SEISMIC BASE ISOLATION:
A FIVE-STORY BUILDING EXAMPLE

By

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Submitted to

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Abstract

The objective of this investigation is to outline all relevant issues concerning the conceptual design of base isolated structures. A five-story building example is used to compare both fixed based and isolated based schemes. Different types of isolation systems are discussed. Techniques used for modeling the complex superstructure and nonlinear isolation system are also described.

Nonlinear dynamic analyses are carried out using the finite element program SAP2000. Time history analysis was conducted for the Loma Prieta earthquake. The mechanical properties of lead-core rubber bearings, such as those provided by Dynamic Isolation Systems, Inc. are explored in depth and analyzed using the DIS in-house program ISOLATE. Base Isolation proves to be effective in reducing the induced accelerations on a structure by increasing the period of vibration.

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CHAPTER 1

MOTION DUE TO EARTHQUAKES

1.1 Earthquake Forces

In order to understand the motion of buildings due to earthquakes, it is necessary to fully identify the applied forces. This section will outline the major technical issues associated with earthquakes.

Ground Motion

The dynamic response of a building to earthquake ground motion is the most important cause of earthquake-induced damage to buildings. The soil and ground beneath buildings can also be a major source of damage. The abrupt movement along a plane of faults causes most earthquakes. This shift of earth releases bursts of energy to form waves. As the waves travel energy is released through the complex soil and rock material. Therefore, the closer a structure is to a fault, the greater the potential damage. The seismic waves first strike the building at the foundation or the substructure. As the forces move through the building, the building deforms to accommodate the loads.

The site of the structure must be examined carefully to fully understand and analyze the effect of seismic loads. The induced loads are dependent on the fault, or source effect, the path of travel, and the local site effect.

Building Frequencies and Periods

An engineer must understand the primary characteristics of earthquake ground motions, which are:

- the duration
- the amplitude of displacement
- the amplitude of velocity
- the amplitude of acceleration
- the frequency of the ground motion.

Frequency, the number of complete cycles of vibration made by the wave per second, describes the waves that travel from the faults. The site, soil, and path of the earthquake create surface ground motions that can be described as a complex superposition of vibrations of different frequencies.

The building also has a set of frequencies that characterize its response. The lowest frequency is called the fundamental frequency. As the induced loading frequency approaches the fundamental frequency, a phenomenon known as resonance initiates. This is described in the next section. A rule of thumb that is quite helpful in preliminary studies relates the fundamental period (inverse of the frequency) to the height of the building.

The fundamental period, T, is related to the number of stories of the structure by the following approximation:

$$T = 0.1*n$$

For a five-story building, the fundamental period is one half of a second. Therefore, the building experiences a full cycle of lateral motion every half second.

Resonance

Again, as the frequencies of the ground motion tend towards the building's natural frequency, the building and the ground motion approach the state of resonance. Resonance can be catastrophic because resonance amplifies the building's response. The Tacoma Narrows Bridge is a famous example of the consequence of resonance. If an earthquake with a period of 0.3 seconds hits the Back Bay in Boston, the amount of damage would be overwhelming (the Back Bay is lined with three-story masonry flats). Tall buildings close to the Back Bay, such as the John Hancock, would suffer little to no damage because its natural frequency is close to 6 seconds.

Response Spectra

A response spectrum concisely represents the building's range of responses to ground motion of different frequency contents. The spectrum is often represented as a graph, which plots the maximum response values of acceleration, velocity and displacement against period and frequency. This graph helps engineers determine the amount of acceleration that a building undergoes during an earthquake. The amount of damage a structure may undergo will be proportional to the inter-story displacements of the structure. Analyzing the structure to find its resonant frequencies is of primary importance when investigating the seismic response of the structure.

1.2 Building Response

Ground Acceleration

In order to understand ground acceleration, one must employ Newton' s Second Law of Motion that states the mass of the building times the building acceleration is equal the force acting on the building. Assuming that the mass does not change, as the acceleration of the building increases, the force that affects the building increases. Therefore, decreasing the building acceleration is an objective of the engineer in charge of mediating structural damage caused by earthquakes. A building that is suddenly forced to move very quickly suffers a great deal of damage.

Inertial Forces

The product of the mass times the acceleration is defined as the inertial force. The inertia force due to ground motion causes the structure to deform, imposing strains upon the beams, columns, load-bearing walls, floors, as well as the connections. If this force is decreased, by means of base isolation, the building responds in a less destructive manner.

Stiffness and Ductility

Stiffness is dependent on the materials, the height, the connections, the lateral load components, the diaphragms, and the building's components. Stiffness greatly affects the building's uptake of earthquake generated force but other methods allow a building to be more flexible. This will be discussed later as base isolation. The ductility of a structure is in fact one of the most important factors affecting the performance. One of the primary tasks of an engineer designing a building to be earthquake resistant is to ensure that the building will possess enough ductility to withstand the size and types of earthquakes it is likely to experience during its lifetime.

Damping

By definition, damping is the decay of the amplitude of vibration over time. Motion during an earthquake has a complex, vibratory nature as the building moves back and forth many times. Without damping, a vibrating object would never stop vibrating, once it had been set in motion. All buildings have different properties and therefore have unique damping characteristics. The damping is due to internal friction and the absorption of energy from the structural and nonstructural elements. The more the building's intrinsic damping the faster the building will effectively dissipate the earthquake energy. Without damping a structure will undergo extensive damage to major components.

Damage

Damage comes in many varieties from minor cracking in surface finishes to major cracks in structural elements. This may lead to complete structural failure.

CHAPTER 2

BASE ISOLATION

2.1 Introduction to Base Isolation

Traditional Design

The traditional design methods to mediate structural damage due to earthquakes are shear walls, braced and moment frames, diaphragms, and horizontal trusses. These components need to have adequate strength, stiffness and inelastic deformation capacity to withstand earthquake-generated forces. One earthquake resistant design strategy does not strengthen or stiffen the structure but reduces the forces that will act on the superstructure by reducing the loading through an isolation medium.

Base Isolation

The concept of isolating the superstructure from the substructure has always been an elegant idea in theory, but only recently has it become a viable solution. The goal is to have a flexible material in the horizontal plane that is capable of preventing energy flow into the superstructure. This flexibility increases the superstructure's period, which, in turn, reduces the induced acceleration. Figure 2.1 demonstrates this idea.

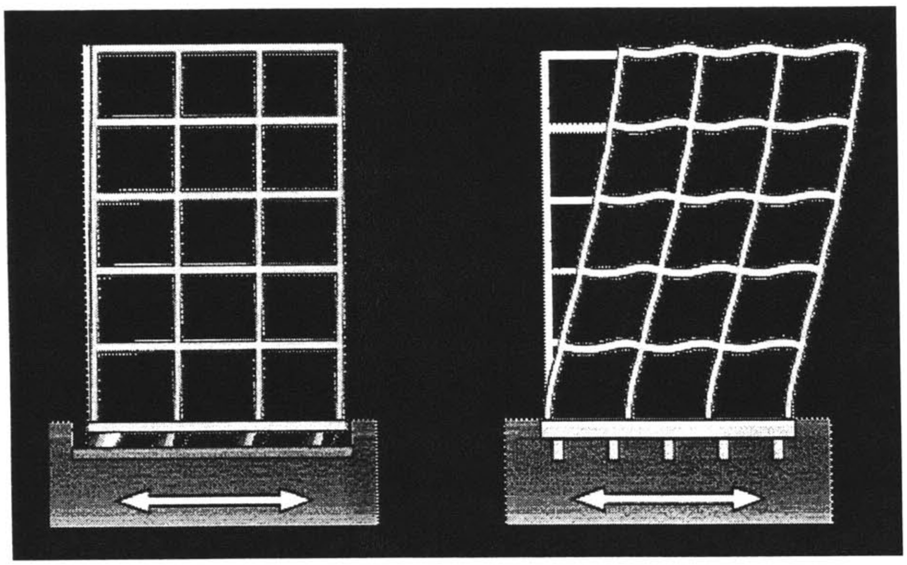


Figure 2.1: Effects of Base Isolation (DIS)

The addition of isolators can reduce the acceleration substantially and many of the stiffening methods need not be used in the superstructure. Without isolators the building itself would have to undergo large-scale movement or sustain increased internal deformation to withstand the large inertial forces. Therefore, by providing isolators at the base of the structure that have high flexibility, it is possible to greatly reduce structural damage.

2.2 *Types of Isolation Devices*

The various types of base isolators are shown in figure 2.2.

High Damping Rubber Bearings

The steel-elastomeric isolator unit consists of steel shims bonded to thin rubber in various layers as specified by the designer. This lay-up process results in large vertical stiffness and low horizontal stiffness

Lead Core Rubber Isolators

Many isolators include lead-cores that deform inelastically during the earthquake induced motions and provide hysteretic damping. The core provides stiffness to withstand wind action. The DIS isolator, in figure 2.3, was chosen as a high performance concept for the cascade building.

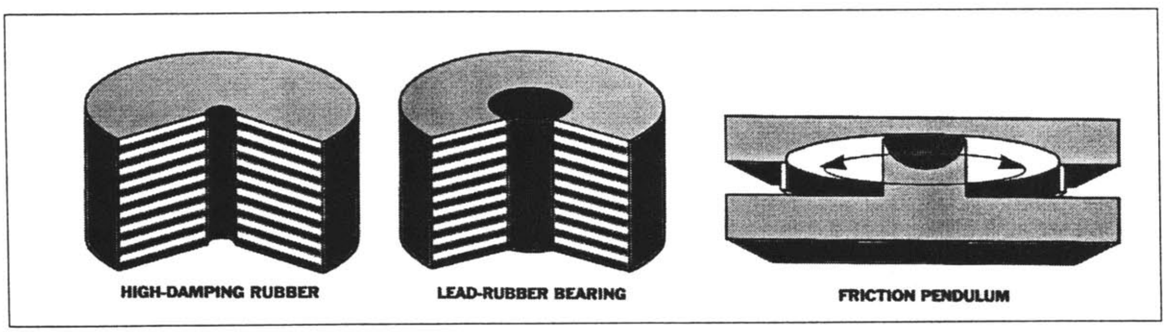


Figure 2.2: Various Types of Isolators (DIS)

Friction Pendulum Bearings

There are a number of different types of base isolation bearings that have now been developed. One type that is currently being used in the San Francisco airport is the friction bearing. Energy dissipation from this system arises from the frictional forces from the weight of the supported superstructure. The sliding surface of the bearing is concave and the structure is forced to rise slightly when it moves horizontally. One can analyze this system as an inverted pendulum to optimize the period of the superstructure. Typically, the sliding surface of the bearing is a low friction bearing material composite. The friction threshold is greater than the forces from wind so this system acts as a fixed based structure until subjected to an earthquake. An advantage of this system is that one does not have to design for stiffness, as opposed to lead-rubber bearings, but designs for friction optimization.

2.3 Dynamic Isolation Systems

DIS, Inc. located in Lafayette, California is the leader in protecting structures from earthquake damage through the application of seismic isolation and energy dissipation technologies. The first DIS installation of a lead-rubber seismic isolator was in the United States in 1984. Figure 2.3 provides an illustration of the most popular type of isolator. DIS has provided design and analysis services on more than fifty percent of the structures isolated in North America.

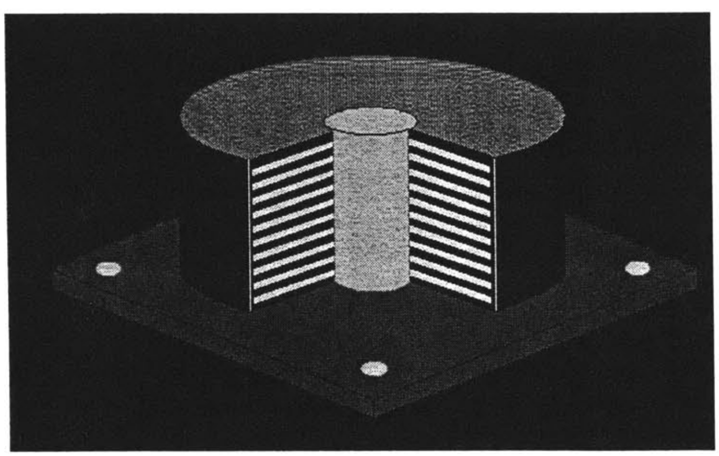


Figure 2.2: Lead Rubber Bearing (DIS)

CHAPTER 3

THE CASCADE BUILDING

3.1 Background

The Civil & Environmental Engineering Department at MIT requested the design of a highly innovative building complex to house its instructional and research needs. The project site lies in the northeast section of MIT's campus. The proposed Civil and Environmental Engineering building will also be located in this area. Directly adjacent to the proposed CEE building will be the new Electrical Engineering and Computer Science (EECS) building, which is currently being designed by renowned architect Frank Gehry. Together these two buildings will form the "new gateway" to the MIT campus as can be seen in figure 3.1.

