BASE ISOLATION CASE STUDY

by

Kenneth A. Ching
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Signature of Author: _____
Department of Civil and Environmental Engineering
May 22, 2008

Certified by: _____
Jerome J. Connor
Professor of Civil and Environmental Engineering
Thesis Supervisor

Accepted by: _____
Daniele Veneziano
Professor of Civil and Environmental Engineering
Chairman, Departmental Committee for Graduate Students

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ABSTRACT

The primary objective of this thesis is the introduction of the current code, ASCE 7-05 into the base isolation design and the analysis of base isolation response due to seismic forces. An eight story irregular structure is modeled using SAP2000 structural program. The time history of Northridge earthquake is used as a seismic forcing function of the structure. The base isolator is designed by using the principle of bilinear modeling. Therefore, the base isolation system is analyzed using the non-linear time history analysis. The response of the isolation system is analyzed, and especially its hysteresis loop. Results show that the inputted energy of the seismic forces is dissipated by hysteretic and modal damping.

Thesis Supervisor: Jerome J. Connor
Title: Professor of Civil and Environmental Engineering

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1. Introduction

1.1. Overview

A common perception on how to resist an earthquake force is by strengthening the structure. The traditional engineering design strategy based on increasing the design capacity and stiffness to accommodate foreseeable lateral forces may not be the most efficient solution. The problem with the latter is that all seismic forces from the foundation will be absorbed by the superstructure. The base isolation technique is exactly the opposite of traditional engineering design strategy.

Base isolation is a system that protects a building from the damaging effects of a seismic movement. If the structure separates from the ground during an earthquake, the ground is moving but the structure is still dormant. However, this scenario is not realistic. The current technology that is active and expanding is the introduction of a low lateral stiffness support that isolates the structure from the ground movement. This technology was introduced as early as the 1900's; however, not until the 1970's did it evolve into the practical strategy for seismic-resistant design.

1.2. Principles and Concepts

The objective of base isolation system is to decouple the structure from the ground. It lowers the effect of ground motion transmitted to the structure. When the ground moves, a perfectly rigid structure will have a total displacement equal to ground displacement, relative displacement equal to zero, and period equal to zero. Thus, a perfectly laterally flexible structure will have a total displacement equal to zero, relative displacement equal to ground displacement, and the period would be infinite.

Structures are neither perfectly rigid nor perfectly laterally flexible, and therefore the response of the structure lies in between the two extremes. Lengthening the fundamental period of the structure reduces the pseudo acceleration and the seismic forces induced in the structure. However, it increases the displacement of the structure, but mostly concentrated in the base isolation system as shown in Figure 1.1. In addition, the added damping by the isolation system allows the seismic energy inputted to be absorbed by the isolation system thereby reducing the energy transmitted to the structure. Finally, the torsional effects due to eccentricities can be reduced by coinciding the center of stiffness of the base isolation system with the center of mass of the superstructure.

In contrast, in Figure 1.2, the conventional structure has distributed displacement along its entire height. In addition, seismic energy and forces are entirely absorbed by the structure, thereby increasing the vulnerability of the whole structure.

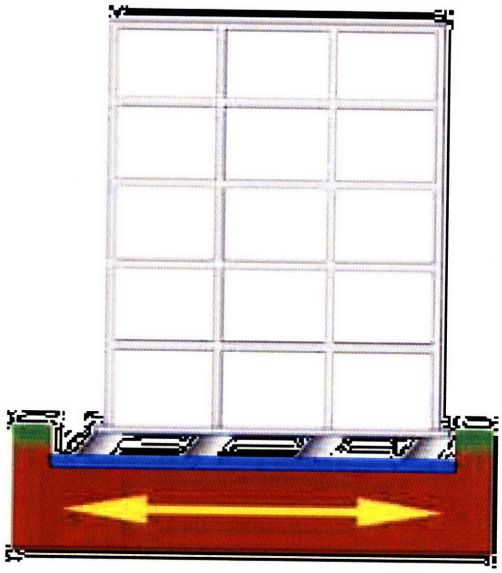

Figure 1-1 : Displacement of Based isolated Structure (DIS)

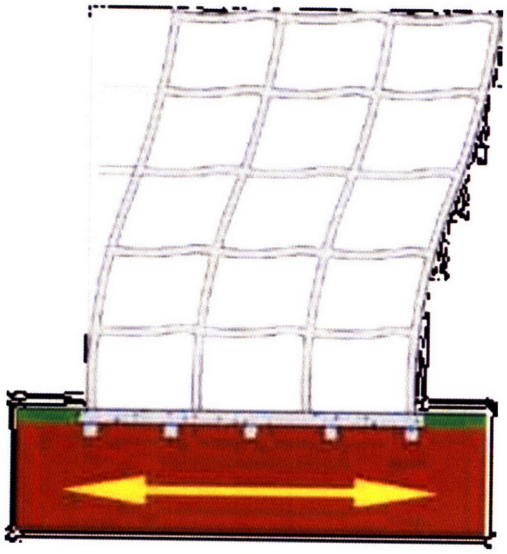

Figure 1-2: Displacement of Fixed Base Structure (DIS)

1.3. Application of Base Isolation

Structures that are midrise in height are the best candidates for base isolation technology. Base isolation design provides a good substitute for fixed based design in locations where very strong seismic activities are likely. The initial cost of base isolation might be higher compared with a fixed based. However, after a seismic event, the cost of repairing a structure plus the loss in opportunity might be considerably higher.

Most of base isolated structures in the west coast of United States are hospitals. These facilities must be in operation after a seismic event. The world's first base isolated structure in the United States is the University of Southern California Hospital, see Figure 1.3. It remained operational after experiencing the Northridge earthquake in 1994.

Figure 1-3: USC University Hospital. (www.uscuh.com)

Other structures that can benefit from base isolation design in the long term are the manufacturing facilities (i.e. semiconductors) that need to be in operation after a seismic event; otherwise economic loss is at stake.

In addition, historic structures have benefitted from base isolation technology through retrofitting. Figure 1.4 shows the installation of base isolators by jacking the columns of San Francisco City Hall.

Figure 1-4: Base Isolation Retrofitting of San Francisco City Hall. (www.celebratingeqsafety.com)

2. Major Types of Isolators

2.1. Elastomeric Bearings

Elastomeric bearings have been used widely in bridges as bearing pads between the girder and the supporting structure for many years. Elastomeric bearings have multiple layers of steel shims and rubber laminated together under high pressure and heat in a mold. Steel shims prevent lateral bulging of the rubber when axially loaded. They do not resist shear forces and do not prevent the horizontal deformation of the layered rubbers. Therefore, steel shims increase the vertical stiffness of isolators but do not increase the lateral stiffness of elastomeric bearings.

Figure 2.1 shows elastomeric bearings subjected to dynamic shear test. The uneven surface of the elastomeric bearings shown is due to its steel shims. Generally, elastomeric bearings have low critical damping resistance, approximately 2% to 3% of critical viscous damping; and have minimal resistance under service loads. Therefore elastomeric bearings need to be improved. The result is a high damping elastomeric bearings and the lead rubber bearings.

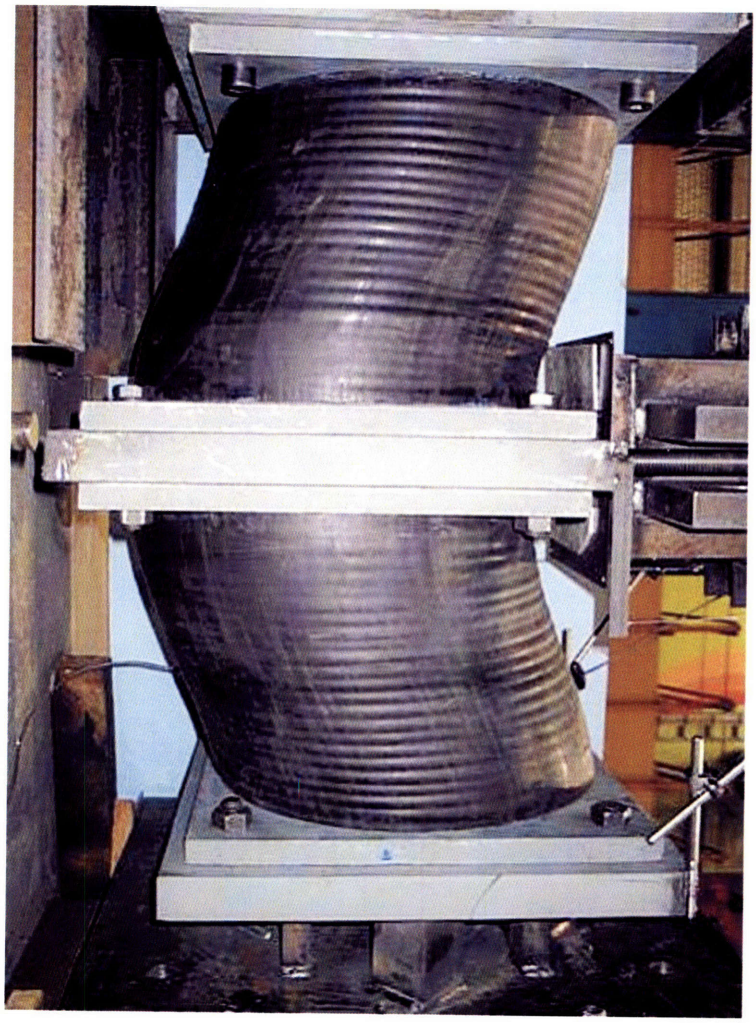

Figure 2-1: Elastomeric Bearings under Shear Test. (Maurer Söhne)

2.2. High Damping Rubber Bearings (HDRBs)

As an alternative to elastomeric bearings, high damping rubber bearings provide critical damping from 10% to 20% at 100% shear strains. The construction methodology is the same with elastomeric bearings; however, the damping is increased by adding carbon block and other fillers. In addition, it has an adequate resistance to service loads.

The damping characteristic is in between hysteretic and viscous. The energy dissipation is linear and quadratic for hysteretic and viscous, respectively. The energy absorption capability help reduced the earthquake energy transmitted to the superstructure.

The load capacity of HDRB can be computed using the same method as elastomeric bearings. The damping value can be computed using the equivalent damping ratios for specific elastomeric compounds.

2.3. Lear Rubber Bearings (LRBs)

Lead rubber bearings are elastomeric bearings that contain one or more lead plugs inserted into their preformed holes. The lead provides significant stiffness under service loads and low lateral loads as compare to the elastomeric bearings. Figure 2.2 shows the alternating sheet of steel shims and rubbers circumscribing a lead core. In addition, the lead serves as energy dissipation mechanism under severe lateral loads.

During high lateral loads, the lead yields and the lateral stiffness of the LRB is significantly reduced. This increases the duration of the period of the structure and thereby serves the purpose of base isolation system. The bearing is cycled into a hysteretic damping as it absorbs the energy. LRB has a range of damping from 15% to 30% which is a function of displacement.

