EARTHQUAKE DAMAGE TO HIGH-TECH INDUSTRIAL FACILITIES
A CASE STUDY

APPROVED BY:

Dr. James O. Jirsa, Supervisor

Dr. Michael D. Engelhardt

A mi padre

EARTHQUAKE DAMAGE TO HIGH-TECH INDUSTRIAL FACILITIES
A CASE STUDY

by

MARIANELA PICADO, Ingeniero Civil

THESIS

Presented to the Faculty of the Graduate School of
The University of Texas at Austin
in Partial Fulfillment
of the Requirements
for the Degree of

MASTER OF SCIENCE IN ENGINEERING

THE UNIVERSITY OF TEXAS AT AUSTIN
December, 1991

ACKNOWLEDGEMENT

The report presented herein is part of a project funded by the National Science Foundation under Grant BCS 9011137. The support of this organization is appreciated.

I would like to express my sincere thanks to Dr. James O. Jirsa, supervisor of this thesis, for his valuable advice, encouragement and support. I consider myself fortunate to have been able to work with him and benefit from his knowledge.

I would like to thank Dr. Michael D. Engelhardt for serving as a second reader for this thesis. His advice and always helpful disposition are deeply appreciated.

The contribution of Ms. Maryann T. Phipps (H.J. Degenkolb Associates) is gratefully acknowledged. Her observations and suggestions to improve this thesis are appreciated.

The assistance of Raoul Karp during the analytical part of this project is greatly appreciated. I would also like to thank Jose Pincheira for his advice and friendly disposition. Special thanks to Santiago Lopez for all his help in the preparation of this report.

I thank all my friends in Austin for their support. Their friendship is invaluable and will always be remembered, especially Virginia Andrade, Santiago Lopez and Elias Saqaan.

I deeply thank Sergio Alcocer for his love and encouragement.

I profoundly thank my father Romulo for his continuous support and love throughout my life. He has always encouraged me to be a better professional and human being, and has given me all the opportunities to improve myself. The support of Rocio and my sisters is greatly appreciated. I especially thank my aunt Maria Eugenia for all she has done for me. The support of the rest of my family is gratefully appreciated.

Marianela Picado

Austin, Texas
October, 1991

EARTHQUAKE DAMAGE TO HIGH-TECH INDUSTRIAL FACILITIES
A CASE STUDY

Marianela Picado, M.Sc.
The University of Texas at Austin, 1991

Supervising Professor: Dr. James O. Jirsa

High technology industrial facilities play an important role in the nation's economy and development. During past earthquakes, these facilities have suffered damages, particularly to nonstructural elements and equipment. This damage has caused interruption of operations and large economic losses in the past. Due to the vital role of industrial facilities, minimizing nonstructural and equipment damage as a result of ground motions is becoming increasingly important.

A program was implemented at The University of Texas to assess the behavior of industrial facilities during recent earthquakes. Current analysis, design and installation techniques for nonstructural elements and equipment were studied. Summaries of damage to industrial facilities are presented, A review of current code provisions and other methods of analysis and design for nonstructural elements are given. Use of current building code design provisions are suggested for elements for which minimal and intermediate seismic protection is required. Dynamic analyses are suggested for elements with maximum seismic protection requirements. Vulnerable nonstructural elements

and equipment are identified. Suggestions for analysis, design and installation techniques for improved seismic performance of these elements are given.

A typical industrial facility was analyzed to study its performance under forces computed using current building code provisions, and under measured ground motions. Elastic analyses of this structure were performed using lateral forces specified by UBC Code 1991 and NEHRP Provisions. The computed displacements and drifts meet the requirements of both provisions. The performance of the structure under the UBC Code is generally acceptable as expected. Under the NEHRP Provisions, the bracing elements in the structure do not meet the strength requirements. Inelastic analyses of the existing structure under measured ground motions show extensive inelastic action and large story drifts, indicative of a strong possibility of nonstructural damage. A strengthening scheme is proposed to improve the performance of the building.

Strengthening consists of adding braces to the existing structure. This scheme effectively reduces both inelastic action in the existing structure and floor displacements and drifts, minimizing the possibility of nonstructural damage. Computed floor velocities and accelerations for both existing and strengthened structure are presented.

Design loads for nonstructural elements are computed using current building code provisions and using the results of the analyses of the building. In general, loads computed using current code provisions are lower than the ones predicted by the analyses of the existing and strengthened structure.

TABLE OF CONTENTS

LIST OF TABLES

Table | Page

LIST OF FIGURES

CHAPTER I
INTRODUCTION

1.1 GENERAL

High-technology industries have a vital role in the nation's defense, banking and finance, electronics research and development, and manufacturing sectors. Loss of productivity in these sectors could have serious impact on the nation's economy and development. Recent earthquakes have shown the vulnerability of existing industrial facilities to moderate and strong earthquakes. The 1989 Loma Prieta earthquake resulted in relatively little structural damage to industrial facilities but high damage estimates due to loss of work hours, productivity and occupancy in the "high-tech" area were reported.

Most of the earthquake related economic losses in the high-tech industry have been product of damage to nonstructural elements. Failure of suspended ceilings, piping, sprinkler systems, and other architectural and/or mechanical/electrical systems has caused entire facilities to shut down, with the consequent loss in production. Because of the importance of these facilities, a new concern has risen. Analysis, design and installation of nonstructural elements to withstand moderate earthquakes without damage that could affect continuity of production in these facilities is becoming increasingly important. The industrial sector is beginning to realize the necessity of having seismic protection programs for its facilities. Design procedures and installation guides are being developed by different companies for their own use. However, most of the available information on earthquake related damage and seismic protection techniques is privately held. It is important to gather and analyze this data, and make it available to the earthquake engineering profession. Also, research on performance of typical industrial buildings, particularly performance of nonstructural and electrical/mechanical systems, as well as adequacy of current building code provisions for analysis and design of such systems is needed to minimize the impact of future seismic events in the high-tech industry.

1.2 OBJECTIVES

This study is part of a research project to study damage to existing industrial buildings implemented after the 1989 Loma Prieta California earthquake, and sponsored by the National Science Foundation.

The objectives of this study are to gather information on performance of industrial buildings during recent earthquakes focusing on performance of nonstructural elements; and to examine current code provisions and design and installation procedures for typical nonstructural elements and equipment in high-tech industrial facilities. Suggestions will be formulated regarding seismic protection techniques for some of the nonstructural elements that are most likely to be damaged during earthquakes, and whose failure could result in interruption of operations in the facility.

A main purpose of this study is to examine the behavior of an existing industrial building representative of typical industrial construction. Response of this building under measured ground motions will be studied to determine possibility of nonstructural and equipment damage. Results will be used to determine the necessity of retrofitting to minimize nonstructural damage. Forces on nonstructural and mechanical/electrical elements will be compared with those proposed in different building codes.

The results of this study will be used later to define quantitatively the emphasis that should be given to maintenance of operations when designing or retrofitting a structure housing high technology industries.

1.3 ORGANIZATION

The study is divided into six chapters. Chapter II describes structural systems and architectural layouts of typical high-tech industrial buildings. Information on behavior and damage to industrial buildings and their components during past earthquakes is included. Chapter III includes a summary of provisions regarding analysis and design of nonstructural and electrical/mechanical systems given by different current building codes. Suggestions for analysis and design of these systems are given. In Chapter IV, suggestions on seismic protection techniques for typical nonstructural elements encountered in high-tech industrial buildings are given. Suggestions include analysis, design and installation procedures. Chapter V includes a case study of a typical industrial building. Performance of the building under lateral forces suggested by different building codes and under measured ground motions is studied. A retrofitting scheme proposed to improve the behavior of the building under seismic motion is presented. Design forces for nonstructural elements computed using the results of the study, and comparison with the forces suggested by different building code provisions are also presented in this Chapter. Chapter VI contains a summary of the study and the results, and the conclusions.

CHAPTER II
HIGH TECHNOLOGY INDUSTRIAL FACILITIES

2.1 INTRODUCTION

The operation of an industrial facility is dependent on the proper functioning of a number of systems, including: industry related equipment, building structure and other supporting structures, nonstructural elements, support services, site and building access, and supporting utility systems. Throughout the years, much attention has been paid to designing structures to provide life safety in major earthquakes and to minimize structural damage in moderate earthquakes. After more recent earthquakes, a new concern has risen. Damage to nonstructural building elements and equipment resulted in significant economic impacts. After the Loma Prieta 1989 earthquake, some corporations reported millions of dollars in property damage and lost work hours, mainly due to failure of nonstructural elements. Damage to gypsum board partitions, glazing, air conditioning units, etc., may not represent a major loss in a particular building but when considered area-wide, the loss becomes significant.

In this chapter, an overview of the typical structural and architectural systems used in high technology facilities are presented. A brief description of the equipment and support utilities found in this group of buildings is also included. A summary of the earthquake related damage suffered by the industry in recent years is presented.

2.2 DESCRIPTION OF HIGH-TECH FACILITIES

The term "high-tech" is usually related to those industrial facilities that manufacture and distribute electronics-based products such as computers and data processing equipment, software, advanced medical equipment, and electronic equipment to be used in other areas. Often, high-tech industries either use or produce highly sensitive and expensive equipment that could be easily damaged during an earthquake if proper precautions are not taken. A description of typical structural and architectural systems used to house high-tech facilities, as well as nonstructural elements and equipment normally encountered in this type of facility is presented next.

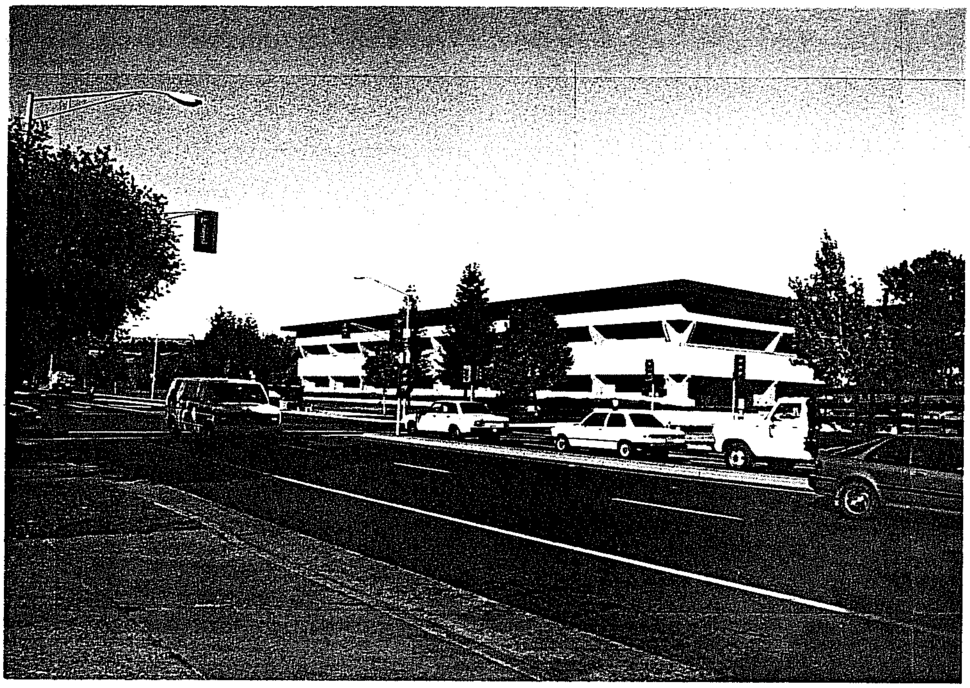


Figure 2.1 Typical Industrial Building in the Silicon Valley Area, California

2.2.1 Structural and architectural systems

Industries are housed in buildings that vary in date of construction and structural type. For example, in the Silicon Valley area in California, industrial buildings date from the late 1940's to the present, most of them were constructed since the 1960's (Figure 2.1). The structural type and space distribution vary according to the use of the building. Many of the structures are of tilt-up construction. Other common types include moment resisting frames, steel braced frames and concrete shear wall systems. Light manufacturing and data processing industries can usually be set up within any conventional building.

2.2.1.1 Architectural layout

Production areas include conventional machine shops, as well as workbenches assembly lines where products are assembled and tested. Portions of the work area are also destined for storage, computer installations, and offices. Due to the necessity of continuously changing the disposition of the work areas, or even the use of the building, most of the industrial facilities have a large open area so that equipment or partitions can be moved around easily (Figure 2.2). Large one or two story structures house the main production or data processing facility, and smaller structures are built around them to house support facilities. Normally, all the buildings in the same industrial complex have either similar or the same structural system and architectural treatment. A common practice among large corporate owners is to use typical or "standard" buildings for primary facilities: production, storage, offices, etc. The standard building is then constructed in different geographic areas. Another common practice is to lease existing buildings. The final decision to lease is rarely based on structural aspects. Location and leasing costs weigh heavily in the decision making process. As a result, high-tech industries are sometimes housed in buildings whose structural system may not perform adequately under earthquake loads.

2.2.1.2 Structural systems

A common type of structural system used in industrial buildings is the moment resisting steel or reinforced concrete frame structure. These are usually flexible systems which dissipate energy by deforming in flexure. When the frame structure is designed properly, it is capable of remaining stable when stressed beyond yield stress level. However, the nonstructural components and systems

Figure 2.2 Typical Computer Installation within Industrial Facilities (Ref. 22)

must be designed to accommodate the expected frame deformations. Such accommodations include providing connection and support details so that the structural frame can deflect without damaging exterior walls, windows or interior partitions. Similarly, provisions to accommodate building deformations must be provided in the design of electrical and mechanical systems. These provisions can add to construction costs. Nevertheless, a moment space frame can provide large open areas and may be the best solution based on building function and use.

Another type of lateral force resisting system used in industrial buildings is the shear (or structural) wall. Shear wall structures are quite rigid and deflect less than a comparable moment resisting frame. Since shear walls deform considerably less, savings on connections of exterior walls, windows and interior partitions to the structural system can result. However, the walls may be less versatile with respect to availability of open spaces, depending on their location. Usually the walls are placed along the exterior sides of the structure, leaving the interior portion of the building open. In these cases, the diaphragm at roof and floor levels has to be properly designed to transfer forces to the walls. In a similar manner, structural framing systems are sometimes stiffened by using cross bracing, or eccentric bracing to reduce the deformations in the building. Damage to nonstructural components is less likely.

Another variation of the structural type mentioned before is tilt-up construction. Tilt-up precast concrete walls are used as either bearing walls to support vertical loads, or as lateral force resisting walls capable of carrying forces induced by seismic motions. Tilt-up panels may be combined with moment resisting frames or braced frames to form the lateral load resisting system of the building. Tilt-up construction has proved to have adequate performance under seismic motion, as long as the connections between wall panels, and between wall panels and floor and roof diaphragms, are designed properly. The design of the diaphragm itself could be critical in this type of structure.

2.2.2 Nonstructural elements and equipment

Nonstructural elements include all the architectural components found in a building system (e.g. cladding, ceilings, partitions, doors/windows, stairs, furnishings and equipment, etc.) in addition to all mechanical, electrical and plumbing components (e.g. elevators, lights, piping, ducts, HVAC systems, security systems, computer equipment, etc.). Common nonstructural elements and equipment present in high-tech facilities are utility systems for ventilation, heating and power supply, which are often housed in attics or rooftop penthouses. Suspended ceilings and light fixtures, sprinkler lines, rod-hung piping, machines and other equipment related to production or data processing systems. Partitions, work stations, files and cabinets normally are not permanently attached to the floor so that changes in space distribution can be made easily.

Nearly identical nonstructural elements may be used for a variety of purposes, each with different relative importance to maintaining function of the facility. Typical equipment and nonstructural elements encountered in data processing facilities are presented in Table 2.1 (Ref. 11). These elements are representative of those present in any kind of high-technology industrial building. Relationships between the elements and their use, operational importance and seismic vulnerability based on past earthquake performance are illustrated in the table. The elements are related to end use categories as follows (from Ref. 11):

a. Data processing power: elements associated with delivering electrical power to the industrial facility.

b. Air conditioning: elements associated with delivering appropriate amount and quality of air conditioning.

c. Communication support: all elements required to support the communication function of the facility. In many instances, this equipment is located in raised floors.

d. Environmental enclosure: all elements that physically make up the facility space, other than access floor and computer equipment.

e. Environmental security: elements related to protecting the space from hazards other than earthquake, such as physical tampering or fire.

f. Environmental lighting and power: elements associated with providing lighting and power other than that necessary for the function of the facility equipment.

g. Environmental heating, ventilating and air conditioning (HVAC).

The seismic vulnerability classification was based on past earthquake performance and is related to the possibility of damage to the elements if installed without seismic protection.

USE CATEGORIES									
ENVIRONMENTAL HVAC	ENVIRONMENTAL POWER & LIGHTING	ENVIRONMENTAL SECURITY	ENVIRONMENTAL ENCLOSURE	DP COMMUNICATION SUPPORT	DP AIR CONDITIONING	DP POWER	VULNERABILITY (3)	ELEMENT/SYSTEM	ELEMENT TYPE
□	□			■	■	■	M	SUBSTATION	ELECTRICAL
□	□			■	■	■	M	SWITCHGEAR	
						■	H	MOTOR GENERATOR	
						■	M–H	UPS EQUIPMENT	
□	□			■	■	■	M	TRANSFORMERS	
						■	H	POWER DISTRIBUTION UNITS (1)	
□	□			□	■	■	H	ENGINE GENERATOR (2)	
□	□			■	■	■	L	DISTRIBUTION PANELS	
□					■		H	MOTOR CONTROL CENTERS	
□	□			■	■	■	L	BUS DUCT. CONDUIT. ETC	
□					■		M	BOILERS (2)	HEATING
□					■		H	HOT WATER HEATERS, TANKS	
□					■		L	PUMPS	
□					■		H	TANKS	
□					■		M	MISC. CONTROL CABINETS	
□					□		L–M	EMERGENCY FUEL STORAGE (2)	
□					■		L–M	ASSOCIATED PIPING	
□					■		H	CHILLERS	COOLING
□					■		H	CONDENSERS	
□					■		M	HEAT EXCHANGERS	
□					■		H	COOLING TOWER	
□					■		L	PUMPS	
□					■		M	CONTROL CABINETS	
□					■		L–M	ASSOCIATED PIPING	
□					■		H	CONTROL AIR COMPRESSORS (2)	
□							H	EXHAUST FANS	AIR
□							M	H/V UNITS	
□					■		M	HVAC UNITS	
□					■		H	HVAC UNITS (1)	
□			□				H	GRILLS, DIFFUSERS (OVERHEAD)	
□					□		L	ASSOCIATED DUCTWORK	
□					□		M	FILTERS	
□					□		M	HUMIDIFIERS	
				■			M	SWITCHING EQUIPMENT	COMMUNI CATION
				■			M	MISC. CABINETRY	
				■			L	ANTENNA	
				■			H	BATTERIES	
		■	■				H	MASONRY PARTITIONS ON STRUCT.	ENCLOSURE
		□	□				L	NONMASON PARTITIONS ON STRUCT.	
		□	□				H	NONMASON PARTITIONS (1)	
□	■		■				H	PANELIZED CEILINGS	
□	□		□				L	SOLID CEILINGS	
			□				M	STORAGE RACKS, SHELVING	
			□				H	STORAGE RACKS, SHELVING (1)	
			■				H	ELEVATOR SHAFT	
			■				H	ELEVATOR EQUIP. ROOM	
		□					M	ENTRY CONTROL SYSTEM	SECURITY
		□					H	VIDEO SURVEILLANCE SYSTEM	
		□					L	FIRE PUMPS	
		□					L	SPRINKLER PIPING	
		□					M	LOCAL EXTINGUISHERS	
		□					M	HALON SYSTEM	
	■		■				–	FIXTURES IN PANELIZED CEILING	LIGHT/ POWER
	■		■				–	HUNG FIXTURES – SOLID CEILING	
	□		□				L	OTHER FIXTURES – SOLID CEILING	

■ GREATER OPERATIONAL IMPORTANCE
□ LESSER OPERATIONAL IMPORTANCE

(1) ON COMPUTER ACCESS FLOOR
(2) INCLUDES RELATED EQUIPMENT
(3) DAMAGE LIKELY TO OCCUR IF INSTALLED WITHOUT SEISMIC CONSIDERATIONS
L = LOW
M = MODERATE
H = HIGH

Table 2.1 Nonstructural Building Element Relationships and Operational Importance (Ref. 11)

2.2.3 Support utilities

The operation of high-tech industrial facilities depends not only on the operation of the manufacturing or processing equipment but on all the structural and nonstructural systems mentioned before. Other necessary utility systems are: electrical power, water supply, sanitary and storm sewers, natural gas and communications. Also site and building access are essential.

Electrical power is necessary for operation of computer equipment, lighting, HVAC systems, pumps, fire detection and suppression systems, communications, building security systems, etc. Water supply is essential for the cooling systems, fire sprinklers, sanitary sewers and drinking water. Sanitary sewers are needed for waste disposal and storm sewers are needed for water runoff from storms to prevent or minimize flooding.

Past earthquakes have shown the large extent of damage and resulting economic losses that occur when support facilities do not function. Therefore, it is important that the reliability of support utility systems be evaluated from several aspects: supplier controlled system, usually local utilities or government agencies, building owner controlled portion and industry owner controlled portion. Consideration should be given to providing back-up systems for the most critical support utilities. This is particularly important in high seismic risk areas when only limited outage can be tolerated. Structural flexibility should also be provided in the construction of a utility system to allow relative movement between piping and structure, or between areas of hard soils and softer soils or engineered fills.

It is important to plan or assess the facility for possible access routes and to secure their dependability. Access to the site after an earthquake is vital to enable the employees to go to work, to perform repairs, to deliver materials, and to allow access to ambulances and repair crews.

2.3 SEISMIC PERFORMANCE OF HIGH-TECH FACILITIES

In general, structural performance of high-tech facilities has been adequate during recent moderate earthquakes. Those buildings that suffered damage after recent earthquakes (e.g. Loma Prieta 1989) were either of known hazardous construction, or more modern buildings with construction deficiencies or poor detailing.

Nonstructural and operational performance has not been so satisfactory. Most of the economic losses experienced in the industrial sector were due to failure of nonstructural elements: broken windows, fallen light fixtures and ceiling panels, broken pipes. These failures interrupted operations in the buildings, causing substantial losses over and above those directly related to the repair of damage.

2.3.1 Structural damage

Industrial facilities are sometimes housed in buildings of known hazardous construction types: pre-1972 San Fernando earthquake tilt-up buildings, nonductile concrete moment resisting frame structures and other structural systems lacking an adequate lateral load resisting system (Ref. 20). Many of these buildings located in the Silicon Valley area suffered little or no observable damage during the Loma Prieta earthquake. This was due to the relatively low level of ground acceleration experienced at the site. The potential for greater damage is present unless strengthening measures are taken. An example of this type of construction is a tilt-up system lacking positive connection between roof diaphragm and walls, which weakens the load resisting system and may produce partial or total collapse of the structure.

In modern engineered buildings designed with current codes, damage occurred as a result of construction deficiencies or improper detailing. These appear to have been isolated cases, with the majority of modern buildings performing adequately. Though the Loma Prieta earthquake was of limited magnitude and duration and did not represent a severe test of modern codes, the lack of significant structural damage provides some confidence in the effectiveness of the code provisions to guarantee life safety and to limit structural damage.

2.3.2 Damage to nonstructural elements and equipment

Damage to nonstructural elements is caused in two primary ways:

a. Damage related to differential distortion of the structure: Occurs when the nonstructural element is not able to withstand or adjust to loads caused by the deformations and deflections of the basic structure, and

b. Damage related to shaking of elements: Damage to elements that may respond to the motion of the structure by vibrating internally, sliding, overturning or swinging.

Both types of damage have been observed in nonstructural elements and equipment in industrial buildings in past earthquakes: damage to exterior unreinforced masonry-brick veneer and facade systems, broken windows, damage to suspended ceilings, all caused by a large drift of the structural system (Figure 2.3).

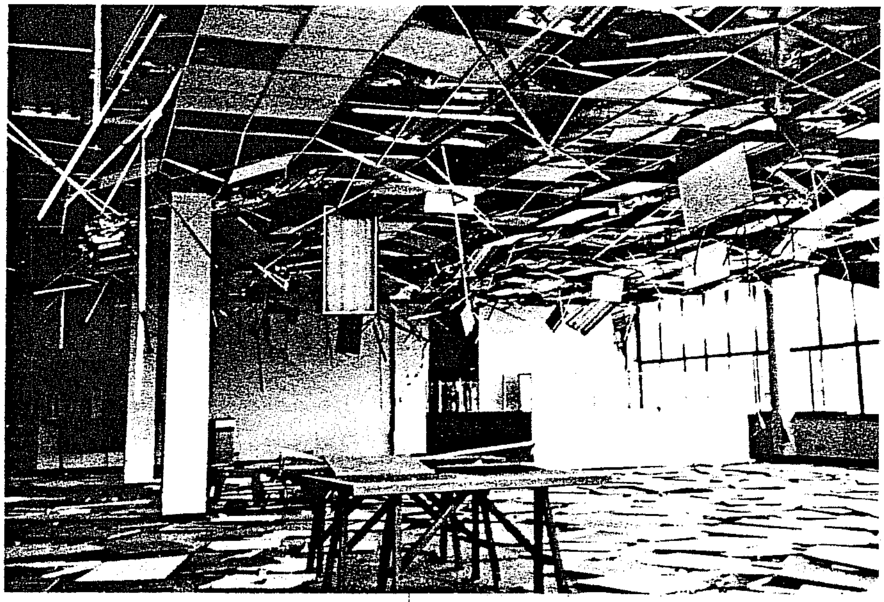


Figure 2.3 Damage to Nonstructural Elements: Collapsed Suspended Ceiling, Lights, and Ventilation Ducts (Ref. 7)

2.3.2.1 Damage to architectural systems and components

A summary of typical damage to architectural systems and components after past earthquakes is shown in Table 2.2 (Ref. 19). Ceiling panels dislodged and fell, chain-suspended light fixtures detached from their chain hooks and fell. Building separation joints have performed as expected, but many of the architectural elements adjacent to the joints have not. Many architectural finish elements such as cladding and interior walls, floors and ceilings have been severely damaged or dislodged due to excessive horizontal deformations and lack of attention to design of details that would allow sufficient movement between the lateral load resisting system and the nonstructural elements.

TABLE 2.2 DAMAGE TO ARCHITECTURAL SYSTEMS AND COMPONENTS DURING PAST EARTHQUAKES (Ref. 19)

SYSTEMS	COMPONENTS	RECORDED DAMAGE
Partitions	Permanent-masonry and tile	Cracking of units; horizontal drift unit losses or compression failures at top of partitions; joint failures; overturning.
	Permanent - stud and gypsum board or plaster	Overturning associated with ceiling failures adjacent to partitions; finish cracking; horizontal drift; delamination of finish from studs.
	Demountable - metal, wood, metal and glass	Separations at top and bottom channel; compression breaks; overturning; cracking or separation of fixed glass from partition body.
Furring	Plaster or gypsum board	Cracks in finish; separation failures from furred structural element due to movement of structure.
Ceilings	Suspended lay-in tile system - exposed splines	Unwinding or breakage of hangers; separation of tiles from suspension system; compression bending of system at room perimeters; breakage at building seismic joints; shear breakage in suspension interconnections.
	Suspended concealed spline systems	Failures similar to exposed splines, except less tiles separate from suspension system.
	Suspended plaster or gypsum board	Plaster spalls from lath; shear cracks in finish; suspension system sustains similar damage to other suspended systems; gypsum board separates from supports.
	Surface-applied tile, plaster, or gypsum board	Generally better performance than suspended systems; plaster cracks and spalls due to structural movement; adhesive failures in ceiling tile.
Light fixtures	Lay-in fluorescent (recessed and semirecessed)	Racking of ceiling suspension causes fixtures to separate from suspension system and fall. Where fixtures are supported separately from ceiling system, performance is better. Failures within fixtures included separation of diffusers, lenses, and lamps from housings.

TABLE 2.2 (cont.)

SYSTEMS	COMPONENTS	RECORDED DAMAGE
Light fixtures	Stem-hung and chain-hung fluorescent	Separation of stem at structural connection point; twisting of fixture causes breakage in stems and chain breakage. Multiple fixture installations, end to end, experience most common damage due to interaction of fixtures with each other. Long-stem fixtures sustain more damage than short-stem. Internal damage similar to lay-in fixtures.
	Surface-mounted fluorescent	Ceiling fixtures perform similarly to lay-in fixtures. Wall fixtures perform better than ceiling fixtures except in instances of wall failure. Internal damage similar to others.
	Stem-hung incandescent	Performance similar to stem-hung fluorescent fixtures except that incandescent fixtures are usually hung with a single flexible stem. Damage due primarily to fixture swaying and encountering other structural and nonstructural components. Internal damage consists of lens and globe separation as well as lamp breakage.
	Surface-mounted incandescent	Ceiling fixtures perform similarly to surface-mounted fluorescent. Wall fixtures perform well.
	Ornamental fixtures	Chandeliers and other fixtures of a similar nature fall similarly to stem-hung fixtures. Internal damage due to multiple moving elements interacting with each other.
Doors and frames	Wood, hollow metal, metal and glass	Frames warp from enclosing wall movement; doors occasionally deform hinges.

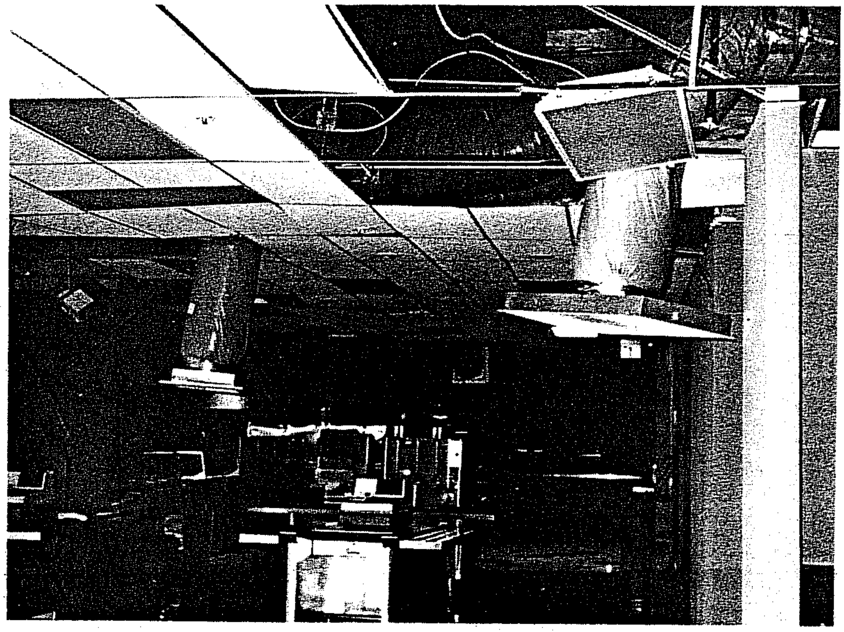


Figure 2.4 Dislodged Lights and Ventilation Diffusers (Ref. 3)

2.3.2.2 Damage to mechanical and electrical systems

A great part of the economic loss in industrial buildings, both in property losses and lost work hours, could be attributed to the failure or complete lack of seismic restraint details in many utility systems. Main pipes of sprinkler systems could not accommodate differential movements between their supports and failed at the connections thereby releasing water into plenum spaces. Heating/ventilation/air conditioning equipment moved from supports causing brief losses of air conditioning (Figure 2.4). Sections of HVAC ducts split due to differential displacements between support points. Other rod-supported piping suffered cracking and failed at the connections due to lack of bracing.

In some extreme cases, entire building floors had to be evacuated as a result of this type of damage, with correspondent loss due to disruption of operations. A summary of typical damage which occurred in mechanical components during past earthquakes is presented in Table 2.3 (Ref. 19).

TABLE 2.3 DAMAGE TO MECHANICAL COMPONENTS DURING PAST EARTHQUAKES (Ref. 19)

SYSTEMS	COMPONENTS	RECORDED DAMAGE
Mechanical	Rigidly mounted equipment such as boilers, chillers, generators, tanks	Generally perform well where there is no damage to structural base. Some shearing of attachment devices and corresponding horizontal displacement; tall tanks overturn; supports fail. Greatest damage is to equipment that rested on structural base without positive anchorage; overturning and horizontal movement sever connected lines and pipes.
	Vibration-isolation-mounted fans, pumps, air handlers, etc.	Devices fail and cause equipment to fail. Some damage due to unrestrained shaking on vibration-isolation device. Suspended equipment fails more often than floor-mounted equipment.
Piping	Water, stream, sprinkler, gas, waste, etc.	Large-diameter rigid piping fails at elbows and bends. Joint separations; hanger failures. Small-diameter piping performs better than larger piping due to bending without breaking. Single failures of hanger assemblies frequently causes progressive overloading and failures at other hangers and piping supports. Piping performs better in vertical runs where there are lateral restraints than in horizontal runs where there is no lateral bracing. failures at building seismic joints due to differential movements.
Ducts	Rectangular, square, and round ducts	Breakage most common at bends. Supporting yokes fail; long runs fail as a result of large-amplitude swaying.

TABLE 2.3 (cont.)

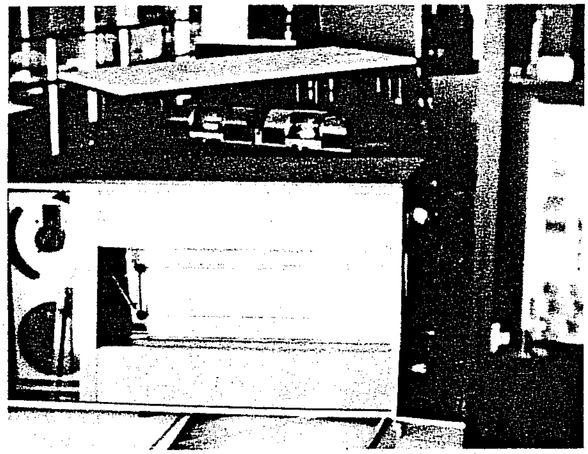
SYSTEMS	COMPONENTS	RECORDED DAMAGE
Elevators	Counterweights, guiderails	Separation of counterweights from rails. Damage from counterweights includes structural beams, cables, and cabs.
	Motor-generator sets	Sheared-off vibration-isolation devices.
	Controller panels	Overturning when unanchored at bases. Hinged panels thrown open.
	Cars-guiding systems	Generally perform well.
	Hoistway doors	Some doors jam or fall outward.
	Hydraulic elevator systems	Generally perform well. Some cylinders shift out of plumb.
Escalators		Generally perform well. Some treads are damaged by falling debris.
Emergency equipment	Generators	Generally perform well when bolted securely to structural bases.
	Communications and lighting equipment	Perform similarly to other electrical equipment. Some battery racks collapse. Unsecured battery-powered emergency lighting falls.
	Exit corridors, doors, lighting	Many exit doors become deformed and jam. Exit corridors are blocked with debris. Exit lights perform well.
	Battery packs	Most remain in place where strapped to walls.
Electrical equipment	Panels, transformers, ducts, switchboards, distribution systems	Tall equipment overturns where not bolted at base or braced at top. Many instances of panels performing better than enclosing partitions, Horizontal movement of large equipment. Rigid conduits with structure support fail.

