A Performance-Based Approach to Retrofitting Unreinforced Masonry Structures for Seismic Loads

by

Keith M. Bouchard

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Signature of Author: ______________________________
Department of Civil and Environmental Engineering
May 14, 2007

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

Accepted by: ______________________________
Professor Daniele Veneziano
Chairman, Departmental Committee for Graduate Studies

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Abstract

The structural inadequacy of existing unreinforced masonry (URM) buildings to resist possible seismic loading is a serious problem in many parts of the United States, including the Northeast and Midwest. The fact that many of these buildings are deemed historic structures or house critical facilities, like firehouses, emphasizes the need for an effective retrofitting program. The Federal Emergency Management Agency published a performance-based design code – FEMA 356 – in 2000 to use for analyzing and retrofitting existing structures. This code includes procedures for URM buildings. This paper applies these performance-based analysis procedures to a URM shear wall and compares the results to a modified analysis proposed by researchers. The wall is then rehabilitated using two common retrofit methods and again analyzed using the code. Recommendations are made for practicing engineers when evaluating URM structures for seismic loads.

Thesis Supervisor: Jerome J. Connor

Title: Professor, Civil and Environmental Engineering

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1. Introduction

Few types of construction are more vulnerable to seismic loading than unreinforced masonry (URM) buildings. These structures, mostly built in the late 19^{th} to mid-20^{th} century, represent a large percentage of the built environment in the United States and are often historically significant or house essential services, like fire stations. Past earthquakes around the world have shown how URM structures can fail catastrophically, causing significant loss of life and property. In an area like California where the seismic danger is well-documented, URM buildings have been gradually retrofitted or simply replaced to reduce the overall risk.

However, the west coast is not the only area of the country susceptible to earthquake events. The East coast and Midwest are also seismic regions, albeit far less active than the famed San Andreas fault. But these regions are still vulnerable to large seismic events and, in some cases, are overdue for one. To make matters worse, the Northeast and Midwest are littered with URM buildings, creating the potential for a disaster that would compare to that wrought by Hurricane Katrina.

With this in mind, the existing infrastructure in these regions needs to be rehabilitated to reduce the overall risk to life-safety and property loss. A program should be instituted to identify and retrofit high-risk URM structures to meet today's seismic codes. This paper reviews the latest publication from the Federal Emergency Management Agency (FEMA 356) which outlines a general procedure for local municipalities to follow when establishing seismic risk mitigation programs. This publication is a performance-based design code, meaning its analysis methods are based on allowable deformations rather than allowable stresses. These analyses methods will be reviewed and applied to a typical URM shear wall in a fictional building in the

Northeastern United States. This report includes defining the seismic loads on the structure, choosing a performance level, and subsequently analyzing the existing structure through both linear and nonlinear procedures. The wall will then be retrofitted using two common strengthening techniques and again analyzed using the code. The results of each analysis will be compared and recommendations will be made for retrofitting existing URM buildings according to performance-based principles.

2. Rehabilitation Procedures

Due to limited resources and the unpredictable nature of seismic events, it is impossible to make a region's infrastructure fully "earthquake-proof." All that can be done is to lower the risk of fatalities due to catastrophic failures and limit property damage. Modern building codes require new buildings to be designed to stringent seismic requirements based on the latest geological information and constructed with sound building practices. As these new, safer buildings are erected and out-dated buildings demolished, a city's inherent seismic risk naturally decreases.

This natural process, however, is not enough. Most owners cannot afford to simply replace their buildings with a new model, especially if that building is perfectly functional besides not meeting seismic codes. That practice is also highly unsustainable. Additionally, many old buildings – especially masonry buildings – are considered historic structures that cannot be demolished. This means a program to retrofit existing buildings to meet today's seismic codes is necessary to lower earthquake risks in vulnerable areas.

2.1. FEMA 356

Realizing the potential risks of a large scale earthquake striking an unprepared region, the Federal Emergency Management Agency (FEMA) issued a publication in November of 2000 to address the problem. This publication is titled the *Prestandard and Commentary for the Seismic Rehabilitation of Buildings,* or simply FEMA 356. As is stated in the standard, the idea is that FEMA 356 "specifies nationally applicable provisions for the rehabilitation of buildings to improve seismic performance" (ATC 2000). This means that regional code officials could use the standard to design retrofit programs for their area to decrease the amount of risky infrastructure.