The LRB are the most common base isolator used for isolating midrise buildings. It is usually designed and optimized according to a specific performance based target design. It combines the stiffness needed for service loads and low lateral loads while providing the flexibility and damping needed for high lateral loads. A wide array of damping and stiffness is possible through the use of LRB. As for the case of HDRB design, the formulas of elastomeric bearings are suitable for the design of LRB.

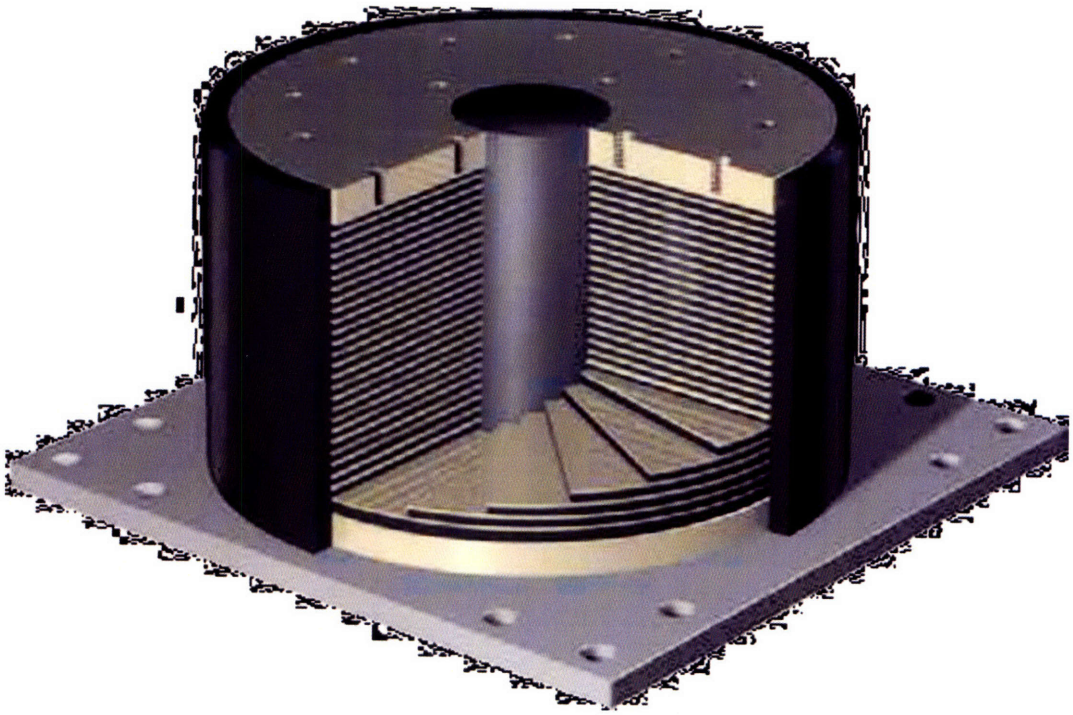

Figure 2-2: Lead Rubber Bearing with Layers of Rubber and Steel; and a Lead Core. (DIS)

2.4. Flat Sliding Bearings

Flat sliding bearings consist of PTFE (Teflon) disc that slides on a stainless steel plate. They provide a perfectly plastic hysteresis shape, and adequate stiffness under service loads with high damping properties. In addition, the coefficient of friction is a function of both pressure and velocity of sliding. It provides the resistance under service loads. However, it must be combined with other bearings (i.e. HDRBs, LRBs) because it has no capability to return to its initial position.

Figure 2.3 shows an assembly of flat sliding bearing, which has the stainless steel plate supporting the circular disc. There is no other part in the assembly that shows that it has the capability to return to its initial position. A modified version of flat sliding bearing that has self restoring force is the friction pendulum bearings.

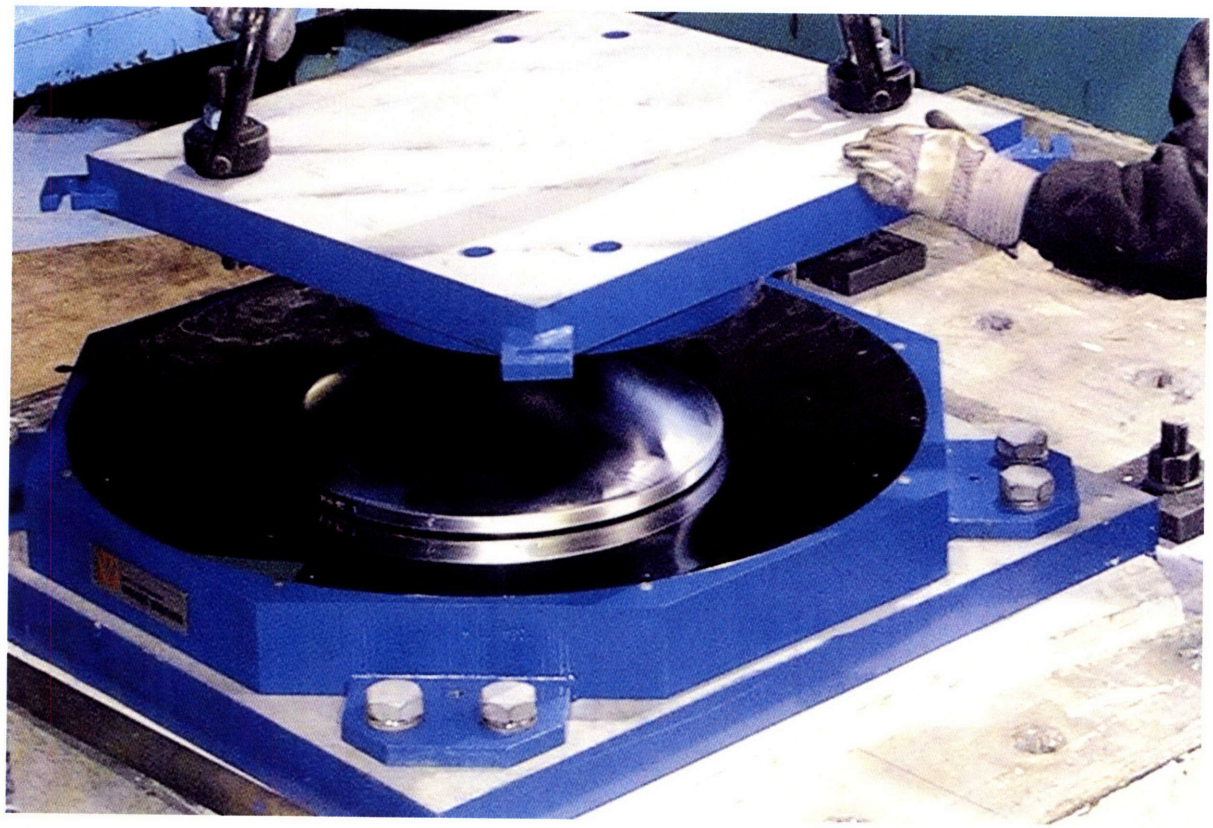

Figure 2-3: Assembly of Flat Sliding Bearing. (Maurer Söhne)

2.5. Friction Pendulum Bearings

The friction pendulum bearings have the same properties as the flat sliding bearings. However, the sliding surface is concave in shape rather than flat as shown in Figure 2.4. The hemisphere at the center of the concave surface is the pendulum slider. The spherical concave surface provides a restoring force to the pendulum slider to return to its initial position.

Varying the radius of the concave surface varies the stiffness of the friction pendulum bearings. In addition, once the coefficient of friction is overcome the lateral movement of the mass is accompanied by a vertical movement of the mass because of the curved shape of the slider. As for the case of LRB, the friction pendulum bearings have a wide array of damping and stiffness design capabilities.

Figure 2-4: Friction Pendulum Bearing Assembly. (EPS)

3. Case Study

3.1. Background of the Base Isolated Structure

The structure is a semiconductor manufacturing facility that is located in Santa Monica City, California. It has a total length of 271.0 ft, width of 147.6 ft, roof height of 78.7 ft, and a total height of 147.6 ft. The total floor area is approximately 191,000 sq. ft. Based on FEMA-273, the structure is an Immediate Occupancy (IO) building performance level. IO performance level mandates that the building remain functional during and immediately after the earthquake.

The first design scheme of the structure has a fixed base as shown in Figure 3.1. The main structural elements that resist seismic forces are the shear walls. In addition, the structure is a moment resisting frame. It has a fundamental period of 0.488 sec.

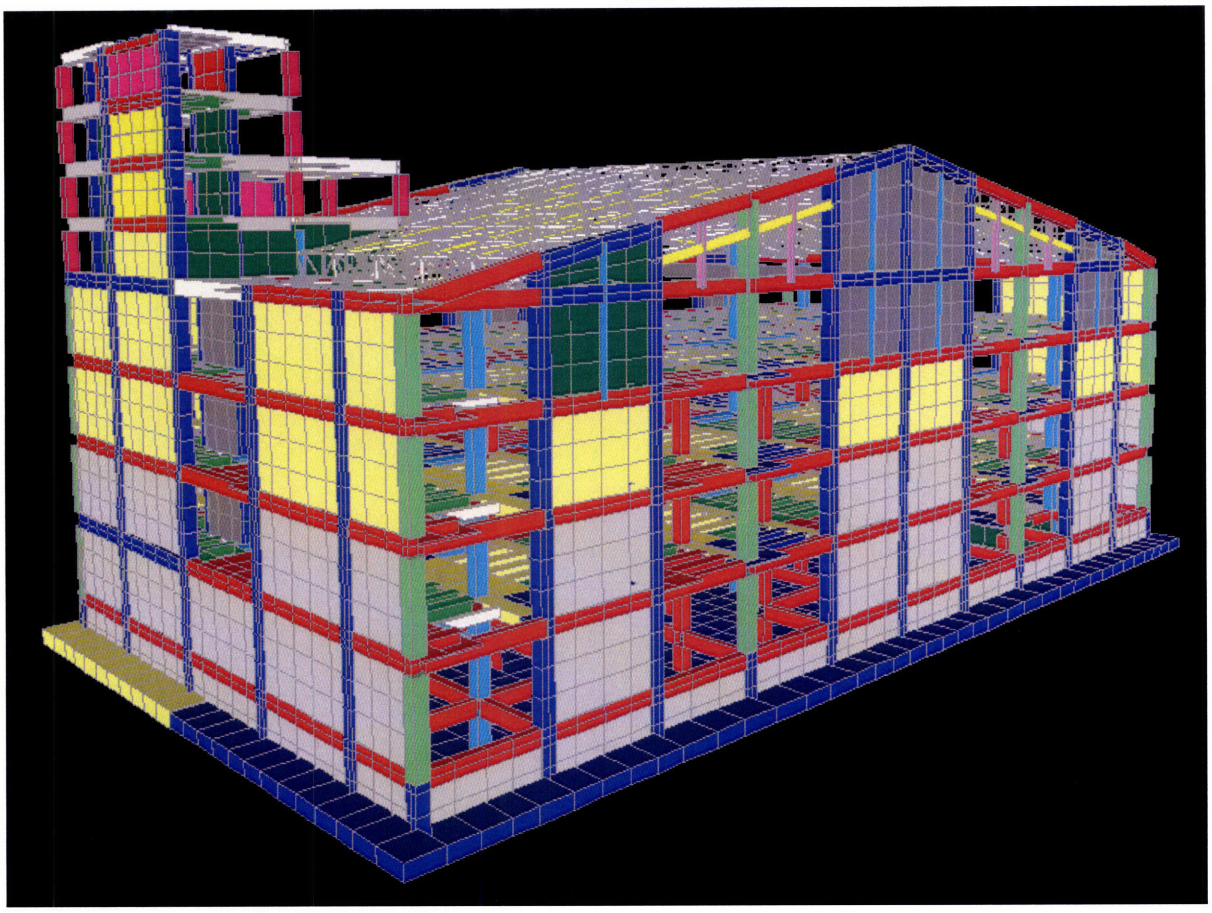

Figure 3-1: A Fixed Base Structure

The second design scheme of the structure has an isolated base as shown in Figure 3.2. The isolation system reduces the acceleration transferred to the sensitive semiconductor manufacturing equipment. Under the base isolated structure, the shear walls and moment resisting frames from previous fixed base design are subject to modification by means of lowering the design capacity and thus lowering the superstructure cost. However, in this case study, the superstructure has not been modified.