2.3.2.3 Damage to data processing equipment

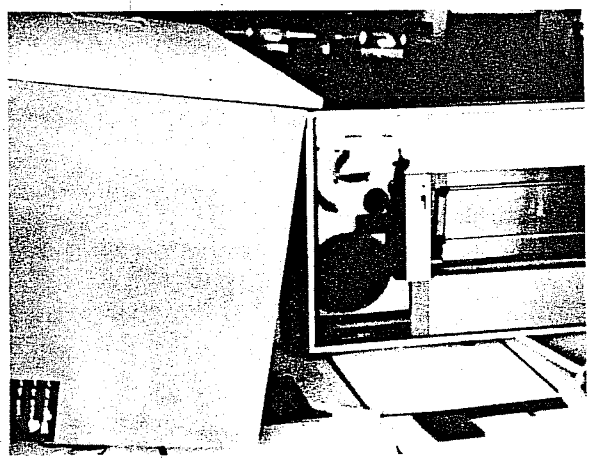
Damage to equipment has usually been a consequence of improper detailing of supports and anchorages. Minor damage in raised floors is due mainly to displaced panels or buckled panels under heavy pieces of equipment. Process equipment supported on casters or leveling pads, or both, rolled or slid distances that range anywhere from a few inches to several feet. Formed wire caster clips have proven to be inadequate to hold the machines in place, except on the ground floor or in areas of mild excitation. The clips were usually ejected from the casters allowing the equipment to slide or roll into adjacent equipment, or into unguarded floor openings resulting in overturned machines or damaged machine covers and skirts (Figure 2.5). In some instances, the equipment rolled until it went into a cable hole or was restrained by its cables. The lack of mechanical strain reliefs at the box entry points resulted in motion being stopped by the cable and connector resulting in damaged cables and connectors. Power or communication cables were separated and fiber-optic connectors broke. Failures of this type could be attributed to lack of sufficient slack in the cables or excessive movement of equipment.

Bent or broken levelers on equipment were common. The levelers were not strong enough to resist the bending forces developed when the machines moved horizontally. The leveler problems were worse if raised floor tiles were uneven and the leveler foot caught on the edge of a tile section.

Cabinets, tape and cartridge racks suffered damage ranging from distortion of the rack frame to overturning (Figures 2.6 and 2.7). Where racks are seismically braced or anchored to walls, tapes are occasionally dislodged by violent motion. Double stacked racks of equipment that are not secure have fallen over and table top units have been thrown to the floor.



A. Tape Drive Overturned



B. Peripheral Equipment Caught in Floor Penetration

Figure 2.5 Damage to Computer Equipment (Ref. 11)

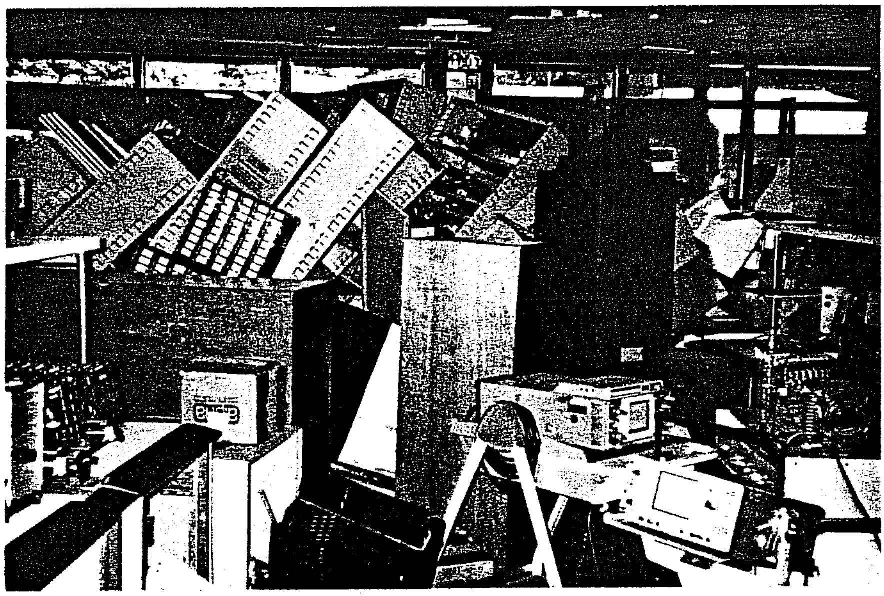


Figure 2.6 Damage to Testing, Storage, and Office Areas (Ref. 7)

Figure 2.7 Tape Spillage in Computer Center (Ref. 11)

Very few cases of equipment damaged due to excessive floor accelerations or vibration have been reported. Most of the damage is the product of fallen ceilings, broken pipes, overturned cabinets or sliding equipment. In many instances, this damage caused the evacuation of the building and interruption of operations ranging from a few hours to months.

2.3.3 Damage to support utilities

Damage to support utilities in past earthquakes has consisted mainly of loss of electrical power, water supply or sewer system breakage or blockage. The loss of electrical power is a major cause of manufacturing and data processing facilities downtime.

CHAPTER III
SEISMIC DESIGN PROVISIONS

3.1 INTRODUCTION

Experience from past earthquakes has focused attention on the need to provide effective guidelines and methods to design nonstructural elements and equipment to withstand seismic forces. Most of the current building codes include a section that deals exclusively with the design of architectural and electrical/mechanical systems. This chapter presents a summary of the current provisions in the Uniform Building Code 1991, Tri-Services Manual and National Earthquake Hazards Reduction Program Recommended Provisions (NEHRP-FEMA). Also, suggestions on the use of more detailed analyses and on the use of each of the manuals mentioned above, are given.

3.2 SEISMIC DESIGN CONSIDERATIONS

Most building code standards are intended to provide life safety. Very few provisions relate the response of the structural system to the limitations imposed by nonstructural systems and components. In a severe earthquake, the deflections are likely to be large, especially if design forces are reduced to take advantage of ductility. Therefore, damage to some elements of the structure will be inevitable. Drift and deflection limitations are imposed primarily to insure that damage is controlled but are not always related directly to nonstructural systems or components. Provisions for the design of nonstructural components and equipment, are normally formulated assuming that the behavior of the nonstructural components in a building can be uncoupled from the response of the primary structural system.

Following is a brief summary of the design provisions that affect the nonstructural systems in a building, proposed by the Uniform Building Code, units of the Department of Defense, and the National Earthquake Hazards Reduction Program.

3.2.1 Structural design criteria

Structural design criteria is taken as that related to the design of the vertical and lateral load resisting systems. In current codes, structural systems are designed taking advantage of their ductility to dissipate energy through deformations. This results in lower design forces and larger deflections. Story drift ratio limitations are set to limit these deflections. Though these limitations are not directly related to the response of nonstructural elements, they are some of the few provisions that help control the level of nonstructural element damage in moderate earthquakes.

3.2.1.1 Uniform Building Code

The Uniform Building Code (Ref. 15) gives different allowable story drift values depending on the period of the structure. For structures having a fundamental period of less than 0.7 seconds, the calculated story drift should not exceed $0.04/R_w$ or 0.005 times the story height. If the period is 0.7 seconds or greater, the calculated story drift should not exceed $0.03/R_w$ or 0.004 times the story height. R_w is a coefficient related to the ductility of the structural system in consideration and varies from 4 to 12.

The design lateral forces used to determine the story drifts may be computed using different procedures. For structures that are regular in plan, under 240 feet in height and with well defined lateral force resisting systems, a static lateral load analysis can be used to compute the story drifts. The seismic coefficient used to compute the equivalent static lateral loads depends on the importance of the building, soil characteristics, seismic zone, type of structural system and period of the structure. Most typical industrial buildings can be analyzed using the static lateral load procedure.

When the building does not meet the requirements specified in the UBC Code for use of the static lateral load procedure, a dynamic analysis should be performed. Two analysis procedures can be used: response spectrum analysis and time-history analysis. Response spectrum analysis is an elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes that significantly contribute to the structural response. Time-history analysis is an analysis of the dynamic response of a structure when the base is subjected to a specific ground motion time history.

The structure should be designed to remain elastic under the loads resulting from the above analyses, though usually spectra or ground acceleration records are scaled down to permit inelastic response. Therefore, the limitation to the drift ratio applies to deflections computed for an elastic structure. The increment on the drift ratios resulting from possible inelastic action in the structure is considered by using the R_w factor to reduce the allowable elastic drift ratio values.

3.2.1.2 National Earthquake Hazards Reduction Program (NEHRP)

Allowable drift ratios in the NEHRP provisions (Ref. 9) depend on the importance of the structure, referred to as seismic hazard exposure group, and the type of building (Table 3.1). Group III includes buildings having essential facilities that are necessary for post-earthquake recovery, Group II includes buildings that constitute a substantial public hazard because of occupancy or use, and Group I includes all buildings not covered in the previous groups.

TABLE 3.1 ALLOWABLE STORY DRIFTS - NEHRP (Ref. 9)

Buildings	Seismic Hazard Exposure Group		
	I	II	III
Single story steel buildings without equipment attached to the structural resisting system and without brittle finishes	No limit	0.020 h_{sx} *	0.015 h_{sx}
4 stories or less without brittle finishes	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}
All others	0.015 h_{sx}	0.015 h_{xs}	0.010 h_{sx}

* h_{xs} is the story height

The drift ratios are computed using procedures similar to those specified for the UBC Code but with different seismic coefficients. The drift ratios resulting from the elastic analyses are amplified by a factor C_d (deflection amplification factor), that varies from 1.25 to 6.5 (Ref. 9). This factor represents the ductility of the system and the properties of the materials from which it is constructed, thus giving an estimation of the expected inelastic deformations.

3.2.2 Nonstructural systems and equipment design

The design standards for nonstructural systems provide some guidance to help mitigate their damage. Most of the existing codes assume that the behavior of the nonstructural components can be uncoupled from the response of the structural system. As a consequence, nonstructural systems are designed using equivalent static lateral forces.

In many design standards, the following basic formula is used to establish the design lateral force:

$$F_p = Z\, C_p\, W_c$$

where:

F_p = lateral force applied to the nonstructural component

Z = numerical coefficient that depends on the seismic zone in which the structure is located

C_p = horizontal force factor that varies with the type of nonstructural component

W_c = weight of the whole, or a part, of the nonstructural component.

The essential difference between design standards is in the value specified for the coefficient C_p.

Nonstructural components whose weights are large in comparison with the weight of the structure tend to affect significantly the overall response of the structure. The response of these nonstructural components is highly dependent on the response of the building, thus uncoupling of the responses cannot be assumed and the static lateral load procedure cannot be applied. Such limitations normally apply to nonstructural components whose weights either exceed 20% of the total dead weight of the floor where they are located, or exceed 10% of the total dead weight of the structure (Ref. 9). For components with such a large mass, a dynamic analysis is usually required.

3.2.2.1 Uniform Building Code (UBC 91)

The design lateral seismic force, F_p, for nonstructural elements is computed using a formula similar to the one presented before:

$$F_p = Z\, C_p\, I\, W_c$$

The values of Z (seismic zone factor) and I (importance factor) are the same used for the building, with few exceptions. Nonstructural elements are classified as rigid or rigidly supported, and nonrigid or flexibly supported. Rigid or rigidly supported equipment is that having a fundamental period less than or equal to 0.06 seconds. Nonrigid or flexibly supported equipment is defined as a system having a fundamental period greater than 0.06 seconds.

The C_p coefficients presented in the UBC Code apply to elements and components and to rigid and rigidly supported equipment (Table 3.2). To design nonrigid or flexibly supported equipment, the lateral forces should be determined considering the dynamic properties of both the equipment and the structure which supports it. In no case, the lateral loads computed for flexible equipment can be less than those that would apply to rigid equipment. In the absence of a detailed analysis, the value of C_p for a nonrigid component may be taken as twice the value listed for the same rigid component. However, it need not exceed 2.0. The value of C_p for elements or components supported at or

TABLE 3.2 HORIZONTAL FORCE FACTOR C_p - UBC 1991 (Ref. 15)

ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS AND EQUIPMENT	VALUE OF C_p
I. PART OR PORTION OF STRUCTURE	
1. Walls including the following:	
a. Unbraced (cantilevered) parapets	2.00
b. Other exterior walls above the ground floor	0.75
c. All interior bearing and nonbearing walls and partitions	0.75
d. Masonry or concrete fences over 6 feet high	0.75
2. Penthouse (except when framed by an extension of the structural frame)	0.75
3. Connections for prefabricated structural elements other than walls, with force applied at center of gravity	0.75
4. Diaphragms	*
II. NONSTRUCTURAL COMPONENTS	
1. Exterior and interior ornamentations and appendages	2.00
2. Chimneys, stacks, trussed towers and tanks on legs:	
a. Supported on or projecting as an unbraced cantilever above the roof more than one half their total height	2.00
b. All others, including those supported below the roof with unbraced projection above the roof less than one half its height, or braced or guyed to the structural frame at or above their centers of mass	0.75
3. Signs and billboards	2.00
4. Storage racks (include contents)	0.75
5. Anchorage for permanent floor-supported cabinets and book stacks more than 5 feet in height (include contents)	0.75
6. Anchorage for suspended ceilings and light fixtures	0.75
7. Access floor systems	0.75

TABLE 3.2 (cont.)

ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS AND EQUIPMENT	VALUE OF C_p
III. EQUIPMENT	
1. Tanks and vessels (include contents), including support systems and anchorage	0.75
2. Electrical, mechanical and plumbing equipment and associated conduits, ductwork and piping, and machinery	0.75

* Other requirements apply (Ref. 15)

below the ground level may be two-thirds of the values of C_p listed in Table 3.2.

However, the design force should not be less than that obtained by treating the item as an independent structure. Equipment weighing less than 400 pounds, furniture and temporary or movable equipment are excluded from these provisions.

3.2.2.2 National Earthquake Hazards Reduction Program (NEHRP)

In the NEHRP provisions (Ref. 9) a static type of analysis similar to that in the UBC Code is used. For determining static design loads, the nonstructural elements are classed as architectural components, or mechanical/electrical components. A performance characteristic level (P) is assigned to each of the nonstructural elements. The levels are: superior (S), good (G) and low (L), and were established by assessing potential hazards to life safety according to the location and function of the component. Values associated with these performance factors are 1.5 for superior performance (S), 1.0 for good performance (G), and 0.5 for low performance (L).

TABLE 3.3 SEISMIC COEFFICIENT (C_c) AND PERFORMANCE CHARACTERISTIC LEVELS REQUIRED FOR ARCHITECTURAL SYSTEMS OR COMPONENTS NEHRP (Ref. 9)

Architectural components	C_c factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels		
		III	II	I
Appendages				
Exterior nonbearing walls	0.90	S	S	S
Wall attachments	3.00	S	S	S
Connector fasteners	6.00			
Veneer attachments	3.00	G	G	L
Roofing units	0.60	G	G	NR
Containers and miscellaneous components (free standing)	1.50	G	G	NR
Partitions				
Stairs and shafts	1.50	S	G	G
Elevator shafts	1.50	S	L	L
Vertical shafts	0.90	S	L	L
Horizontal exits including ceilings	0.90	S	S	G
Public corridors	0.90	S	G	L
Private corridors	0.60	S	L	NR
Full-height area separation partitions	0.90	S	G	G
Full-height other partitions	0.60	S	L	L
Partial-height partitions	0.60	G	L	NR
Structural fireproofing	0.90	S	G	L
Ceilings				
Fire-rated membrane	0.90	S	G	G
Nonfire-rated membrane	0.60	G	G	L

TABLE 3.3 (cont.)

Architectural components	C_c factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels		
		III	II	I
Raised access floors	2.00	S	G	L
Architectural equipment - ceiling, wall, or floor mounted	0.90	S	G	L
Architectural components - elevator and hoistway structural systems				
Structural frame providing supports for guide rail brackets	1.25	S	G	G
Guide rails and brackets	1.25	S	G	G
Car and counterweight guiding members	1.25	S	G	G

a. Architectural design requirements: Architectural components include appendages, partitions, structural fireproofing, ceilings, raised access floors, architectural equipment and other architectural components (Table 3.3). Architectural systems and components and their attachments are to be designed to resist seismic forces determined as follows:

$$F_p = A_v C_c P W_c$$

where:

F_p = seismic force applied to the component at its center of gravity,

A_v = Effective Peak Velocity-Related Acceleration (same as that used for the building),

C_c = seismic coefficient for components of architectural systems, as shown in Table 3.3.

P = performance criteria factor (S=1.5, G=1.0 and L=0.5), and

W_c = weight of component.

b. Mechanical and electrical design requirements: All mechanical and electrical components listed in Table 3.4 are included. The seismic forces are determined as follows:

$$F_p = A_v\, C_c\, P\, a_c\, a_x\, W_c$$

where:

F_p, A_v, P and W_c are as defined before, and

C_c = seismic coefficient for components of mechanical or electrical systems as shown in Table 3.4.

a_c = amplification factor related to the response of a system or component as affected by the type of attachment (e.g. fixed or direct attachment to building, resilient mounting system, or mounted on the ground or on a slab in direct contact with the ground).

a_x = amplification factor at level x related to the variation of the response in height of the building, determined as: $a_x = 1.0 + (h_x/h_n)$, where h_x is the height above the base to the level in consideration, and h_n the height above the base to the top floor of the structure.

TABLE 3.4 SEISMIC COEFFICIENT (C_c) AND PERFORMANCE CHARACTERISTIC LEVELS REQUIRED FOR MECHANICAL END ELECTRICAL COMPONENTS NEHRP(Ref.9)

Mechanical/Electrical Components	C_c factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels		
		III	II	I
Emergency electrical systems (code required) Fire and smoke detection system (code required) Fire suppression systems (code required) Life safety system components	2.00	S	S	S
Elevator machinery and controller anchorage	1.25	S	G	G
Boilers, furnaces, incinerators, water heaters, and other equipment using combustible energy sources or high-temperature energy sources, chimneys, flues, smokestacks and vents Communication systems Electrical bus ducts and primary cable systems Electrical motor control centers, motor control devices, switchgear, transformers, and unit substations Reciprocating or rotating equipment Tanks, heat exchangers, and pressure vessels Utility and service interfaces	2.00	S	G	L
Machinery (manufacturing process)	0.67	S	G	L
Lighting fixtures	0.67	S	G	L

TABLE 3.4 (cont.)

Mechanical/Electrical Components	C_c factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels		
		III	II	I
Ducts and piping distribution systems				
Resiliently supported	2.00	S	G	NR
Ridigly supported	0.67	S	G	NR
Electrical panelboards and dimmers	0.67	S	G	NR
Conveyor systems (nonpersonnel)	0.67	S	NR	NR

* S = superior (1.5)
G = good (1.0)
L = low (0.5)
NR = non rated

3.2.2.3 Tri-Services Manual (U.S. Department of Defense)

The Tri-Services Manual (Ref.4) has different provisions according to the type of element to be designed. Elements are classed as architectural components or mechanical and electrical elements. Provisions for design of architectural components are similar to those proposed by the UBC Code presented above, and the resulting static lateral forces are the same as those obtained with the UBC Code. Mechanical and electrical elements are classed as: rigid and rigidly mounted, flexible or flexibly mounted, or equipment mounted directly on ground.

a. Rigid and rigidly mounted equipment in buildings: Equipment units and equipment supporting systems for which the period of vibration is estimated to be less than 0.05 seconds. The period is measured assuming that the equipment responds as a single-degree-of-freedom system, with $T_a = 2\pi\sqrt{W/k}$, where T_a is the period of the equipment, and W and k are their weight and stiffness respectively. The equivalent static lateral force is computed as $F_p = ZIC_pW_p$, the same forces prescribed by the UBC Code.

b. Flexible and flexibly mounted equipment in buildings: Appropriate seismic design forces are determined with consideration given to both the dynamic properties of the equipment and to the building or structure in which it is placed. The approximate design procedure is based on the equipment responding as a single-degree-of-freedom system to the motion of one of the predominant modes of vibration of the building at the floor level in which the equipment is placed. The period of the equipment is estimated as $0.32\sqrt{W/k}$, the same as for rigid equipment. The period of the building is computed by considering the building as a multiple-degree-of-freedom system with more than one mode of vibration. The fundamental period of vibration is that used for the design of the building. Higher periods and modes of vibration must also be considered.

The appendage (equipment) motion amplification with respect to the peak motion of the floor level that it is mounted on is considered through the Appendage Magnification Factor (M.F.). The M.F. factor is generally based on steady-state motion due to the floor responding as a uniform sine wave. Since the buildings respond to earthquakes is a random fashion, the M.F. factors generated are not as large as computed by steady-state response. In order to approximate a realistic value for a design M.F. factor, it is assumed that: (1) the periods Ta (equipment) and T (building) will differ by at least 5%, (2) buildings are not perfectly linear elastic, especially at high amplitudes of response; (3) the floor response is not an exact, uniform sine wave, and (4) the number of high amplitude floor response cycles is substantially less than 25. The resulting design M.F. factor curve is shown in Figure 3.1, and can be used as a design aid in lieu of a more rigorous analysis.

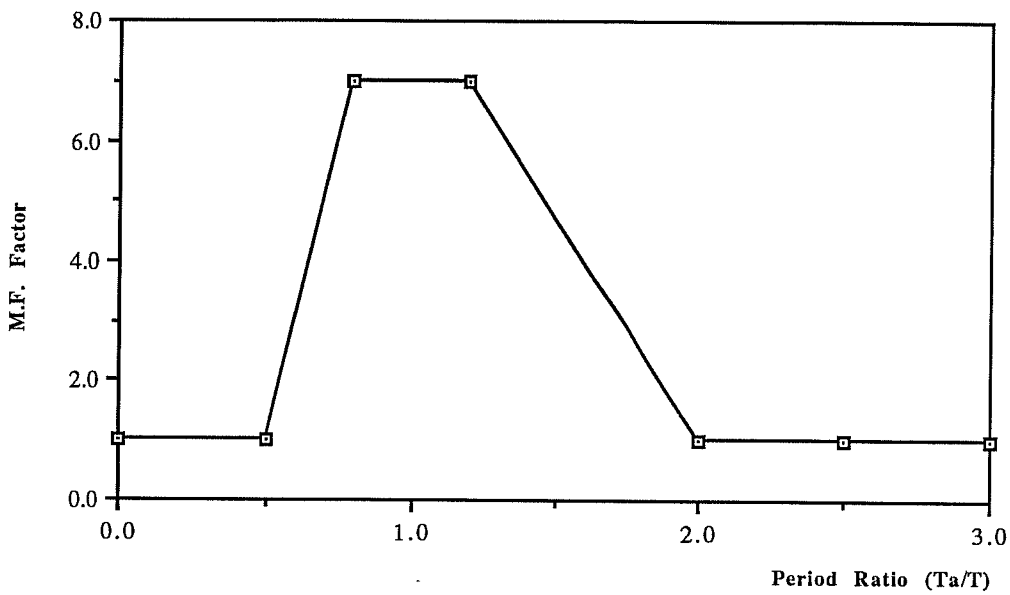


Figure 3.1 Design Magnification Factor (M.F.) vs. Period Ratio (Ref. 4)

The equivalent static force for the anchorage of flexible or flexibly mounted equipment is given by:

$$F_p = Z\, I\, A_p\, C_p\, W_p$$

which is a modification of the formula used for rigid equipment, where A_p is the amplification factor for the coefficient C_p. A_p is related to the M.F. values shown in Figure 3.1; however, the maximum value of 7.5 is reduced to 5.0 to account for multimode effects that are assumed to be included in the C_p values. C_p values are the same used in the UBC Code. The values of A_p will be determined by:

1. If the periods of the building and equipment are not known, A_p=5.0.

2. If the fundamental period of the building is known, but the period of the equipment is not known, A_p is determined by Table 3.5.

TABLE 3.5 AMPLIFICATION FACTOR Ap FOR FLEXIBLE OR FLEXIBLY MOUNTED EQUIPMENT (Ref. 4)

Building period (sec)	Less than 0.5	0.75	1.00	2.00	Greater than 3.00
A_p	5.00	4.75	4.00	3.30	2.70

3. If building and equipment periods are both known, A_p may be approximated using the graph in Figure 3.2 (Ref. 4).

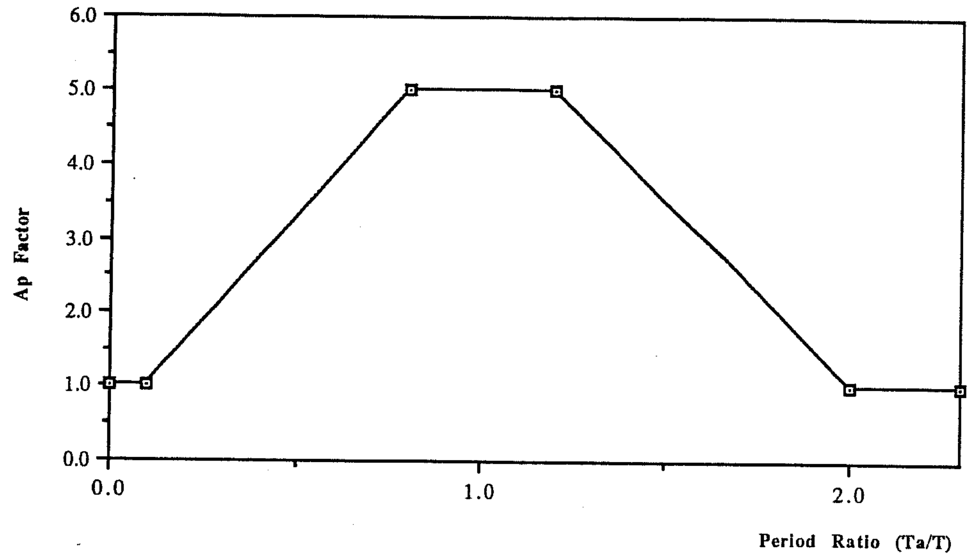


Figure 3.2.a Ap vs. Period Ratio (Ta/T)
(T < 0.5 sec)

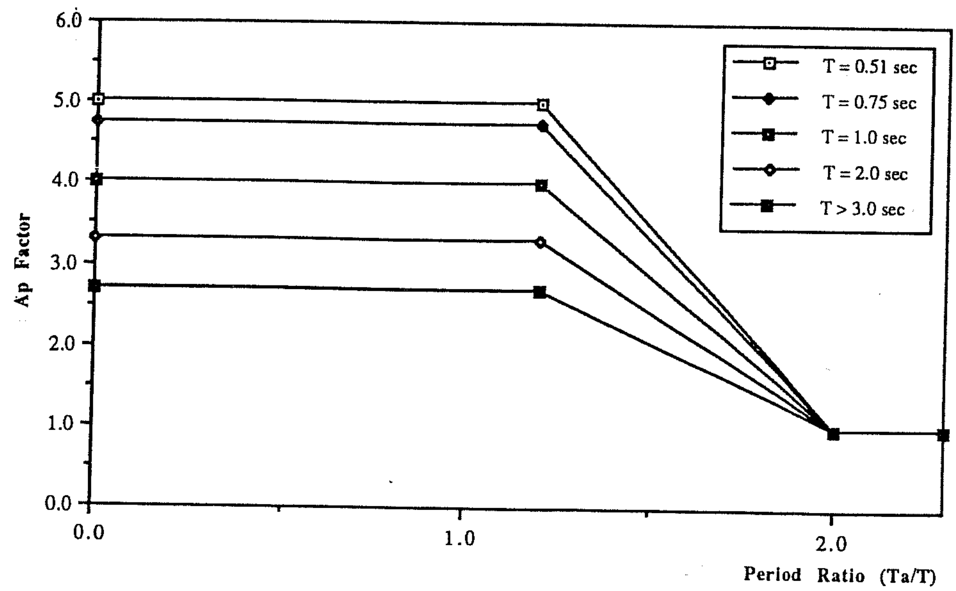


Figure 3.2.b Ap vs. Period Ratio (Ta/T)
(T > 0.5 sec)

Figure 3.2 A_p vs. Period Ratio (T_a/T) (Ref. 4)

c. Equipment on the ground: this is equipment placed in contact or buried in the soil, and is classified as:

1. Rigid and rigidly mounted: defined in the same manner as rigid equipment discussed above. The equivalent static lateral force is given by:

$$F_p = Z\,I\,(2/3\;C_p)\;W_p$$

2. Flexible or flexibly mounted equipment: this equipment is generally not subjected to the additional magnification factors that apply to similar equipment located in the elevated stories of buildings. Equivalent static lateral force is given by:

$$F_p = Z\,I\,(2\;C\;S)\;W_p$$

where $C = 1/15\sqrt{T_a}$ and S is determined as:

$$S = 1.0 + T_a/T_s - 0.5\,(T_a/T_s) \quad \text{for } T_a/T_s \leq 1.0$$
$$S = 1.2 + 0.6\,T_a/T_s - 0.3\,(T_a/T_s)2 \quad \text{for } T_a/T_s > 1.0$$

T_s is the characteristic site period established from geotechnical data. If T_s is not known, $S = 1.5$. When the periods are not known, (2CS) is equal to a maximum value of 0.28.

3.3 SUGGESTIONS FOR SEISMIC ANALYSIS OF EQUIPMENT

As discussed above, there are several levels of standards for the determination of seismic design forces. Equivalent static force design requirements and seismic force design recommendations for different elements are contained in the UBC Code. In the UBC code, the seismic performance level used for design and detailing is set by giving different values to the Importance Factor (I) of each unit (nonstructural element or piece of equipment). The value given to this factor is normally established based on vulnerability of the element and time required to repair or replace it in case of damage. Common practice is to analyze the equipment for different performance levels. One level would normally correspond to a reoccupancy time of about 60 to 90 days, usually for $I=1.0$. On a higher level, with $I=1.5$ or with larger values of Cp, reoccupancy times of about two weeks would be anticipated. All the detailing, installation, and inspection should correspond with those levels of desired performance. The seismic level used for design and detailing of nonstructural restraints and equipment should also be consistent with the seismic level used or desired for the building as a whole.

The UBC Code requirements do not necessarily assure continuous operation and do not address specific dynamic characteristics of a given building on a given site, or specific dynamic characteristics of the nonstructural element or equipment. More detailed and in some cases more accurate force determination is contained in the seismic design requirements proposed by the U.S. Department of Defense (Ref. 4) and in the provisions of the NEHRP-FEMA 1988 (Ref. 9). Both of these documents recommend the use of dynamic characteristics of the element and the building, specific site characteristics and location of the element in the building to determine design static forces.

The most elaborate and possibly most accurate method of determining design forces and response is a dynamic analysis, using measured earthquake motions. A floor response spectrum can be used to evaluate the seismic forces acting on the equipment installed within a building. The response spectrum has to be generated for each level of the structure since the motion experienced by various floors in the building is not the same.

Where the equipment is flexible and is supported by a flexible structure, the equipment-structure interaction is important. If the mass of the equipment is significantly less than the mass of the floor on which the element is supported (10% or less according to Ref. 11), the equipment and the structure can be uncoupled. The design forces can be established using the design spectra computed for each floor or performing a dynamic analysis of a model of the equipment using the acceleration history for each floor as input.

If the mass of the flexible equipment is not significantly less than the mass of the floor that supports it, then the equipment and the structure must be analyzed together as a coupled system. Most likely, this would not be required for equipment in conventional industrial building structures.

To determine the level of analysis to be used to design nonstructural systems and equipment, it is necessary for the owner/operator of the facility to establish the degree of risk of loss of function and length of shutdown that could be accommodated without serious economic impact. General guidelines given in Ref. 11 are presented in Table 3.6. Special attention should be paid to making the expected deformations of the type of building framing compatible with the nonstructural components and systems, and equipment in the building.

TABLE 3.6 DETERMINATION OF DESIGN FORCE FOR ALL ELEMENTS (Ref.11)

PROTECTION	DETERMINATION OF DESIGN FORCE
Minimum Recommended Protection	Equivalent static lateral force proposed in the UBC Code (Ref. 15). For $T>0.05$ sec., or for other potential response compliance, use 2 x C_p. For highly irregular or tall buildings, use method shown for Intermediate Protection.
Intermediate Protection	Use equivalent static lateral force similar to NEHRP-FEMA (Ref. 9) or Tri-services Manual (Ref. 4). Ratio of building period and element period used to determine response magnification up to a recommended maximum of 5. For highly irregular or tall buildings, use method shown for Maximum Protection.
Maximum Protection	Specific building floor response calculated from dynamic analysis and used as dynamic input for design of element.

CHAPTER IV
SEISMIC PROTECTION TECHNIQUES

4.1 INTRODUCTION

Examples of damage suffered by nonstructural elements, and electrical and mechanical equipment were given in Chapter II. Much of the damage shown could have been avoided, or at least minimized, if proper seismic protection techniques had been used to design and install the elements and equipment. It is important that industrial owners realize the benefits that a seismic preparedness program could provide in terms of lessening the economic impact of damage and loss of production due to earthquakes. As part of a seismic preparedness program, a seismic protection plan including adequate analysis of earthquake design forces, development of anchoring and restraining techniques, and prohibition of certain hazardous or high risk conditions should be included. This plan should address all nonstructural elements and equipment whose failure would affect life safety and continuity of operations of the facility.

For nonstructural elements and equipment, seismic protection details depend mainly on the element configuration and support condition, rather than on the element function. Nevertheless, the element function is critical to determining its importance to operations, the level of risk that can be allowed, likely damage and time to repair or replace. These variables will indicate the proper analysis and design techniques to use for each element, and the best procedure to either install or retrofit.

In general, high-technology facilities have similar architectural characteristics, mechanical/electrical features, and equipment which is independent of their specific production process or operation. Architectural features that are usually encountered are raised floors, suspended ceilings, and cabinets and tape storage racks. Mechanical/electrical features include piping systems, HVAC systems and lighting. Equipment usually includes computers and other data processing machines. This chapter presents suggestions for seismic protection techniques of nonstructural elements and mechanical/electrical equipment that are considered representative of those encountered in high-tech industry facilities. The elements chosen are those vital to the functioning of the facility and those that have experienced the most damage during past earthquakes. Most of the suggestions are based on past earthquake experience, current code provisions and/or engineering practice. The intent is not to provide an exhaustive compilation of seismic protection details but rather general examples of what can be done to mitigate earthquake damage to generic elements. In practice, each element or equipment component has to be carefully analyzed to determine the best procedure to improve its seismic response and decrease its vulnerability.

Unless otherwise mentioned, the analysis and design techniques to be used are those specified in Chapter III. The proper technique will be chosen according to the importance of the element in consideration. In all cases, current building code provisions should be reviewed and applied when designing the elements. Manufacturers specifications for the materials and the equipment should also be reviewed to determine the best restraining techniques, analysis and installation procedures.

4.2 RAISED COMPUTER FLOORS

4.2.1 Description of typical systems and components

Raised computer floors are used to accommodate the different utility connections and circulation of cooling air required for the operation of machine or computer rooms. In general, raised floors consist of standardized 2 foot by 2 foot modular systems composed of removable floor panels, stringers, and pedestals. Floor panels are made of wood, steel, or aluminum. They can be supported by the stringers, or span directly to the pedestal heads. The stringers, when present, are made of light gage bent steel or aluminum that either snap or are screwed to the pedestal head. When stringers are not used, the floor panels are attached to the pedestal heads with metal screws at each panel corner. There are different types of pedestals composed in general of the following parts: a pedestal head used to support the floor panels and stringers, a threaded stud and adjustment nut used to adjust the height of the pedestal to level the floor, a locking collar to prevent the leveling nut from loosening under normal floor vibration, a tubular column into which the threaded stud is inserted, and a base plate used for pedestal support and anchorage to the structural slab (Figure 4.1).

There are four basic raised floor systems commonly used to resist lateral earthquake loads: cantilever pedestals, braced pedestals, braced panels, and pedestal-stringer frame (Ref. 11).

