EARTHQUAKE DAMAGE AND CONTINUITY OF OPERATIONS IN HIGH-TECH INDUSTRIAL STRUCTURES

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by

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In recent earthquakes, notably San Fernando and Loma Prieta, the susceptibility of many high-tech industrial structures to nonstructural, building content and limited structural damage, has been exposed. This damage places a large financial burden on the corporation, not only due to general damage, but also as a result of disruption to operations. The ability of the nation to rebound quickly following a moderate to major earthquake is dependant on the ability of such industries to maintain operations through, or recover operations shortly after, a seismic event.

The objectives of this research are twofold. In the first instance, data has been collected in regard to the performance of buildings and their contents during the Loma Prieta earthquake of October 17th, 1989. The data will focus on facilities in the high-tech industries. The data was collected so as to make much of the information previously available only to individual companies now available to the entire

engineering community. By analyzing the data it will be possible to define more clearly the emphasis that should be given to maintenance of operations when designing or retrofitting a structure.

Based on the results of the data collected, further analytical research is conducted to determine the forces likely to be experienced by a high-tech industrial structure under various levels of ground motion. Recommendations are made on maintaining continuity of operations during and after ground motion that may occur more than once during the lifetime of a structure. These suggestions are aimed at meeting service limit state objectives of preventing structural and nonstructural damage during moderate ground motion. United States seismic code provisions and philosophy are compared to that of a Japanese building code.

A case study is conducted to try and determine whether past, and current, seismic code provisions adequately meet service limit state objectives. Suggestions made in this research are also tested to determine there effectiveness in limiting structural damage to only the most severe seismic events. An elastic analysis is performed on three designs of a typical high-tech industrial structure using ETABS. An inelastic analysis is conducted on the same three designs using DRAIN-2D. Results indicate that current code procedures may not meet all service limit state objectives for the high-tech industrial structures.

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CHAPTER I
INTRODUCTION

1.1 GENERAL

Located in the middle of what is commonly referred to as "Silicon Valley" is the heart of the nation's high-technology computer industry. As early as the mid-seventies thirty percent of the electronic computer equipment industry and twenty five percent of the U.S. electronic component industry was located in the state of California [1]. These and other industries in this high-tech heartland play a crucial role in many of the nations key industries. These industries include; defense, banking and finance, electronics research and development, and the manufacturing sector. Unfortunately, these same facilities are located in an area of relatively high seismic risk. The ability of the nation to rebound quickly following a moderate to major earthquake is dependant on the ability of such industries to maintain operations through, or recover operations shortly after, a seismic event. If competitive advantage is to be maintained in many of these key industries, and by individual companies, then loss of productivity and prolonged disruption of operations cannot be tolerated.

In recent earthquakes, notably San Fernando and Loma Prieta, the susceptibility of many high-tech industrial structures to predominantly nonstructural, and some structural, damage has been exposed. Until now, most of the data concerning seismic damage and the associated economic losses has been privately held by the numerous high-tech companies. In light of the data available to individual corporations many of these high-tech corporations have undertaken seismic retrofit programs on there own impetus. These programs are aimed at reducing the extent of damage, both structural and nonstructural, and length of downtime for repair following a seismic event. By collecting the privately held data and making it available to others, the impact of future seismic events on high-tech industrial buildings and on the nation's economy can be minimized.

Design guidelines are currently available for both new construction as well as for retrofitting of existing facilities. However, these guidelines are based predominantly on life safety criteria and only to a limited extent on serviceability concerns. While life safety criteria are focused on protecting the occupants of a structure in a large earthquake, serviceability criteria are typically based on preventing damage to the building's nonstructural elements under service and long term loads. In this report serviceability issues have been broadened to include minimizing disruption of operations and structural damage during and after a moderate earthquake. If future losses in these key high-tech industries are to be avoided then certain performance standards should be outlined which address serviceability and life-safety issues more equitably.

1.2 OBJECTIVES

The objectives of this research are twofold. First, data has been collected with regard to the performance of high-tech industrial buildings and their contents during the Loma Prieta earthquake of October 17th, 1989. The data was collected so as to make much of the information previously available only to individual companies now available to the entire engineering community. By analyzing the data it will be possible to define more clearly the emphasis that should be given to maintenance of operations when designing or retrofitting a high-tech industrial structure.

Second, based on the results of the data collected, further analytical research is conducted to attempt to define quantitatively a set of performance standards for design or retrofit of high-tech industrial structures. These standards will provide the engineer with some recommendations on the serviceability issues to be addressed in order to minimize economic losses that can result from a significant seismic event. The recommendations focus on serviceability issues for small to moderate seismic events, while maintaining life safety as an over-riding concern in all seismic events.

1.3 ORGANIZATION

This thesis is divided into six chapters. In the second chapter a short description of the data collection process and an analysis of the collected data is given. In chapter three a summary of the seismic provisions of two current U.S. codes are given, and a comparison is made with the provisions of a Japanese design code. In chapter four an analytical study is performed to define quantitatively the forces that must be resisted to prevent disruption of operations and structural damage in a moderate earthquake. In the fifth chapter a case study is performed on a typical industrial facility. The existing building, the building designed according to current code and the building designed based on the results and recommendations of chapter four, are all analyzed. The three designs are subject to four different ground motions and their performance is evaluated. In chapter six the findings and results are all summarized. Conclusions and final recommendations are also provided in this final chapter.

CHAPTER II
THE PERFORMANCE OF HIGH-TECH INDUSTRIAL STRUCTURES DURING LOMA PRIETA

2.1 INTRODUCTION

The operation of a high-tech industrial facility is based on the symbiosis of structure, contents and surroundings. Industrial facilities rely on the integration of the structure, contents, equipment, nonstructural elements, support services, site location, building access, and utilities [2]. Until now much attention has been given to ensuring life-safety in the design of these buildings. However, since the 1971 San Fernando earthquake, and now the 1989 Loma Prieta earthquake, the necessity of minimizing disruption of operations in high-tech industrial buildings is becoming more apparent.

Following the Loma Prieta earthquake many companies retained structural engineers and kept records of the nature of damage their buildings sustained. The resulting loss of productivity and economic impact was also recorded for corporate use. This data has been kept private because of the competitive nature of the industry and the need for industrial security. In an attempt to make this data more accessible and useful to the industry and to the structural design profession, a number of corporations in the high-tech industry agreed to provide data for this project. Surveys were sent to the participating companies requesting data on the performance of buildings and their contents during the Loma Prieta earthquake.

In this chapter the data collection form and the data collection procedure are detailed. The data collected has been analyzed and conclusions drawn from the information are discussed. Suggestions are made for data collection following future seismic events, taking advantage of the lessons learnt in this study.

2.2 DESCRIPTION OF TYPICAL HIGH-TECH FACILITIES

The definition of what exactly constitutes a high-technology industry is relatively subjective. For the purposes of this study a high-tech industrial structure is defined as any building whose function is somehow related to the operation, production, storage or distribution of any advanced technological product, normally electronics-based. More scientific and quantitative definitions (some based on the proportion of technical occupations in the industry) have been used in the past [1]. In this research the facilities that were considered part of a high-technology industry range in function from structures that house data processing equipment, to assembly plants, to administrative offices crucial to the successful operation of a facility. The high-technology products produced at these facilities include; computer hardware, data processing equipment, advanced medical equipment, defense hardware and software, and other electronic equipment.

The typical high-tech industrial building can be constructed using any one of several structural systems. However, the general design and layout of these buildings is fairly uniform across the industry. The typical high-tech industrial facility has a large, relatively open, floor space (Figure 2.1). The floor space is versatile and can be partitioned as needed.

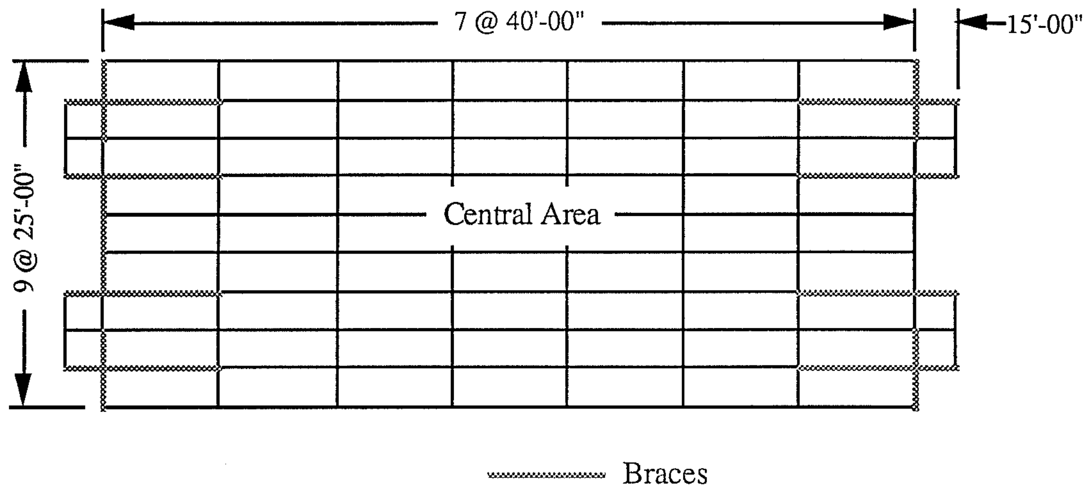


Figure 2.1 Floor Plan of Typical High-Tech Industrial Facility

Within this floor space are located the workbenches, assembly lines and conventional machine shops. Other portions of these structures are often used to house computer, data processing and data storage equipment. The administrative and other supporting operations are often housed in this same structure. These structures are typically one or two stories in height to enable quick and easy access to products and equipment on the different levels. It is also not uncommon for a high-tech company to construct many of their buildings, at different sites, from one "generic" design.

2.3 OUTLINE OF DATA COLLECTION

Following the Loma Prieta earthquake many large companies maintained records on the performance of their structures. To acquire the data on the structural system, function, and damage statistics, a data collection form was developed (Figure 2.2) in conjunction with Degenkolb & Associates of San Francisco. The form was sent to a number of high-tech companies in the area effected by the Loma Prieta earthquake, and replies were received for over two hundred buildings. It was agreed that all data would be reported and analyzed in a manner that would not identify specific company buildings or damage.

The facilities for which data was procured cover a wide spectrum of building types and functions. The data on each building was provided by the individual companies and compiled by persons with varying technical backgrounds and with different responsibilities within the corporation. In some cases structural engineers may have been responsible, while in other cases an employee responsible for maintenance, security or management of the building, may have provided the information. For this reason the data provide was not always uniform. Data was sometimes missing and was extremely subjective in certain areas. Even though results are presented anonymously, some of the companies were still reluctant to provide details on damage and economic impact estimates. Not all the data collected was received in a format consistent with that of the original data collection form. However, all data provided has been incorporated into a database in the form judged most

Data Collection Form

"A Study of Damage To Existing Industrial Facilities"

LOMA PRIETA EARTHQUAKE, OCTOBER 17, 1989

NSF GRANT NO. BCS-9011129

Company Name______________________________

Building Name/Location______________________________

Any Ground Motion Records that represent motion at the site ? Yes No

If yes, source of ground motion estimate and magnitude________________

Building Area *(square feet)*______________________________

Number of Stories________________

Structural System *(see list of model building types opposite side of form)*________

Year of Construction________________

Damage Observed in Loma Prieta Earthquake *(include structural damage, nonstructural damage, and damage to contents)*________________________

Length of time to substantially restore building operations to pre-earthquake status *(describe stages of restoration in detail if possible)*________________

How long until employees were permitted to re-enter building?________________

Nature of Repair or Rehabilitation Undertaken________________________

Cost of Repairs or Rehabilitation________________________

$ Losses Due to Business Interruption________________________

How was cost of business interruption estimated?________________________

Figure 2.2 Original Data Collection Form relevant. As more data becomes available following future seismic events the database, trends and conclusions drawn from the Loma Prieta data should be verified or updated.

2.3.1 Data Collection Form

The data collection form (Figure 2.2) was compiled with the express purpose of collecting data relevant to the performance of buildings and their contents during the Loma Prieta earthquake. The form was compiled to make data collection as simple as possible, while still providing the necessary information. The name and location of the company and building were requested for catalogue purposes but are not revealed in the analysis or this report. The approximate ground motion at the building site was provided where available. The height, floor area, structural system and year of construction were used to determine if any of these variables had an influence on the performance of the building. Data on the type of damage observed, length of time for repair, loss of productivity and cost of repair provided a measure of the economic impact of damage and disruption to operations.

2.4 RESULTS

Replies from eight different companies with complete data on ninety three buildings and partial data on another one hundred thirty seven buildings were received.

The companies from which data was procured include such diverse operations as; computer development and fabrication, defense industries and chemical or processing companies. Limited information was also provided on approximately four hundred additional buildings. This limited information covered a large square footage of corporate buildings in which no damage was observed. In the few cases where damage did occur additional notes were supplied. However, this limited information could only be used for estimating the fraction of high-tech industrial building inventory that were damaged in the Loma Prieta earthquake.

2.4.1 Structural Types

The respondents were asked to identify the structural system according to the ATC-14 [3] definitions (Figure 2.3). For analytical purposes the structural types have been divided into the following general categories; tilt-up with or without a secondary frame or braces, steel structures with or without concrete/masonry shear wall or steel braces, reinforced concrete moment resisting frames, masonry structures, and wood structures. As shown in Figure 2.4 approximately 49% of the buildings on which data was provided were tilt-up structures. The steel structural system, sometimes used in conjunction with masonry/concrete shear walls or braces, comprised almost 39% of the total. The remaining 12 to 13% were comprised of either concrete moment resisting frame, wood or masonry. A general description of the two most popular structural systems is presented below.

Model Buildings
<u>Wood Buildings</u>
Wood A, Wood Frame Dwellings and Light Frames (W1)
Wood B, Commercial or Industrial Wood Structures (W2)
<u>Steel Buildings</u>
Steel Moment Resisting Frame Buildings (S1)
Braced Steel Frame Buildings (S2)
Light Moment Frame Buildings with longitudinal Tension Only Bracing (S3)
Steel Frame Buildings with Cast -in-Place Concrete Shear Walls (S4)
Steel Frame Buildings with Infilled Walls of Unreinforced Masonry (S5)
<u>Cast-in-Place Reinforced Concrete Buildings</u>
Reinforced Concrete Moment Resisting Frame (C1)
Shear Walls Buildings (C2)
Concrete Frame Buildings with Infilled Walls of Unreinforced Masonry (C3)
<u>Buildings with Precast Concrete Elements</u>
Tilt-up Buildings with Precast Bearing Walls (PC1)
Buildings with Precast Concrete Frames and Concrete Shear Walls (PC2)
<u>Reinforced Masonry Buildings</u>
Reinforced Masonry Bearing Wall - Wood or Metal Diaphragm (RM1)
Reinforced Masonry - Precast Concrete Diaphragm Building (RM2)
<u>Unreinforced Masonry Buildings</u>
Unreinforced Masonry Bearing Wall Buildings (URM)

Figure 2.3 Structure Types According to ATC-14 [3]

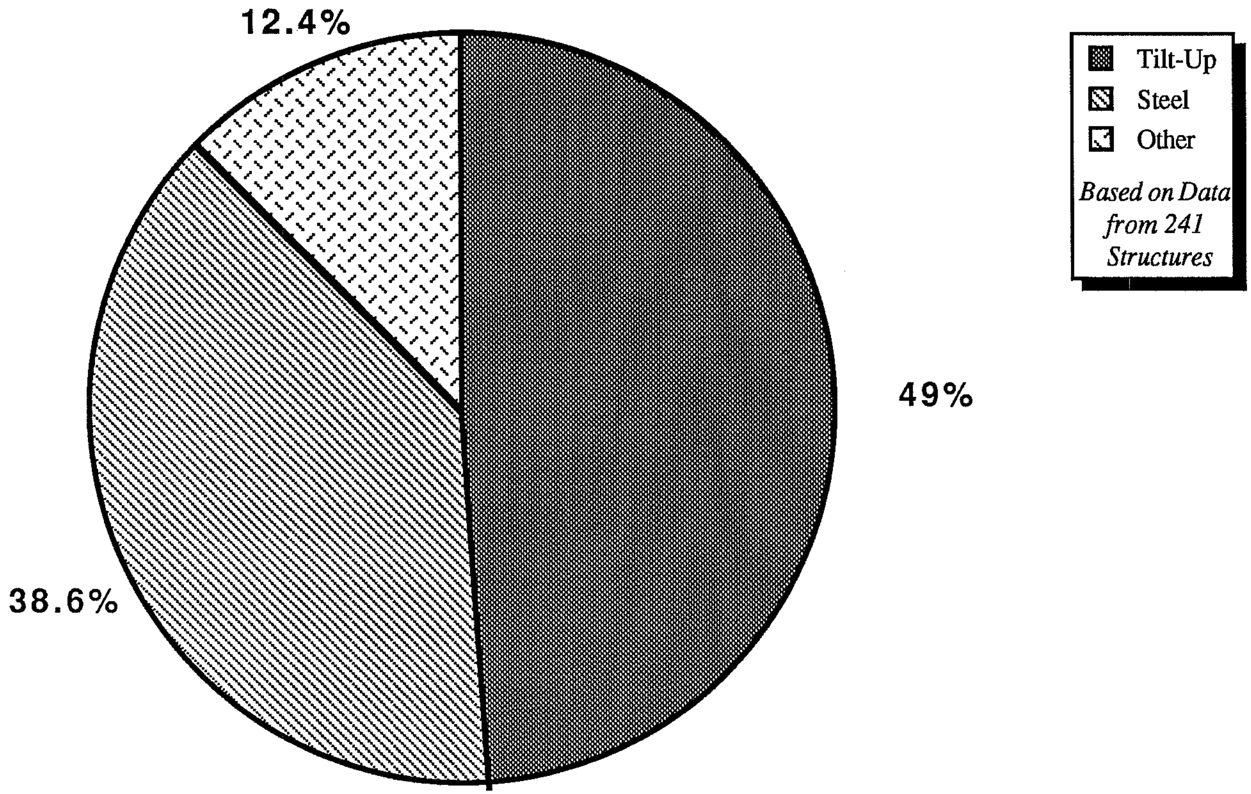


Figure 2.4 Structural Systems of High-Tech Industrial Structures

2.4.1.1 *The tilt-up structure*

The most common industrial facility surveyed, the tilt-up structure, remains a popular form of construction in the high-tech industry. Figure 2.5 shows that the tilt-up structure has steadily gained in popularity from its almost non-existence in the first half of this century to a peak in popularity in the late seventies. Over sixty percent of high-tech industrial structures constructed in this period were tilt-up. The typical tilt-up structure is constructed with plywood sheathed wood-frame roofs and diaphragms supported by perimeter concrete walls (Figure 2.6) [4,5]. Tilt-up structures are popular due to their ease and speed of construction. The precast walls are often constructed on top of the floor slab and then tilted into position around the building. The main lateral force resisting system is the in plane stiffness and strength of the tilt-up panels, but a secondary braced or moment resisting frame is also common.

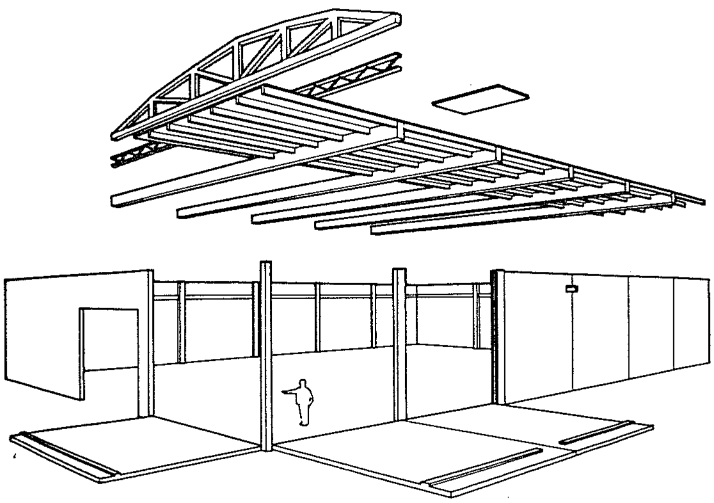


Figure 2.6 Typical Tilt-Up Construction used in the High-Tech Industry [4]

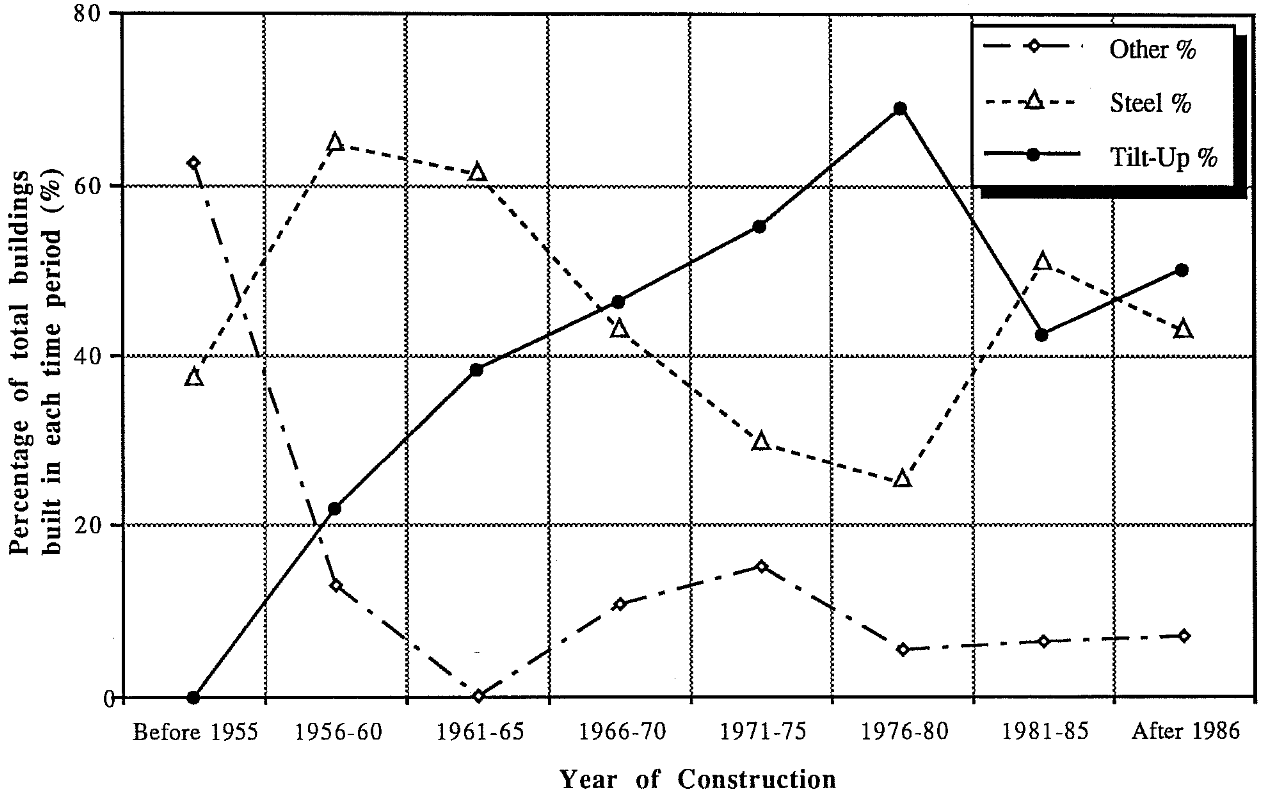


Figure 2.5 Structural Systems used in High-Tech Industrial Buildings Through Time

The vulnerability of tilt-up structures was exposed during the 1971 San Fernando and the 1987 Whittier earthquakes. Seismic provisions improving the roof-to-wall, floor-to-wall and wall-to-wall anchorage requirements were implemented following the San Fernando event. Many tilt-up buildings, constructed prior to the 1973 code revisions, continue to pose life-safety hazards in a major earthquake. The magnitude of the problem is apparent in Figure 2.7 which shows that 37% of the tilt-up structures in the survey, and currently in use, were built prior to the 1973 code revisions. The number that have since been retrofitted is unknown. In the Loma Prieta earthquake most of the damage to tilt-up structures was a result of non-structural items falling against the tilt-up walls, and as a result of known structural deficiencies (such as weak roof-wall anchorage)[5,6]. Fortunately, many of these buildings had little or no damage because the level of ground motion was low.

2.4.1.2 *Steel MRF and braced frame structures*

The steel moment resisting frame (MRF), steel braced frame and steel frame with concrete shear wall, are also popular structural systems used in the high-tech industry. These types of construction for high-tech facilities were most popular in the late fifties and early sixties (Figure 2.5). During this time period the survey showed that over sixty percent of the structures built were of these types of steel structures. The steel structural systems rely on moment connections, concentric/eccentric bracing or shear walls for lateral resistance. The modern-steel frame building often has exterior cladding which could include corrugated metal siding or precast concrete panels (Figure 2.8).

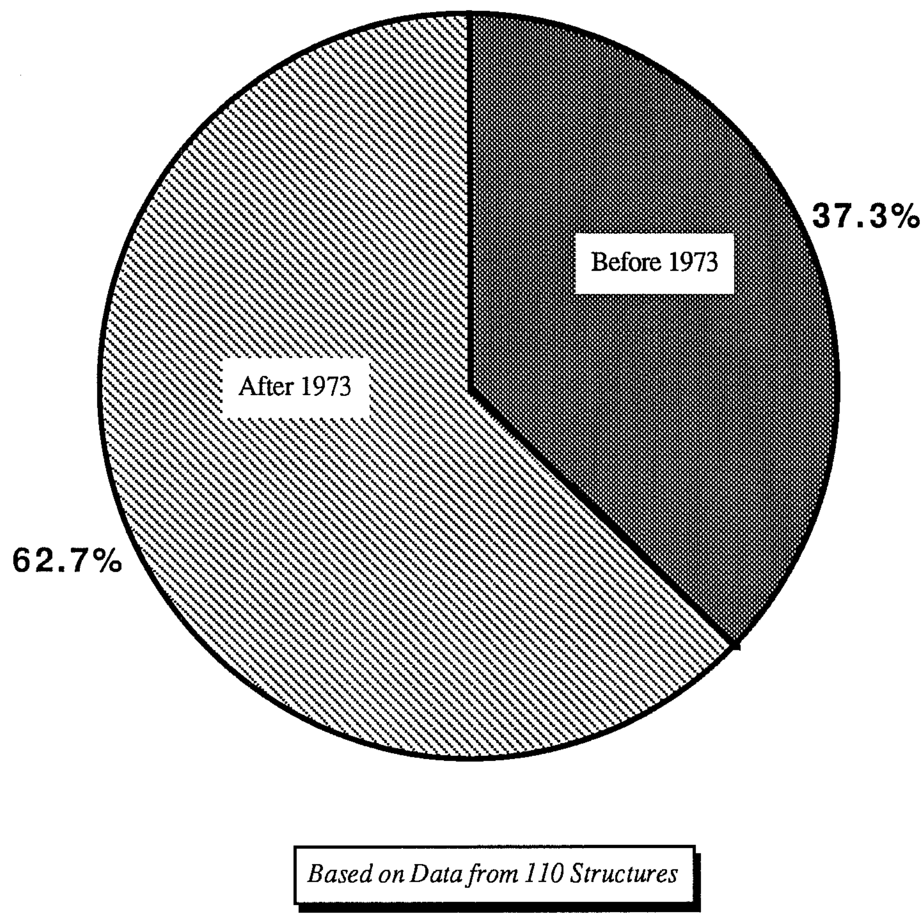


Figure 2.7 Fraction of Tilt-Up Structures Surveyed by Code Year of Design

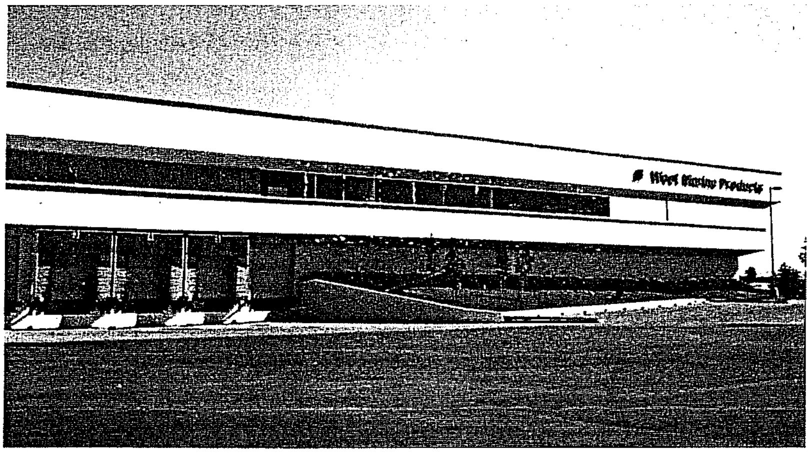


Figure 2.8 A Typical Steel High-Tech Industrial Structure

The steel structure relies on flexural hinging, brace buckling and yielding, or shear wall hinging to dissipate the most energy in a major earthquake. As with many steel structures (in particular with the steel MRF) these buildings are often considered to be too flexible [2,5]. Due to the expected building frame deformations additional care is required when designing and installing non-structural elements and equipment into these buildings.

2.4.1.3 *Other structural systems*

Other structural systems that were surveyed include wood, concrete and reinforced masonry structures. These types of construction were, however, not as common as the previously mentioned systems. Most of these types of structures suffered only minor damage in the Loma Prieta for several reasons. Wood construction is a particularly light form of construction and as such the inertial forces are relatively low in these structures. Most of the wood facilities were built prior to the sixties and as therefore probably house older and less expensive equipment so losses are not as substantial. The concrete frame structure is used both for industrial plants and as supporting administrative offices. While many of the larger and older concrete frame structures sustained major damage during the Loma Prieta earthquake [5], damage to the smaller concrete frame high-tech industrial structures was minimal. The concrete moment resisting frame is, however, a relatively flexible system and care is required to ensure that equipment and nonstructural elements can sustain the expected frame deformations. In the masonry buildings damage was relatively minor, probably because they were fairly well reinforced. However, damage was observed in many of the unreinforced masonry structures [5].

Probably the greatest factor that kept damage in these types of facilities to a minimum was the relatively low levels of ground motion. While most of the wood, masonry and concrete high-tech industrial structures experienced little structural damage, there was damage to nonstructural elements, partitions and veneers in several of these structures [5].

2.4.2 Typical Damage

One of the main objectives behind the data collection survey was to ascertain the amount of damage and the form of that damage. Respondents were asked to provide data on the type of damage that was observed, both structural, nonstructural and to contents. Information was received for two hundred and thirty facilities. Of all these facilities only forty five or 19.6% had significant damage (Figure.2.9). For purposes of this study significant damage is defined as any financial loss over one thousand dollars. Where costs were less than one thousand dollars they were typically required to pay for time to do minor patchwork, painting and reorganizing of contents.

2.4.2.1 *Structural damage*

Structural damage was reported in eight (3.5%) of the two hundred and thirty structures. Typical structural damage included buckled braces, gusset plates and weld failures in steel structures. Two steel buildings experienced major cracks to concrete floor slabs. Concrete cracking and spalling in tilt-up structures were also reported. Of the eight that were structurally damaged four were tilt-up structures and four were steel structures. Two of the tilt-up structures damaged were constructed prior to the 1973 code revisions and suffered predictable damage. Damage to the concrete floor slabs in the steel buildings was typically caused by design and construction deficiencies. Unfortunately, the level of structural damage that can be expected in an earthquake of the magnitude of Loma Prieta is not specified in the codes. As each structural system has different ductility properties they are designed for different lateral strength. For this reason two buildings at the same location, each with a different structural system, may experience different levels of structural damage.