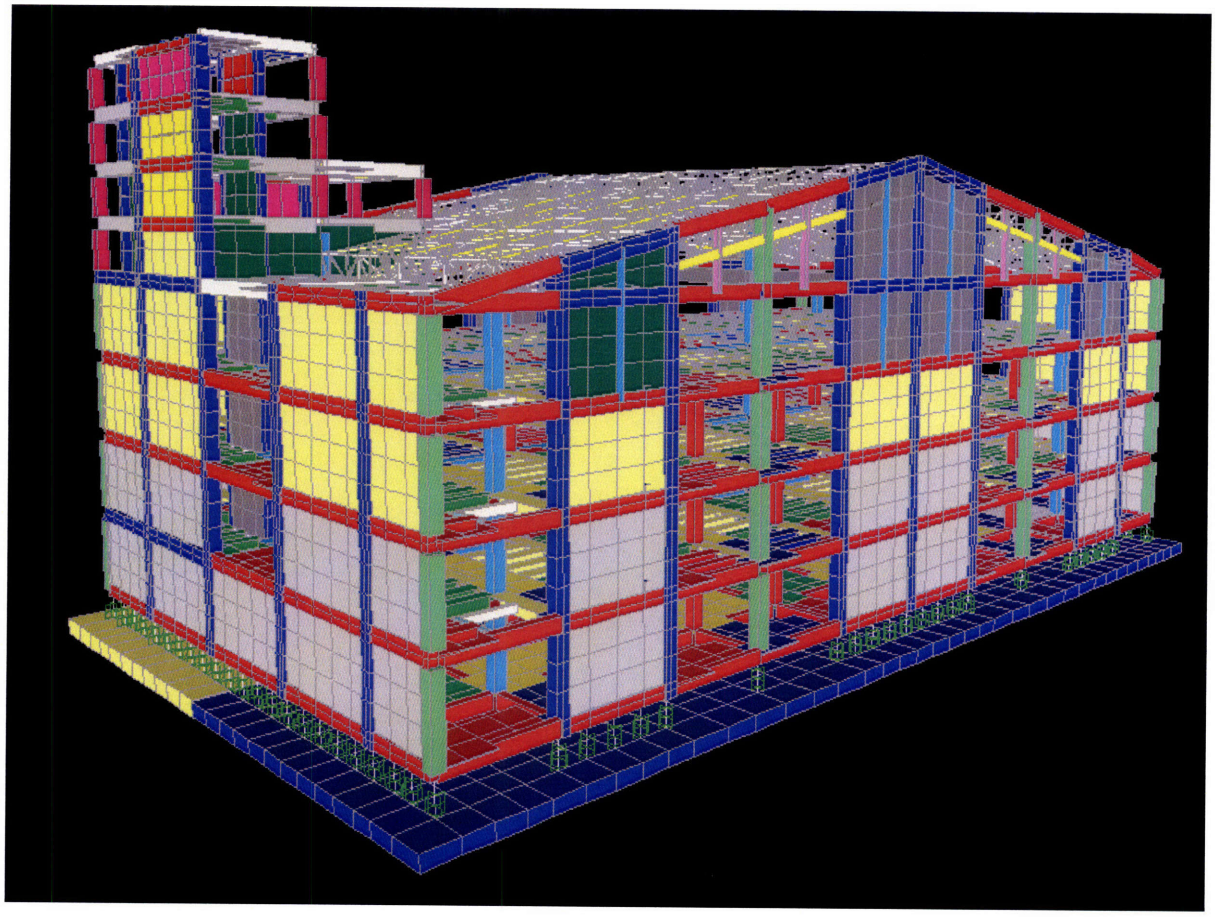

Figure 3-2: A Base Isolated Structure

3.2. Equivalent Lateral Force Procedure

The equivalent lateral force procedure based on ASCE 7-05 will be used for the preliminary analysis only. The sample structure does not conform to the code requirement, particularly the response spectral acceleration, S_1, should be less than 0.6g. The sample building has S_1 = 0.8091g, which will be further discussed in the succeeding chapters. Next, the height of the structure should not be more than four stories or 65 ft. Therefore, the case study needs to include the dynamic analysis based on time history procedure. However, the output data of equivalent lateral force procedure will be a reference data for the dynamic analysis. In addition, it will be included in the SAP2000 output case as EQX.

3.3. Response History Procedure

ASCE 7-05 stated that a response history procedure is permitted for the design of any seismically isolated structure.

Where a response history procedure is performed, a suite of not fewer than three appropriate ground motions shall be used in the analysis and the ground motion pairs shall be selected and scaled in accordance with ASCE 7-05 Section 17.3.2.

The Santa Monica City provided three pairs of appropriate recorded ground motions and therefore scaling is not necessary anymore. The first recorded ground motion pairs is Santa Monica City Hall Grounds 0 degree and 90 degree, SMC1 and SMC2, respectively. The second pair is Sylmar County Hospital Parking Lot 0 degree and 90 degree, SYL1 and SYL2, respectively. The Century City Lacc 90 degree, LAC, has only one seismic direction data available. The recorded ground motions are the time history forcing functions for the dynamic analysis in SAP2000.

ASCE 7-05 stated that each pair of ground motion components shall be applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal displacements at each time step. However, this case study is interested in the seismic response in one direction without orthogonal effect. Therefore each pair is applied separately. The longitudinal, X direction is subjected to 0 degree seismic force and then the 90 degree seismic force.

3.4. Computer Modeling Procedure

SAP2000 was used to model the whole structure. The model is able to qualify to the standard set by ASCE 7-05. The structure is modeled and analyze as a three dimensional structure. It has six degrees of freedom. They are the two horizontal

movements, vertical movement, rotation about X-axis, rotation about Y-axis, and rotation about Z-axis. Therefore, the floor diaphragms can appropriately distribute the seismic forces to lateral resisting elements according to its stiffness. In addition the model considered the cracked sections properties of concrete and shear walls.

There is a hierarchy of analysis procedures in using computer modeling. It is advisable to model and analyze the structure in a simpler procedure before the complex procedure. The basic step is the three dimensional linear analysis for member design forces. The intermediate step is the three dimensional linear structure with non-linear isolators. The complex step is the three dimensional non-linear structure with non-linear isolators.

Three Dimensional Linear Model

Current CSi programs such as SAP2000 are well-equipped to handle the three dimensional linear model. The response spectrum was frequently used as a forcing function for dynamic loading. The isolators can be modeled by assigning an element (NLink) with stiffness and damping properties. However, there is a possibility of under-estimating the overturning moments of the non-linear isolators if the linear analysis is used.

Three Dimensional Linear Structure and Non-Linear Isolators

Three dimensional linear structure and non-linear isolators is used in this case study. Response history cannot be used for non-linear yielding isolators. Therefore, non-linear response history analysis is used to model and analyze the performance of the isolators. However, the structure is still modeled and analyzed as linear structure for design purposes. This type of model is sufficient where there is little yielding of structures above the isolators.

Three Dimensional Non-Linear Structure and Non-Linear Isolators

The last step is full non-linear structural modeling. SAP2000 is capable of performing full non-linear structural analysis; however, the required makes this approach impractical for this case study. It takes more than ten times longer to analyze a full non-linear model than a partial non-linear model. This discourages the iteration needed for optimal isolators and member design, which are more important. Most often, fully non-linear analysis are used for special structures (i.e. nuclear power plant).

4. Equivalent Lateral Force Procedure for Base Isolation

4.1. Design Parameters

Importance Factor of Base Isolated Structure

The importance factor for, I, for the base isolated structure shall be taken as 1.0 regardless of its occupancy category.

$I = 1$

Mapped Acceleration Parameters

The mapped acceleration parameters, S_S and S_1 shall be determined from ASCE 7-05 Figures 22-3 and 22-4 respectively for Sta. Monica City California.

$S_S = 2.023$

$S_1 = 0.8091$

Site Class and Site Coefficients

The site class refers to the soil properties on site. This design problem assumed that it is located in Site Class B. Site class is necessary to determine the short-period site coefficient (at 0.2 s-period), F_a; and long-period site coefficient (at 1.0 s-period), F_v. F_a and F_v shall be determined from ASCE 7-05 Tables 11.4-1 and 11.4-2, respectively.

$F_a = 1$

$F_v = 1$

Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters

The adjusted maximum considered earthquake (MCE) spectral response acceleration parameters are the S_{MS} and S_{M1} for short periods and at 1 s period, respectively. MCE is defined as the most severe earthquake effects considered and as defined by ASCE 7-05 Section 11.4. They are the adjusted S_S and S_1 from ASCE 7-05 Equations 11.4-1 and 11.4-2, respectively, which considered the Site Class effects.

$$S_{MS} = F_a S_S \qquad (11.4\text{-}1)$$
$$= 2.023$$
$$S_{M1} = F_v S_1 \qquad (11.4\text{-}2)$$
$$= 0.8091$$

Design Spectral Acceleration Parameters

The design spectral acceleration parameters are short period, S_{DS}; and at 1 s period, S_{D1}. They are determined from ASCE 7-05 Equations 11.4-3 and 11.4-4, respectively.

$$S_{DS} = 2/3 S_{MS} \qquad (11.4\text{-}3)$$
$$= 1.35$$
$$S_{D1} = 2/3 S_{M1} \qquad (11.4\text{-}4)$$
$$= 0.54$$

Damping Coefficients

For the design purposes, we use 15% effective critical damping to determine B_D and B_M. B_D and B_M shall be determined using ASCE 7-05 Table 17.5-1.