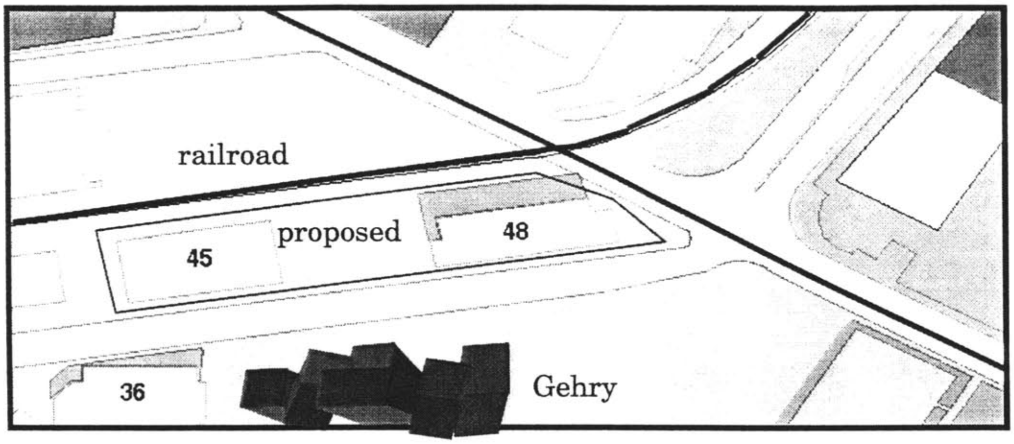


Figure 3.1: The Location of the New Civil Complex

3.2 Architecture

For completeness, the architecture, interior, site and soil properties of the "Cascade" are briefly discussed. By integrating and displaying high levels of civil engineering performance in the building's design, MIT's Civil and Environmental Engineering Department is demonstrating the development of high tech structures for the 21st century. Figure 3.2 depicts the structure

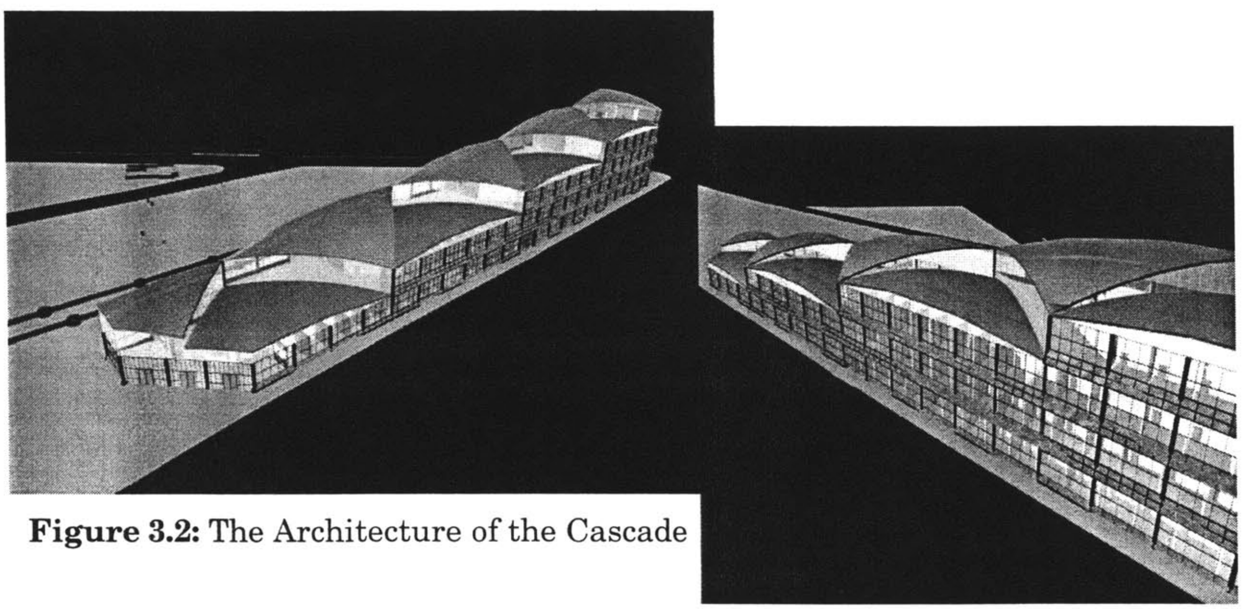


Figure 3.2: The Architecture of the Cascade

on the site. The inside of the structure is represented in figure 3.3. As can be seen, the

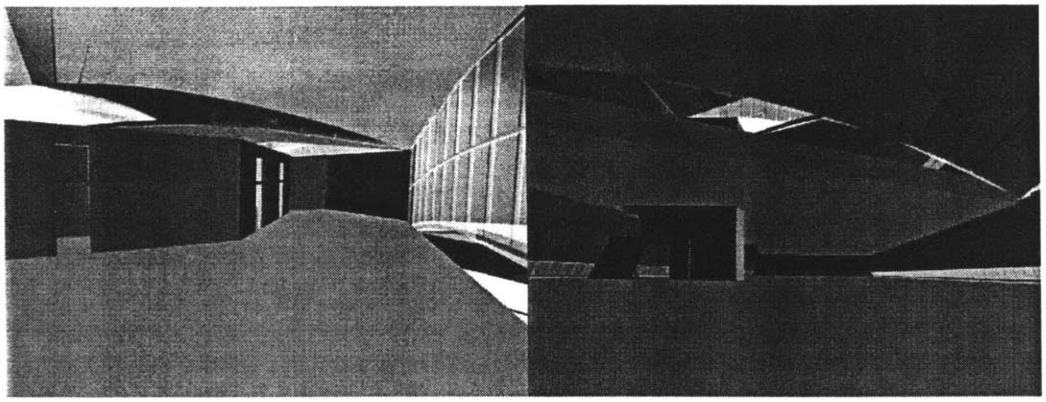


Figure 3.3: Inside the Cascade (DF, JT)

top floor maximizes light by provides two light sources close to the ceiling. The team members, Pushkar V. Deshpande, David Farnsworth, Nick Gianferante, and Joeseph Matthew Tripi and myself used high performance building ideas as primary importance. The use of dampers or base isolation techniques that limit the amount of vibration experienced by motion sensitive research laboratories. Further examples include the use of a space truss roof system as an aesthetically pleasing way to efficiently carry roof loads and a slurry wall construction to create excavation diaphragm-walls.

Figure 3.4: Photovoltaics (DF)

Each of these high-performance systems addresses a specific need, and therefore became a part of the HPS Team's proposed designs. An arrangement of elliptical paraboloids triangular in plan was chosen for the roof system. The corresponding step angle was 45 degrees, still facing generally south. This "cascading shells design" provided for both an aesthetically pleasing and structurally exciting building. In addition, in an effort to increase the energy efficiency of the building, a system of photovoltaic panels was investigated as in figure 3.4.

The final result is a building that is flexible, aesthetically pleasing, structurally exciting, and energy efficient. (DF)

3.3 Static Analysis

A wire frame representation of the 'Cascade' building can be seen in Figure 3.5. It consists of an entirely steel framework with five key features:

- Beams and Columns of 'I' cross-section, pin connected
- No internal columns
- Trusses spanning the 75-ft width of the building
- Bracing systems along the elevator/stair cores

The design of the building took into consideration the LRFD Codes and Massachusetts State Design Codes for wind and snow loads. The actual design analysis was performed using the software program SAP2000 and supplemented, wherever necessary, by hand calculation checks of critical structural systems. Also, expansion joints would need to be placed in the building since it has a length of 600Ft. It may be important to note that all member sizes and relevant force magnitudes may be found in the appendices.

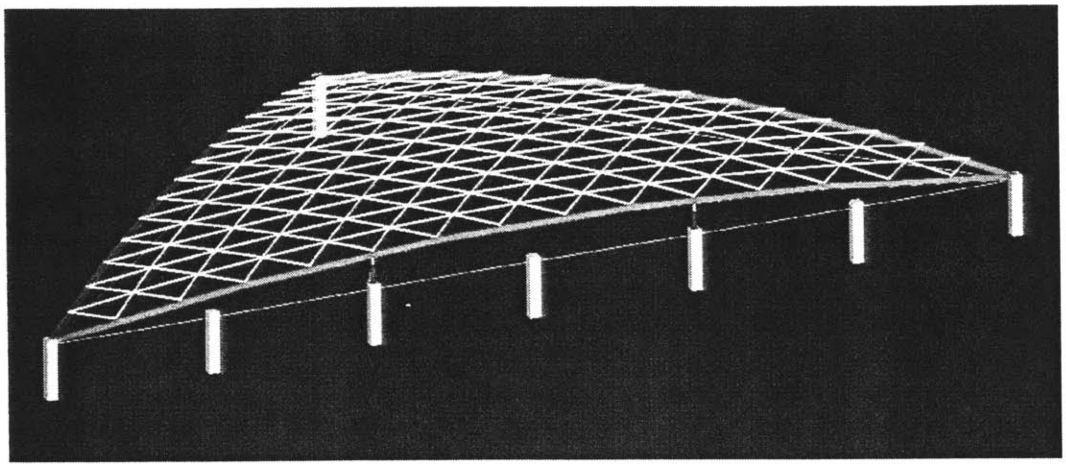


Figure 3.5: Shell Roof Structure (DF)

The structural design and analysis of the roof system, in figure 8, began with the generation of a model of the reticulated shell. The nodal point coordinates were calculated using the above equation. The initial members and properties were then assigned. All data was then imported into SAP 2000 for the analysis under various loading conditions.

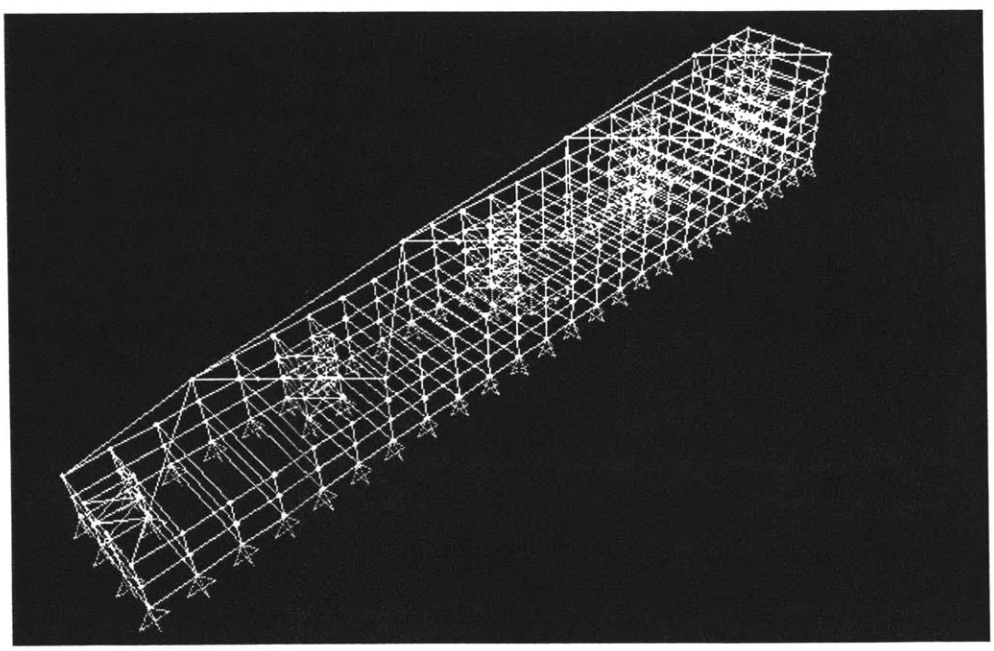


Figure 3.6: Structural Wire Frame

The table 3.1 and 3.2 below represents the critical load combinations designed for and assumptions pertaining to the site.

Table 3.1: Load Cases

Case Number	Load Combination
Case 1	1.4Dead
Case 2	1.2Dead + 1.6Live
Case 3	1.2Dead + 1.3Wind + 0.5Live

Table 3.2: Material Loads

Load	Magnitude (psf)
Concrete	50*
Live	150
Self Weight of Structure	Included as per size

*Concrete Slab 4inch thick

Table 3.3: Live Static Loads

Load	Magnitude (psf)
Wind	21 – Zone 3 Exposure B
Snow	40

The figures 3.7 and 3.8 are two sections taken from the building that display the uniformly distributed loads as well as the point loads coming in from the roof system.