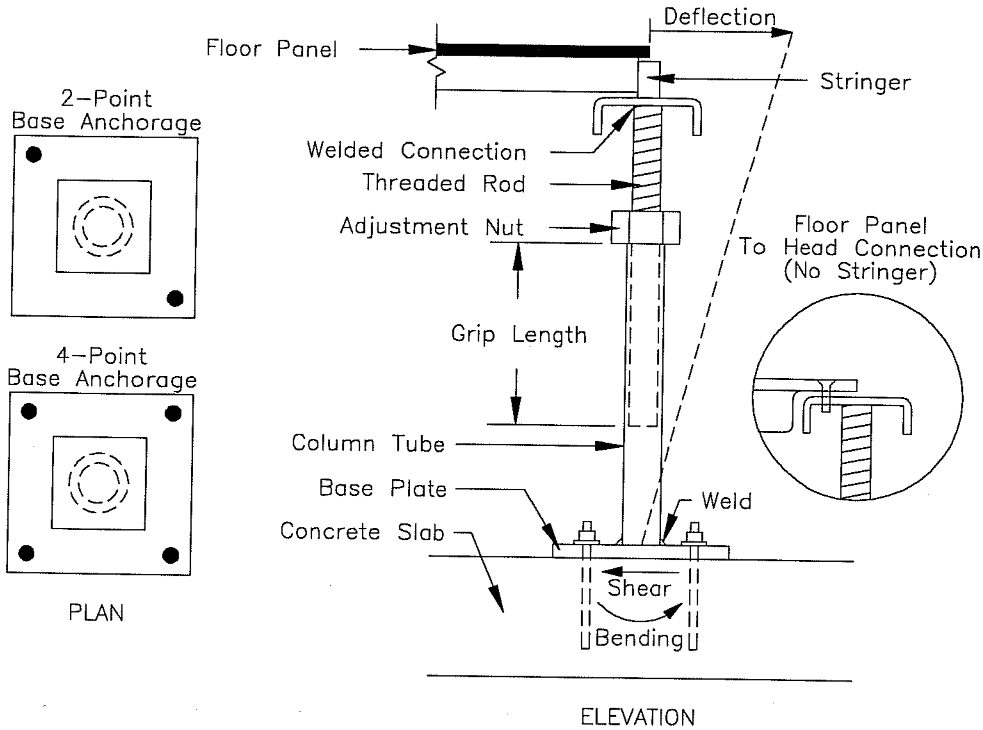


Figure 4.1 Cantilever Pedestal (Ref. 11)

4.2.1.1 Cantilever pedestals

Figure 4.1 shows the basic components of cantilever pedestals. The lateral loads produced by earthquake motions are transmitted from the raised floor to the structural slab through cantilever bending action of the pedestals. The design of the connections for this system, particularly the column to base and column to pedestal head connections, is critical for the proper transfer of forces. In the past, the normal practice in industrial buildings was to glue the base plate to the supporting slab using a mastic or epoxy compound. These products proved to deteriorate with time, loosing their adhesiveness and leaving the pedestals with no positive connection to the structural slab. A better practice is to use drilled anchors, designed to transmit to the slab the shear and bending forces resulting from the cantilever action of the pedestals. These anchors can be placed either in the four corners of the plate, or in diagonal corners, as shown in Figure 4.1. The metal base plate must be strong enough to transmit the bending forces from the pedestal to the supporting floor, and should be stiff enough to minimize rotational deflection at the base.

Typical pedestal columns are generally formed from a tubular section and/or threaded stud or rod. These columns must be designed to resist the lateral earthquake loads through cantilever action, and should be stiff enough to prevent large floor deflections. Pedestal columns should be able to develop plastic action to allow redistribution of seismic overloads and/or heavy floor loads without failure. The threaded rod is used to provide vertical adjustability to the pedestal. Generally, the threaded rod fits into the tubular column without positive connection to resist vertical uplift. The connection between the rod and the columns transmits bending movement through side bearing of the rod against the walls of the column. This connection should be designed to transmit the bending forces for all pedestal heights and grip lengths. The pedestal head-to-threaded rod connection is usually welded, but in some cases, a slip-fit head-in tube socket connection may be used. The connection should be designed to transmit shear and bending forces resulting from lateral loads.

Positive attachment between the floor panels and pedestal heads is necessary to transfer the vertical and lateral floor loads to the pedestals, but is usually omitted due to the decrease in ease of access below the floor. Positive attachment can be provided using screws or other mechanical fasteners. When stringers are provided, typical stringer to pedestal connections consist of a raised metal lip engaged in a hole in the stringer or a threaded screw through the stringer into the pedestal head. Sometimes, a frictional nut is used to clamp the stringer to the pedestal head. When stringers are used, a redundant load resisting system is provided to redistribute overloads from one section of the floor to other areas.

The major advantage of the cantilever pedestal system is that it leaves the floor area unobstructed for placement of subfloor utilities. However, the deflections under earthquake loads may be very large, especially when tall pedestals are used. Because of this, deflections usually become the controlling design consideration, resulting in heavier pedestal columns and base plates than required by strength alone.

4.2.1.2 Braced pedestals

Bracing is an effective scheme to provide lateral force resistance and minimize deflections on raised floors supported on tall pedestals. Two basic bracing systems are commonly used: diagonal brace to pedestal (Figure 4.2.a), and a cruciform pattern of diagonal bracing connected to either the panel or to four pedestal bases and the floor panel (Figure 4.2.b).

The critical elements in the design of a braced pedestal system are the pedestal column under tension and compression, the brace and its connection to the pedestal head and floor slab, and the stringers that act as collectors. The pedestal columns are usually designed for leveling purposes only, and will pull apart easily under tension loads. In order to make the diagonal bracing scheme work properly, it is necessary to provide an adequate tension connection between the building floor and the base plate, a tension connection between the column and the base plate, and a tension connection between the tubular

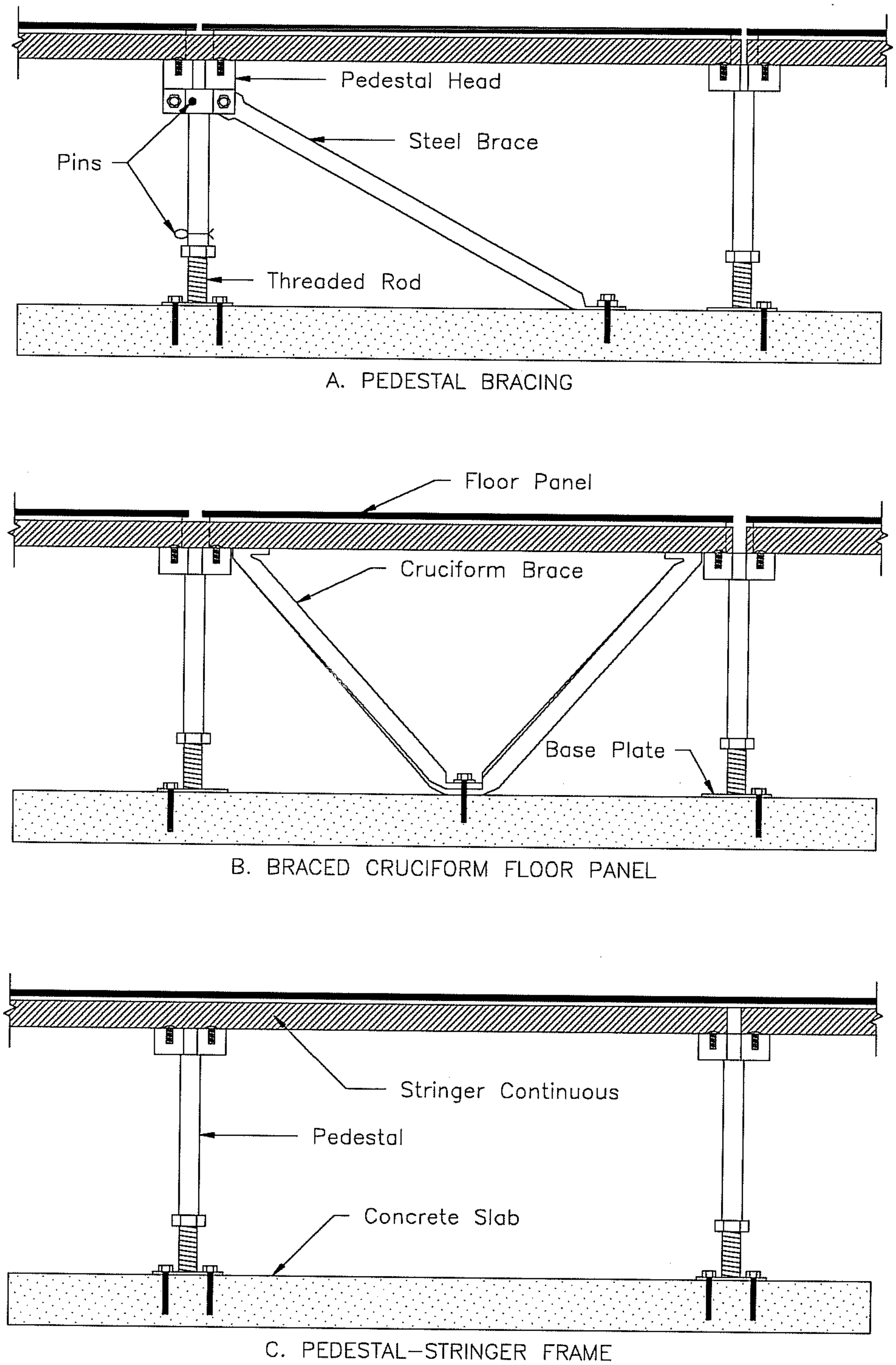


Figure 4.2 Raised Floor Bracing Schemes (Ref. 11)

column and the threaded rod/stud at the adjustment nut. The diagonal brace should be designed to carry tension and compression loads without buckling laterally. The connection between the brace and the pedestal head should be designed to resist forces higher than those the brace can develop, to allow for overload redistribution and to prevent brittle failure of the connection. The brace should be positively anchored to the floor slab with devices such as anchor bolts that could be torque-tested after installation. This connection scheme should withstand normal day-to-day vibrations without loosing anchorage or strength. The stringer system and its connection to the pedestal heads should be designed to adequately transfer the floor loads to the braced pedestals. Since the floor panels cannot carry tension, the stringers are the only elements that can assure the transfer of the tension loads to the braced system.

The braced pedestal system provides an economical solution for tall pedestal floor systems. However, the braces may interfere with the placement of subfloor utilities.

4.2.1.3 Braced panel system

The braced panel or cruciform system (Figure 4.2.b) utilizes braces attached to the four corners of the panel and anchored to the building floor with a bolt torqued in tension. This scheme produces a hold down force on four corner pedestals, minimizing the tension uplift in those locations. As in the braced pedestals, the stringer system serves as a collector to transfer loads from adjacent unbraced panels. Stringer size and connections should be designed for such purpose and to accommodate the vertical component from the diagonal brace force. The metal skin on the bottom side of the floor panel should be reinforced at the corners to prevent tearing at the brace-panel connection. The floor anchor must be designed to accommodate variations in floor height from area to area. This induces bending forces in the anchor bolt due to lateral brace forces. The bolt must be designed for combined tension, shear and bending.

Besides having the same advantages of the braced pedestal system, the braced panel scheme may be an economical method for bracing new floors that require additional strength under heavy equipment. It has the disadvantage that placement of subfloor utilities is difficult in the braced area, and removal of braced panels is also difficult.

4.2.1.4 Pedestal-stringer frame

In the pedestal-stringer frame system (Figure 4.2.c), lateral loads are transferred to the building floor through bending of the stringer and pedestal. Bending resistance is developed in the pedestal column at the pedestal head and at the pedestal base. The connection of the pedestal base to the building floor is similar to the one used in the cantilever pedestal system. The same applies to the strength of the pedestal column. Design considerations for the pedestal column and the pedestal base are the same as those used in the cantilever pedestal system. The connection between pedestal head and threaded rod is critical to develop the bending capacity of the system. This connection is usually spot-welded or fillet-welded. The connection between stringer and pedestal head is normally made with two metal screws attaching both elements. Heavy duty stringers and special connection designs are generally required to avoid brittle tear-out of the screws in tension resulting from bending forces.

This systems utilizes all the conventional elements in a raised floor system to develop an efficient lateral load resisting system. It also minimizes interference to placement of subfloor utilities. It has the disadvantage that relatively large deflections of the floor system are possible and the connections between all the elements must be inspected regularly.

4.2.2 New raised floor construction

4.2.2.1 Design considerations

The most important design consideration for raised floor systems is control of horizontal deflections induced by earthquake loads. These deflections are usually ignored in the building code seismic design for raised floor systems. However, floor deflections have to be considered to control pounding of the floor system with adjacent building columns and perimeter walls. Also, large deflections must be avoided if there is any rigidly braced or anchored equipment supported on independent frames that penetrate the floor. Rigid support framing is usually stiffer than the floor system and will tend to resist most of the seismic floor loads if the raised floor is considerably more flexible. Pounding of the floor system against the rigid support frames may produce failure of the support frames or subfloor bracing schemes used under heavy equipment.

Vertical earthquake motion effects should also be considered in the design of raised floors. The use of flexible structural systems to support raised floors may amplify the vertical motions on the raised floor. The vertical motions can cause floor panels to float free from the stringers of pedestal supports, and may enhance the tendency for floor buckling. Panel tie-downs to pedestal heads and tension uplift connections in the pedestal columns should be provided when large vertical earthquake motion on the building floor is expected.

4.2.2.2 Methods of analysis and design

There are two basic methods of analysis and design of raised floors: static building code coefficients, and dynamic seismic analysis. Static building code coefficient methods were presented in Chapter III. These methods are usually very simple to use, requiring only an estimate of the weight of the floor and equipment and the seismic zone. However, they do not always consider the variation in lateral forces with height in the building, nor the variation in earthquake motion characteristics and amplitude from site to site, and the dynamic characteristics of each raised floor system.

In the dynamic seismic analysis approach, site-specific ground motions can be utilized, and the level of shaking at each floor in the structure, along with the dynamic characteristics of each raised floor system can be included in the analysis. Computed motions at the raised floor may also be used to select the appropriate methods of equipment support and anchorage. This is especially important for vibration-sensitive equipment. This method has the disadvantage that is more difficult to use, requires specific computer software and has a greater cost.

Analysis and design methods used are chosen according to the degree of earthquake performance required for the raised floor, e.g. acceptable outage time in case of damage or failure, and importance of the supported equipment. Important components of the design strategy are: a realistic assessment of the design forces, reasonable limits on lateral floor deflections, proper appraisal of the supporting floor structural system,its failure modes and design safety factors, program of field testing and verification, and a program of field inspection and certification. General guidelines for each of these components are summarized in Table 4.1 (Ref. 11).

a. Assessment of force levels: The factors that should be considered when determining the design force level are site seismic exposure level, location of the raised floor in the building to account for possible amplification of ground motions, dynamic amplification of building motion due to raised floor flexibility, and ductility or reserved capacity of the lateral force resisting system of the raised floor. As a minimum, seismic forces must satisfy building code requirements. Design safety factors may be increased according to the importance of the raised floor and the performance level required.

b. Limits on horizontal deflections: Since specific horizontal deflection limits for raised floors are not specified in building codes, the designer should select the limits to minimize damage and buckling of the floor system, and damage to equipment and other elements supported on the raised floor. Factors that should be considered when establishing the deflection limits are: presence of glass partitions or exterior cladding adjacent to the raised access floor, height of the raised floor system, type of lateral load resisting system, extent and location of rigid building elements such as columns or walls that penetrate the raised floor and degree of seismic separation between the floor system and those elements, and potential amplification of vertical ground motions in the raised floor due to its flexibility.

c. Appraising the lateral force resisting system: The lateral force resisting system must be properly identified, and must have well defined load paths to transfer forces from the raised floor to the structural slab. Typical systems were described previously in this chapter, along with the most important design

TABLE 4.1 SEISMIC DESIGN STRATEGIES FOR RAISED ACCESS FLOORS (Ref.11)

DESIGN STRATEGIES	LEVELS OF PROTECTION		
	Maximum	Intermediate	Minimum
1. Lateral Seismic Force	Required	Required	Required
Static Building Code	Increase design safety margin	Increase design safety margin	Acceptable
Dynamic Analysis	Recommended	Suggested	Optional
2. Deflection Limits	Required	Required	Recommended
3. Lateral Force Resisting System	Required	Required	Required
Cantilever Pedestal	B*	B	A
Pedestal Braced	A	A	A
Panel Braced	A	A	B
Pedestal-Stringer Frame	C	C	B
Stringers	Required	Required	Recommended
Panel Tied Down	Recommended throughout the floor	Recommended around heavy floor loads	Optional
Bracing for Tall Floors	Recommended	Recommended	Optional
4. Testing			
Laboratory Test Program	Required	Recommended	Optional
Field Test Program	Required	Required	Required
5. Inspection and Certification	Required	Required	Required

* Ranking of lateral force resisting system:
A = Usually the best solution
B = Acceptable, but may have limitations
C = Optional, but generally not recommended

considerations. Adequacy of each system according to the level of protection required is presented in Table 4.1.

d. Testing program: The designer should determine the adequacy of the materials to be used in the raised floor system from test data provided by the manufacturer. Field test programs of the different elements of the floor systems as well as an assembled floor system should be conducted in the early stages of construction. Guidelines for conducting these tests are presented in Table 4.1.

e. Inspection and certification: Inspection of the floor system during construction should always be required to verify that the system has been assembled as specified and that all mechanical fasteners have either been torque-tested or otherwise installed to meet design specifications.

4.2.2.3 Specifications

Detailed seismic specifications for the design, fabrication and installation of raised access floors should be given by the designer. These specification should contain the design loads, limits on lateral floor deflections under design loads, all necessary specifications for construction of the raised floor along with information on its ductility, redundancy and safety factor against failure; and test, inspection and certification requirements.

4.2.3 Existing raised floors

The following are guidelines for evaluation and retrofit of existing raised floors.

4.2.3.1 Methods of evaluation

The existing raised floor should meet the same performance requirements for new raised floor systems, therefore, the design guidelines mentioned before also apply. Some of the evaluation methods that can be used to assess the condition of existing raised floors are:

a. Preliminary survey: General aspects to determine when conducting a preliminary survey are loads on the floor, height of floor, location of floor relative to height of building, value of the equipment and its importance to function of the facility, and overall seismic exposure of the site. It is important to determine the type of lateral load resisting system used, along with the presence and physical condition of all the elements necessary for the adequate performance of the particular system. Attention should be given to the condition of the connections. If possible, connections should be physically tested. This can be easily done by applying lateral and/or uplift forces using body weight.

The results of the preliminary survey should either lead to the conclusion that the raised floor system is adequate, or that more tests and evaluation are required.

b. Analytical evaluation: Once the preliminary survey is completed, the information gathered can be used to build a model of the raised floor. Analysis can be performed using any of the methods discussed in Section 4.2.2.

c. Test evaluation: The most direct and complete method of evaluation is to perform physical tests of all the elements and the assembled floor system.

Results should indicate if strengthening and/or stiffening should be implemented.

4.2.3.2 General guidelines for retrofit

The need to retrofit the raised floors, the retrofit scheme and the appropriate time to do it should be determined considering the existing lateral force resisting capacity of the floor, the value and functional importance of the equipment on the floor, the potential functional disruption caused by the retrofit process, and the schedule for equipment replacement, reconfiguration or facility remodeling. Considering the fact that the facility usually must remain operational during the retrofit process, the scheme that interferes least with ongoing operations is the best. Shutdown or relocation of equipment may not always be permitted, removal of floor panels adjacent to certain equipment may not be possible, dust must be controlled to minimize disruption of sensitive electronic equipment. With these limitations, systems such as the braced pedestal or braced panel that provide flexibility in location of braces may have an advantage over strengthening each pedestal and its base.

Once the strengthening scheme is constructed, the complete system should be tested to prove its adequacy. In some cases, the addition of strengthening elements may introduce premature failure in the system at a new location, and this may not have been predicted in the analysis. Testing of the installed system is critical to verify that it meets the design intent.

4.3 COMPUTER EQUIPMENT

4.3.1 Description of equipment

Equipment covered in this section is that usually encountered in light industrial facilities: mainframes (e.g. processors, power distribution units, control consoles), peripherals (e.g. disk drives, printers, communications controllers), mass storage facilities, document processors and telecommunications gear. Most of this equipment is constructed with a metal frame stiffened by steel panel siding or internal bracing. The frame is enclosed with doors and metal panels to control dust, air circulation, and electrostatic effects. The frame is generally supported on leveling pads, casters, or metal skids on the raised floor. In some cases, the frame is supported by special supports through the raised floor. Internal components are generally rigidly attached to the frame, except for vibration-sensitive equipment for which shock isolation supports are provided. All utilities, such as chilled air and water, power, and electrical signal cabling, enter the unit from the underside through penetrations in the raised floor.

The potential earthquake behavior or failure modes that may be experienced by computer or electronic equipment are shown in Table 4.2 (Ref. 11). All the main variables that may cause damage resulting in down time are presented, including components of the equipment itself, and external environmental and support services. The principal sources of failure along with an estimate of probability of down time are shown. These estimates were based on engineering judgement and assume that other significant failure modes with longer down time have not also occurred, and that the computer facility has

TABLE 4.2 POTENTIAL EARTHQUAKE BEHAVIOR OR FAILURE MODES AND ESTIMATED OUTAGE FOR ELECTRONIC EQUIPMENT (Ref. 11)

BEHAVIOR OR FAILURE MODES	SOURCES OF FAILURE						OUTAGE TIME
	Vibratory			Mech.	Thermal	Elect.	Best Estimate
	Acc.	Disp.	Impact				
1. Raised floor							
Partial collapse	S		P				>5 da.
Total floor collapse	P						>1 mo.
2. Equipment support							
Catch in floor penetration		P					>4 hr.
Slide or roll into equipment wall		P	S				<24 hr.
Tip over (internal damage)		P					>5 da.
Buckle and collapse support pedestals	P						>24 hr.
Cables and hoses disconnect		P					>24 hr.
3. Equipment Frame							
Warp, rack, misalign	P	S	S				>5 da.
Doors pop off, hinges break	P	S	S				>5 da.
Segments separate (rupture internal cables and hoses)		P					>5 da.

TABLE 4.2 (Cont.)

BEHAVIOR OR FAILURE MODES	Acc.	Disp.	Impact	Mech.	Thermal	Elect.	Best Estimate
Frame fails, braces buckle	P	P					>5 da.
4. Internal Components							
Support braces and bolts fail	P	S					>24 hr.
Vibration isolators fail	P			S			>24 hr.
Head crash	P			S			>24 hr.
Circuit boards slide out	P					S	>4 hr.
Cooling hoses rupture		P		S			>4 hr.
ICUs overheat					P	S	>24 hr.
Cables short circuit		P				S	>24 hr.
Mass storage cartridges dump	P						>5 da.
5. External environment							
Falling ceiling and piping	P		S				>24 hr.
Dust and particles in air						P	-
Flooding - broken lines				P			>24 hr.
Interaction (impact) from other equipment			P				>24 hr.
Power surge (destroys ICUs)						P	>5 da.

TABLE 4.2 (Cont.)

BEHAVIOR OR FAILURE MODES	Acc.	Disp.	Impact	Mech.	Thermal	Elect.	Best Estimate
6. External support services and utilities							
Loss of primary power						P	>4 hr.
Loss of emergency power						P	>24 hr.
Loss of primary coolant				S	P	S	>4 hr.
Loss of room air conditioning					P	S	>4 hr.
Loss of telecommunications						P	>4 hr.

P = Primary source or cause of behavior or failure
S = Secondary source or cause

readily available support staff and spare parts to inspect and repair the damage. Failures due to human error and negligence are not identified, however, these may contribute to earthquake failure modes.

4.3.2 Methods of seismic design

The methods for seismic design of electronic and computer equipment have to consider the potential modes of failure listed in Table 4.2. These methods may vary significantly depending on the part of the equipment being designed: enclosing structure or internal parts. Included among the different methods of analysis and testing are: static building code analysis, rigid body dynamic analysis, elastic and nonlinear dynamic analysis, dynamic testing of components and dynamic testing of the assembled unit.

4.3.2.1 Static building code analysis

The static method of analysis applies only to the design of the rigid connections between equipment and supporting structure, and to the design of equipment cabinet frame and frame bracing. The method is not applicable when the equipment is supported on leveling pads, skids, spring isolation devices or rollers. Advantages and limitations of this method are the same listed previously in this chapter for the design of raised floors .

4.3.2.2 Rigid body dynamic analysis

This is a simplified dynamic analysis that emphasizes the mode of equipment motion during an earthquake: rocking about one edge without sliding, sliding on the floor without rocking, rocking and sliding, or movement with the floor, without sliding or rocking. The equipment is idealized as a rigid rectangular box supported on a rigid floor. The model considers only one vertical and one horizontal floor motion component. The basic parameters used to solve the dynamic equations resulting from this modelling are aspect ratio (normally height of the center of gravity of the equipment divided by one-half the base width of the equipment), peak horizontal and vertical floor accelerations, and coefficient of friction between equipment base and floor surface. This method provides a simple way of identifying the potential modes of equipment response and a relatively realistic assessment of the extent of motion and the need for tethering or anchoring to prevent excessive sliding or overturning. The method does not account for cabinet or frame distortions when the equipment is rigidly anchored to the raised floor, nor does it account for impact loading on the internal components of the equipment resulting from equipment motion.

4.3.2.3 Elastic and nonlinear dynamic analysis

A numerical dynamic computer analysis may be performed using two-dimensional linear elastic and/or nonlinear finite element models capable of representing large deformation behavior of the equipment. Recorded or computed earthquake motions of the building can be used in the analysis. This method is usually suggested for the design of equipment supports, especially when an accurate assessment of anchoring forces is required. The dynamic finite element analysis can provide a better estimate of potential frame distortions, areas of overstress in connections, and peak accelerations and displacements that might induce equipment malfunction. The disadvantages of this method are that it cannot assess the thermal or electrical performance of the equipment, and is dependent on the realism of the computer model used.

4.3.2.4 Dynamic testing of components

Component testing is mainly applied to the various internal components of the unit. Dynamic vibratory motions are specified and programmed into a shaking table to simulate the motion at the anchorage points of the component in the equipment cabinet. The component is tested either under normally functioning conditions, or under a wide range of vibrations to assess its vulnerability at any frequency and amplitude of motion. Disadvantages of this test method, besides its cost, are that it does not verify that all elements of the system have an adequate margin of safety to remain functional under earthquake conditions. New tests must be performed if any of the components are altered.

4.3.2.5 Dynamic test of assembled unit

The safest and surest way to verify the functional performance of electronic equipment under vibratory motions is to conduct a full-scale assembly test with dynamic vibrators or shaking tables. The assembled unit test provides information that cannot be obtained by the simple component tests, but it is generally time-consuming, and requires costly laboratory instrumentation and recording processes.

4.3.3 General design guidelines

The following are basic factors that should be considered in the seismic design of computer and electronic equipment: amplitude and frequency characteristics of ground motion at the site, location of equipment in the building and motion characteristics at the floor level, design strength and stiffness of the raised floor supporting the equipment, method of equipment support, geometry of equipment and internal mass distribution, strength and stiffness of cabinet/frame, vulnerability of internal components to vibration and impact, location of equipment inside the room and possible interaction with other equipment or structural/nonstructural elements, and extent of analysis, testing and inspection required for equipment design and installation. Some of these factors have already been discussed in previous sections of this chapter. Specific design guidelines for new and existing equipment are presented in the next sections.

4.3.3.1 Design guidelines for new equipment

According to Ref. DPF, minimal potential damage and loss of function should result if the following strategy is adopted when designing and placing new equipment:

a. Equipment should be located on the ground floor or basement of the building, thus reducing amplification of ground motions with height.

b. Raised floors should be designed following the guidelines given in Section 4.2.

c. Adequate rattle space should be provided around the equipment to prevent it from impacting adjacent equipment or structure. As an approximate guide, equipment is considered closely spaced at 1/2 to 1/2 feet, moderately spaced at 1/2 to 3 feet, and adequately spaced at more than 3 feet on all four sides. These ranges apply to equipment placed at ground level and should be increased if the equipment is placed on higher levels in the building.

d. Internal components should have as low a vulnerability to earthquake vibration and other related sources of failure as practical.

e. Equipment cabinet should have sufficient strength and stiffness to prevent component damage and loss of function.

f. The combination of cabinet geometry and equipment support or attachment should provide safe stable equipment behavior.

Suggested types of equipment support or anchorage are the following:

a. Fixed anchorage or bracing: Figures 4.3 and 4.4 illustrate different types of equipment with through the floor anchorage and vertical seismic bracing. These schemes may be used when the analysis shows that the equipment will tip, when the equipment is closely spaced and there is a high possibility of impact, when the equipment is located in upper floors of the building, when the internal components have low vulnerability to vibrations, and when the cabinet frame has sufficient strength and stiffness to support equipment without additional supplemental bracing.

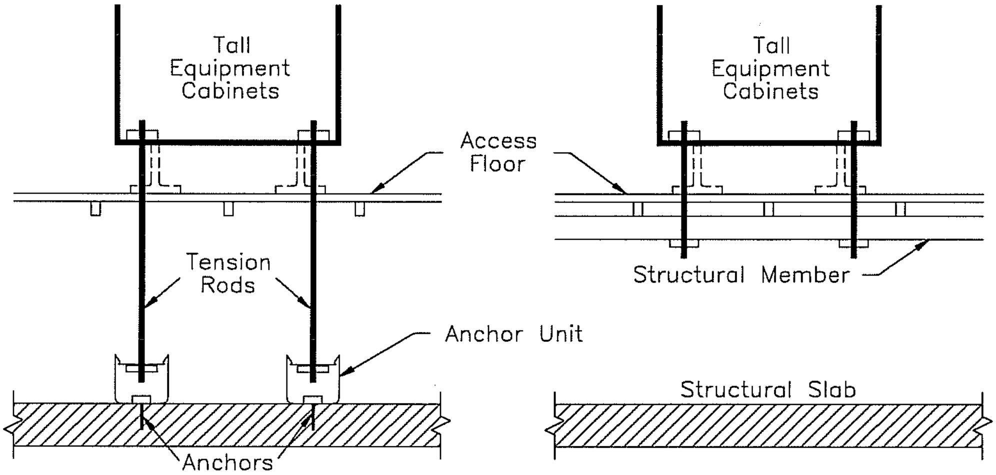


Figure 4.3 Computer Equipment Through Floor Anchorage (Ref. 11)

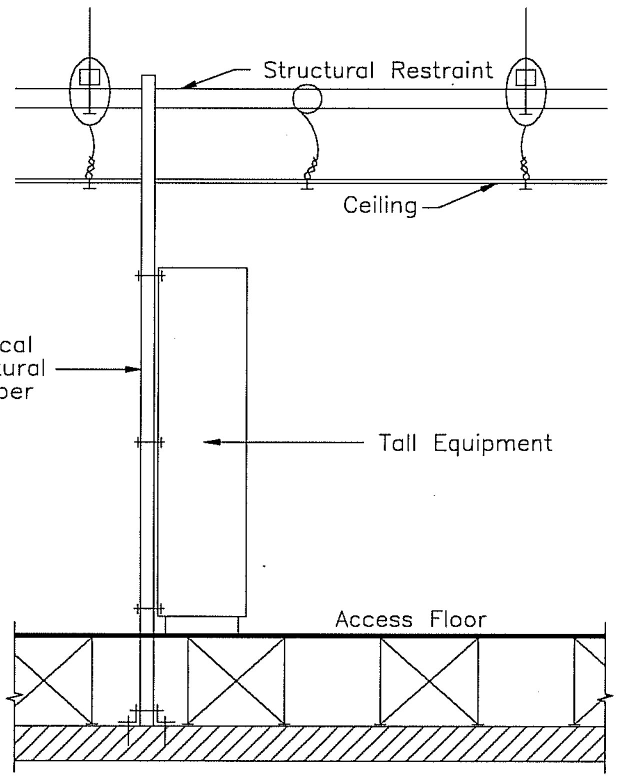


Figure 4.4 Computer Equipment Vertical Seismic Bracing (Ref. 11)

b. Isolation from horizontal floor motions: Equipment is supported on low-friction seismic leveling pads or casters that allow it to move relative to the floor with minimal load transfer at low levels of floor acceleration (Figure 4.5). This method can be used under the following conditions: dynamic stability analysis indicates the equipment will not tip over if it is supported on casters or low-friction leveling pads, there is adequate space for the equipment to move without impacting other objects, a tether is introduced to limit the distance to roll (Figure 4.6), floor penetrations have been guarded, equipment cabinet is weak and flexible and cannot be strengthened and stiffened sufficiently to endure strong shaking, internal components are highly vulnerable to horizontal floor motions, computed earthquake motions are intense, and the raised floor system is unable to resist large lateral equipment loads without possible collapse.

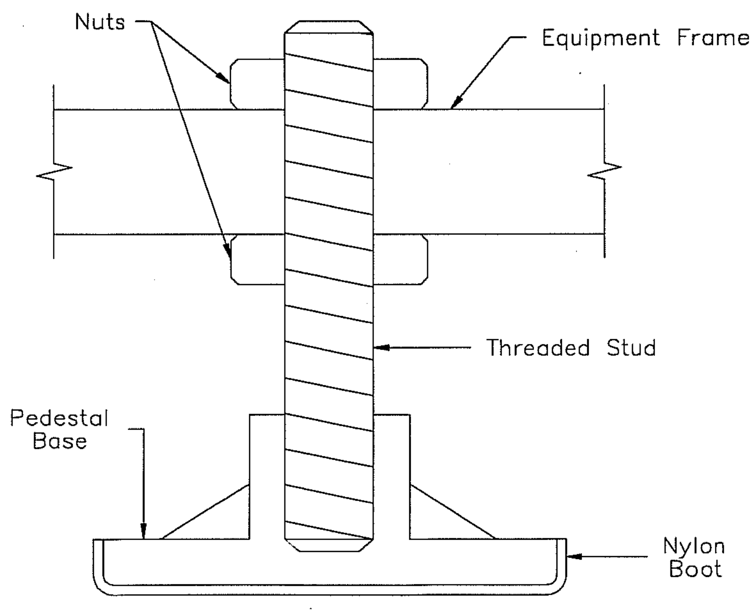


Figure 4.5 Leveling Pad for Seismic Zone (Ref. 11)

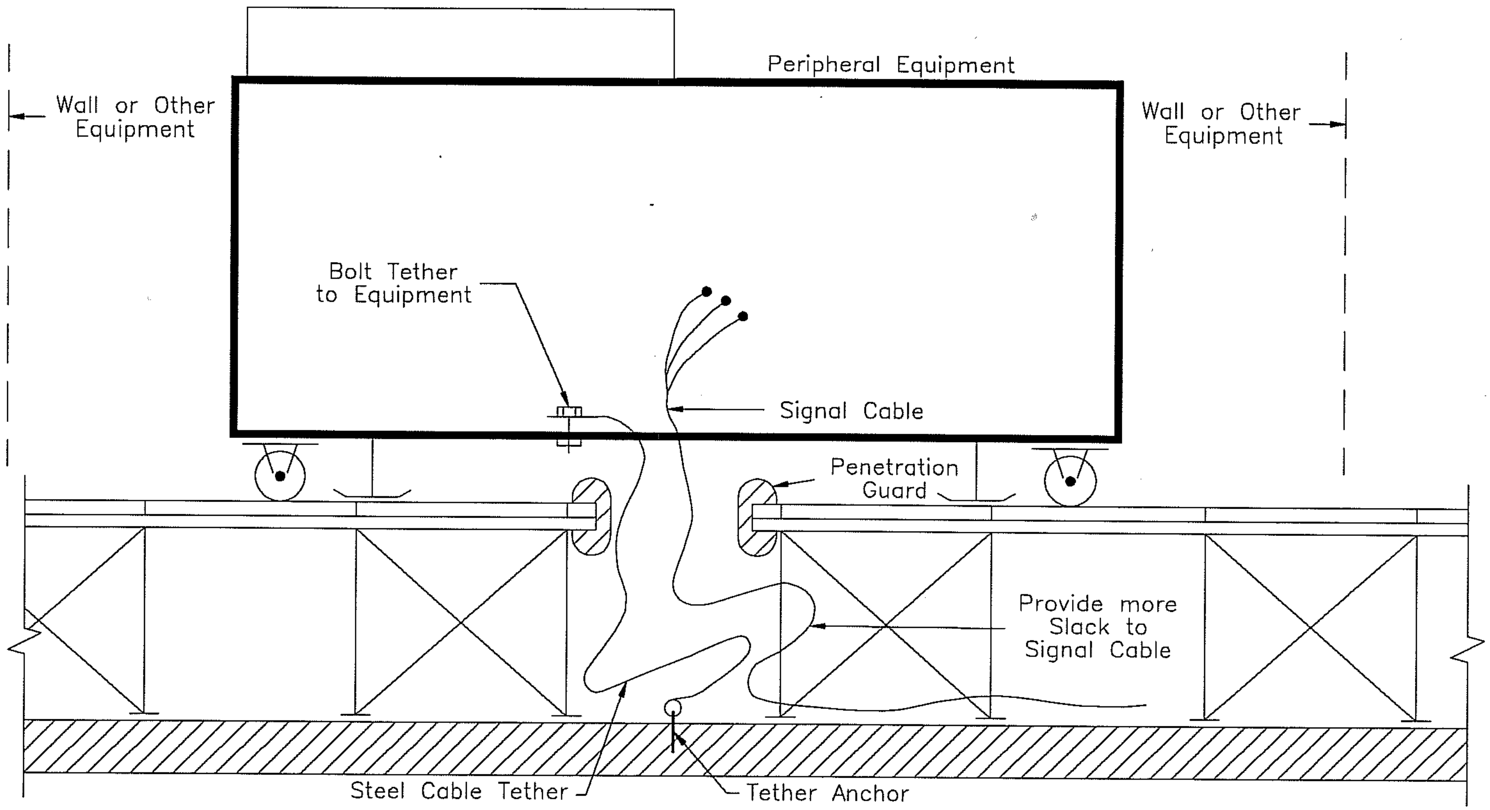


Figure 4.6 Computer Equipment Steel Cable Tether (Ref. 11)

c. Support on high-friction skids and leveling pads: This method of support should be used when dynamic stability analysis indicates the equipment will not tip or slide, when the support system will not fail due to collapse or disengagement of the leveling pads, when there is adequate space to prevent other items from impacting the equipment, when the cabinet is strong and stiff enough to resist the motions without excessive distortion, and when the internal components have low vulnerability to vibration effects.