B_D = 1.35 (15% used for the design of base isolators)

B_M = 1.35 (15% used for the design of base isolators)

Effective Seismic Weight

The effective seismic weight, W, in the mathematical structural model includes the total dead load and 25% of the floor live load as defined in ASCE 7-05 Section 12.7.2.

$W = 68621$ kips

4.2. Period of the Structure

Fundamental Period of Fixed Base Structure

The fundamental period of the structure with fixed base was determined from mathematical structural model using SAP 2000 and was in accordance with ASCE 7-05. The detailed analysis of the fixed base fundamental period is not in the scope of this thesis. The movement of fundamental mode is in X-direction. For the notation of movement direction, a subscript "x" is incorporated in all period, T; and displacement, D.

$T_X = 0.49$ Sec (Fundamental period of fixed base)

Effective Period at Design Displacement

The effective period at design displacement, $T_{D,X}$ is the design fundamental period for the base isolators and structural model.

$T_{D,X} = 2.50$ Sec (Design Fundamental Period)

Effective Period at Maximum Displacement

The $T_{M,X}$ is the effective period at maximum displacement. The maximum displacement occurs at MCE.

$T_{M,X} = 6.16$ Sec (MCE Period)

4.3. Stiffness of the Structure

Minimum Effective Stiffness of the Isolation System at Design Displacement

The minimum effective stiffness of the isolation system at design displacement, K_{Dmin}, shall be determined from ASCE 7-05 Equation 17.5-2.

$$T_{D,X} = \frac{2\pi(W)^{0.5}}{(K_{D,min}\, g)^{0.5}} \qquad (17.5\text{-}2)$$

$K_{D,min}$ = 1121.76 kips/in.

Minimum Effective Stiffness of the Isolation System at Maximum Displacement

The minimum effective stiffness of the isolation system at maximum displacement, $K_{M,min}$, shall be determined from ASCE 7-05 Equation 17.5-4.

$$T_{M,X} = \frac{2\pi(W)^{0.5}}{(K_{M,min}\, g)^{0.5}} \qquad (17.5\text{-}4)$$

$K_{M,min}$ 184.99 kips/in.

Maximum Effective Stiffness of the Isolation System at Design Displacement

Typically, a margin of ±10% variations in stiffness from the mean values of effective stiffness of the isolation system at design displacement is assumed.

$$K_{D,max} = \frac{(1.1)*K_{D,min}}{0.9}$$

= 1371.04 kips/in.

Maximum Effective Stiffness of the Isolation System at Maximum Displacement

Typically, a margin of ±10% variations in stiffness from the mean values of effective stiffness of the isolation system at maximum displacement is assumed.

$$K_{M,max} = \frac{(1.1)*K_{M,min}}{0.9}$$

$$= 226.09 \quad \text{kips/in.}$$

4.4. Displacement of the Structure

Design Displacement

The design displacement, $D_{D,X}$, of the isolation system shall be designed to resist minimum lateral earthquake displacement. $D_{D,X}$ shall be determined from ASCE 7-05 Equation 17.5-1.

$$D_{D,X} = \frac{gS_{D1}T_{D,X}}{4\pi^2 B_D} \qquad (17.5\text{-}1)$$

$$= 9.78 \quad \text{in}$$

Maximum Displacement

The maximum displacement, $D_{M,X}$, of the isolation system shall be designed to resist maximum lateral earthquake displacement. $D_{M,X}$ shall be determined from ASCE 7-05 Equation 17.5-3. The maximum displacement will be referred as MCE displacement, considered displacement, and allowable displacement in the next chapter

$$D_{M,X} = \frac{gS_{M1}T_{M,X}}{4\pi^2 B_M} \qquad (17.5\text{-}3)$$

$$= 36.11 \quad \text{in}$$

Total Displacement

Total displacement is accounted due to accidental torsion. Total design displacement, $D_{TD,X}$, and total maximum displacement, $D_{TM,X}$ shall be determined from ASCE 7-05 Eqs. 17.5-5 and 17.5-6, respectively shall not be less than 1.1 times $D_{D,X}$ and $D_{M,X}$, respectively.

$D_{TD,X}$	=	$D_{D,X}$	$(1+y_y 12e_y/(b^2+d^2))$			(17.5-5)
	=	9.79	in	<	1.1 $D_{D,X}$	
	=	10.75	in	used	1.1 $D_{D,X}$	

$D_{TM,X}$	=	$D_{M,X}$	$(1+y_y 12e_y/(b^2+d^2))$			(17.5-6)
	=	36.15	in	<	1.1 $D_{M,X}$	
	=	39.72	in	used	1.1 $D_{M,X}$	

Where,

B	=	147.64	ft
D	=	271	ft
y_y	=	1.04	ft
e_y	=	$0.05(w)+y_y$	ft
	=	8.422	ft

For the calculation of y_y, see Appendix.

Dynamic Displacement

For dynamic analysis procedures, the design displacement, $D'_{D,X}$; and the maximum displacement, $D'_{M,X}$ shall be determined from ASCE 7-05 equations 17.6-1 and 17.6-2, respectively.

$$D'_{D,X} = \frac{D_{D,X}}{(1+(T_X/T_{D,X})^2)^{0.5}} \qquad (17.6\text{-}1)$$

$$= 9.60 \quad \text{in}$$

$$D'_{MX} = \frac{D_{M,X}}{(1+(T_X/T_{M,X})^2)^{0.5}} \qquad (17.6\text{-}2)$$

$$= 36.00 \quad \text{in}$$

4.5. Lateral Forces

Minimum Lateral Forces

The design forces for the substructure which include the isolators, the foundation and other structural elements shall be designed to resist a minimum lateral seismic force, $V_{b,X}$, and shall be determined from ASCE 7-05 Equation 17.5-7.

$$V_{b,X} = K_{D,max}D_{D,X} \qquad (17.5\text{-}7)$$

$$= 13404.31 \quad \text{kips}$$

Structural Elements above the Isolation System

The design forces for the superstructure shall be designed to resist a minimum force, V_S, and shall be determined from ASCE 7-05 Equation 17.5-8. The R_I factor is calculated by three-eighths of the R from ASCE 7-05 Table 12.2-1, but should not be lower than 1.0 or higher than 2.0.

$$V_S = \frac{K_{D,max}D_{D,X}}{R_I} \qquad (17.5\text{-}8)$$

$$= 6702.15 \quad \text{kips}$$

The value of V_S shall not be less than the following:

First, the equivalent lateral force procedure from ASCE 7-05 Section 12.8 where W is the same effective seismic weight $T_{D,X}$ is the isolated period. The seismic response coefficient, C_S shall be determined from ASCE 7-05 Equations 12.8-2, but should not

exceed ASCE 7-05 Equations 12.8-3 and 12.8-4. In addition, for structures located where $S_1 \geq 0.6g$, CS shall not be less than ASCE 7-05 Equation 12.8-6. Finally, C_S shall not be less than 0.01 from ASCE 7-05 Equation 12.8-5.

R	=	7			
$T_{D,X}$	=	2.50	sec		
T_L	=	12.00	sec		
C_s	=	$\frac{S_{DS}}{(R/I)}$			(12.8-2)
	=	0.19	unit less		
C_s	=	$\frac{S_{D1}}{T(R/I)}$		(for $T \leq T_L$)	(12.8-3)
	=	0.03	unit less		
C_s	=	$\frac{S_{D1}T_L}{T^2_{D,X}(R/I)}$		(for $T \geq T_L$)	(12.8-4)
	=	0.15	unit less		
C_s	=	$\frac{0.5S_1}{T^2_{D,X}(R/I)}$		(for $S1 \geq 0.6g$)	(12.8-6)
	=	0.0577929	unit less		
Cs	=	0.01	unit less	(minimum)	(12.8-5)
W	=	68621	kips		
V	=	C_sW		(seismic base shear)	
	=	3965.8037	kips		
V_S	>	V		(Used Vs)	

Where,

C_s	=	seismic response coefficient determined in Section 12.8.1.1
R	=	response modification coefficient as given in Table 12.2-1
T_L	=	long-period transition period as defined in Section 11.4.5
V	=	total design lateral force or shear at the base

Other limits for V_S are not critical for the design sample. See ASCE 7-05 Section 17.5.4.3 for complete limits on V_S.

Items	Units	Values		
Design Period	Sec	1.50	2.50	3.50
MCE Period	Sec	6.16	6.16	6.16
T_X	Sec	0.49	0.49	0.49
$K_{D,min}$	kips/in	3116.00	1121.76	572.33
$K_{M,min}$	kips/in	184.99	184.99	184.99
$K_{D,max}$	kips/in	3808.44	1371.04	699.51
$K_{M,max}$	kips/in	226.09	226.09	226.09
$D_{D,X}$	In	5.87	9.78	13.69
$D_{M,X}$	In	36.11	36.11	36.11
$D_{TD,X}$	In	6.45	10.75	15.06
$D_{TM,X}$	In	39.72	39.72	39.72
$D'_{D,X}$	In	5.58	9.60	13.56
D'_{MX}	In	36.00	36.00	36.00
$V_{b,X}$	In	22340.51	13404.31	9574.51
V_S	Kips	11170.26	6702.15	4787.25

Table 4-1: Summary of Equivalent Lateral Force Procedure

Table 4.1 shows the summary of equivalent lateral force procedure based with different design fundamental period using the procedure described above. The highlighted part of the table will be discussed thoroughly in the next chapter. Three design periods are given as a first step of iteration toward base isolation design scheme. The MCE period as discussed, is the maximum period at maximum displacement. The MCE period is fixed to 6.16 sec to limit our allowable dynamic displacement, D'_{MX}, to 36 inches based on equivalent lateral force procedure. The $D_{TD,X}$ is the linear static displacement that will be compared in element deformation of SAP2000's EQX output case. The $K_{D,min}$ is total stiffness of the individual isolators or simply the stiffness of the isolation system. The $K_{D,min}$ will be used in the next chapter, as K_{eff}. The $V_{b,X}$ is the calculated static base shear that will be used to scale the base shear of the SAP2000 model.

5. Design and Analysis of Bilinear Modeling

5.1. Bilinear Design Modeling Procedure

Recommended Lead Rubber Bearing Modeling Procedure

K_1	=	Initial and elastic stiffness of the isolator
K_2	=	Secondary stiffness of the isolator
K_{eff}	=	Effective Stiffness
K_V	=	Vertical Stiffness
Q	=	Hysteretic Strength
F_y	=	Yield force
D	=	Displacement
D_y	=	Yield Displacement

A lead rubber bearing isolator is a nonlinear system that is modeled as a bilinear system based on three parameters K_1, K_2, K_{eff}, F_y and Q as shown in Figure 5.1. The elastic stiffness K_1 is usually taken as 10 times of K_2 or by elastomeric bearing test. The first value, K_1, is important in resisting service loads such as wind and the second value, K_2, is important in resisting activated when the lateral forces exceeded Fy. Fy is the point where the initial stiffness changes to secondary stiffness. Q is the hysteretic strength and where the hysteresis loop intercepts the force axis. For bilinear modeling F_y and Q are usually taken to be equal. The K_v is the vertical stiffness of the base isolator.