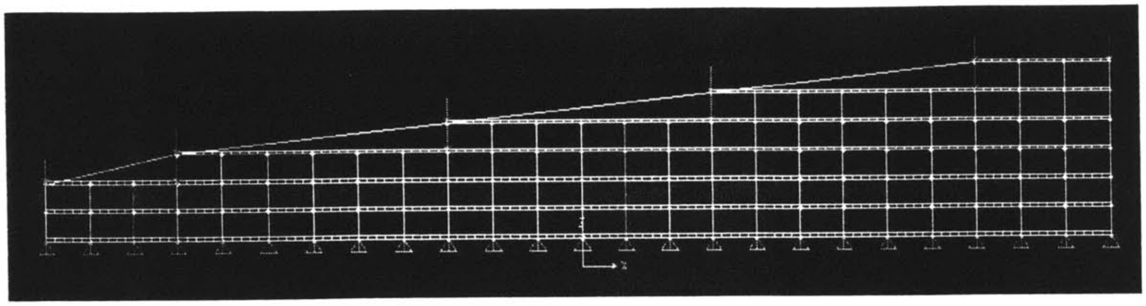


Figure 3.7: Cascade structural section xz

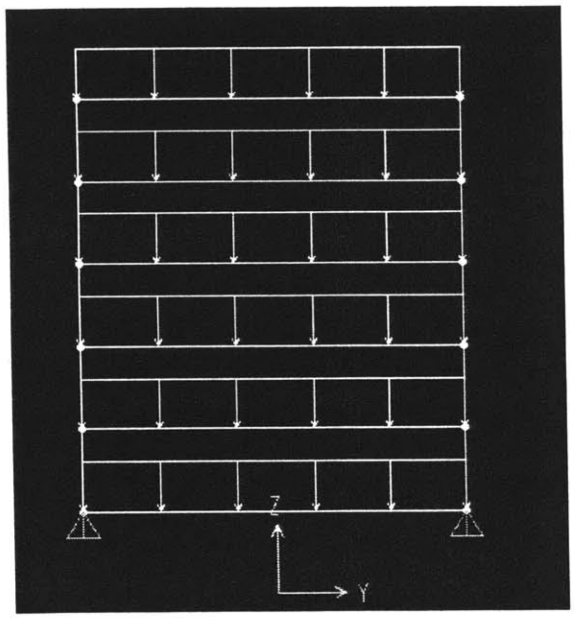


Figure 3.8: Cascade structural section xy

Floor System

Figure 3.9 gives a closer look at the floor system. Briefly, this consists of:

- Trusses spanning the 75ft width of the building along the entire length of the building
- Beams at 5ft connecting these trusses
- A four-inch thick cast in place concrete floor slab

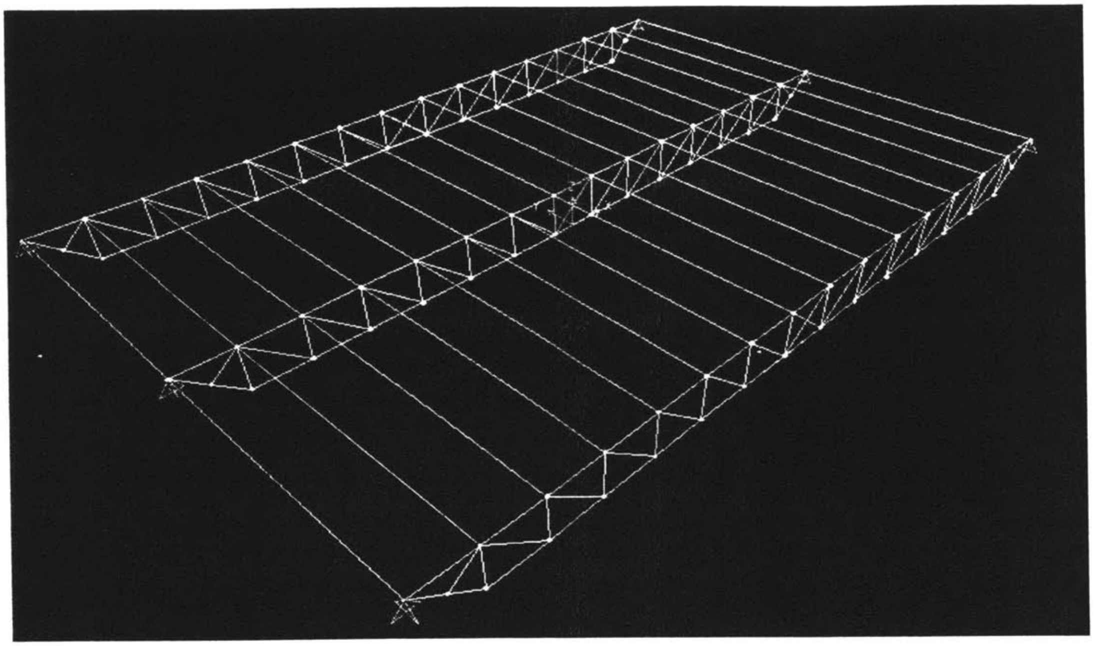


Figure 3.9: SAP 2000 floor model

A single truss was analyzed a shown in Figure 3.10 as one with point forces acting at the points of contact of the crossbeams to the truss.

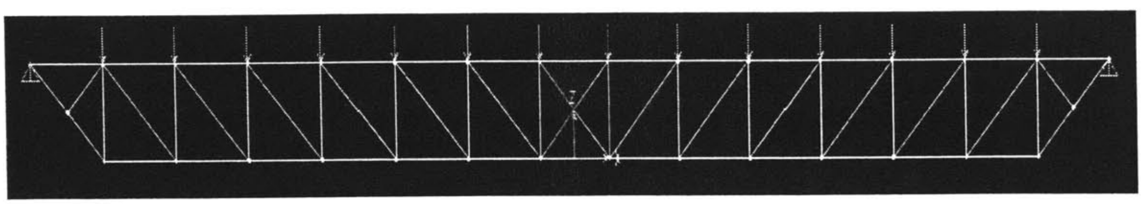


Figure 3.10: SAP 2000 truss model

Lateral Resistance System

The structure consists of five braced cores along the elevator shafts that help resist the lateral loads. In addition to this, we have moment frames on the two shorter sides of the building. The floor diaphragms on every floor are designed to transmit the wind loads into the bracing systems. A graphic representation of the structural mechanism can be seen in the Figure 3.11 and 3.12.

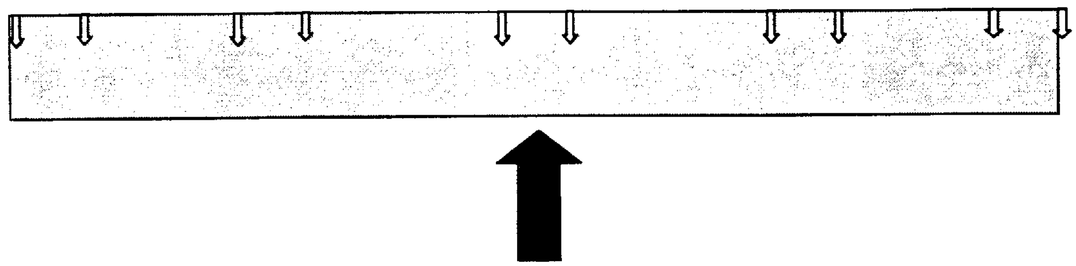


Figure 3.11: Lateral Force Model (PD)

In this case, the wind load comes in perpendicular to the longer face of the building and is resisted by the forces in the lateral braces as may be seen in the figure above. This is known to be a relatively simple model and will cause no design problems.

However, when the wind force ( ) is incident on the shorter face of the building the lateral forces that are generated to resist it ( ) create an imbalance and a resulting moment (figure 3.12). This must be countered and the resistance is created from the bracing ( ) system as shown below. This moment may, in other cases, cause some twisting or torsional effects but in our case posses no design threat since the magnitude of the wind force as well as the moment generated is extremely small. (PD)

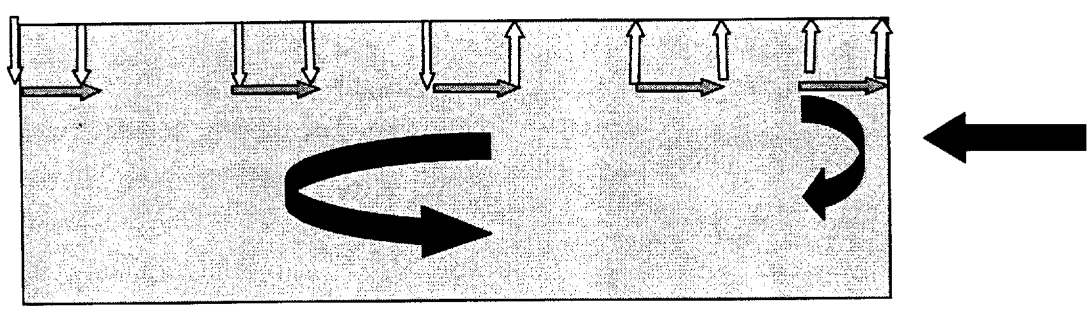


Figure 3.12: Lateral Force Resistance (PD)

3.4 Site and Soil Conditions

Generally, the stiffer the soil, the more effective the isolation. If the soil is soft, the high-frequency content of the earthquake motion is filtered out and results in long-period motions. Thus, with seismic isolation that lengthen the period of a structure, the soft soil condition will amplify rather than reduce the ground motion. The mat profile is shown in figure 3.13.

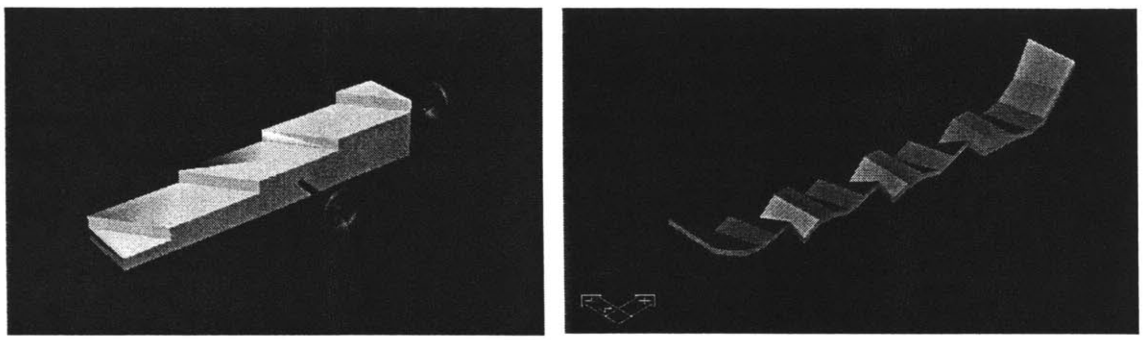


Figure 3.13: Mat Profile (PI, JF)

Although the soil profile is beyond the scope of this project it is interesting to note the settlement of a stepped building. The location where the mass is the greatest will want to heave the structure.

CHAPTER 4

BASE ISOLATION OF THE CASCADE

4.1 Guidelines

Seismic isolation of a building achieves a reduction in earthquake forces by increasing the length of the period of vibration at which the structure responds to earthquake motions. The most desirable characteristics for isolation are a building subjected to zone 4 earthquakes and a building that has two to twenty stories. The Cascade building satisfies both of these requirements. Also, the motion sensitive equipment of the Civil and Environmental Engineering department will be better protected.

The engineer and architect must communicate with the owner on the level of seismic performance necessary for the structure. Table 4.1 lists levels of earthquake performance where, * is the basic objective, ** is the essential/hazardous objective, and *** is the safety critical objective. The fully operational – very rare cell, ****, is not an economically viable solution.

Table 4.1: Seismic Performance Level (Asher, 1998)

Return Period	Fully Operational	Operational	Life-Safe	Near Collapse
Frequent (43 year)	*			
Occasional (72 year)	**	*		
Rare (470 year)	***	**	*	
Very Rare (970 year)	****	***	**	*

4.2 Flexibility

One of the main concerns of an isolated building is the need for special details to accommodate movement within the structure. Gas, water, and electricity pass through the substructure and the isolated superstructure must be designed for movement as is shown in figure 4.1. This design allows the utilities the flexibility to move to bridge the moat gap if an earthquake hits the structure.

Moats must also be designed to accommodate for deflections at entrances, exits, stairs, and lift wells. There are moat covers to occupy the surface above the isolation plane. This provides movement over the adjacent sidewalk so as to prevent the structure to be locked against the pavement in an earthquake.