4.3.3.2 Design guidelines for existing equipment

Guidelines for seismic upgrade of existing equipment are similar to those used for new equipment. Though the areas for potential upgrade are normally limited to the raised floor and the equipment support or attachment to the floor. Seismic upgrading of the raised floor system has already been discussed in Section 4.2. Seismic design strategies for existing computer equipment are summarized in Table 4.3 for typical equipment types. These strategies assume that the equipment geometry, cabinet strength, and vulnerability of internal components cannot be readily modified, which is normally the case for existing equipment. When applying any of the design strategies, or in general when upgrading the equipment to improve its seismic behavior, the designer should make sure that the solution does not create other problems that may cause failure of the equipment.

Among other possible solutions to common equipment problems are tying equipment together when enough movement space cannot be provided, or installing impact bumpers on the equipment to minimize shock loads from rolling or tipping impact. The use of easily accessible clamps rather than bolts is suggested to attach frames together. Free casters or caster clips that tend to eject during earthquake motion resulting in free casters should be eliminated. Casters with screw type locks should be considered. Skirts on machines which get damaged easily when machines roll around should not be used. When levelers are necessary, enough time should be taken to install them properly. Small diameter levelers are not recommended. Levelers should be strong enough to hold up the machines without bending under earthquake motions and allowing the machine to roll on its casters. Loose machines such as uninstalled box or book carts should be restrained by tethering them to a bar on the wall or floor. A parking area could be designated to place such elements. Locked casters could also be used as restraining devices. A curbing or restraining molding should be placed around cable holes to restrict the equipment from rolling into the holes during an earthquake (Figure 4.6).

4.3.4 Inspection

A program of systematic review and inspection of all computer equipment should be included as part of the design and construction process. Also, due to the frequent movement of equipment around floor areas, it is necessary to have a program of inspection developed and put in practice to guarantee proper installation and anchorage of equipment. Periodic inspections should also be conducted to verify the condition of all anchorages and connections of the equipment.

TABLE 4.3 SEISMIC DESIGN STRATEGIES FOR EXISTING COMPUTER EQUIPMENT (Ref. 11)

EQUIPMENT TYPE	ESTIMATED EQUIPMENT CHARACTERISTICS			Internal component vulnerability	Suggested seismic design strategy	Comments
	Support or attachment	Cabinet geometry (aspect ratio)	Cabinet strength and stiffness			
1. Mainframes						
Processors	Leveling pads	Average to tall (1.5 to 2.5)	Weak and flexible	High	Anchor or brace/Isolate	Impact and vibration sensitive
Power distribution unit	Leveling pads	Average (1.5 to 2.0)	Medium	Medium	Anchor or brace	
Coolant distribution unit	Leveling pads or skids	Average (1.5 to 2.0)	Medium to strong	Low	Anchor or brace	
Control console	Leveling pads	Low (0.5 to 1.0)	Strong	Medium	Anchor or brace	
2. Peripherals						
Disk drives	Leveling pads	Average (1 to 2)	Medium	High	Isolate	Vibration sensitive
Disk drive controllers	Leveling pads	Average to tall (1.5 to 2.5)	Weak to medium	Medium to high	Isolate	
Magnetic tape drives	Casters and clips	Average to tall (1.5 to 2.5)	Medium	High	Isolate	Vibration sensitive

TABLE 4.3 (cont.)

EQUIPMENT TYPE	Support or attachment	Cabinet geometry (aspect ratio)	Cabinet strength and stiffness	Component vulnerability	Seismic design strategy	Comments
Tape drive controllers	Leveling pads	Average to tall (1.5 to 2.5)	Weak to medium	Medium to high	Isolate	
Card reader	Skids/pads	Average (1.5)	Strong and stiff	Low	Leave as is	
Card punch	Skids	Low (1.5)	Strong and stiff	Low	Leave as is	
Communications controller	Leveling pads	Average to tall (1.5 to 2.5)	Medium to weak	High	Anchor or brace/Isolate	
High speed printer	Leveling pads	Average to tall (1.5 to 2.5)	Weak to medium	Medium	Isolate	Alignment sensitive
Printer controller	Leveling pads	Average (0.5 to 1.0)	Weak to medium	High to medium	Isolate	
Switching management system	Skids	Average	Medium	Medium	Anchor or brace/Isolate	
3. Mass storage facilities	Leveling pads	Average to tall (1.5 to 2.5)	Medium to weak	High	Anchor or brace	Alignment sensitive
4. Telecommunications gear						
Modem cabinets	Skids/Leveling pads	Tall (>2.5)	Medium to weak (flexible)	High	Anchor or brace	
5. Air handling units controllers	Leveling pads / Support frame	Tall	Medium to strong	Low	Anchor or Brace	

4.4 TAPE AND DISK STORAGE

4.4.1 Description of tape and disk storage types

The demand for storage space for data bases generated by electronic data processing equipment has resulted in taller racks, more closely spaced shelving, and space-saving track mounted racks. The typical fixed tape storage racks and shelving systems are shown in Figure 4.7. These include: the pendulum-suspended, double-sided, tree-column rack (Figure 4.7.a), the shelf-supported, double-sided, tree-column tape rack (Figure 4.7.b), the pendulum-supported, single-sided or half-sided tree column rack (Figure 4.7.c), the enclosed shelf storage unit with either an open front, top closing shelf doors, vertically sliding door, or cabinet door (Figure 4.7.d), and the cubbyhole shelf storage rack for tape cartridges (Figure 4.7.e). All these units are designed to be supported on raised floors or structural floor slabs, and to be anchored to the floor or braced to an adjacent wall. The first three rack systems can be attached to a mobile track-mounted unit.

Magnetic storage disks are commonly stored in shelf units similar to those shown in Figure 4.7.d. Normally, these shelf units have doors on the front to protect the disk packs from falling out.

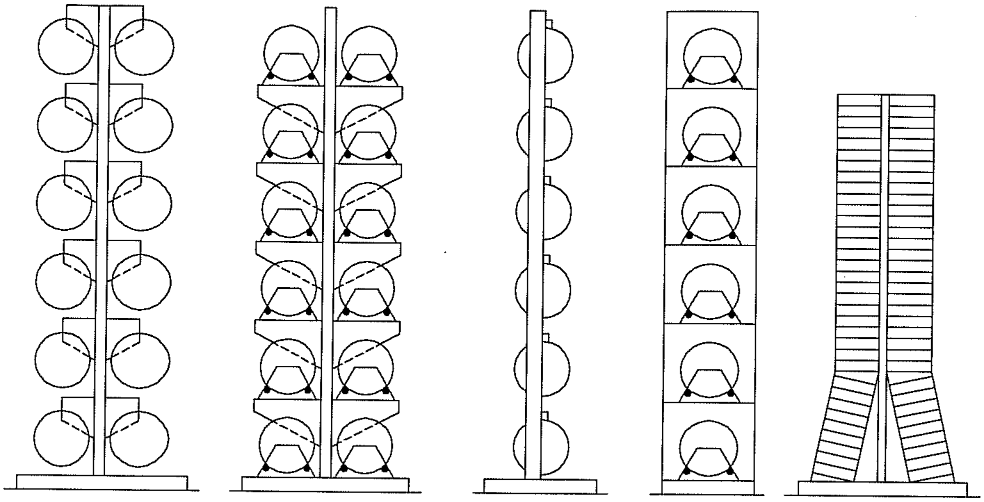


Figure 4.7 Tape and Disk Storage (Ref. 11)

4.4.2 Methods of design and analysis

4.4.2.1 Static lateral force

The simplest method for evaluating seismic design of tape storage racks is the static lateral force procedure. Selection of the design coefficient (see Chapter III) as well as distribution of the lateral force with height requires a thorough understanding of the dynamic behavior or the tapes and rack system.

4.4.2.2 Dynamic analysis

An approximate linear dynamic analysis may be performed to realistically assess the lateral force and its distribution in the rack system. Vertical and horizontal earthquake motions may be applied to the model. A nonlinear analysis could also be performed to represent swinging and impacting of tapes against the rack system as well as tipping of the rack on its support.

4.4.2.3 Dynamic test

Dynamic shaking table tests may be used for tape storage racks and disk storage shelf units to simulate their performance under earthquake motions. These tests provide a realistic assessment of the complex dynamic behavior/interaction of the tapes and their support system.

4.4.3 General design guidelines

Some of the basic design considerations that must be taken into account to provide seismic safety to the tape and disk storage systems are: provision of adequate constraint to prevent the tapes and disks from falling from the shelf or impacting the rack, design of the rack or shelf unit assuming that all the tapes remain in place, seismic loads imposed by the rack or shelf system should not exceed the capacity of the supporting building elements, including raised floors, and rolling rail or caster-mounted racks must remain stable or upright without dumping the tapes.

Some measures that can be used to achieve adequate performance of tape and disk storage systems are the following. The most obvious system to prevent the tapes from rolling off the shelves or bouncing free from pendulum support rails is to provide an enclosed cabinet. This solution is not always feasible due to the added cost in terms of labor to access the tapes. Other alternatives have been developed, such as sloping shelves or wire racks, or passive seismic straps, but all have the same disadvantages mentioned before. Schemes that have been suggested by rack manufacturers include an active seismic constraint, consisting of a strap that would fall in place under small amplitude earthquake motions; and to support the entire tape rack system on low-friction bearings, a form of base isolation, to limit the seismic force that would act on the rack and the tapes. If this last scheme is used, a dynamic nonlinear analysis should be employed to verify that the system works.

Wherever possible, heavy storage racks should be supported directly on the structural slab. If they must be supported on the raised floor, additional pedestals and seismic bracing should be provided in the more heavily loaded areas. When rows of racks are tied together overhead with horizontal bracing, these braces should be adequate anchored to adjacent structural walls, or provide a vertical system of structural supports running from the structural floor to the roof.

For rolling tape units, adequate space must be provided to allow them to move without impacting adjacent equipment or structural members. Adequate braces tying the individual units together to avoid overturning, overhead guiderail bracing and/or floor rail guides and safety guards should be provided to avoid derailing the units.

4.5 CEILINGS

4.5.1 Description of the system

In general, the many different types of suspended ceilings can be divided into solid, constructed from gypsum board or plaster, and panelized, made of suspended prefabricated squares or rectangles of light insulating material. The most common ceiling type in industrial facilities is the lay-in or tee-bar ceiling. This type of ceiling system is composed of suspended sheet metal inverted tee shaped sections called runners. The runners support the ceiling elements or panels, light fixtures and mechanical grills. This type of ceiling system is the most susceptible to damage during an earthquake because of the light sections used and the lack of in-plane continuity and stiffness.

4.5.2 General design guidelines

Design of suspended ceilings should be performed following the methods suggested in Chapter III, either a static lateral force analysis or a more complex dynamic analysis according to the importance assigned to the system. When estimating the weight of the ceiling system, all the elements that will be supported on it should be included: light fixtures, grills and any other electrical or mechanical element.

Differential swaying or swinging motion in panelized ceilings can cause gaps to open between support members. This will allow the ceiling panels and other suspended elements to fall. This problem can be minimized by diagonally bracing the main ceiling suspension runners to the structure above, by reinforcing the inter-ties between runners, or by physically attaching ceiling elements to the runners. Without diagonal bracing, the ceiling swinging motion will be incompatible with the surrounding partitions, and damage will occur at the ceiling perimeter. For large suspended areas, bracing is required to avoid damage due to pounding with walls, columns or other non-yielding ceiling penetrations.

All types of ceiling systems should comply with the provisions of current building codes regarding their design and installation. The following suggestions can be used to minimize damage to suspended lay-in panel systems:

a. Suspended ceiling systems must be laterally braced to limit lateral movement in earthquakes. Diagonal wires and compression struts as shown in Figure 4.8 are suggested.

b. When possible, the ceiling system should not be fastened to the surrounding walls or partitions. Where the ceiling must join a wall or partition, an angle wall trim, wide enough to allow for differential movements, should be provided. Main and cross runners should have hangers at the perimeter so that wall trims do not support the ceiling. Additional hangers, struts or braces should be provided as required at all ceiling breaks, soffits or discontinuous areas. Splices should not be permitted in any hanger wires.

c. Ceiling grid members may be attached to not more than two adjacent walls. Ceiling grid members should be at least 1/2 inch free of other walls. If walls run diagonally to ceiling grid system runners, one end of main and cross runners should be free a minimum of 1/2 inch from the wall.

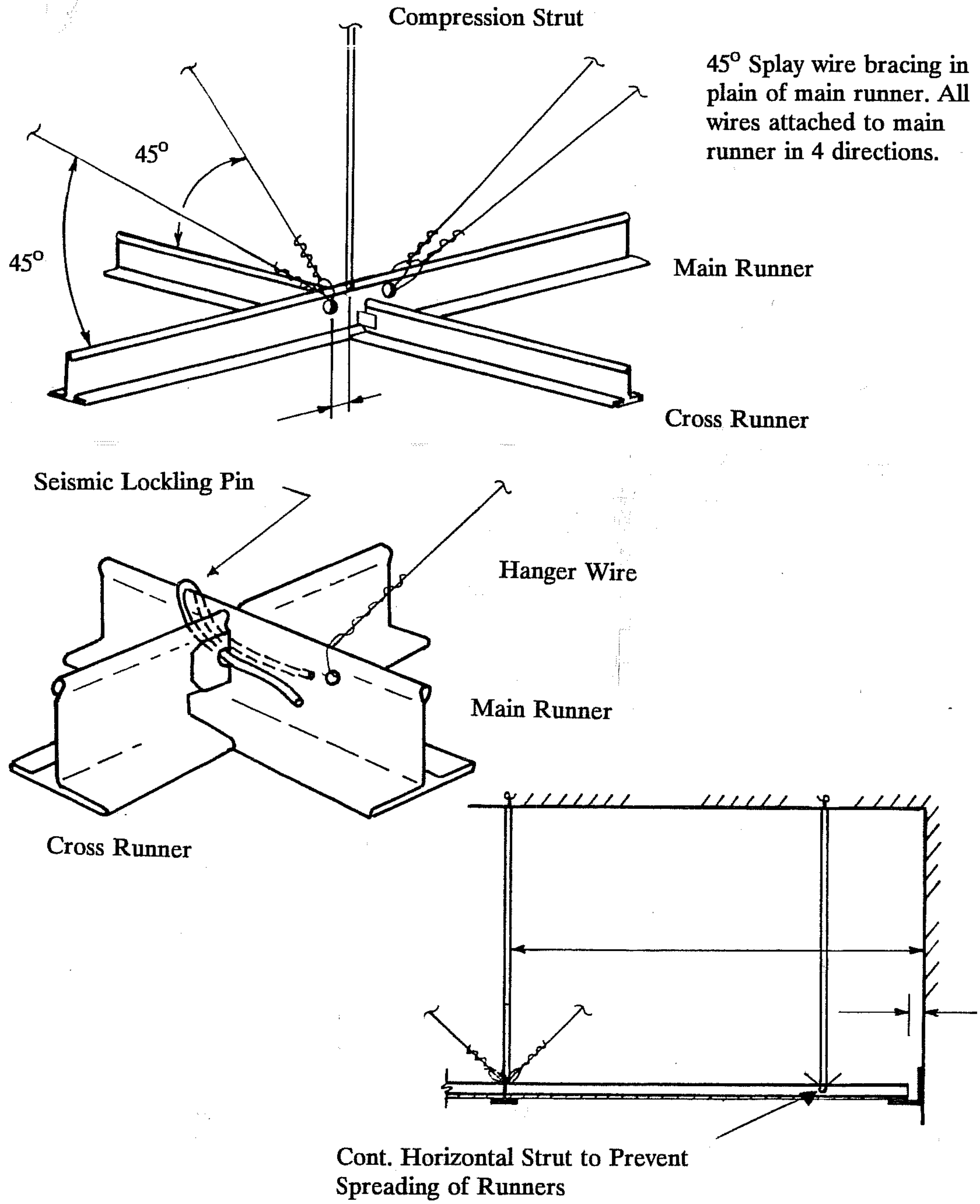


Figure 4.8 Bracing Details for Suspended Ceiling Systems (Ref. 4)

d. Cross runners should be fastened to the main runners using locking clips or similar devices to prevent cross tees pulling or twisting out of the main runners. Interconnection between runners at the free ends should be provided to avoid lateral spreading.

e. In Seismic Zones 3 and 4, a compression strut at each intersection of bracing wires should be provided.

4.6 PARTITIONS

Partitions in buildings with flexible structural frames should be anchored to only one structural element, such as a floor slab, and separated by a physical gap from all other structural elements (Figure 4.9). Reinforced concrete masonry units attached to more than one structural element should be considered as structural elements and designed as such. Unreinforced concrete masonry units should not be used for partitions or filler walls.

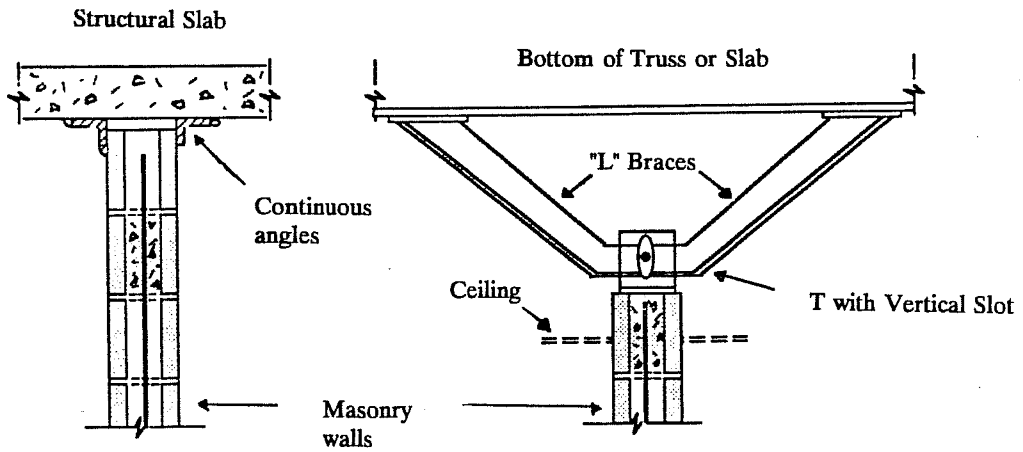
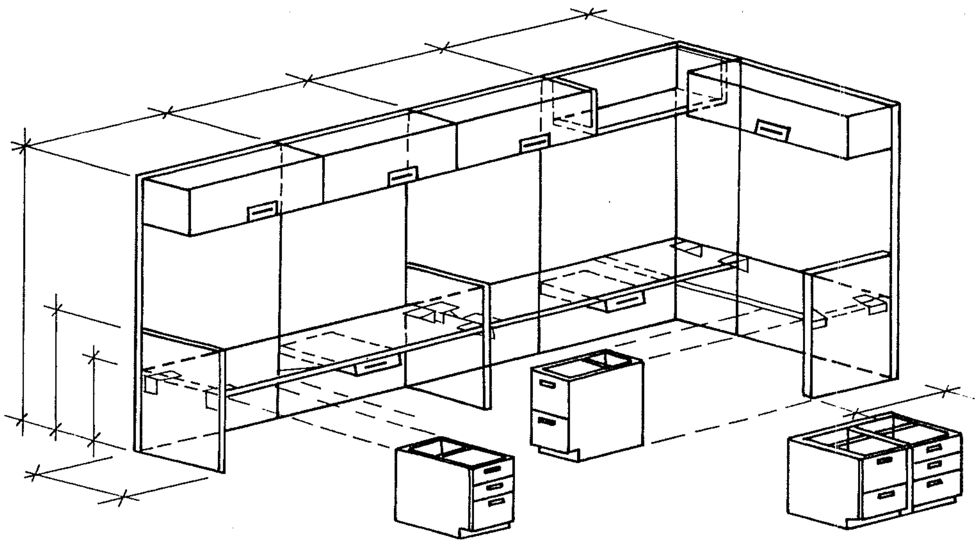


Figure 4.9 Lateral Supports for Nonstructural Partitions (Ref. 4)

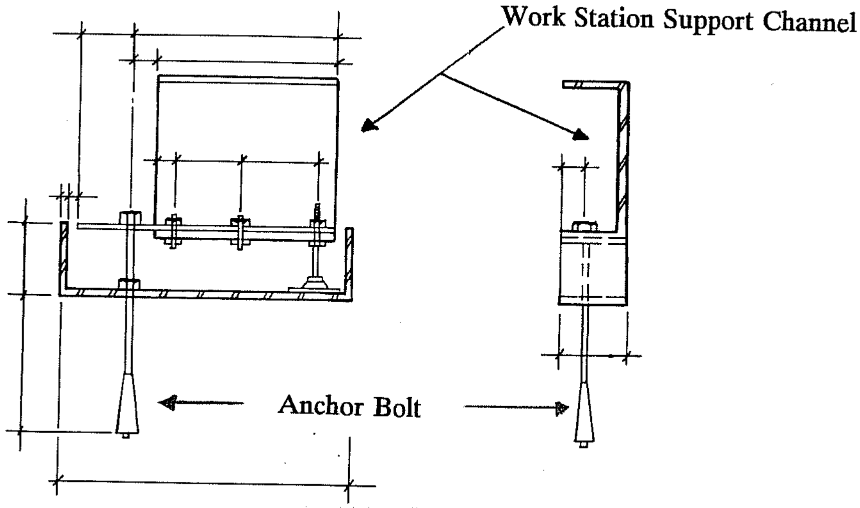
Gypsum board or plaster partitions supported at top and bottom generally sustain minimal damage during earthquakes, unless the structure itself is severely damaged or distorted much in excess of code allowable deformations. The main damage to these partitions consists of cracking. This may not be of great concern except for the dust created which can cause damage to computer equipment. The walls can be covered with a material able to distort without fracturing, so dust would be confined within the wall.

Ceiling supported partitions are susceptible to damage and overturning failure, especially if the ceiling is of panelized type. If the ceiling is not adequately suspended or braced, the partitions should have independent bracing along the top track. Cantilever or movable partitions usually gain lateral support from base brackets or from attachments to perpendicular walls. Such devices must be designed to resist the forces induced by seismic motions, especially if the partitions support any shelving. Partitions on access floors are particularly vulnerable to seismic damage due to the concentrated loading on already vulnerable access floor systems. The response of the partitions and the raised floors as a unit should be considered when designing both systems.

Work stations can be viewed as a form of partition. Work stations usually consist of metal or wood walls with overhead shelves, desks and drawers attached to the walls. Typical configuration and anchorage to the structure are shown in Fig. 4.10. Work stations have to be designed to resist lateral loads resulting from seismic motions, normally computed using the worst configuration possible, that is the maximum number of overhead shelves and weight that can be applied to the work area. They should be adequately anchored to transmit those forces to the structural slab.



A. Typical Work Station



B. Typical Anchor Bolt Detail

Figure 4.10 Work Station Configuration and Anchorage

4.7 MECHANICAL AND ELECTRICAL SYSTEMS

4.7.1 General description

Mechanical and electrical systems often have a greater potential for damage because the structural engineer has little influence over the methods used for design and installation. Most electrical and mechanical equipment is purchased as premanufactured items rather than being designed specifically for a project. Thus, the manufacturer establishes the particular characteristics of the equipment, and these in turn, determine the damage potential during an earthquake. Typical damage suffered by electrical and mechanical equipment was shown in Chapter II.

4.7.2 General guidelines for design and installation

The design of mechanical and electrical equipment, including enclosing structure, contents, and anchorage to structural elements should be performed using the methods described in Chapter III. The selection of the specific method will depend on the importance of the element and the degree of earthquake protection desired.

The following suggestions are based on characteristics of installation that have proved to work during past earthquakes.

4.7.2.1 Equipment with vibration isolation

Vibration isolated equipment can be mounted either directly on the floor, or it can be hung from structural elements on the building. Vibration isolated floor mounted equipment are usually units containing internal moving parts such as pumps, motors, compressors, and engines. All vibration-isolation mounts should be carefully analyzed for earthquake resistance. Generally, the isolation material has poor lateral load resistance capacity and often its presence significantly increases the seismic response of the equipment. Also, the isolator housing tends to overturn easily. It is necessary to either supplement conventional isolators with separate stops, or install isolators that have built-in restraints and overturning resistance designed for use under seismic loading. Careful examination of the anchorage of the isolator to the structure is necessary because often its capacity is not specified by the manufacturer. Additionally, the forces produced by seismic motion may require separate restraints or snubbers to be provided to control the energy stored in the isolation springs. Snubbers with restraint in three dimensions are better because fewer are required. Typical anchorage details are given in Figure 4.11.

Vibration isolation hung elements are the most difficult to restrain, especially if only a small movement can be tolerated. The system that has proved to be the best is to place an independent laterally stable frame around the equipment with proper operating gaps padded with resilient material, similar to a snubber. Otherwise, vibration-isolation hangers for suspended equipment should be tightly installed against the supporting structural member. For lightweight equipment, cross bracing should be provided between hanger rods on all four sides of the suspended element. An alternative for heavy equipment is to provide a self contained laterally stable suspended platform upon which conventional seismic isolators or snubbers can be mounted. Typical restraining details for a vibration isolation hung element is shown in Figure 4.12.

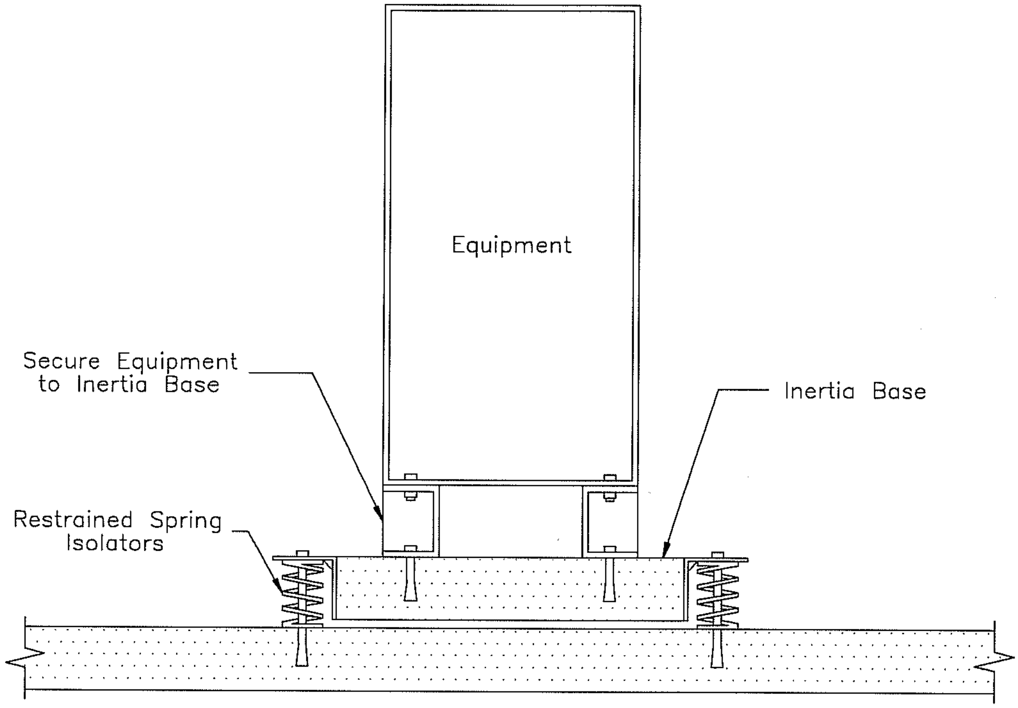


Figure 4.11 Equipment on Vibration Isolated Inertia Base (Ref. 4)

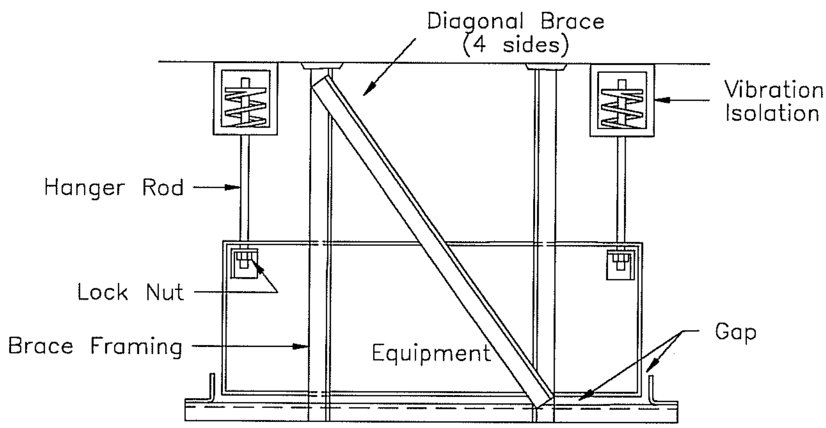


Figure 4.12 Hung Equipment with Vibration Isolation (Ref. 4)

4.7.2.2 Equipment without vibration isolation

Excessive movement of equipment placed directly on the floor can cause damage to the equipment itself, but generally most of the damage is caused to the connected services. Floor mounted elements should be designed to withstand earthquake forces and should be anchored to the floor or otherwise secured. Anchoring can be accomplished using cast-in-place anchor bolts and other inserts, or drilled-in concrete anchors. For elements with high center of gravity, bracing at the top should be provided to prevent overturning. Braces can be placed diagonally to the floor, to the structure above, or to adjacent structural walls if they can resist the resulting lateral forces. Threaded pipe should not be used for tank or equipment legs since the weakened plane created by the threads has lead to several failures in the past (Ref. 19). Any frame supporting elevated tanks or equipment should be adequately braced and anchored to the structural slab and walls. Figure 4.13 shows typical anchorage details.

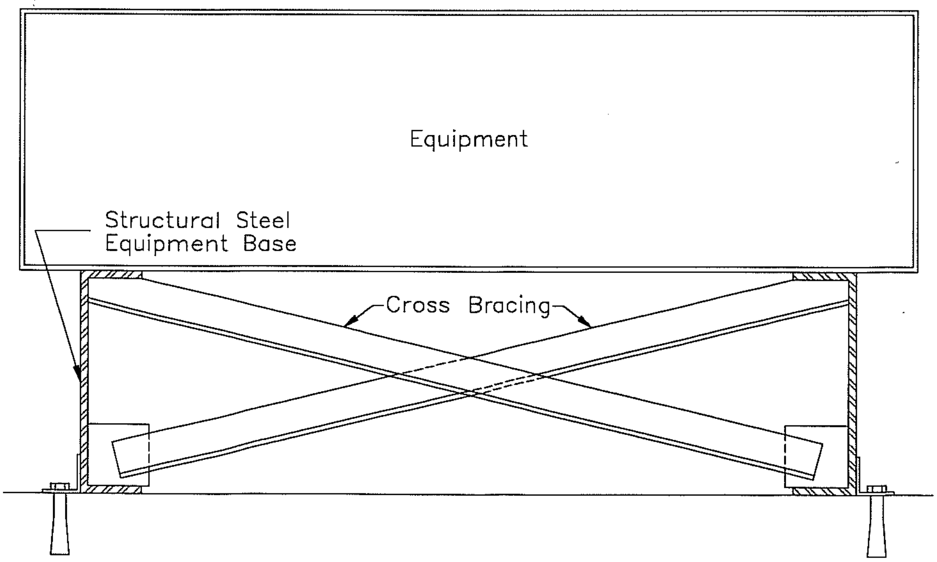


Figure 4.13 Anchorage of Elevated Equipment

Fixed suspended equipment can suffer excessive swinging movement that can damage connections, cause damage by impacting other elements, or induce failure of the suspension system posing life safety hazards. Suspended tanks and heavy equipment should be strapped to their hanger systems and provided with lateral bracing in all directions (Figure 4.14). Where possible, tanks or other equipment should be installed tightly against the structural member above, eliminating the need for braces, and secured to the suspension system to prevent slipping.

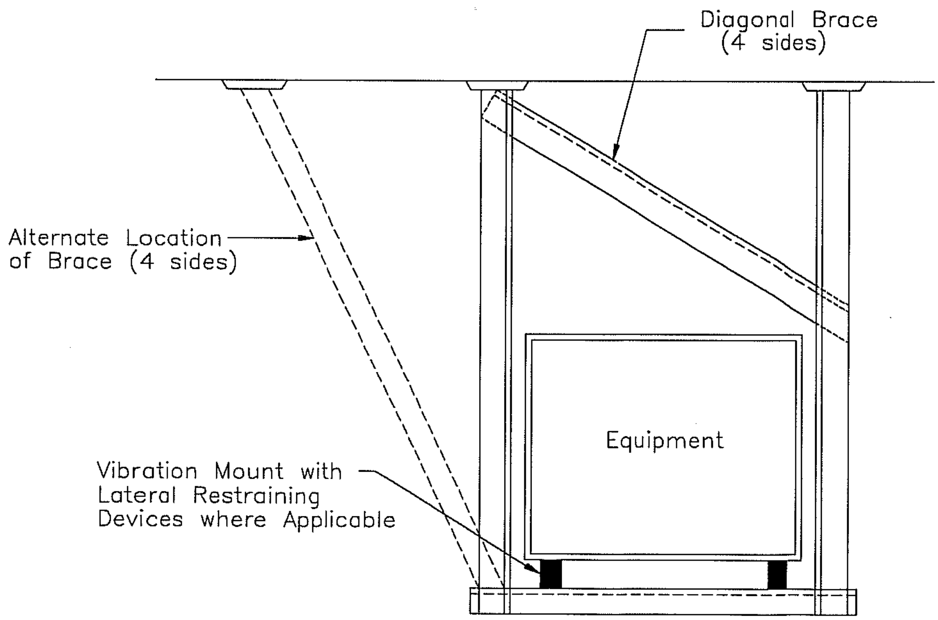


Figure 4.14 Bracing of Hung Equipment (Ref. 4)

4.7.3 Piping systems

Failure of piping systems usually occurs at or near connectors where the equipment is allowed to move, or where the main pipe is forced to move and small branches connected to the main pipe are clamped to structural elements. It is usually difficult to define a lateral restraining scheme for a piping system before its construction since its exact configuration is seldom known by the designer, and even if it were, key brace locations are not easy to determine. Current practice specifies provision of complete restraint where seismic protection is required.