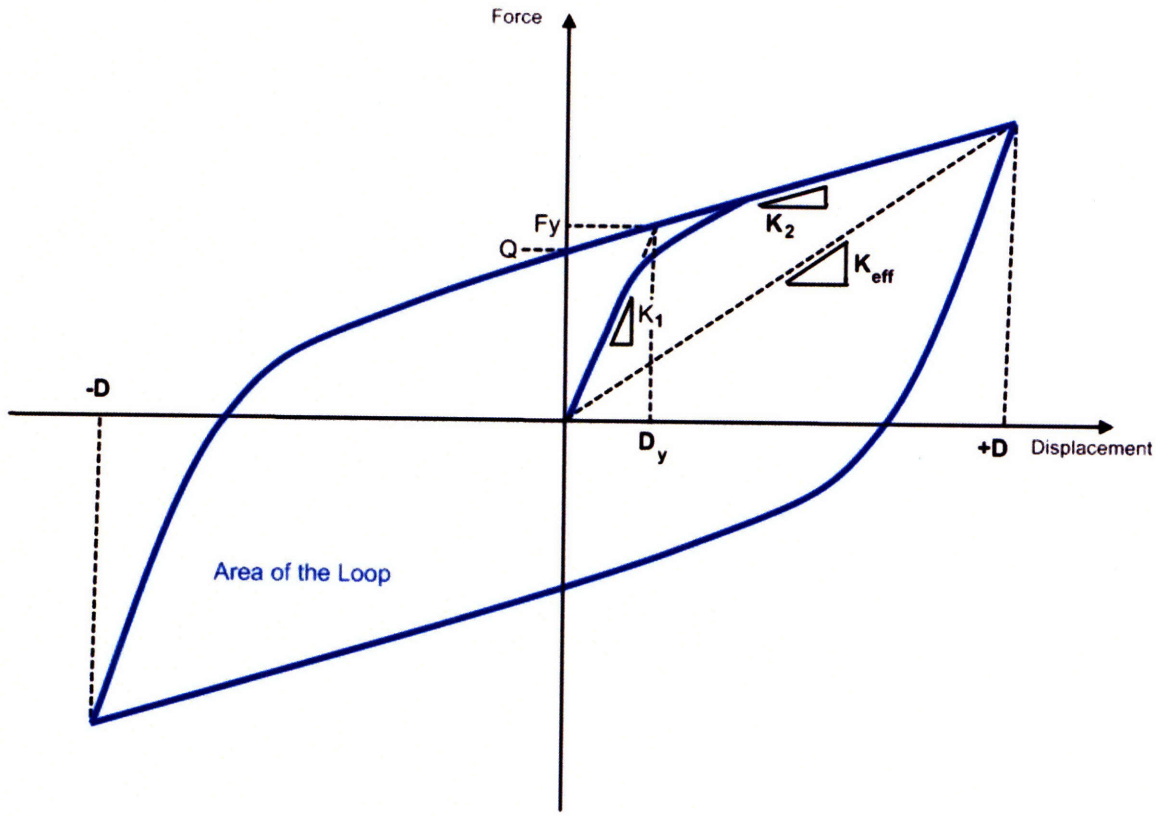

Figure 5-1: Hysteresis Loop Properties (Pierre Ghisbain)

W	=	68621	Kips	(Given)
$T_{D,X}$	=	2.5	Sec	(Targeted)
K_{eff}	=	1121.76	kips/in	(Calculated)
D	=	36	In	(Targeted)
D_y	=	0	In	(Initial Assumption)
β	=	0.15	Unit less	(Designed)

Table 5-2: Base Isolation System Design Parameters

The value of K_{eff} as determined from Table 5.2, is the secant slope of the peak to peak values in the hysteresis loop. The β is the designed critical damping.

For the nonlinear properties of the base isolation system the following procedure shall be taken. The energy dissipated per cycle of the hysteresis loop shall be determined from Equation 5.1.

$$W_D = 2\pi K_{eff} D^2 \beta \qquad (5.1)$$
$$= 1370174.16 \text{ kips}$$

The hysteretic strength, Q, of the isolation system with an initial assumption that D_y is 0 shall be determined from Equation 5.2a.

$$Q = \frac{W_D}{4D} \qquad (5.2a)$$
$$= 9515.10 \text{ Kips}$$

The first estimate of secondary stiffness, K_2, of the isolation system with $D_y = 0$ shall be determined from Equation 5.3.

$$K_2 = K_{eff} - Q/D \qquad (5.3)$$
$$= 857.45 \text{ kips/in}$$

The hysteretic strength, Q, of the isolation system with an iterated value of D_y equals to 1.25 shall be determined from Equation 5.2b.

$$Q = \frac{W_D}{4(D - D_y)} \qquad (5.2b)$$
$$= 9856.46 \text{ kips}$$

Then

$$K_2 = K_{eff} - Q/D \quad (5.4)$$
$$= 847.97 \quad \text{kips/in}$$

The initial stiffness, K_1, of the isolation system shall be determined from Equation 5.5.

$$K_1 = 10K_2 \quad (5.5)$$
$$= 8479.69 \quad \text{kips/in}$$

Finally, the iterated value of Dy shall be determined from Equation 5.6.

$$D_y = \frac{Q}{9K_2} \quad (5.6)$$
$$= 1.25 \quad \text{unit less}$$

Design Period	sec	**1.50**	**2.50**	**3.50**
No. of Isolators	pieces	127	127	127
K_{eff}/isolators	kips/in	24.54	8.83	4.51
K_1/isolators	kips/in	185.47	66.77	34.07
K_2/isolators	kips/in	18.55	6.68	3.41
Q/isolators	kips	215.58	77.61	39.60
Ration of K_2/K_1	unitless	0.10	0.10	0.10

Table 5-3: Base Isolation System Summary at Different Design Periods

The Table 5.3 shows the summary for the base isolation system for each isolator at several design periods. The values from Table 5.3 are the inputted design of isolators (link) to be used in SAP2000 structural model. The K_{eff} is the effective stiffness that is used for the linear analysis cases, which is the equivalent lateral force procedure in X-direction, (EQX). The K_1 and the Q are the stiffness and yield strength, respectively, that are used for the nonlinear analysis cases, which are the time history forcing functions. The post yield stiffness ratio is the ratio of K_2/K_1. Clearly, all the stiffness, Ks, and the yield force, Q, increase as the period decreases.

The fundamental periods for the SAP2000 structural model are based on equivalent lateral force procedure of design period 1.5 sec, 2.5 sec and 3.5 sec are 1.69 sec, 2.60 sec, and 3.55 sec. There is a 12.7%, 4.0% and 1.4 % variation for the design period 1.5 sec, 2.5 sec and 3.5 sec respectively compare with its SAP2000 counterpart.

Base Reactions:			**Fx (kips) at D = 36 in**		
OutputCase	**CaseType**	**StepType**	**T=1.693 sec**	**T=2.60 sec**	**T=3.55 sec**
EQX	LinStatic		-22338.43	-13405.28	-9570.85
LAC	NonModHist	Max	44597.07	19606.21	11861.05
LAC	NonModHist	Min	-46426.96	-20805.17	-9812.10
SMC1	NonModHist	Max	49041.26	21466.99	11847.41
SMC1	NonModHist	Min	-47100.69	-19341.36	-9905.19
SMC2	NonModHist	Max	49871.39	24626.37	15383.97
SMC2	NonModHist	Min	-53918.17	-22633.40	-10778.63
SYL1	NonModHist	Max	82645.09	40117.28	21402.31
SYL1	NonModHist	Min	-69693.07	-32780.06	-20299.03
SYL2	NonModHist	Max	68566.25	32790.57	16266.24
SYL2	NonModHist	Min	-55047.74	-28659.31	-17711.58

Table 5-4: Base Reactions at Different Given Fundamental Period

Table 5.4 and other SAP2000 output analysis tables display the Max and Min Step Type except for the Output Case EQX. The Max and Min was taken from the maximum and minimum forces from the oscillating base shear reactions due to dynamic acceleration of time history. The LinStatic and NonModHist in the Table 5.4 represent linear static analysis and nonlinear modal history analysis, respectively.

Table 5.4 displays the output base reactions based on the inputted properties from Table 5.3. The static base reaction (EQX) has only one value for base reaction. The applied equivalent lateral force was scaled to have the same value with the $V_{b,X}$ in Table 4.1. In this case the lateral deflection due to static has been magnified. See EQX in Table 5.5 for the static deflection isolation system based on SAP2000 output values and $D_{TD,X}$ in Table 4.1 for the static deflection based on calculated value from design fundamental period. There is a discrepancy between the static deflections from SAP2000 and the calculated deflection values based on design periods. As mentioned previously, the structure does not fit the criteria to equivalent lateral force procedure. Therefore, the dynamic procedure should be done using nonlinear time history analysis.

The time history considered was actual ground motion for the Santa Monica City based on previous records, therefore no scaling factor is required. Based on Table 5.4 the fundamental period of 3.55 sec yielded the lowest base shear among the three periods considered. This is because among the three periods it has the lowest stiffness, K_{eff} and K_1 is equal to 4.51 kip/in and 34.07 kip/in, respectively based on Table 5.3. The low stiffness enables the building to move more and thus lengthens the period. Lengthening the period reduces the induced acceleration in the building resulting in the lowering of applied lateral forces on the structure.

Element Deformations:			Unit (in) at D = 36		
OutputCase	CaseType	StepType	T=1.69 sec	T=2.60 sec	T=3.55 sec
EQX	LinStatic		-8.65	-13.49	-18.48
LAC	NonModHist	Max	2.33	4.24	9.87
LAC	NonModHist	Min	-2.25	-5.35	-3.65
SMC1	NonModHist	Max	3.06	6.40	10.65
SMC1	NonModHist	Min	-2.56	-3.62	-4.43
SMC2	NonModHist	Max	3.88	12.53	22.15
SMC2	NonModHist	Min	-4.36	-9.52	-4.53
SYL1	NonModHist	Max	23.34	39.88	42.55
SYL1	NonModHist	Min	-16.54	-27.86	-34.79
SYL2	NonModHist	Max	14.26	26.77	25.17
SYL2	NonModHist	Min	-4.97	-20.24	-32.87

Table 5-5: Element Deformations at Different Given Fundamental Periods

Table 5.5 is the output lateral deflection of the isolation system base on the inputted properties from Table 5.3. The Max and Min element deformation is due to the oscillating deflection of the dynamic acceleration of time history. Table 5.5 illustrates that the deflection of the critical isolators increases as the period increases. It also illustrates that the dynamic deflection from SYL1 gave the highest deflection, 42.55 in and -34.79 in for the Max and Min, respectively.