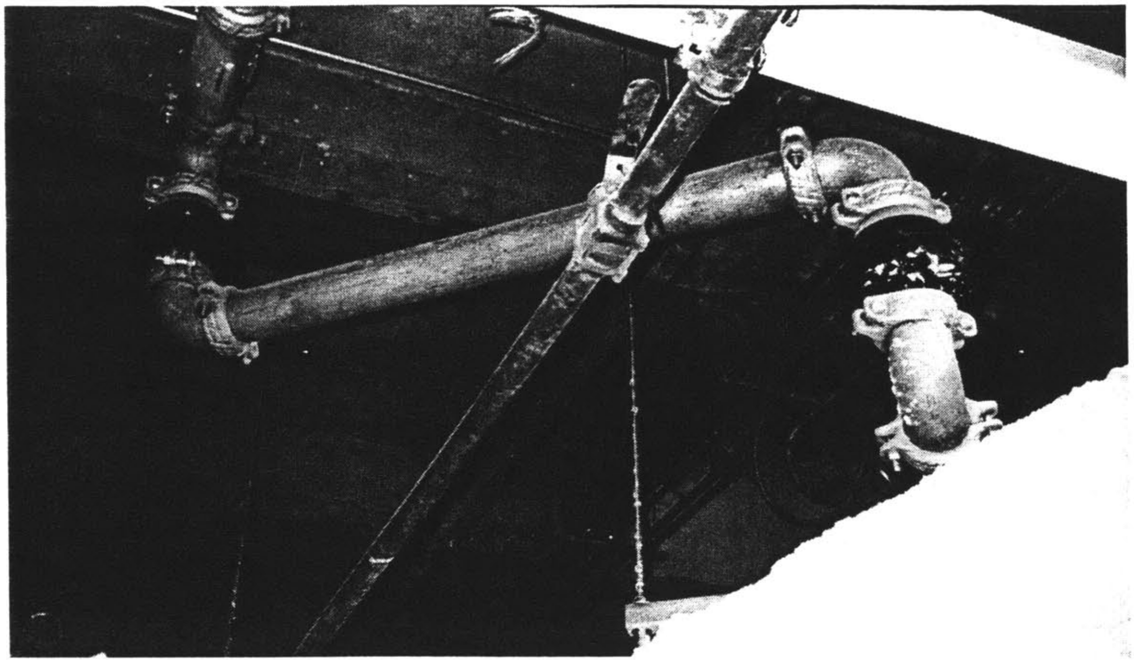


Figure 4.1: Utility Flexibility

Figures 4.2 and 4.3 depict the isolator and the isolation plane between the superstructure and the substructure.

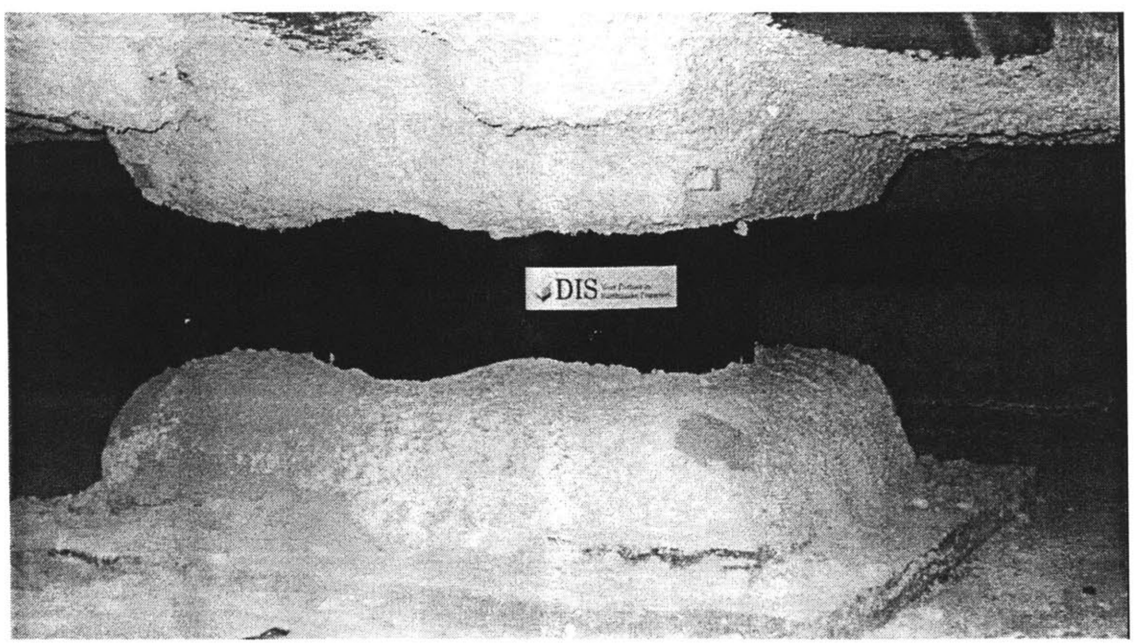


Figure 4.2: Installed Isolator

The movable joint also provides a safety backup system. In the event of a fire, where the isolators are severely deteriorated the building could settle on top of the substructure. Later the structure can be jacked and the isolators can then be replaced.

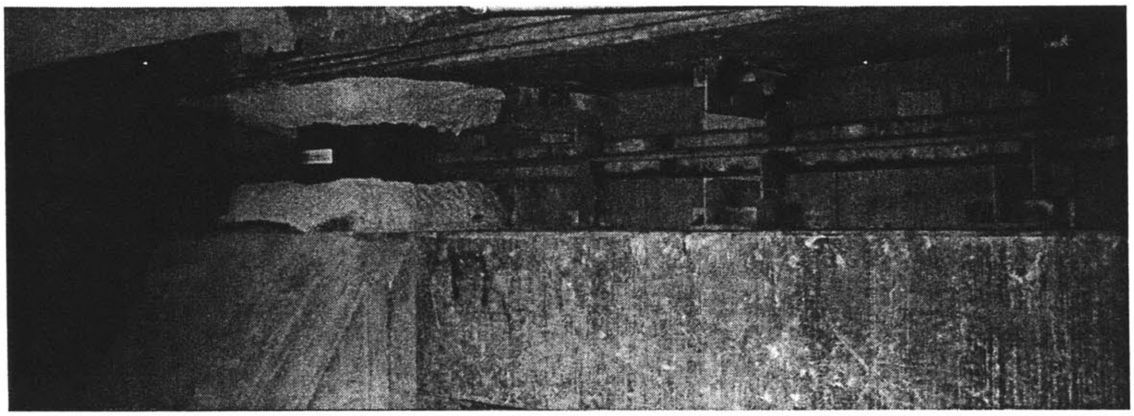


Figure 4.3: Movable Joint between Substructure and Superstructure

4.3 Bearing locations

The various locations of the isolation plane are depicted in figure 4.4. The isolators can be placed at the foundation level, basement level, ground level, or the top, bottom, or mid-height of columns. Location of the appropriate level for the plane of isolation is of primary importance. The foundation level and above the basement were considered. Eventually, the case where the isolators are above the basement was analyzed in depth. The major difference between the locations is the elevator detail. If the isolators were placed above the basement level the elevator must be able to move with the superstructure. To accommodate this, an elevator detail was completed and is shown in figure 4.5. This detail allows for the elevator to be isolated itself. The system is then attached to the superstructure and clip angles are placed to allow movement below the first floor.

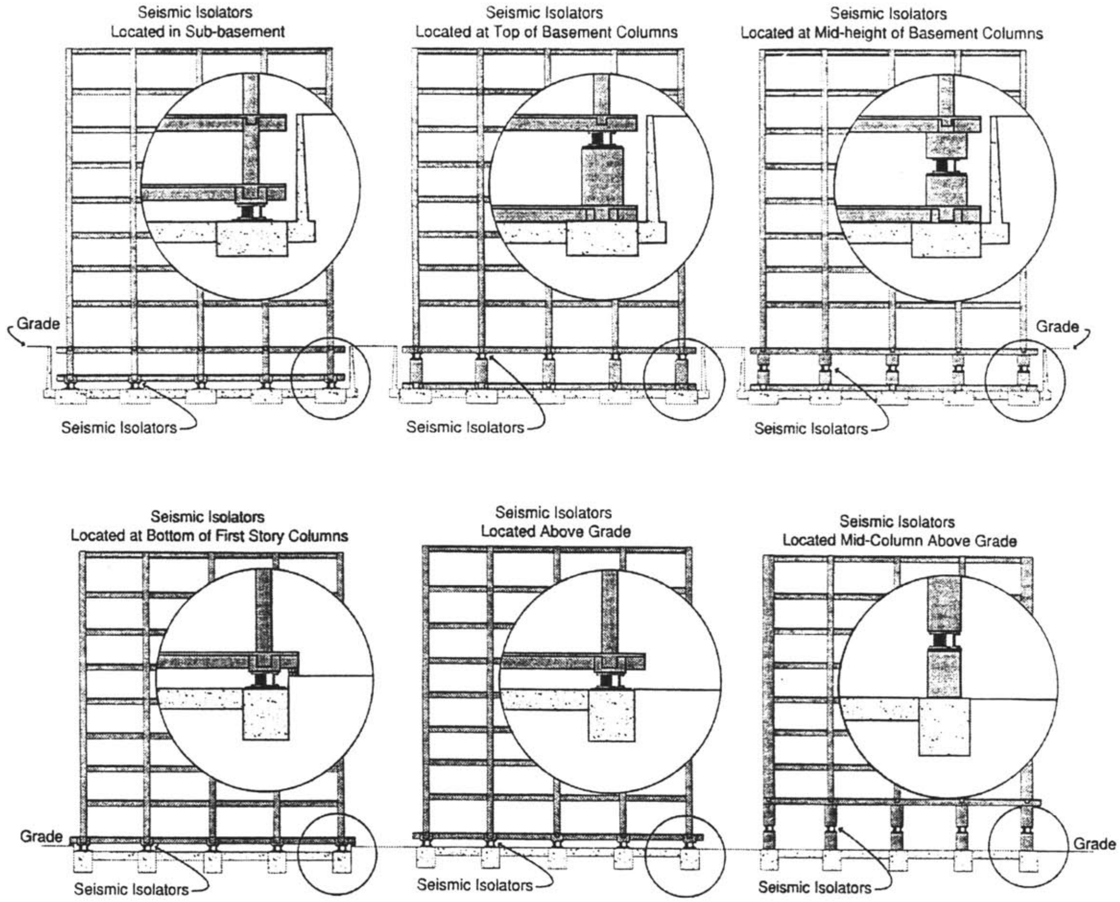


Figure 4.4: Six Locations for Isolators (DIS)

4.4 Details

As described earlier, the elevator detail is critical and can tolerate no separation or discontinuities in their geometry throughout the height. If the locations of the isolators are placed above the foundation the elevator will have to be free to move with the superstructure. This promotes the need for individual isolators under each elevator.

The assembly of the DIS isolator is provided in figure 4.6.

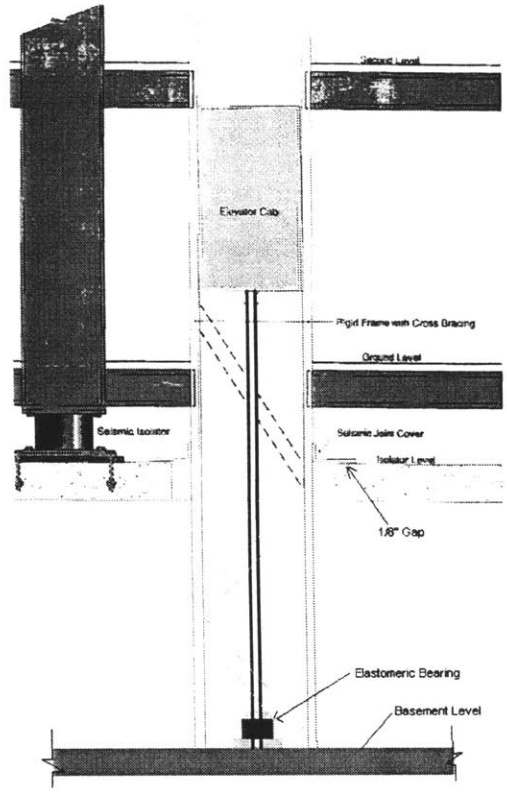


Figure 4.5: Elevator Detail

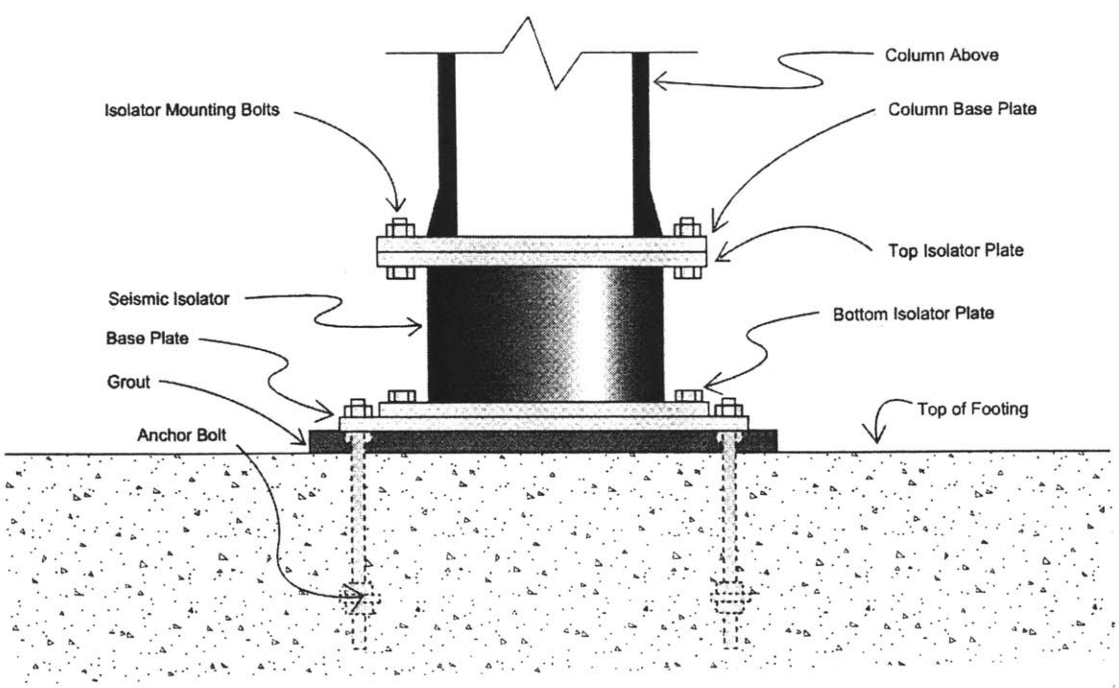


Figure 4.6: Isolator Assembly Detail (DIS)

4.5 Uplift and Overturn

For a structure that is under 10 stories, overturn is not a major concern. There may be tensile forces in the isolators during earthquake loading and these forces need to be analyzed. The keeper plates prevent uplift by large tensile loads.

