F BU90 TU90 SU890 AU990 0.21 0.42
X1 and X7, Y1 and Y3, 4F to 6F BU91 TU91 SU891 AU991 0.21 0.42
X1 and X7, Y1 and Y3, 6F to 8F BU50 TU50 SU850 AU950 0.21 0.42
X1 and X7, Y1 and Y3, 8F to 10F BU51 TU51 SU851 AU951 0.21 0.42
X1 and X7, Y1 and Y3, 10F to 12F BU52 TU52 SU852 AU952 0.21 0.42
X1 and X7, Y1 and Y3, 12F to 14F BU53 TU53 SU853 AU953 0.21 0.42
X1 and X7, Y1 and Y3, 14F to 16F BU54 TU54 SU854 AU954 0.21 0.42
X1 and X7, Y1 and Y3, 16F to 18F BU55 TU55 SU855 AU955 0.21 0.42
X1 and X7, Y1 and Y3, 18F to 22F BU56 TU56 SU856 AU956 0.21 0.42
X1 and X7, Y1 and Y3, 22F to 24F BU57 TU57 SU857 AU957 0.21 0.42
X1 and X7, Y1 and Y3, 24F to 26F BU58 TU58 SU858 AU958 0.21 0.42
X1 and X7, Y1 and Y3, 26F to 28F BU59 TU59 SU859 AU959 0.21 0.42
X1 and X7, Y1 and Y3, 28F to 32F BU60 TU60 SU860 AU960 0.21 0.42
X1 and X7, Y1 and Y3, 32F to 34F BU61 TU61 SU861 AU961 0.21 0.42
X1 and X7, Y1 and Y3, 34F to 36F BU62 TU62 SU862 AU962 0.21 0.42
X1 and X7, Y1 and Y3, 36F to 40F BU63 TU63 SU863 AU963 0.21 0.42
X1 and X7, Y1 and Y3, 40F to 44F BU65 TU65 SU865 AU965 0.21 0.42
X1 and X7, Y1 and Y3, 44F to 48F BU67 TU67 SU867 AU967 0.21 0.42
X1 and X7, Y1 and Y3, 48F to 50F BU65 TU65 SU865 AU965 0.21 0.42
X1 and X7, Y1 and Y3, 50F to 52F BU64 TU64 SU864 AU964 0.21 0.42
*/
<table>
<thead>
<tr>
<th>X1 and X7, Y1 and Y3, 52F to 53F BU66 TU66 SU866 AU966</th>
<th>0.21 0.42</th>
</tr>
</thead>
<tbody>
<tr>
<td>/* side columns */ X1 and X7, Y1 and Y3, 52F to 53F BU66 TU66 SU866 AU966</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 1F to 2F BU93 TU93 SU893 AU993</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 2F to 4F BU93 TU93 SU893 AU993</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 4F to 6F BU70 TU70 SU870 AU970</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 6F to 8F BU71 TU71 SU871 AU971</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 8F to 10F BU72 TU72 SU872 AU972</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 10F to 12F BU73 TU73 SU873 AU973</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 12F to 14F BU74 TU74 SU874 AU974</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 16F to 20F BU76 TU76 SU876 AU976</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 20F to 22F BU77 TU77 SU877 AU977</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 22F to 24F BU78 TU78 SU878 AU978</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 24F to 26F BU79 TU79 SU879 AU979</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 26F to 28F BU80 TU80 SU880 AU980</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 28F to 32F BU81 TU81 SU881 AU981</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 32F to 34F BU82 TU82 SU882 AU982</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 34F to 36F BU83 TU83 SU883 AU983</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 36F to 38F BU84 TU84 SU884 AU984</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 38F to 40F BU85 TU85 SU885 AU985</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 40F to 42F BU86 TU86 SU886 AU986</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 42F to 44F BU87 TU87 SU887 AU987</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 44F to 46F BU88 TU88 SU888 AU988</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 46F to 48F BU89 TU89 SU889 AU989</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X1 and X7, Y4 and Y5, 48F to 50F BU90 TU90 SU890 AU990</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 1F to 2F BU93 TU93 SU893 AU993</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 2F to 4F BU93 TU93 SU893 AU993</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 4F to 6F BU70 TU70 SU870 AU970</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 6F to 8F BU71 TU71 SU871 AU971</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 8F to 10F BU72 TU72 SU872 AU972</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 10F to 12F BU73 TU73 SU873 AU973</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 12F to 14F BU74 TU74 SU874 AU974</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 14F to 16F BU75 TU75 SU875 AU975</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 16F to 20F BU76 TU76 SU876 AU976</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 20F to 22F BU77 TU77 SU877 AU977</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 22F to 24F BU78 TU78 SU878 AU978</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 24F to 26F BU79 TU79 SU879 AU979</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 26F to 28F BU80 TU80 SU880 AU980</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 28F to 32F BU81 TU81 SU881 AU981</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 32F to 34F BU82 TU82 SU882 AU982</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 34F to 36F BU83 TU83 SU883 AU983</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 36F to 38F BU84 TU84 SU884 AU984</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 38F to 40F BU85 TU85 SU885 AU985</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 40F to 42F BU86 TU86 SU886 AU986</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 42F to 44F BU87 TU87 SU887 AU987</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 44F to 46F BU88 TU88 SU888 AU988</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 46F to 48F BU89 TU89 SU889 AU989</td>
<td>0.21 0.42</td>
</tr>
<tr>
<td>X8 and X9, Y1 and Y3, 48F to 50F BU90 TU90 SU890 AU990</td>
<td>0.21 0.42</td>
</tr>
</tbody>
</table>