5.2. Optimizing Base Isolator Design

Iteration is essential to optimize a base isolation design. The 2.5 range period is chosen to be used to represent the optimizing procedure. The recommended lead rubber bearing modeling procedure from previous discussion was used with some modifications. The D = 36 in is replaced by the D = 33 in and D = 39 in for iteration purposes. The displacement, D, represents the limit displacement of the bilinear modeling of base isolator as shown on Figure 5.1.

Base Reactions:			Force (kips) at T = 2.60 sec		
OutputCase	**CaseType**	**StepType**	**D = 33 in**	**D = 36 in**	**D = 39 in**
EQX	LinStatic		-13405.28	-13405.28	-13405.28
LAC	NonModHist	Max	18156.99	19606.21	21182.95
LAC	NonModHist	Min	-19533.35	-20805.17	-21913.70
SMC1	NonModHist	Max	20126.39	21466.99	22816.79
SMC1	NonModHist	Min	-17920.79	-19341.36	-20760.52
SMC2	NonModHist	Max	23595.68	24626.37	25666.31
SMC2	NonModHist	Min	-21252.33	-22633.40	-24031.85
SYL1	NonModHist	Max	39146.20	40117.28	41087.39
SYL1	NonModHist	Min	-31986.79	-32780.06	-33560.00
SYL2	NonModHist	Max	31992.99	32790.57	33573.98
SYL2	NonModHist	Min	-28353.06	-28659.31	-29075.35

Table 5-6: Base Reactions at T = 2.60 sec

Table 5.6 illustrates that changing the displacement, D, does not change the linear static base shear of the isolation system. The change in the bilinear property only works for the nonlinear dynamic analysis. However, for the engineering design aspect the change in base shear reactions is negligible even taking the two end extreme limits of considered (MCE) displacement, D.

Element Deformations:			Displacement (in) at T = 2.60 sec		
OutputCase	**CaseType**	**StepType**	**D = 33 in**	**D = 36 in**	**D = 39 in**
EQX	LinStatic		-13.49	-13.49	-13.49
LAC	NonModHist	Max	4.56	4.24	4.00
LAC	NonModHist	Min	-5.41	-5.35	-5.15
SMC1	NonModHist	Max	6.66	6.40	6.16
SMC1	NonModHist	Min	-3.60	-3.62	-3.63
SMC2	NonModHist	Max	13.38	12.53	11.56
SMC2	NonModHist	Min	-9.83	-9.52	-9.31
SYL1	NonModHist	Max	40.73	39.88	38.96
SYL1	NonModHist	Min	-29.62	-27.86	-26.42
SYL2	NonModHist	Max	28.20	26.77	25.52
SYL2	NonModHist	Min	-22.31	-20.24	-18.16

Table 5-7: Element Deformations at T = 2.60 sec

The importance of optimization of the isolation system is to determine the consistency of the considered (MCE) displacement with respect to the output displacement from a computer model using the inputted properties of the isolation system. Table 5.7 shows that using a considered displacement of 36 in and 33 in will crush the isolation system if it is subjected to SYL1 seismic forces. Taking the considered displacement of 39 in is needed. The allowable displacement that the isolation system can accommodate is 39 in and the maximum displacement due to SYL1 is 38.96 in, therefore using a considered displacement of 39 in is the optimal design for a fundamental period of 2.60 sec. Table 5.8 shows the inputted properties of the isolation system for a considered displacement of 33 in, 36 in and 39 in for a design period of 2.50 sec (SAP2000, T = 2.60 sec). The effective MCE periods are 5.65 sec, 6.16 sec, and 6.67 sec for MCE displacement. MCE period should not be confused with design period which the former is the effective period at maximum displacement and the latter is the effective period at design displacement (fundamental period).

Design Period	sec	**2.50**	**2.50**	**2.50**
MCE Displacement	in	33	36	39
MCE Period	sec	5.65	6.16	6.67
No. of Isolators	pieces	127	127	127
K_{eff}/isolators	kip/in	8.83	8.83	8.83
K_1/isolators	kip/in	66.77	66.77	66.77
K_2/isolators	kip/in	6.68	6.68	6.68
Q/isolators	kips	71.14	77.61	84.08
Ration of K_2/K_1	unit less	0.10	0.10	0.10

Table 5-8: Different Base Isolation Properties at T = 2.50 sec

Table 5.8 shows the different base isolation properties at design period of 2.50 sec (SAP2000, T = 2.60 sec) with various MCE displacement (considered displacement). The Ks in the Table 5.8 in different MCE displacement are all the same. The change that mitigates the displacement is the increase of the yield strength, Q, of the material.

Therefore the characteristic of our design base isolator is based on a design period of 2.50 sec and a MCE displacement (allowable) of 39 in.

5.3. Base Isolator Response

The Figure 5.9 is the three dimensional plan view of the isolation system of the structure. It shows the location of the different isolators in the isolation system of the structure. The isolation system's isolator is connected together by a tie beam (3.3'x2') to integrate the isolators, and support the slab above. It can be seen in the right side that there are a few missing tie beams; this is to accommodate the needed space for mechanical, electrical, plumbing (MEP) of the structure. The replacement of tie beam (frame) design is an integrated slab and wide beam design using finite element method of analysis. The isolator that is inscribed in the white polygon is the isolator Link 45. It is one of the critical isolators who are in the proximity of allowable displacement of 39 in. This isolator's response will be analyzed in detail using Sylmar County Hospital Parking Lot 0 degree (SYL1) time history function.

The Figure 5.10 shows the side view of the isolation system. It had a minimal spacing (6.7 ft) between isolators to support the shear walls of the superstructure. The Figure 5.11 shows the front view of the isolation system. The isolator that has a distance adjacent is supporting a column instead of a shear wall.

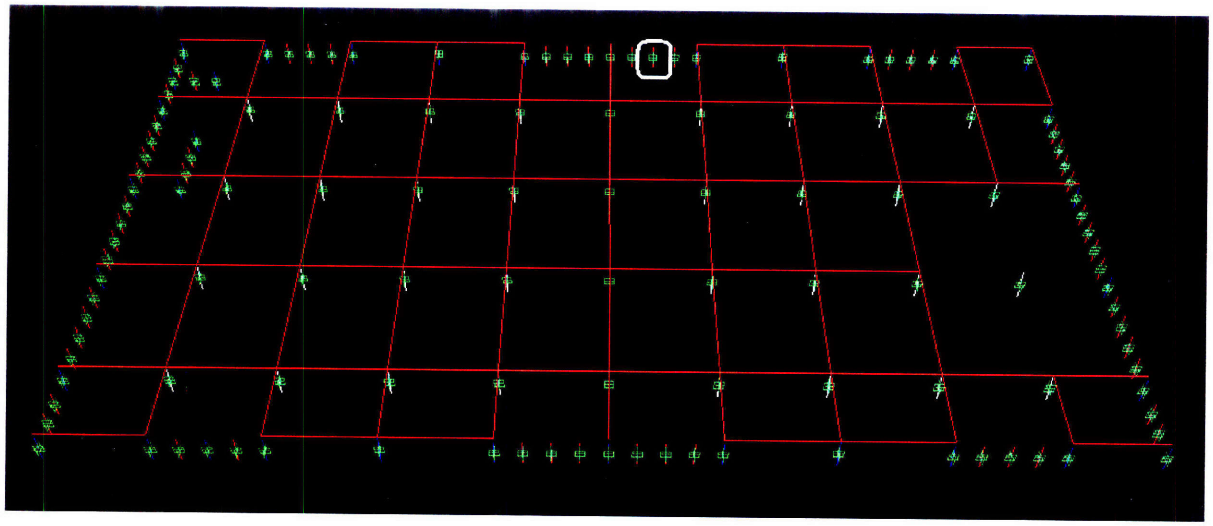

Figure 5-9: The Isolation System

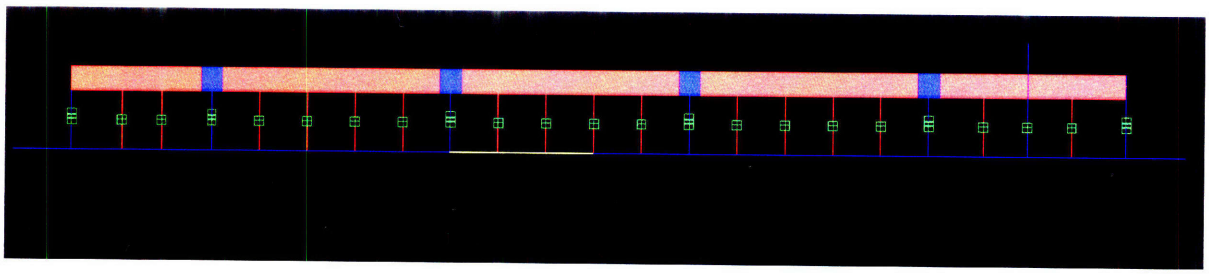

Figure 5-10: Isolation System Side View

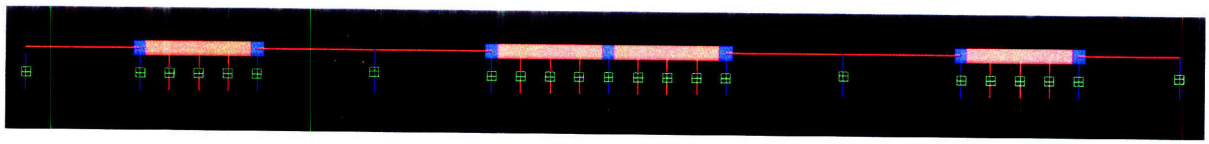

Figure 5-11: Isolation System Front View

The SYL1 time history function gave the critical response in the isolation system. It had the highest peak acceleration among the recorded time histories. In addition, the absolute maximum value of the base shear and the lateral displacement was due to SYL1 forcing function.