The most innovative aspect of FEMA 356 is that its methodology is based on performance-based design. Unlike previous building codes that were focused on force-based design, the analysis procedures in FEMA 356 concentrate on allowable deformations. The code allows the engineer to consider nonlinear behavior and energy absorption after the elastic phase when analyzing an existing structure and the subsequent rehabilitation technique. This leads to a much better understanding of a structure's behavior as well as allowing for more innovative retrofit methods like base isolation or added damping. Since this is the first code of its type in the United States for existing buildings, the authors deemed it a "Prestandard" and left the design and analysis procedures open to discussion among the engineering community. The following sections will explain the procedures outlined by FEMA 356.

2.1.1. Design Procedure

The purpose of FEMA 356 is to provide a structured method to assess an existing building, no matter what the building material, and determine if and how it should be rehabilitated. The general procedure is as follows (ATC 2000):

1. Review initial considerations – These include site conditions, use and occupancy, historical status, prior evaluations, economic considerations, societal issues, and local jurisdictions.

2. Select rehabilitation objective – This involves finding the earthquake hazard level for the area and specifying a desired building performance level. This will be explained in greater detail later.

3. Obtain as-built information – This includes identifying the load carrying elements, obtaining component properties and observing adjacent buildings

4. Select rehabilitation method – Many factors are involved with this, including the type of building, the desired performance level, and the allowable budget.

5. Design rehabilitation scheme – Taking into account multi-directional seismic effects, torsion, P-Delta effects, overturning, diaphragm behavior, continuity, non-structural components, etc.

6. Verify adequacy of design – The standard outlines both acceptable linear and non-linear analyses to check the design. These will be detailed later.

7. Prepare construction documents – If the design is acceptable and financially feasible, construction can proceed.

This basic methodology can be applied to any project. The main focus of the standard is to define the design performance levels, explain the analysis procedures, and show how design values can be obtained from steel, concrete, masonry, and wood structures. FEMA 356 also includes a chapter on base isolation and energy dissipation (damping) as well as considerations for architectural and mechanical components. A flow chart describing the design procedure in detail can be found in Figure 1.