/*X4, Y1 and Y3, 1F to 2F BU68 TU68 SU868 AU968 0.21 0.42 */
/*X8 and X9, Y4 and Y5, IF to 50F BU13 TU13 SU813 AU913 0 0

// stiffness and hysteresis parameters

| U01  | 14 | 2.0e+8 | 0.0103 | C(8200,8205) Y(8235,8235) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U02  | 14 | 2.0e+8 | 0.00847 | C(6900,6900) Y(6907,6907) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U03  | 14 | 2.0e+8 | 0.00718 | C(5900,5900) Y(5920,5920) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U04  | 14 | 2.0e+8 | 0.00625 | C(5100,5101) Y(5141,5141) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U05  | 14 | 2.0e+8 | 0.00596 | C(5500,5501) Y(5541,5541) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U06  | 14 | 2.0e+8 | 0.00571 | C(5100,5109) Y(5199,5199) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U07  | 14 | 2.0e+8 | 0.00526 | C(4400,4400) Y(4402,4402) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U08  | 14 | 2.0e+8 | 0.00510 | C(3100,3103) Y(3130,3133) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U09  | 14 | 2.0e+8 | 0.00493 | C(2800,2800) Y(2850,2850) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U10  | 14 | 2.0e+8 | 0.00481 | C(2500,2507) Y(2577,2577) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U11  | 14 | 2.0e+8 | 0.00468 | C(2300,2300) Y(2330,2330) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U12  | 14 | 2.0e+8 | 0.00450 | C(2000,2009) Y(2099,2099) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U13  | 14 | 2.0e+8 | 0.00436 | C(18400,18402) Y(18482,18482) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U14  | 14 | 2.0e+8 | 0.00426 | C(17700,17709) Y(17799,17799) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U15  | 14 | 2.0e+8 | 0.00408 | C(17300,17304) Y(17344,17344) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U16  | 14 | 2.0e+8 | 0.00399 | C(16100,16107) Y(16167,16167) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U17  | 14 | 2.0e+8 | 0.00392 | C(15000,15020) Y(15028,15028) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U18  | 14 | 2.0e+8 | 0.00387 | C(14100,14106) Y(14156,14156) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U19  | 14 | 2.0e+8 | 0.00380 | C(13300,13300) Y(13397,13397) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U20  | 14 | 2.0e+8 | 0.00373 | C(12100,12100) Y(12167,12167) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U21  | 14 | 2.0e+8 | 0.00366 | C(11100,11107) Y(11157,11157) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U22  | 14 | 2.0e+8 | 0.00360 | C(10100,10101) Y(10171,10171) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U23  | 14 | 2.0e+8 | 0.00354 | C(90600,90609) Y(90639,90639) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U24  | 14 | 2.0e+8 | 