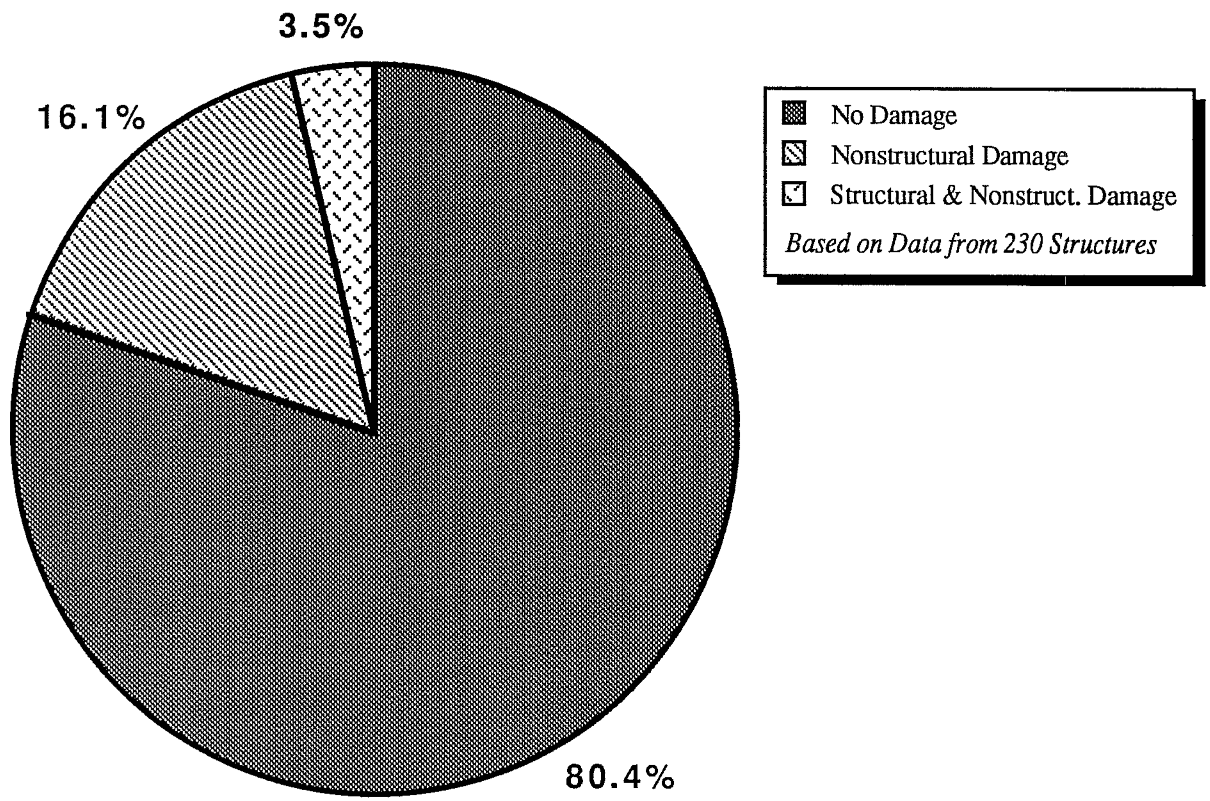


Figure 2.9 Typical Damage to High-Tech Industrial Structures during Loma Prieta

2.4.2.2 *Nonstructural and building content damage*

All forty five of the facilities that had over one thousand dollars in avoidable damage suffered nonstructural or building content damage. Previous earthquake damage has been fairly well documented [7,8] and correlates closely to what was reported in this event. The most common damage observed was dislodging and overturning of furniture, storage racks and their contents (Figure 2.10). Ceiling tile damage and minor cracking were just as prevalent. Burst pipes, dislodged air conditioning ducts, dislodged light fittings and broken windows were observed to a lesser, but still significant, extent. Other forms of nonstructural damage observed include roof purlin rotation, equipment and scanning electron microscope chillers damaged, storage tank buckling and some minor buckling in floor systems.

2.4.3 Economic Impact

The financial loss estimates of damage as a result of Loma Prieta range from four to over eight billion dollars [9,34]. While only a small part of this is as a result of damage to high-tech industrial buildings, the indirect economic impact of damage to these structures is immeasurable. While direct repair costs are easily obtained, the loss of productivity and related economic impact to a specific company, industry and the community is more difficult to ascertain. Fortunately, in the Loma Prieta earthquake the ability of individual corporations and of the regional economy to rebound following a large earthquake was not severely tested. Had the earthquake occurred closer to many of the facilities, or been of greater intensity, the socioeconomic consequences could easily have been devastating. Loma Prieta did, however, allow

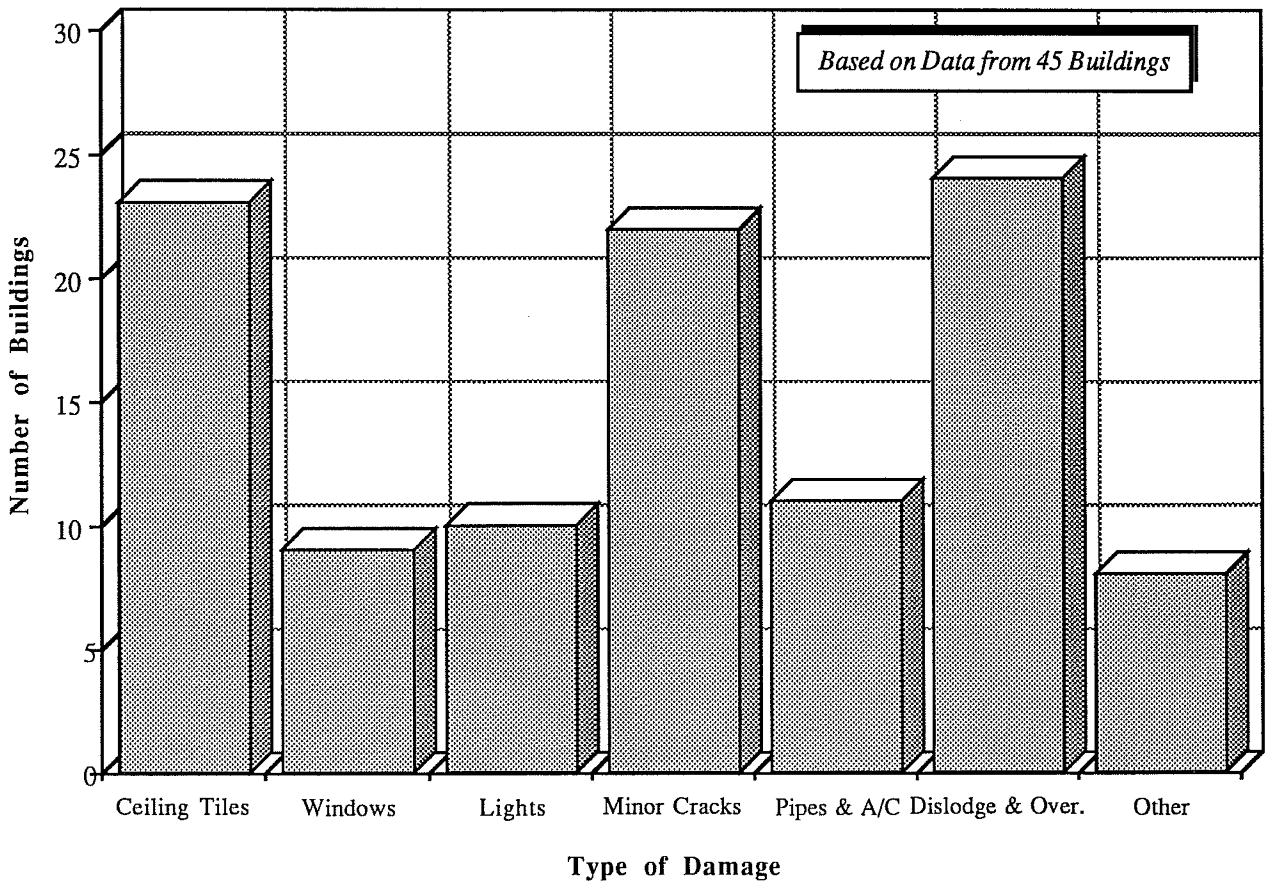


Figure 2.10 Typical Nonstructural Damage in High-Tech Industrial Buildings during Loma Prieta

for invaluable data to be obtained on the economic impact associated with disruption of operations. This data can be used to determine the emphasis that should be placed on ensuring continuity-of-operation when retrofitting or designing a structure for use in the high-tech industry.

2.4.3.1 *Loss-of productivity*

As previously mentioned complete data was received for ninety three buildings. The assessment of loss due to business interruption proved to be one of the most subjective pieces of data received. Some companies were willing to provide costs related to lost contracts, salary expenses during repair, overtime expenses, and loss of productivity during clean up or moving, while other companies declined. For this reason only twenty four of the facilities have economic figures associated with the loss of productivity. These figures serve to highlight one significant point; the extremely large expense associated with disruption of operation as compared with the expense associated with both structural and nonstructural repair. As shown in Figure 2.11 the losses associated with disruption of operation is over three times that associated with repair. The dollar values shown in this figure are only for those facilities where both loss of productivity and repair costs were supplied. This emphasizes that in many cases the disruption of operations could mean a greater financial set back to corporations than does the cost of repair. This data would seem to justify the need to sometimes retrofit structures while operations within the structure continue.

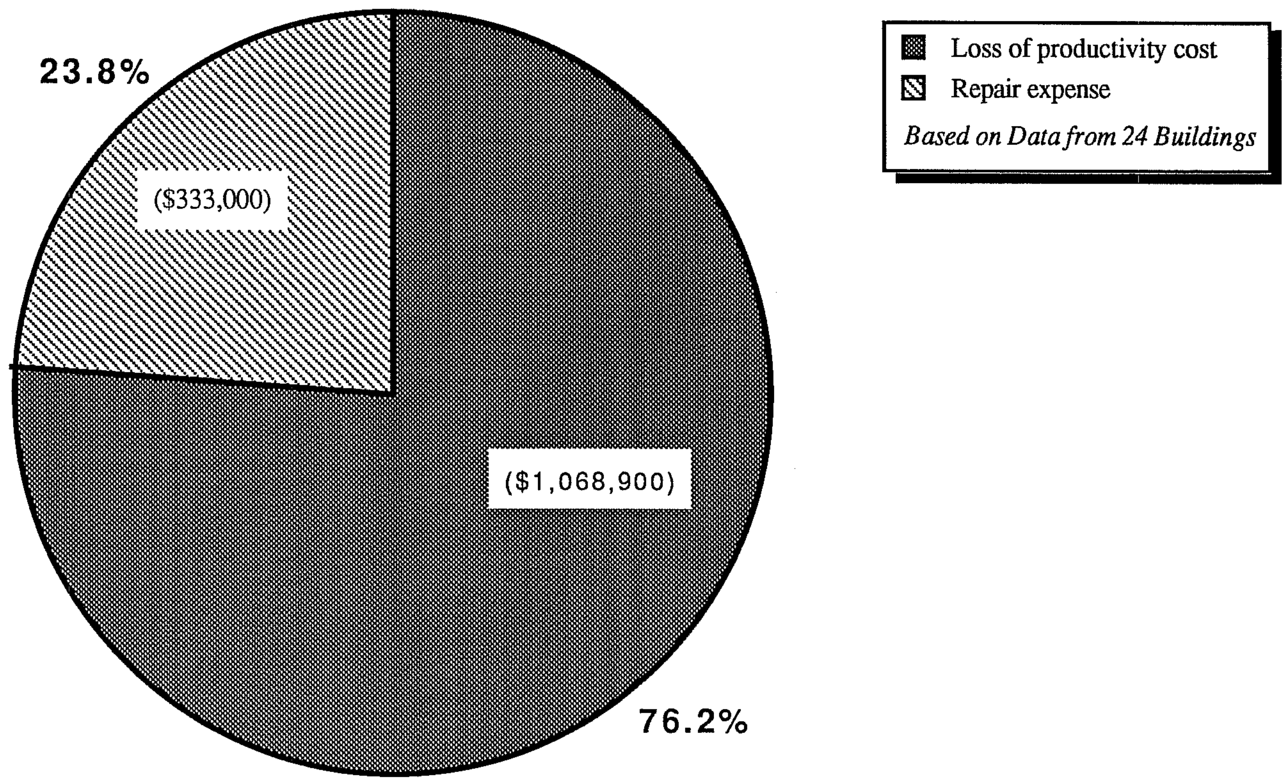


Figure 2.11 Loss of Productivity Costs vs Repair Expense from Loma Prieta

2.4.3.2 *Time taken for repair*

The adage that "time is money" is certainly evident in the correlation between losses and repair time following Loma Prieta. Corporations provided data on the length of time that building operations were interrupted following the earthquake. As shown in Figure 2.12 the longer the facility was out of commission the larger the financial loss (structural, nonstructural and loss of productivity). While the values in the figure are not necessarily applicable to all facilities, the over-riding message is that downtime increases losses dramatically. What Figure 2.12 does not indicate is that ninety one percent of the facilities surveyed had repair times of less than two weeks. Once again it should be remembered that many facilities were not severely tested in this earthquake. Had the epicenter been closer to many of these facilities the average repair time would be longer and the economic losses would have increased accordingly. Of equal importance is the fact that those structures that required more than two weeks to repair all suffered structural damage. This highlights the fact that structural damage, when it occurs, has high associated recovery costs and must be avoided if possible. The importance of ensuring continuity-of-operations and not just life safety is seen to be crucial.

2.4.4 Effect of Code Revisions

Since the first Uniform Building Code was published in 1927 there have been multiple revisions at approximately three year intervals. There have been numerous additional codes and local city ordinances that have also come into existence. As our knowledge of materials, structural behavior and analysis has improved through time,

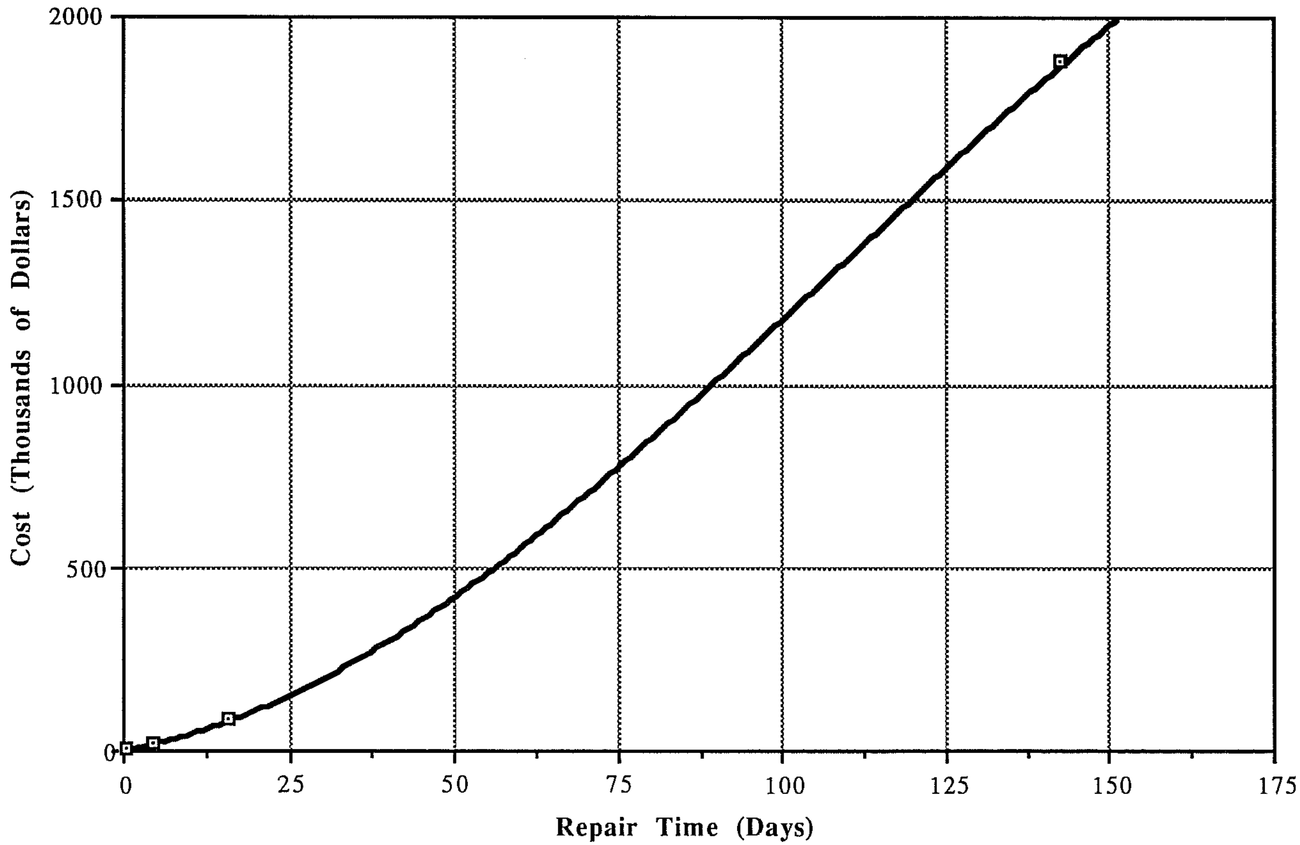


Figure 2.12 Repair Time versus Total Cost

so too has the size and shape of our structures. Through time the materials used in construction have improved and the acceptable working stresses increased [10]. What has not changed through time is the basic code philosophy concerning life safety.

2.4.4.1 *Effect of code revisions on observed damage*

The Uniform Building Code (UBC) is discussed in this section because it is most often referred to in California building regulations. An attempt has been made to determine whether those facilities built in any specific time period were more susceptible to damage than those in other time periods. Figure 2.13 illustrates the point that in these high-tech facilities no trend is apparent in the age of buildings versus the number of structures that experienced substantial restoration costs. Also, structural damage did not appear to be more prevalent in any particular type of structure. The buildings that were structurally damaged were constructed in the time period from 1962 to 1985. Although there are many additional factors that influence Figure 2.13, until serviceability issues are addressed more directly in the code no improvement is likely to be observed.

2.5 DISCUSSION

In the process of compiling and analyzing the data, several problems were identified. These problems provide valuable lessons that should be considered in developing future data collection forms of this nature. The problems can be divided into two broad categories, these being subjectivity and trust.

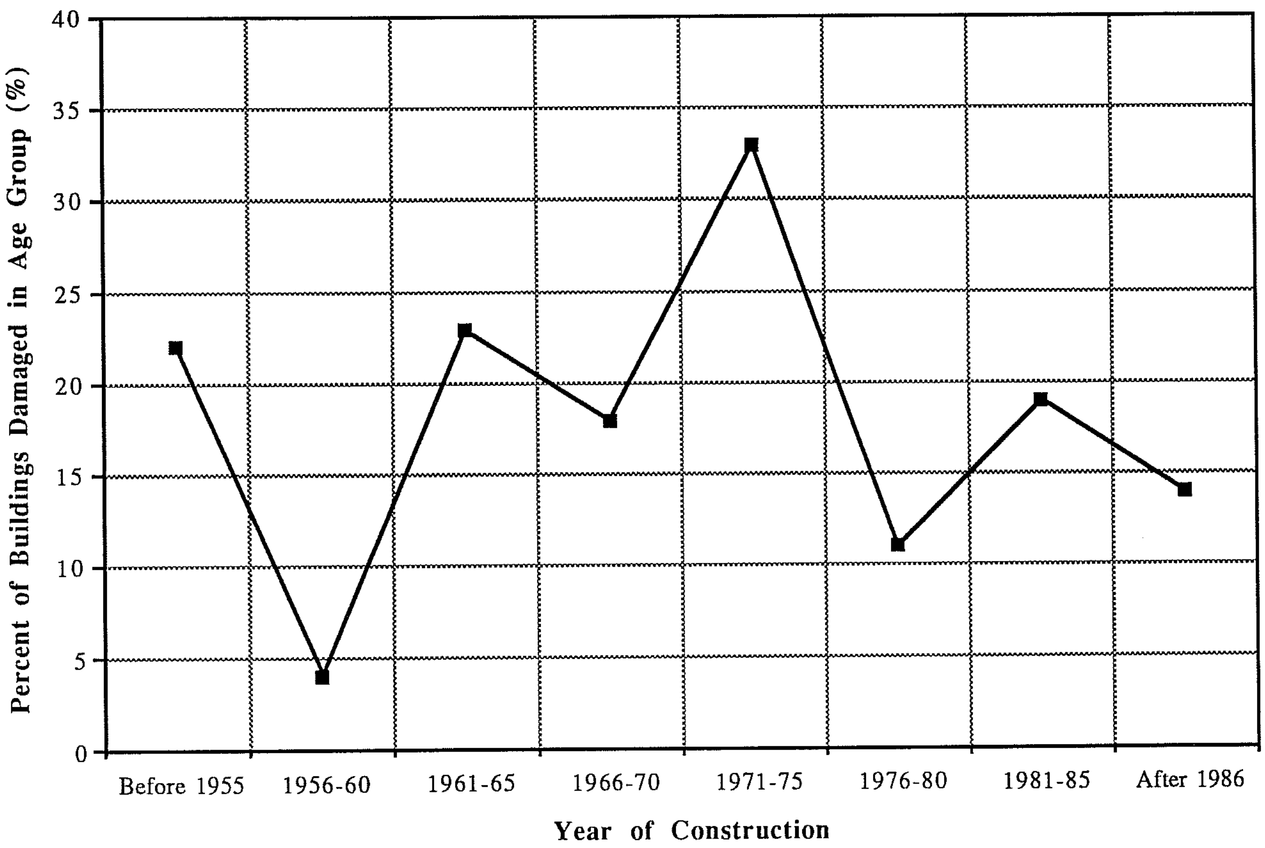


Figure 2.13 Trend in Quantity of Buildings Damaged during Loma Prieta by Age of Construction

Unfortunately, several questions in the original data collection form (Figure 2.2) proved to be open to interpretation. This allowed for subjectivity to become an issue which may have resulted in less scientific data being received. For example, the word substantially in the question on the length of time to substantially restore building operations could be interpreted in various different ways. The question of estimates on dollar losses due to business interruption also needs to be more clearly defined. For example, if production was delayed, but could be caught up in overtime, then the only loss is overtime expense. If a production line is damaged and all the work in progress is destroyed then product loss and potential loss of business should be included in the loss estimates. Salaries paid to employees who are not normally responsible for maintenance, and must take time from other duties to conduct clean up and minor patch work, should also be included in loss estimates.

Finally, companies need to be more willing to reveal data on the general function of each facility, the type of equipment present, and the damage sustained in an earthquake. Two similar structures can suffer widely different damage and/or losses. Only when this type of data is available can it be used to reduce the losses, not only to the individual corporations and insurance companies, but to the industry group as a whole.

2.5.1 Proposed Survey Form

A preliminary survey questionnaire is suggested (Figure 2.14). Simplicity is essential to ensure that responses are accurate and quick. The intention of this

DATA COLLECTION FORM

To be completed by the structural engineer before the earthquake

Company Name______________________

Building Name______________________ Location________________

Structural System (ATC-21)_____________ Stories 1 2 3

Code Year of Design__________________ Year of Construction_______

Final Score (ATC-21)_________________ Comments______________

Describe Soil Conditions (UBC-91)________________________________

Location of Closest Seismic Recording Station__________________________

What Function is Performed in this Building____________________________

Approximate Number of Employees at the Facility________________________

To be completed by an employee or engineer after the earthquake

Structural Damage

Is there any structural damage that requires repair (columns, floor diaphragm, beams, walls, braces etc) ? Yes No

If the answer to the above question is Yes then please complete the following.

What structural damage was observed, please describe ? ______________________

____

How long were structural repairs to restore the structure to original condition ? ____

What was the cost of structural repair performed by independent contractors ? _____

Nonstructural Damage

Is there any nonstructural damage that requires repair (pipes, infill walls, partitions, windows, lights, exterior cladding, air conditioning, tiles) ? Yes No

If the answer to the above question is Yes then please complete the following.

What nonstructural damage was observed, please describe ? __________________

____

How long were nonstructural repairs to restore the structure to original condition ? _

What was the cost of nonstructural repair performed by independent contractors?___

Figure 2.14 Recommended Survey Questionnaire

To be completed by an employee or engineer after the earthquake

Content Damage

Is there any damage to contents that requires repair (equipment or machinery damaged, racks overturned, furniture damaged, product damage, water damage) ?

Yes No

If the answer to the above question is Yes then please complete the following questions;

What damage to contents was observed, please describe ? ____________________

____

How long were repairs to restore the contents to original condition ? ____________

What was the cost of restoring, or replacing, contents (NOT including time spent by employees that were in employment prior to the earthquake) ?__________________

Disruption of Operation Costs

Were operations at the facility disrupted or could they have gone on unabated following a safety check of the facility ? Disrupted No disruption

If disrupted, what length of time was required before production could be resumed at pre-earthquake levels (even if at a new facility) ? ___________________________

What length of time was allowed employees to sort out private matters ? _________

How many hours where spent by **employees** in restoring operations to pre-earthquake levels (either for clean-up, moving, calibration, reorganization etc.) ? __

What were the salary costs associated with **employees** time used to restore operations to pre-earthquake levels ? _______________

What was the cost associated with lost contracts or overtime required to regain the original scheduled output or meet deadlines ? ______________________________

Additional Comments

Please use the space provided to provide any additional information that you find relevant ? ____

____

____

Figure 2.14 Recommended Survey Questionnaire

questionnaire is to try and minimize subjectivity and provide more uniform and accurate data following a major earthquake. This aim can be achieved by getting engineers to identify structural types and evaluate seismic risk of buildings prior to future earthquakes. At the same time the accuracy of evaluation techniques can be tested. The closest ground motion recording station should be located and recorded prior to the earthquake. This will relieve the burden on the person completing the form of determining the approximate ground motion experienced by the structure. Some important data such as site soil conditions are also necessary. The importance of local site geology was highlighted in the Loma Prieta event [11]. This type of questionnaire should be distributed to as many high-tech industrial facilities as possible, prior to a future earthquake. Following an earthquake a specific employee should be made responsible for collecting all the appropriate information and completing the questionnaire. The completed form should then be directed to a central location where the data can be used to improve and extend the current database.

2.6 CONCLUSIONS

Despite all the problems associated with the data collection and analysis, the results do show several points that should be examined further. While data has long been collected on the types of damage observed in major earthquakes, the effect of disruption of operations on individual companies has long been withheld from the engineering community. Now that this data has begun to surface there are several interesting trends observed.

i) It is apparent from Figure 2.4 that a large proportion of high-tech industries, located in the northern half of California, are housed in low rise steel or tilt-up structures. The performance of these particular types of structures needs to be examined to determine whether they adequately address the serviceability needs of the high-tech industries.

ii) The damage observed in high-tech industrial structures during Loma Prieta is consistent with what has been observed in previous seismic events [7,8]. Structural damage was observed in some structures but nonstructural and content damage was more prevalent.

iii) It was also evident that where losses due to business operation were provided they proved to be over three times as costly as the total repair cost at a facility. This reaffirms the claim that minimizing disruption to operations in an earthquake needs to receive increased emphasis when designing or retrofitting a structure. This is particularly true when considering the moderate seismic events that a building is likely to experience more than once in its lifetime. Structural damage, when it occurred, was seen to result in huge repair and loss of business costs. This type of damage needs to be limited to occur in only the most severe of earthquakes. This is particularly true in the high-tech industry where months of business disruption for structural repair cannot be tolerated.

iv) Last, it was seen that although there have been many changes made to the codes through the years, the serviceability problem still exists. Recently built high-tech industrial structures did not appear to perform any better than did the older structures. Unless the nonstructural and structural problems are addressed in greater detail they will continue to cause unnecessary business interruption in future earthquakes.

In the remainder of this study the third of these above mentioned conclusions will be examined more closely. That is whether additional emphasis needs to be placed on minimizing disruption of operations, due to structural damage, when designing or retrofitting a structure. Many of the concerns over nonstructural and building content damage has been addressed in other references [7,12] and are not extensively covered in this research. However, the importance of properly protecting and installing nonstructural elements and building contents has once again been highlighted in the above data.

CHAPTER III
SEISMIC DESIGN CODE PROVISIONS

3.1 INTRODUCTION

Current United States seismic code provisions focus on providing life safety in the event of a major earthquake. As this philosophy is intended to ensure the survival of building occupants it does not always prevent damage or disruption of operations in a structure. However, in the previous chapter it was observed that maintaining continuity of operations, and not only life safety, is a crucial issue to the high-tech industries. This issue is particularly important in light of the fact that many high-tech industrial structures are likely to be exposed to more intense ground motion than that experienced in Loma Prieta. For this reason there is potential for even greater disruption to operations and economic losses in future earthquakes.

In this chapter the seismic provisions of the Uniform Building Code 1991 (UBC-91) [13], National Earthquake Hazard Reduction Program Recommendations for New Buildings (NEHRP-88) [14] and the Japanese Building Standard Law (BSL) [15] are summarized. A comparison of the lateral design force prescribed by the UBC-91, NEHRP-88 and the BSL, for a typical west coast, high-tech industrial structure, are presented.

3.2 OUTLINE OF BUILDING CODES

Provisions of both the U.S. and Japanese seismic codes can be categorized by the two performance standards on which they are based. A description of the objectives of each performance standard is presented below.

The first is a serviceability standard intended to keep a structure operational and to minimize damage during a moderate earthquake. This performance standard is outlined in the Structural Engineers Association of California (SEAOC) *Recommended Lateral Force Requirements* [16] on which the UBC-91 and NEHRP-88 are based. According to SEAOC a structure built to their recommendations should be able to perform as follows:

- Resist minor levels of earthquake motion without damage.
- Resist moderate levels of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.

This performance standard is commonly referred to as the service limit state (SLS).

The second standard is intended to ensure that a structure remains standing and that life safety is preserved in an extreme earthquake. This is summarized in the SEAOC recommendations as follows:

- Resist major levels of earthquake ground motion having an intensity equal to the strongest either experienced or forecast at the building site, without collapse, but possibly with some structural as well as nonstructural damage.

This performance standard is referred to as the ultimate limit state (ULS). Table 3.1 summarizes the provisions specified by the codes to meet each of these limit states.

Table 3.1 Code Provisions for Service and Ultimate Limit State

LIMIT STATE	UBC-91 & NEHRP-88	BSL
Service	•Provisions that limit the lateral story drift under code prescribed forces. •Provisions that prescribe lateral design forces on elements of structures and nonstructural components supported by the structure.	•Provisions that limit the lateral story drift under code prescribed forces. •Provisions that prescribe lateral design forces on elements of structures and nonstructural components supported by the structure. •Provisions that prescribe the lateral design forces to be resisted by the structure and still stay operational.
Ultimate	•Provisions that prescribe the lateral forces that must be resisted by the structure to ensure life safety. •Provisions that prescribe the ductility required in a system to justify design approach and ensure life safety.	•Provision that ensure the built in over-strength and ductility from the service limit state design are adequate to prevent structural collapse in an extreme earthquake.

As shown in Table 3.1 the widely used UBC and the model NEHRP-88 both attempt to meet the service limit state by limiting lateral drifts and specifying design forces for nonstructural elements. Although not explicitly specified, it is the intention of both the UBC-91 and NEHRP-88 that the strength prescribed for the ultimate limit state is sufficient to prevent structural damage in the service limit state. This assumption has recently been challenged by some who feel it may not meet the performance criteria of the service limit state [17]. In the Japanese BSL, however, the strength required to keep a structure operational is specified explicitly under the service limit state provisions.

Both the U.S. and the Japanese seismic provisions meet the ultimate limit state by prescribing minimum levels of strength and ductility for a structure. Building configuration, layout, element sizing and detailing are also addressed by the codes to prevent structural collapse in major earthquakes.

3.3 PROVISIONS FOR THE SERVICE LIMIT STATE

As outlined earlier the service limit state is based on serviceability criteria which should keep a structure operational and damage to a minimum during a moderate earthquake. The code provisions for meeting the service limit state objectives are based on design forces for nonstructural elements (all codes), restricting lateral drift (all codes) and specifying minimum design lateral forces on a structure (BSL only).

3.3.1 Provisions for Nonstructural Elements

Provisions for nonstructural elements provide guidelines for determining the design level of seismic forces for architectural, mechanical, electrical and other nonstructural elements and equipment. The most common approach for specifying these design forces is the equivalent static lateral force procedure. In certain cases, however, a more detailed analysis considering the dynamic properties of the nonstructural element and the supporting structure is prescribed. The seismic performance and design provisions for nonstructural elements and equipment has been well researched and documented [7,8], and are discussed briefly below.