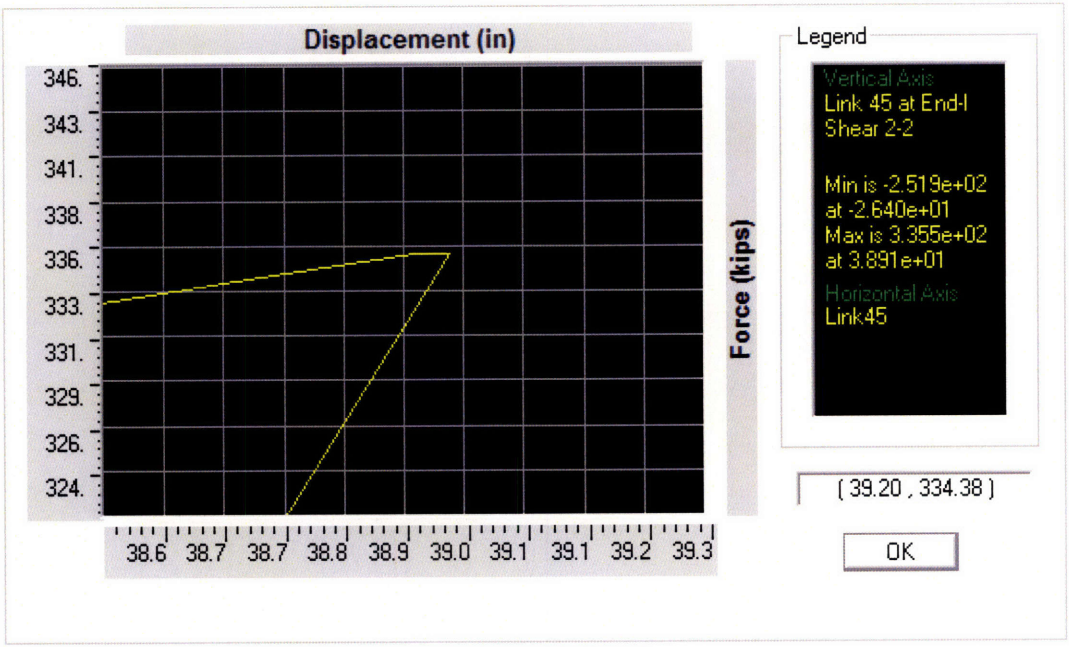

Figure 5-13: Magnified Location of Maximum Force and Displacement

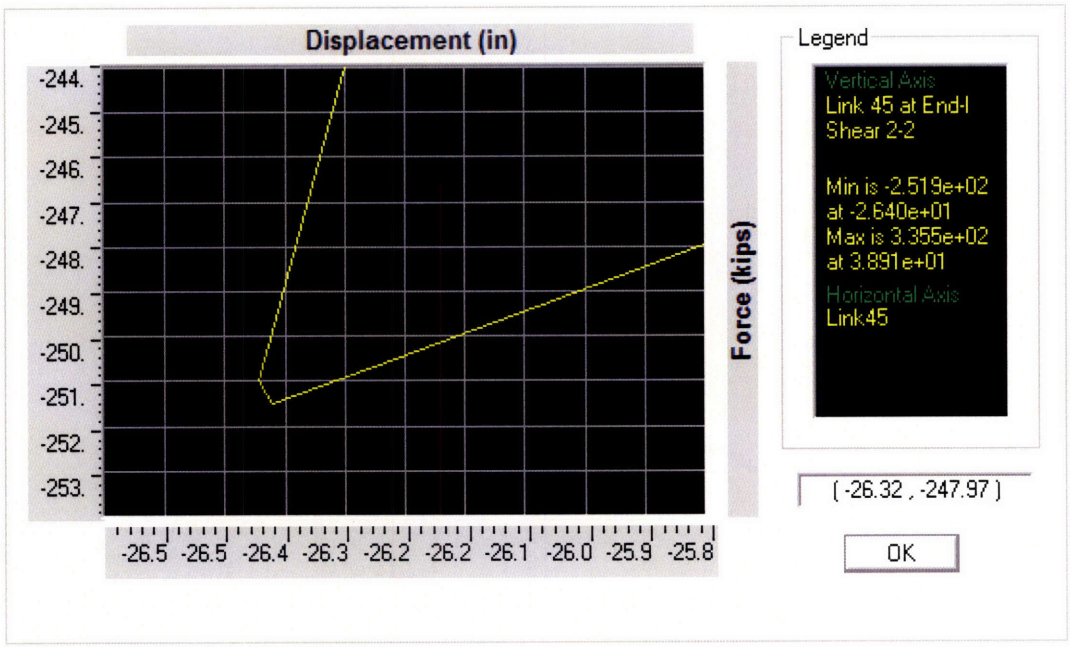

Figure 5-14: Magnified Location of Minimum Force and Displacement

The internal shear force of Link 45 with respect to time history is shown in Figure 5.15. The maximum shear force is 335.5 kips and the minimum shear force is 251.9 kips occurred at period 4.98 sec and 6.26 sec, respectively. The maximum and minimum shear force is consistent with Figure 5.12. The displacement of Link 45 with respect to time history is shown in Figure 5.16. The maximum element deformation, 38.96 in; and the minimum element deformation, 26.42 in, occurred at period 5.0 sec and 6.28 sec, respectively. The maximum and minimum element deformation is consistent with Figure 5.12. The period of maximum shear force and displacement are almost equal. Similarly, the period of minimum shear force and displacement are almost equal. 4.98 sec and 5.0 sec are for the maximum shear and displacement, respectively. 6.26 sec and 6.28 sec are for the minimum shear and deformation, respectively. However for engineering analysis, it can be assumed that the maximum shear force occurred at the maximum displacement. The minimum shear force occurred at the minimum displacement.

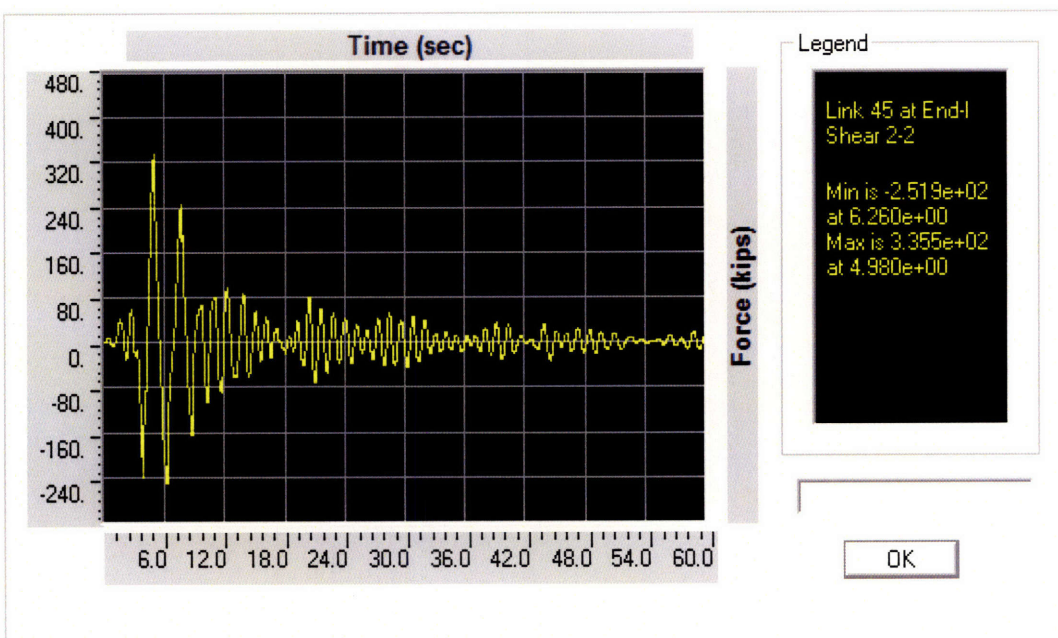

Figure 5-15: Internal Shear Force of Base Isolator Link 45

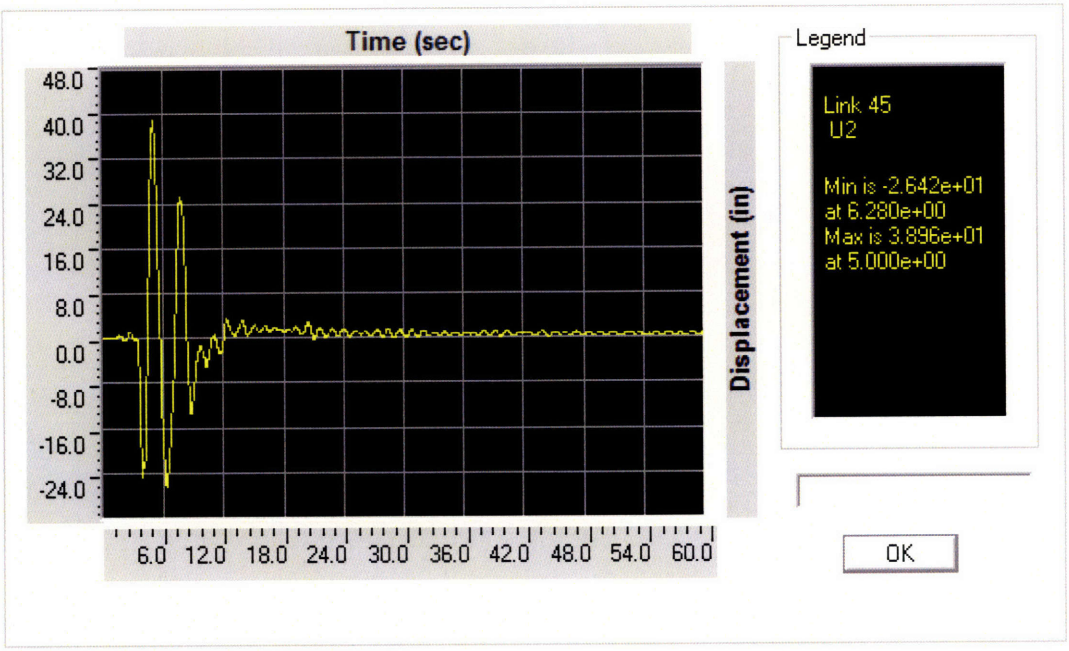

Figure 5-16: Displacement of Base Isolator Link 45

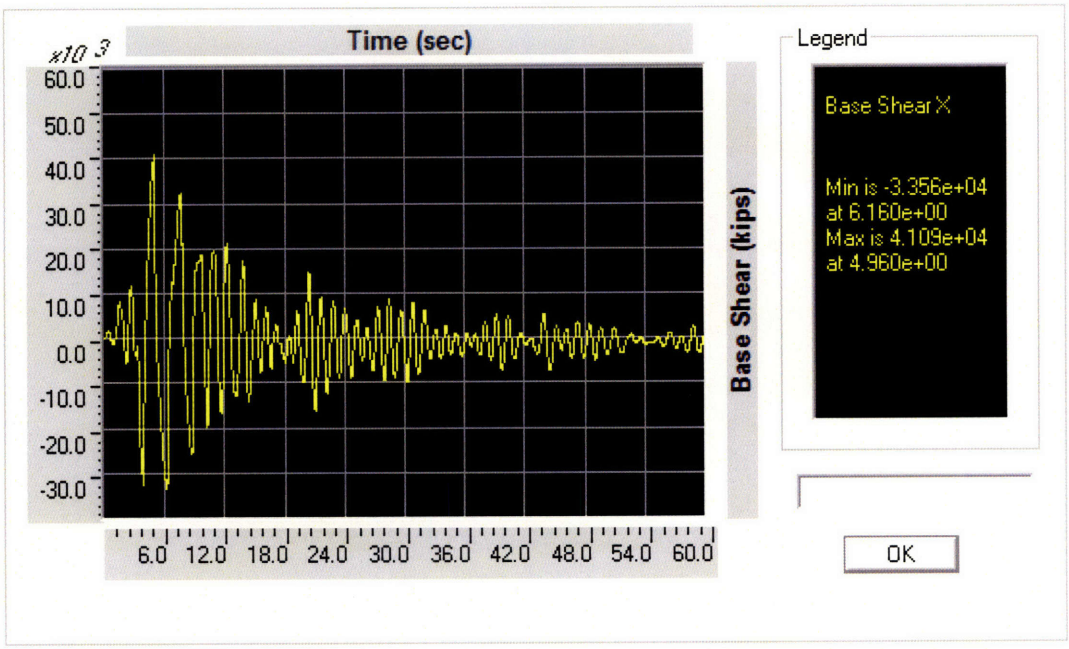

Figure 5-17: Base Shear of the Isolation System

The base shear of the isolation system with respect to time history in the X direction corresponding to the SYL1 forcing function is shown in Figure 5.17. The maximum base shear is 41087.393 kips at 4.96 sec and the minimum base shear is -33560.004 kips at 6.16 sec. These force values are consistent with the Table 5.6 at D = 39 in. The maximum and minimum base shear can be estimated by multiplying the number of isolator against and its internal shear force. Table 5.18 illustrates the conservative procedure in estimating the maximum and minimum base shear. The maximum and minimum shear force of the isolator is taken from Link 45. The estimated base shear is the product of the number of isolator and shear force.