0.00348 | C(87800,87806) Y(87856,87856) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U25  | 14 | 2.0e+8 | 0.00342 | C(77000,77000) Y(77780,77780) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U26  | 14 | 2.0e+8 | 0.00336 | C(67300,67302) Y(67362,67362) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U27  | 14 | 2.0e+8 | 0.00330 | C(57300,57302) Y(57362,57362) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U28  | 14 | 2.0e+8 | 0.00324 | C(47300,47302) Y(47362,47362) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U29  | 14 | 2.0e+8 | 0.00318 | C(37300,37302) Y(37362,37362) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U30  | 14 | 2.0e+8 | 0.00312 | C(27300,27302) Y(27362,27362) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U31  | 14 | 2.0e+8 | 0.00306 | C(17300,17302) Y(17362,17362) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U32  | 14 | 2.0e+8 | 0.00300 | C(10700,10700) Y(10762,10762) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U33  | 14 | 2.0e+8 | 0.00294 | C(09700,09700) Y(09762,09762) A(1,1) B(0.02,0.02) P(0.3 0.75) |
| U801 | 7.7e+7 0.0266 |
| U802 | 7.7e+7 0.0224 |
| U803 | 7.7e+7 0.0193 |
| U804 | 7.7e+7 0.0183 |
| U805 | 7.7e+7 0.0173 |
| U806 | 7.7e+7 0.0107 |
| U807 | 7.7e+7 0.0228 |
| U808 | 7.7e+7 0.0191 |
| U809 | 7.7e+7 0.0174 |
| U810 | 7.7e+7 0.0157 |
| U811 | 7.7e+7 0.0140 |
| U812 | 7.7e+7 0.0125 |
| U813 | 7.7e+7 0.0110 |
| U814 | 7.7e+7 0.00998 |
| U815 | 7.7e+7 0.00904 |
| U816 | 7.7e+7 0.00813 |
| U817 | 7.7e+7 0.00722 |
| U818 | 7.7e+7 0.00638 |
| U819 | 7.7e+7 0.00595 |
| U820 | 7.7e+7 0.00552 |
| U821 | 7.7e+7 0.00213 |
| U822 | 7.7e+7 0.00473 |
| U823 | 7.7e+7 0.0503 |
| U824 | 7.7e+7 0.0472 |
| U825 | 7.7e+7 0.0439 |
| U826 | 7.7e+7 0.0407 |
| U827 | 7.7e+7 0.0391 |
| U828 | 7.7e+7 0.0381 |
| U829 | 7.7e+7 0.0355 |
| U830 | 7.7e+7 0.0351 |
| U831 | 7.7e+7 0.035 |
| U832 | 7.7e+7 0.035 |
| U833 | 7.7e+7 0.0321 |
| U834 | 7.7e+7 0.0299 |
| U863 | 7.7e+7 0.0271 |
| U864 | 7.7e+7 0.0283 |
| U865 | 7.7e+7 0.0232 |
| U866 | 7.7e+7 0.0230 |
| U867 | 7.7e+7 0.0218 |
| U868 | 7.7e+7 0.0629 |
| U870 | 7.7e+7 0.1197 |
| U871 | 7.7e+7 0.1135 |
| U872 | 7.7e+7 0.108 |
| U873 | 7.7e+7 0.0978 |
| U874 | 7.7e+7 0.0908 |
| U875 | 7.7e+7 0.0784 |
| U876 | 7.7e+7 0.0722 |
| U877 | 7.7e+7 0.0664 |
| U878 | 7.7e+7 0.0611 |
| U879 | 7.7e+7 0.0553 |
| U880 | 7.7e+7 0.0516 |
| U881 | 7.7e+7 0.0515 |
| U882 | 7.7e+7 0.0475 |
| U883 | 7.7e+7 0.0429 |
| U884 | 7.7e+7 0.0384 |
| U885 | 7.7e+7 0.0348 |
| U886 | 7.7e+7 0.053 |
| U887 | 7.7e+7 0.043 |
| U888 | 7.7e+7 0.041 |
| U889 | 7.7e+7 0.040 |
| U890 | 7.7e+7 0.19 |
| U891 | 7.7e+7 0.19 |
| U892 | 7.7e+7 0.0542 |
| U893 | 7.7e+7 0.186 |
| U894 | 7.7e+7 0.0413 |
| U895 | 7.7e+7 0.0348 |