3.3.1 1 *Equivalent static lateral force procedure*

Nonstructural components that are considered rigid (fundamental period less than 0.05 seconds), or rigidly anchored to the structure, will essentially respond to the motion of the structure. For this reason the static lateral force procedure is traditionally prescribed for rigid or rigidly mounted nonstructural components [2]. This procedure assumes that the equipment has a mass less than 10% of the mass of the supporting floor, and therefore does not significantly influence the response of the structure. In the static lateral force procedure, design loads are determined as a function of the equipment weight, building use or occupancy, storey shear force coefficient, seismic risk zone and sometimes on the location of equipment in the building. Codes such as NEHRP-88 consider the effect of height by prescribing an amplification factor anywhere from 1.0 to 2.0 depending on the location of the equipment between the ground and roof level respectively. The basic form of the lateral force equation is shown below.

$$\mathbf{F} = \mathbf{Z\ I\ C\ W} \qquad (3\text{-}1)$$

where, $\mathbf{F}$ = the equivalent static lateral load for design purposes

$\mathbf{Z}$ = a variable dependent on the level of ground motion expected

I = coefficient dependent on building use or occupancy

C = a storey shear force coefficient which may vary with the location (height), and type, of the nonstructural element in the building

W = the weight of the element.

The UBC-91 and NEHRP-88 both use this method to specify design forces for nonstructural components, with additional criteria on their use contained in the respective codes.

3.3.1.2 *Dynamic analysis*

In a dynamic analysis the dynamic properties of both the nonstructural component and the supporting structure must be considered. This procedure is traditionally prescribed for flexible or flexibly mounted components, and for equipment crucial to the operation of an essential facility. The supporting structure can be analyzed using either the time-history, modal superposition or direct integration method in order to determine the time histories of motion at various floor levels. These time histories can then be used to calculate single degree of freedom damped response spectra for each floor. If the equipment is fairly rigid (natural period between 0.04 and 0.05) it can be designed for a static force coefficient equal to the peak acceleration at its supports. If the equipment is flexible its response can be determined from the response spectrum, modal superposition method, or a static coefficient equal to 1.5 times the peak response acceleration from the response spectrum used [2]. When nonstructural components significantly influence the behavior of the supporting structure (i.e. equipment mass larger than 10% of the supporting floor) then the two must be analyzed together as a coupled system [2]. These dynamic procedures are described in both the UBC-91 and the NEHRP-88 provisions.

3.3.2 Provisions Limiting Lateral Drift from Seismic Loads

All building codes limit the lateral deformation that is acceptable under the code prescribed lateral forces. These lateral drift limits are meant to prevent major nonstructural damage in the event of an earthquake, but are based predominantly on preventing damage to architectural elements [13,14]. As such, these limits do not prevent damage to specific equipment within a structure [7], nor do they prevent disruption of operations during an earthquake. In both the UBC-91 and the BSL the story drift ratios are based on elastic structural response and range between 0.33% to 0.5%, depending on building function and structural system. Higher levels are acceptable where it can be shown that both nonstructural and structural elements can sustain the deformation expected. The NEHRP-88 prescribes drift limits that consider the inelastic action that is likely to occur in a major earthquake. As such these limits are higher than those of the UBC-91 and BSL, and range from 1% to 2% depending on the structures function.

3.3.3 Provisions for Minimum Design Lateral Force

Only the BSL explicitly specifies a minimum design lateral force in the service limit state. While the UBC-91 and NEHRP-88 provisions do prescribe a minimum design lateral force it is based on life safety performance criteria (ultimate limit state), and not on achieving the service limit state objectives. The Japanese seismic design provisions underwent a major revision in 1980-81, at which time a two-phase seismic design approach was introduced. This new Building Standard Law (BSL) was rapidly implemented following the significant damage caused to modern buildings in the 1978 Miyagi-ken-oki earthquake. The basic philosophy subscribed to by the Japanese BSL is summarized in this statement [15]:

> ..the adopted two-phase design procedure can be regarded as the design for two different intensities of earthquake motion. The purpose of the first phase design is now to protect buildings from loss of function in case of earthquakes which can occur several times during the life of the building.

3.3.3.1 *First phase design of BSL*

As is shown in Figure 3.1 buildings are initially classified according to their height. The typical high-tech industrial structure would be less than 31 meters (101.7 feet) (Box 2), and would therefore require a two phase design. The first phase design (Box 5) is for general (dead, live, wind, etc.) and earthquake loads. To determine the design base shear the BSL prescribes a static lateral force procedure comparable with the UBC-91 and the NEHRP-88 recommendations. The equation to determine the total seismic shear at floor level i is given in Equation (3-2) below.

$$\mathbf{Qi = Ci \sum_{i=i}^{n} Wi} \qquad (3\text{-}2)$$

Where, **Wi** = weight of i-th storey, including all other applicable loads

Ci = **Z Rt Ai Co** (3-2a)

where, **Z** = seismic zone factor

Rt = vibrations characteristic factor

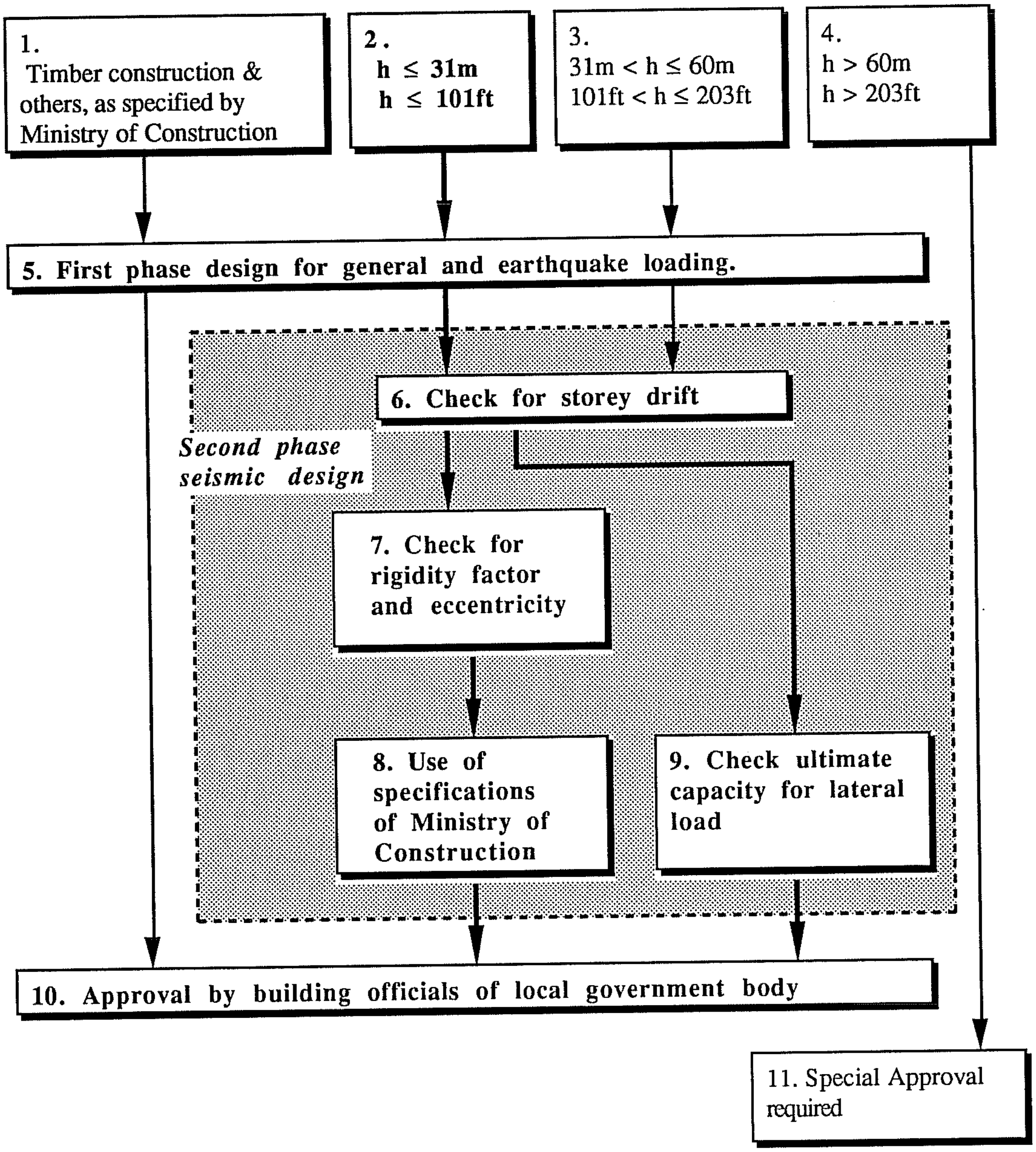


Figure 3.1 Japanese Building Standard Law Seismic Design Outline

Ai = vertical distribution factor

Co = standard shear coefficient.

This calculation determines the total earthquake shear force at a particular floor level (i.e. includes the contribution of the shear force from the floors above the floor level of interest). The variables in Equation (3-2a) are described below.

i) The seismic zone factor (Z) is a value given to the expected level of seismicity in the building location and can vary from 0.7 to 1.0. The larger value is representative of an area of greatest seismicity and would be appropriate for the west coast of the United States.

ii) The vibration characteristic factor (Rt) is meant to take into consideration the soil characteristics and the soil structure interaction. The Rt value is a function of the structures natural period, T, and the type of subsoil Tc (Figure 3.2).

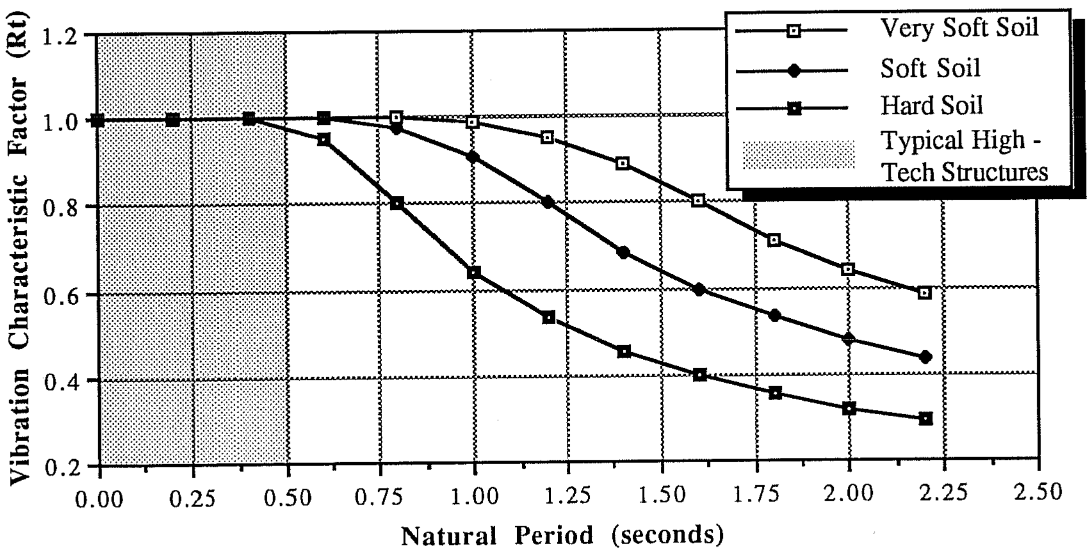


Figure 3.2 Vibration Characteristic Factor (Rt)

In the case of a typical high-tech industrial structure the period of the structure is less than 0.5 seconds which results in a Rt value of 1.0, independent of the soil type.

iii) The vertical distribution factor (Ai) is a coefficient that determines how the seismic design forces are distributed vertically on the structure. The distribution of the design shear forces, unlike the UBC and NEHRP, is imbedded in the lateral force Equation (3-2). The distribution of these lateral design forces is calculated by the following expression

$$\mathbf{A}_i = 1 + \left(\frac{1}{\sqrt{\beta_i}} - \beta_i\right)\frac{2\mathbf{T}}{1+3\mathbf{T}} \qquad (3\text{-}3)$$

where, ßi is a non dimensional weight coefficient given by

$$\beta_i = \frac{\sum_{i=i}^{n} \mathbf{W}_i}{\sum_{i=1}^{n} \mathbf{W}_i} \qquad (3\text{-}3a)$$

Where, **Wi** = weight of i-th storey, including other applicable loads,
n = number of storeys.

This vertical distribution factor Ai is different for every floor and increases in the upper levels of a building (Figure 3.3).

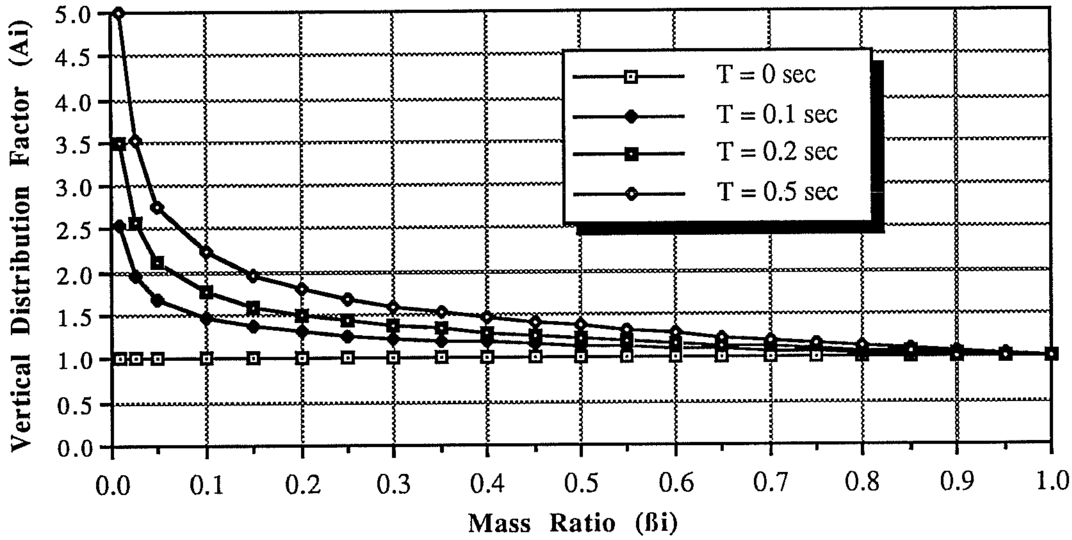


Figure 3.3 Vertical Distribution Factor versus Mass Ratio

The taller, longer period structures also receive higher values than the typical high-tech industrial structure. For the base level $ß_1$ (Equation 3-3a) is unity and A_1 from Figure 3.3 also equals one.

iv) The <u>standard shear coefficient (Co)</u> has been given a value of 0.2 for the first phase seismic design. This seismic coefficient originated after the heavy damage from the 1923 Kanto earthquake in Tokyo. This coefficient is meant to protect buildings in case of earthquakes with an intensity V on the Japanese Meteorological Agency (JMA) with an approximate effective peak acceleration of 100 gal (0.1g). Buildings are supposed to respond to earthquakes of this level without loss of function [15].

3.3.3.2 *Typical west coast high-tech industrial structure*

For the typical west coast, high-tech industrial structure Equation (3-2) can be simplified. The vertical distribution factor (Ai) is always one for the first floor and the vibration characteristic factor (Rt) equals unity for a typical high-tech industrial structure. The seismic zone factor (Z) is given a value of one in the most seismically active area of Japan, the equivalent of the U.S west coast. For this reason the base shear coefficient C_1 from Equation 3-2a would equal 0.2, and the design base shear (Equation 3-2) would simplify to the following expression:

$$\mathbf{Q_1 = 0.2\ W} \qquad (3\text{-}4)$$

Equation (3-4) is observed to be independent of the ductility available in the structural system. All low-rise structures, irrespective of function or structural system, must satisfy this same minimum strength requirement according to BSL.

3.4 PROVISIONS FOR THE ULTIMATE LIMIT STATE

As outlined in Table 3.1 both the U.S. and the Japanese building codes attempt to satisfy the ultimate limit state by prescribing minimum strength and ductility levels. In keeping with the U.S. design philosophy of life safety the primary structural requirement is strength. A minimum design lateral force is prescribed by the UBC and NEHRP provisions to prevent structural collapse and loss of life in a major earthquake. In the BSL approach the service limit state design is checked to ensure that the built in over-strength and ductility is adequate to meet the ultimate limit state performance criteria. In this section the seismic design provisions relevant to achieving the ultimate limit state of the UBC-91, NEHRP-88 and the BSL are presented.

3.4.1 Uniform Building Code 1991

The primary requirements specified by the UBC-91 to meet life safety objectives (ultimate limit state) is strength and ductility. To determine the minimum level of lateral force to be resisted by the structure, UBC-91 allows for either a static or dynamic lateral force procedure to be used. The typical high-tech structure (as described in Section 2.2) normally meets the criteria (UBC-91, §2333 (h) 2) for analysis by the static lateral force procedure outlined below. The base shear force prescribed by the UBC-91 static lateral force procedure is given by the following expression:

$$V = \frac{Z\,I\,C}{R_W}\,W \qquad (3\text{-}5)$$

Where, Z = seismic zone factor (0.075 to 0.4)

I = importance factor dependant on occupancy (1.00 to 1.25)

R_W = force reduction factor, account for system ductility etc. (4 to 12)

W = total dead weight of the structure, and any additional applicable loads

$$C = \frac{1.25\ S}{T^{2/3}} < 2.75 \qquad (3\text{-}5a)$$

Where, S = soil site coefficient (1.0 to 2.0)

T = period of structure

The seismic zone (Z) coefficient is meant to represent the approximate maximum effective peak ground acceleration that is likely to be experienced at the building site with a 475 year return period. The value of the seismic zone factor varies from 0.0 to 0.4 depending on the site location. The ductility factor (Rw) is meant to be representative of (amongst others) the available ductility of the structural system (Table 3.2). Those structures able to sustain more cycles of inelastic activity and large deformations are typically given a higher Rw value. This factor reduces the minimum required lateral strength of the structure so that the larger the Rw (ductility) factor the smaller the required strength. This reduction in lateral design load makes the structure more economical to construct while stability and life safety are maintained through additional code requirements.

The importance factor (I) classifies a structure according to its function, occupation and public reliance in case of emergency. Structures are classified as standard or special occupancy unless they house significant quantities of toxic or explosive substances. In this later case a hazardous facility classification would be placed on the structure. Only facilities that must remain operating, such as hospitals, emergency shelters or facilities required for emergency response, are classified as essential facilities [13]. It is apparent that this classification system is based on concern for life safety, and not serviceability. The value of the importance factor are shown in Table 3.3.

Table.3.2 UBC and NEHRP Ductility Reduction Factors

BASIC STRUCTURAL SYSTEM	LATERAL LOAD RESISTING SYSTEM	NEHRP R	UBC Rw
A. Bearing Wall System	1. Light-framed walls with shear panels		
	a. Plywood walls, less than 3 stories	6.5	8
	2. Shear walls		
	a. Concrete	4.5	6
	b. Masonry	3.5	6
B. Building Frame System	1. Steel eccentrically braced frame (EBF)	8	10
	2. Light-framed walls with shear panels		
	a. Plywood walls, less than 3 stories	7	9
	3. Shear walls		
	a. Concrete	5.5	8
	b. Masonry	4.5	8
	4. Concentrically braced frames	5	8
C. Moment-resisting Frame System	1. Special moment-resisting frame (SMRF)		
	a. Steel	8	12
	b. Concrete	8	12
	2. Concrete intermediate moment-resisting	4	8
	3. Ordinary moment-resisting frame (OMRF)		
	a. Steel	4.5	6
	b. Concrete	2	5
D. Dual Systems	1. Shear walls		
	a. Concrete with SMRF	8	12
	b. Concrete with steel OMRF	6	6
	c. Masonry with SMRF	6.5	8
	d. Masonry with steel OMRF	5	6
	2. Steel EBF		
	a. With steel SMRF	8	12
	3. Concentrically braced frames		
	a. Steel with steel SMRF	6	10
	b. Steel with OMRF	5	6

Table 3.3 UBC-91 Occupancy Requirements [13]

Occupancy Category	Importance Factor (I) Earthquake
I. Essential facilities	1.25
II. Hazardous facilities	1.25
III. Special occupancy structures	1.00
IV. Standard occupancy structures	1.00

The influence of soil-structure interaction is represented in the base shear coefficient (C) value from Equation (3-5a). This value is typically dependent on both the type of soil (S) and the period of the structure (T). As shown in Figure 3.4 an upper bound of 2.75 has been placed on the value of C in the lower period ranges.

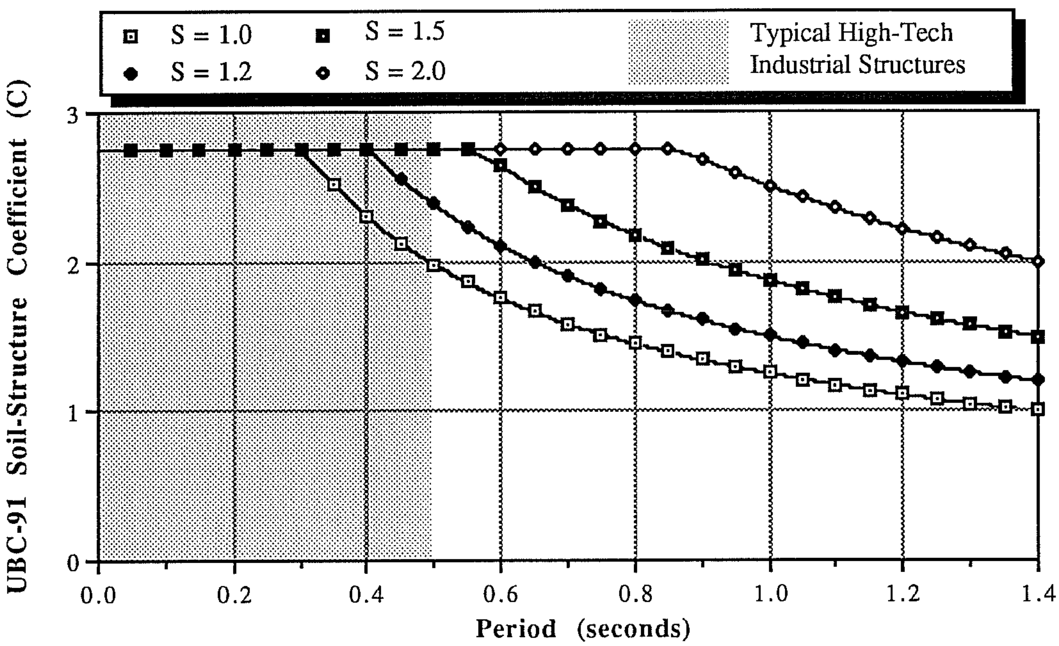


Figure 3.4 UBC 1991 Soil-Structure (C) Coefficient

3.4.1.1 *Typical west coast high-tech industrial structure*

In the typical one to three level high-tech industrial structure the period of the structure is likely to be less than 0.5 seconds. As indicated in Equation (3-5a) and Figure 3.4 a value of 2.75 typically controls in this period range (assuming the structure is not located on rock). The design base shear then simplifies to become a function of the seismic zone (Z), importance factor (I), weight (W) and a reduction of forces for the ductility (Rw) of the system (Equation 3-6).

$$V = \frac{2.75\ Z\ I}{Rw} W \qquad (3\text{-}6)$$

The typical high-tech industrial structure would normally fall into the standard or special occupancy category which implies an importance factor (I) of 1.0. On the west coast of the U.S. the seismic zone factor is given a value of 0.4. The design base shear Equation (3-6) further simplifies to become a function only of weight (W) and the available ductility (Rw) of the structural system.

$$V = \frac{1.10}{Rw} W \qquad (3\text{-}7)$$

As the Rw values range from four to twelve (Table 3.2) the design base shear can lie anywhere between ten and twenty seven percent of the structures weight, depending on the type of structure. This design base shear is then distributed over the height of the structure.

3.4.2 National Earthquake Hazard Reduction Program (NEHRP-88)

Similar to UBC-91, in NEHRP-88 [14] the primary design objective is to ensure life safety in a major earthquake (ultimate limit state). The general philosophy subscribed to by NEHRP-88 recommendations, similar to that of the UBC, is expressed in this quote [14]:

> The 'design earthquake' ground motion levels specified herein may result in both structural and nonstructural damage, but such damage is expected to be repairable. For ground motions larger than the design levels, the intent of these provisions is that there be a low likelihood of building collapse.

The primary requirements specified by NEHRP-88 to meet the life safety criteria are for strength (lateral forces) and ductility. In the NEHRP-88 Recommended Provisions for the Development of Seismic Regulations for New Buildings, design lateral forces are determined in a manner similar to that of the UBC-91.

According to NEHRP-88 structures are initially placed in a Seismic Performance Category according to the expected level of seismicity (location), and a Seismic Hazard Exposure Group based on building function. The expected level of seismicity is defined in terms of effective peak acceleration (Aa) and effective peak velocity related acceleration (Av) for a seismic event with a 475 year return period. The values representative of expected levels of seismicity at a site are taken from U.S.G.S.-NEHRP maps (values range between 0.05 and 0.4) or can be calculated from an alternative method such as that defined in the NEHRP-88 Appendix to chapter one.

The seismic hazard exposure group, similar to the UBC importance factor, classifies a structure according to its function, occupancy and public reliance in case of emergency. Buildings are placed in seismic hazard group II if they constitute a substantial public hazard because of occupancy or use. Seismic Exposure Group III is assigned to structures requiring the highest level of protection, such as, hospitals and emergency facilities. All other structures are designated seismic hazard group I. The structure is then placed in a seismic performance category according to Table 3.4.

Table 3.4 NEHRP-88 Seismic Performance Categories [14]

	Seismic Hazard Exposure Group		
Value of Av	I	II	III
$0.20 \leq Av$	D	D	E
$0.15 \leq Av < 0.20$	C	D	D
$0.10 \leq Av < 0.15$	C	D	D
$0.05 \leq Av < 0.10$	C	D	D
$Av < 0.05$	A	A	A

The seismic performance category is representative of the level of protection provided for the structure, and is defined by NEHRP-88 as, quote:

> a measure of the degree of protection provided for the public and building occupants against the potential hazards resulting from the effects of earthquake motions on buildings

Buildings in Seismic Performance Category B through D can all be analyzed with the equivalent lateral force procedure, provided they do not have vertical stiffness, mass or geometrical irregularities as defined in NEHRP-88. As in the UBC-

91, the typical high-tech industrial structure satisfies the NEHRP-88 criteria for design by the static lateral force procedure. Using this procedure the seismic base shear prescribed by NEHRP-88 is calculated from the following expression

$$V = Cs\ W \qquad (3\text{-}8)$$

Where, **V** = seismic base shear
W = the total structure dead load and applicable live loads

<u>If the fundamental period is calculated</u>

$$Cs = \frac{1.2\ Av\ S}{R\ T^{2/3}} < Cs \text{ from Equation (3-8b,c)} \qquad (3\text{-}8a)$$

Where, **Av** = Effective Peak Velocity-Related Acceleration coefficient (0.05-0.4)
S = coefficient for soil profile site characteristics (1.0 to 2.0)
R = response modification factor to account for system ductility (2 to 8)
T = the fundamental period of the structure.

<u>If the fundamental period is not calculated</u>

$$Cs = \frac{2.5\ Aa}{R} \qquad (3\text{-}8b)$$

but, if the soil type is S3 or S4 or Aa ≥ 0.3 then,

$$Cs = \frac{2.0\ Aa}{R} \qquad (3\text{-}8c)$$

Where, **Aa** = coefficient representing effective peak acceleration (0.05 to 0.40)
R = response modification factor to account for system ductility (2 to 8)

The response modification factor (R), similar to the UBC Rw factor, is representative of the available ductility of the structural system (Table 3.2). Soil structure interaction is considered in Equations (3-8a) but the upper bound of Cs is set by Equations (3.8b,c) which are independent of the soil type or the structures period.

3.4.2.1 *Typical west coast high-tech industrial structure*

The high-tech industrial building will typically fall into seismic hazard exposure group I or II. U.S.G.S. maps indicate that on the west coast of the U.S. the expected effective peak velocity-related acceleration is greater than 0.2. As shown in Table 3.4 the typical west coast, high-tech industrial building will be classified in seismic performance category D, and satisfy NEHRP-88 criteria for analysis by the static lateral force procedure.

The base shear coefficient (Cs) from Equation (3-8a) for low-rise structures on the U.S. west coast will typically be controlled by Equation (3-8c). The base shear coefficient is thus observed to be independent of the site soil conditions. In this location an effective peak acceleration (Aa) of 0.4 is obtained from U.S.G.S.-NEHRP maps. The NEHRP-88 base shear Equation (3-8) can simplify to become dependent only on the ductility factor (R) and mass (W).

$$V = \frac{0.8}{R} W \qquad (3\text{-}9)$$

The NEHRP-88 system ductility factor (R) value ranges between two and eight (Table 3.2). Based on this ductility factor (R) the design base shear can vary between ten and forty percent of the structures weight. Once again this base shear is distributed over the height of the structure for analysis purposes.

3.4.3 Japanese Building Standard Law Earthquake Provisions

As outlined earlier the BSL is unique in that it prescribes a two phase design approach. The previously discussed first phase of design prescribes a minimum design strength to satisfy the service limit state objectives. The second phase of design, discussed below, is intended to ensure that the ultimate capacity of the structure is sufficient to resist severe ground shaking [15]. The second phase is thus aimed at ensuring life safety in a once in a life time earthquake.

3.4.3.1 *Second Phase Design*

The second phase design is provided to ensure that the assumed built in over-strength and available ductility of a structure is sufficient to prevent major damage in the case of severe earthquake motion. The second phase design requires an initial storey drift check, followed by one of two different procedures to check the ultimate capacity of the structure. Additional strength and ductility requirements may be specified in this second phase of design if required.

The first provision in this second phase is to ensure that the storey drift ratios resulting from code prescribed forces are maintained to some acceptable level (Box 6, Figure 3.1). This provision is truly aimed at achieving the service limit state objectives and has therefore been discussed in the section of provisions for the service limit state (Section 3.3.2).

The next step in the second phase design is to decide which of two approaches is to be taken (Figure 3.1). A typical high-tech industrial structure, under 31meters (101 feet) in height, meets the criteria for the first approach (Box 7). In the first approach the structure is checked to determine if it is well-balanced with a regular lay-out. This is done by performing a rigidity test which identifies potential soft "story" problems, and an eccentricity test, that identifies the potential for problems that may result from eccentricity between the mass center and the center of rigidity. Should the structure meet these criteria, then as long as it meets certain additional Ministry of Construction requirements (Box 8, Figure 3.1), it can be passed for approval by the local government body. These Ministry requirements pertain to issues such as assuring adequate strength and ductility in braces and frames of steel buildings, and the same with shear walls and frames in concrete structures [15].

Should the structure under design not meet any one of the above criteria then the second approach, a check for the ultimate lateral load capacity of the structure, is required. This provision ensures that the ultimate lateral load carrying capacity in each storey exceeds the shear force likely to be experienced at that level, as represented by this expression.

$$\mathbf{Q_{ui} \geq D_s\, F_{es}\, Q_{ni}} \qquad (3\text{-}10)$$

Where, Q_{ui} = the ultimate lateral load carrying capacity of storey i calculated by any method such as incremental nonlinear analysis or limit analysis.

D_S = a factor that takes into account the energy dissipation capacity, or ductility, of the structural system. This factor reduces the elastic response storey shear according to available ductility and varies from 0.25 to 0.55.

F_{es} = a factor that takes into account the irregularity of the structure in terms of the rigidity and eccentricity factors. This factor varies from 1 to 1.5 as the irregularity of the structure increases.

Q_{ni} = the seismic storey shear as determined in the first phase design (Equation 3-2) except that the standard shear coefficient (Co) has a value of 1.0 instead of 0.2.

If the structure does not satisfy the requirement of Equation (3-10) then it must be redesigned prior to being sent for approval at the local government level.

3.5 MINIMUM DESIGN BASE SHEAR COMPARISON

All the codes prescribe a minimum level of lateral force that a structure must be designed to resist. While all the codes allow for dynamic analysis procedures in the structural analysis, the static lateral force procedure was found to be applicable to the typical high-tech industrial building. In order to compare the three codes the prescribed base shear of a typical high-tech industrial building, located on the U.S. west coast, will be discussed.