Pipelines should be tied only to one structural system. Where structural systems change and deflections are anticipated, movable joints should be installed to allow movement of the pipes. Suspended piping systems should have consistent freedom of motion throughout. If the main pipe line is free to sway, branch lines should not be anchored to any structural system. When the pipe system is allowed to sway, movable joints should be provided at the equipment connections. Required operational movement of piping due to thermal and/or pressure loadings must also be considered.

Guidelines for the seismic restraint of ducts and pipes have been developed by the Sheet Metal and Air Conditioning Contractors National Association (SMACNA) and the Plumbing and Piping Industry Council (PPIC). These guidelines have been developed for use in Seismic Zones 3 and 4 and are generally accepted and widely used in areas of high seismicity. Their use is suggested for installation of piping systems in industrial buildings.

4.7.4 Air distribution systems

The following comments refer to ducted systems only. Seismic protection of duct work is more a matter of careful consideration of possible differential movements than anchoring and bracing to minimize response. Bracing should be considered near stationary equipment, or when duct swaying could damage adjacent elements. Braces are also needed when long hangers and supports for ductwork are used. Horizontal ducts should be supported as close as possible to the supporting structural element. Pipe sleeves or duct openings through walls or floors should be large enough to allow for movement of the pipes and ducts. Ceiling diffusers and registers should be secured to ductwork with sheet-metal screws. Diffusers connected to the flexible ducts should have positive ties to the ductwork and/or the wall opening.

Guidelines for the installation of air ducts are also given in the SMACNA manual and are suggested for use in industrial buildings.

4.7.5 Light fixtures

Seismic protection for light fixtures is achieved mainly through careful detailing and installation. Design and installation should follow current building code provisions. Some of the important aspects of installation of light fixtures are:

a. Recessed fixtures: These are the most common in lay-in ceilings. Recessed fixtures should be secured to and supported by a ceiling suspension system designed to carry their weight. Alternatively, independent safety supports can be provided. The UBC Building Code (Ref. 15) requires securing recessed lighting fixtures at diagonal corners by installing two 12 gauge wires connected directly to the structure. In all cases, these wires should be connected to the fixture, as specified in the code, and not to the tee-bars that support it. The wires should be able to resist at least four times the weight of the fixture. All hooks used to hang lighting fixtures should have safety latches.

b. Surface mounted fixtures: This type of fixture is usually undamaged during earthquakes because normal installation methods securely fasten the fixture to the ceiling system. The devices used to attach the fixture to the ceiling system should be able to withstant dynamic loads. Manufacturer's specifications must be examined to verify that all devices comply with this requirement.

c. Pendant fixtures: Usually the most susceptible to earthquake damage. During earthquakes, pendant fixtures can swing and hit adjacent elements or they can hit the ceiling. Long runs of interconnected pendant fixtures have also been shown particularly susceptible to earthquake damage. There is no positive way to prevent damage unless a lateral supporting grid is installed at the fixture level to prevent swaying. The practicality of this solution depends on the room size, fixture spacing, and type of surrounding walls. An alternative is to install safety cables through the supporting stems. This would prevent the fixtures from falling but the fixture would still be susceptible to damage.

d. Chain-hung fixtures: This type of fixture is also susceptible to earthquake damage due to swaying and pounding with adjacent elements. When installed near other building elements such as ventilation ductwork, suspended equipment, or building columns or walls, the fixture or group of fixtures should be laterally stayed by a rigid grid or taut cables at fixture level. The chains should be properly designed and open hooks to hold the fixture should not be used.

4.7.6 Building electrical systems

Critical building electrical distribution elements should be installed to be independent of the failure of other items. All electrical equipment such as transformers, switch gear, and control panels must be anchored or braced to the building, as shown previously in Figure 4.4. Flexible braided connections should be used, where possible, in place of rigid copper bus whenever relative movement may occur between switchboard components. Additional pull boxes with slack conductors should be provided in long conduit runs to avoid tensioning of conductors. Large rigid systems such as bus ducts should be braced when suspended.

Crossing of seismic joints with conduits and bus ducts should be avoided, otherwise arrangements to permit the required deflections should be used. Separate ground conductors should be provided in all conduit runs that cross seismic joints and elsewhere in the electrical system where grounding systems could be broken.

Emergency power generators should be mounted on adequately designed vibration isolators. The vibration isolators and the connecting service piping should be provided with horizontal restraints. Any other emergency power and lighting systems should be securely anchored and braced to the structure.

CHAPTER V

CASE STUDY OF A BUILDING HOUSING A "HIGH-TECH" INDUSTRY

5.1 INTRODUCTION

A study of the behavior of existing structures subjected to earthquake motions is needed to evaluate adequacy of current code design provisions. The response of a typical industrial building under different loading conditions will be evaluated. Expected performance under loads specified in current code provisions as well as response of the structure under observed ground motion will be investigated.

The study is divided into three sections: elastic analysis of the existing structure subjected to static lateral loads and recorded ground motions, inelastic analysis of the existing structure subjected to recorded ground motions, and inelastic analysis of the strengthened structure under recorded ground motion. The case study includes a description of the building's structural and architectural system. The nonstructural elements and equipment located inside are discussed. Different models used for the analysis, the computation of static lateral loads, and the characteristics of the ground motion are described. In the first section, the discussion is centered on performance of the structure with current code provisions. In the second and third sections, floor displacements, velocities, and accelerations computed considering inelastic action on the structure are presented. Of special interest is a comparison of design forces for nonstructural elements using the UBC Code and NEHRP Provisions, and those forces computed using the response of the structure under recorded ground motion.

5.2 DESCRIPTION OF THE STRUCTURE

The building is located in an industrial zone in a high seismic risk area. It is a typical office and/or light manufacturing building used by a data processing or "high-tech" industry (Figure 5.1). It was designed and built between 1982 and 1985. The building has three stories, with an area of approximately 71000 sq.ft. per floor. The central open area of 64000 sq.ft. (Figure 5.2) is the main working area, where movable partitions and manufacturing equipment are placed. The remaining space located at the east and west ends of the building, between axes A to C and T to V serve to locate elevators, stairs, restrooms and other services. The building has three stories: ground floor, interstitial level and upper level. The ground floor and upper level serve as work areas, while the interstitial level supports all the mechanical and electrical utility systems.

5.2.1 Structural system

The lateral load resisting system of the structure consists of braced steel frames and composite concrete slabs. The braced frames are formed with W-section beams and columns and square-tubular-section braces. The braced frames are located along axes 3, 5, 7 and 9 in the longitudinal direction. In the transverse direction, braced frames are located along axes C and T. Elevation of the braced frames is shown in Figures 5.3 and 5.4. In the central part of the building, between axes C and T, the vertical load resisting system consists of steel frames made with W-section columns and deep trusses located at the upper and roof levels. A typical frame is shown in Figure 5.5. The floor and roof trusses are made of steel angles. The connection between the trusses and the columns was designed and constructed to transmit vertical loads. Horizontal

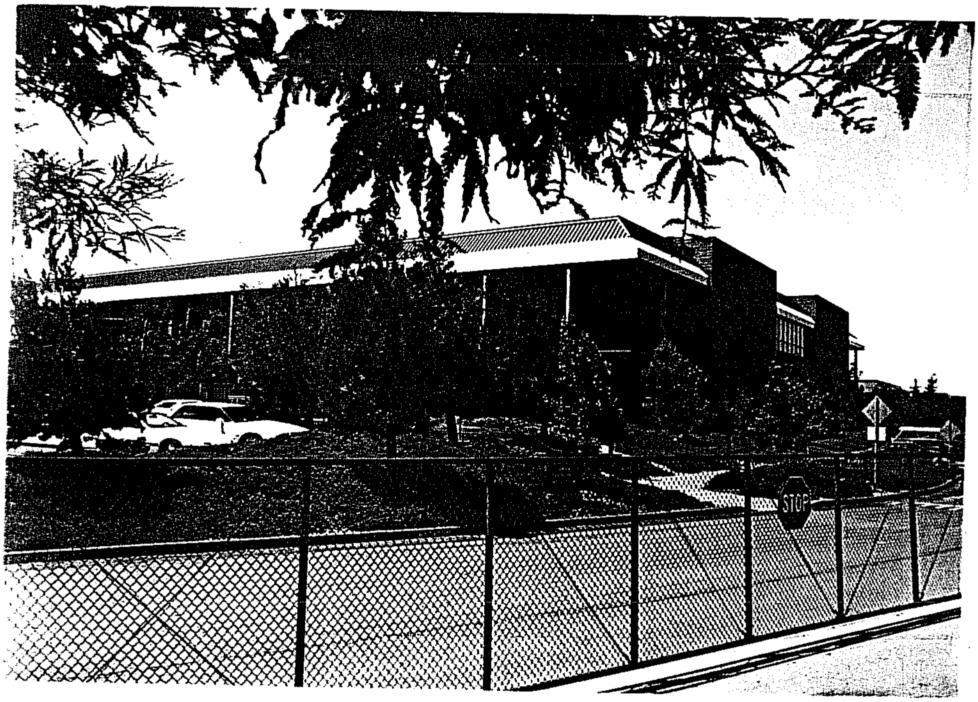


Figure 5.1 Typical Industrial Building

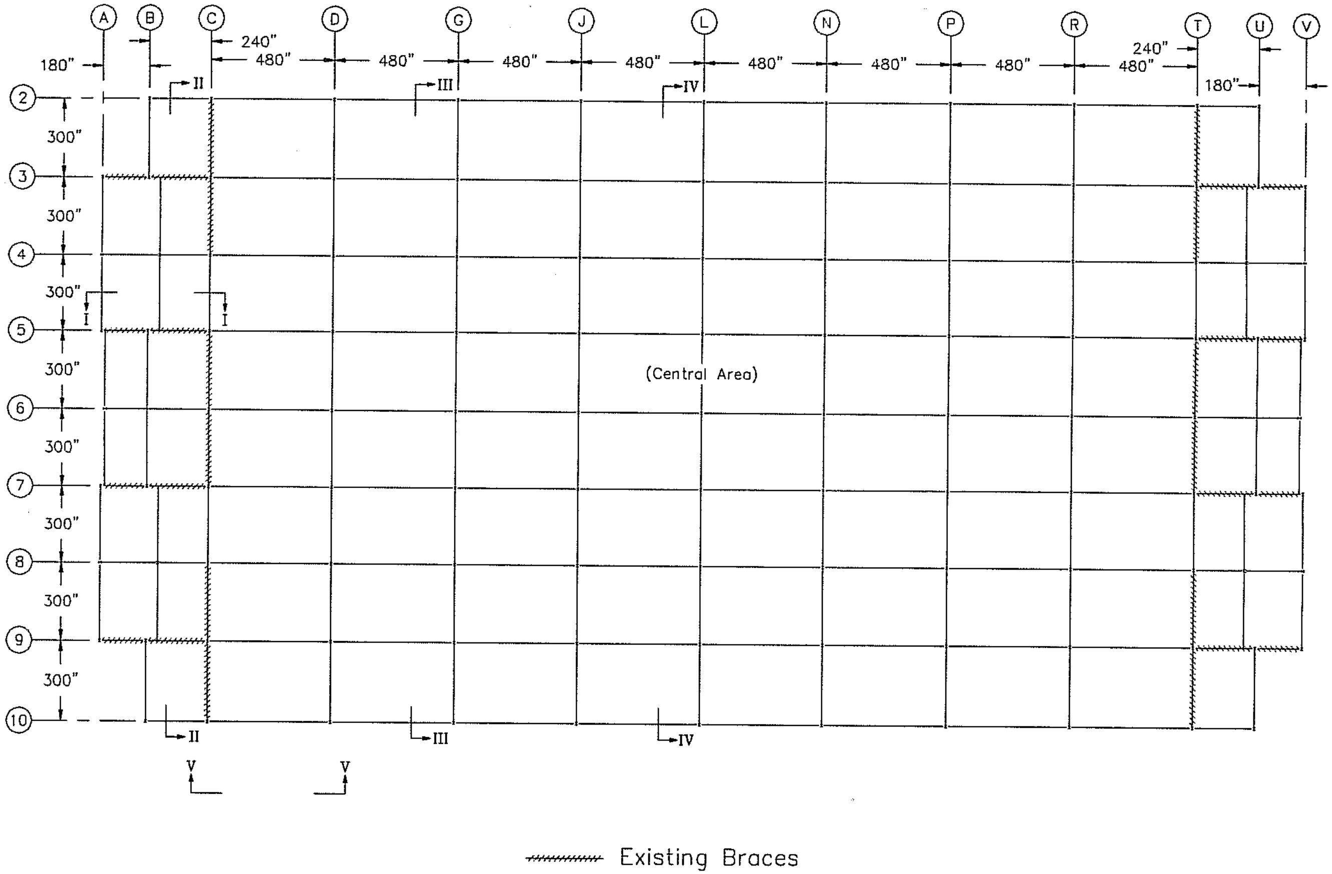


Figure 5.2 Plan View of Existing Building

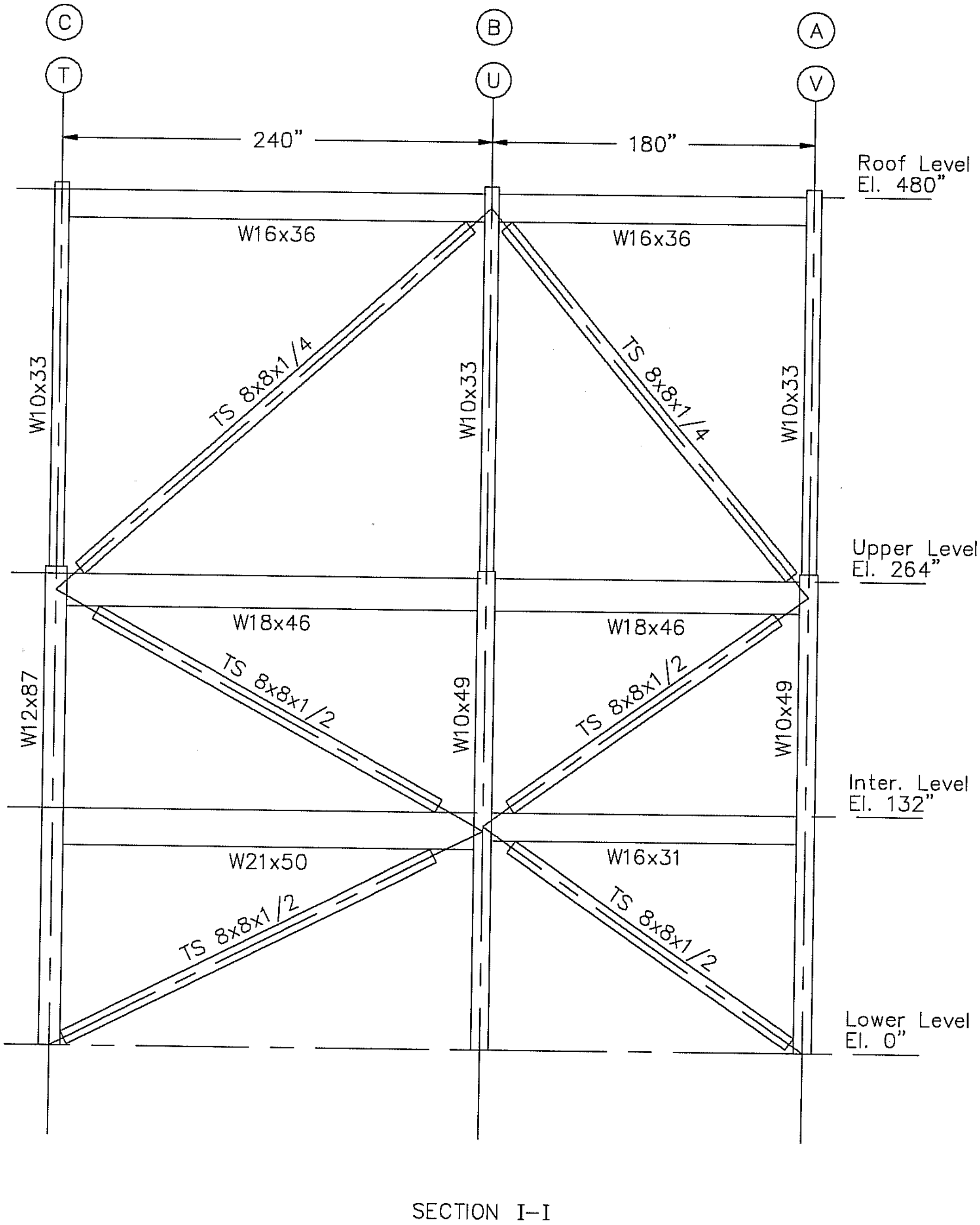
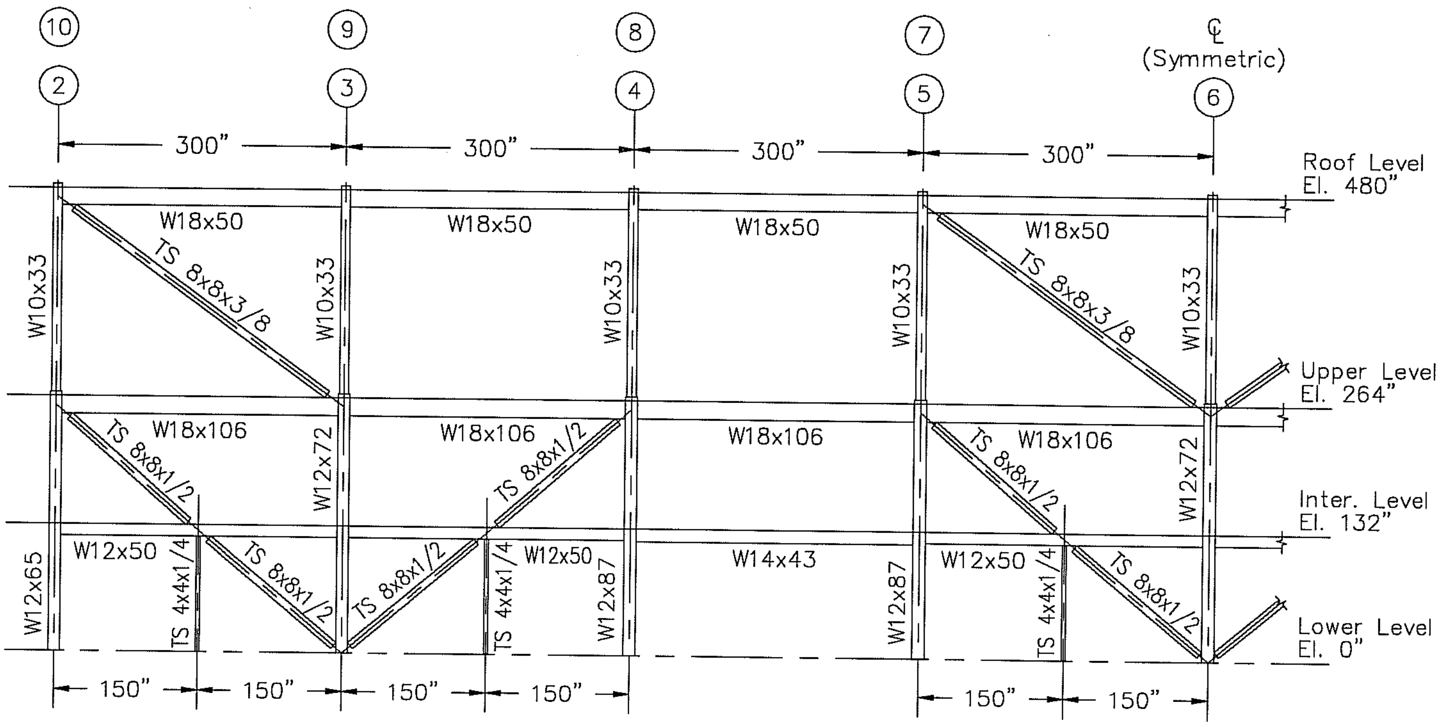


Figure 5.3 Existing Braced Frames in Axes 3, 5, 7 and 9



SECTION II–II

Figure 5.4 Existing Braced Frame in Axes C and T

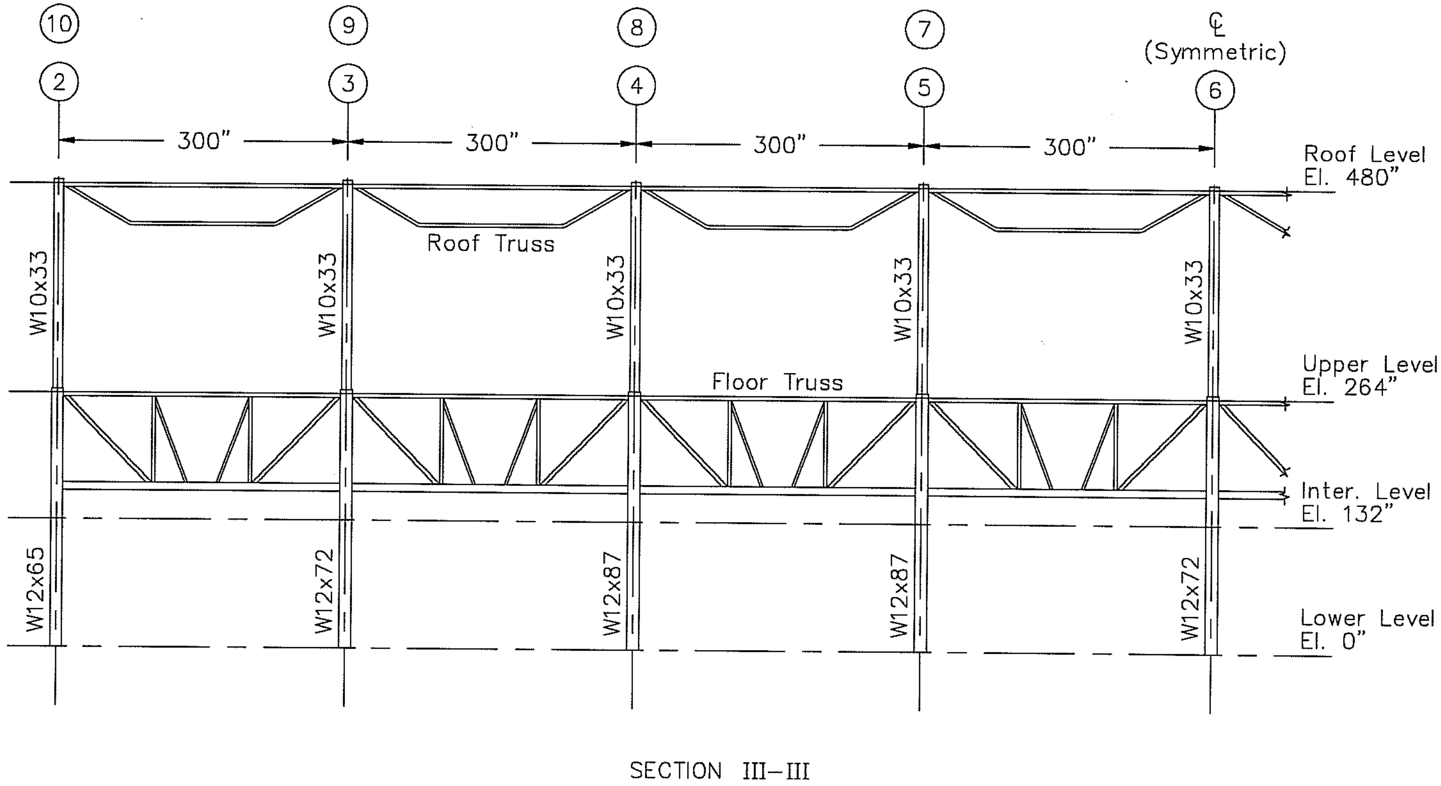


Figure 5.5 Existing Frame in Axes E to R

loads are transmitted at the diaphragm level only. There are no moment resisting connections between structural members in the unbraced frames.

A concrete slab placed over cold-formed steel sections forms the upper level diaphragm. At the interstitial level, the diaphragm in the central part of the building, between axes C and T, is made of plywood sheets. In the rest of the area, between axes A to C and T to V, the diaphragm consists also of a concrete slab placed over cold-formed steel sections. The roof is made of cold-formed steel sections with a built-up roofing material.

At the east and west ends of the building, where the braced frames are located, the foundation consists of a continuous reinforced concrete slab supported by concrete piles. The columns in the central part of the building are supported on reinforced concrete footings. The connections between the columns and the foundation are not designed to transmit any flexure. The foundation is designed to carry axial loads only.

5.2.2 Nonstructural elements and equipment

Nonstructural elements and equipment are those usually encountered in facilities of this type. Nonstructural elements include suspended ceilings with acoustic tiles, gypsum board partitions and concrete masonry walls not attached to the lateral load resisting system, precast concrete fascia elements. Mechanical/electrical installations include all those necessary for the functioning of typical industrial facilities, including HVAC systems and piping systems, water pumps, fire detection and sprinkler systems, telephone installations, security systems, and power and wire systems for computer and other production or data processing equipment installation.

5.2.3 Loads, construction materials and specifications

The dead and live loads used in the analyses were obtained from the original drawings. There is no clear specification that dictates the amount of live load to include when computing the mass of the building. Common engineering practice is to include the live load corresponding to the weight of the partitions in the mass computations. For this study, a live load of 10 psf. was included in the mass of the building. The design loads obtained from the drawings are:

a. Dead loads:

Roof	32 psf.
Upper level	75 psf.
Interstitial level	25 psf.

b. Live loads:

Roof	32 psf.
Upper level	165 psf.
Interstitial level	25 psf.

These loads were specified for the central part of the building. The loads on the end sections, where the braces are located, were not specified and were computed by hand. However, most of the mass is located in the central area.

According to the specifications in the original building drawings, the materials used for the structure are the following:

Concrete:	f'_c = 4000 psi for precast concrete panels
	f'_c = 3000 psi for all structural elements

Steel reinforcement:	ASTM A615 Grade 40	
Structural steel:	Structural shapes	ASTM A36
	Tubes	ASTM A500 Grade B
	Pipes	ASTM A53 Type E Grade B
	Welds	As specified in A.W.S. Standard
Concrete blocks:	Grade N blocks ASTM C90	

Inspection as specified in UBC 306 (1982) was required for construction of concrete footings, grade beams, piers, structural floor fills over metal decks, placement of reinforcement, field welding including braced frames and metal decks, high strength bolting and at fabricator's plant for precast elements.

5.3 MODEL OF STRUCTURE FOR ANALYSIS

Two different analyses were performed to study the seismic response of the structure. An elastic analysis using the program ETABS (Ref. 12) was conducted to analyze the building using the equivalent static lateral forces proposed in the UBC Code and the NEHRP Provisions. The same program was used to perform dynamic analyses using the EL Centro 1940 and records from the Loma Prieta 1989 earthquakes. Inelastic dynamic analyses were conducted using the program DRAIN 2D (Ref. 21), subjecting the structure to the same records mentioned above.

5.3.1 Equivalent static loads

Elastic analyses were performed using the equivalent static lateral load procedure proposed in the UBC Code (Ref. 15) and the NEHRP Provisions (Ref. 9). In both cases, the structure meets the requirements for use of the static method for analysis and design: it is a regular structure with no significant eccentricities or changes of stiffness or mass between floors, it is less than 240 ft. in height, and it has a well defined lateral force resisting system, according to descriptions in both documents. The classification of the structure and seismic coefficient for each of the mentioned provisions are summarized below.

5.3.1.1 UBC Code 1991

The analysis of the existing structure was done using the current UBC Code 1991, though these were not the provisions used originally to design the building. When evaluating the response of the structure, the differences in Code requirements will be discussed.

According to the UBC Code 1991, the structure is classified as follows:

-. Importance factor: special to standard occupancy category. $I=1.0$ for standard industrial buildings.

-. Soil profile: since the soil conditions are not known, a value of $S=1.5$ was chosen as representative of a typical condition.

-. Seismic zone factor: the building could be located in any zone of the country. For analysis purposes, it will be assumed to be located in California, in a zone with $Z=0.40$.

-. Structural system: the structure has a basic building frame system, with a lateral force resisting system consisting of concentrically braced steel frames, $R_w=8$.

-. Structure period: the period of the structure was estimated using ETABS. Periods and mode shapes are given in Table 5.1.

TABLE 5.1 PERIODS AND MODE SHAPES OF ORIGINAL STRUCTURE

LONGITUDINAL FRAMES			
	MODE 1	MODE 2	MODE 3
Period (sec)	0.44	0.19	0.07
Level	MODE SHAPES		
Roof	1.00	1.00	0.04
Upper	0.57	-0.59	-0.10
Interstitial	0.25	-0.40	1.00
TRANSVERSE FRAMES			
Period (sec)	0.51	0.24	0.12
Level	MODE SHAPES		
Roof	1.00	1.00	0.04
Upper	0.56	-0.60	-0.11
Interstitial	0.27	-0.45	1.00

Considering these factors, and following the provisions of the UBC Code, the total base shear for which the building was analyzed is:

$$V_b = (Z\,I\,C / R_w)\,W_t$$

$$C = 1.25\,S / T^{2/3} = 1.25 * 1.5 / 0.51^{(2/3)} = 2.93$$

The value of C need not be greater than 2.75, as specified in the UBC Code. Also, the value of C to be used cannot be less than 80% of the value of

C computed with a period of $T = 0.02 * h_T^{3/4}$, where h_T is the total height of the building. For $h_T = 40$ feet, $T = 0.32$ sec and $C = 4.0$. Again, C need not be greater than 2.75. With $C = 2.75$ and the values presented above:

$$V_b = (0.4 * 1.0 * 2.75 / 8) * W_t = 0.14 W_t$$

where W_t is the total weight of the building, including a portion of the live load corresponding to the partitions (10 psf), as mentioned in Section 5.2.3. The distribution of the equivalent lateral forces along the height of the structure followed the procedure described in the UBC Code.

5.3.1.2 NEHRP Provisions

The structure is classified according to the NEHRP Provisions (Ref. 9) as follows:

-. Soil profile: same conditions mentioned before, $S=1.5$.

-. Response modification factor: same structural system described before, $R=5$.

-. Effective Peak-Velocity Related Acceleration: $A_v=0.40$ for the California area.

-. Effective Peak Acceleration: $A_a=0.40$ for the California area.

-. Period of the structure: see Table 5.1.

The total base shear used in the analysis is:

$$V_b = C_s W_t$$

where $C_s = 1.2 A_v S / R T^{2/3}$ if the period of the building (T) is known.

$$C_s = 1.2 * 0.4 * 1.5 / 5 * 0.51 2/3 = 0.23$$

The value of C_s need not be larger than $2.5 A_a / R = 2.5 * 0.4 / 5 = 0.20$. With $C_s = 0.20$, the design base shear is:

$$V_b = 0.20 W_t$$

Vertical distribution of lateral forces followed the procedure the NEHRP Provisions.

5.3.2 Earthquake records

Three ground motions recorded at different locations were used in this study to evaluate the dynamic response of the building: (1) El Centro 1940, California, (2) Corralitos-Eureka Canyon Rd., Loma Prieta 1989, California, and (3) Oakland-Outer Harbor Wharf, Loma Prieta 1989, California. The El Centro record has been widely used in analytical studies and is classified as a medium-size earthquake. It has a wide frequency content for a record obtained in stiff soil conditions. The Loma Prieta records were chosen as representative of ground motion in the California area, for rock and soft soil conditions. Table 5.2 presents basic information about the records. Figure 5.6 shows the elastic pseudo-acceleration spectra for the three records.

TABLE 5.2 GROUND MOTIONS CONSIDERED

Location	Direction	Soil Type	Maximum Acceleration	Magnitude M_s
El Centro, California 1940	N00E	Alluvium	0.35 g	6.7
Corralitos, Loma Prieta 1989	N00E	Rock	0.63 g	7.1
Oakland Harbor, Loma Prieta 1989	N55W	Bay Mud	0.27 g	7.1

5.3.3 Model of lateral force resisting system

For both elastic and inelastic analyses, the structure was modeled using two dimensional frames. The model includes only those frames that participate in the lateral load resisting system. Since the structure is completely symmetric, only one frame for each direction was analyzed. The total mass was assigned to the frames according to the number of braced frames in each direction: one-half of the total mass for each frame in the transverse direction (axes C and T), and one-eight of the total mass for each frame in the longitudinal direction (axes 3, 5, 7 and 9). The floor diaphragms at the interstitial and upper levels at the location of the braced frames are considered rigid, so that the lateral forces would be distributed among the frames according to their stiffness. The roof diaphragm is considered flexible. The configuration of the frames that were analyzed is shown in Figures 5.3 and 5.4. Rayleigh damping was used to specify a 2% critical equivalent viscous damping for the first mode in all calculations.

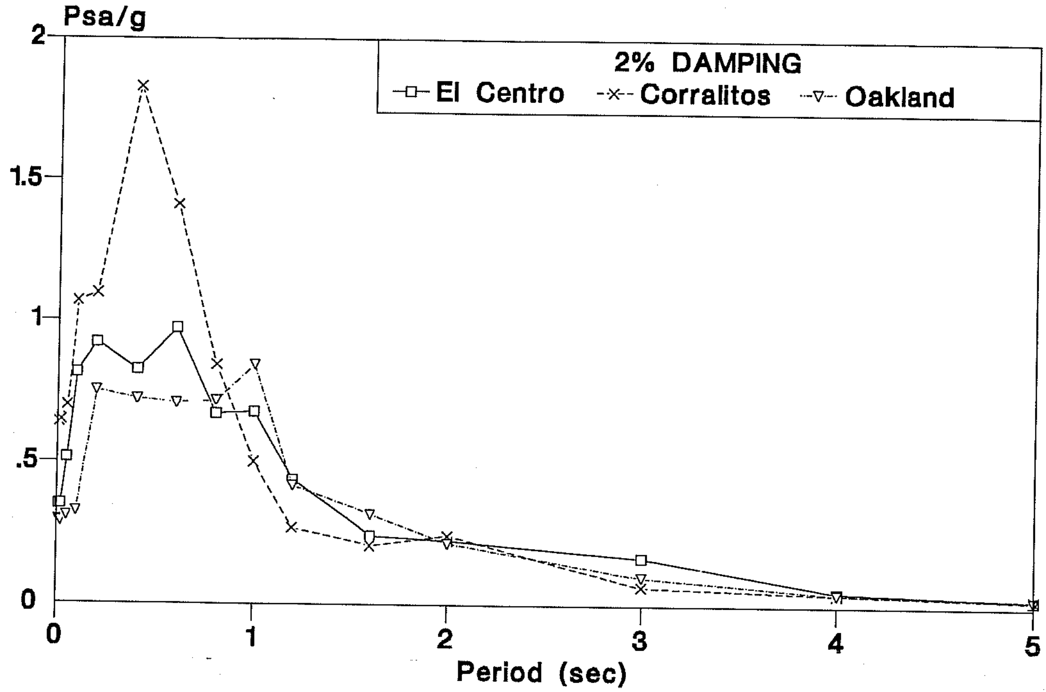


Figure 5.6 Elastic Pseudo - Acceleration Spectra

5.3.3.1 Model for ETABS

The base of the columns at the foundation level is assumed to be pinned, no moment capacity is considered. The columns are continuous from foundation to roof level. The beams are pinned ended and transmit axial and shear forces only at the connections. The beam and column elements are capable of carrying axial and shear loads, and bending moments along their length. The braces are also pinned ended and are modeled as truss members, carrying only axial loads. Since ETABS performs elastic analyses only, the variables needed to model the elements are their geometric properties: area and moments of inertia, and type of connection between elements. No member capacities are specified.