4.6 Bearing Removal and Backup Safety System

The structure can be lifted by the use of hydraulic jacking devices that are easily available. Every isolated structure should have test bearings that are provided with sensors to test and monitor the behavior. These bearings are not needed for serviceability requirements but for determining adequacy of the isolator after a seismic event.

A backup system can be developed to prevent the structure from deflecting beyond the limits. Keeper angles can be placed near the isolator to prevent the superstructure from displacing as much as the width of the moat. The angles can be placed one inch less then the width of the moat to prevent increased substructure damage.

CHAPTER 5

COMPUTER MODELING AND RESULTS

5.1 Isolation Analysis and Theory

Deformable Isolators are capable of moving the fundamental vibration period to high values so as to obtain a significant reduction of the spectral acceleration and of the expected input energy. (Rossi, 1996) Figure 5.1 illustrates a SDOF beam type idealization of an isolated structure. From this, the equivalent SDOF properties of the structure can be estimated by assuming that the structural response is dominated by the fundamental mode. This assumption is reasonable for low-rise buildings subjected to seismic excitation. The governing equations for the lumped mass model, figure 23c, consist of an equilibrium equation for the mass, and an equation relating the shear forces in the spring and the bearing. (Connor, 1998)

$$K \cdot u + C \cdot \dot{u} + M \cdot \ddot{u} = -m(\ddot{u}_b + \ddot{u}_g)$$

and

$$K_b \cdot u_b + C_b \cdot \dot{u}_b = K \cdot u + C \cdot \dot{u}$$

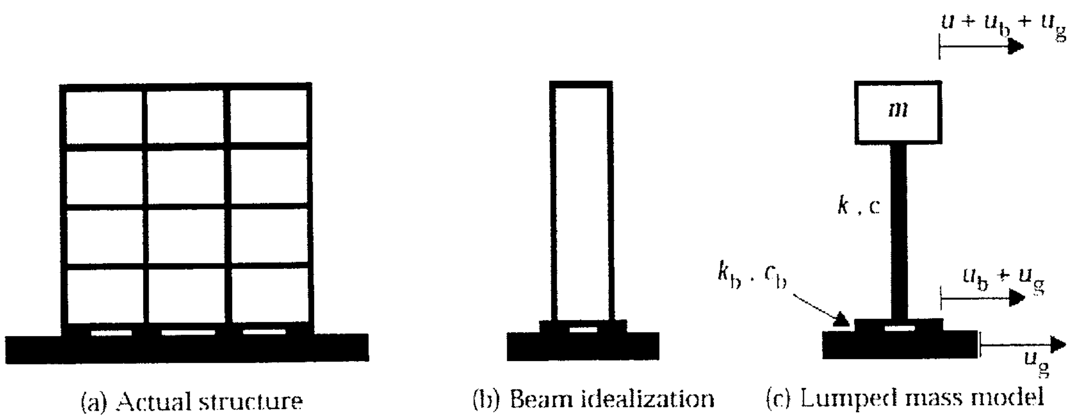


Figure 5.1: Base Isolation Models (Connor, 1998)

5.2 SAP2000

The structural analysis tool SAP2000 was used to analyze the structure under time history loading. This powerful tool has a nonlinear analysis module ideal for understanding the behavior of isolators, or nonlinear elements. The method of nonlinear time-history analysis used in SAP2000 is an extension of the Fast Nonlinear Analysis (FNA) method developed by Wilson (Ibrahimbegovic and Wilson, 1989; Wilson, 1993). The method is extremely efficient and is designed to be used for structural systems which are primarily linear elastic, but which have a limited number of predefined nonlinear elements, referred to as Nlink. In SAP2000, all nonlinearity is restricted to the Nllink elements.

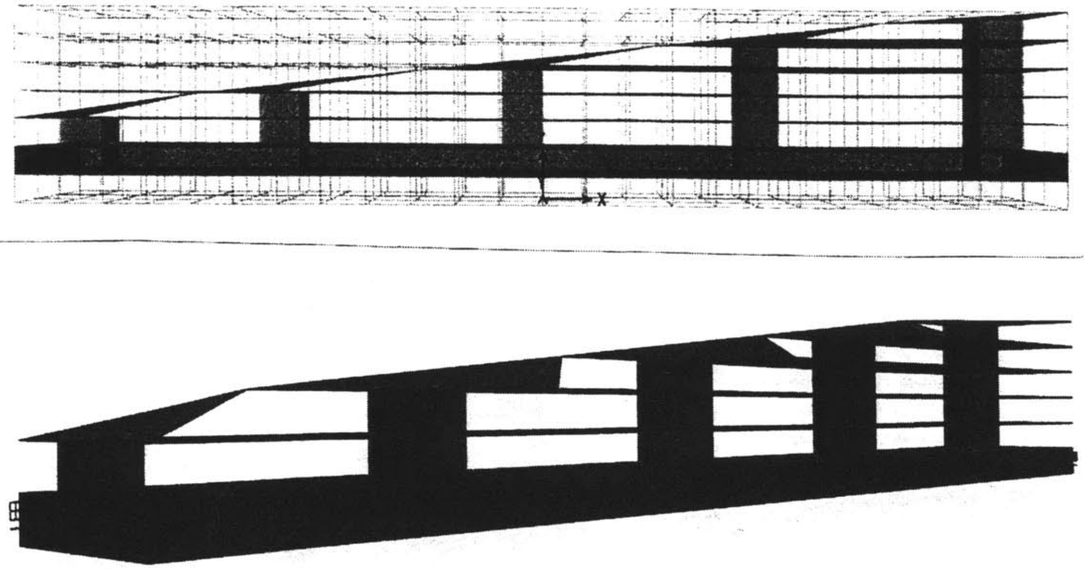


Figure 5.2: SAP 3-D Model

The dynamic equilibrium equations of a linear elastic structure with the isolators as nonlinear elements subjected to an arbitrary load can be written as:

$$K_l \cdot u + C \cdot \dot{u} + M \cdot \ddot{u} + r_n = r(t)$$

where, K is the stiffness matrix for the linear elastic elements (all elements except the Nllinks); C is the proportional damping matrix; M is the diagonal mass matrix; r is the vector of forces from the nonlinear degrees of freedom in the Nllink elements; u is the relative displacement with respect to the ground; and r(t) is the vector of applied loads.

A linear effective stiffness is defined for each degree of freedom of the nonlinear elements. The equilibrium equation can then be rewritten as:

$$K_l \cdot u(t) + C \cdot \dot{u}(t) + M \cdot \ddot{u}(t) = r(t) - [r_N(t) - K_N \cdot u(t)]$$

where, K equals the linear stiffness, K_l, plus the linear effective-stiffness matrix for all of the nonlinear degrees of freedom, K_N. From this, modal analysis is performed using the full stiffness matrix, K, and the mass matrix, M using the Ritz-vector method. The effective stiffness at nonlinear degrees of freedom varies between zero and the maximum nonlinear stiffness of that degree of freedom. The Ritz-vector method should be used to determine the Modes, unless all possible structural Modes are found using eigenvector analysis.

The assumption of proportional modal damping is being made with respect to the total stiffness matrix, K, which includes the effective stiffness from the nonlinear elements. The effective stiffness should be selected such that the modes for which these damping values are specified are realistic. (SAP2000)

Time History

The time history of Loma Prieta is shown in figure 5.3. The characteristics of this earthquake were recorded and are supplied in the earthquake-input module in SAP2000. It is important to note that earthquakes used for time history analysis should be representative of the site for accurate results. Assuming the new Civil and Environmental Complex was located near a fault in zone 4, Loma Prieta will be used. Again, time history analysis is more accurate then response spectrum analysis because of the non-periodic nature of earthquakes.

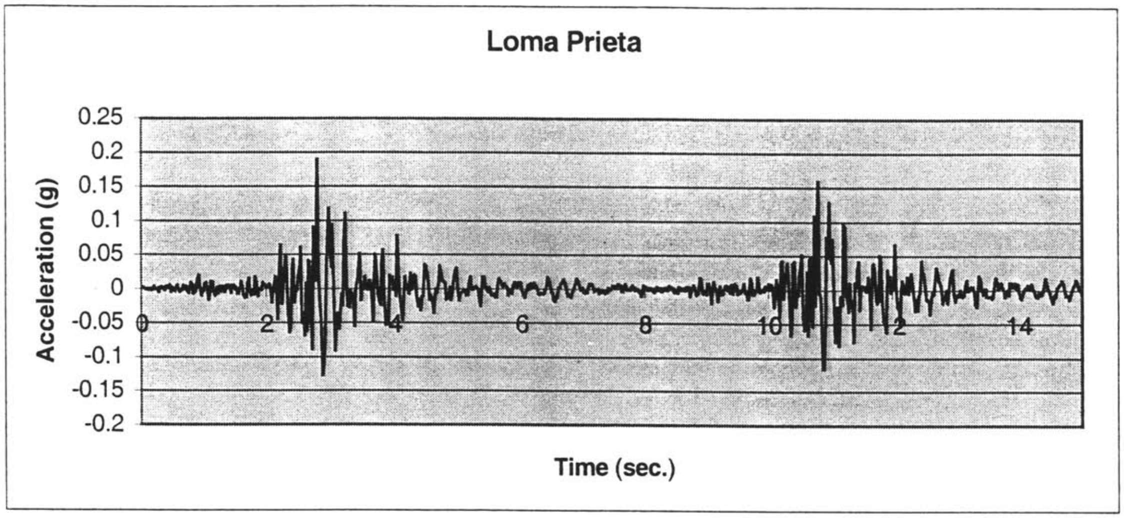


Figure 5.3: Loma Prieta Time History

The time history of Loma Prieta is seen as impulsive around the 3-second mark and ground accelerations are slow at the beginning. This is much different then the time history of El Centro which is quite impulsive early. It is very important to note that one should always use earthquake loading that is representative of the site. For purposes of this project available time-histories were used (Loma Prieta, El Centro, and Northridge). The analysis had 933 degrees of freedom and 30 modes were solved. Again, Ritz analysis was used because it takes into account the spatial distribution of the loading.

The two figures, 5.4 and 5.5, are stick models generated in SAP and demonstrate the advantage of base isolation. Most of the deformation happens within the isolator itself as these two modes depict. This was generated by the time history analysis of the Loma Prieta earthquake. Note that all of the earthquakes result in the same deflected shape but the magnitudes of the forces will differ.