3.5.1 Design Base Shear

The major difference between the design base shear specified by the U.S. and the Japanese codes, is that the U.S. one-phase seismic design approach is based on meeting life safety criteria, whereas the BSL prescribed base shear is based on achieving service limit state criteria. The U.S. codes consider the ductility available in the structure when determining the minimum required design base shear force, while the BSL prescribes a minimum lateral strength requirement independent of the structural system used.

The minimum base shear force requirement for a typical west coast, high-tech industrial structure, as prescribed by the three codes, were simplified to the following three expressions

UBC: $V = \frac{1.10}{Rw} W$ NEHRP: $V = \frac{0.8}{R} W$ BSL: $V = 0.2\ W$

While the UBC equation appears as the more conservative of the two U.S. codes, the UBC ductility factor (Rw) is consistently larger than that of the NEHRP ductility factor (R) (Table 3.2). Also, while the UBC-91 base shear is intended for use with an allowable stress design, NEHRP-88 is intended for use with limit state design. The BSL prescribed base shear was shown to be 20% of the structures mass, independent of the structural system ductility. Figure 3.5 illustrates the design base shear, normalized for mass, for typical high-tech industrial structures. This figure shows that the two U.S. codes prescribe comparable base shear forces while the

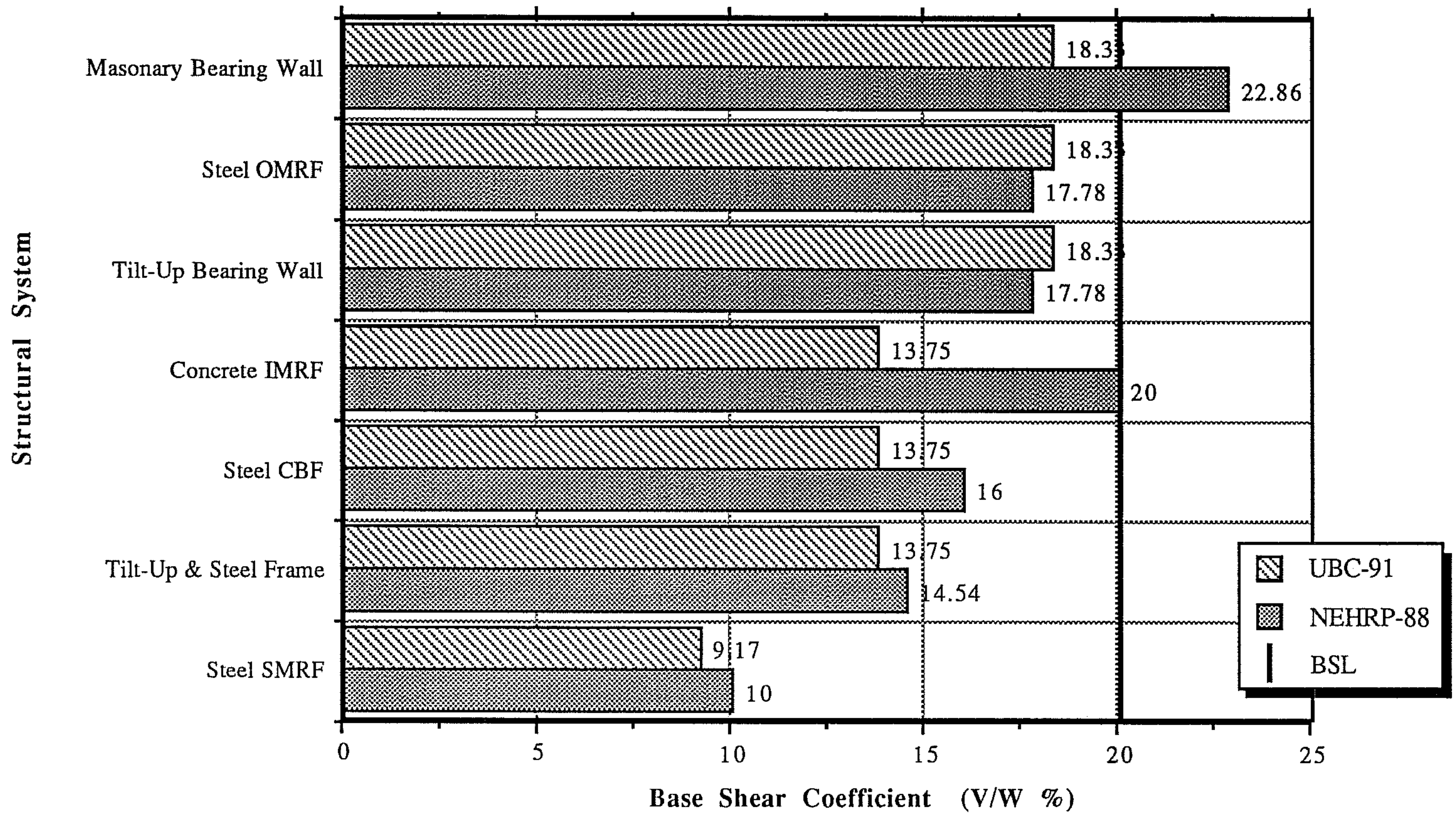


Figure 3.5 BSL, NEHRP-88 and UBC-91 Minimum Design Base Shear

Japanese base shear force is independent of the structural system and more conservative in most instances. In those structural systems with large available ductility (steel special moment-resisting frame, steel braced frame, etc.) the UBC-91 and NEHRP-88 prescribed base shear is substantially less than that of the BSL. In the less ductile structures (masonry bearing wall, tilt-up bearing wall, etc.) the Japanese and U.S. codes show better agreement in the level of base shear prescribed.

3.6 CONCLUSIONS

In this chapter the seismic provisions of the UBC-91, NEHRP-88 and BSL codes were summarized. Code provisions were divided into two broad categories, those provisions aimed at achieving service limit state objectives, and those aimed at achieving ultimate limit state objectives. All the codes provide comparable provisions in the area of design forces for nonstructural elements and limits to lateral drift. The most striking difference between the U.S. and the Japanese BSL codes is in the area of minimum lateral design force determination.

Conceptually it is apparent from the discussion in this chapter that the lateral forces prescribed by the UBC-91 and NEHRP-88 are based on the inelastic design spectra for the ultimate limit state. This procedure specifies lateral design forces that are reduced from an elastic level to account for a structures ductility (R and Rw factor). This design approach implies that the strength of a structure is a function of the structural system (type). This approach does not represent the elastic design spectra for the service limit state (a moderate earthquake). The UBC-91 and NEHRP-88 provisions may therefore adequately meet the ultimate limit state objectives, but not necessarily those of the service limit state.

In the BSL the lateral design forces are determined independent of the structural system (type). The design lateral forces are therefore a function of the strength of earthquake that must be resisted in order to meet the service limit state objectives. In the BSL the service level earthquake is characterized as one having a peak ground acceleration of approximately 0.1 g. All structures, irrespective of structural system should remain operational in an earthquake of this approximate peak ground acceleration.

The current U.S. codes need to be examined to see if the minimum design lateral forces that they prescribe for the ultimate limit state are sufficient to prevent structural damage in less significant ground motion. In some instances a two-phase design approach, where the minimum strength of a structure is based on serviceability criteria, may be desirable. This is particularly important in those highly ductile systems which may have a relatively low lateral strength requirement according to the UBC and NEHRP provisions, when compared to the BSL.

If structural damage occurs repeatedly during the life of a high-tech industrial structure the associated economic costs of restoring operations could financially ruin the building owner. As shown in the previous chapter the economic loss associated with structural damage (and associated disruption of operations) can accumulate rapidly with length of downtime. If these costs are to be minimized in the high-tech industry then structural damage should be prevented in all but the most extreme earthquake. High-tech corporations should be made aware of the potential risk for structural damage that exists in their buildings. If necessary, preventative action such as retrofitting could then be considered as an alternative to performing structural repair after a moderate earthquake.

CHAPTER IV
SERVICE LIMIT STATE DESIGN FOR HIGH-TECH INDUSTRIAL STRUCTURES IN THE U.S.

4.1 INTRODUCTION

In the previous chapter it was shown that the seismic provisions of the UBC-91 and NEHRP-88 can be divided into two categories. The first category encompasses all those provisions aimed at meeting service limit state (SLS) objectives. The second category includes all those provisions aimed at meeting the ultimate limit state (ULS) objectives such as life safety. Based on this design philosophy, structural damage should be prevented in small and moderate levels of earthquake ground motion. During Loma Prieta, however, structural damage was observed in several high-tech industrial structures, even though the level of ground motion they experienced could not be classified as major. For this reason the design forces prescribed by the UBC-91 and NEHRP-88 for life safety need to be examined to determine if they adequately prevent structural damage in high-tech industrial structures in moderate earthquakes.

In this chapter different methods of gaging the "size" of earthquakes and earthquake ground motion are discussed. An analytical study is conducted in which the base shear force for a high-tech industrial structure is related to different levels of ground motion. Recommendations are made on the minimum level of static lateral force that must be resisted by a structure to meet the SLS objectives. Based on these recommendations the adequacy of current seismic codes to meet the SLS objectives is discussed.

4.2 GROUND MOTION QUANTIFICATION

To determine the potential of a particular earthquake to produce damage in a building is a complex process. Of particular interest to the engineer is both the effect that the earthquake will have on the response of a structure and the damage that may result. The damage potential of an earthquake is a function of many factors that include the magnitude or severity of the earthquake, the distance of the epicenter from the structure, and local geologic conditions. These factors all influence the characteristics of the ground motion that may occur at a particular site. The ground motion characteristics must be used to establish the criteria for the design of earthquake-resistant structures.

4.2.1 Magnitude and Intensity of Earthquakes

Two techniques have established themselves as predominant measures of earthquake 'size' or 'strength'. These techniques measure the magnitude and intensity of the earthquake and ground motion respectively. The magnitude and intensity scales can then be used to compare seismic events around in the world.

4.2.1.1 *Magnitude*

The amount of strain energy released at a source of an earthquake is indicated quantitatively as the magnitude. In the U.S. the Richter magnitude [18] is widely referenced as a measure of earthquake strength. The empirical observation is that earthquakes of magnitude 5.0 and above have the potential to cause structural damage [19].

4.2.1.2 *Intensity*

While magnitude is a measure of earthquake strength at the epicenter, the level of ground motion at a site is commonly referred to as the earthquake intensity. The intensity of an earthquake normally reduces with distance from the source, and is typically based on a subjective measure of damage to natural and man-made objects at a site [18]. In the U.S. the standard measure of intensity is the Modified Mercalli (MM) scale.

While both magnitude and intensity are useful in measuring and comparing earthquake strength, they are not meaningful criteria for seismic design. Of more importance to the engineer are the characteristics of ground motion that influence the response of a structure at a particular site.

4.2.2 Ground Motion Characteristics

Earthquake records can be characterized by several different ground motion parameters. The most important of these parameters are those that influence the response of the structure at the site. The parameters that are traditionally used in the characterization of a specific time-history are:

- Maximum Ground Motion - Peak Ground Displacement (PGD)
Peak Ground Velocity (PGV)
Peak Ground Acceleration (PGA)
- Frequency Content
- Duration of Ground Motion

4.2.2.1 *Maximum ground motion*

The peak ground acceleration, velocity and displacement are often used to scale design spectra and acceleration time-histories. These peak ground motion parameters primarily influence the vibration amplitude of the structural response. The parameters which influence maximum ground motion include: site geology, magnitude of earthquake and distance to epicenter [18]. While peak ground acceleration is the ground parameter most often associated with potential for structural damage, peak ground velocity and displacement are often related to the potential for nonstructural damage.

4.2.2.2 *Frequency content of ground motion*

The energy associated with specific frequencies of vibration in ground motion have a substantial influence on the response of a structure. The greater the ground motion energy in the range of the fundamental frequency of the structure, the larger the response [20]. Many factors influence the frequency content at a specific site, amongst these are: site soil conditions, distance to epicenter, magnitude of earthquake and cause of earthquake [18].

4.2.2.3 *Duration of ground motion*

The duration of ground motion can have a significant effect on the performance of a structure. Ground motion with a moderate peak acceleration but long duration could potentially cause more damage than ground motion with a high peak acceleration but low duration. The duration of an earthquake is particularly important when repeated loading could cause progressive deterioration of critical segments of a structure. Several authors have recommended different methods to determine the duration of significant ground motion of an earthquake record. Some of these methods include: bracketed duration between specified acceleration levels [21], or ground motion between 5% and 95% contribution to total acceleration intensity [22].

The damage potential of ground motion cannot be adequately quantified from any of the above parameters alone, instead, a measure that considers the interaction of these factors is desirable. A more precise measure of the influence of specific ground motion on the response of a structure can be obtained from a response spectrum.

4.3 RESPONSE SPECTRUM

A response spectrum describes the maximum response of an elastic, damped single-degree-of-freedom (SDOF) system, at various natural periods, when subjected to an earthquake record. The response spectrum can be used both as a measure of the ground motion intensity and as a design tool.

4.3.1 Response Spectrum as a Measure of Ground Motion Intensity

First developed by Housner [23] the spectrum intensity (S.I.) is representative of the effect a particular earthquake record will have on the response of an elastic structure. As the response spectrum covers all periods of vibration the integral of the spectrum over an appropriate period range is probably the most appropriate measure of ground motion intensity. The spectrum intensity is therefore defined as the integral of the pseudo-velocity, displacement or acceleration response spectrum, between two specified periods of interest, as illustrated below.

$$\mathbf{S.I.}(\xi)=\int_{T1}^{T2} \mathbf{PSv}(\xi,\omega)\ \mathbf{dT} \tag{4-1}$$

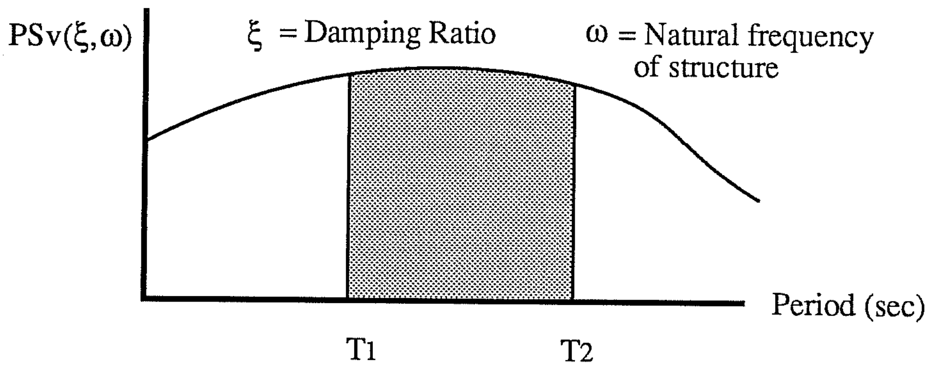


Figure 4.1 Pseudo-Velocity Response Spectrum

4.3.2 Response Spectrum as a Design Tool

The design response spectrum is a commonly used in the design of single and multi-degree-of-freedom (MDOF) structures. Of particular interest to the engineer is the lateral forces induced on a structure by a specific design earthquake record. According to dynamic theory [20] the maximum base shear due to the *n* th mode of a MDOF structure can be expressed as:

$$\mathbf{V}_{n\max} = \frac{\mathbf{W}_n^* \mathbf{Sa}_n}{\mathbf{g}} \tag{4-2}$$

where,
- $\mathbf{n}$ = vibration mode
- $\mathbf{g}$ = gravitational acceleration
- $\mathbf{Sa}_n$ = response spectrum acceleration for n th mode, and
- $\mathbf{W}_n^*$ = effective mass of n th mode.

The sum of effective masses for all modes of a MDOF model must equal the total mass of the structure. The effective mass of the n th mode is defined as:

$$\mathbf{Wn}^* = \frac{\left(\sum_{i=1}^{H} \mathbf{W}_i \varnothing_{in}\right)^2}{\sum_{i=1}^{H} \mathbf{W}_i \varnothing_{in}^2} \tag{4-3}$$

where,
- $\mathbf{H}$ = number of modes considered in structure;
- $\mathbf{W}_i$ = mass of the i th floor; and
- $\varnothing_{in}$ = i th ordinate of the n th mode shape.

From Equation (4-2) the maximum base shear due to each mode can be calculated. The base shear due to the response of each individual mode are unlikely to occur simultaneously. For this reason the total maximum base shear (for all modes) is calculated by combining the response of each individual mode using statistical methods such as square root sum of squares (SRSS) or complete quadratic combination (CQC) [20]. As the contribution of each mode to the total base shear diminishes rapidly with the higher modes, the number of modes that are required to calculate the maximum total base shear to within some acceptable tolerance must be established. This number is traditionally determined by the number of modes whose sum of effective masses is greater than a predefined percentage of the total mass. In the UBC-91 this level is set at ninety percent (90%) of the total mass.

4.4 DESCRIPTION AND RESULTS OF ANALYTICAL STUDY

In this section an analytical study is conducted to determine the base shear force a high-tech industrial structure experiences when subjected to different levels of ground motion. To conduct this study the use of the SDOF response spectrum as a realistic measure of seismic forces induced on a MDOF structure is examined. A method of categorizing different levels of ground motion must also be established.

4.4.1 Categorizing Ground Motion

The first step in this study is to determine which of the previously discussed ground motion parameters (Section 4.2) is the best indicator of the level of forces a ground motion record is likely to produce in a high-tech industrial structure. Structural forces produced by an earthquake are largely as a result of the acceleration/inertia of the structures mass and contents. For this reason ground motion acceleration (particularly the peak ground acceleration ordinate) is traditionally used as an indicator of the expected level of forces likely to be produced by that record. Codes such as UBC-91 and NEHRP-88 have traditionally selected the effective peak acceleration (or velocity-related acceleration) to define the seismic risk at a site (Figure 4.2).

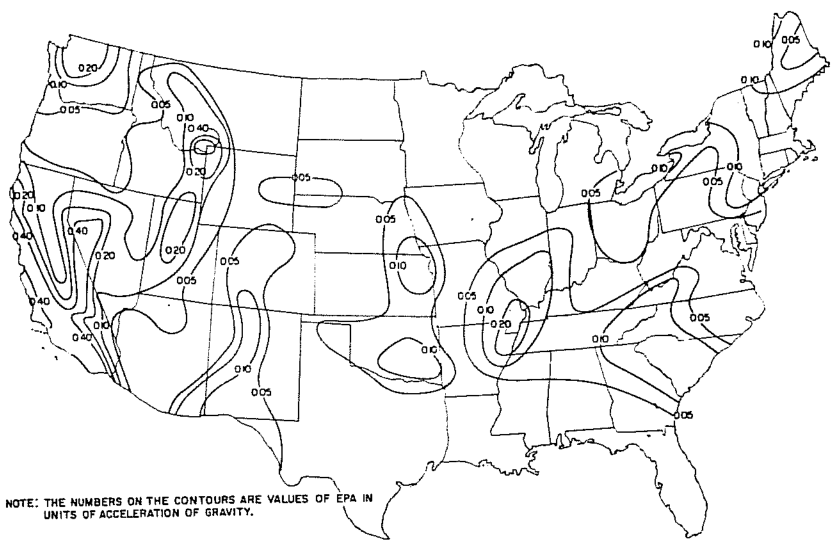


Figure 4.2 Seismic Zone Map of U.S.

Studies conducted by Chang [24] suggest that peak ground acceleration is an appropriate indicator of the forces a ground motion record will produce in a structure, but only for low period structures (<0.5 seconds). In the study conducted by Chang, the influence of several different ground motion parameters on the forces produced in various structure were examined. In his study forty five earthquake records were each normalized (scaled) by the same ground motion parameter (Figure 4.3, Step 1 & 2). For example, all the records were scaled to have the same peak ground acceleration (PGA). Acceleration response spectra were then calculated, one for each of the forty five normalized earthquake records (Figure 4.3, Step 3). A measure of the variation (coefficient of variation) between the acceleration ordinates at each period of the forty five individual response spectrum was described. As a result of this procedure a 'spectrum' of coefficient of variation (Figure 4.3, Step 6) was calculated. This process was repeated several times with the earthquake records normalized by a different ground motion parameter each time.

The reasoning behind Chang's study is as follows: If numerous different ground motion records are normalized by the same single parameter, and the acceleration response spectrum resulting from these records are all identical, then the magnitude of that normalizing parameter in a particular record is an exact indicator of the forces likely to be produced by the record. In other words, from the results of his research the parameter that results in the lowest coefficient of variation 'spectrum' is considered the best indicator of the force likely to be produced by a particular earthquake record.

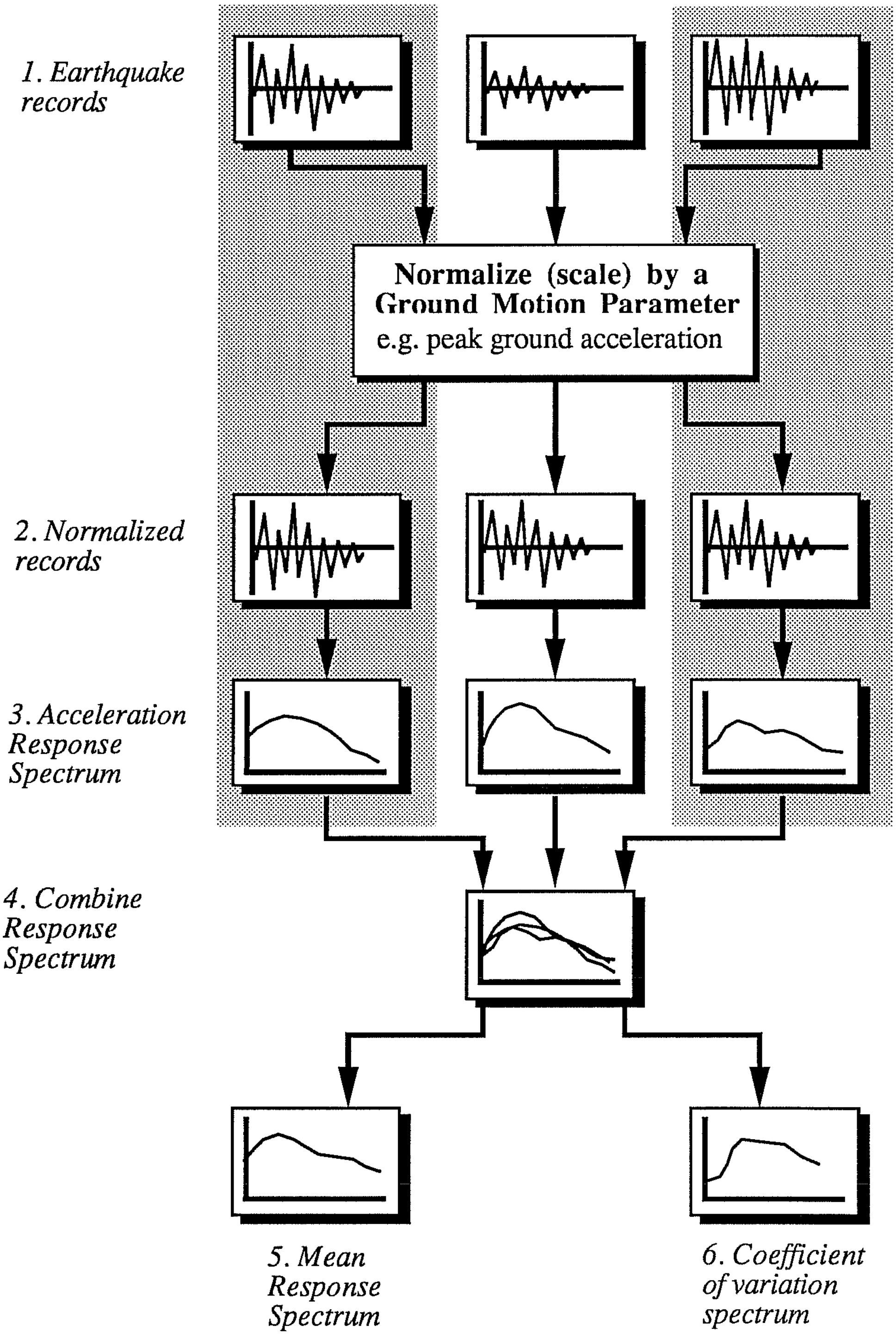


Figure 4.3 Sequence of Study Performed by Chang [24]

The normalizing parameters tested by Chang include:

- Maximum ground displacement (PGD)
- Maximum ground velocity (PGV)
- Maximum ground acceleration (PGA)
- Relative displacement spectrum intensity displacement (Sd SI)
- Relative velocity spectrum intensity displacement (Sv SI)
- Absolute acceleration spectrum intensity displacement (Sa SI)

The results of this study, shown in Figure 4.4, indicate that the influence of a particular parameter varies depending on the period of interest. Normalization by a particular parameter may produce the lowest coefficient of variation but only in a specific period range.

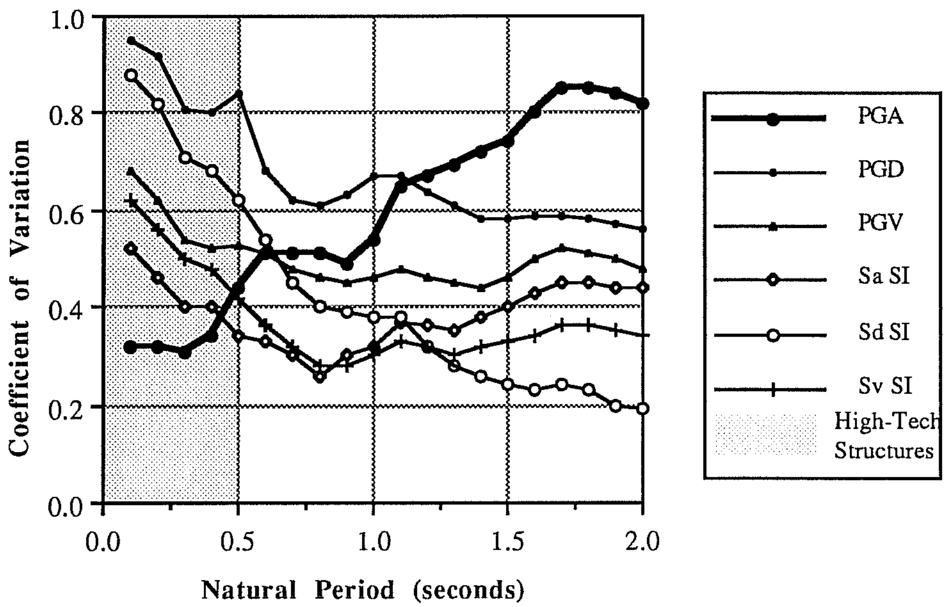


Figure 4.4 Coefficient of Variation Spectra from Acceleration Response Spectra Normalized for Various Ground Motion Parameters (5% Damping)

It is apparent from Figure 4.4 that in the period range below 0.5 seconds normalization by peak ground acceleration results in the lowest coefficient of variation.

For structures whose fundamental period lies in this period range it is concluded that peak ground acceleration of a particular ground motion record is the best indicator of the forces likely to be produced by that record.

4.4.2 Predicting Maximum Ground Acceleration at a Site

Having established PGA as a reasonable indicator of the forces likely to be produced by an earthquake record, the probability of observing a particular PGA at a specific site must be established. In research conducted by Algermissen, et.al [25] the annual risk of a certain effective peak acceleration (EPA) levels being exceeded in various regions of the U.S. was established. The results of this research are shown in Figure 4.5.

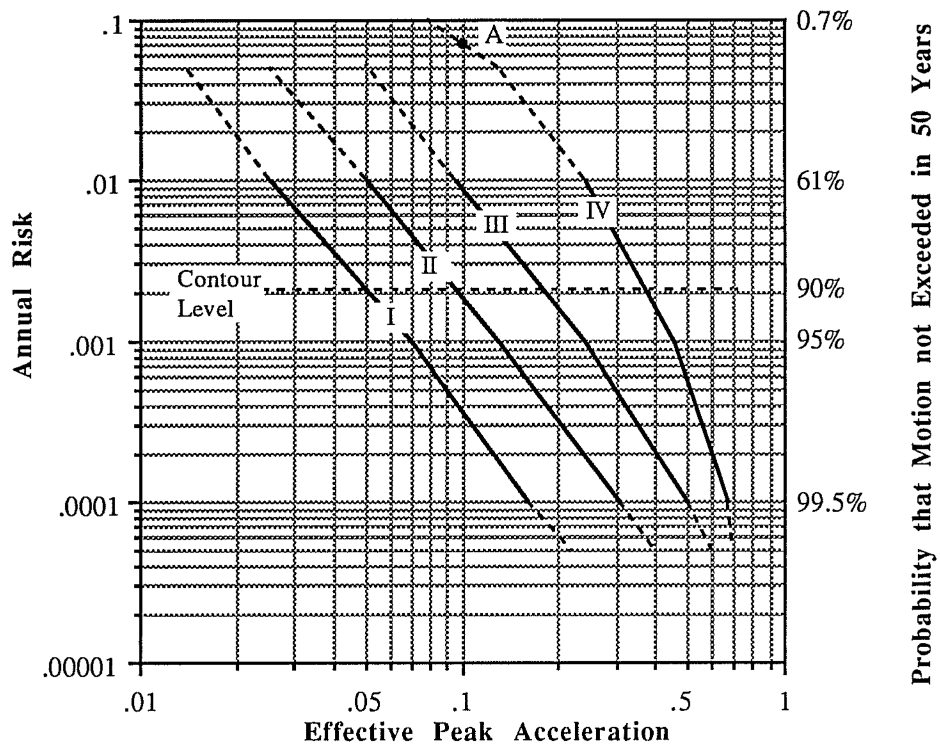


Figure 4.5 Annual Risk of Exceeding Various Effective Peak Accelerations [14]

Figure 4.5 shows the probability of experiencing a particular EPA in each seismic zone. The four lines in the figure represent the four different seismic zones of the UBC seismic zone map. To determine the annual risk of a specific EPA being exceeded in a particular seismic zone the following sequence is followed. Locate the EPA on the horizontal axis and follow this point vertically until intersecting with the line that represents the seismic zone of interest. Moving horizontally from the point of intersection, the left vertical axis indicates the annual risk of the selected EPA being exceeded in that particular seismic zone. For example, the annual risk of an EPA of 0.1g being exceeded in UBC seismic zone IV is about 0.08 (Point A). That is, once every 12.5 years an earthquake with an EPA of 0.1g can be expected at a site in seismic zone IV.

The results of this study conducted by Algermissen and Perkins were also used as the basis on which the contours for the UBC and NEHRP-88 Seismic Zone Map were drawn. The contours of the US seismic zone map (Figure 4.2) are based on a probability of 90% that the indicated effective peak acceleration (EPA), in a particular zone, will not be exceeded in 50 years. This probability translates to an annual risk of approximately 0.002 (≈500 year return period) as indicated by the dashed line in Figure 4.5.

4.4.3 Design Forces from Response Spectrum

Of particular interest to engineers is the maximum base shear likely to be induced by a particular earthquake. These maximum base shear values are easily calculated using a design response spectrum as discussed in Section 4.3.2. However, the acceleration response spectrum is typically calculated for an elastic SDOF system and must be scaled to account for the multiple modes of a MDOF structure. In this section the value by which the acceleration response spectrum must be scaled to adequately predict the maximum total base shear of all modes of a MDOF system is examined.

The contribution of each mode maximum base shear to the total maximum base shear of a structure is illustrated by example. Consider a low-rise structure for which the sum of effective masses (Equation 4-3) from the first two modes totals at least 90% of the total mass of the structure. Assume that the spectral acceleration (Sa) ordinate for both first and second modes is the same (this assumption is acceptable for low-rise structures whose periods of vibration fall in the range of the acceleration response spectrum that is relatively flat). As a specific example assume the effective mass in the first mode (W_1*) is 80% of the total mass (W), and the effective mass in the second mode (W_2*) is 10% of the total mass. Based on these assumptions and substituting into Equation (4-2) the base shear for the first two modes can be expressed as follows:

$$V_1 = \frac{0.8\ Sa\ W}{g} \qquad\qquad V_2 = \frac{0.1\ Sa\ W}{g} \tag{4-4}$$

As only two modes are considered in this example the total base shear calculated using the SRSS is defined as:

$$V_{max} = \sqrt{V_1^2 + V_2^2} \tag{4-5}$$

Substituting the values from Equation (4-4) into Equation (4-5) returns a maximum total base shear of:

$$V_{max} = \frac{0.806\ Sa\ W}{g} \qquad (4\text{-}6)$$

As is apparent from this example, the contribution of the first mode to the total base shear force is substantial (99%). Table 4.1 indicates the contribution of the first mode response to the maximum total base shear, as the effective masses of the first two modes vary.