5.3.3.2 Model for DRAIN 2D

To perform inelastic analysis using DRAIN 2D, the capacity of the elements must be specified. For this purpose, the program includes a variety of element models considering different typical behaviors. The models used for each of the elements in the structure are:

a. Beams: The beams are pinned ended in the real structure, so they were modeled as truss members transmitting axial loads only. Since the analysis considers only lateral loads, the shear and bending properties of the beams are not relevant. The beams are continuously connected to the concrete slab, therefore buckling is not allowed. An equivalent area was specified to include the contribution of the slab.

b. Columns: A beam-column element was used to model the columns. The columns are continuous along their height and are capable of carrying axial, shear and bending forces. The interaction surface used is shown in Figure 5.7. The value of P_y corresponds to the yield load in tension and compression computed according to the provisions of the AISC Manual (Ref. 1). There is a zone of undefined axial load values when the moments on the section approach zero. Because of the computational procedure used in the program, axial forces in excess of yield for zero moments can be computed. In the printed results, axial forces approaching or exceeding yield in a column would indicate that column damage is probably implied. Plastic hinges are developed at the ends of the members when the combination of axial force and bending moment applied to the columns lies outside the interaction surface. In the analysis presented in this report, the columns did not reach the points of maximum tension or compression loads. The values used for P_y and M_p correspond to the maximum capacity of the member, without including strength reduction factors. The effective length factor of the columns was taken from $k=0.7$ to 1.0, according to the support condition.

c. Braces: braces were modeled as truss elements including cyclic buckling and yielding. The failure surface of the braces is controlled by the axial force only. The model used for this type of element is shown in Figure 5.8. An initially straight member loaded first in tension follows an elastic slope as shown by segment AE. When the elements yields, it follows a horizontal plateau (segment EF). If the direction of displacement is reversed, the member unloads elastically, parallel to the initial elastic slope. Continued compression will result in first buckling of the member (point B). If the compression is sustained, the member will reach its post buckling capacity (point C) and will keep on deforming until the direction of axial displacement is reversed (segment CD). When the axial displacement is reversed, the compression load decreases to zero followed by an increasing tensile load (segment DG). The slope of the increasing tensile load

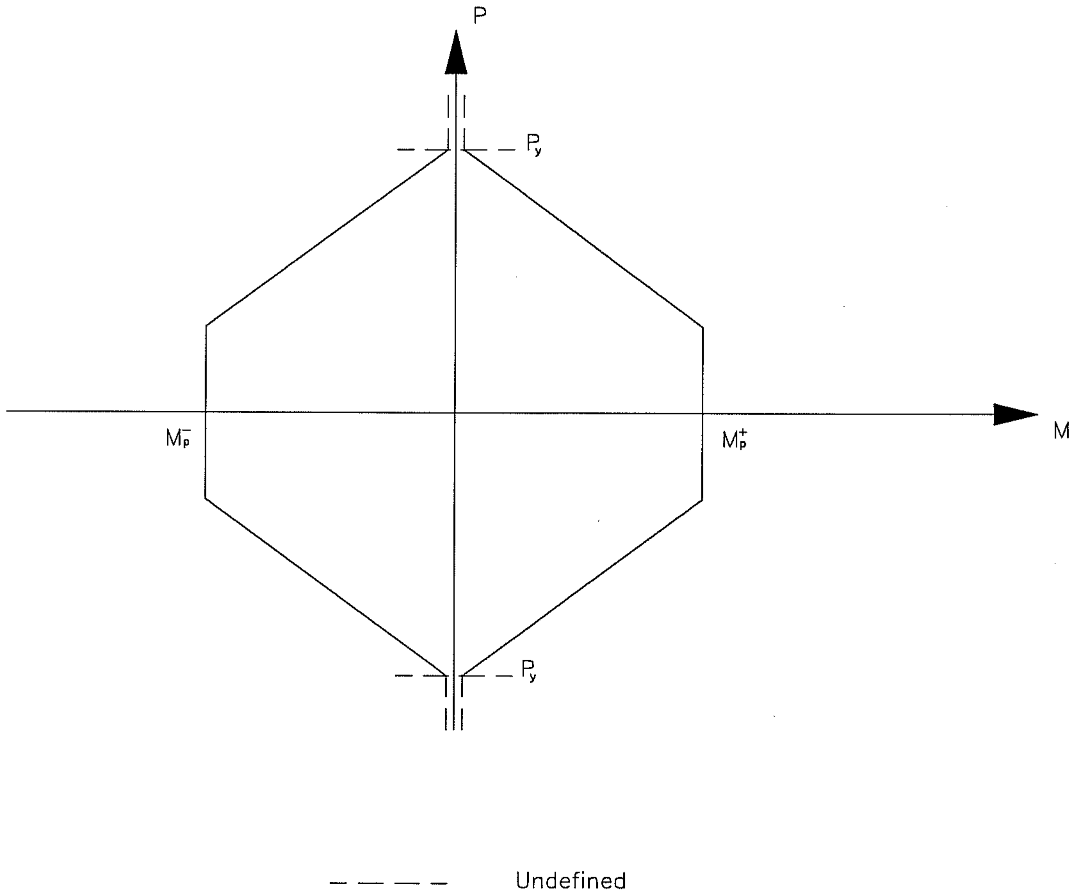


Figure 5.7 DRAIN 2D Model for Beam–Column Elements Interaction Surface (Ref. 21)

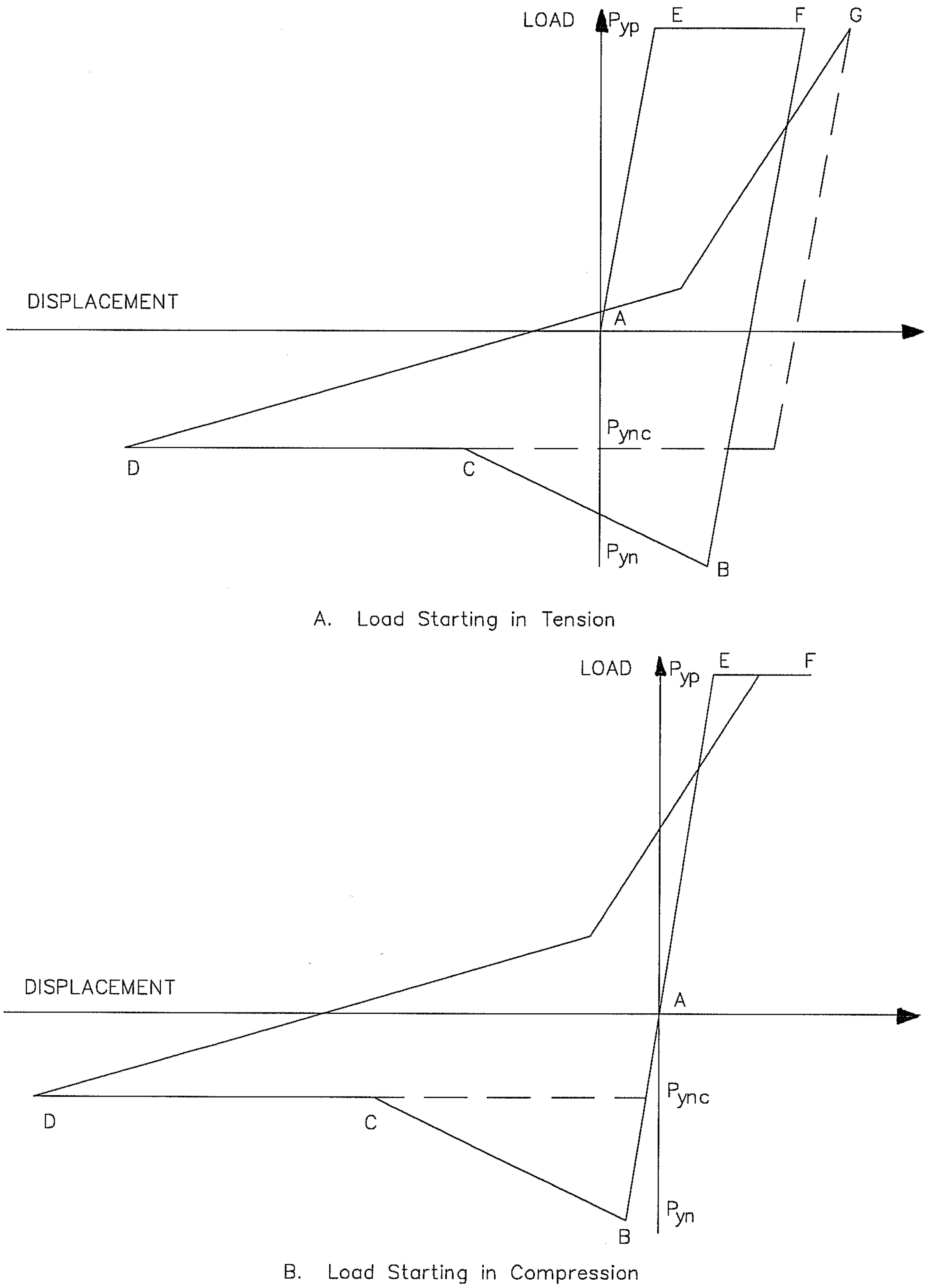


Figure 5.8 DRAIN 2D Model for Bracing Elements (Ref. 16)

is smaller than the initial elastic slope, indicating degradation of stiffness of the element. If the member deforms again in compression, it will reach the post-buckling load level and continue along the same path described above. When the member is loaded first in compression (Figure 5.8.b), it will follow an elastic slope until it reaches the first-buckling load (point B). From that point, the deformation path is similar to the one described above. A more detailed description of the model can be found in Ref. 16.

Yield load in tension (P_{yp}) and first buckling capacity (P_{yn}) are computed according to the provisions for axially loaded members in the AISC Manual (Ref. 1), without including strength reduction factors. The post-buckling strength factor ϕ is computed as 18/(kl/r), according to the recommendation given in Ref. 14. Post-buckling strength P_{ync} is computed as ϕ P_{yn}. The slenderness factor kl/r varies for each element. The effective length factor k was taken as 1.0 for all the braces in the original structure. When cross (X) bracing is used, k is taken as 0.5 for in-plane buckling and 0.6 for out-of-plane buckling, as recommended in Ref. 5 and 13. Fracture criteria are not included in the model.

Time history analyses were performed using an integration time step of 0.005 seconds.

5.4 RESULTS OF ELASTIC ANALYSES

Elastic analyses were performed using ETABS and subjecting the structure to the lateral loads specified by the UBC Code and the NEHRP Provisions (see Section 5.3).

5.4.1 Floor displacements

In Table 5.3, the floor displacements and drifts obtained for the structure under lateral loads computed with the UBC Code and the NEHRP Provisions are presented. The table also shows the allowable drifts specified in both the mentioned provisions. The floor displacements shown correspond to those computed at the braced frame lines. Some increase in floor displacements is expected at the symmetry line of the structure (center of floor diaphgragm), but diaphragm analyses showed that the difference in displacements is negligible at the upper and interstitial levels.

TABLE 5.3 FLOOR DISPLACEMENTS AND DRIFTS FROM ELASTIC ANALYSIS

UBC CODE 1991					
Level	Longitudinal Frames		Transverse Frames		Allowable drift (%)
	Displ.(in)	Drift (%)	Displ.(in)	Drift (%)	
Roof	0.45	0.10	0.62	0.13	0.50
Upper	0.25	0.11	0.34	0.14	0.50
Interstitial	0.11	0.08	0.16	0.12	0.50
NEHRP PROVISIONS*					
Roof	2.66	0.52	3.65	0.75	1.00
Upper	1.53	0.64	2.03	0.79	1.00
Interstitial	0.68	0.52	0.99	0.75	1.00

* Displacements presented for the NEHRP Provisions correspond to the elastic displacement multiplied by a C_d factor of 4.5. These are projected inelastic displacements according to Ref. 9.

In all cases, the computed interstory drifts meet the allowable drift specified in the UBC Code and NEHRP Provisions. The displacements obtained with the loads computed following the UBC Code procedure are well below the allowed drift limit. No suggestions are given in the UBC Code to predict the expected inelastic displacements, though a multiplier of 3/8 R_w is implied. Values obtained using the NEHRP Provisions represent the possible inelastic displacements. The C_d factor used depends on the structural system of the structure (Ref. 9). In this case, the projected displacements and drifts are also below those allowed by the Provisions, though they are closer to the limit than those specified by UBC Code.

5.4.2 Member forces

All the members were checked to verify that their strength equals or exceeds the values specified by current UBC Code and AISC-LRFD Provisions. Capacity of the beams and columns is, in all cases, larger than the capacity required to sustain the specified lateral loads. The braces, however, do not always meet the design requirements. In Table 5.4, some of the properties of the braces along with their computed design capacities are presented.

There are some detailing provisions in the current UBC Code and the AISC-Seismic Provisions Manual (Ref. 2) that the braces must meet. Limits to the slenderness ratio of the braces are given to avoid use of very slender members in seismic zones, which would result in deterioration of the compressive axial strength of the member in the post-buckling range. L/r (ratio of length over radius of gyration of the element) is limited to $720/\sqrt{Fy}$. This gives a value of 106 for A500 Grade B steel (Fy=46 ksi). A limit is also specified in the AISC-Seismic Provisions Manual for the b/t (width over thickness) ratio of the braces to avoid local buckling and fracture of the braces under repetitive loading cycles such as those imposed by seismic motions. The limit is $110/\sqrt{Fy}$ for seismic zones, which gives a value of b/t=16 for Fy=46 ksi.

TABLE 5.4 PROPERTIES OF EXISTING BRACING ELEMENTS

FRAMES LONGITUDINAL DIRECTION								
Level	Length (in)	Section	KL/r*	b/t	Pcd (kip)*		Ptd(kip)*	
					ASD	LRFD	ASD	LRFD
Roof	315	TS8x8x1/4	100	***29***	101	122	278	314
	272	TS8x8x1/4	86	***29***	125	146	278	314
Upper-Interst	274	TS8x8x1/2	91	13	222	262	528	596
	223	TS8x8x1/2	74	13	275	311	528	596
FRAMES IN TRANSVERSE DIRECTION								
Roof	370	TS8x8x3/8	120	***18***	101	135	407	460
Upper-Interst.	200	TS8x8x1/2	66	13	303	334	528	596

* K=1 for all braces

Pcd and Ptd are the first buckling and tensile capacity respectively of the brace elements, computed according to the provisions in the UBC Code (Ref. 15) using Allowable Stress Design (ASD) and the AISC Manual (Ref. 1) using Load and Resistance Factor Design (LRFD). Capacities computed with ASD consider a reduction factor of $1/(1 + (Kl/r)/2Cc)$ and are amplified by 1/3 for earthquake loading (Ref. 15). Capacities computed with LRFD (Ref. 1) consider strength reduction factors of 0.85 in compression and 0.90 in tension. An additional reduction to 0.8 times the design capacity in compression is considered as specified in the reference.

As shown in Table 5.4, the braces in the transverse and longitudinal directions at roof level do not meet the slenderness ratio. Since the structure is low, with only three stories, this requirement may be omitted if amplified design loads are used (see Ref. 1). The b/t requirement is not met by the TS8x8x1/4 braces in the longitudinal direction. For this section b/t=29 and exceeds the allowable limit set by the AISC Provisions. The TS8x8x3/8 braces in the transverse direction barely exceed the b/t limit. It is important to notice that these limits are proposed by current codes, and were not required in earlier code versions. Codes at the time of original design of the building did not specifiy these b/t and L/r requirements.

In Table 5.5, the forces acting on the braces for the structure subjected to the UBC Code and NEHRP lateral forces are presented. Ratios of demand (Pr) over design capacity (Ptd and Pcd as shown in Table 5.4) are also shown. The required strength according to the forces specified by the UBC 1991 Code are very close to the design capacity of the braces in the longitudinal direction. In the transverse direction, the demand exceeds the design capacity of the interstitial and roof level braces. The required strength in the elements in the longitudinal direction is less due to the presence of a larger number of braced bays. In the transverse direction there are fewer braced bays, therefore demand on the braces is higher.

TABLE 5.5 RATIOS OF REQUIRED OVER DESIGN CAPACITY OF BRACES

BRACES LONGITUDINAL DIRECTION						
Level	UBC CODE 1991			NEHRP PROVISIONS		
	Pr (kip)	Pr/Pcd	Pr/Ptd	Pr (kip)	Pr/Pcd	Pr/Ptd
Roof	81	0.80	0.29	103	0.84	0.33
Upper	160	0.96	0.30	214	0.82	0.36
Interst.	152	0.93	0.29	204	0.78	0.34
BRACES TRANSVERSE DIRECTION						
Roof	133	***1.32***	0.33	169	***1.25***	0.37
Upper	227	0.75	0.43	303	0.91	0.51
Interst.	243	***1.08***	0.62	326	0.98	0.55

Computed demand over capacity ratios using the NEHRP forces are similar to those obtained with UBC. Again the demand on the braces on the transverse direction is higher and exceeds the design capacity of the braces of the roof level. At the other levels in both longitudinal and transverse directions, demand on the members is very close to their capacity in compression.

5.5 INELASTIC ANALYSES OF EXISTING STRUCTURE

Inelastic analyses were performed using the earthquake records described in Section 5.3. The results in terms of formation of plastic hinges, floor deflections, velocities and accelerations, are presented next.

5.5.1 Description of inelastic behavior of structure

The amount of inelastic action experienced by the lateral load resisting frames varies according to the record to which they are subjected. In general, dissipation of energy through inelastic deformations occurs mainly on the first floor. The first floor braces reach yielding in tension and first buckling capacities when subjected to all the records. In general, maximum plastic extensions experienced by the first floor braces are about twice as large as those of the braces in the other floors. Following is a description of the plastic action in the longitudinal and transverse frames. Layout of these frames, along with the member sections are shown in Figures 5.3 and 5.4. The model used for each element and the characteristics of the earthquake records were described in Section 5.3 of this chapter. Figures 5.9 and 5.10 show the plastic hinges in columns and buckling of braces for each of the frames and records. Plastic hinges and extensions in the elements do not necessarily occur simultaneously,

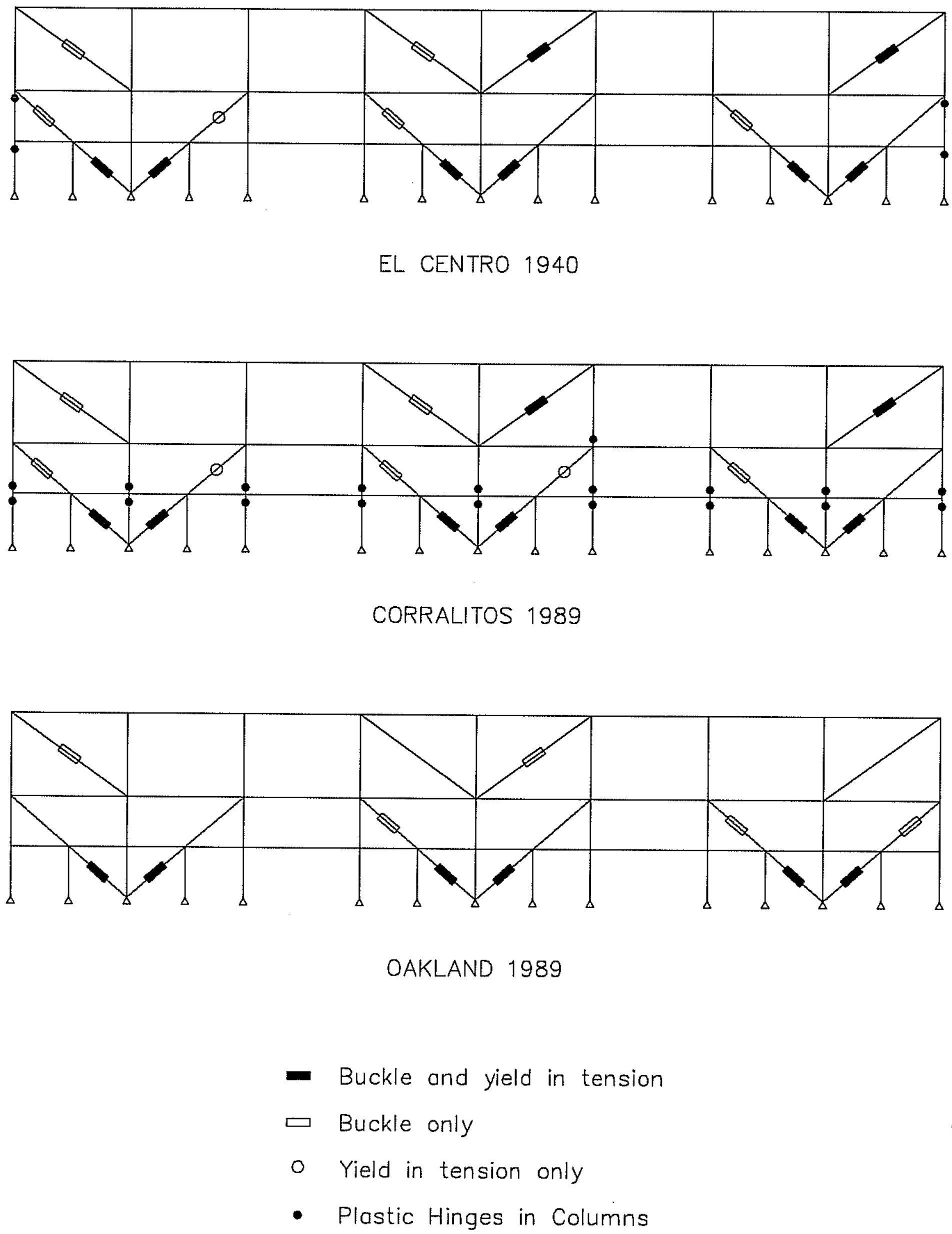


Figure 5.9 Inelastic Action in Existing Transverse Frames (Axes C and T) – Original Structure

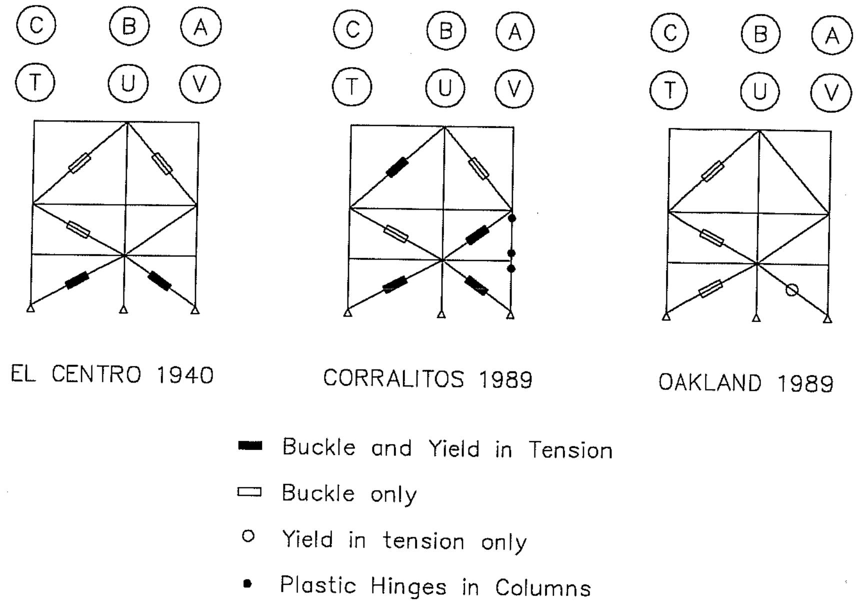


Figure 5.10 Inelastic Action in Longitudinal Frames Axes 3, 5, 7 and 9 – Original Structure

those shown in the figures are all the hinges and inelastic deformations which occurred during the time interval analyzed.

5.5.1.1 El Centro 1940

Inelastic behavior of the braces is concentrated mainly on the first floor. Maximum plastic extensions of these braces are about three times those experienced in the braces in the level above in the longitudinal direction, and about six times those in the transverse direction. The first floor braces in both directions reach maximum yield and first buckling capacity early in the record, between 1.8 sec. and 5.2 sec. in the longitudinal direction, and between 1.8 sec. to 2.3 sec. in the transverse direction. Once beyond those points, the braces continue to deform inelastically, losing compressive strength and stiffness. For these braces, post-buckling strength is assumed to be about 0.20 to 0.25 times the first buckling capacity depending on the slenderness ratio Kl/r. The elements above the interstitial level remain almost elastic, with very few deformation cycles entering the inelastic range. The columns and beams in all floors remain elastic in the longitudinal frames. The exterior ground and interstitial level columns in the transverse frames develop plastic hinges at the top, as shown in Figure 5.9. The beams in this frame remain elastic.

5.5.1.2 Loma Prieta Corralitos

The Loma Prieta Corralitos record is much stronger than the other records used for this analysis. Inelastic behavior of the frames is therefore more extensive. Dissipation of energy through inelastic deformations occurs mainly on the ground and interstitial levels, with considerable plastic extension of the braces and extensive formation of plastic hinges in the columns. In both the longitudinal and transverse directions, the maximum plastic extensions experienced by the braces are about twice as large those under the El Centro record. Again for this record, the braces reach their maximum tension and compression capacities early in the history of the record, between 2.0 and 7.0 seconds. Figures 5.9 and 5.10 show the braces that buckle. Formation of plastic hinges in the columns is more extensive. In the longitudinal direction, plastic hinges are developed in the columns on axes A and V, on ground and interstitial levels. In the transverse direction, all the columns in the ground and interstitial floors along with one column on the upper floor reach their maximum elastic capacity and develop plastic hinges at some time during the record.

5.5.1.3 Loma Prieta Oakland

Inelastic response of the frames is minimal under the Oakland record. Due to the characteristics of this record (see Section 5.3), it does not adversely affect the structure. Inelastic action is concentrated in the braces, as shown in Figures 5.9 and 5.10. The maximum plastic extensions are much lower than those experienced under the El Centro and Corralitos records. The structure remains elastic up to the 10th second of the record, when one of the braces in the top floor reaches its buckling capacity. Between the 10th and 13th seconds buckling and yielding of the braces occurs, coinciding with the time where peak ground accelerations are reached. All the columns in both longitudinal and transverse directions remain elastic, as do all the beams.

5.5.2 Description of floor displacements

In Table 5.6, the floor displacements and interstory drifts experienced by the longitudinal and transverse frames when subjected to each of the earthquake records are listed. The displacement values presented correspond to the time when displacement at the roof level was maximum. The largest interstitial drift values experienced are presented in parenthesis in the Table.

The displacements shown correspond to the diaphragm or floor levels in the east and west ends of the building, where the braced frames are located. In the central part, between axes C and T, the interstitial level is supported by the upper level through the floor trusses, as discussed in Section 5.2.1. For analytical purposes, the mass of the interstitial level in the central part was all concentrated at the upper level, no diaphragm action is considered at the interstitial level. The displacements at the interstitial level in the central part of the building are probably closer to those shown in Table 5.6 for the upper level. Due to the flexibility of the roof diaphragm, the displacements at the roof level at the symmetry line of the structure (axis L) are expected to be higher than the displacements at roof level computed at the braced frame lines, shown in Table 5.6. Roof displacements at axis L were not computed for this study.

TABLE 5.6 INELASTIC FLOOR DISPLACEMENTS AND INTERSTORY DRIFTS IN ORIGINAL STRUCTURE

FLOOR	EL CENTRO		CORRALITOS		OAKLAND	
	Floor displ. (in)	Drift (%)	Floor displ. (in)	Drift (%)	Floor displ. (in)	Drift (%)
LONGITUDINAL DIRECTION						
Roof (40 ft)	2.02	0.30	3.10	0.35	1.66	0.31
Upper (22 ft)	1.37	0.30	2.34	1.04	1.00	0.30
Inter (11 ft)	0.98 (1.20)*	0.74 (0.91)	0.97 (1.10)	0.73 (0.83)	0.60 (0.60)	0.45 (0.45)
TRANSVERSE DIRECTION						
Roof (40 ft)	2.78	0.54	4.34	0.37	1.63	0.18
Upper (22 ft)	1.62	0.31	3.54	0.45	1.25	0.26
Inter (11 ft)	1.21 (1.78)	0.92 (1.35)	2.95 (3.07)	2.23 (2.33)	0.91 (0.91)	0.69 (0.69)

* Maximum displacement of first floor. Does not coincide in time with maximum displacement of roof.

Interstory drifts in the longitudinal direction are always below or very close to 1%, which could be considered adequate taking into account that it corresponds to the structure in the inelastic range. The low drifts experienced in the top floors are an indication that the inelastic action is occurring mainly in the first floor which is the one that experiences higher relative displacements (Figure 5.11). For the Corralitos record, the displacements show that inelastic action occurs in the first two levels, with the top floor moving almost as a rigid body when compared with the bottom floors.

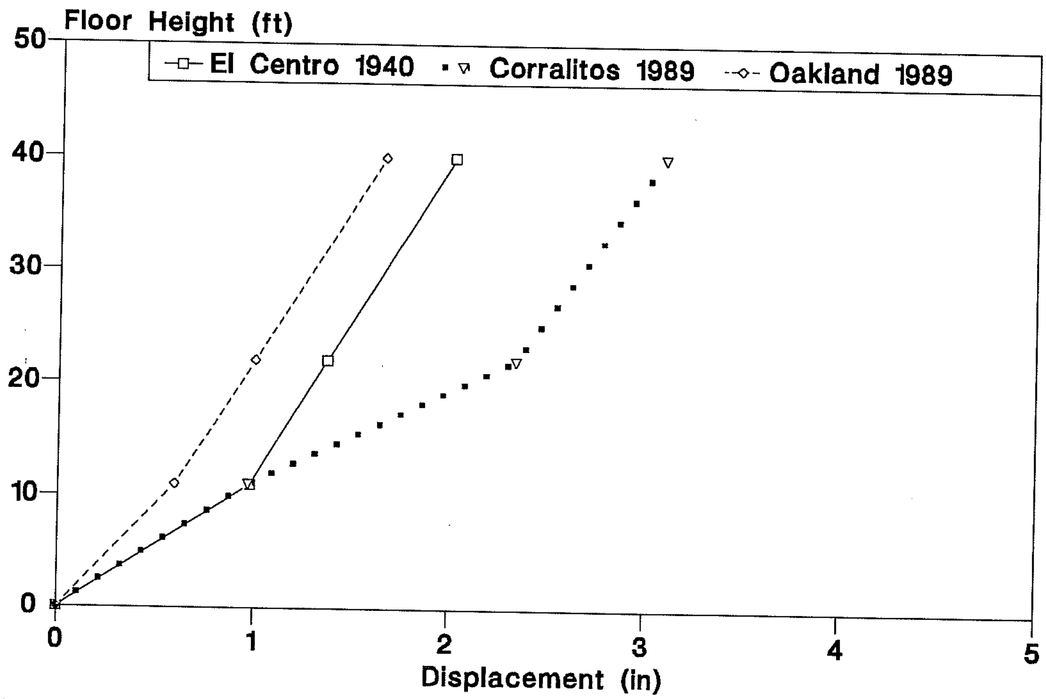


Figure 5.11 Inelastic Displacements in Original Structure Longitudinal Direction

In the transverse direction, the floor displacements also indicate that most of the inelastic action is occurring on the ground floor. The drifts between the ground and interstitial levels are very high, particularly for the Corralitos record (Figure 5.12). Drifts obtained with both El Centro and Corralitos records exceed the 1% value assumed to be adequate for inelastic deflections (Ref. 9). The high drifts indicate a strong possibility of non-structural damage in the building. According to the NEHRP Provisions, drift could go as high as 1.5% depending on the importance of the building and the non-structural elements in it. The results obtained for the Corralitos record do not meet this limit.

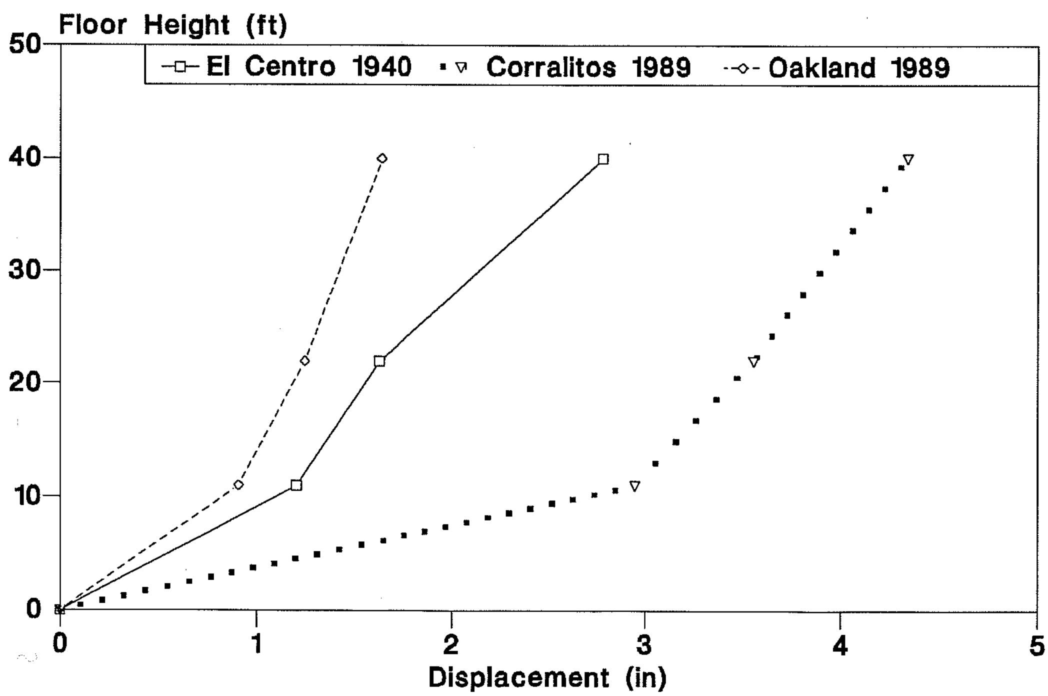


Figure 5.12 Inelastic Displacements in Original Structure Transverse Direction

5.5.3 Floor velocities and accelerations

Floor velocities and accelerations obtained with each of the three earthquake records used are shown in Table 5.7. The values shown were computed at the braced frame lines. As discussed before for floor displacements, differences on the velocity and acceleration values are expected at the center of the diaphragm. Nevertheless, analyses of the diaphragm showed that the amplification of motion at the center of the diaphragm is not significant, resulting in values of displacements, velocities and accelerations very close to those computed at the braced frame lines.

Though there are not any limitations set for velocities or accelerations on the floors, it is important to note the high amplification of ground motion in the upper levels. The amplifications are important when computing design loads for non-structural elements and equipment on the floor. For this structure, expected ground motion amplifications would be around 1.3, 1.6 and 2.0 for the interstitial, upper and roof levels (Ref. 11). The values shown in Table 5.7 are similar for the first two floors. The acceleration amplification in the roof is higher in the longitudinal direction, and a little lower than 2.0 in the transverse direction. The increase in floor acceleration in the longitudinal direction seems to be related to higher frame stiffness in that direction.