Modes

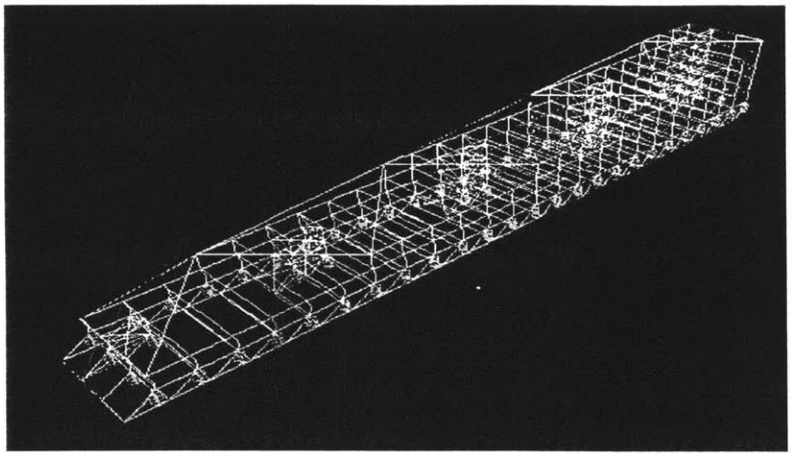


Figure 5.4: Building Shifted at 90 degrees

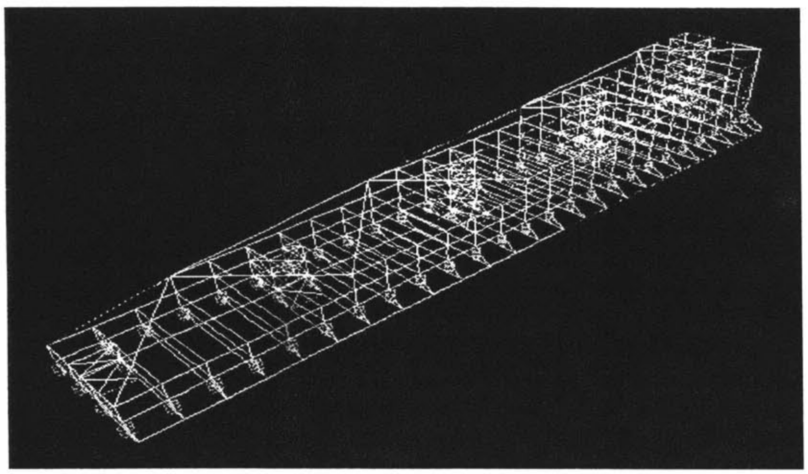


Figure 5.5: Building Shifted at 0 degrees

The second mode, in figure 5.6, presents a good model of how an isolated building will behave. The isolators are above the basement and take over ninety percent of the deformation.

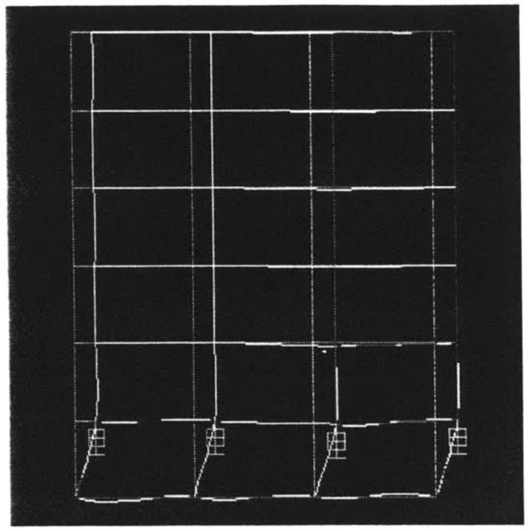


Figure 5.6: Mode 2

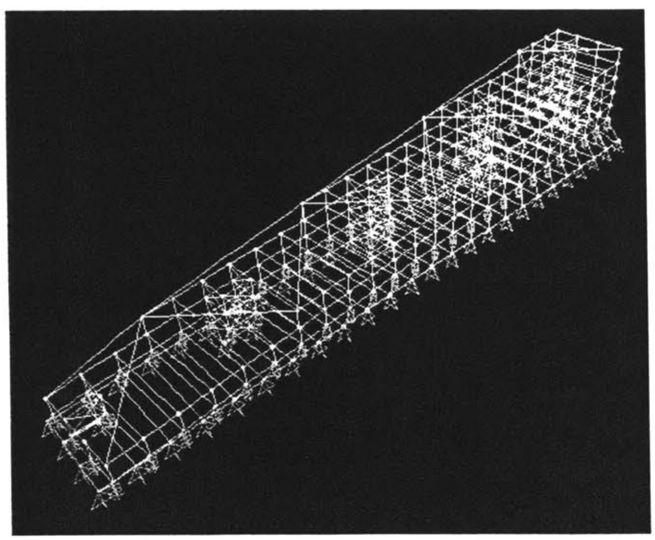


Figure 5.7: Mode 5

The fifth mode above is the twisting mode and is important because it represents the torsional response of the building.

5.3 Bearing Model and Analysis

The isolator, or Nllink element, is used to model local structural nonlinearities. Nonlinear behavior is exhibited only during non-linear time-history analyses and was completed using the Loma Prieta earthquake. Each Nlink element is assumed to be composed of six separate "springs," one for each of six deformational degrees-of-freedom (axial, shear, torsion, and pure bending). Each of these springs possesses a dual set of properties:

- Linear effective-stiffness and effective-damping properties used for all linear analyses.
- An optional nonlinear force-deformation relationship used only for nonlinear time-history analyses.

The linear effective damping property is only used for response-spectrum analyses and linear time-history analyses. The nonlinear force-deformation relationships of these springs may be coupled or uncoupled, depending on the type of behavior modeled. (SAP2000)

Six independent internal deformations are defined for the Nllink element. For two-joint link elements the internal deformations are defined as:

- Axial: du1 = u1j – u1i
- Shear in the 1-2 plane: du2 = u2j – u2i – dj2 r3j – (L – dj2) r3i
- Shear in the 1-3 plane: du3 = u3j – u3i + dj3 r2j + (L – dj3) r2i
- Torsion: dr1 = r1j – r1i
- Pure bending in the 1-3 plane: dr2 = r2i – r2j
- Pure bending in the 1-2 plane: dr3 = r3j – r3i

where:

u1i, u2i, u3i, r1i, r2i, and r3i are the translations and rotations at joint i,

u1j, u2j, u3j, r1j, r2j, and r3j are the translations and rotations at joint j,

dj2 is the distance you specify from joint j to the location where the shear deformation du2 is measured (the default is zero).

dj3 is the distance from joint j to the location where the shear deformation and du3 is measured (the default is zero) and
L is the length of the element.

All translations, rotations, and deformations are expressed in terms of the element local coordinate system.

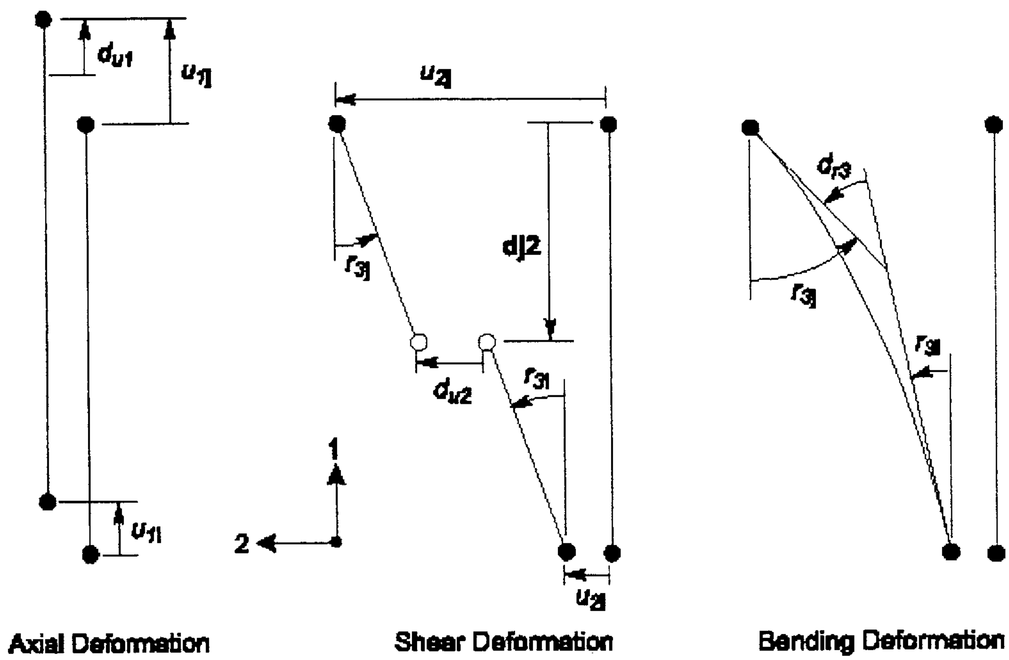


Figure 5.8: Axial, Shear, and Bending Deformation (SAP)

Each Nllink element has its own element local coordinate system used to define force-deformation properties and output. The axes of this local system are denoted 1, 2 and 3. The first axis is directed along the length of the element and corresponds to extensional deformation. The remaining two axes lie in the plane perpendicular to the element and correspond to shear deformation. Both systems are right-handed coordinate systems. It is up to you to define local systems, which simplify data input and interpretation of results. (SAP2000)

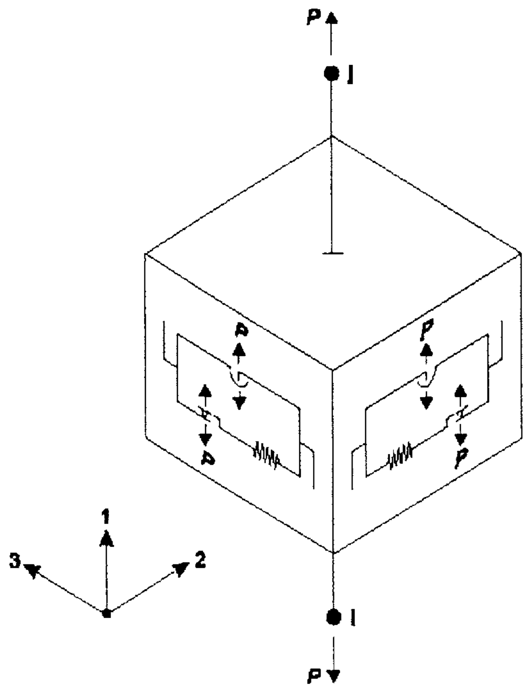


Figure 5.9: Isolator Idealization and Coordinate Geometry (SAP)

The spring force relationships are listed that govern the behavior of the element, one for each of the internal springs:

- Axial: fu1 vs. du1
- Shear: fu2 vs. du2 , fu3 vs. du3
- Torsional: fr1 vs. dr1
- Pure bending: fr2 vs. dr2 , fr3 vs. dr3

where f u1 , f u2 , and f u3 are the internal-spring forces; and f r1 , f r2 , and f r3 are the internal-spring moments.

5.4 DIS Program ISOLATE

ISOLATE facilitates the design of seismically isolated buildings and other structures that fall within the scope of the Uniform Building Code (UBC). It automates preliminary design of DIS Lead-Rubber seismic isolators.

Complete requirements for the design and analysis of a seismically isolated building are contained in the 1994 Uniform Building Code, Appendix Chapter 16, Division III - Earthquake Regulations for Seismic-Isolated Structures.

Project Input

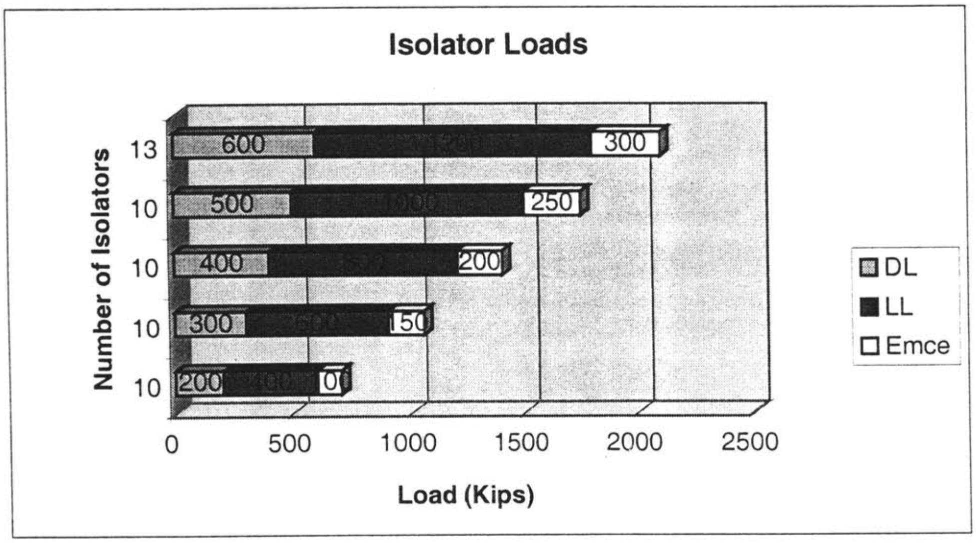


Figure 5.10: Isolator Loads

Design acceleration level and soil profile types are identified and the earthquake loads are specified according to the 1994 Edition of the UBC. The seismic load level is specified by two factors: a Design Acceleration Level, which is the product of the seismic zone factor, Z, and the near-field coefficient, N, and a Soil Profile Type (1, 2 or 3). The UBC requires that the design displacements used for evaluation of the isolation system include the effects of torsion. The default for this ratio is conservatively assumed to be 1.30. For a well designed lead-rubber system, the actual value is often less than 1.20. (DIS ISOLATE)

Secondly, the loads need to be determined from the SAP2000 output. All of the columns of the cascade building had a different load because of the stepped geometry of the structure. There were five different vertical loads for the input and ISOLATE designed three different sizes of bearings. In addition, the lead core option was selected and the output included both isolators with and without lead cores.