Table 4.1 Contribution of First Mode to the Maximum Total Base Shear using SRSS

W_2^*/W (%)				
40	83 %	----	----	----
30	89 %	92 %	----	----
20	----	96 %	97 %	----
10	----	----	99 %	99 %
	60	**70**	**80**	**90**

W_1^*/W (%)

From the results shown in Table 4.1 it can be concluded that when the first mode effective mass is over eighty percent of the total mass, the maximum total base shear is almost entirely produced by this first mode response (97+%). For low-rise structures the effective mass in the first mode response is typically close to eighty percent of the total mass [26]. For this reason the first mode response alone will provide a relatively accurate representation of the maximum response of a low-rise MDOF structure.

For design purposes base shear is traditionally given as a function of actual structure mass (W) as shown below.

$$V_{max} = \frac{W\ Sa}{g} \tag{4-7}$$

However, as indicated in Equation (4-2) and from the above discussion, the base shear of a MDOF system is a function of the effective mass in the first mode, as shown below

$$V_{max} = \frac{W_1^*\ Sa}{g} \tag{4-8}$$

In low-rise structures the effective mass of the first mode can conservatively be taken as eighty percent of the total structure mass ($W_1^*=0.8W$). Equation (4-8) can then be expressed as a function of the total structure mass.

$$V_{max} = \frac{0.8\ W\ Sa}{g} \tag{4-9}$$

Equation (4-9) indicates that a SDOF acceleration response spectrum must be scaled by a factor of 0.8 if the base shear of a MDOF system is to be accurately represented as a function of the total structure mass (Figure 4.6).

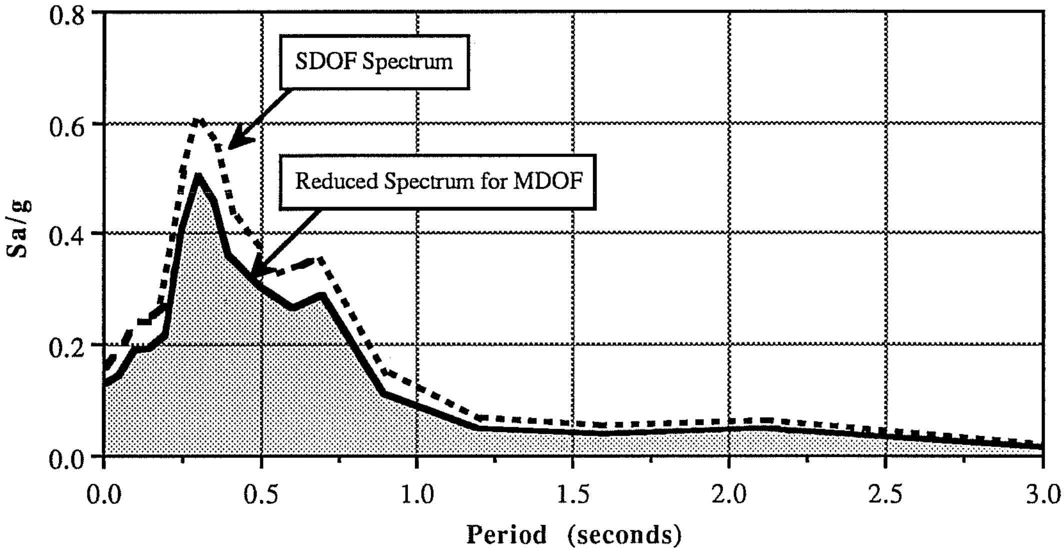


Figure 4.6 Schematic of Acceleration Response Spectrum Scaled for MDOF Design

4.4.4 Sequence of Analytical Study

As discussed in Section 4.4.1 the peak ground acceleration (PGA) is the best indicator of the the base shear force a particular ground motion record will produce in a high-tech industrial structure. For this reason twenty eight earthquake records are normalized to a specific level of peak ground acceleration (Figure 4.7, Step 1&2). Acceleration response spectra are calculated for each of the normalized earthquake records (Figure 4.7, Step 3) and a mean spectrum (plus one standard deviation) is calculated (Figure 4.7, Step 5). The mean spectrum is scaled by a factor of 0.8 to account for multi-mode effects as discussed in Section 4.4.3. An average spectrum value (average over all periods) is determined from the mean acceleration spectrum (Figure 4.7, Step 7). The procedure is repeated for the same earthquake records

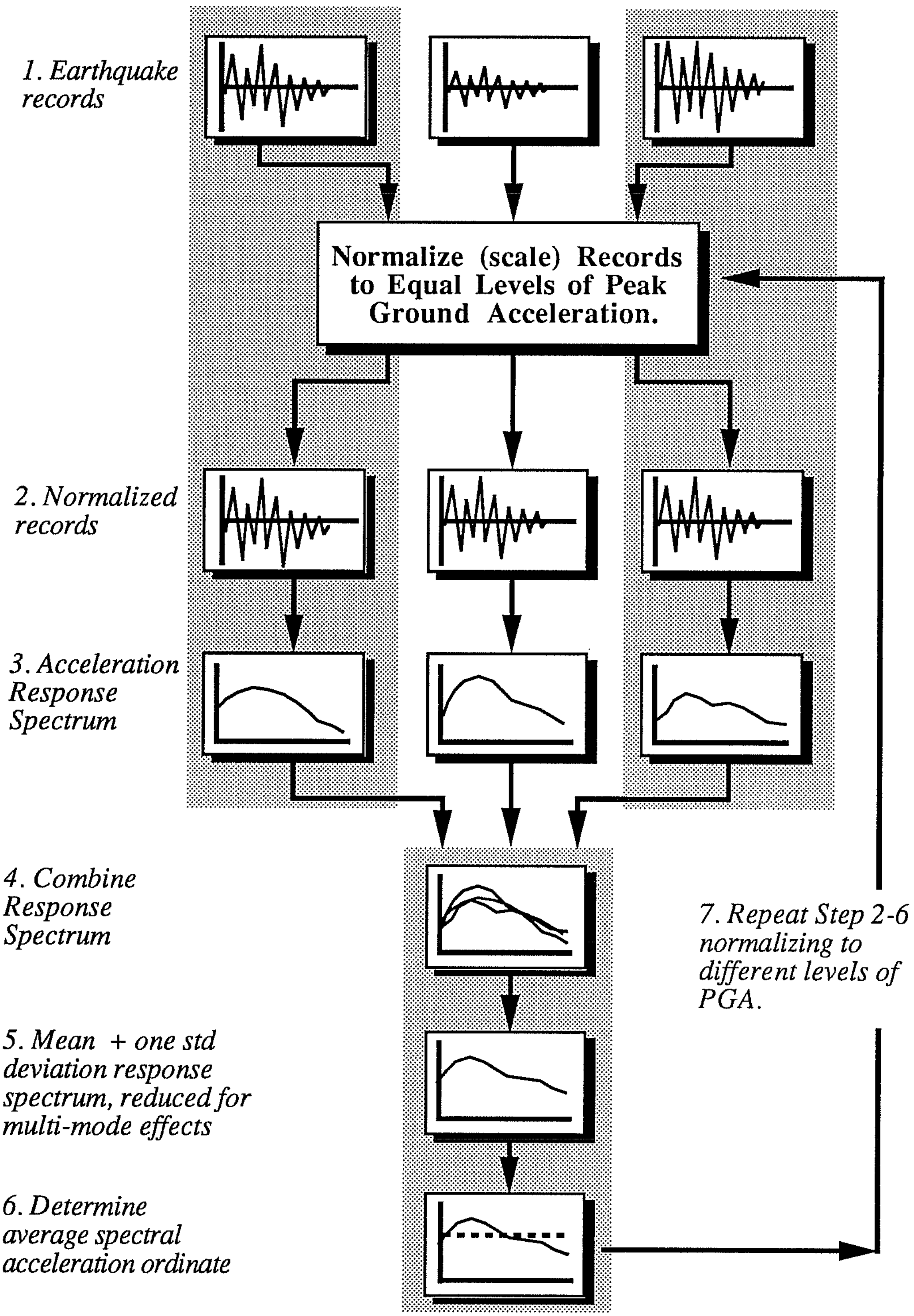


Figure 4.7 Sequence of Analytical Study

normalized to a different level of PGA. The result is a single value of base shear force for each level of PGA to which the earthquake records are scaled.

4.4.4.1 *Input earthquake records*

Twenty eight strong-motion earthquake time-histories were utilized in this study (Table 4.2). These records represent eleven earthquakes measured at nineteen different stations. The records were selected to represent various magnitude earthquakes, near and far field ground motion responses, as well as different site geologic conditions. While these records are predominantly West Coast earthquake records, the same study could be conducted for other earthquake records with similar results expected.

4.4.4.2 *Damping*

In calculating the response spectrum for a structure from a particular earthquake record the percent of critical damping provided by the structure must be estimated. In this study an elastic response was assumed and an appropriate damping range of three to five percent (3-5%) of critical was selected.

4.4.5 Results of Analytical Study

The result of this study is a relationship between the PGA of an earthquake record and the expected base shear force that record produces on a low-rise structure (Figure 4.8). It is apparent from this figure the base shear force on a low-rise structure increases linearly with the magnitude of the PGA of the earthquake record to

Table 4.2 Earthquake Records used in Analytical Study

Earthquake	Recording Station	Dir.	PGA
Imperial Valley, 1940	El Centro	N00E	0.35 g
Kern County, 1952	Pasdena-Caltech Athenaeum	S90W	0.05 g
	Santa Barbara-Court House	S48E	0.13 g
	Hollywood Storage-Basement	S00W	0.06 g
San Francisco, 1957	Golden Gate Park	S80E	0.11 g
Long Beach, 1933	Vernon CMD Building	S08W	0.13 g
Lower California, 1934	El Centro	S00W	0.15 g
		S90W	0.18 g
Helena, Montana, 1935	Carrol College	S90W	0.15 g
Western Washington, 1949	Seattle-Distr. Engs. Office	S02W	0.07 g
	Olympia-Hwy. Test Lab.	N04W	0.16 g
Puget Sound, 1965	Olympia-Hwy. Test Lab.	S04E	0.14 g
Parkfield, California, 1966	Cholame-Shandon Array #2	N65E	0.49 g
	Cholame-Shandon Array #8	N50E	0.24 g
	Cholame-Shandon Array #12	N50E	0.05 g
		N40W	0.06 g
San Fernando, 1971	E-First St.-Basement.	N36E	0.10 g
		N54W	0.13 g
	Hollywood Storage-Basement	S00W	0.11 g
		N90E	0.15 g
	Caltech Athenaeum	N90E	0.11 g
	JPL-Basement	S82E	0.21 g
	Palmdale Fire Station	S60E	0.11 g
		S30W	0.14 g
	Ventura Blvd.	N11E	0.23 g
Loma Prieta, 1989	VA Hospital	N212E	0.38 g
	Oakland	N55W	0.27 g
	Corralitos	N00E	0.63 g

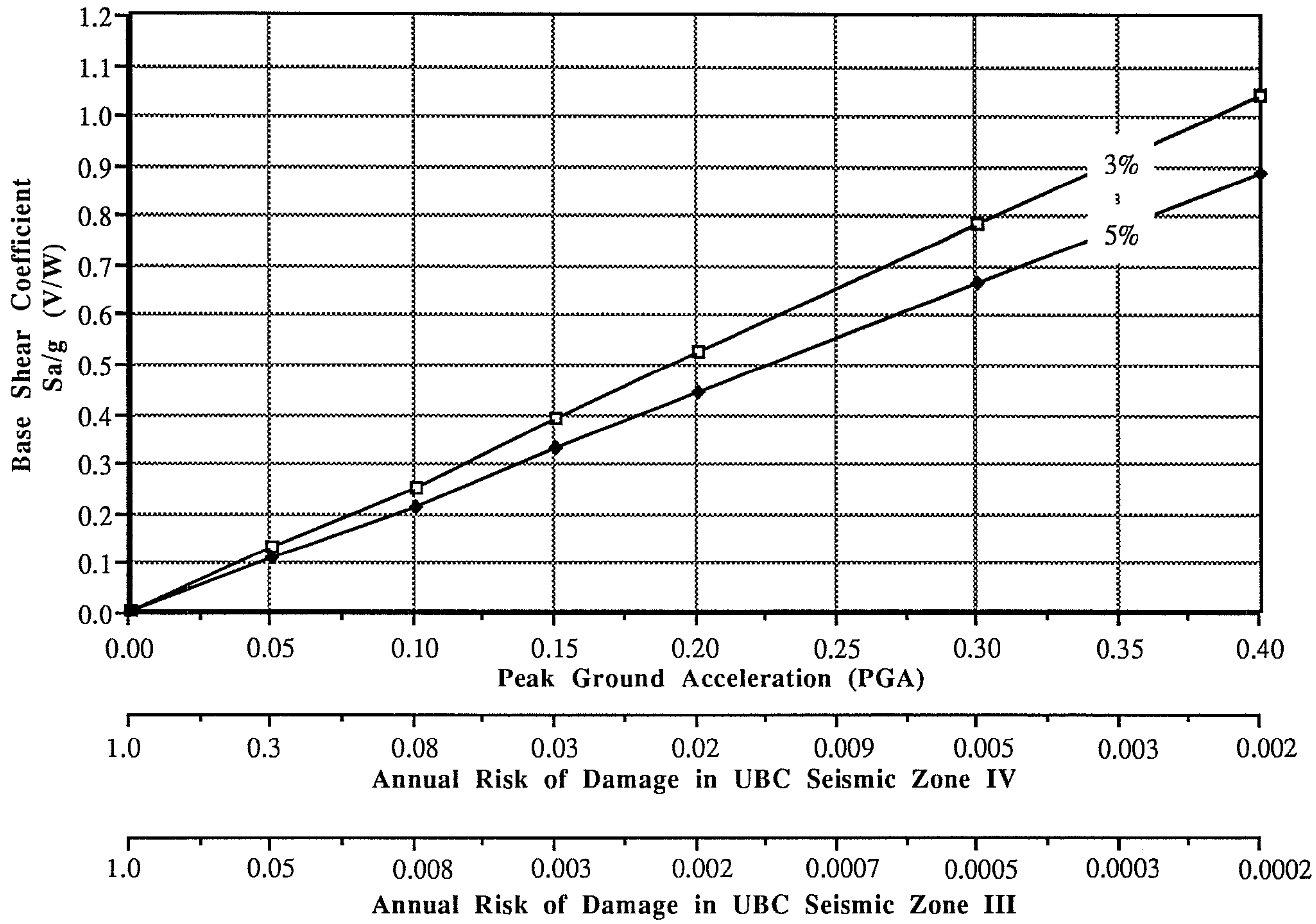


Figure 4.8 Lateral Force Due to Earthquakes with Various Levels of PGA

be resisted. The base shear force produced by a particular earthquake record is obviously greater with a lower damping value for the structure. The vertical axis in Figure 4.8 is representative of the base shear coefficient (shear force as a percent of structure mass) that is expected to be produced by an earthquake record with a PGA of the magnitude indicated on the top horizontal axis. The second and third horizontal axes indicate the annual risk that structural damage will result in the UBC-91 seismic zones III and IV respectively. The annual risk values were determined from the results of the Algermissen, et. al study [25] (Figure 4.5).

4.4.6 Code Prescribed Base Shear and Expected Forces

In the previous chapter the base shear force equations of the UBC-91 and BSL, were simplified for a typical west-coast high-tech industrial building to the following expressions:

UBC: $V = \frac{1.10}{R_w} W$ BSL: $V = 0.2\ W$

According to the BSL the minimum lateral force is meant to be representative of an earthquake of peak ground acceleration around 0.1g (Section 3.3.3.1). The BSL lateral force of 20% of structure mass (0.2W) is independent of the structural system ductility. However, for typical high-tech industrial buildings the UBC-91 minimum design base shear varies depending on the structure type (Section 3.5.1) to give values between 10% (SMRF) and 18% (Tilt-Up) of structure mass (0.1W-0.18W). Higher design base shear values are specified by the UBC-91 for more brittle structures (e.g.

masonry buildings), but these types of structures are typically not found in the high-tech industry.

Figure 4.8 shows a comparison of the UBC-91 and BSL code prescribed base shear for low-rise structures, and the base shear force expected from an earthquake with a PGA of 0.1g at low periods.

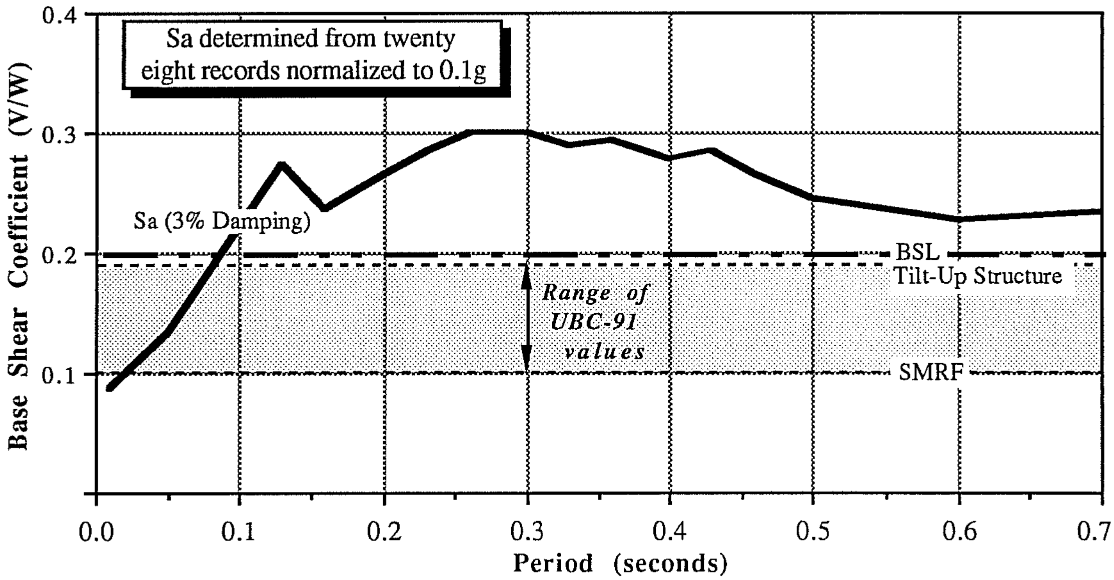


Figure 4.9 Capacity of Structures Designed According to UBC-91 and BSL Prescribed Base Shear, Compared with the Expected Base Shear Force due to Earthquakes with a PGA of 0.1g

As indicated in Figure 4.9 the base shear force prescribed by the UBC-91 is typically lower than the forces expected from an earthquake with PGA of 0.1g. However, material safety and load factors considered in the design process ensure that the true capacity of a structure is typically two or more times greater than the minimum design base shear prescribed by the codes. For this reason a typical low-rise structure is less likely to experience structural damage than is indicated by Figure 4.9. The minimum design base shear prescribed by the BSL is closer than the UBC-91 to the expected level of forces as shown in figure 4.9. However, even in the BSL, material safety and load factors are required to provide sufficient strength to avoid structural damage in moderate earthquakes.

While low-rise structures may be capable of resisting the force from an earthquake with a PGA of 0.1g, it would only be as a result of additional strength provided by material safety and load factors. However, these material and load factors are meant to provide a level of safety against the possibility of an unlikely, extreme loading. According to the results of Algermissen & Perkins (Figure 4.5), in UBC-91 seismic zone IV, the annual risk of exceeding an earthquake with an EPA of 0.1g is 0.08 (i.e. a return period of 12.5 years). As such, an earthquake with a PGA of 0.1g cannot be considered an unlikely, extreme event. For this reason the current levels of UBC prescribed design base shear is not likely to be acceptable for many critical buildings in the high-tech industry where the large losses associated with structural damage and disruption of operations must be avoided (Section 2.5.3). The minimum level of lateral force that the codes specify must be resisted by low-rise structures may therefore not be large enough to meet the SLS objective of avoiding structural damage in moderate earthquakes.

Figure 4.9 also indicates how the difference in UBC-91 prescribed base shear versus expected force is greater for the more ductile structural systems with higher ductility factors (Rw). For this reason a low-rise structure constructed with a special steel moment resisting frame (SMRF) is likely to incur structural damage from a lower level of ground motion than the same building built with concrete tilt-up bearing walls.

4.5 RECOMMENDATIONS

It is apparent from the above discussion that the strength prescribed by the UBC-91 and NEHRP-88, for low-rise structures, may not be sufficient to prevent structural damage in a moderate earthquake. This problem is of particular interest to the high-tech industry where disruption of operations may be as important as life safety. It is recommended that when continuity of operations is important, a two phase design procedure may be more appropriate than the current UBC and NEHRP approach. In the first phase, a minimum strength aimed specifically at meeting the SLS objectives would be determined. In the second phase of design, life safety issues for an extreme earthquake would be addressed.

In the first phase design it is obvious that the greater the strength required the higher the final cost of construction. For this reason the owner and engineer should agree on a level of risk that is acceptable. Based on the acceptable annual risk and the site location (seismic zone), the effective peak ground acceleration expected at the site can be ascertained from Figure 4.5. The results of the analytical study (Figure 4.8), or a similar study based on more local or non west-coast earthquake records, could then be used to determine the base shear force that must be resisted to meet the accepted level of risk.

4.6 SUMMARY AND CONCLUSIONS

In this chapter, a previous study was described in which it was shown that peak ground acceleration of an earthquake record is the best indicator of the base shear force likely to be induced on a low-rise structure. The base shear force that can be expected from earthquake records of various levels of peak ground acceleration were calculated and presented. Based on these results the ability of the current seismic code provisions to meet the service limit state objectives was examined and the following can be concluded.

Current UBC-91 and NEHRP-88 seismic code provisions adequately meet ultimate limit state objectives for life safety and structural integrity. However, the code provisions may not adequately meet the service limit state objective of avoiding structural damage in moderate earthquakes. It was shown that ductile low-rise structures, located on the west coast of the U.S., may not resist forces that result from an earthquake with a return period as low as thirteen years. This may not be adequate protection for structures in the high-tech industry where structural damage and disruption of operations should be avoided in the design-life (expected life) of the structure.

Inherent in the current design provisions of both the UBC and NEHRP is the assumption of inelastic activity and structural damage. Both UBC and NEHRP prescribe minimum lateral forces as a function of the ductility of the structural system. The higher the ductility of the structural system the lower the design lateral forces. However, the ductility of a structure only factors into the response of a structure once inelastic activity (structural damage) is initiated. For this reason, if meeting the service limit state objectives (preventing damage in moderate earthquakes) is a design objective, then the lateral forces to be resisted should be determined independent of the ductility available in the structural system. By this logic a structure constructed with a SMRF would experience damage in the same level of ground motion as the identical structure built with a concrete OMRF.

A two-phase design approach, similar to the BSL, is recommended where service limit state objectives are important. The first phase would be a design for strength that is dependent on the owners acceptable risk of structural damage. The second phase of design would be for life safety to ensure sufficient strength and ductility to meet the ultimate limit state objectives. This design approach would only be necessitated where sensitive processes critical to the overall economic viability of an industry or corporation are occurring within the structure. It would therefore be up to the owner to decide what level of risk of structural damage is acceptable, and for the engineer to consider that risk in the design.

CHAPTER V
CASE STUDY OF A TYPICAL HIGH-TECH INDUSTRIAL BUILDING

5.1 INTRODUCTION

A study of existing high-tech industrial structures subject to various ground motion records is necessary to evaluate the ability of current seismic codes to prevent structural damage in moderate earthquakes. In this study the response of three different designs of a typical high-tech industrial structure subject to a variety of ground motion records will be evaluated.

First, the original, existing structure designed in 1970 is studied. Second, the original structure is redesigned to meet current code provisions (UBC-91). Third, the structure is redesigned for service limit state (SLS) lateral loads. The study is divided into two sections, in the first an elastic analysis of all three designs under UBC-91 prescribed lateral loads is performed. An inelastic analysis of all three designs, each subject to four different ground motion records, is conducted in the second section. The performance of the designs are compared and the adequacy of current codes to meet service limit state objectives is discussed. The analytical response of the original structure is compared with the actual behavior of the structure observed during Loma Prieta. Any problems identified in the response of the actual structure, or in the analytical model, are described.

5.2 DESCRIPTION OF STRUCTURE

The building is a typical high-tech industrial structure in an area of high seismic risk (Figure 5.1). The two story structure was designed and built between 1970 and 1973, with a floor area of approximately 12,300 sq-ft per floor (Figure 5.2). The building functions as a typical office and/or light manufacturing building used by a high-tech industry.

Figure 5.1 Typical High-Tech Industrial Structure

5.2.1 Structural System

The lateral load resisting system of this structure consists of a chevron braced frame, concrete precast panels and composite floor slabs (Figure 5.3). The braced

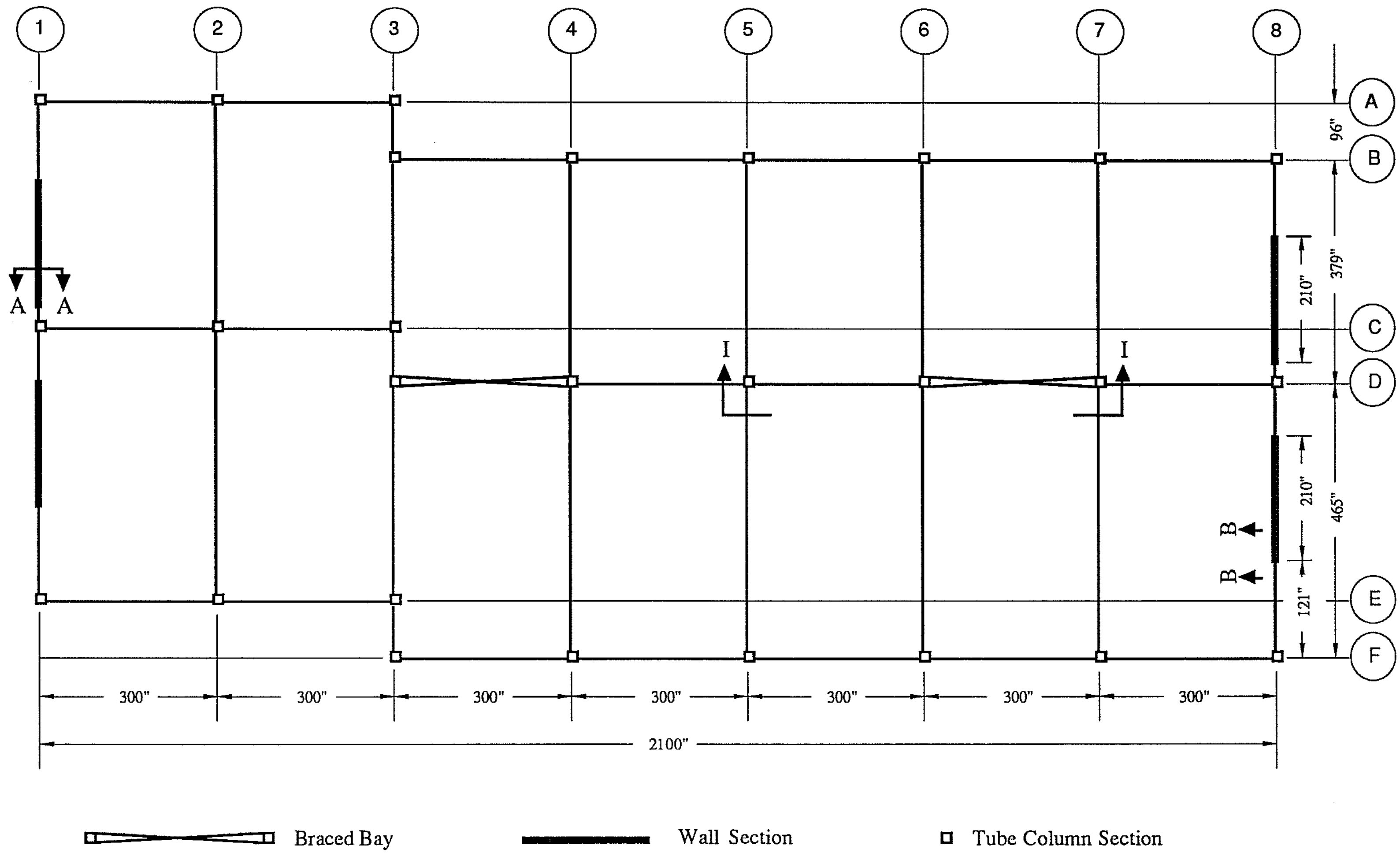


Figure 5.2 Floorplan of High-Tech Industrial Structure

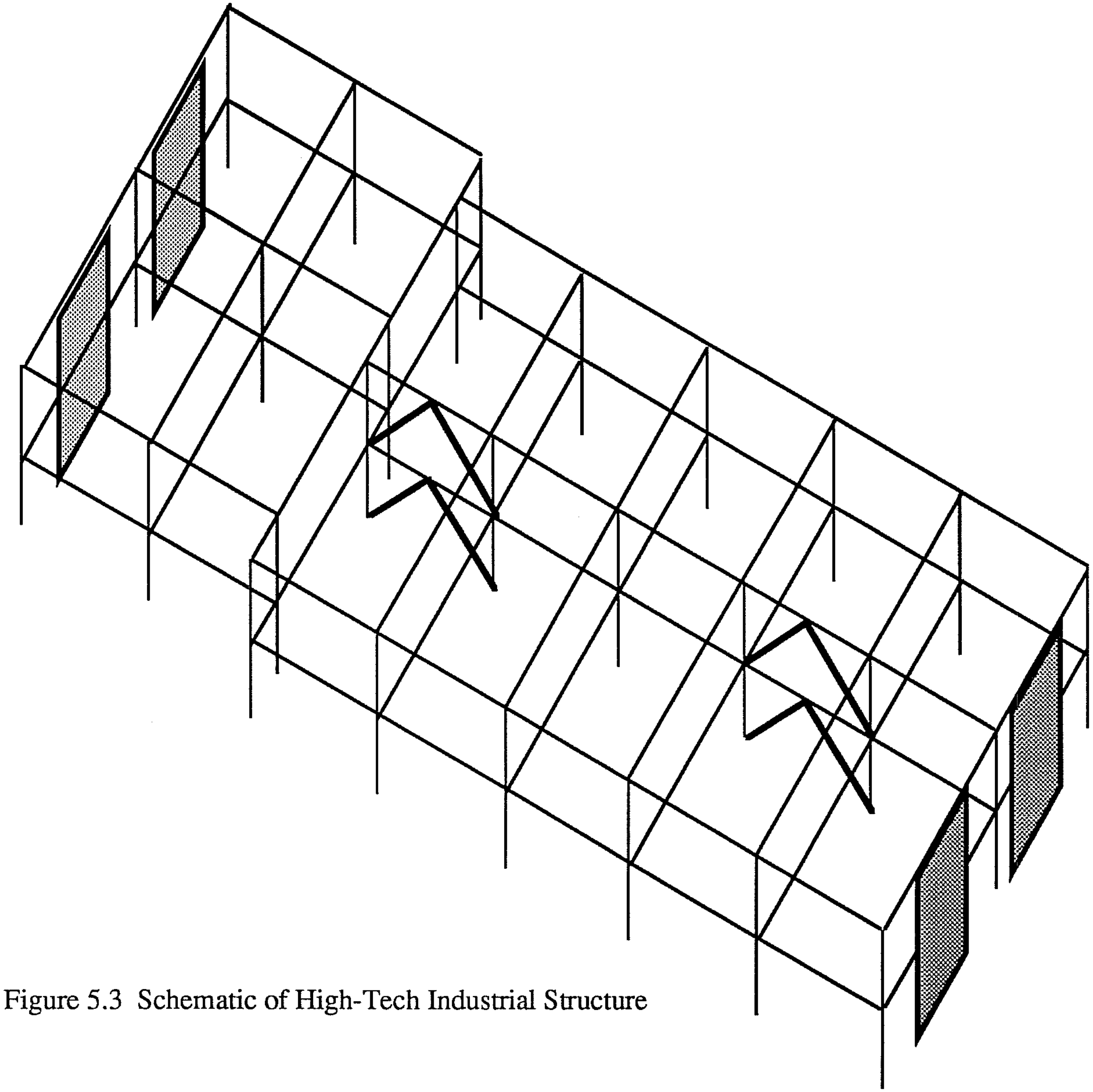


Figure 5.3 Schematic of High-Tech Industrial Structure

frame consists of double angles for the braces with tube sections for columns and W-sections for beams. The braced frame is located in the longitudinal direction along axis D of Figure 5.2, with an elevation of the frame shown in Figure 5.4. While the connections between columns and beams are capable of transmitting moment, the lateral stiffness provided by the braced frame (elastic range) ensures that the majority of any horizontal load is carried by the braces in the longitudinal direction.