TABLE 5.7 FLOOR VELOCITIES AND ACCELERATIONS IN ORIGINAL STRUCTURE

LONGITUDINAL DIRECTION									
Level	Maximum Velocity (in/sec)			Maximum Acceleration (in/sec^2)					
	El Centro	Corralitos	Oakland	El Centro	Amplific.	Corralitos	Amplific.	Oakland	Amplific
Roof	20.0	29.7	17.6	336 (0.87g)	2.5	294.0 (0.76g)	1.2	242.0 (0.63g)	2.2
Upper	13.3	19.2	9.2	159.0 (0.41g)	1.2	184.9 (0.48g)	0.8	169.0 (0.44g)	1.5
Interst	11.0	9.2	6.4	182.0 (0.47g)	1.4	250.4 (0.65g)	1.0	166.0 (0.43g)	1.5
TRANSVERSE DIRECTION									
Roof	22.5	26.1	11.5	243.0 (0.63g)	1.8	252.0 (0.65g)	1.0	188.0 (0.49g)	1.7
Upper	17.4	19.1	9.5	143.0 (0.37g)	1.1	155.0 (0.40g)	0.6	113.0 (0.29g)	1.0
Interst	18.2	22.0	5.8	159.0 (0.41g)	1.2	186.0 (0.48g)	0.8	130.0 (0.34g)	1.2

5.6 INELASTIC ANALYSIS OF STRENGTHENED STRUCTURE

The strengthening technique selected for the structure was based on the following aspects:

a. Floor displacements resulting from the inelastic analyses are large, particularly in the transverse direction. Non-structural and equipment damage is expected for such large deformations.

b. The original structure undergoes extensive inelastic deformation when subjected to typical earthquake records, which may result in damage to the structure and loss of stiffness and strength for resisting subsequent earthquakes. Also hinge formation on the columns indicates a possibility of buckling, which could affect the stability of the structure. The b/t ratio of the braces on the top floor is very high and local buckling or fracture could result if extensive inelastic deformation is allowed.

The objective of the strengthening scheme is to reduce the force levels on the existing frames, to reduce inelastic action on the braces and hinge formation of the columns, and to reduce drifts to a reasonable value. A 1% drift was set as an adequate limit to avoid extensive non-structural damage. For stronger earthquakes, such as the one represented by the Corralitos record, a higher drift could be allowed. A maximum of 1.5% is considered acceptable based on the recommendations of the NEHRP Provisions.

The strengthening scheme is intended to provide a stiffer lateral load resisting system by adding new braces in both the longitudinal and transverse directions. Adding bracing elements to the braced frames that constitute the lateral load resisting system of the existing structure is not possible. In the longitudinal direction all the bays between axes A to C and T to V are braced and in the transverse direction, the only remaining open bays in the same area are needed for the function of the facility. Along the axes of the unbraced frames addition of braces to the existing structure in the transverse direction is difficult due to the presence of the trusses supporting the floors. The solution selected involves placing a new braced frame next to the existing gravity load frame at the location shown in Figure 5.13. The new frame would interfere with the open space in the central area of the building, and would impair arrangement of office space and equipment, but if the owner of the facility is willing to accept this inconvenience, the construction of the new frame is feasible and produces acceptable performance. In the longitudinal direction, additional braces can be placed in axes 2 and 10 without interfering with the central open areas. The location of the new braces is shown in Figure 5.13. The layout of the new braces and the section sizes of the elements are shown in Figures 5.14 and 5.15.

The periods of the strengthened structure in the longitudinal and transverse directions are shown in Table 5.8. The period of the existing structure is reduced in both directions.

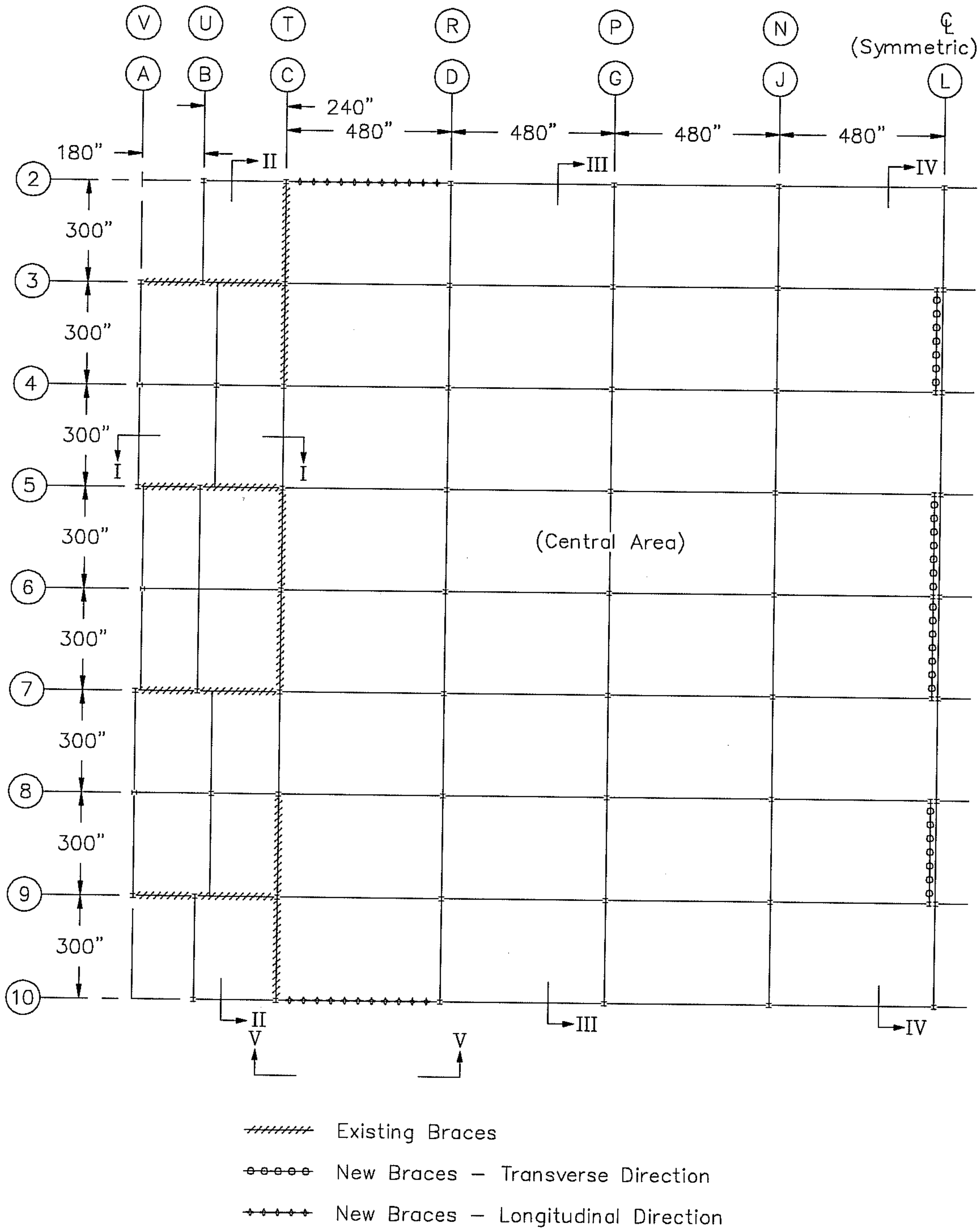
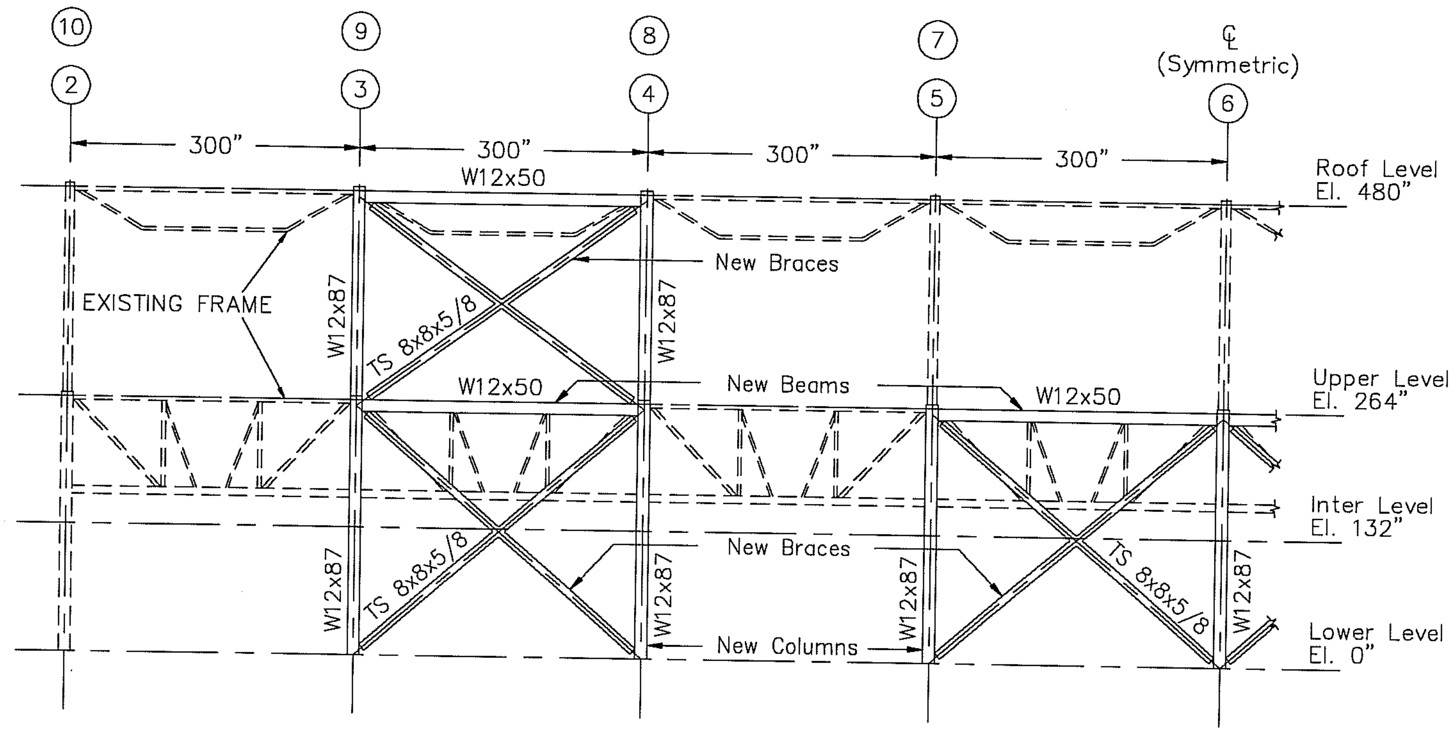


Figure 5.13 Plan View of Strengthened Structure



SECTION IV-IV

Figure 5.14 Proposed Bracing in Axis L – Transverse Direction

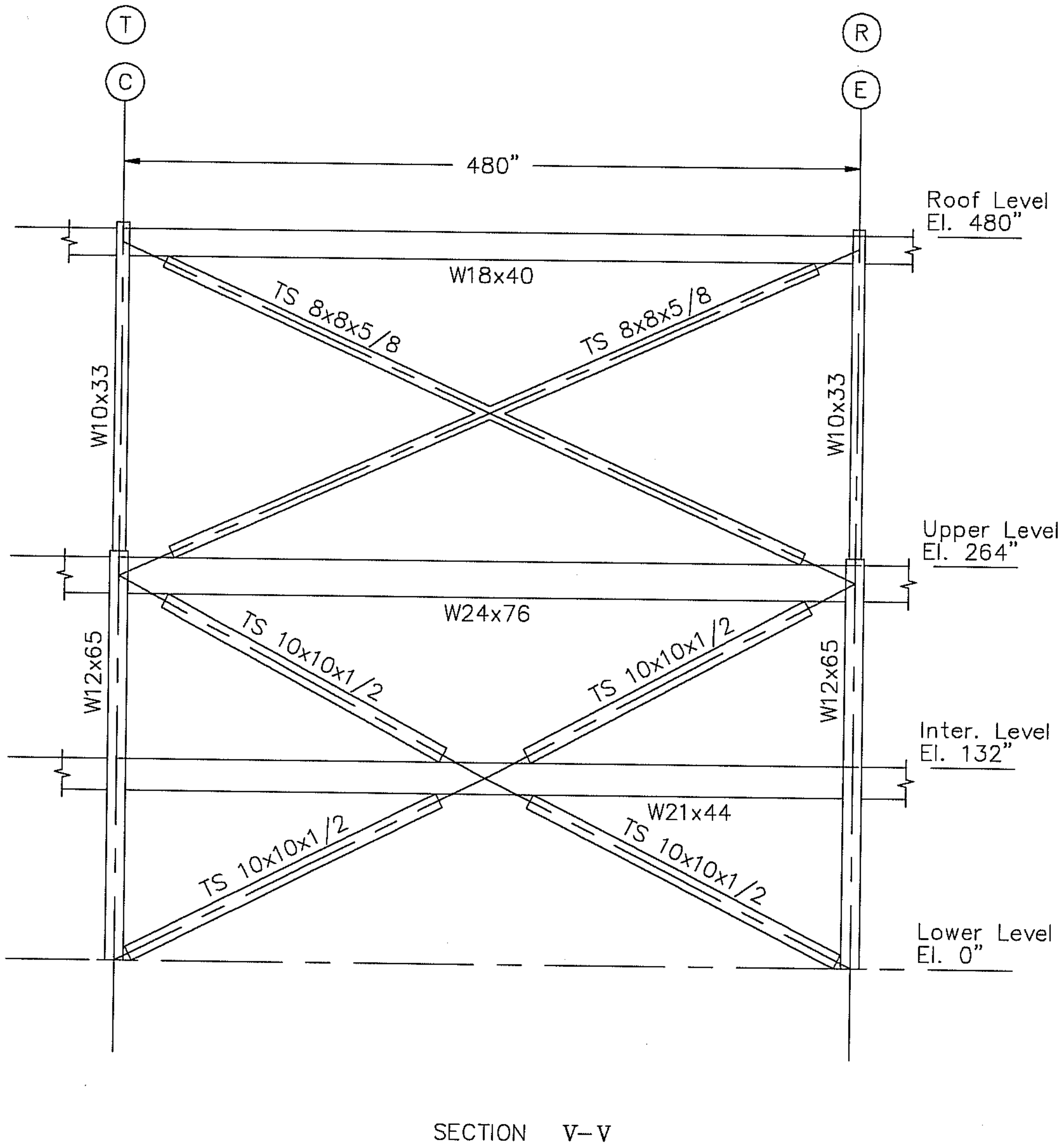


Figure 5.15 Proposed Bracing in Axes 2 and 10 Longitudinal Direction

TABLE 5.8 PERIODS OF ORIGINAL AND STRENGTHENED STRUCTURE

LONGITUDINAL DIRECTION		
MODE NUMBER	PERIOD (SEC)	
	Original structure	Strengthened structure
Mode 1	0.44	0.35
Mode 2	0.19	0.15
TRANSVERSE DIRECTION		
Mode 1	0.51	0.41
Mode 2	0.24	0.30

5.6.1 Description of inelastic behavior of the structure

Adding braces in both longitudinal and transverse directions effectively reduces the amount of inelastic deformation in the original structure. Figures 5.16 to 5.18 show the accumulated plastic hinges in the columns and buckling in the braces at the end of each earthquake record. Plastic hinging and buckling shown do not necessarily occur at the same time.

5.6.1.1 El Centro 1940

In the transverse direction, overall buckling and yielding of the braces is reduced. None of the braces reach their tensile capacity, and buckling is reduced on the first two floors. This indicates that the forces are being effectively distributed among the three transverse frames. In the ground floor, maximum plastic deformations in the braces of the strengthened structure are about 1/6 of the maximum plastic deformations in the braces of the original unstrengthened structure. Maximum plastic extensions on the braces of the top

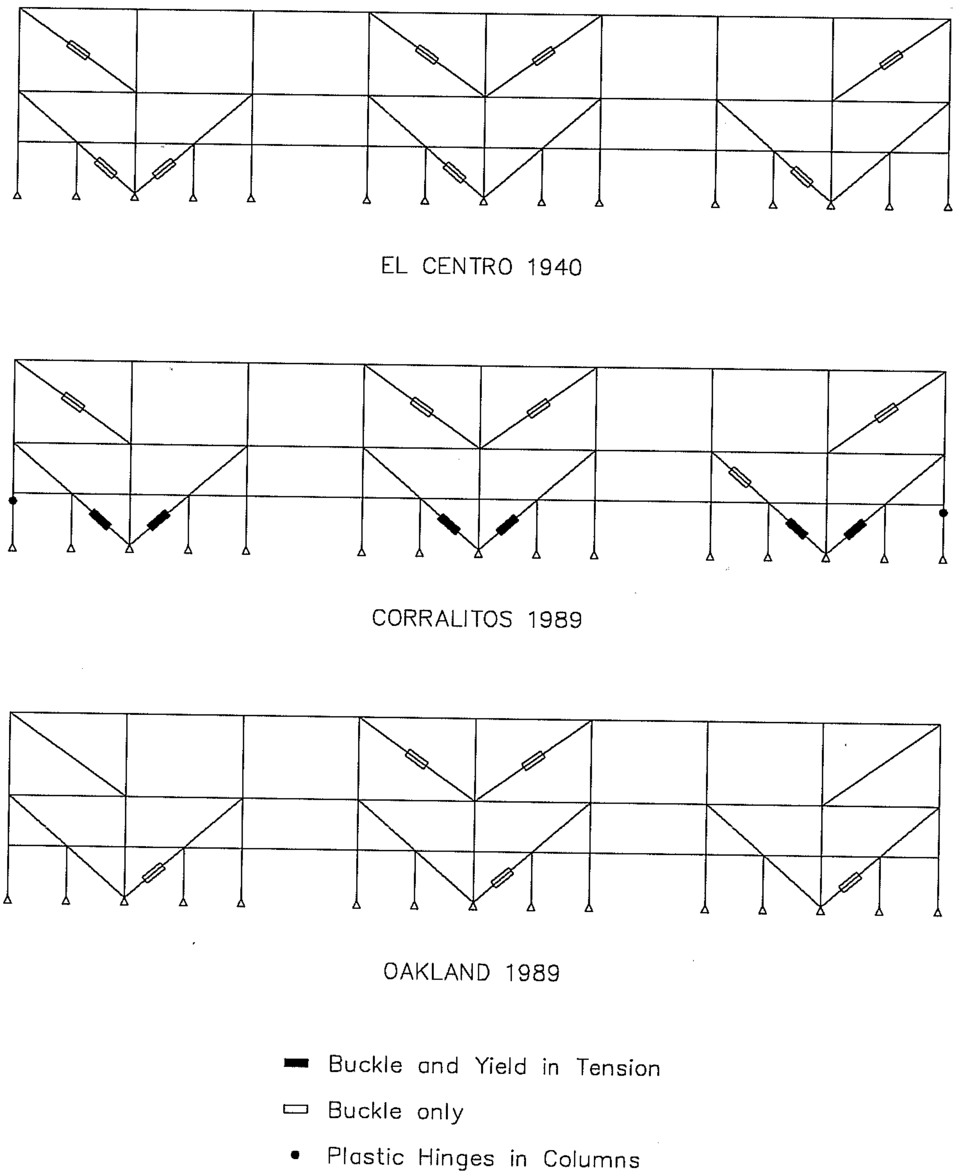


Figure 5.16 Inelastic Action in Existing Transverse Frames (Axes C and T) – Strengthened Structure

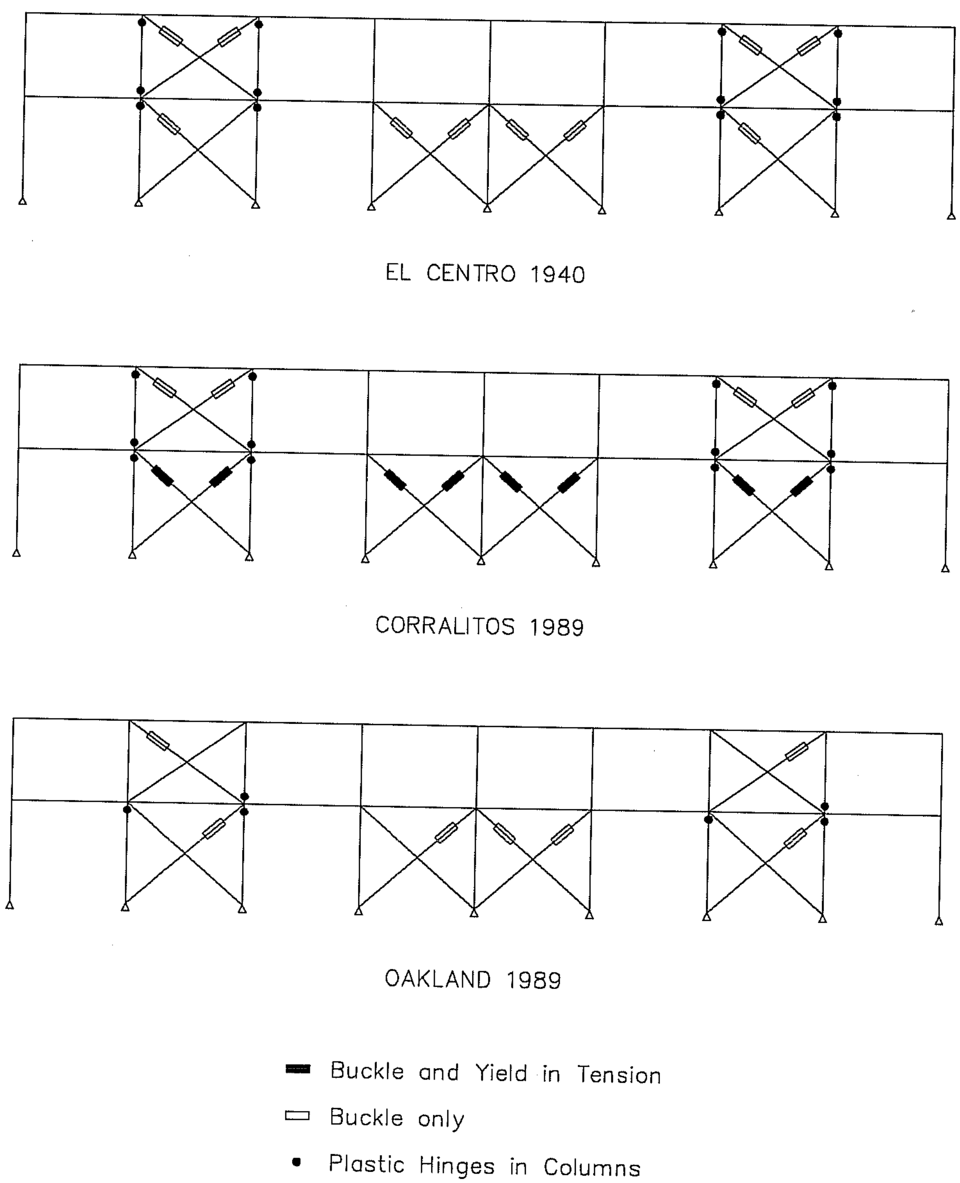


Figure 5.17 Inelastic Action in New Transverse Frame (Axis L) Strengthened Structure

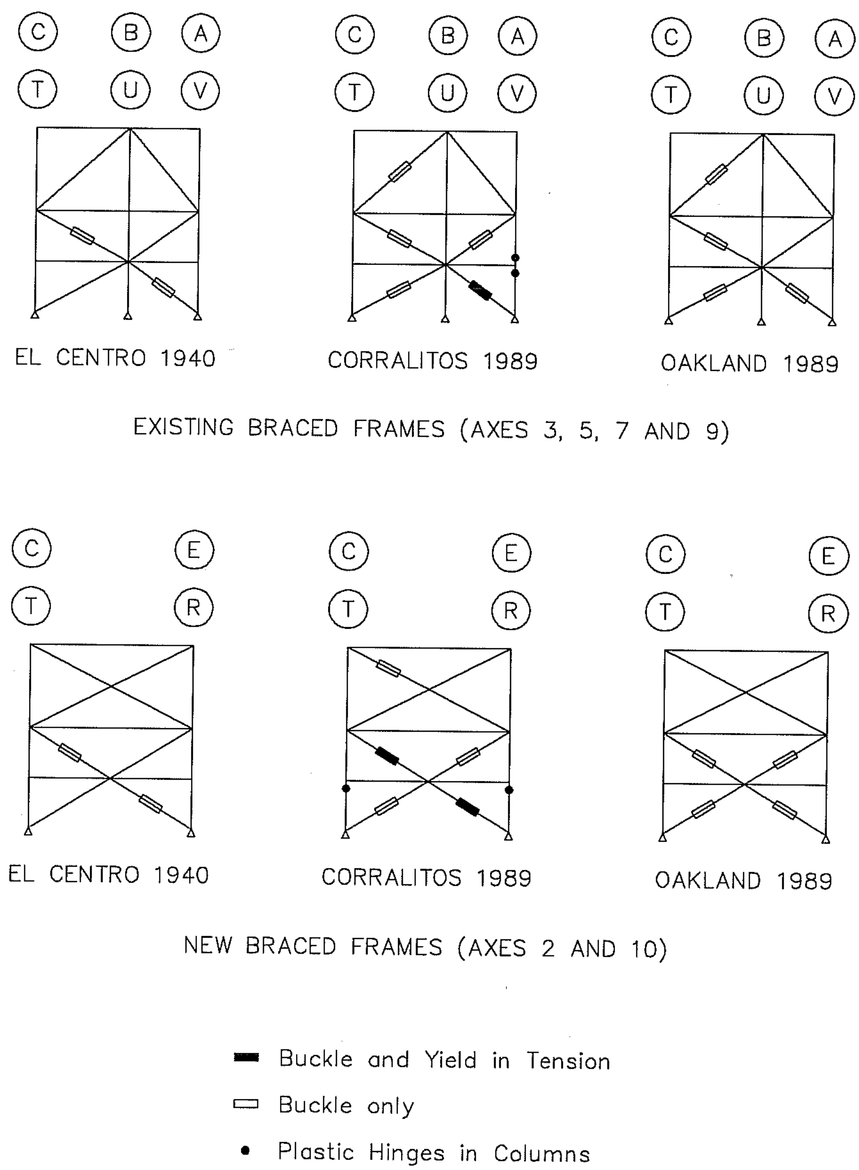


Figure 5.18 Inelastic Action in Longitudinal Frames Strengthened Structure

floor are also reduced. This reduction is desirable to avoid local buckling or fracture of those braces since the b/t ratio is large. None of the columns develop plastic hinges.

In the new transverse frame, buckling occurs in almost all the braces, as shown in Figure 5.17. The braces on the ground floor do not reach their yielding capacity in tension. Plastic deformations of the new braces are about the same as those experienced by the braces in the transverse frames of the unstrengthened structure. Plastic hinging of columns occurs on both levels of the exterior braced bays, as shown in Figure 5.17. Since the new columns will be attached to the existing columns, their buckling and bending capacity is increased. This is not reflected in the results of the analysis because only the properties of the new columns were used to model the frames. Therefore, even when plastic hinges are developed, buckling of the new columns is not expected.

In the longitudinal direction, buckling occurs only in the two braces on the existing braced frames shown in Figure 5.18. Yield capacity of the braces in tension is not reached. The columns do not develop plastic hinges. In the new braced frames, only two braces buckle and none reach their maximum tensile capacity (Figure 5.18). None of the columns develop plastic hinges.

5.6.1.2 Loma Prieta Corralitos 1989

In the transverse direction, all the braces on the ground floor reach their first buckling load and tensile capacity. Maximum plastic extensions in the braces are about the same as those experienced by the original structure. The ground floor braces have a considerable number of cycles with deformations in the inelastic range. There is no yielding of the braces on the interstitial floor and only one brace buckles. Braces on the upper floor buckle but do not reach tensile capacity. Maximum plastic extensions in these braces are also reduced. Hinging of the columns is considerably reduced. Plastic hinges develop only at the ground floor, in the exterior columns.

For the Corralitos record, the new transverse frame experiences considerable inelastic deformations. All the braces in the new transverse frame reach first buckling and tensile yield capacity (Figure 5.17). Columns in the exterior braced bays experience higher demand. Plastic hinges are developed in both floors.

In the longitudinal direction, only one brace on the first floor reaches its buckling and tensile yield capacities. The remaining braces do not reach their capacity in tension (Figure 5.18). The amount of plastic deformation in the braces is reduced compared to that experienced by the original braced frame. Column hinges are developed on both ground and interstitial levels as shown in Figure 5.18. The new braces also buckle and some reach their tensile capacity. Maximum plastic deformations are about the same as those experienced by the braces of the existing structure. Plastic hinges in the columns in axes C and T occur at both first and second levels.

5.6.1.3 Loma Prieta Oakland 1989

Buckling and plastic hinge formation in the transverse direction is shown in Figure 5.16. The braces reach their buckling capacity but do not reach their maximum tensile capacity. Inelastic deformations in the braces are less than those experienced by the original structure. The columns do not develop plastic hinges. The braces of the new transverse frame also reach buckling capacity without yielding in tension. Plastic hinges in the columns occur on the ground floor of the columns of the exterior braced bays, and in two of the columns in the upper floor (Figure 5.16).

In the longitudinal direction, the braces of the existing frames as well as the new braces reach their buckling capacity without reaching yielding in tension (Figure 5.18). Plastic deformations of the existing braces are smaller than those experienced before strengthening. None of the columns develop plastic hinges.

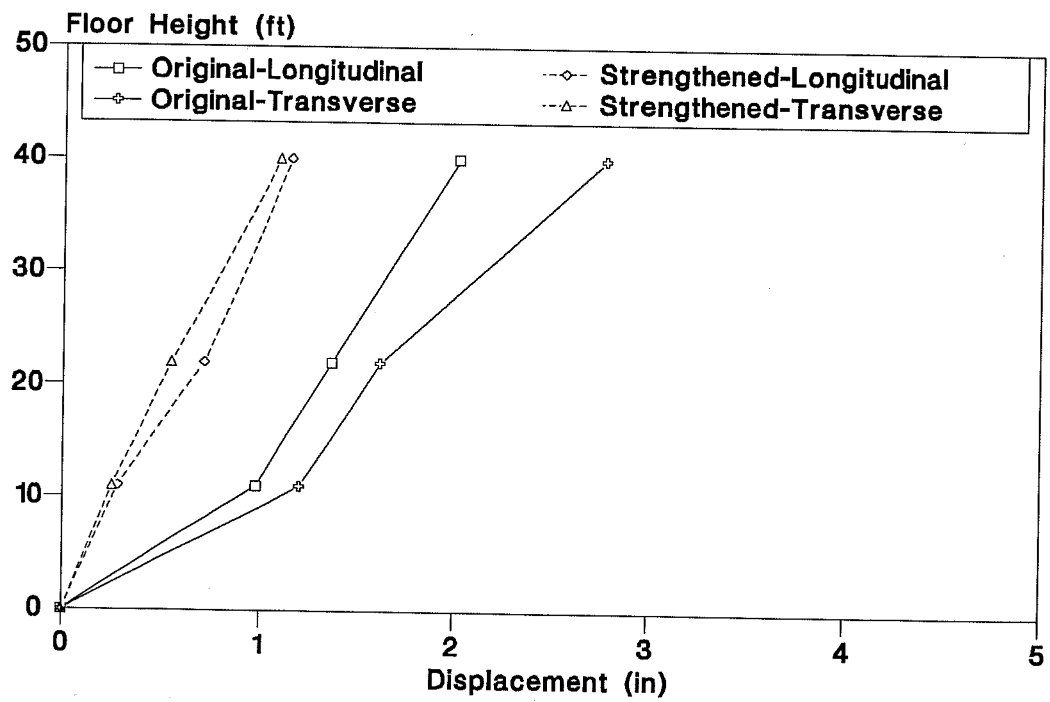
5.6.2 Floor displacements

Floor displacements and drifts obtained for the strengthened structure are shown in Table 5.9. The values shown correspond to the displacements in the structure at the time the top floor displacement is maximum. Displacements for the new frame in the transverse direction are also shown. The displacements of the new frame were computed at the upper and roof levels only. In the central part of the building, between axes C and T, there is no diaphragm at the interstitial level. The drifts are computed between ground level and upper level, and between upper level and roof. Since the upper level diaphragm is rigid, the displacements of the new frame and the existing braced frames in axes C and T are the same at this level. The roof diaphragm is not rigid, therefore the displacements of the new and existing frames are not the same as shown in Table 5.9. In the model of the structure, no connection was assumed between the roof level of the existing braced frames and the roof level in the new frame in axis L. In the real structure, the frames are connected through steel beams and roof trusses, nevertheless the stiffness of the whole diaphragm at roof level is not large enough to be considered rigid so that the forces could be distributed among frames according to their stiffnesses. A mass corresponding to the tributary area was assigned to each of the existing and new frames. The displacements, velocities and accelerations at the roof level of the new frame in the structure are expected to be between those computed at the new frame line (axis L) and the ones computed at the existing transverse braced frames in axes C and T, shown in Table 5.9 and 5.10.

TABLE 5.9 INELASTIC FLOOR DISPLACEMENTS AND INTERSTORY DRIFTS IN STRENGHTHENED STRUCTURE

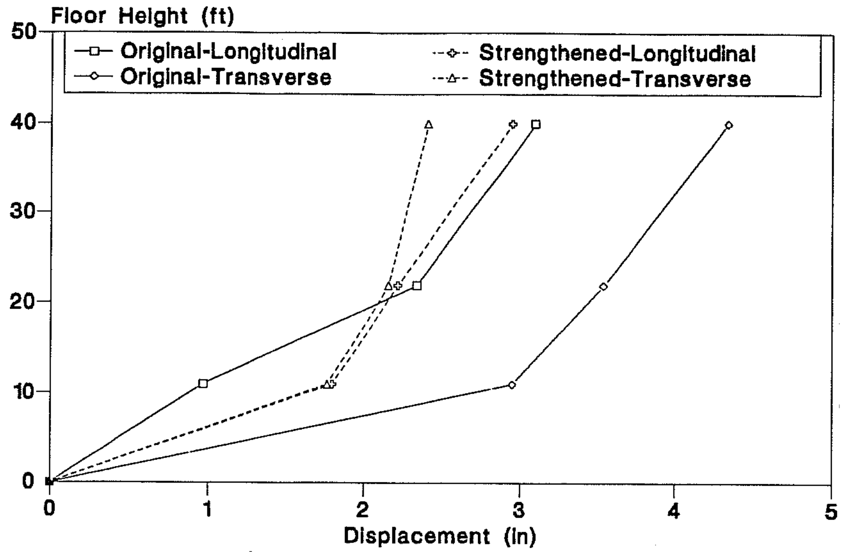
FLOOR	EL CENTRO		CORRALITOS		OAKLAND	
	Floor displ. (in)	Drift (%)	Floor displ. (in)	Drift (%)	Floor displ. (in)	Drift (%)
LONGITUDINAL DIRECTION						
Roof (40 ft)	1.16	0.20	2.95	0.34	1.32	0.24
Upper (22 ft)	0.72	0.33	2.22	0.32	0.80	0.36
Inter (11 ft)	0.29	0.22	1.80	1.36	0.33	0.25
EXISTING FRAMES IN TRANSVERSE DIRECTION						
Roof (40 ft)	1.10	0.25	2.41	0.12	1.21	0.21
Upper (22 ft)	0.56	0.23	2.16	0.30	0.76	0.28
Inter (11 ft)	0.26	0.20	1.77	1.34	0.39	0.30
NEW BRACED FRAME (AXIS L) IN TRANSVERSE DIRECTION						
Roof (40 ft)	1.82	0.58	3.07	0.42	1.91	0.53
Upper (22 ft)	0.56	0.21	2.16	0.82	0.76	0.29

A comparison between the displacements of the original structure and the displacements of the strengthened structure as indicated in Table 5.9 is presented in Figure 5.19. It is evident that the strengthening scheme effectively reduced the floor displacements and drifts. The computed drifts are below the 1% limit for control of non-structural damage. Only the drifts corresponding to the structure subjected to the Corralitos record exceed this limit, but meet the 1.5% requirement discussed previously.