Project Output

Three different sizes of bearings as well as two types were given. All of the isolators have the same height for ease of construction. The second type of twenty isolators did not have the lead core because they were not needed. Since there were no interior isolators, besides under the elevators, the lead and no-lead bearings were dispersed evenly throughout the perimeter of the structure.

Table 5.1: Types of Isolators

Type	1	2	3
Number of Isolators:	20	20	13

The design conditions are the maximum for any isolator load condition for each type. The final design condition listed is the total maximum displacement, Dtm, calculated from the response to the specified seismic loading and the specified displacement ratios (Dt/D and Dtm/Dt).

Table 5.2: Loads

Maximum DL + LL	900	1500	1800
Maximum 1.2DL + LL + Emce:	1110	1850	2220
Total Maximum Disp., Dtm :	6.93	6.93	6.93

The isolator dimensions below list the plan shape, plan dimension, the total height including mounting plates, and the lead core diameter. The estimated weight of the isolator is also listed.

Table 5.3: Isolator Properties

Plan Shape	Circular #1	Circular #2	Circular #3
Plan Dimension	25.50	31.50	33.50
Height	9.375	9.375	9.375
Lead Core Diameter	4.50	0.00	6.00
Weight (lbs)	934	1345	1573

The vertical response of the isolators is assumed to be linear elastic. Hence a single stiffness value is reported. The effective stiffness (Keff) is defined as the lateral force in the isolator divided by the corresponding lateral displacement. Due to the bilinear behavior, this stiffness is a function of the displacement. The effective stiffness reported in the output corresponds to the design displacement (D). The Initial Stiffness (Ku), the Yield Force (Fy) and the Post-yield Stiffness Ratio (Kd/Ku) are for use in a nonlinear time-history analysis (SAP2000). For isolators without lead cores, linear elastic behavior is assumed. (ISOLATE)

Table 5.4: Stiffness Properties

Isolator	#1	#2	#3
Vertical Stiffness	6593	14515	17272
Horizontal Effective Stiffness	13.19	11.88	23.04
Initial Stiffness	67.91	11.88	118.11
Yield Force	19.36	0.00	34.38
Post-yield Stiff. Ratio	0.111	1.000	0.110

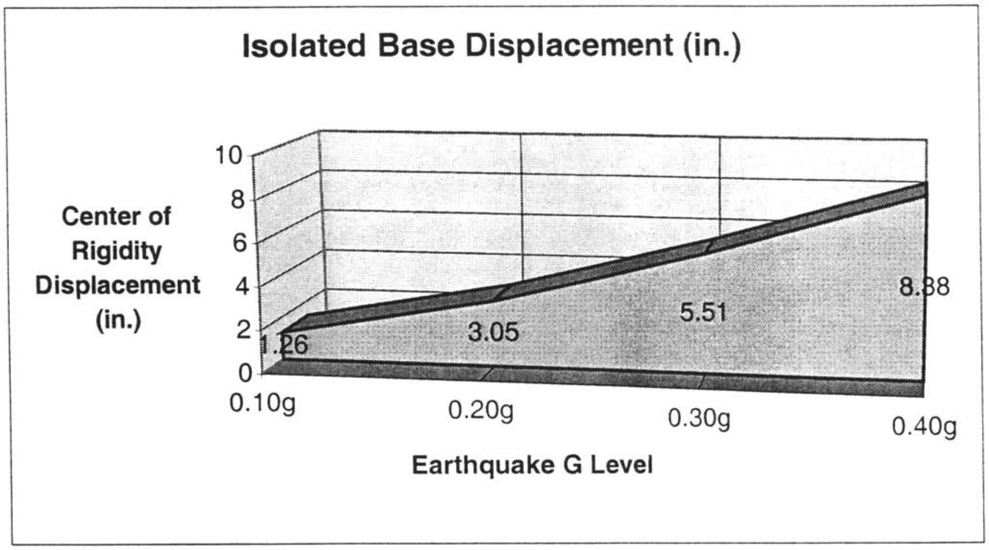


Figure 5.11: Isolator Base Displacement

The seismic performance of the isolated structure has been calculated assuming a rigid substructure below and a rigid superstructure above the isolators. The mote will have to accommodate 8.38 inches of movement and will be design for 12 inches around the perimeter of the structure. A moat detail from DIS is shown in figure 5.12.

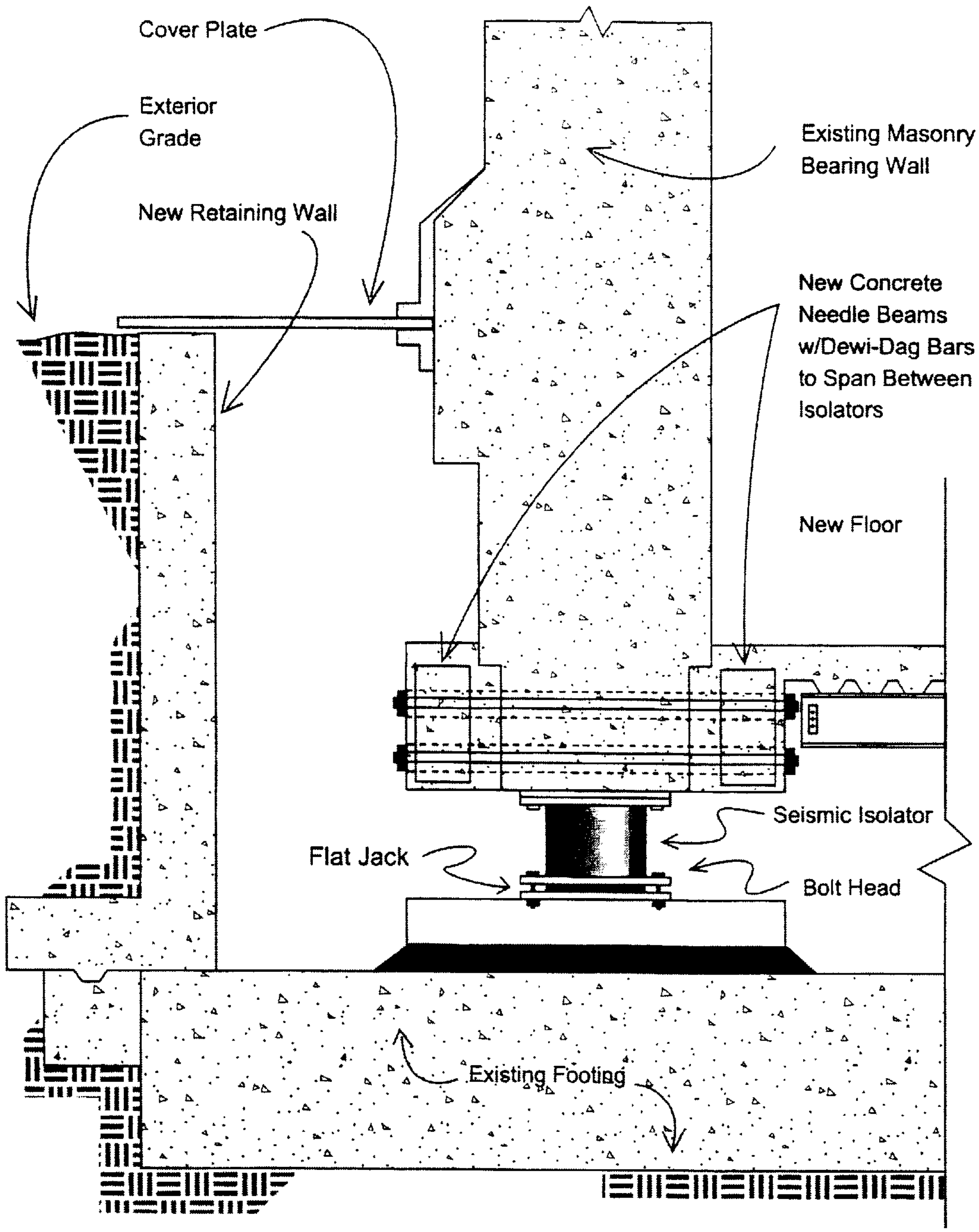


Figure 5.12: Moat Detail (DIS)

Superstructure and/or substructure flexibility will result in an isolated period longer than that estimated from these procedures. The values of isolated period, center of rigidity displacement and elastic base shear coefficient are based on the nonlinear response spectra developed for ISOLATE.

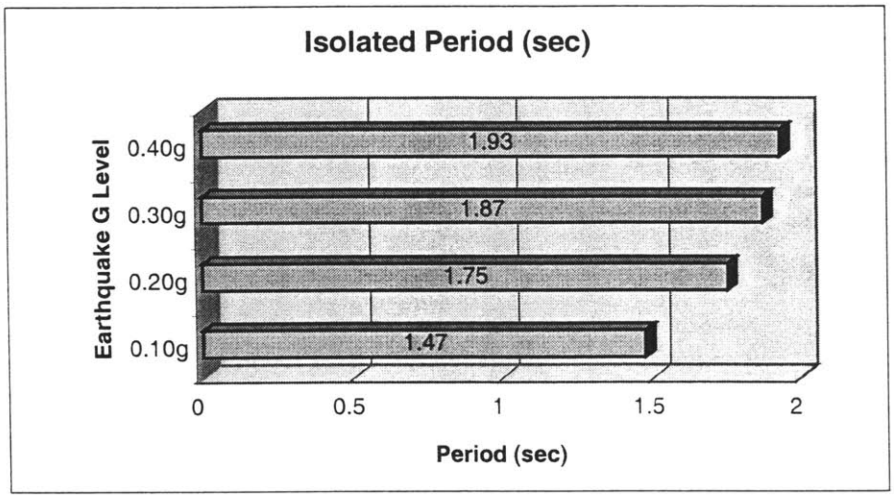


Figure 5.13: Isolated Period

5.5 Comparison of Fixed vs. Isolated

The isolated period, the center-of-rigidity displacement, and the elastic base shear coefficient characterize the seismic performance of the isolated structure. The periods of the structure increased from a fixed base period of 0.3 seconds to an average of 1.75 seconds, which dramatically reduces the floor accelerations. The two similar graphs indicate the differences in displacements between top and bottom floors for both the isolated and non-isolated cases.

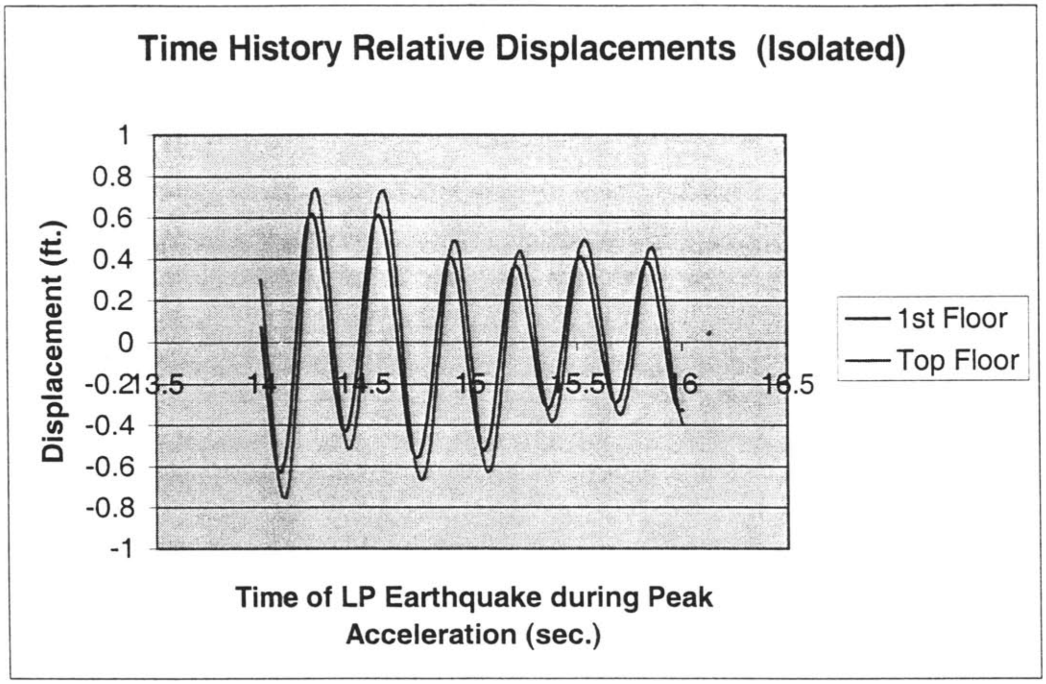


Figure 5.14: Time History Relative Displacements (Isolated)