Two six-and-a-half inch concrete precast panels along column lines 1 and 8 (Figure 5.2) provide the lateral resistance in the transverse direction. Shear forces in the transverse direction are transmitted from the precast panels to the foundation through shear keys between the panels and the ground floor diaphragm (Figure 5.5). The precast panels are connected to the roof and second level diaphragm by steel angles. The stiff concrete panels carry the overturning moment in the transverse direction and transmit this moment to the foundation through steel straps attached to each side of the panels (Figure 5.5).

A two-and-a-half inch post-tensioned concrete slab on a one-and-a-half inch deep corrugated metal deck forms the diaphragm of the second floor. The corrugated deck is supported by truss joists that span in the transverse direction distributing gravity loads between the exterior and interior beams and columns. At the roof level cold-formed steel sections and built up roofing material comprise the roof diaphragm. Steel truss joists in the transverse direction carry the gravity loads to the beams and columns. Horizontal forces are transmitted at the diaphragm level at both second and roof levels.

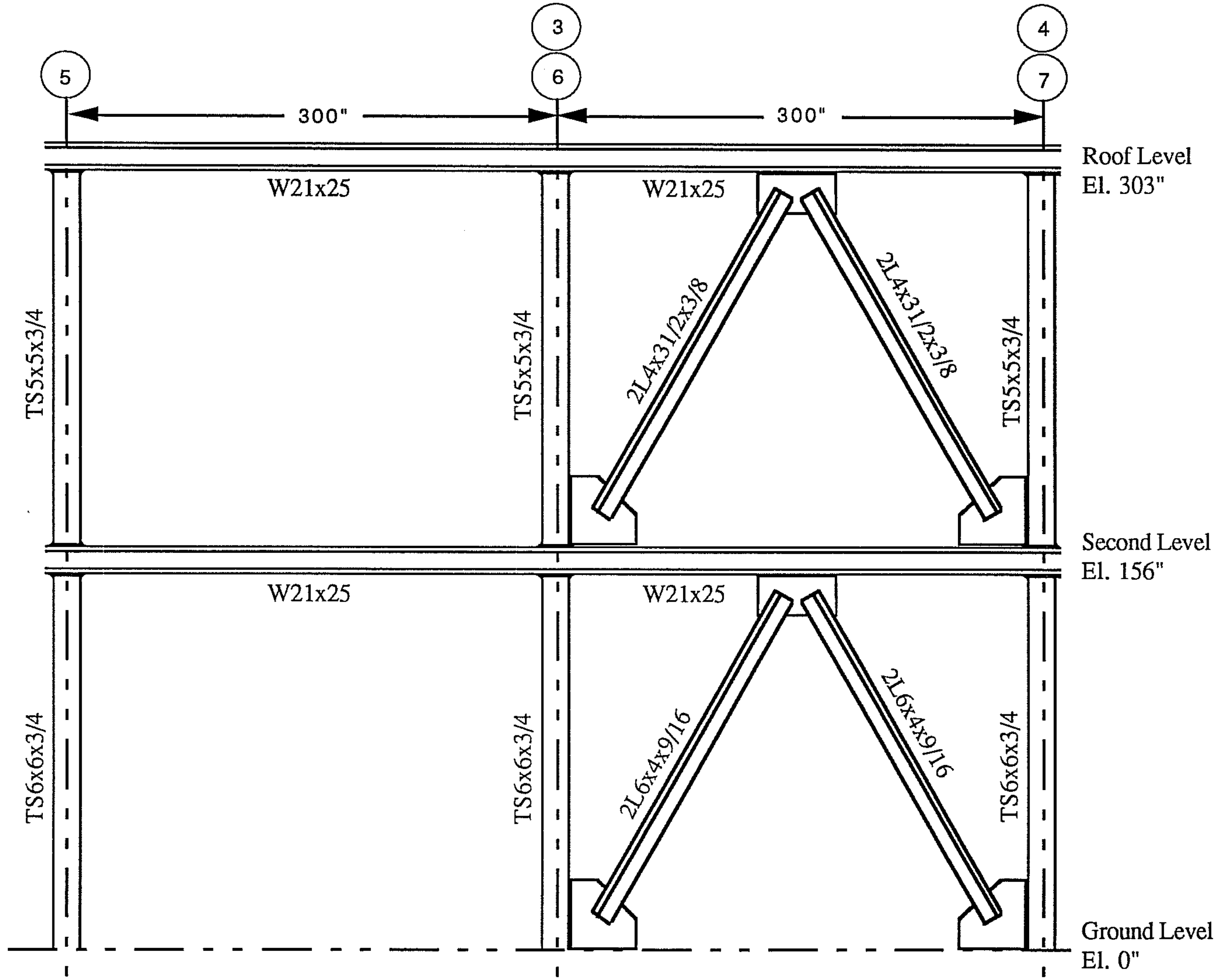
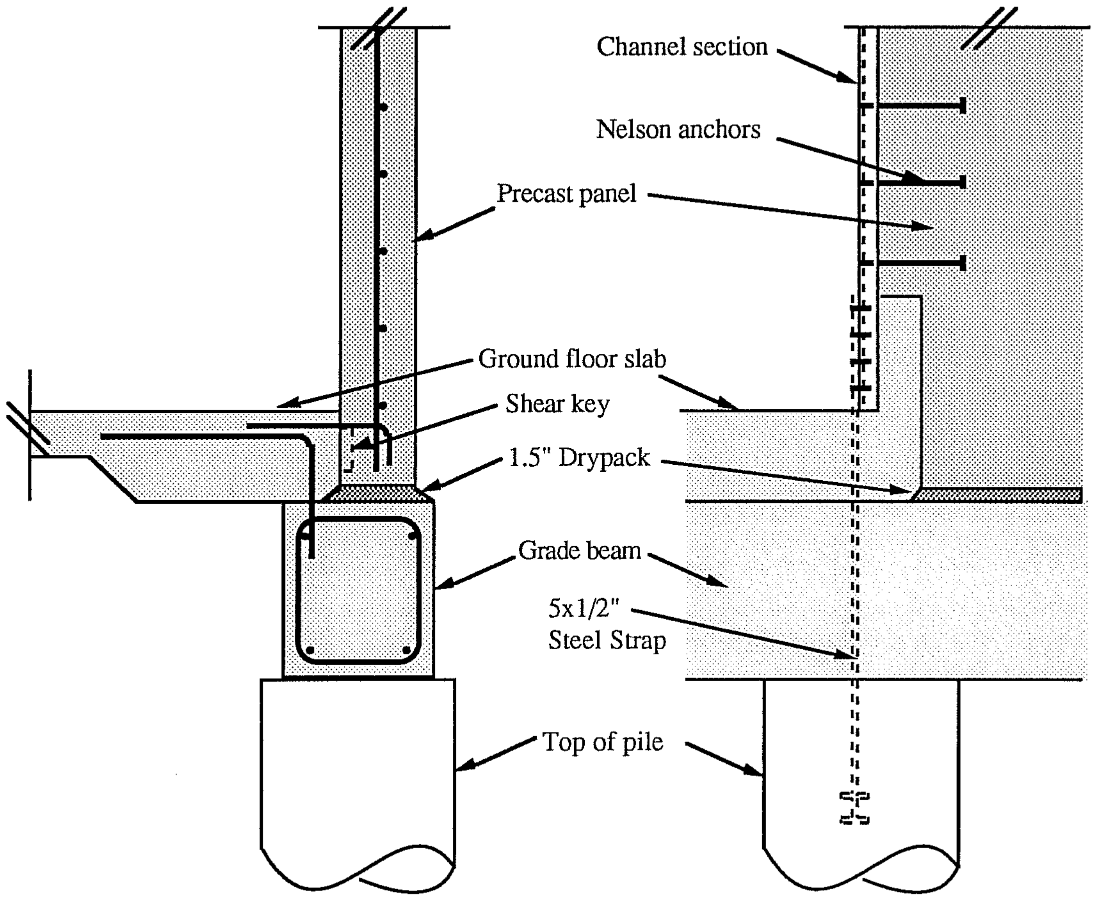


Figure 5.4 Elevation of Braced Frame



Section A-A*
Cross-section through precast panel

Section B-B*
Elevation of precast panel (end connection)

* *See Figure 5.2*

Figure 5.5 Details of Precast Panel in Transverse Direction

The foundation of this structure consists primarily of capped pile footings. A pile footing is located under each column and at each end of the precast panels. A grade beam is located around the entire edge of the structure as well as along column line 3 (Figure 5.2). The ground floor diaphragm consists of a four inch concrete slab on grade. Lateral shear forces are transmitted through the ground floor diaphragm to the pile footings below the floor.

5.2.2 Loads and Construction Materials

The dead and live loads used in the analyses were obtained from the original drawings and are shown in Table 5.1.

Table 5.1 Dead and Live Loads for the Typical High-Tech Industrial Structure

LEVEL	DEAD LOAD (psf)	LIVE LOAD (psf)
Roof Level	25	20
Second Level	40	65

The following are the material properties obtained from the original drawings:

Concrete:	f'c = 4000 psi for the post tensioned slab	
	f'c = 3000 psi for all other concrete	
Steel Reinforcement:	ASTM A615 Grade 40	
Structural Steel:	Tubes	ASTM A500 Grade B
	Structural Shapes	ASTM A36

5.3 ELASTIC ANALYSIS

A three dimensional elastic analysis using ETABS [27] was performed on the original structure, and the two redesigned structures. These three structures are referred to as follows:

- Original Structure - Existing structure built to resist UBC-70 lateral forces.
- Redesign I - Structure redesigned to resist UBC-91 code forces.
- Redesign II - Structure redesigned to resist a lateral force that meets SLS recommendations of an acceptable level of risk against structural damage during the life of the building.

5.3.1 Model for ETABS

Using ETABS the three structures are modelled in three dimensions by using the as-built conditions of the original structure as closely as possible. The bases of all columns are fixed at the foundation level and the columns are continuous to the roof level. The beams are continuous in the longitudinal direction, while the floor and roof joists in the transverse direction are pinned. The beams and columns are capable of carrying axial, shear and bending moments. While beam and column connections in the longitudinal direction are capable of transmitting moment, the lateral stiffness of the braces attracts over ninety percent of all lateral loads in the elastic analysis. Although the brace-beam connection is capable of carrying moment, the braces are assumed pinned and modeled as truss elements carrying only axial load. The precast walls are modeled as panels that carry shear loads and overturning moments. Material properties, element characteristics, boundary and loading conditions are the only variables required by ETABS to model these structures. Since ETABS performs an elastic analysis no member capacities are required.

5.3.2 Equivalent Static Lateral Forces

In this section the lateral forces prescribed by UBC-70 [35] and UBC-91 for this high-tech industrial structure are calculated. The lateral forces determined from SLS recommendations made in Section 4.5 are also presented. The results of the calculations are given in Table 5.2. Current UBC-91 minimum design base shear is 21% greater than UBC-70, but UBC-91 forces is still less than half that recommended to prevent structural damage in moderate earthquakes. Table 5.2 also shows the design base shear as a fraction of the mass of the structure. The overturning moment is given for a single precast panel.

Table 5.2 Comparison of Minimum Design Base Shear and Overturning Moment

DESIGN CODE	MINIMUM BASE SHEAR (kip)	MINIMUM BASE SHEAR (Fraction of W)	OVERTURNING MOMENT (kip-in)
UBC-70	105	0.11W†	6,050
UBC-91	127	0.14W*	7,300
SLS	277	0.30W*	15,800

† W is the weight of the structure as defined by UBC-70.

* W is the weight of the structure as defined by UBC-91.

5.3.2.1 *Lateral loads for original structure*

According to original drawings the design of the structure was completed in 1973. For this reason UBC-70 provisions were used to determine the lateral forces used in the design of the original structure. UBC-70 prescribed total base shear is calculated from the following expression:

$$\mathbf{V = ZKCW} \qquad (5\text{-}1)$$

where,

Z = 1.0 for the highest seismic zone

K = 1.0 for both directions

C = 0.10 for a two story structure in both directions

W = 1046 kip (Dead load plus 20 psf for partitions on second level)

The resulting base shear force (Vb) for which this structure was probably designed is:

Both Directions: $\quad$ Vb = 0.10 W = 105 kip

For analysis purposes this base shear is distributed equally between the two floor levels, as specified by the UBC-70 for structures less than two stories in height. These forces result in a base overturning moment of 6,050 kip-in that must be resisted by each of the precast panels in the transverse direction.

5.3.2.2 *Lateral loads for redesign I*

In order to predict the behavior of the structure when designed according to current design codes, the building was redesigned for lateral forces prescribed by UBC-91. According to UBC-91 the total base shear which the structure must resist is calculated from the following expression:

$$V = \frac{Z\,I\,C}{R_W}\,W \tag{5-2}$$

where the parameters of Equation (5-2) are designated as follows:

Z = 0.4 Seismic zone factor for the highest seismic zone
I = 1.0 Importance factor for typical office building
C = 2.75 (irrespective of soil and building period)
R_W = 8 Ductility factor is the same in both directions
W = 923 kip (Dead load plus 10 psf for partitions on second level)

The base shear force for the structure (used in the elastic analysis of all designs) is:

Both Directions $\quad V = 0.138\ W = 127$ kip

For analysis purposes this base shear force is distributed between the two floor levels according to UBC-91 specifications (64.4 kip at second floor, 62.6 kip at roof). These forces result in a base overturning moment of 7,300 kip-in that must be resisted by each of the precast walls in the transverse direction.

5.3.2.3 *Lateral loads for redesign II*

In Section 4.6 recommendations on a design approach to meet SLS objectives in high-tech industrial structures is presented. According to these recommendations a building owner could specify an acceptable level of risk against structural damage for which the engineer must design. For example, the owners of the new structure might specify that the useful life of a structure is to be twenty-five years and that they would like to avoid structural damage during that time. To avoid structural damage and meet all SLS objectives during this period of time, an earthquake with a return period of twenty five years should be the basis for design (a 25 year return period translates into a 4% risk of damage per year). As the structure is located in the highest seismic zone, the effective peak acceleration expected from an earthquake with a 4% annual risk is approximately 0.13g (Figure 4.5). Based on Figure 4.8 for a peak ground acceleration of 0.13g a structure with between 3% and 5% damping should be designed to resist a lateral force of thirty percent (30%) of the structure mass. This result could also be obtained by using the value of acceptable annual risk (4%) to determine the base shear coefficient (V/W) off the vertical axis of Figure 5.6. The design lateral force needed to meet the defined SLS level of risk is thus:

Both Directions: $V = 0.3\ W = 277$ kip

The mass (W) is the same as in UBC-91 and is distributed to each floor according to UBC-91 specifications (141 kip at 2nd level, 136 kip at roof level). This lateral force results in a base overturning moment of 15,800 kip-in that must be resisted by each of the precast walls in the transverse direction.

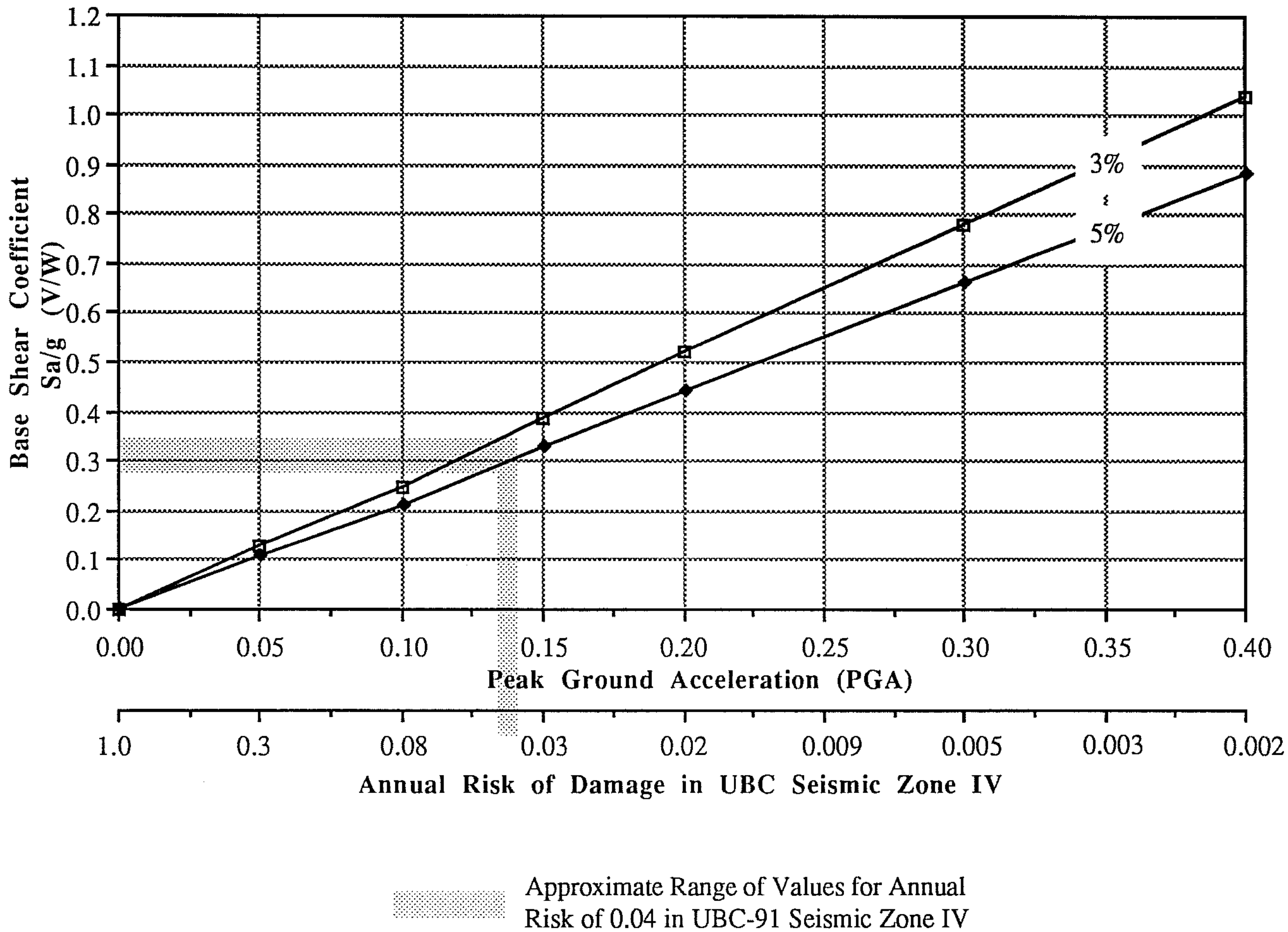


Figure 5.6 Lateral Force Due to Earthquakes with Various Levels of PGA

5.3.3 Member Forces

The lateral force resisting members of the original structure were checked to verify that they were capable of resisting UBC-70 and UBC-91 prescribed overturning moments and lateral forces. The structural members chosen for the two redesigned structures are presented along with the forces for which they were designed. Besides strength, all the braces are checked to ensure conformance with the current UBC and American Institute of Steel Construction (AISC) [28] design and detailing requirements.

The UBC and the AISC both prescribe limits on the slenderness ratio of braces in seismic regions to prevent rapid degradation of the brace strength and stiffness in the post buckling range. Specifically, the L/r ratio (length of brace over lowest radius of gyration) is limited to $720/\sqrt{Fy}$ by the UBC. This results in a L/r limit of 120 for ASTM A36 steel (Fy=36 ksi), and 106 for ASTM A500 Grade B steel (Fy=46 ksi). A further limit of $110/\sqrt{Fy}$ is specified by the AISC-Seismic Provisions Manual [28] for the b/t ratio (width to thickness) of the braces. This specification limits the b/t ratio to 18.33 for Fy=36 ksi steel and 16 for Fy=46 ksi steel, and is intended to prevent local buckling and brace fracture under repeated cyclic loading. However, the UBC-91 does allow member sections to be used that do not satisfy the UBC requirements provided the strength of the brace is capable of resisting (3Rw/8) times the code specified lateral load.

5.3.3.1 *Original structure*

As indicated in Table 5.3 from calculations according to the AISC-ASD [29] the braces of the original structure have adequate strength to resist both the UBC-70 and UBC-91 prescribed lateral loads. However, the second level braces do not meet the current slenderness requirements of the UBC-91. The L/r ratio of 160 would satisfy all the UBC-70 requirements, but exceeds the 120 limit specified in the UBC-91.

Table 5.3 Braces in Original Structure Longitudinal Frame

	LEVEL		
	FIRST	**SECOND**	**NOTES**
Length (in)	195	200	
Section	2L6x4x$^9/_{16}$	2L4x3$^1/_2$x$^3/_8$	
KL/r	120	**160**	Second level exceeds 120 limit [13]
b/t	10.7	10.7	
Pcd (kip)	117	32	Compression capacity of brace using AISC-ASD [29] and UBC-91 [13]. Includes a 33% increase specified by AISC-ASD and a reduction of 1/(1+(kl/r/2Cc)) specified by the UBC91
Ptd (kip)	252	115	Tension capacity of brace sections using AISC-ASD and UBC-91.
UBC-70: Pr (kip)	35.6	18.7	Member force as a result of UBC-70 prescribed base shear
UBC-91: Pr (kip)	43.0	22.3	Member force as a result of UBC-91 prescribed base shear

The shear and moment capacity of the precast panels are capable of carrying the lateral forces prescribed by the UBC-91. Table 5.4 presents the shear and overturning moment due to the UBC-91 prescribed loads, the capacity of the steel straps at the end of the walls, and the shear capacity of the concrete in the precast walls. The design strength due to the UBC-91 specified forces are seen to be well below the capacity of the existing precast panels and straps.

Table 5.4 Walls in Transverse Direction of Original Structure

	BASE LEVEL	NOTES
Wall Dimension (in x in)	210 x 6 1/2	
Strap Dimension (in x in)	5 x 1/2	
Overturning Moment (kip-in)	7,300	
Pt (kip)	34.8	Force in the straps at the sides of each precast wall from the overturning moment
Ptd (kip)	54.0	Tensile capacity of the steel straps according to AISC-ASD [29]
Shear (kip)	31.8	Shear force due to UBC-70 base shear in each panel
Vc (kip)	129	Shear capacity of the concrete in the precast wall calculated according to ACI 318-89 ($2\sqrt{f'c}hd$)

5.3.3.2 *Redesign I*

The original structure was redesigned for UBC-91 prescribed lateral loads and in accordance with the UBC slenderness and local buckling provisions. Table 5.5 shows the redesigned brace member sizes, the capacity of these members and the

Table 5.5 Braces in Longitudinal Frame of Redesign I

	LEVEL		NOTES
	FIRST	SECOND	
Length (in)	195	200	
Section	TS5x5x5/16	TS4x4x5/16	
L/r	103	**135**	Second level exceeds L/r<106 limit [13]; however, the brace section has the strength to carry 3Rw/8=3 times the UBC-91 specified lateral load which therefore exempts it from the local buckling and slenderness.requirements of the UBC-91.
b/t	15	12	
Pcd (kip)	70.9	37.3	Compression capacity of brace. The value (for second level) does not include an increase of 33% nor a reduction of 1/(1+(kl/r/2Cc)) because the brace strength is defined by the UBC-91 as 1.7Pcd, where Pcd is the brace strength without any increases or reductions. The <u>second</u> level braces (strength = 1.7Pcd) are capable of resisting 3 times the UBC-91 load (Pr)
Ptd (kip)	155	120	Tension capacity of brace sections using AISC-ASD and UBC-91.
UBC-91:Pr (kip)	42	21	Member force as a result of UBC-91 prescribed base shear
Pr/Pcd	0.59	0.56	

UBC-91 load that they were designed to resist. These member sections are 33% more efficient (lighter) than those of the original structure. The additional efficiency results in a decrease in stiffness of this redesigned structure over the original structure. However, as tube sections replace the original double angles, the compressive capacity of the top braces are increased but that of the lower braces is reduced. The precast panels in the transverse direction were unchanged as they met current code requirements and contain adequate strength to resist the UBC-91 prescribed lateral loads.

5.3.3.3 *Redesign II*

The original structure was redesigned for SLS prescribed lateral loads and in accordance with the UBC-91 slenderness and local buckling provisions. Table 5.6 shows the redesigned brace member sizes, the capacity of these members and the forces they experience due to UBC-91 lateral loads. The strength of both roof and second level braces is seen to increase over the previous two designs. The brace member sections selected for redesign II are 7% lighter than those of the original structure; therefore, the lateral stiffness is slightly decreased from that of the original design. For this reason Redesign II has greater strength but less stiffness than the original structure. Once again the precast walls in the transverse direction were not changed as they provide adequate strength to resist the SLS prescribed lateral loads and meet all UBC-91 requirements.

Table 5.6 Braces in Longitudinal Frame of Redesign II

	LEVEL		NOTES
	FIRST	SECOND	
Length (in)	195	200	
Section	TS6x6x3/8	TS5x5x5/16	
L/r	85.9	103	
b/t	15	15	
Pcd (kip)	132	70.9	Compression capacity of brace sections using AISC-ASD and UBC-91.
Ptd (kip)	223	155	Tension capacity of brace sections using AISC-ASD and UBC-91.
SLS: Pr (kip)	90	45	Member force as a result of SLS recommended lateral loads
UBC-91:Pr (kip)	42	21	Member force as a result of UBC-91 prescribed base shear
UBC-91 Pr/Pcd	0.32	0.32	

5.3.4 Response Characteristics

Using ETABS the dynamic characteristics of the three designs were estimated. The dynamic characteristics of the three designs are calculated to verify that these designs meet the definition of a typical high-tech industrial structure. Also, the recommendations presented in Section 4.5 are only applicable to low-rise high-tech industrial structures whose total response is predominantly due to first mode response.

5.3.4.1 *Original structure*

The dynamic characteristics of the original structure were estimated by ETABS and are shown in Table 5.7. The low fundamental period of this structure is consistent with that of a typical high-tech industrial structure. As a three dimensional analysis was conducted with ETABS the response modes were determined in both major directions. The first two modes of response are in the more flexible longitudinal direction, while the third mode of response lies in the more rigid transverse direction.

Table 5.7 Response Characteristics for Original Structure (UBC-70)

	MODE 1	**MODE 2**	**MODE 3**
Period (sec)	0.212	0.115	0.080
	Mode Shapes		
	Longitudinal Frame		**Transverse Dir.**
Roof	1.00	0.94	1.00
Second	0.39	-1.00	0.38
	Effective Mass Factor (%)		
	80.93	19.07	80.23

5.3.4.2 *Redesign I*

The original structure was redesigned for UBC-91 lateral loads and the dynamic characteristics of this redesigned structure are shown in Table 5.8. While the response characteristics of this redesigned structure are similar to the original structure, more efficient sections were selected in the redesign which reduced the stiffness of the structure. The reduced stiffness resulted in a slight increase in the periods of the first two modes as shown in Table 5.8. The larger effective mass in the first mode of redesign I, compared to that of the original structure, indicates a greater contribution of the first mode response to the total response.

Table 5.8 Response Characteristics for Redesign I (UBC-91)

	MODE 1	MODE 2	MODE 3
Period (sec)	0.265	0.129	0.078
	Mode Shapes		
	Longitudinal Frame		**Transverse Dir.**
Roof	1.00	1.00	1.00
Second	0.54	-0.92	0.38
	Effective Mass Factor (%)		
	91.2	8.8	79.81

5.3.4.3 *Redesign II*

The original structure was redesigned for SLS lateral loads and the dynamic characteristics of this redesigned structure are shown in Table 5.9. Due to the increased stiffness this structure exhibits a lower fundamental period, but a smaller first mode contribution to the total response, than in redesign I.

Table 5.9 Response Characteristics for Redesign II (SLS-Acceptable Risk)

	MODE 1	MODE 2	MODE 3
Period (sec)	0.228	0.114	0.078
	Mode Shapes		
	Longitudinal Frame		**Transverse Dir.**
Roof	1.00	1.00	1.00
Second	0.49	-1.00	0.38
	Effective Mass Factor (%)		
	88.54	11.46	79.81

5.3.5 Floor Displacements

The floor displacements and drift ratios for each of the structural designs subject to UBC-91 lateral loads were estimated using ETABS and are presented in Table 5.10. The drift values are compared to the allowable drift limits specified by UBC-91.

Table 5.10 Floor Displacements for Structures Subject to UBC-91 Prescribed Loads

	LONGITUDINAL DIR.		TRANSVERSE DIR.		Allowable
LEVEL	**Displ. (in)**	**Drift (%)**	**Displ (in)**	**Drift (%)**	**Drift (%)**
Original Structure					
Roof	0.092	0.037	0.013	0.008	0.50
Second	0.037	0.024	0.005	0.003	0.50
Redesign I					
Roof	0.140	0.044	0.013	0.008	0.50
Second	0.075	0.048	0.005	0.003	0.50
Redesign II					
Roof	0.106	0.036	0.013	0.008	0.50
Second	0.053	0.034	0.005	0.003	0.50

In all cases the computed inter-story drifts are well below the allowable drift limits specified by the UBC-91 provisions. The drift limits specified by the UBC-91 are based on an elastic analysis and are not representative of the inelastic deformation that can be expected in a major earthquake.

5.4 INELASTIC ANALYSIS

An inelastic analysis was conducted using DRAIN-2D [30] to evaluate the performance of the three designs of the high-tech industrial structure, subject to various ground motion records. An inelastic analysis is not conducted for the precast walls in the transverse direction as they were previously shown to exhibit substantial over-strength and are not likely to suffer any significant damage under seismic loading.

5.4.1 Model for DRAIN-2D

DRAIN-2D is a computer program capable of modelling the inelastic response of certain structures subject to dynamic loading. The program contains various element models whose individual behavior are combined to determine the response of the entire structure. In the inelastic analysis of this structure the beam-column and buckling elements were used to model the behavior of the beams, columns and braces of the braced frame. For all the DRAIN-2D models, three percent (3%) of critical damping was considered to be appropriate for the steel structure being analyzed.

5.4.1.1 *Beams*

While the beams in the longitudinal direction are continuous throughout the length of the structure, it was shown in the elastic analysis that over ninety percent of the lateral load is carried by the braces. For this reason the beams are modelled as pin ended allowing all the lateral force to be carried by the braces. The beams are, however, continuous over the junction of the chevron braces where large bending moments will occur if any of the braces buckle. As the beams are continuously connected to the floor slab and roof diaphragm, properties of the composite section were used in the analysis. A beam-column element which is capable of carrying axial, shear and bending forces was used to model the beams. The interaction surface used by DRAIN-2D to model the beam-column element is shown in Figure 5.7. The value of Py corresponds to the compression and tension yield load of the beams computed according to the AISC manual [28,29] without strength reduction factors. Plastic hinges are developed at the end of beam-column elements when the combination of axial and bending forces lies outside of the interaction surface.

5.4.1.2 *Columns*

Like the beams, the columns are also modelled as beam-column elements. The columns are continuous through the height of the structure and are modelled with a fixed base. Bending moments, shear forces and axial forces are capable of being carried by the columns.