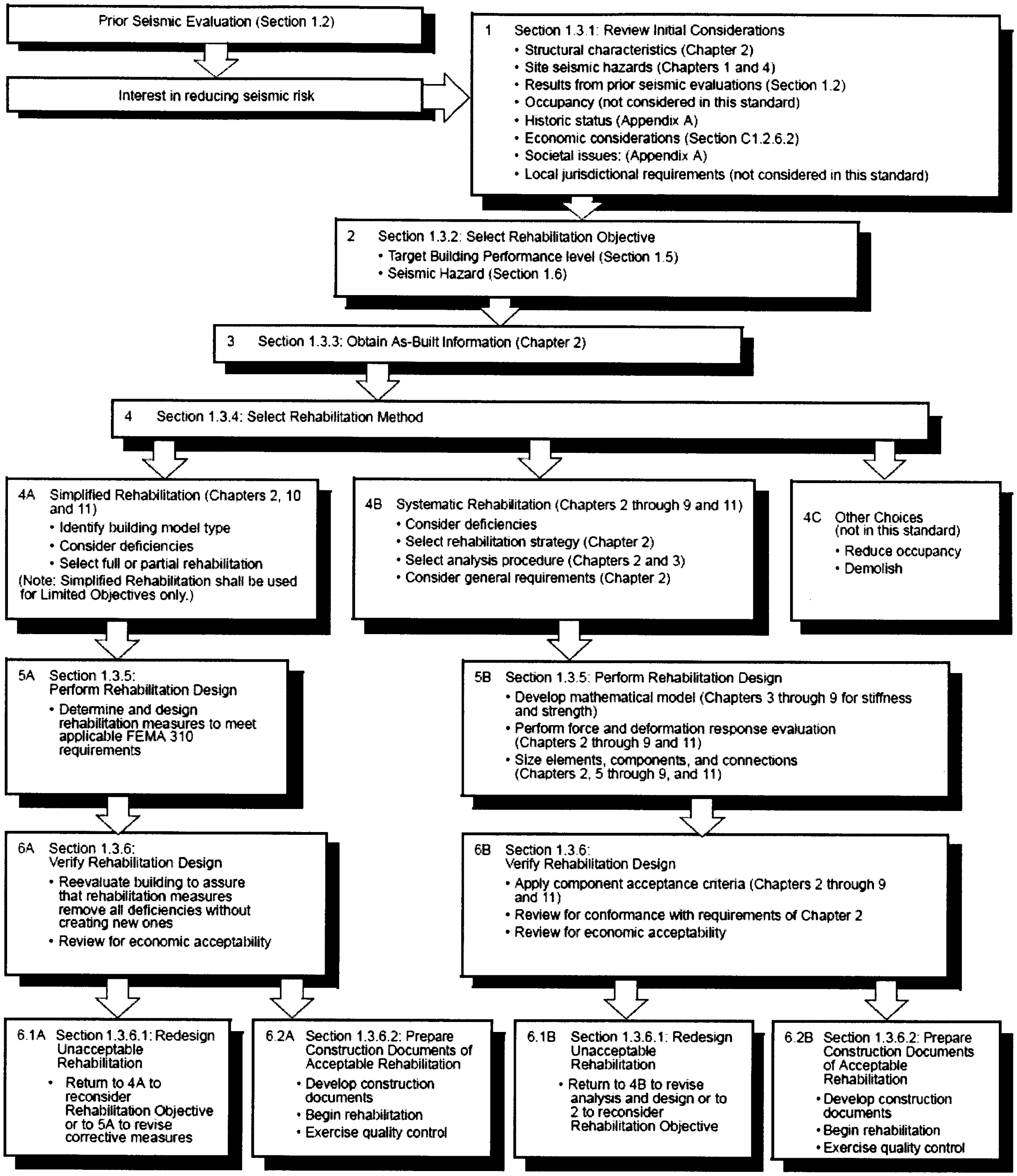

Figure 1 : FEMA 356 Rehabilitation flow chart (ATC 2000)

2.1.2. Rehabilitation Objectives

When implementing a systematic retrofit program, it is important to clearly define the goals of the rehabilitation. As was mentioned earlier, limited resources and the random nature of earthquakes make it impossible to fully protect a structure from seismic loading. Because of this, FEMA 356 specifies some simple rehabilitation objectives for existing buildings. These objectives are based on both the earthquake risk of the site and the desired condition of the building after an event.

The standard designates a "Basic Safety Objective (BSO)" that is meant to approximate the acceptable life-safety risk in the United States. From the FEMA 356 commentary: "Buildings meeting the BSO are expected to experience little damage from relatively frequent, moderate earthquakes, but significantly more damage and potential economic loss from the most severe and infrequent earthquakes that could affect them" (ATC 2000). The exact design consequences of this objective varies for each building depending on the required performance level and location, as will be explained in the following sections. The standard admits that the performance of existing buildings retrofitted to meet the BSO still may not be as desirable as new buildings designed to the same level, but this must be accepted. Designers may also choose to target a limited or enhanced rehabilitation objective if

		Target Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p

Table 1 : Rehabilitation Objectives: k + p equals the basic safety objective (ATC 2000)

financial or other considerations deem it justified. Table 1 shows the different rehabilitation objectives as defined by FEMA 356 as a function of the building performance level and earthquake hazard level.

2.1.2.1. Target Building Performance Levels

FEMA 356 defines a number of target building performance levels that can be used to assess an existing building. The main performance levels are as follows:

- Operational (O)
- Immediate occupancy (IO)
- Life-safety (LS)
- Collapse prevention (CP)

These performance levels are fairly self-explanatory and based on the desired condition of structural and architectural components in the building after an earthquake. Figure 2 shows the range of performance levels and the expected damage state after the seismic event. These performance levels are combined with the earthquake hazard level of the site to obtain the rehabilitation objective for the project.

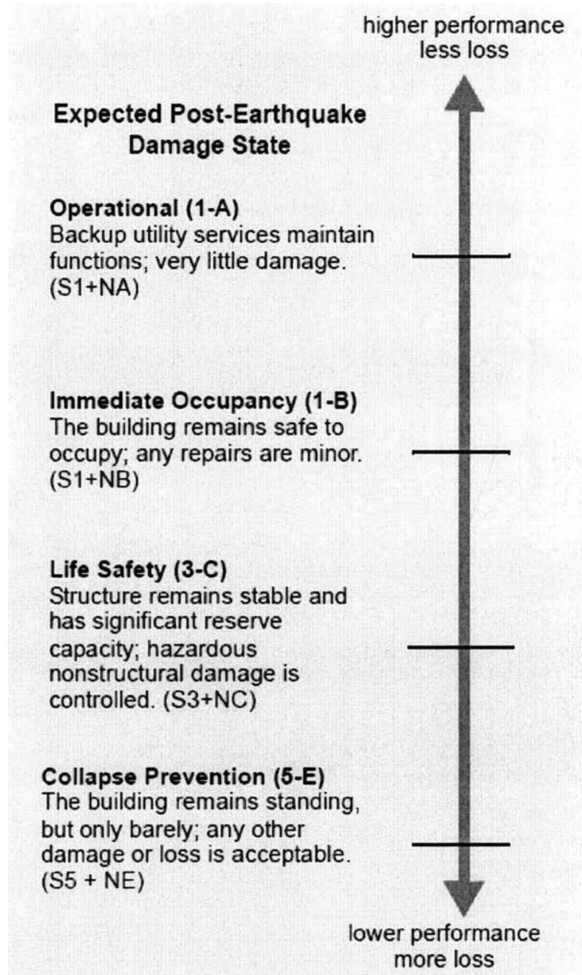

Figure 2 : Range of performance levels (ATC 2000)