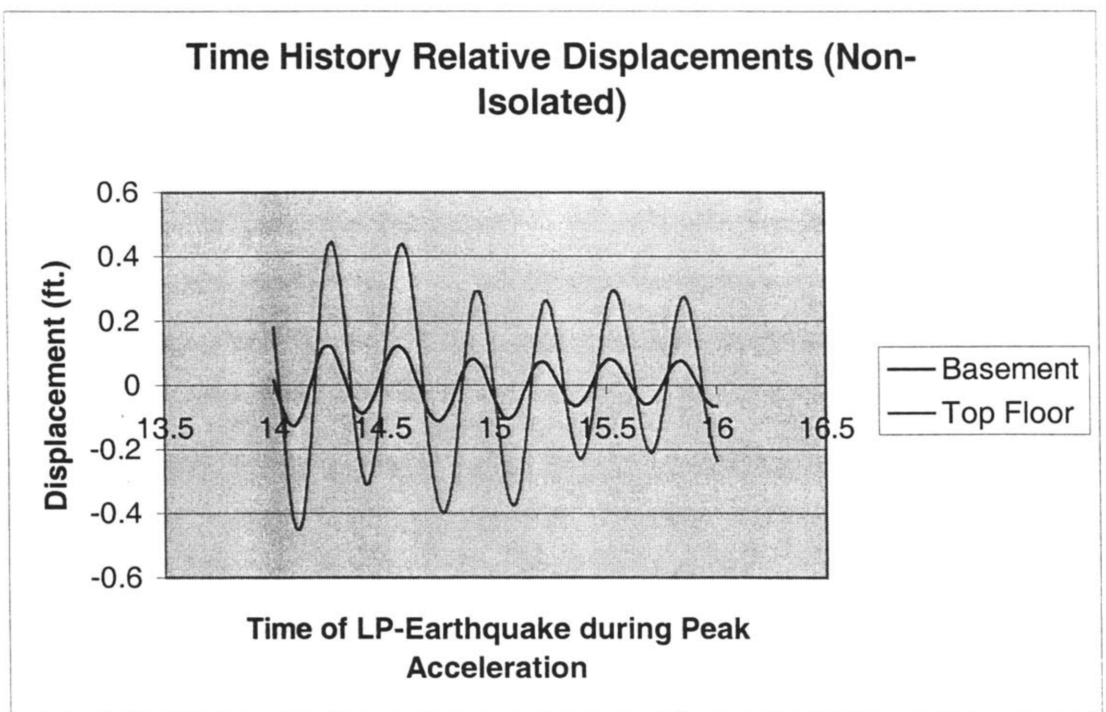


Figure 5.15: Time History Relative Displacements (Non-Isolated)

The non-isolated case has a greater difference in the displacements between the two floors. Although this case has a smaller absolute displacement, the superstructure will have to be strong and stiff enough to handle the inter-story displacements. Comparing the two, the base isolated case depicts the higher performance.

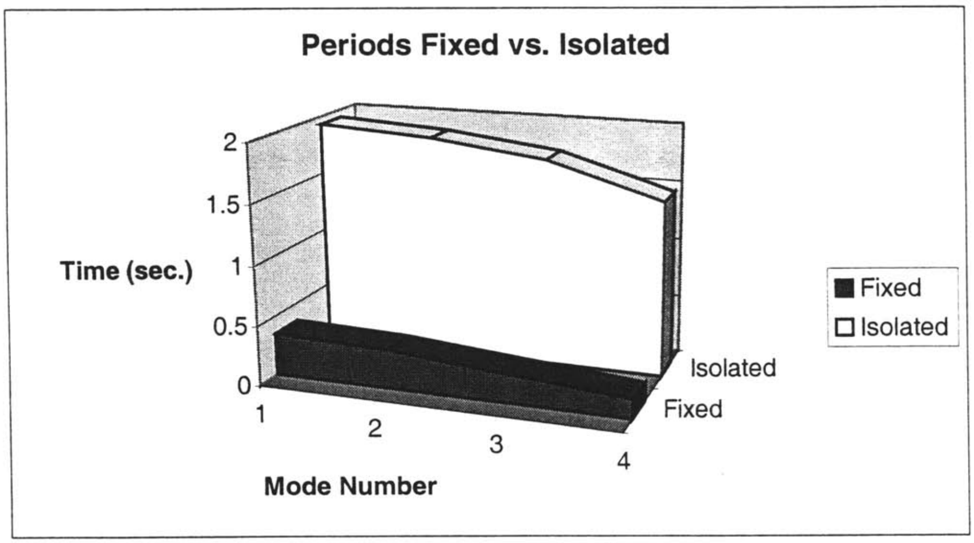


Figure 5.16: Comparison of Periods

Figure 5.16 indicates the difference between the periods between fixed and isolated case. As the period increases the inter-story accelerations decrease and the building is better behaved.

CHAPTER 6

CONCLUSION

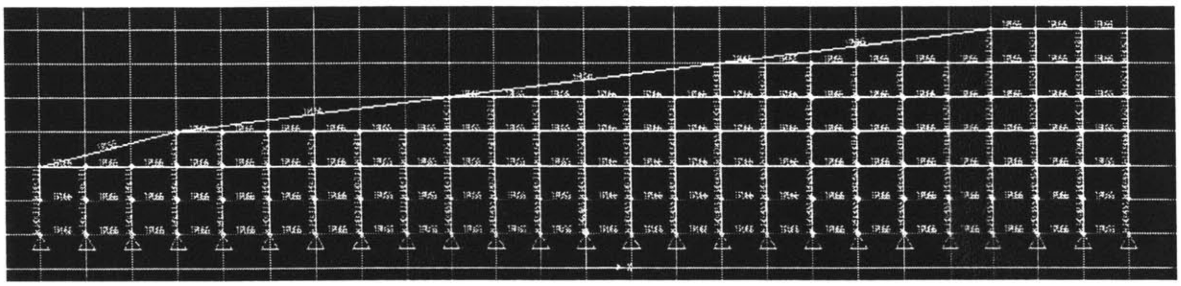
6.1 Conclusion

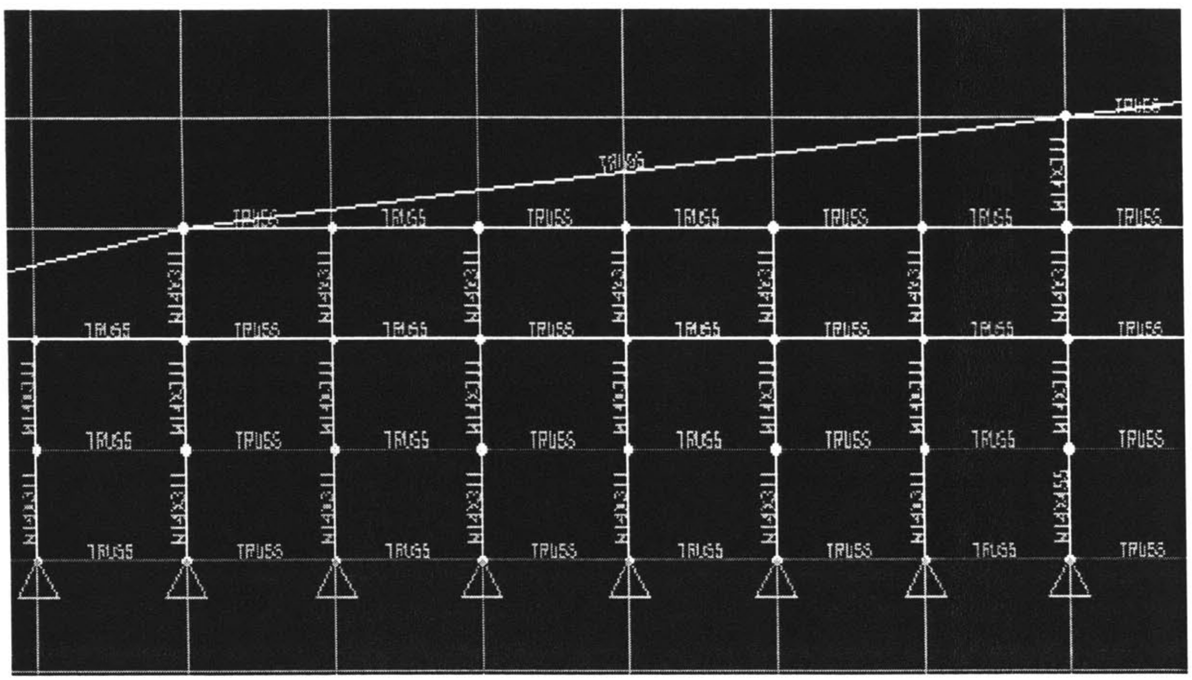
Nonlinear dynamic analysis has proven the benefits of base isolation. The mechanical properties of lead-core rubber bearings, such as those provided by Dynamic Isolation Systems, Inc. were designed and analyzed to maximize the performance of the entire structure. Base Isolation has shown positive effects by reducing the induced accelerations on a structure by increasing the period of vibration. This investigation outlined all relevant issues concerning the conceptual design of base isolated structures. Base isolated structures will prove to be successful in the future as new performance based codes are introduced.

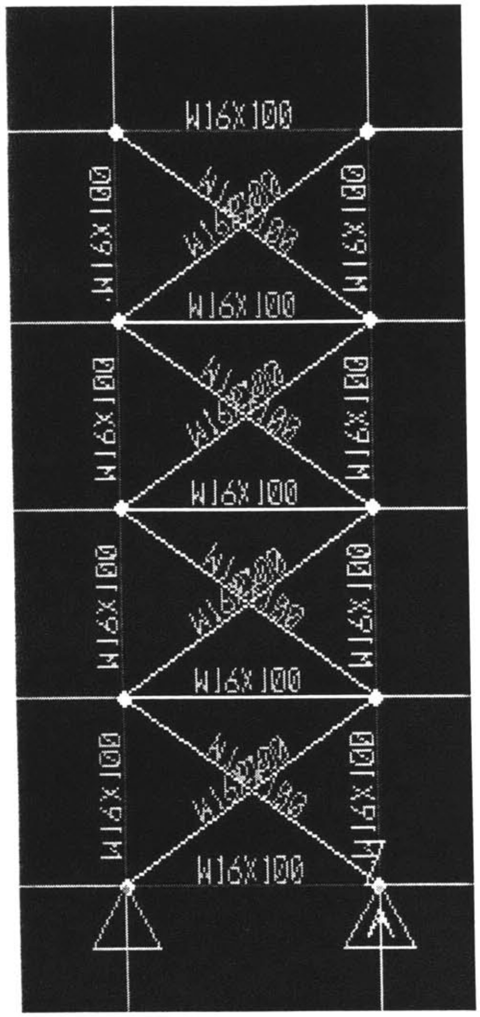
CHAPTER 7 APPENDIX A - SOFTWARE LIST

The following programs were used during the course of the structures group project and this thesis:

Seismic Analysis	Dynamic Isolation Systems Isolate
Structural Analysis	SAP 2000
Operating System	Windows NT 4.0
Document Composition	MS Word 97
Calculations	MS Excel 97
Drafting	AutoCAD 14.01
Visualization / VR Modeling	3D Studio VIZ and MAX
Geotechnical Analysis	Plaxis 7.1



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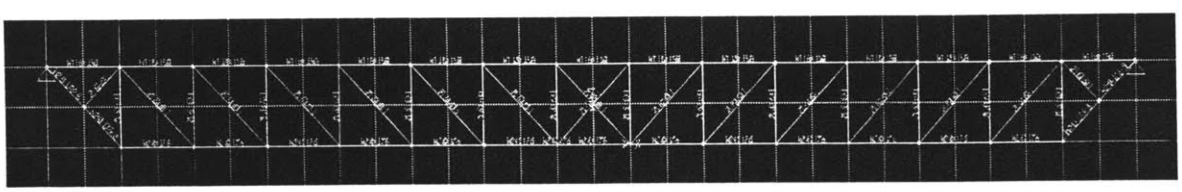

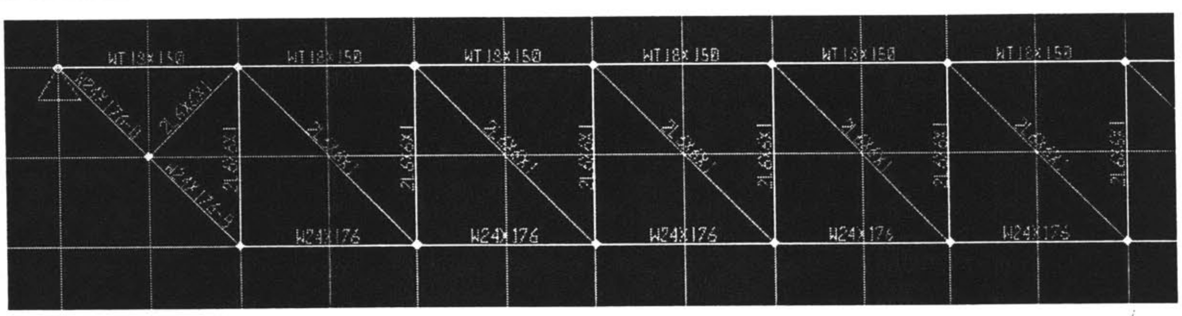
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