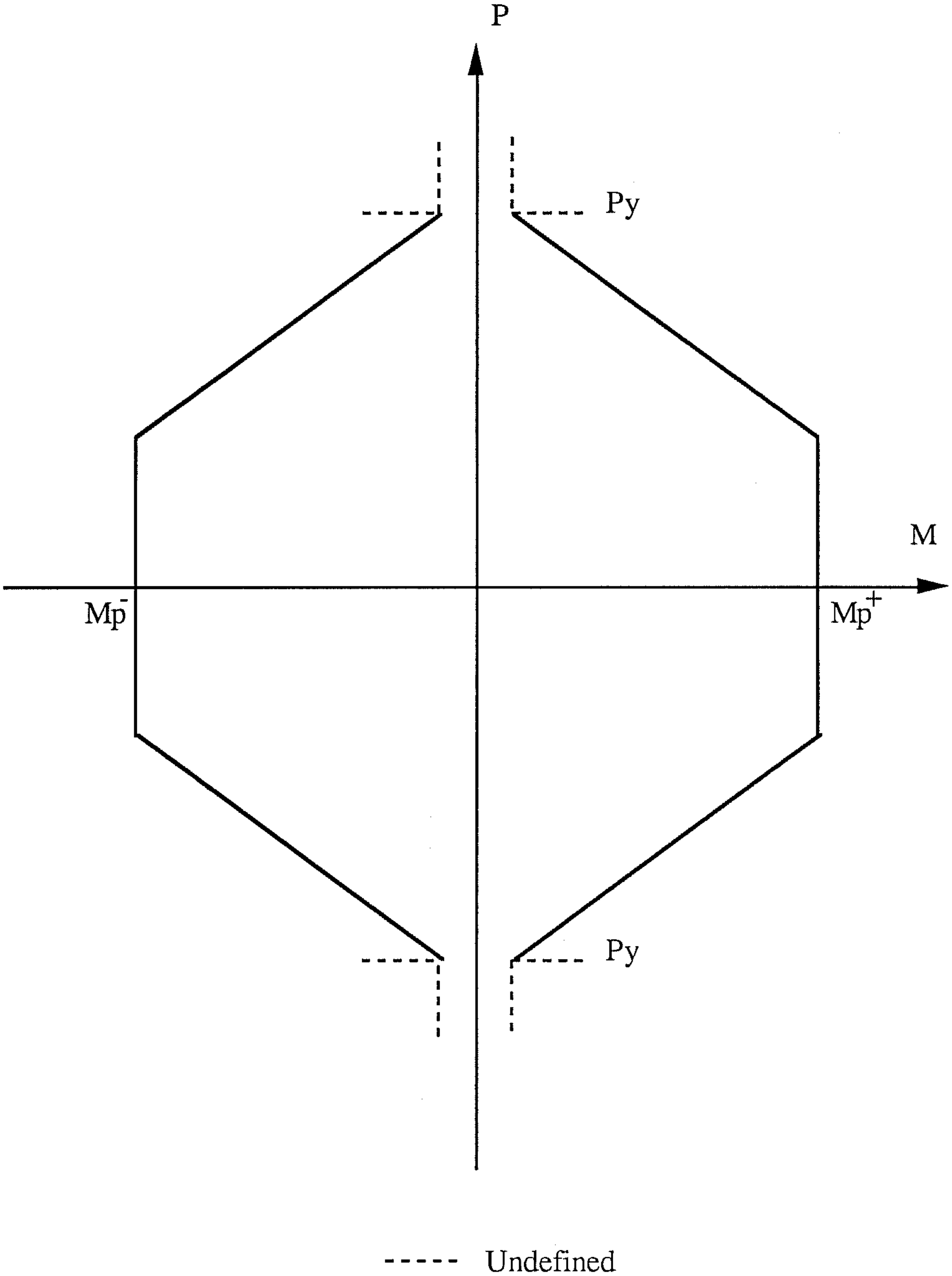


Figure 5.7 DRAIN-2D Model for Beam-Column Elements Interaction Surface [30]

5.4.1.3 *Braces*

The braces of this structure were conservatively modelled as buckling elements which only transmit axial forces [33]. However, in the real structure the braces are bolted to gusset plates at the connections to the beams. While these connections are capable of transmitting moment the majority of resistance is provided by the axial stiffness of the braces. In DRAIN-2D the buckling element model is based on a set of rules that describe the post-buckling/yielding behavior of the braces. Figure 5.8 shows typical loading paths defined by the buckling element in DRAIN-2D. The post-buckling/yielding rules followed by DRAIN-2D include a degradation of stiffness and strength after first buckling. The yield load in tension (Pyp) and the first buckling capacity (Pyn) are computed according to the provisions of the AISC manual and do not include any strength reduction factors. The post-buckling strength factor (Ø) is computed as $0.35/(L/\pi r(\sqrt{}Fy/E))$ [31], with the post-buckling strength (Pync) is calculated as ØPyn. As the section properties changed with each design so did the post-buckling strength factor. Fracture criteria were not included in this model.

5.4.2 Earthquake Records

The three designs were subject to four different ground motion records to evaluate their dynamic response. The first record is El Centro 1940, a record widely used in previous analytical studies. The other three records were produced by the Loma Prieta earthquake of 1989. The Corralitos-Eureka Canyon Road, Oakland-Outer Harbor Wharf and Palo Alto-Veterans Administration Hospital (VA) records were chosen. Table 5.11 shows basic information on the ground motion records.

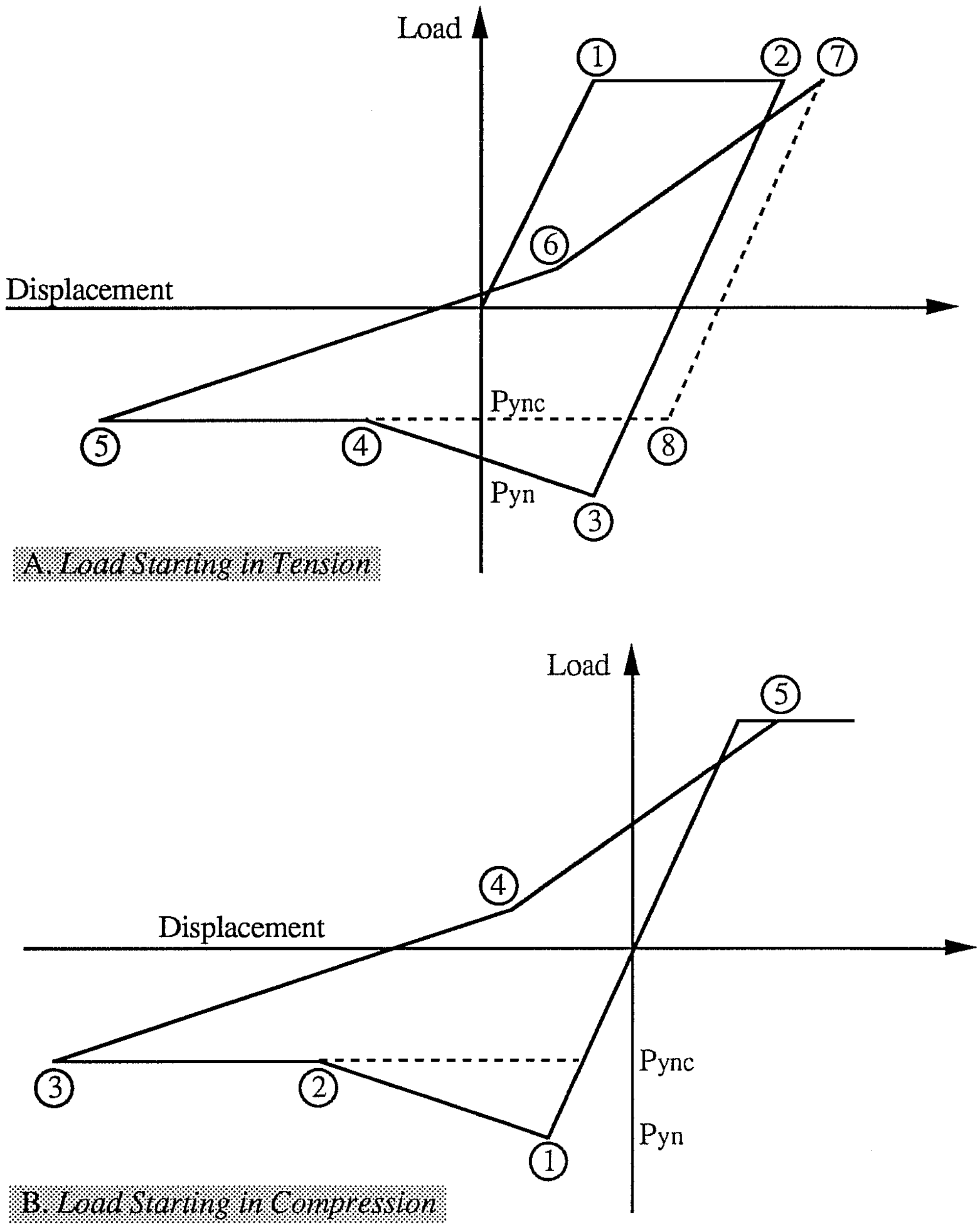


Figure 5.8 DRAIN-2D Model for Buckling Element [33]

Table 5.11 Ground Motion Records used in the DRAIN-2D Analytical Modelling

Recording Station	Direction	Soil Type	Peak Acc.	Magn.(M)
El Centro, 1940	N00E	Alluvium	0.35g	6.7
Corralitos, 1989	N00E	Rock	0.63g	7.1
Oakland Harbor, 1989	N55W	Bay Mud	0.27g	7.1
Veterans Hospital, 1989	N212E	Alluvium	0.36g	7.1

The VA record is of particular interest as this record was recorded in the vicinity of the original structure. For this reason the analytical model of the original structure subject to the VA record can be compared to the actual behavior of the original structure during the Loma Prieta Earthquake. Figure 5.9 shows the elastic pseudo-acceleration spectra for the four earthquakes with 3% damping in the low period range.

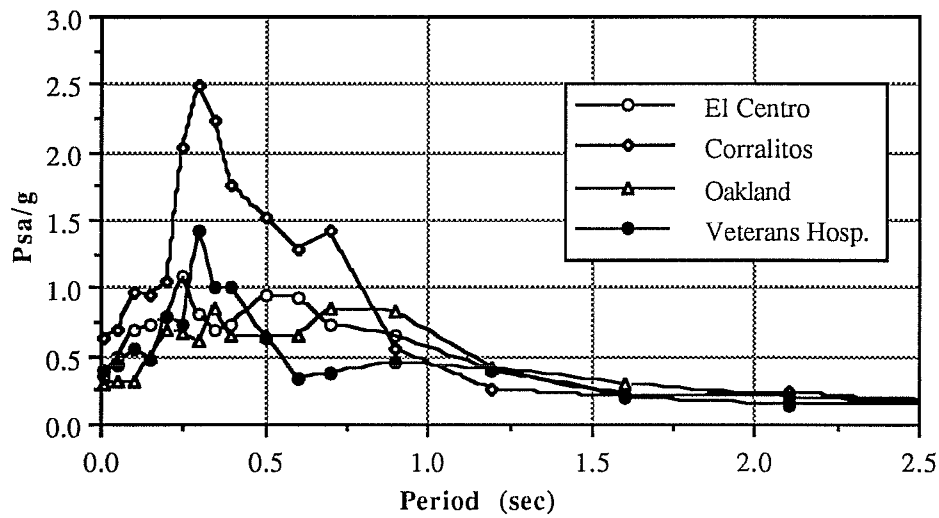


Figure 5.9 Elastic Pseudo-Acceleration Spectra

5.4.3 Description of Inelastic Behavior of Structure

Twelve different inelastic analyses were conducted in this case study. The three different designs of the structure are each subject to four different ground motion records. The results of the analyses in terms of structural damage (buckling of braces), floor displacements and accelerations are presented.

5.4.3.1 *Original structure*

Figure 5.10 indicates the buckling of braces and plastic hinges that occurred during the dynamic analysis of the braced frame of the original structure for each of the ground motion records. Also shown in this figure is the maximum cumulative plastic deformation for one brace at each level. For all earthquake records the braces were observed to buckle on the second level of the original structure. The large difference in stiffness and strength between the first and second level braces results in all the inelastic action and structural damage being concentrated in the second level. The large slenderness ratio of the second level braces results in low post-buckling strength (20% of initial capacity) and reduces the ability of the braces to dissipate energy. The Corralitos record is notably stronger than the other three records, resulting in over 12 inches of accumulated plastic deformation in the braces of the second level. Analysis of the original structure by the Corralitos record indicated the formation of plastic hinges in the beams above the roof braces.

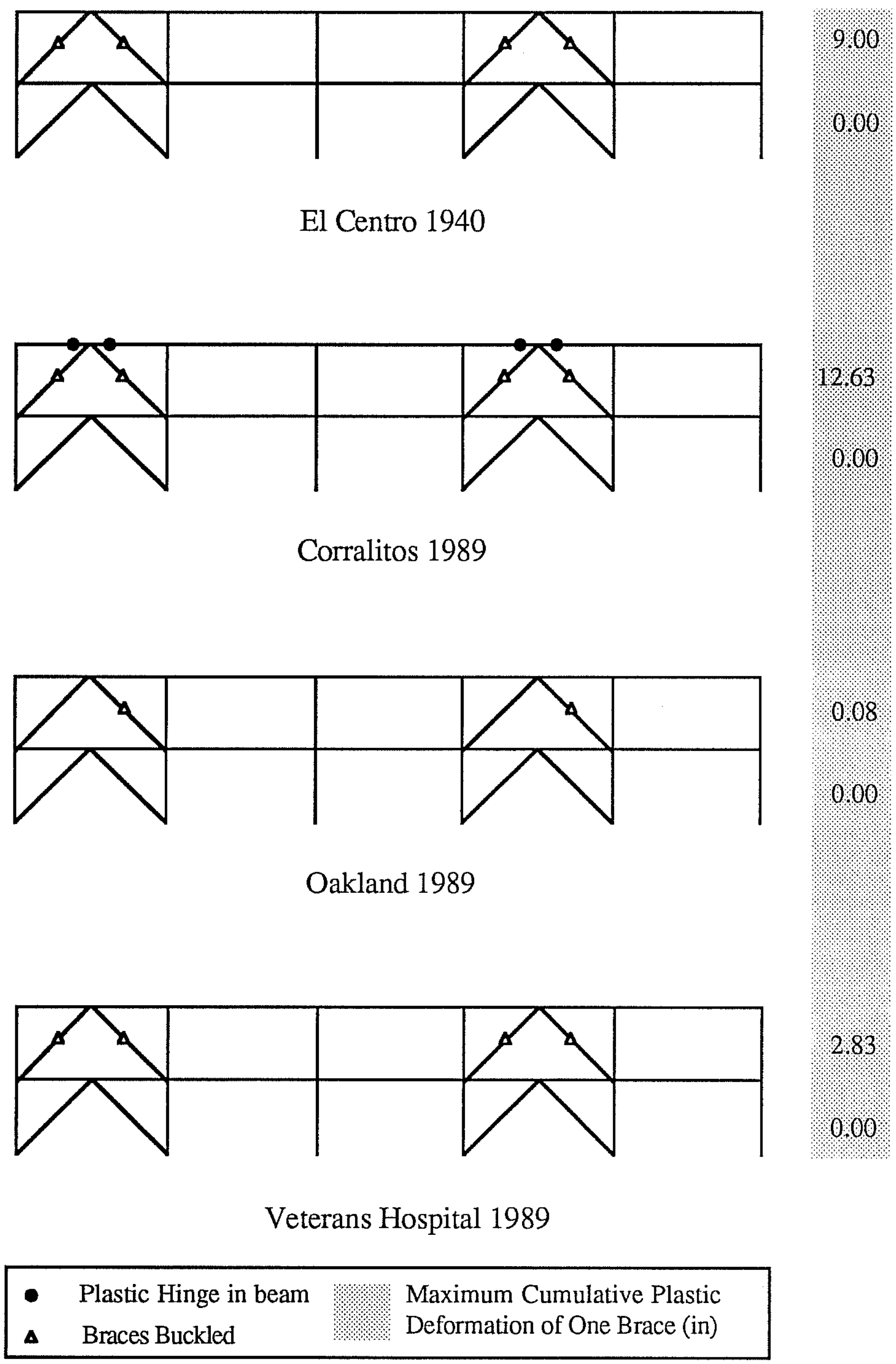


Figure 5.10 Inelastic Action of Original Structure

5.4.3.2 *Redesign I*

In the analysis of the original structure redesigned to resist UBC-91 lateral loads the braces are observed to buckle on both levels for all records. This result is consistent with the philosophy of the UBC-91 to distribute any inelastic activity evenly over the height of the structure. Plastic hinges are formed at the column bases and lower beams during analysis by the Corralitos earthquake record. Figure 5.11 indicates the buckling of braces and plastic hinges that occurred during the dynamic analysis of the redesigned braced frame for each of the ground motion records. Even though structural damage is more wide spread in redesign I than in the original structure, the maximum cumulative plastic deformations are reduced in redesign I for the El Centro and Oakland records. It is apparent that even when designed for UBC-91 lateral loads and provisions, this high-tech industrial structure is still likely to experience structural damage under moderate ground motion

5.4.3.3 *Redesign II*

As shown in Figure 5.12 only in the El Centro and Corralitos records was any structural damage observed in the analysis of the structure redesigned to meet specific SLS lateral loads. In these two cases where structural damage was observed it resulted in a maximum cumulative plastic deformation well below that of the original or redesign I structure. For the El Centro record the roof braces only buckled once due to the particular loading sequence of the record. Once again, any inelastic activity is distributed evenly among the braces of the two levels. In the case of the Oakland and Veterans Hospital earthquake records no structural damage was observed in the analysis.

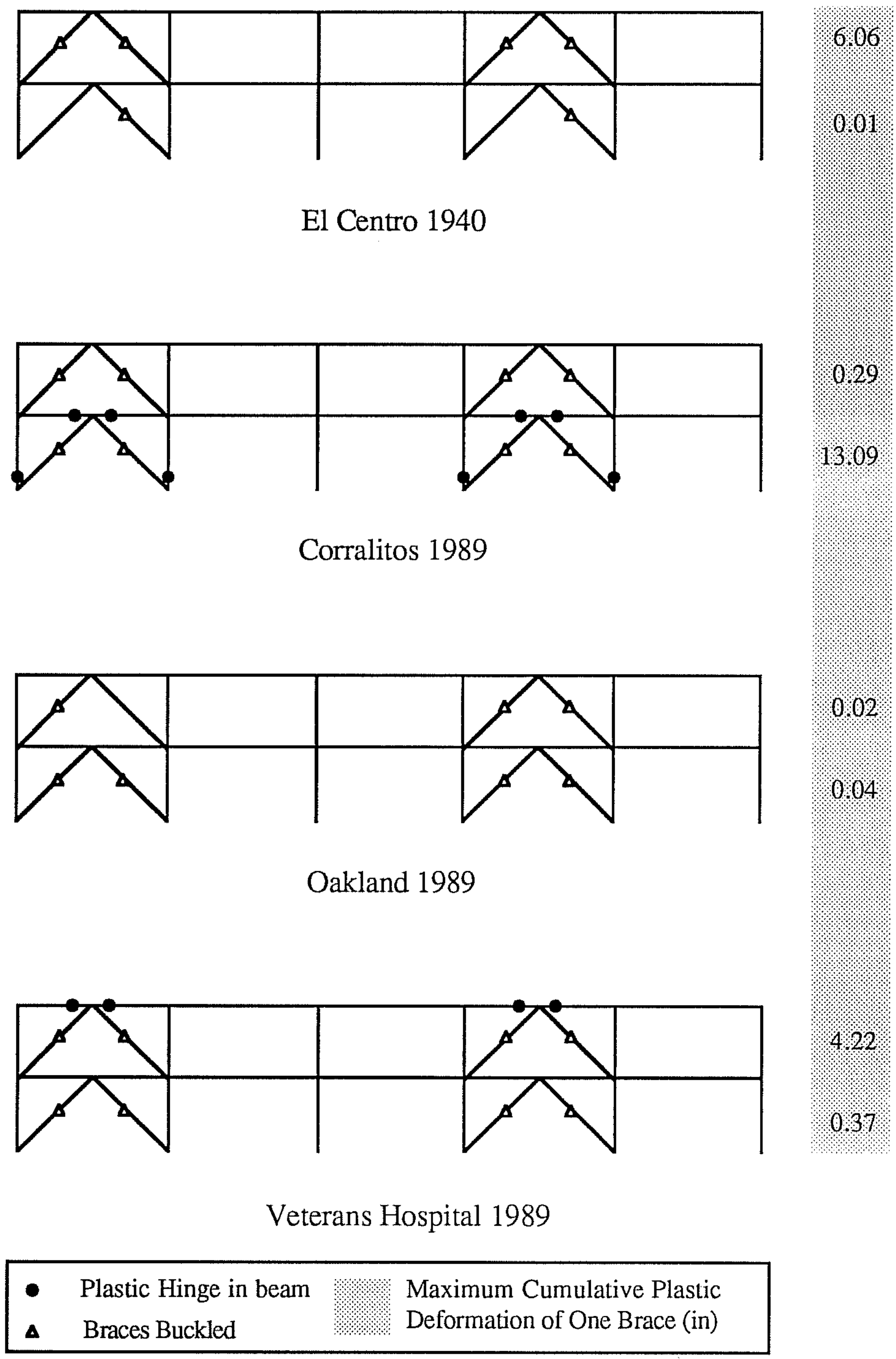


Figure 5.11 Inelastic Action of Redesign I

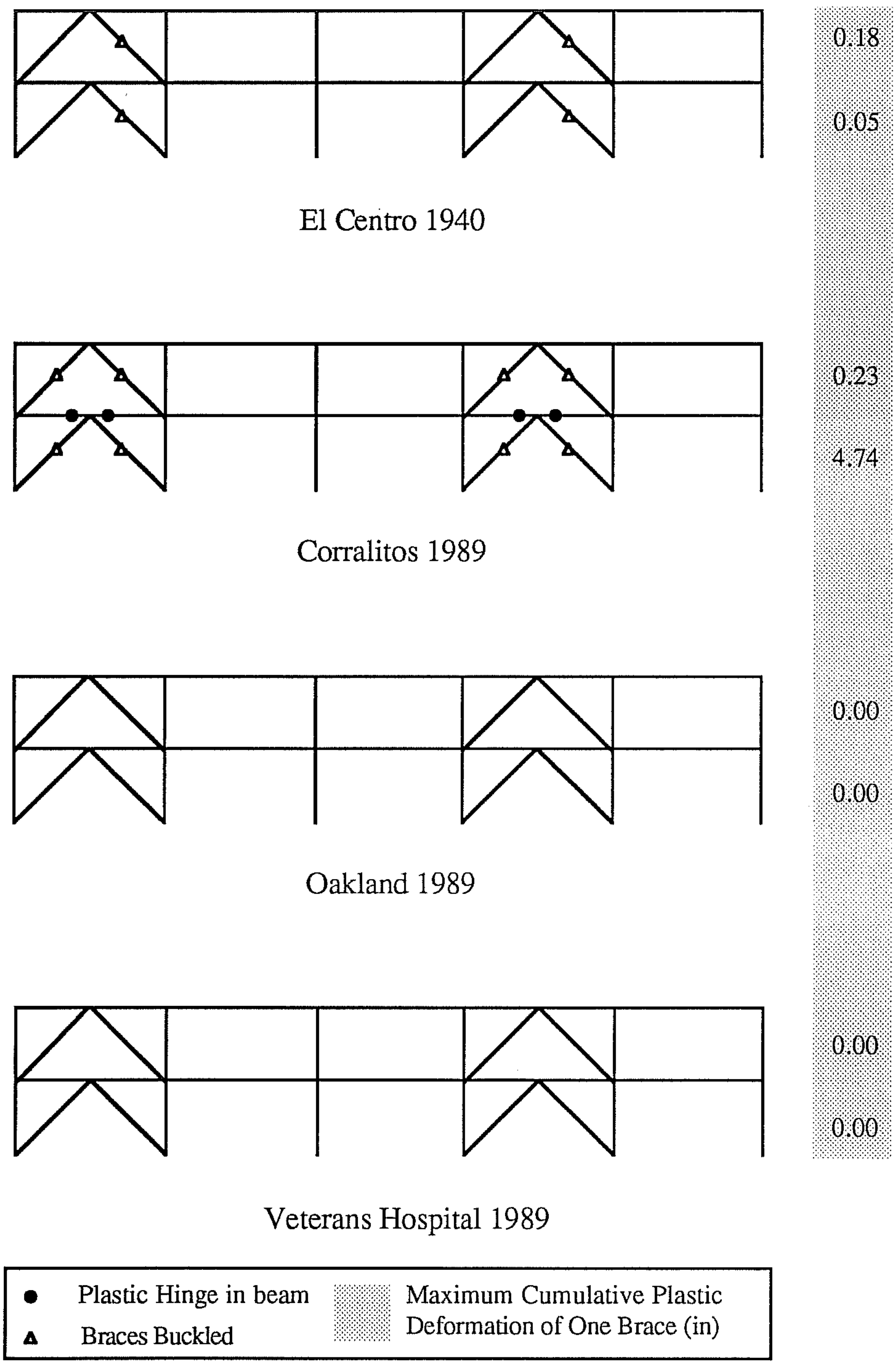


Figure 5.12 Inelastic Action of Redesign II

5.4.4 Response of Structures during Inelastic Analysis

The large losses associated with nonstructural damage observed in Loma Prieta (Chapter 2) has increased the awareness of the impact that drift and accelerations can have on the contents of a typical high-tech industrial structure. The maximum floor displacements and accelerations for each structure under the various ground motion records was calculated and is presented below.

5.4.4.1 *Inelastic floor displacements*

Figure 5.13 shows the drifts that were calculated in the DRAIN-2D analysis of the original and two redesigned structures subject to the four ground motion records. The largest drift for all three structures was observed using the Corralitos ground motion record. For this record a drift ratio of 1.7% was observed in the first level of redesign I and 1.4% in the second level of the original structure. This was the only case where drift ratios exceeded the 1.0-1.5% value assumed adequate for inelastic deflections [14]. The lighter brace members in the case of redesign I resulted in the largest inelastic drifts for all records except El Centro.

It is important to note that even though the brace sections selected for redesign II were lighter than those of the original structure, the drifts observed for redesign II were less than that of the original and redesign I in all but two instances. This is because the braces in redesign II had higher strength and deformed in the elastic range longer than the other designs.

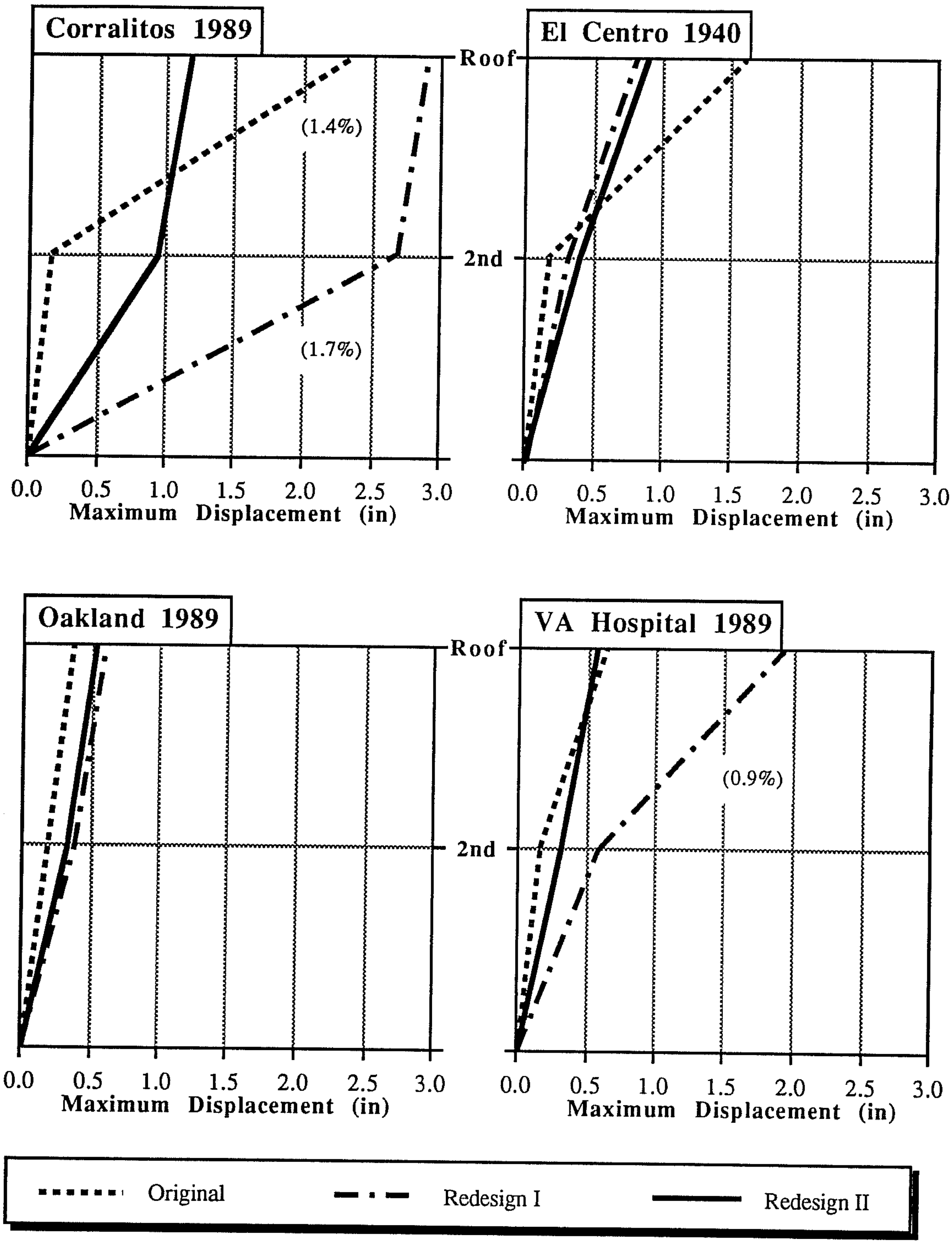


Figure 5.13 Maximum Floor Displacements

5.4.4.2 *Maximum floor accelerations*

Although there are no code guidelines limiting the allowable acceleration levels in a building subject to earthquake loading, there is some information that suggests limiting accelerations is essential where sensitive equipment is present [32]. Figure 5.14 shows all the peak floor accelerations for each of the structures subject to the four ground motion records.

It is apparent from Figure 5.14 that the largest accelerations occurred in the structure redesigned to resist the SLS lateral forces. This result is because redesign II has greater strength than the other two designs and is therefore capable of resisting higher inertia forces (accelerations). Figure 5.14 also illustrates that once the braces on a specific floor level have buckled the stiffness is reduced and the accelerations transmitted to that floor are reduced. For example, in El Centro 1940 the second level braces of the original structure buckle early in the event and the maximum acceleration at the roof level is no greater than that at the second level. However, for redesign II the braces buckle only once and the upper level maintains a large portion of its initial stiffness. As a result, roof level accelerations are almost twice that of the second level.