The following damage levels are allowed in an unreinforced masonry building for each performance level. The collapse prevention level allows extensive cracking, peeling off of face course and veneer, and noticeable in-plane and out-of-plane offsets in the main shear/load bearing walls. Non-structural walls can completely dislodge but drift must not exceed 1%. For the life safety level, extensive cracking and noticeable in-plane offsets are allowed in both structural and non-structural elements. Out-of-plane offsets must be minor and drift cannot exceed 0.6%. Finally, for the immediate occupancy and operational performance level, only minor cracking and spalling of the veneer is allowed with no noticeable out-of-plane offsets. Drift must not exceed 0.3% (ATC 2000).

2.1.2.2. Seismic Hazard Zones

The second element involved in defining a rehabilitation objective is determining the earthquake hazard level of the site. FEMA 356 uses the national hazard maps developed by the United States Geological Survey (USGS) to find the expected ground motion for a region. Two "Basic Safety Earthquakes (BSE)" are used when defining the rehabilitation objective:

- BSE-1: 10% chance of occurrence per 50 years (500 year return period)
- BSE -2: 2% chance of occurrence per 50 years (2500 year return period)

The BSO requires that a building meet the life-safety performance level after a BSE-1 event and also meet the collapse prevention performance level after the BSE-2 event. Anything beyond this is considered an enhanced objective.

FEMA 356 outlines a basic procedure to calculate the general response spectrum for a building during both the BSE-1 and BSE-2 event. The spectral response acceleration parameters are taken from the USGS contour maps for both short-period (0.2 second) and long-period (1.0 second) response. These values are then modified according to the geotechnical site class for the specific location. The response spectrum is then calculated using the modified results. Figure 3 shows how the spectrum

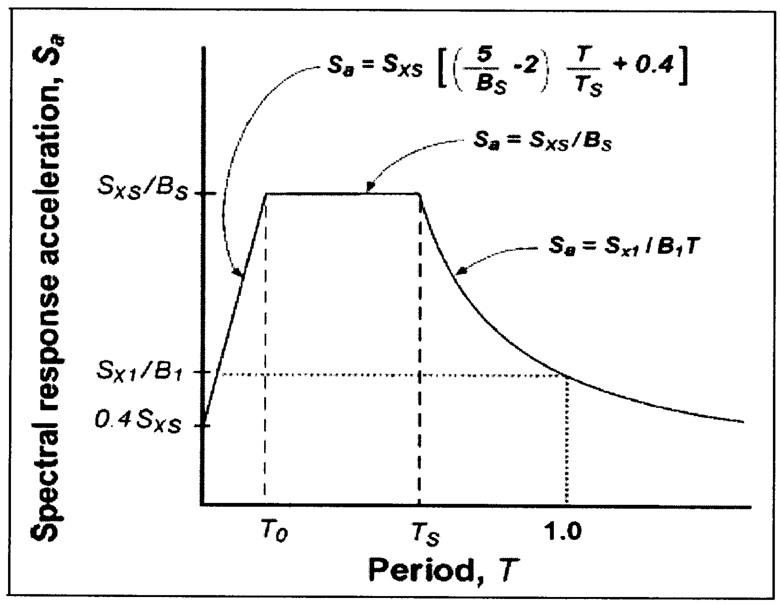

Figure 3 : Generic response spectrum (ATC 2000)

is created using the FEMA procedure. A damping ratio of 5% is assumed for all buildings. While this damping ratio may be an overestimation when considering steel or wood framed buildings, studies have shown that URM structures tend to have a lower-bound damping ratio around 5% (Griffith 2004).

2.2. *Rehabilitation Objective for Test Wall*

The first step to analyzing retrofitting strategies was to choose the type of building, its location, and the goals of the retrofit. This information would lead to a defined performance level and the design earthquakes. A fictitious two-story URM building in the Northeast United States, specifically the Boston area, was chosen for the analysis. The building is considered to house essential services, like a police station or fire house, meaning an enhanced rehabilitation objective is needed. Since these services are crucial after a seismic event, the building must be fully operational after moderate earthquakes and must remain safe during larger events. This decision leads to the "Immediate Occupancy" performance level for the BSE-1 and the "Life-Safety" performance level for the BSE-2.