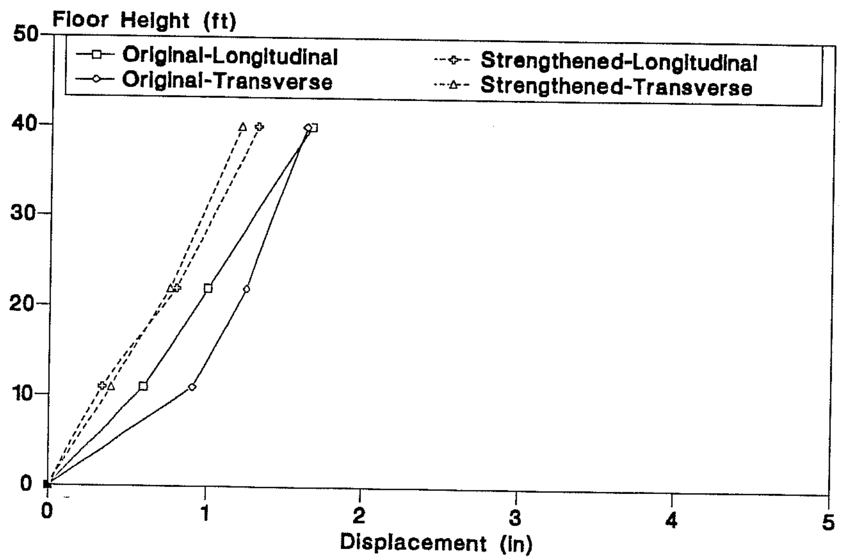


A. El Centro 1940

Figure 5.19 Inelastic Displacements Original vs. Strengthened Structure



B. Loma Prieta - Corralitos 1989



C. Loma Prieta - Oakland 1989

Figure 5.19 Inelastic Displacements Original vs. Strengthened Structure

5.6.3 Floor velocities and accelerations

Floor velocities and accelerations obtained for the strengthened structure are shown in Table 5.10. The velocities are about the same for the original and the strengthened structure. They do not seem to be affected very much by the change in stiffness. In the longitudinal direction, the accelerations obtained with the El Centro record are very close to those experienced by the original structure. For the Corralitos and Oakland records, accelerations in the strengthened structure are higher. Amplification of ground motion is particularly important for the Oakland record. Top floor amplification is 3.3, much higher than the 2.0 value usually predicted for the top of the building (Ref. 9 and 11). This seems to be related to the fact that more elements remain elastic under the Oakland record, resulting in a stiffer structure and higher floor accelerations.

In the transverse direction, along the existing frame lines, the accelerations experienced by the strengthened structure are higher than the ones obtained for the original frames. The addition of the new braced frame affected the structure's stiffness more in the transverse direction than the addition of braced bays in the longitudinal direction. This increase of stiffness results in higher floor accelerations. Particularly important is the acceleration amplification on the top floor, which surpasses the 2.00 factor usually recommended for design of mechanical and electrical elements (Ref. 9 and 11). At the new braced frame line, floor velocities and accelerations at the upper level are the same as the accelerations at the upper level in the existing braced frames (axes C and T), as shown in Tables 5.10 and 5.11. Accelerations in both existing and new frames at the upper level are the same since the diaphragm at this level is rigid, making all the frames move in the same manner. The velocities and accelerations of the new frame at the roof level are not the same as those shown for the braced

TABLE 5.10 FLOOR VELOCITIES AND ACCELERATIONS IN STRENGTHENED STRUCTURE

Level	Maximum Velocity (in/sec)			Maximum Acceleration (in/sec^2)					
	El Centro	Corralitos	Oakland	El Centro	Amplific.	Corralitos	Amplific.	Oakland	Amplific
LONGITUDINAL DIRECTION									
Roof	26.3	32.3	25.3	360.0 (0.93g)	2.7	401.0 (1.04g)	1.6	364.0 (0.94g)	3.3
Upper	9.3	20.1	13.7	252.0 (0.65g)	1.9	285.9 (0.74g)	1.2	230.0 (0.60g)	2.1
Interst	4.5	11.9	4.8	157.0 (0.41g)	1.2	287.0 (0.74g)	1.2	186.0 (0.48g)	1.7
TRANSVERSE DIRECTION									
Roof	17.4	26.4	15.9	295.0 (0.76g)	2.2	356.0 (0.92g)	1.5	268.0 (0.69g)	2.4
Upper	9.8	21.3	7.4	204.0 (0.53g)	1.5	216.0 (0.56g)	0.9	180.0 (0.47g)	1.6
Interst	5.8	20.7	5.0	122.0 (0.32g)	0.9	228.0 (0.59g)	0.9	145.0 (0.38g)	1.3

TABLE 5.11 FLOOR VELOCITIES AND ACCELERATIONS IN NEW TRANSVERSE FRAME

TRANSVERSE DIRECTION									
Level	Maximum Velocity (in/sec)			Maximum Acceleration (in/sec^2)					
	El Centro	Corralitos	Oakland	El Centro	Amplific.	Corralitos	Amplific.	Oakland	Amplific
Roof	26.3	32.3	25.3	360.0 (0.93g)	2.7	401.0 (1.04g)	1.6	364.0 (0.94g)	3.3
Upper	9.8	21.3	7.4	200.0 (0.52g)	1.5	215.9 (0.56g)	0.9	182.0 (0.47g)	1.6

frames in axes C and T because the roof diaphragm is not considered rigid for the analysis, as discussed in Section 5.6.2. At the roof level, the new frame is the stiffest of the three and therefore is the one that experiences highest accelerations. The strengthening technique effectively reduces inelastic deformations and floor displacements in the structure. However, floor accelerations and ground acceleration amplifications increase as the structure becomes stiffer.

5.7 LOAD COMPUTATION FOR NONSTRUCTURAL ELEMENTS

Design forces for architectural components and mechanical/electrical systems will be computed using the UBC Code, NEHRP Provisions and Tri-Services Manual, following the procedures described in Chapter III. These forces will be compared with those computed using the resulting floor acceleration histories for the original and strengthened structures.

5.7.1 Design forces computed with UBC 1991 Code

5.7.1.1 Architectural components

Design forces were computed for a typical architectural component. The following are the coefficients used to compute the equivalent lateral load:

I (Importance factor) = 1.0, the same used for the design of the building.

Z (zone factor) = 0.40, for the California area.

C_p (component coefficient) = 0.75, for typical architectural components.

The equivalent lateral load to be used in the design of the architectural component would be:

$$F_p = Z\ C_p\ I\ W_c = 0.4 * 0.75 * 1.0 * W_c = 0.30\ W_c$$

This value can be used for rigid components or rigidly fixed components, with $T < 0.06$ seconds. For flexible components, $T > 0.06$ seconds, the design force should by multiplied by 2, so $F_p = 0.60\ W_c$, where W_c is the total weight of the component.

5.7.1.2 Mechanical/electrical systems

For typical mechanical/electrical systems the same procedure is used to computed the design forces. Typical systems have component coefficients C_p of 0.75, resulting in the same design forces computed above. $F_p = 0.30\ W_c$ for rigid or rigidly fixed equipment, and $F_p = 0.60\ W_c$ for flexible or flexibly fixed equipment.

5.7.2 Design forces computed with NEHRP Provisions

5.7.2.1 Architectural components

The following are the coefficients used to compute design forces for a typical architectural component:

P (performance factor) = 1.0, equivalent to I=1.0 in the UBC Code.

A_v (Zone factor) = 0.4, for the California area.

C_c (Component coefficient) = 0.90, for the same type of element chosen to compute forces with the UBC Code.

The resulting design force is:

$$F_p = A_v\, C_c\, P\, W_c = 0.4 * 0.9 * 1.0 * W_c = 0.36\, W_c$$

No special provisions are specified to differentiate between rigid and flexible equipment.

5.7.2.2 Mechanical/electrical components

Coefficients used to compute design forces for a typical mechanical/electrical component are the following:

P (Performance factor) = 1.0, for good performance.

A_v (Zone factor) = 0.40, for the California area.

C_c (Component coefficient) = 2.0 for typical mechanical/electrical components, equivalent to the one chosen to compute forces with the UBC Code.

a_c (Amplification factor related to attachment of element to building)
a_c = 1.0, for fixed or direct attachment to building.

a_x (Amplification factor related to position of element with respect to height of the building) = 1.0 + 264/480 = 1.55, where 264 and 480 correspond to the height of upper level and total height of the building respectively, in inches.

The design force is computed as:

$$F_p = A_v\, C_c\, P\, a_c\, a_x\, W_c = 0.4 * 2.0 * 1.0 * 1.0 * 1.55 = 1.24\ W_c$$

5.7.3 Design forces computed using Tri-Services Manual

5.7.3.1 Architectural components

For design of architectural components the same forces used by the UBC Code are suggested. $F_p = 0.30\ W_c$ for rigid equipment and $F_p = 0.60\ W_c$ for flexible equipment.

5.7.3.2 Mechanical/electrical components

a. Rigid components with $T_a < 0.05$ seconds, where T_a is the period of the component: the following are the coefficients used to compute design forces for typical rigid mechanical/electrical components,

Z (zone factor) = 1.0, for the California area and equivalent to the 0.4 value specified by the UBC Code.

I (Importance factor) = 1.0, for standard occupancy.

C_p (Component coefficient) = 0.30, for all rigid components.

The design force is computed as:

$$F_p = Z\, I\, C_p\, W_p = 1.0 * 1.0 * 0.30 * W_p = 0.30\, W_p$$

where W_p is the weight of the component.

b. For flexible components with $T_a > 0.05$ seconds: the same values for Z, I and C_p are used. An amplification factor A_p that considers the period of the component is used. The value of this factor depends on the ratio of component period to building period (T_a/T). The procedure used to compute this factor is explained in Chapter III. Different vibration periods were chosen for the component to compute the design forces. A summary of the design forces obtained is shown in Table 5.12 for component periods of 0.10 sec, 0.40 sec and 1.00 sec.

TABLE 5.12 DESIGN FORCES FOR MECHANICAL/ELECTRICAL COMPONENTS ACCORDING TO TRI-SERVICES MANUAL

ORIGINAL STRUCTURE (T = 0.51 SEC)		
Period Ratio (T_a/T)	A_p factor*	Design Force F_p**
0.10/0.51=0.20	5.0	1.50 W_p
0.40/0.51=0.78	5.0	1.50 W_p
1.00/0.51=1.96	1.2	0.36 W_p
STRENGTHENED STRUCTURE (T = 0.41 SEC)		
0.10/0.41=0.24	1.8	0.54 W_p
0.40/0.41=0.98	5.0	1.50 W_p
1.00/0.41=2.44	1.0	0.30 W_p

* A_p factors from Figure 3.2

** $F_p = Z\, I\, A_p\, C_p\, W_p$

The periods of the structure were taken from the results presented in Section 5.3 and 5.6 of this Chapter for the original and the strengthened structure.

5.7.4 Design forces computed using results from Case Study

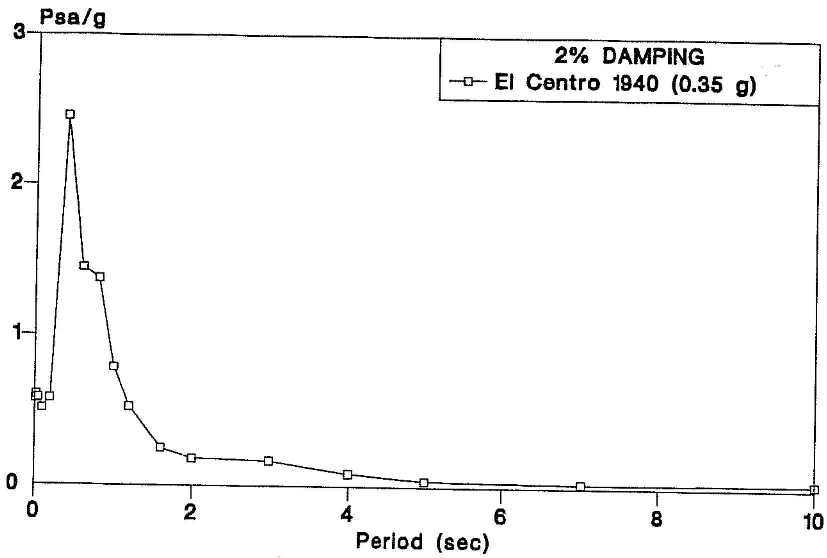
To compute the nonstructural element design forces, response spectra for the upper floor were computed based on the acceleration histories obtained for each of the records used for the original and the strengthened structure. The results will be presented for the El Centro and Corralitos records. Examples of the spectra obtained are shown in Figure 5.20. These spectra were computed using a computer program developed by Dr. Richard Klingner at The University of Texas at Austin (Ref. 17).

From the response spectra computed, the maximum acceleration values were chosen for periods of the component of 0.05 sec, 0.10 sec, 0.40 sec and 1.00 sec. The design forces are computed as:

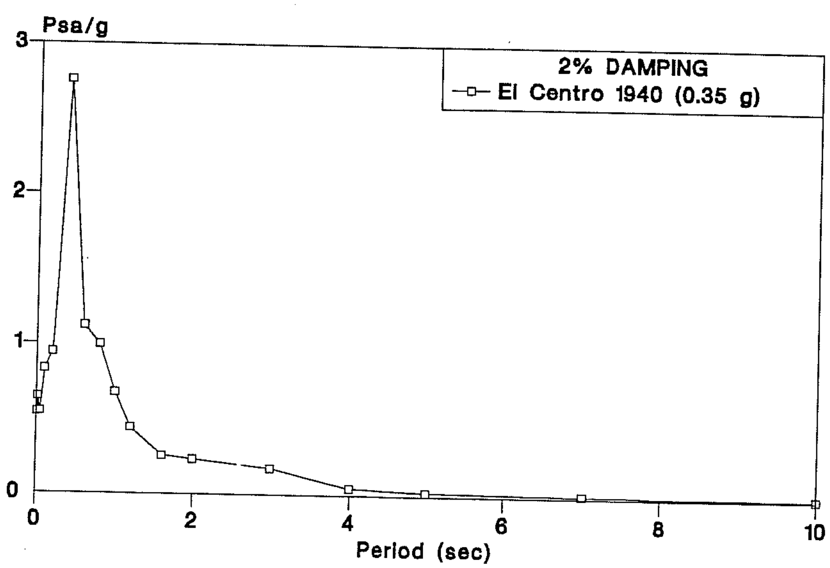
$$F_p = P_{sa}/g * W_c$$

where P_{sa} is the pseudo-acceleration value from the response spectra divided by the gravity acceleration g, and W_c is the weight of the component. In Table 5.13 the resulting design forces are presented.

It is important to notice that elements having periods between 0.2 sec and 0.8 sec experience the larger acceleration amplification, and therefore should be designed to withstand larger forces. Elements with periods close to the period



A. Original Structure - Longitudinal Direction



B. Strengthened Structure - Transverse Direction

Figure 5.20 Computed Upper Floor Response Spectra of the building, and at the same time, within the range of the main period of the ground motion, will experience larger acceleration amplifications. Dynamic interaction between structure and element is important to consider for flexible elements, especially those within the above mentioned period ranges.

TABLE 5.13 NONSTRUCTURAL ELEMENT DESIGN FORCES AT UPPER LEVEL COMPUTED USING RESULTS FROM CASE STUDY

ORIGINAL STRUCTURE				
Record	Design Forces F_p			
	T_a = 0.05 sec	T_a = 0.10 sec	T_a = 0.40 sec	T_a = 1.00 sec
El Centro 1940	0.58 W_c	0.71 W_c	2.46 W_c	1.16 W_c
Corralitos 1989	0.56 W_c	0.70 W_c	1.16 W_c	1.19 W_c
STRENGTHENED STRUCTURE				
El Centro 1940	0.63 W_c	0.86 W_c	2.75 W_c	0.67 W_c
Corralitos 1989	0.75 W_c	1.16 W_c	2.12 W_c	0.89 W_c

5.7.5 Comparison of design forces for rigid elements

In Table 5.14 the computed design forces for rigid nonstructural elements and mechanical/electrical equipment are summarized.

The forces suggested by the studied building codes for design of architectural components are lower than the values computed using the floor accelerations at the building floor. For mechanical/electrical equipment, the forces are again lower, except for those computed using the NEHRP Provisions. These forces are amplified considering a ground motion amplification factor of

1.55 (see Section 5.7.2.2), close to the amplifications experienced by the upper floor for the El Centro and Corralitos earthquakes (Tables 5.7 and 5.10).

TABLE 5.14 DESIGN FORCE COEFFICIENTS FOR RIGID NONSTRUCTURAL ELEMENTS

Element (Ta<0.05sec)	Design Force Coefficient C				
	UBC 91	NEHRP	Tri-Services	Case Study Original str.	Case Study Strength.str.
Architectural component	0.30	0.36	0.30	0.58*	0.63*
Mechanical/ Electrical Component	0.30	1.24	0.30		

* Forces shown correspond to those computed with El Centro (see Table 5.13)

5.7.6 Comparison of design forces for flexible nonstructural elements

In Table 5.15 the computed design force coefficients for flexible architectural and mechanical/electrical components are summarized.

In general, UBC design forces are lower than the forces computed using amplified acceleration histories at the upper floor level. The NEHRP Provisions and the Tri-Services Manual also appear to underestimate the design forces for architectural components. A distinction in the NEHRP Provisions should be made regarding the flexibility of the component being designed.

TABLE 5.15 DESIGN FORCE COEFFICIENTS FOR FLEXIBLE NONSTRUCTURAL ELEMENTS

<table>
<tr><th colspan="7">ARCHITECTURAL COMPONENTS</th></tr>
<tr><th rowspan="3">Element Period (sec)</th><th colspan="6">Design Force Coefficients</th></tr>
<tr><th rowspan="2">UBC 1991</th><th rowspan="2">NEHRP</th><th colspan="2">Tri-Services</th><th rowspan="2">Case Study Original str.</th><th rowspan="2">Case Study Strength. str.</th></tr>
<tr><th>Origin.</th><th>Strength.</th></tr>
<tr><td>0.10</td><td>0.60</td><td>0.36</td><td colspan="2">0.60</td><td>0.71*</td><td>0.86*</td></tr>
<tr><td>0.40</td><td>0.60</td><td>0.36</td><td colspan="2">0.60</td><td>2.46</td><td>2.75</td></tr>
<tr><td>1.00</td><td>0.60</td><td>0.36</td><td colspan="2">0.60</td><td>1.16</td><td>0.67</td></tr>
<tr><th colspan="7">MECHANICAL/ELECTRICAL COMPONENTS</th></tr>
<tr><td>0.10</td><td>0.60</td><td>1.24</td><td>1.50</td><td>0.54</td><td>0.71</td><td>0.86</td></tr>
<tr><td>0.40</td><td>0.60</td><td>1.24</td><td>1.50</td><td>1.50</td><td>2.46</td><td>2.75</td></tr>
<tr><td>1.00</td><td>0.60</td><td>1.24</td><td>0.36</td><td>0.30</td><td>1.16</td><td>0.67</td></tr>
</table>

* Forces shown correspond to those computed with El Centro (see Table 5.13)

For electrical/mechanical components, UBC specifies forces lower than those computed from the Case Study data. NEHRP and Tri-Services Manual specify higher forces than the ones proposed by the UBC. NEHRP takes into account ground motion amplification on the floor and Tri-Services Manual considers the influence of the flexibility of the element. Nevertheless, design forces for elements with periods between 0.2 and 0.8 seconds are lower than the forces computed using the computed accelerations in the building. Forces representative of this range are those shown in Table 5.15 for a period of 0.40 seconds.

In general, design forces for architectural elements proposed by current building codes are lower than those obtained from the computed response of the building under different ground motions. Design forces for mechanical/electrical systems proposed by current codes are usually higher than the ones proposed for architectural components, as a result of considering ground motion amplification factors and influence of flexibility of the elements in the design procedure. The design forces for mechanical/electrical systems may actually be closer to the forces experienced by the element in the real structure. Though code provisions seem to underestimate the forces resulting from the computed accelerations in the building, recent earthquakes have shown that failure of nonstructural elements was a result of poor detailing or lack of proper anchorage rather than low design forces. However, the results of the case study indicate that more detailed analyses should be performed when critical equipment vital for the facility function is designed.

CHAPTER VI
SUMMARY AND CONCLUSIONS

6.1 SUMMARY

A brief description of typical high-tech industry facilities is presented, including common architectural and structural systems, nonstructural components and electrical/mechanical equipment. Summaries of structural and nonstructural damage suffered past earthquakes are also presented.

A review of current code provisions regarding structural and nonstructural design criteria is given. The codes studied are the UBC 1991 Code, the NEHRP Provisions and the U.S. Department of Defense Tri-Services Manual. Analysis and design criteria for nonstructural systems are also reviewed.

Seismic protection techniques for elements usually encountered in high-tech industrial buildings are presented. Elements included are raised floors, computer equipment, tape and disk storage, ceilings, partitions, and mechanical and electrical systems. Seismic protection techniques include analysis and design of new and existing elements, and general installation guidelines.

A case study of a typical existing industrial building is presented. The study is mainly centered on the nonstructural aspects of the response of the building. The study includes elastic and inelastic analyses of the existing structure. Discussion is focused on resulting floor displacements, velocities and accelerations. Possible design and response deficiencies are identified. General criteria for desirable behavior are established for high-tech industrial buildings. A strengthening scheme is proposed to improve the response of the building to meet this criteria. An inelastic analysis of the strengthened structure is presented. Discussion is again focused on floor displacements, velocities and accelerations. Finally, a comparison of design forces for typical nonstructural elements is presented. Forces are compared using the building codes mentioned above and the results from the analysis of the existing and the strengthened structure.

6.2 CONCLUSIONS

6.2.1 General

In general, buildings occupied by the high-tech industry survived the recent Loma Prieta earthquake with minor structural damage. Most of the economic loss was the product of nonstructural damage. Despite the consequences of earthquake damage, many companies do not dedicate appreciable efforts to prepare for future events. In many cases, decisions regarding building leases are taken without considering the building's structural adequacy, vital information regarding equipment design criteria is held privately and does not appear to be shared among companies, information on building and nonstructural element performance during earthquakes is not disseminated. It is important for the industry in general to realize that sharing this information with the engineering profession is vital to determining the adequacy of current design practices and minimizing damage and economic losses due to earthquake motions. It is also important to realize that seismic preparedness programs are necessary to minimize damage to structures and equipment. Such programs should include rehabilitation of existing buildings housing high-tech facilities, revision or formulation of adequate design and installation procedures for the nonstructural elements and equipment within the facility, and periodic inspection of the facility and its architectural and mechanical/electrical systems to insure that proper installation techniques are used and that systems adequate at installation have not been changed by the occupants. Most of all, a revision of the decision making process is necessary. The adequacy of the structural system of a building may be as important as its location when computing economic advantages. Potential losses due to inadequate structural systems or inadequate design and installation of nonstructural systems can be devastating for the industry.

Nowadays, with the extensive use of computer and data processing equipment, it is difficult not to classify almost all business facilities as "high-tech" facilities. Most of the equipment discussed in this study is used in banks, insurance companies, engineering firms, and other office buildings: raised floors, computers and other electronic equipment, tape and disk storage systems. The use of architectural components such as movable partitions and suspended ceilings is also extensive. It is important to realize that for these facilities, as for the "high-tech" industry, damage due to earthquakes can result in large economic losses and closed businesses. Seismic preparedness is as important for these businesses as it is for the "high-tech" facilities discussed in this study.

6.2.2 Performance of high-tech industries in past earthquakes

In general, structural performance of high-tech facilities during moderate earthquakes, as the Loma Prieta 1989 earthquake, has been adequate. Buildings that have suffered damage have been those of known hazardous construction (pre-1972 San Fernando tilt-up systems), or modern buildings with construction deficiencies or poor detailing. Nonstructural performance has not been so satisfactory. Damage has resulted from poor detailing and anchorage schemes.

Elements are not designed or installed to withstand dynamic motions, and consequently suffer extensive damage during earthquakes.

6.2.3 Current building code provisions

Taking advantage of the structure's ductility may result in large deflections during moderate and strong earthquakes. The limits given in the codes to drifts computed from an elastic analysis of the structure are not enough to insure that the final building displacements will not result in nonstructural damage. Limitations to maximum deflections of buildings including possible inelastic effects under moderate and strong earthquakes are necessary to minimize nonstructural damage. This may result in higher design forces and larger or stiffer elements.

Architectural and mechanical/electrical system design provisions usually do not consider the dynamic characteristics of the elements, nor the interaction between structure and element response. Comparison of suggested design forces in the UBC, NEHRP Provisions and Tri-Services Manual, and forces computed using the results of the analysis of a typical structure indicates that the design code forces can underestimate the forces experienced by the elements during an earthquake. Variables such as ground motion amplification with height in the building and dynamic characteristics of the nonstructural element (period of vibration) should be considered when computing design forces for these elements.

6.2.4 Seismic Protection Techniques

The proper analysis and design techniques, and the best procedure to install or retrofit a particular element must be chosen based on the importance of the element, the level of risk that can be allowed, likely damage and time to repair or replace. This criteria is normally set by the facility operator and must be known by the designer prior to beginning the design procedure. It may not be sufficient to design a structure to meet minimum requirements for structural performance following simple code procedures. More accurate force estimations and analyses may be necessary.

Particularly critical elements in a high-tech or data processing facility are raised floors, computer equipment and tape and disk storage elements. Loss or damage to any of these elements can result in serious economic losses or loss of information that may be vital for the function of the facility. Other elements that have shown to be vulnerable to earthquake motions are ceilings, partitions and electrical/mechanical equipment. Attention must be paid to proper installation of such elements.

6.2.5 Case study of typical structure

A typical office and/or light manufacturing building used by the "high-tech" industry was studied. The structure was designed and built between 1982 and 1985. It has three stories, with an area of approximately 71000 sq.ft, divided into two sections: a central open area of 64000 sq.ft. where movable partitions and manufacturing equipment are placed, and the east and west end sections that serve to locate elevators, stairs, restrooms and other services.

The lateral load resisting system of the structure consists of braced steel frames and composite concrete slabs. The braced frames are located in the east and west ends of the building, and are formed with W-section beams and columns, and square-tubular-section braces. In the central part of the building, the vertical load resisting system consists of steel W-columns and deep trusses located at the upper and roof levels. These frames are not considered to be a part of the lateral load resisting system. The lateral forces are transmitted to the braced frames through the concrete diaphragm.

6.2.5.1 Elastic analysis of original structure

Most of the elements in the building meet current UBC Code provisions. Though some braces in the transverse direction have a demand higher than the computed design capacity, the overall structure could be considered adequate to resist the forces prescribed by this Code. Computed elastic displacements are well within the allowable values suggested by the UBC.

In terms of design capacity, the structure meets the NEHRP provisions. Demand on the braces exceeds the design capacity only for the braces at the roof level on the transverse direction. All the braces at the roof level exceed the b/t ratio specified by the AISC-Earthquake Provisions. This indicates a possibility of local buckling and fracture under repetitive loading. Computed displacements using the forces specified by the NEHRP are within the allowable values suggested in the these provisions.

6.2.5.2 Inelastic Analysis of the original structure

Inelastic action in the structure under the El Centro and Corralitos earthquakes is extensive. Analyses indicate that buckling of almost all the braces would be expected under these earthquakes. It is likely that in a major earthquake, this building could experience structural damage that would take a long time to repair. Floor displacements and drifts are very high, especially in the transverse direction. Drifts in the first floor of 1.35% for El Centro and 2.33% for Corralitos strongly indicate a possibility of nonstructural damage. Maximum floor velocities are usually under 30 in/sec. Ground acceleration amplification factors are within those suggested for mechanical/electrical equipment design in the NEHRP Provisions and in Reference DPM. Acceleration amplification is higher in the longitudinal direction, where the structural system is stiffer due to the presence of a higher number of braced bays.

Due to the extensive inelastic action in the frames and the large floor displacements, a strengthening scheme is suggested.

6.2.5.3 Inelastic analysis of strengthened structure

Adding braces in both longitudinal and transverse directions effectively reduces the demand on the existing elements, hence reducing inelastic action in the frames. Buckling of the braces is expected under the earthquake records used. Though this is how the braced frame system is expected to respond, it indicates a possibility that buildings designed to comply with modern codes may experience structural damage during major earthquakes that could take a long time to repair and would affect operations in the building. Drifts are reduced to a maximum of 0.6% in the transverse direction for El Centro, and 1.36% in both transverse and longitudinal directions for Corralitos. Both values are within the 1% and 1.5% limits set for each record respectively. Displacements at the roof level are also reduced, minimizing the possibility of damage to the roof diaphragm.

Floor velocities remain almost the same, under 34 in/sec. Floor accelerations increase in both longitudinal and transverse direction, with the subsequent increase in ground acceleration amplification factors. Most of these values exceed those proposed in References NEHRP and DPM. Stiffening the structure reduces floor displacements but increases floor accelerations.

In general, performance of the strengthened structure is adequate. Performance in terms of equipment criteria (floor velocities or accelerations) cannot be assessed due to lack of information on equipment design criteria. This information is very important to completely assess the adequacy of existing industrial facilities to withstand minor and moderate earthquakes without significant interruption of operations.

6.2.5.4 Computed design loads for nonstructural systems

In general, loads computed using current code design provisions are lower than those computed using the results of the analysis of the building. This stresses the importance of including variables such as ground motion amplification factors and dynamic characteristics of the nonstructural elements in the design procedures. Recent earthquakes have shown that failure of nonstructural elements is usually a result of poor detailing and/or lack of adequate anchorage rather than low design forces. However, the results of the case study show that in the case of high-tech industrial buildings, simplified design procedures such as those proposed by the UBC Code for nonstructural elements may not be the best to use for critical equipment vital for the facility function. Dynamic analysis of critical equipment, considering interaction with the response of the building is suggested in such cases.

6.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The following aspects are needed to further improve structural and nonstructural design criteria regarding high-tech industrial facilities, and to asses the importance of continuity of operations in the design and retrofitting of high-tech industrial buildings:

a. A more detailed study of damage suffered in buildings used by high-tech industries during past earthquakes is needed, identifying elements likely to be damaged and impact on continuity of operations.

b. Information on computer, and in general production or data-processing equipment design criteria is needed. Dynamic characteristics of equipment and vulnerability of vibration-sensitive equipment is needed to determine analysis and design procedures, and installation techniques. This information is also needed to assess adequacy of different structural systems to minimize damage to vibration-sensitive equipment.

c. Further study of existing industrial buildings with different structural systems is needed to assess their performance under earthquake motions, and to examine the adequacy of current code provisions for design of structural and nonstructural elements.

d. Nonstructural element and electrical/mechanical systems design methods should be studied further. Importance of variables such as ground motion amplification with height in the building and dynamic characteristics of nonstructural elements and equipment should be assessed to formulate improved design procedures.

BIBLIOGRAPHY AND REFERENCES

1. AISC, "Manual of Steel Construction - Load and Resistance Factor Design", American Institute of Steel Construction, Chicago, 1986.

2. AISC, "Seismic Provisions for Structural Steel Buildings - Load and Resistance Factor Design", American Institute of Steel Construction, Chicago, 1990.

3. Benuska, L. et al., "Loma Prieta Earthquake Reconnaissance Report", EERI, Earthquake Spectra, Supplement to Vol. 6, May 1990, pp. 189-238 and 339-376.

4. Departments of the Army, the Navy and the Air Force, "Seismic Design for Buildings", Army TM 5-809-10, Navy NAVFAC P-355, Air Force AFM 88-3 Chapter 13, February, 1982.

5. El-Tayem, Adel A. and Goel, S.C., "Effective Length Factor for the Design of X-Bracing Systems", AISC Engineering Journal, Vol. 23, No. 1, 1st. Quarter, 1986.

6. Estes, K., "The Whittier Narrows, California Earthquake of October 1, 1987 - Ticor Title Insurance Building Damage", Earthquake Spectra, EERI, Vol. 4, No. 2, May 1988, 265-276 pp.

7. EQE, "The July 16, 1990 Philippines Earthquake: A Quick Look Report", EQE Engineering, San Francisco, 1990.

8. EQE Engineering, "The October 17,1989 Loma Prieta Earthquake: A Quick Look Report", EQE Engineering, San Francisco, 1989.

9. Federal Emergency Management Agency, "NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings - Part 1 Provisions and Part 2 Commentary", Earthquake Hazards Reduction Series 17, Building Seismic Safety Council, Washington, 1988.

10. Federal Emergency Management Agency, "Techniques for Seismically Rehabilitating Existing Buildings (Preliminary)", Earthquake Hazards Reduction Series 49, March 1989.

11. Finance, Insurance and Monetary Services Committee, "Data Processing Facilities: Guidelines for Earthquake Hazard Mitigation", VSP Associates Inc., Sacramento, 1987.

12. Habibullah, A., "ETABS - Three Dimensional Analysis of Building Systems User's Guide", Computers and Structures Inc., Berkeley, 1990.

13. Hanson, R.D. and Goel, S.C., "Behavior of Concentrically Braced Frames and Design of Bracing Members for Ductility", Proceedings of the 56th. SEAOC Convention, San Diego, 1987.

14. Hassan, O.F. and Goel, S.C., "Modelin of Bracing Members and Seismic Behavior of Concentrically Braced Steel Structures", Research Report UMCE 91-1, The University of Michigan, Ann Arbor, 1991.

15. International Conference of Building Officials, "Uniform Building Code", 1991 Edition, California, 1991.

16. Jain, A.K. and Goel, S.C., "Hysteresis Model for Steel Members Subjected to Cyclic Buckling and Cyclic End Moments and Buckling - User's Guide for El9 and El10 of DRAIN-2D", Report No. UMEE-78R6, The University of Michigan, Ann Arbor, 1978.

17. Klingner, R., "Program Spectra", Notes from the course Dynamic of Structures, The University of Texas at Austin, Austin, 1990.

18. Lagorio, H., "Earthquakes: An Architect's Guide to Nonstructural Seismic Hazards", John Wiley & Sons Inc., New York, 1990, 107-138 pp.

19. Naeim, F. Editor, "The Seismic Design Handbook", Van Nostrand Reinhold, New York, 1989, 119-141 and 387-412 pp.

20. Phipps, M., "Performance of Buildings for High-Tech Industries in Silicon Valley in the Loma Prieta Earthquake", Proceedings of a Workshop on Evaluation, Repair, and Retrofit of Structures - U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Gaithersburg, MD, USA, May 1990.

21. Powell, G.H., "DRAIN-2D User's Guide", Report No. 73-22, Earthquake Engineering Research Center, The University of California at Berkeley, Berkeley, 1973.

22. Swan, S., Miller, D. and Yanev, P., "The Morgan Hill Earthquake of April 24, 1984 - Effects on Industrial Facilities, Buildings, and Other Facilities", EERI,

Earthquake Spectra, Vol. 1, No. 3, May 1985, 457-568 pp.

23. Taly, N., "The Whittier Narrows, California Earthquake of October 1, 1987 - Performance of Buildings at California State University, Los Angeles", EERI, Earthquake Spectra, Vol. 4, No. 2, May 1988.

VITA

Marianela Picado was born in San Jose, Costa Rica, on September 10, 1964, the daughter of Romulo Picado Chacon and Mayra Maykall Mora. In February, 1982, she entered the Universidad de Costa Rica in San Jose, Costa Rica, where she graduated as Licenciada en Ingenieria Civil in January, 1987. In 1987, she became a member of the Colegio Federado de Ingenieros y Arquitectos de Costa Rica (CFIA). In 1986, she became an Assistant Engineer for the firm Franz Sauter y Asociados Ltda., a design and consulting office in San Jose, Costa Rica. From January, 1987 to August, 1989, she worked for the same firm as Structural Engineer. In September, 1989, she entered the Graduate School of The University of Texas.

Permanent address: P.O. Box 5890-1000
San Jose, COSTA RICA

This thesis was typed by the author.