| U901 | 2.0e+8 0.068 |
| U902 | 2.0e+8 0.057 |
| U907 | 14.2e+8 0.0889 C(30604,18309) Y(30664,18399) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U910 | 14.2e+8 0.065 C(22301,13404) Y(22391,13434) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U911 | 14.2e+8 0.0591 C(20300,12208) Y(20380,12228) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U912 | 14.2e+8 0.0536 C(18402,11009) Y(18482,11089) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U913 | 14.2e+8 0.0486 C(16704,10004) Y(16774,10064) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U914 | 14.2e+8 0.0439 C(15103,09006) Y(15143,09086) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U915 | 14.2e+8 0.0403 C(13800,08304) Y(13880,08334) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U916 | 14.2e+8 0.0369 C(12704,07609) Y(12714,07629) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U917 | 14.2e+8 0.0331 C(11404,06804) Y(11424,06854) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U918 | 14.2e+8 0.0304 C(10404,06204) Y(10474,06284) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U919 | 14.2e+8 0.0275 C(09408,05603) Y(09488,05693) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U920 | 14.2e+8 0.0251 C(08603,05102) Y(08653,05192) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U921 | 14.2e+8 0.0227 C(07808,04600) Y(07818,04690) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U922 | 14.2e+8 0.0207 C(07105,04201) Y(07135,04281) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U923 | 14.2e+8 0.0188 C(06400,03804) Y(06490,03894) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U924 | 14.2e+8 0.0156 C(05603,03406) Y(05693,03416) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U931 | 14.2e+8 0.0182 C(06202,03708) Y(06262,03758) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U932 | 14.2e+8 0.0165 C(05603,03406) Y(05693,03416) A(1,1) B(0.02,0.001) P(0.3 0.75) |
| U933 | 14.2e+8 0.0150 C(05101,03007) Y(05161,03097) A(1,1) B(0.02,0.001) P(0.3 0.75) |
U934 14 2.0e+8 0.0135 C(04608,02800) Y(04668,02801) A(1,1) B(0.02,0.001) P(0.3 0.75)
U940 14 2.0e+8 0.0393 C(13509,08109) Y(13549,08129) A(1,1) B(0.02,0.001) P(0.3 0.75)
U941 14 2.0e+8 0.0369 C(12704,07609) Y(12714,07629) A(1,1) B(0.02,0.001) P(0.3 0.75)
U942 14 2.0e+8 0.0321 C(11004,06606) Y(11044,06626) A(1,1) B(0.02,0.001) P(0.3 0.75)
U943 14 2.0e+8 0.0474 C(16307,09804) Y(16357,09814) A(1,1) B(0.02,0.001) P(0.3 0.75)
U945 14 2.0e+8 0.0417 C(14304,08601) Y(14384,08631) A(1,1) B(0.02,0.001) P(0.3 0.75)
U946 14 2.0e+8 0.0340 C(11707,07007) Y(11727,07037) A(1,1) B(0.02,0.001) P(0.3 0.75)
U947 14 2.0e+8 0.0301 C(10308,06209) Y(10398,06239) A(1,1) B(0.02,0.001) P(0.3 0.75)
U950 1 2.0e+8 0.322
U951 1 2.0e+8 0.302
U952 1 2.0e+8 0.281
U953 1 2.0e+8 0.2605
U954 1 2.0e+8 0.25
U955 1 2.0e+8 0.244
U956 1 2.0e+8 0.227
U957 1 2.0e+8 0.217
U958 1 2.0e+8 0.205
U959 1 2.0e+8 0.194
U960 1 2.0e+8 0.188
U961 1 2.0e+8 0.174
U962 1 2.0e+8 0.162
U963 1 2.0e+8 0.147
U964 1 2.0e+8 0.162
U965 1 2.0e+8 0.133
U966 1 2.0e+8 0.132
U967 1 2.0e+8 0.125
U968 1 2.0e+8 0.106
U970 1 2.0e+8 0.29
U971 1 2.0e+8 0.275
U972 1 2.0e+8 0.262
U973 1 2.0e+8 0.237
U974 1 2.0e+8 0.22
U975 1 2.0e+8 0.19
U976 1 2.0e+8 0.175
U977 1 2.0e+8 0.161
U978 1 2.0e+8 0.148
U979 1 2.0e+8 0.134
U980 1 2.0e+8 0.125
U981 1 2.0e+8 0.114
U982 1 2.0e+8 0.105
U983 1 2.0e+8 0.095
U984 1 2.0e+8 0.085
U985 1 2.0e+8 0.077
U990 1 2.0e+8 0.39
U991 1 2.0e+8 0.35
U993 1 2.0e+8 0.33
U995 1 2.0e+8 0.77
U996 14 2.0e+8 0.070 C(24208,16000) Y(24288,16030) A(1,1) B(0.02,0.001) P(0.3 0.75)
U997 14 2.0e+8 0.062 C(21202,12701) Y(21252,12751) A(1,1) B(0.02,0.001) P(0.3 0.75)
U998 14 2.0e+8 0.053 C(18206,10900) Y(18316,10930) A(1,1) B(0.02,0.001) P(0.3 0.75)
U999 14 2.0e+8 0.051 C(17407,10404) Y(17457,10474) A(1,1) B(0.02,0.001) P(0.3 0.75)
// initial load before step by step analysis

beam X1&X7 Y1 to Y3 2F to 53F, loade 49.4
beam X1&X7 Y3 to Y4 2F to 50F, loade 30.6
beam X1&X7 Y5-Y1 2F to 50F, loade 30.6

beam Y1&Y3 X1 to X7 2F to 53F, loade 41.7
beam Y1&Y3 X7 to X8 2F to 50F, loade 34.6
beam Y1&Y3 X9-X1 2F to 50F, loade 34.6

/*beam Y4 and Y5, X1 to X7, 2F to 50F, loade 19.9
/*beam Y4 and Y5, X7 to X8, 2F to 50F, loade 17.2
/*beam Y4 and Y5, X9-X1, 2F to 50F, loade 17.2

/*beam X8 and X9, Y1 to Y3, 2F to 50F, loade 23.5
/*beam X8 and X9, Y3 to Y4, 2F to 50F, loade 15.3
/*beam X8 and X9, Y5-Y1, 2F to 50F, loade 15.3

//