Item	Shear Force per Isolator (kips)	No. of Isolators (unit less)	Estimated Base Shear (kips)	Actual Base Shear (kips)	Percentage Difference
Max	335.5	127	42608.5	41087.4	3.70
Min	-251.9	127	-31991.3	-33560.0	-4.67

Table 5-18: Calculated Base Shear Based on Base Isolator Link 45

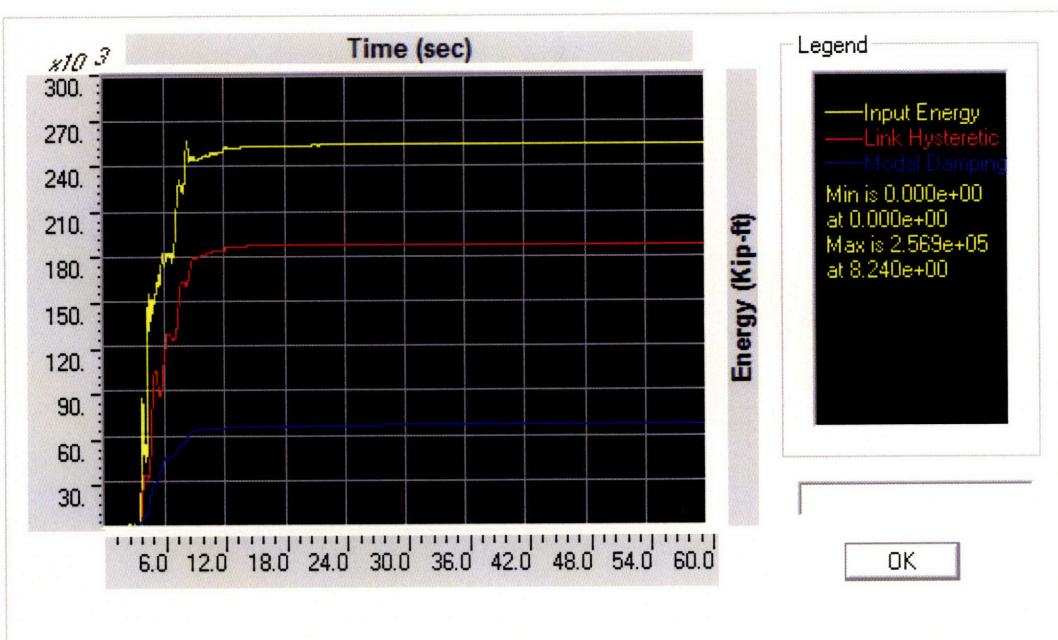

Figure 5-19: Inputted Energy and Energies Dissipated

The base shear of the isolation system with respect to time history in the X direction corresponding to the SYL1 forcing function is shown in Figure 5.17. The maximum base shear is 41087.393 kips at 4.96 sec and the minimum base shear is -33560.004 kips at 6.16 sec. These force values are consistent with the Table 5.6 at D = 39 in. The maximum and minimum base shear can be estimated by multiplying the number of isolator against and its internal shear force. Table 5.18 illustrates the conservative procedure in estimating the maximum and minimum base shear. The maximum and minimum shear force of the isolator is taken from Link 45. The estimated base shear is the product of the number of isolator and shear force.

Item	Shear Force per Isolator (kips)	No. of Isolators (unit less)	Estimated Base Shear (kips)	Actual Base Shear (kips)	Percentage Difference
Max	335.5	127	42608.5	41087.4	3.70
Min	-251.9	127	-31991.3	-33560.0	-4.67

Table 5-18: Calculated Base Shear Based on Base Isolator Link 45

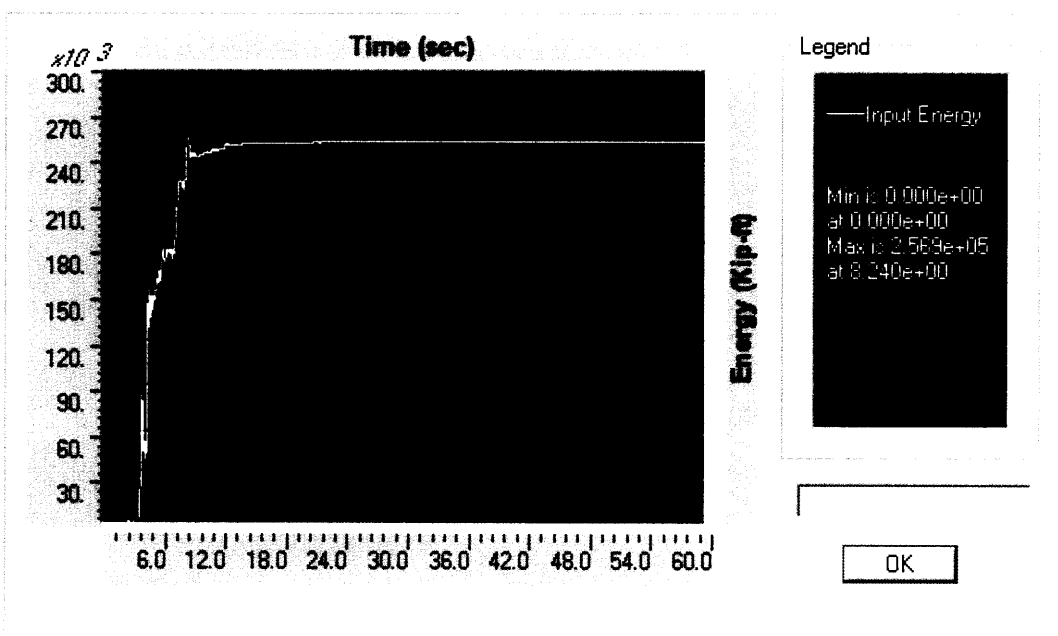

Figure 5-19: Inputted Energy and Energies Dissipated

The Figure 5.19 shows the inputted energy (yellow line) in the entire structure due to SYL1 time history forcing function. The maximum value of the inputted energy is 2.569E5 kip-ft. The energy dissipated through the link hysteresis loop is 1.878E5 kip-ft. The other energy dissipated through modal damping is 0.671E5 kip-ft. The modal damping is used in dynamic response analysis (i.e. response spectrum and time history). The material modal damping ratio, r, used in this analysis is r = 0.05. The damping ratio, r_{ij}, contributed to mode i by element j of this material is given by

$$r_{ij} = \frac{r\Phi_i^T \mathbf{K}_j \Phi_i}{\mathbf{K}_i}$$

Where Φ_i is mode shape for mode I, $\mathbf{K}_j$ is the stiffness matrix for element j, and $\mathbf{K}_i$ is the modal stiffness for mode I given by

$$K_i = \sum_j \Phi_i^T \mathbf{K}_j \Phi_i$$

summed over all elements, j, in the model (CSI Analysis Reference Manual, January 2007).

The sum of the energy dissipated is equal to the input energy. In the Figure 5.19, the sum of the magnitude of the red line and the magnitude of the blue line is equal to the magnitude of the yellow line at 8.24 seconds and onwards.

6. Conclusion

Base isolation seems to be a very promising seismic technology to use for all types of structures. However, it is not; base isolation is recommended only for midrise structure (i.e. less than 20 story high) or structures with a height to width ratio that is less than two to one. This is because the base isolation is designed to resist lateral shear forces and not to carry moments. In addition, base isolation is recommended for rigid structures that can act as a single degree of freedom. Otherwise the framing system will act as multiple single degrees of freedom with different frequencies. In this case, the building will collapse.

In addition, base isolation design should start first in a simpler way, the equivalent lateral force procedure should occur before the complicated time history analysis. If dynamic response is desired; consider first the response spectrum which yields less output to analyze. The three recorded pairs of time history produce significant amount data that can easily obscure the structural designer. Therefore, it is advisable to design first with the equivalent lateral force procedure, then by response spectrum, and finally use the time history analysis for code requirement purposes.

Lastly, in designing the base isolation system, proper coordination with the supplier is needed. In our sample design, the supplier (DIS) does not have a ready market for a displacement of 39 inches. It is made to order; therefore, the base isolator system is customized that can be very costly, and requires considerable lead time.

7. References

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Appendix

Length (l)	=	271	ft
Width (w)	=	147.64	ft

Grid for Isolators	Location of Isolators	No. of Isolators	Distance from Grid 23	Isolators x Distance
1	155.84	23	147.64	3395.72
2	148.21	1	140.01	140.01
3	142.06	3	133.86	401.58
4	135.91	1	127.71	127.71
5	128.28	11	120.08	1320.88
6	121.59	2	113.39	226.78
7	114.9	3	106.7	320.1
8	108.2	3	100	300
9	101.51	3	93.31	279.93
10	94.82	12	86.62	1039.44
11	88.12	2	79.92	159.84
12	81.43	2	73.23	146.46
13	74.74	2	66.54	133.08
14	68.04	2	59.84	119.68
15	61.35	11	53.15	584.65
16	54.66	2	46.46	92.92
17	47.97	2	39.77	79.54
18	41.27	2	33.07	66.14
19	34.58	2	26.38	52.76
20	27.89	11	19.69	216.59
21	20.83	2	12.63	25.26
22	15.26	2	7.06	14.12
23	8.2	23	0	0
Sum		**127**		**9243.19**

y	=	9243.19/127	ft
y	=	72.78	ft
y_y	=	w/2-y	
	=	**1.04**	ft