With the location of the building known, general response spectrums could be created using ground acceleration values from the USGS hazard maps. Table 2 shows the acceleration parameters for Boston, MA. These acceleration parameters are then adjusted with respect to the geotechnical site class where the building is founded. For this case, site class D was used, which is common for shallow soils in the area. The modified values were then used to create two response spectra, one for BSE-1 and the other for BSE-2. These graphs can be found in Figure 4 and Figure 5. Calculations are found in Appendix A. The design loads for the building will later be calculated using these response spectra.

	BSE-1	**BSE-2**
S_s (%g)	0.090	0.280
S_1 (%g)	0.025	0.090

Table 2 : Acceleration parameters for Boston, MA

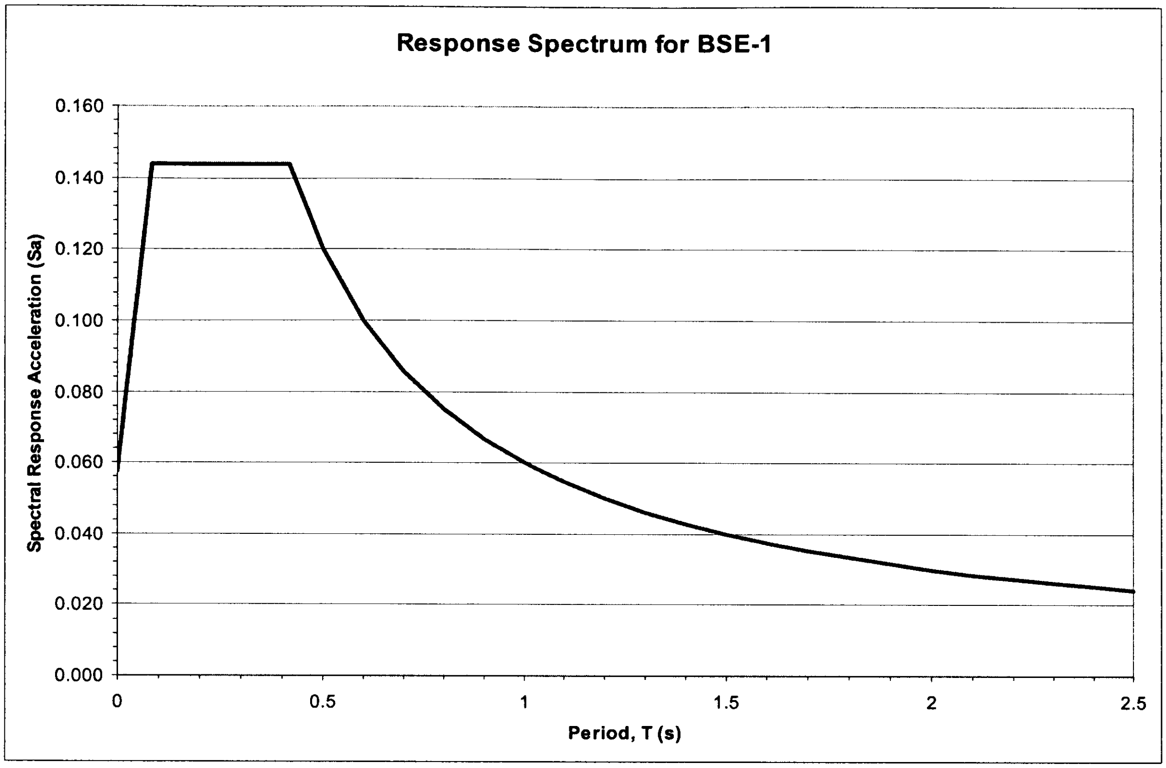

Figure 4 : Response spectrum for BSE-1 – 500 year return period (Boston, MA)