The amplification of the peak ground acceleration (PGA) in each floor level is also shown in Figure 5.14. Table 5.12 shows the floor accelerations and amplification of PGA for the El Centro and Corralitos records. While analysis by the Corralitos record was shown to produce the largest floor displacements and absolute floor accelerations, analysis by the El Centro record produced the largest amplification of PGA on the two floor levels. This result is likely due to the longer duration of intense

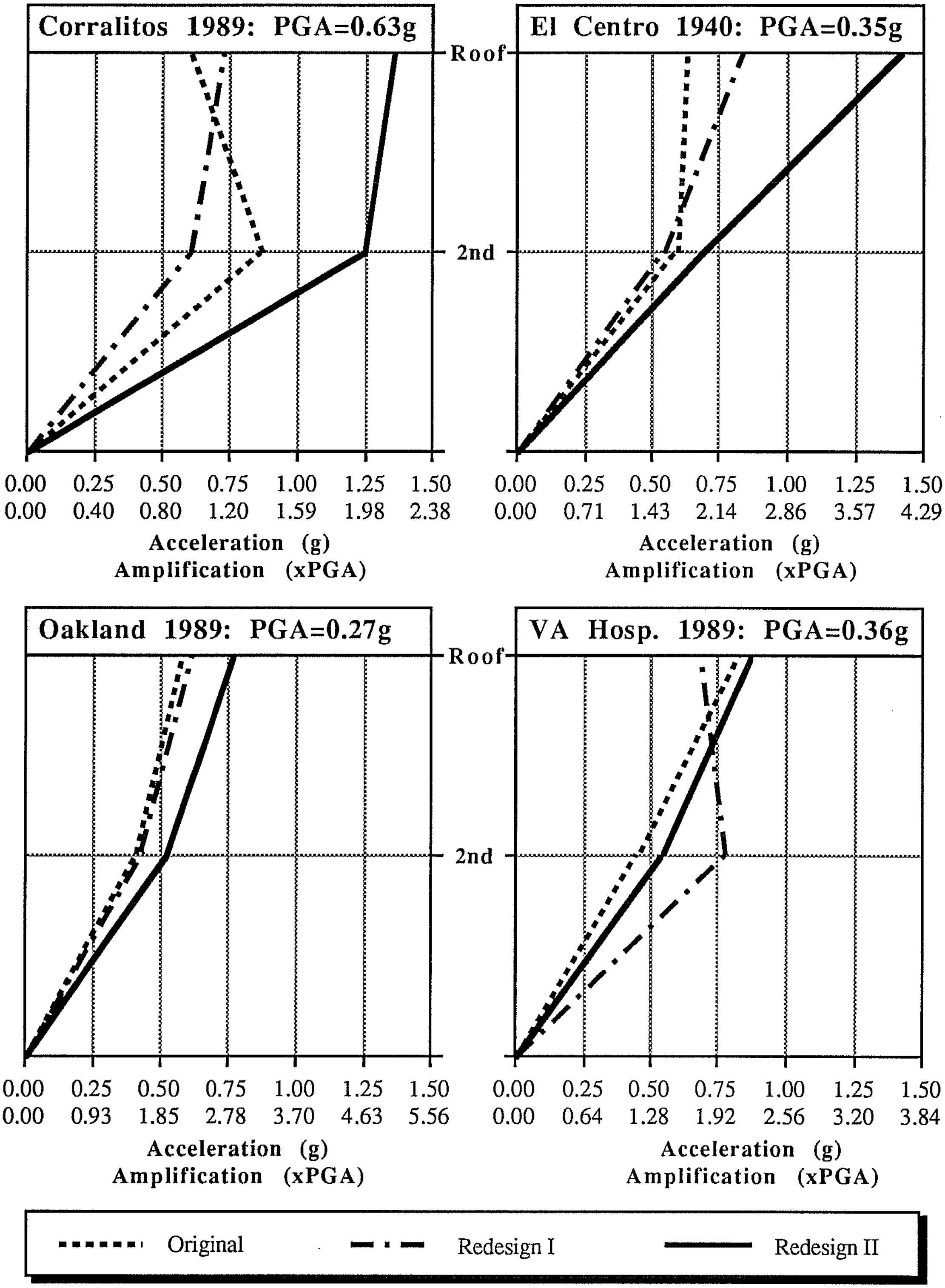


Figure 5.14 Peak Floor Accelerations and Amplification of Ground Acceleration

motion found in the El Centro record, and because the large acceleration of the Corralitos record cause the braces of the structures to buckle early in the analysis process. Table 5.12 also indicates that one of the disadvantages of designing a structure to sustain higher lateral forces is an increase in the acceleration amplification levels that will likely be observed during an earthquake. The stronger redesign II has less lateral stiffness than the original structure but amplifies the ground motion more that of the original structure.

Table 5.12 Floor Acceleration Levels for El Centro and Corralitos

	CORRALITOS (PGA = 0.63g)		EL CENTRO (PGA = 0.36g)	
Original Structure				
Level	*Acceleration (g)*	*Amplification*	*Acceleration (g)*	*Amplification*
Roof	0.61	0.96	0.62	1.71
Second	0.86	1.37	0.58	1.62
Redesign I				
Level	*Acceleration (g)*	*Amplification*	*Acceleration (g)*	*Amplification*
Roof	0.72	1.14	0.84	2.34
Second	0.60	0.95	0.52	1.45
Redesign II				
Level	*Acceleration (g)*	*Amplification*	*Acceleration (g)*	*Amplification*
Roof	1.35	2.15	1.44	4.00
Second	1.25	1.98	0.68	1.90

5.5 DISCUSSION OF RESULTS

The analytical response of the original structure is compared to the response observed in the real high-tech industrial structure during the 1989 Loma Prieta earthquake. The reason for the formation and impact of the plastic hinges that were observed in the beams in several of the analytical models is discussed. In redesigning the brace members for redesign I and redesign II it became apparent that the design force in the brace members did not always control the design. In some instances other code requirements may increase the strength of the selected member well above the required design force. This is discussed in light of the results presented in the previous section.

5.5.1 Analytical Response versus Real Response

As mentioned previously the high-tech industrial structure, on which the case study was based, experienced structural damage during the 1989 Loma Prieta earthquake. The most critical damage observed in the real structure was buckled braces. This correlates well with the results of the analytical study of the original structure when subject to the VA Hospital ground motion recorded closest to the real structure. Besides the buckled braces, the real structure also showed twisting damage to the gusset plate connection at the junction of the braces. This damage may be due to a number of different phenomena. First, eccentricity in the connection between the braces framing into the gusset plate results in large moments being induced in the gusset plate during earthquake loading. These moments may result in local buckling of the gusset plate. Second, the braces of the bottom level buckle out of plane. If these braces buckle they can cause the gusset plate to be bent out of its own plane and subsequent load reversal result in out of plane forces on the gusset plate. Third, the sudden redistribution of forces at the gusset plate when one of the braces buckles during cyclic loading may also buckle the gusset plate.

5.5.2 Behavior of Chevron Braced Frame

The deficiencies of the chevron braced frame system have already been identified and documented [14]. Some of these deficiencies were observed in the inelastic models and are explained schematically in Figure 5.15. The top row of Figure 5.15 describes the compression brace load-deformation response in a chevron braced frame subject to monotonic loading. The second row is a schematic of the frame response under an increasing lateral load. The frame is assumed to have columns fixed at the base with a continuous pin-ended beam spanning between the columns. The braces are assumed to be pin-ended and transmit only axial forces (k=1). The third row describes the tension brace load-deformation response, while the fourth row describes the overall load deformation response of the entire frame.

As shown in Figure 5.15, step A, as the braced frame is loaded the tension and compression braces carry equal magnitudes of load as they frame into the top beam at the same angle. The vertical component of the tension brace cancels the vertical component of the compression brace and no moment is induced in the beam above the braces. In step B the compression brace is at its buckling capacity; however, the

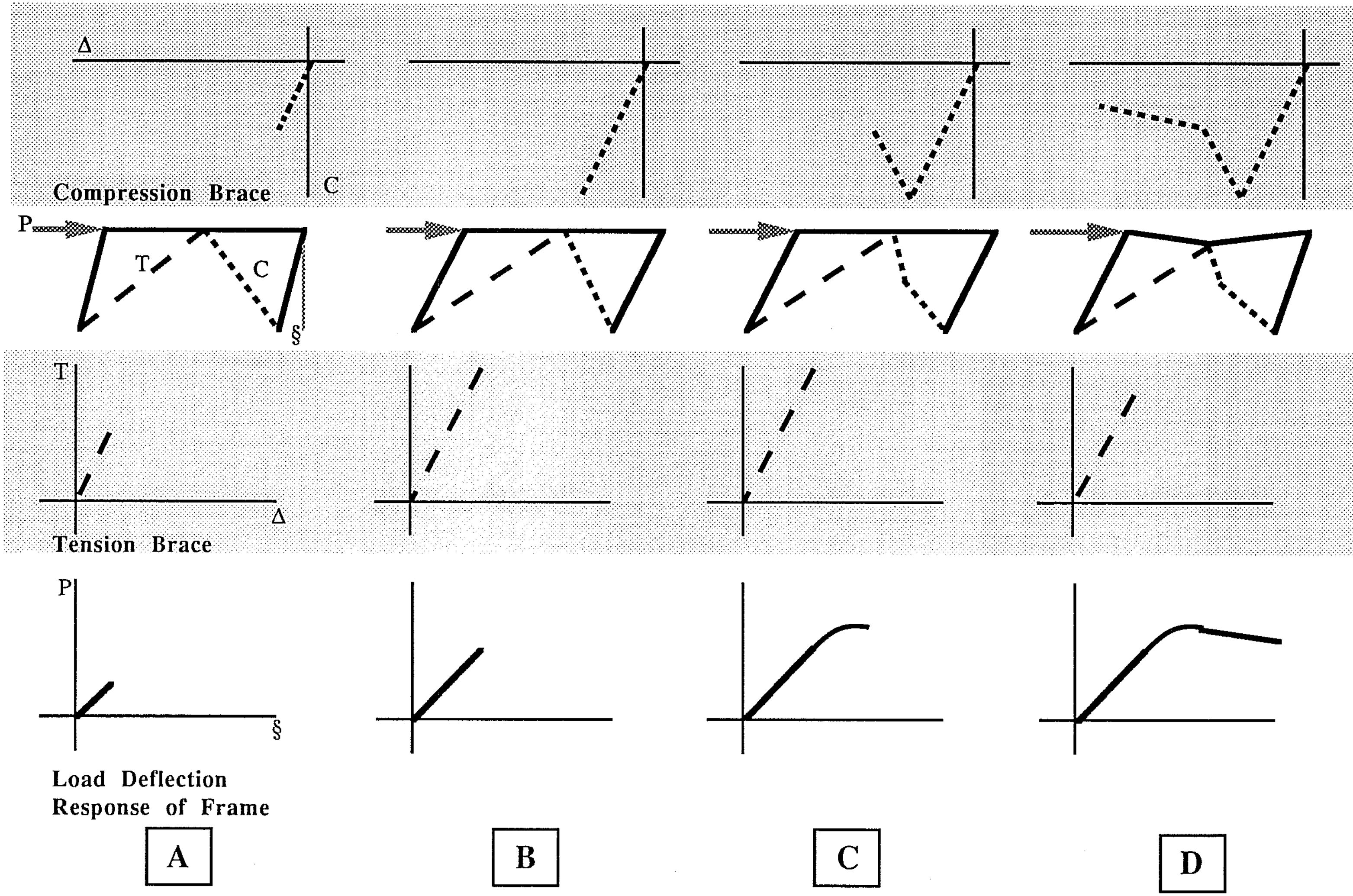


Figure 5.15 Post-Buckling Behavior of Chevron Braced Frame

tension and compression braces still carry equal loads and the beam moment is zero. In step C the compression brace is observed to have buckled reducing its strength dramatically and disrupting equilibrium at the junction of the braces. Any imbalance between the tension brace force and the compression brace force after buckling must be resisted by the beam. After buckling, the large vertical component of the tension brace must be resisted mainly by the flexural stiffness and capacity of the beam. As the lateral load is increased the post-buckling capacity of the compression brace reduces and the moment induced in the beam from the tension brace can cause hinging in the beam (step D). When this happens the load carried by the tension brace remains constant or may be reduced. The result is a decrease in lateral stiffness of the entire frame with only the moment carried at the column bases resisting lateral load and preventing collapse.

It is apparent that when a brace buckles, and the lateral load continues to increase, the beam above the braces will be pulled downward and may hinge. This behavior was observed in the DRAIN-2D analysis of all three structures when subject to the Corralitos record. Figure 5.16 (right axis) shows the vertical deflection of the beam at the junction of the top braces of the original structure subject to the Corralitos record. Also shown in Figure 5.16 (left axis) is the force in the braces of the roof level. The large spikes in the vertical deflection of node A correspond to a buckling of either brace. This figure also shows that plastic rotation and extension of the beams and braces can result in a permanent vertical deflection of the beam. This deficiency in the behavior of the chevron braced system can be avoided by either increasing the

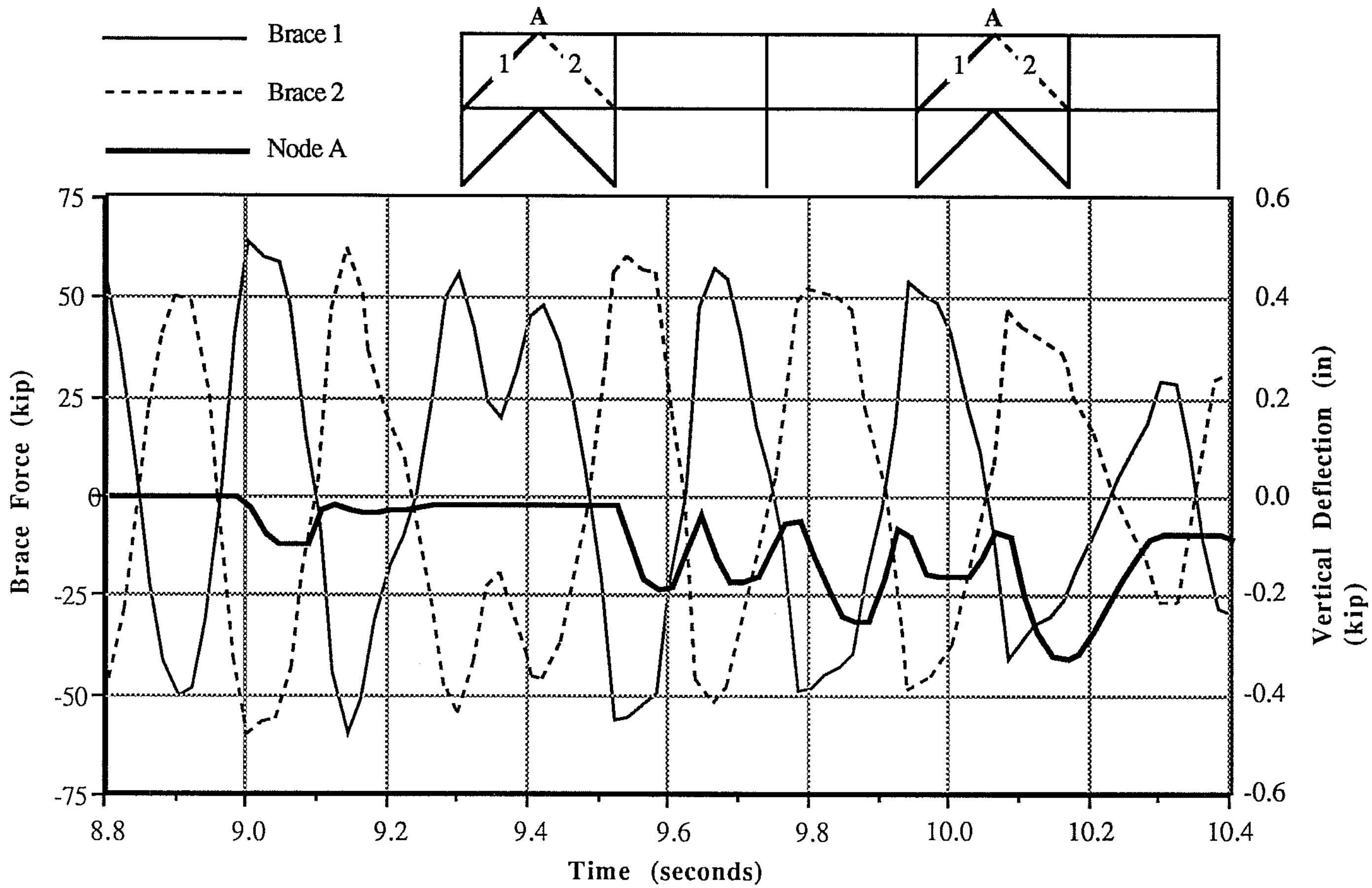


Figure 5.16 Forces in Second Level Braces and Vertical Deflection at Junction of Braces of Original Structure under Corralitos Record

capacity of the braces or by extending a column from the junction of the chevron brace to the foundation.

5.5.3 UBC Provisions and Brace Member Selection

In selecting brace members for redesign I and redesign II it became apparent that particularly for members with long unbraced lengths and small loads, other UBC and AISC requirements besides required strength control the design. In particular the limitation on member slenderness ratio and requirements on preventing local buckling increase the size of the selected section. In all cases the compression capacity of the member was less than the tensile capacity and controlled the member selection. According to UBC-91 and AISC (ASD/LRFD), a pure compression chevron brace member is required to meet several requirements:

(1) sufficient strength to resist force due to 1.5 times UBC lateral loads,

(2) L/r less than 106 for 46ksi steel, OR

(3) the selected brace must resist (3Rw/8) the forces due to UBC loads.

Also, (4) the selected section must have a b/t less than 16 for 46ksi steel

Figure 5.17 shows the compression force in the first and second level braces that result from the UBC-91 and SLS prescribed lateral loads for this structure (Section 5.3.2.2/3). This is the required strength according to the UBC-91 and SLS lateral loads. The total height of the bar charts in Figure 5.17 is representative of the true capacity of the selected member, and the magnitude by which the true capacity exceeds the required strength. The UBC or AISC requirement that controlled the selection of the particular member is indicated by the pattern on the bar chart and the numbers in the legend refer to the above list of design requirements.

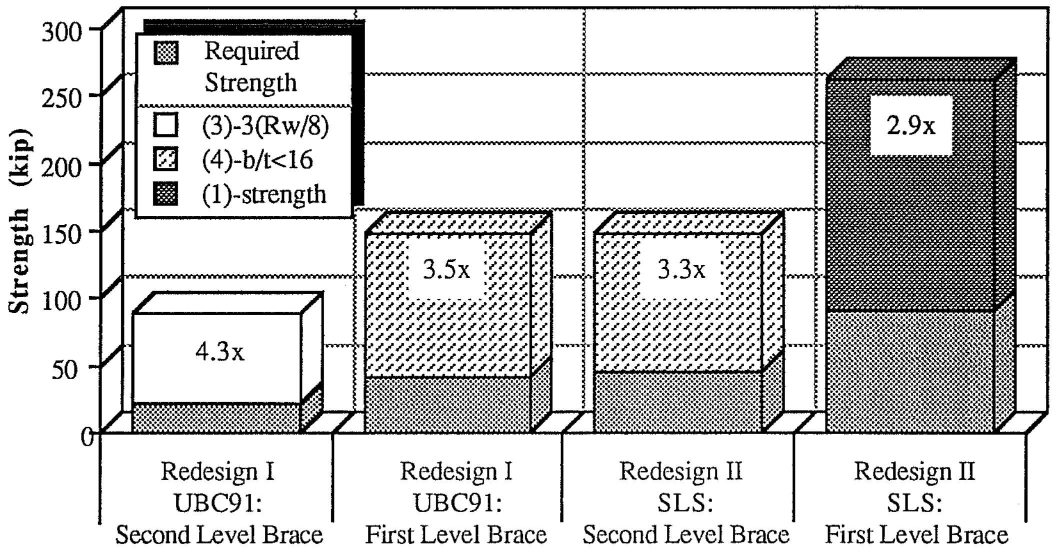


Figure 5.17 Required Strength and True Capacity of Brace Members of Redesign I and II

It can be concluded from this figure that although a minimum lateral design strength is prescribed by the UBC, the final member selection is often controlled by other requirements that result in a member with a capacity well above the required design strength. This figure also shows that ~~the~~ the lower the required strength of the section the larger the increase in the strength of the selected member. It is important to note that these results were observed for the low-rise structure of the case study, and cannot be generalized to all sizes and types of structures.

5.6 CONCLUSIONS

This case study has shown how the UBC minimum design base shear force may not be sufficient to prevent structural damage in the high-tech industrial structure during moderate earthquakes. According to the inelastic analysis, structural damage could not be avoided for any of the ground motion records in the structure redesigned according to UBC-91 lateral forces and provisions.

More widespread structural damage in high-tech industrial structures during earthquakes such as Loma Prieta has been avoided in the past for several reason. First, the actual lateral strength of a structure is typically two or three times greater than the minimum base shear it was designed to resist. This is in keeping with the discussion in Section 4.4.6 concerning the difference between the design base shear and the true lateral capacity of a structure. In the case study it was calculated that the minimum design base shear according to the UBC-91 was approximately 14% of the structure mass. Once the UBC and AISC safety factors and other detailing requirements are considered the true lateral capacity of the structure is approximately 45% the mass of the structure. However, as shown in Figure 5.6 a building with a lateral strength of 45% of the structure mass is only likely to experience structural damage under a ground motion with a PGA greater than 0.18g. The earthquake records used in the inelastic analysis all had PGA higher than 0.18g (Table 5.11) and as expected redesign I suffered structural damage under all records.

Other reasons that there has not been more widespread damage to low-rise ductile structures in the past are: soil-structure interaction may reduce the intensity (and increase the damping) of the ground motion experienced by the structure, the actual structure mass may be less than that assumed in the design, and the direction of the seismic waves may influence the response of the structure.

In the case study, better results were obtained for the structure redesigned for SLS lateral loads that are calculated independently of the structural system and ductility. In this case were the SLS design base shear was 30% of the structure mass, the actual lateral capacity of redesign II is shown to be closer to 90% the mass of the structure. According to Figure 5.6 redesign II should only experience damage under earthquake records with a PGA greater than 0.35g. This was verified in the case study were no damage was observed for two records, and slight damage in a third record, with PGA levels of between 0.27g and 0.36g. However, increased strength and lower story drifts came at the expense of increased peak floor accelerations.

It can be concluded from the case study that if avoiding structural damage is a priority and equipment is uncoupled from the structure (to avoid large accelerations) then the SLS recommendations can achieve this objective. The increased cost to provide the additional strength to meet the SLS recommended lateral forces needs to be weighed against the risk of incurring structural damage and total cost of repair of such damage.

CHAPTER VI
SUMMARY AND CONCLUSIONS

6.1 OUTLINE OF THESIS

A brief description of typical high-tech industrial structures is presented. A database was created containing information on the performance of over two hundred high-tech industrial buildings that were subjected to ground motion resulting from the October 17th, 1989 Loma Prieta earthquake. The data was collected to examine the damage to buildings and their contents, the reconstruction efforts required to resume or restore operations and the associated economic losses. The data was analyzed and the results presented.

A review of current U.S. seismic code provisions, particularly related to structural design philosophy and criteria, is presented. The code philosophy and seismic provisions of a Japanese building code are presented and compared to that of the U.S. codes. A brief description of U.S. design provisions for non-structural elements is also presented.

An analytical study is conducted in which the base shear force on low-rise structures resulting from different levels of earthquake motion is calculated. These base shear forces are compared to the code prescribed forces. The ability of the U.S. seismic codes to prevent damage in moderate levels of earthquake motion is discussed.

A case study of a typical existing high-tech industrial building is presented. The study examines the response of the existing structure to ground motion records recorded during the Loma Prieta earthquake. The analytical response and damage is compared to the actual damage observed in the structure during the Loma Prieta earthquake. An inelastic analysis of the same structure redesigned according to current code provisions, and the same structure redesigned from recommendations to prevent structural damage, is also conducted. The acceleration and drift response of the original and the two redesigned structures are compared. Also, structural damage, drift and acceleration levels of the three designs, under a moderate earthquake, is discussed.

6.2 SUMMARY OF RESULTS

6.2.1 General

The desire to limit damage to structures during an earthquake is not a problem unique to the high-tech industry. However, the impact of damage and disruption of operations in a high-tech industrial building can greatly influence not only the corporate building owner, but society at large. By making data on earthquake damage and associated economic losses available to engineers, potential problems can be identified and avoided in future events. Unless the corporate owners are willing to divulge this information it will not be possible to define quantitatively the emphasis that should be given to maintenance of operations when designing or retrofitting a structure.

It is also important for the high-tech industries to determine acceptable performance standards for their own structures and contents. Too often the structural adequacy of a building is not considered by a corporation leasing property. Data on the capacity of sensitive equipment (computers, manufacturing equipment etc.) to withstand dynamic loading is also lacking in the public domain. Without the owners of a structure specifying acceptable performance criteria (acceptable risk against damage, acceleration or drift limits) for their own needs, they run the risk of potentially crippling damage even in moderate earthquakes.

The desire to limit damage is not unique to the high-tech industries. Banking and finance, manufacturing, research centers all stand to lose by a disruption of operations at their facilities. Perhaps it is up to the insurance industry to offer incentives to those companies with proven seismic preparedness programs and structures that offer adequate protection against disruption of operations and structural damage. While life safety is imperative, the cost to insurance companies and the federal government due to physical damage alone was estimated at over $6 billion dollars [9,34] for Loma Prieta. That does not include secondary costs due to disruption of operations and layoffs in many industries. Prevention is truly better than cure, companies taking measures now can perhaps minimize future disruptions and damage.

6.2.2 Performance of High-Tech Industrial Structures During Loma Prieta

Data collected on over two hundred buildings indicated that the majority of high-tech industries in northern California are housed in low-rise tilt-up (49%) and steel (39%) structures. Many of the tilt-up structures were constructed prior to major 1973 code revisions which resulted from deficiencies in the tilt-up system exposed in the 1971 San Fernando earthquake. Structural damage was observed in relatively few structures (4% of those surveyed) but nonstructural and content damage was more prevalent (20%). Where data was provided on the economic cost of loss of productivity at a facility, it proved to be over three times as costly as the total repair cost of the structure. This emphasizes the importance of preventing disruption of operations in many high-tech industrial buildings. Of equal importance was the fact that in those cases where structural damage was observed the economic impact was immense. For this reason, current codes must indicate the importance of preventing structural damage in all but the most severe earthquakes. Finally the data showed that recently constructed facilities were just as likely to suffer damage and serviceability problems as older structures. Recent code revisions have improved life safety, but have not addressed serviceability issues.

6.2.3 Current Building Code Provisions

The minimum level of design lateral forces prescribed by the U.S. codes is lower than those prescribed by a Japanese building code (BSL). The minimum design lateral force prescribed by the U.S. codes is dependent on the ductility of the structure being designed. However, ductility is only a factor in the response of a structure once structural damage has occurred. Therefore, current US seismic codes implicitly accept structural damage in the determination of minimum design strength. This design approach is adequate for ensuring life safety but suggests that a structures capacity is dependant on what structural system is used. US codes may therefore not adequately address service limit state objectives such as limiting damage in moderate earthquakes. The BSL minimum design lateral load is independent of the structural system and based on preventing damage to all structures in a specific size earthquake.

6.2.4 Service Limit State Design for High-Tech Industrial Structures in the U.S.

An analytical study indicates that the minimum lateral force prescribed by the UBC-91 code is significantly less than the base shear force expected from a moderate earthquake. Results from a previous study suggest that a typical high-tech industrial structure, on the west coast of the US, will be subject to lateral forces larger than the lateral force for which the structure was designed more than once during the life of the structure. Recommendations are made on the minimum design lateral force that should be resisted by a high-tech industrial structure, to avoid structural damage from specific levels of ground motion and to meet acceptable levels of risk. When continuity of operations is essential, then the minimum design base shear should be a function of the expected level of ground motion at a site and not the type of structural system used in the building.

6.2.5 Case Study of a Typical High-Tech Industrial Structure

A case study of a typical high-tech industrial building located on the west coast of the US was performed.

6.2.5.1 *Original structure*

A typical office building used by the high-tech industry was studied. The two story, 24,600 sq-ft structure was built in 1973, according to UBC-70 code provisions. The lateral load resisting system of the structure consisted of tilt-up precast concrete walls in the transverse direction and a chevron braced frame of double angles in the longitudinal direction. Steel tube columns and W-section beams form the gravity load resisting system, and were not considered as part of the lateral load resisting system. Lateral forces were transmitted to the braced frame and precast walls through the concrete floor and roof diaphragm.

6.2.5.2 *Redesign I*

A study was also conducted on the original structure redesigned to resist UBC-91 minimum lateral forces and to meet all other UBC-91 requirements. The precast walls were not altered but the higher UBC-91 design lateral forces resulted in stronger braces being required for the top level but slightly weaker braces required on the lower level. This redesigned structure was more flexible than the original structure as the redesigned tube braces were significantly lighter than the double angle braces of the original structure.

6.2.5.3 *Redesign II*

A third study was conducted on the original structure redesigned to resist lateral loads based on preventing damage in a twenty five year earthquake. The more efficient tube braces of this redesigned structure were stronger than both the original and redesign I. The braces are also lighter than those of the original structure producing a structure more flexible than the original structure, but less flexible than that of redesign I. Redesign II is therefore stronger, but less stiff, than the original design, proving that an increase in strength does not necessarily result in an increase in stiffness.

6.2.5.4 *Elastic analysis*

The concrete precast panels in the transverse direction met all current UBC code requirements for strength and drift. In the longitudinal direction the top braces of the original structure exceeded the current UBC slenderness limit but satisfied all other requirements including strength. Elastic drift ratios resulting from UBC-91 equivalent static lateral loads were all well below the allowable values suggested by the UBC.

6.2.5.5 *Inelastic analysis*

All three structures subject to the Corralitos earthquake record (PGA=0.63g) exhibited extensive damage. Analyses indicated that braces buckled and hinges were formed in the roof or second floor beams. Under a major ground motion of this kind significant structural and nonstructural damage is to be expected. Analyses also show that the Veterans Hospital record (PGA=0.38g), the Oakland record (PGA=0.27g) and the El Centro record (PGA=0.36g), would also likely produce significant damage in the El Centro record (PGA=0.36g), would also likely produce significant damage in the original structure and redesign I. Analysis shows that for redesign II, no structural damage produced from the VA record and the Oakland record, and only slight damage resulted from the El Centro record.

The inelastic drift ratios observed in the analyses were all below 2%. The largest floor drift ratios occurred as a result of the strong Corralitos record and were 1.7% for second floor of Redesign I and 1.4% for the top floor of the original structure. A consequence of the increased strength of redesign II was large amplification of ground acceleration at the second and roof level. Analyses shows that the largest amplification occurred in redesign II under the El Centro record. The levels of acceleration predicted for all three designs, but particularly for redesign II, suggest that nonstructural and equipment damage is to be expected as a result of all these ground motion records. Buckled braces and plastic hinging reduced the accelerations (but increased the drift) on floors above the damage.

The damage predicted from the analyses closely matches the damage observed in the original structure during Loma Prieta. It is apparent from the inelastic analysis that even under current US codes structural damage and disruption of operations would have occurred. If these earthquake records are representative of the typical ground motion likely to be experienced by high-tech industrial buildings on the west coast, then it is apparent that the current code approach may not be adequate. More widespread damage was fortunately not observed in Loma Prieta due to large safety factors, material factors and other requirements that increase the actual strength of a structure by more than 200% over the code prescribed minimum level. Also, soil-structure interaction may increase damping and the bi-directional nature of seismic waves could all reduce the effective seismic forces experienced by a structure.

6.3 CONCLUSIONS

Data from Loma Prieta suggests that approximately 20% of high-tech industrial structures experienced structural and/or nonstructural damage. The total cost of nonstructural damage and loss of productivity greatly exceeded the cost of structural damage during Loma Prieta; however, when structural damage did occur it resulted in large direct and indirect economic losses for the corporations involved. The economic losses associated with disruption of operations, loss of productivity and structural repair procedures could cripple a corporation. Therefore, if avoiding structural damage and disruption to operations is to be a priority, building owners should establish acceptable performance standards for their buildings and equipment within the structures. The increased cost necessary to provide the additional strength to meet higher performance standards needs to be weighed against the risk of structural and nonstructural damage occurring, and the total economic impact it could have on the business. Care should be taken to determine the impact on drift and floor accelerations that result from strengthening a structure.

Current US seismic codes implicitly accept structural damage in the determination of minimum design base shear. By using the ductility available in a structure as a variable in the minimum base shear calculation, the codes are assuming that inelastic action will occur during an earthquake. It seems obvious that if structural damage is to be avoided in a moderate earthquake, then the minimum design base shear should be a function of the 'size' of the earthquake, and independent of the ductility available in a structural system.

Current codes address life safety issues directly, but in the case of the high-tech industrial structure they may need to focus more attention on serviceability issues. More widespread structural damage to high-tech industrial structures was avoided during Loma Prieta mainly because of low levels of ground motion and due to over-strength in structures designed to current codes. However, relatively wide-spread nonstructural damage was observed. Low-rise structures will continue to be exposed to lateral forces larger than that specified by the base shear calculations of current codes. A two-phase design approach (addressing serviceability and life safety independently) would be most appropriate for the buildings of the high-tech industry. However, until such time that a two-phase design is introduced, engineers should become more aware of the contents and operations within a high-tech industrial structure when designing or retrofitting the building.

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