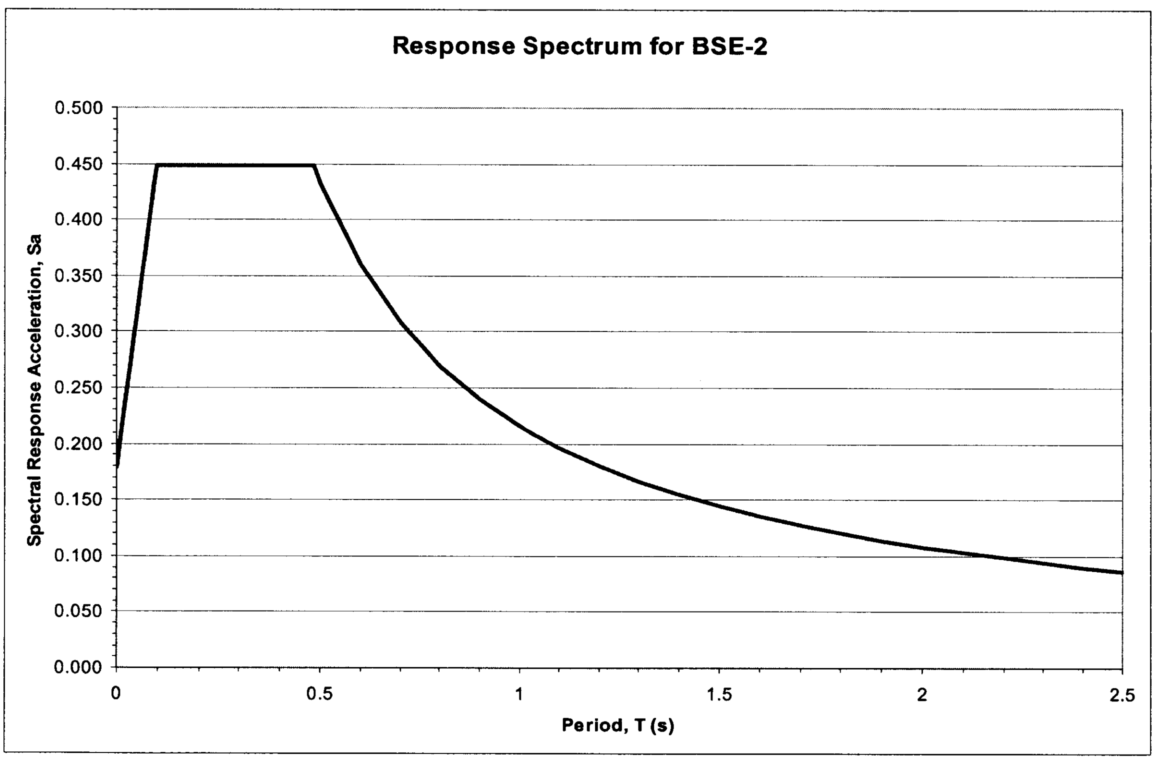

Figure 5 : Response spectrum for BSE-2 – 2500 year return period (Boston, MA)

3. Behavior of Unreinforced Masonry Buildings

A variety of factors makes predicting the response of unreinforced masonry buildings during seismic events complicated. Masonry is orthotropic with high strength in compression and often negligible strength in tension. It is also heterogeneous as it is composed of both masonry units (bricks or concrete) and mortar. The mechanical properties of the components vary greatly depending on the type of construction, location, and time of erection. The masonry tends to have a short elastic period before cracking and subsequent non-linear behavior.

In addition to difficult component properties, the global behavior of masonry buildings under lateral loading is not well understood. The failure modes typically depend on the type of construction, the amount and size of openings, the type of diaphragm (flexible vs. rigid), and how the vertical elements are connected to the diaphragms. It is also highly dependent on the direction of the ground motion, and whether it occurs parallel to the walls (in-plane) or perpendicular to the walls (out-of-plane). The behavior under large loads is highly non-linear and has been assumed to be brittle, although recent studies have shown that URM buildings can dissipate large amounts of energy after cracking through global rocking and sliding mechanisms (Griffith 2004). This makes analyzing and retrofitting URM structures very complex. The following sections explain in greater detail the behavior of URM buildings.

3.1. Typical URM Building Construction

Before discussing the typical failure modes of URM buildings, it is necessary to briefly mention the different types of construction methods that have been commonly used. This includes simple load bearing/shear wall construction, buildings with masonry facades, and moment frame structures with masonry infills. Reinforced masonry construction is not detailed in this report.

3.1.1. Load Bearing/Shear Walls

The simplest means of URM construction is a structure that is supported both vertically and laterally by unreinforced masonry walls. These buildings often have timber floor systems that create an essentially flexible diaphragm. It is also possible, however, to have steel floor framing with terra cotta or concrete slabs that create a rigid or semi-rigid diaphragm. The vertical masonry walls and columns were designed by experienced builders to carry the gravity loads. This was clearly adequate for normal gravity and wind load conditions since so many of these structures are still in use.

Many factors are involved when determining the lateral capacity of this type of construction under seismic loading. The in-plane behavior of a shear wall depends on the thickness of the wall and the amount and size of openings (windows and doors). Out-of-plane behavior is often governed by component properties and the effectiveness of the diaphragm-wall connection. The amount of vertical load on the walls will also have a major effect on behavior in both directions. Finally, global characteristics should be considered when analyzing behavior, as will be explained later.

3.1.2. Masonry Facades

Another common construction method uses masonry as a façade material. The façade is often clay bricks covering a reinforced CMU back-up wall. A gap is left between the back-up wall and the façade for drainage and various anchorages are used to support

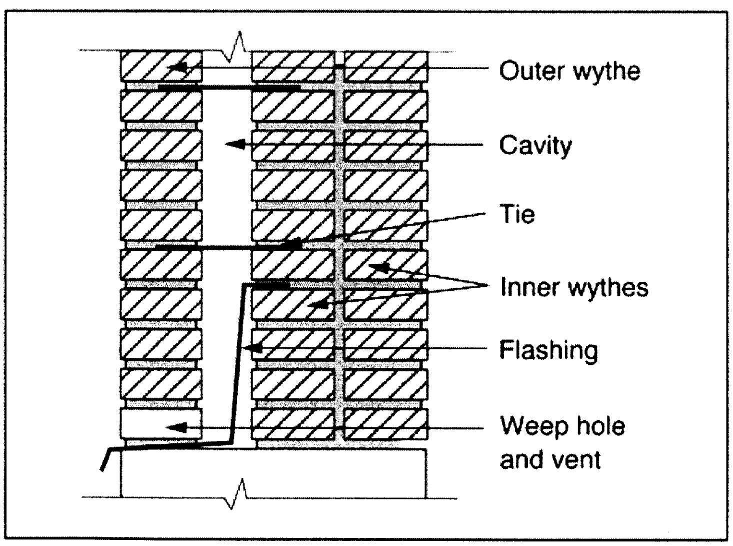

Figure 6 : Typical cavity wall section
http://irc.nrc-cnrc.gc.ca/images/ctus/ctu9/fig2-e.jpg

and hold back the bricks. This is typically called a cavity wall and can be seen in Figure 6. It is also common for the brick façade to cover a steel or concrete framed structure with anchorages at floor levels. In most cases, the façade is not a main structural component and is only designed to carry its self-weight and distribute the wind load that acts on it. Failure of the façade during a seismic event does not pose an immediate danger to building collapse; however, the falling bricks pose a large life-safety threat to pedestrians. The most important components in controlling this type of failure are the anchorages to the structural frame and the condition of the mortar.

3.1.3. Masonry Infills

A common building practice in many countries involves concrete or steel moment frames infilled with CMU units to help resist shear. This type of construction received a lot of attention in 1999 after many of these buildings collapsed during a large earthquake in Turkey resulting in a dramatic loss of life. This report, however, does not cover this type of structure.

3.2. Failure Modes

As previously mentioned, failure modes of masonry elements are difficult to predict and depend on a variety of factors. The direction of the loading, the amount of vertical stress, the number and size of openings, and the strength of the mortar relative to the blocks all play important roles in determining failure patterns. The following sections explain in detail the different types of failures for masonry elements and how they are dealt with in FEMA 356.

3.2.1. In-Plane Properties

In-plane shear walls are the main lateral load resisting element in most URM buildings. Their behavior depends largely on the size and location of the window and door openings. Walls are usually classified as either strong spandrel-weak pier (coupled) or strong pier-weak spandrel (uncoupled) depending on their geometry. In a strong spandrel-weak pier wall, the pier will fail first and limit the capacity of the wall. The piers tend to be considered as fixed-fixed elements and "coupled" since the stiff spandrel makes the piers resist lateral load like shear springs in parallel. With strong pier-weak spandrel walls, the spandrel fails first to limit the wall capacity. In this case, the piers are usually modeled as cantilever elements acting separately from each other since the weak spandrel does not provide enough stiffness for the piers to act together, hence "uncoupled."

Since the FEMA 356 analysis methods are based on allowable drifts, it is necessary to estimate the in-plane stiffness of the shear walls. The standard takes into account both flexural and shear deformations to calculate stiffness. For coupled shear walls, each pier is modeled as separate shear springs which are then added together to determine the stiffness of each floor level. Piers of uncoupled shear walls are modeled independently. The standard recommends two separate equations for stiffness depending on if the wall is coupled or uncoupled (ATC 2000):

$$\text{Coupled: } k = \frac{1}{\dfrac{h_{eff}^3}{12E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \qquad \textbf{(Eq. 3-1)}$$

$$\text{Uncoupled: } k = \frac{1}{\dfrac{h_{eff}^3}{3E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \qquad \textbf{(Eq. 3-2)}$$

Where:

h_{eff} = Effective wall height. See Figure 7.

A_v = Shear area

I_g = Uncracked moment of inertia

E_m = Masonry elastic modulus

G_m = Masonry shear modulus

The modulus values are either determined through testing or by using default values specified in the standard. The wall is considered homogenous and the effective wall height is defined as the height of the adjacent openings. Equation 3-1 considers the piers to be perfectly fixed-fixed while equation 3-2 assumes the piers are fixed-free. The design engineer needs to keep in mind that this is an idealization.

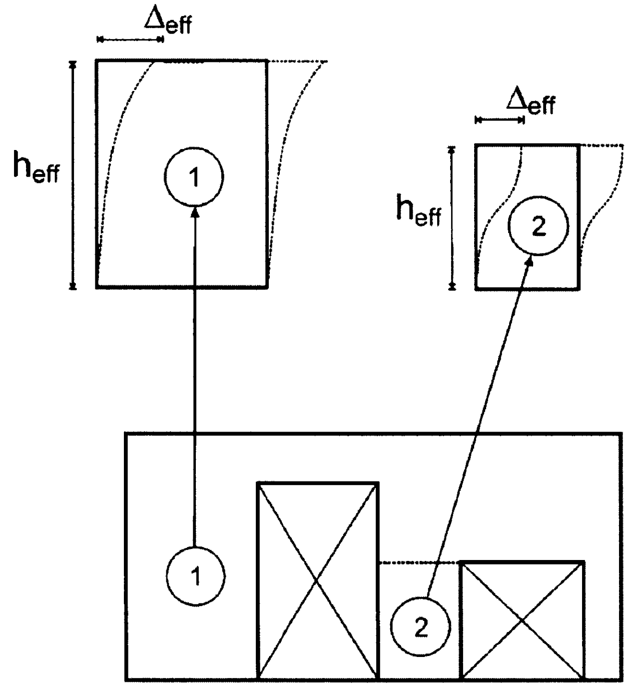

Figure 7 : Illustration of effective pier heights and displacements (ATC 2000)

To calculate the strength of a wall in-plane, it is necessary to consider all possible failure modes. For performance-based design, it is also important to recognize if the failure mode is ductile (deformation-controlled) or brittle (force-controlled). The following sections will explain the different types of in-plane failures and how they are calculated in FEMA 356.

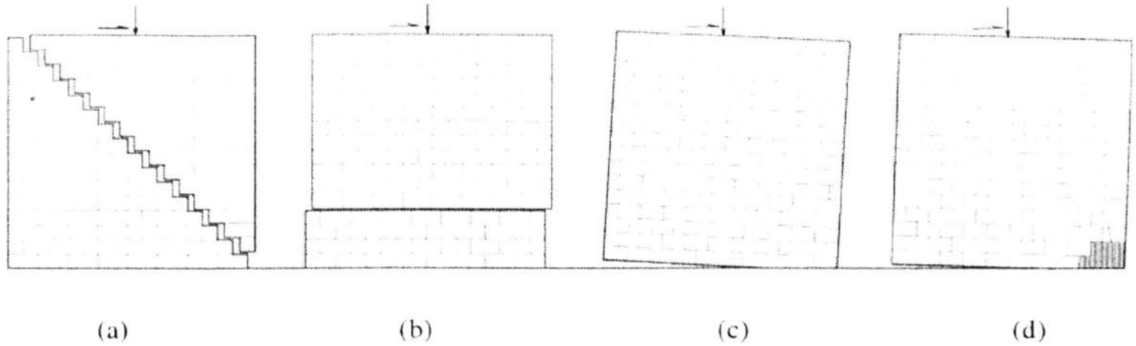

Figure 8 : Typical failure modes of a URM wall. a) Diagonal tension b) Bed-joint sliding c) Rocking d) Toe crushing (El Gawady 2007)

3.2.1.1. Deformation-Controlled Failures

Deformation-controlled failures are more ductile than force controlled and lead to less risk of sudden collapse. A deformation-controlled failure has the ability to absorb considerable amounts of energy after yield or cracking and therefore leads to a much safer design. For in-plane masonry shear walls, rocking and bed-joint sliding failures are both considered deformation-controlled modes. These two failure modes can be seen in Figure 8 (b) and (c). A rocking failure begins with a horizontal crack at the base of a wall or pier and subsequent rocking around the vertical axis of that element. This behavior can absorb significant energy and can be considered nonlinear but elastic since after the lateral load is gone, the element tends to return to its original position. A bed-joint sliding failure also begins with a horizontal crack but is then followed by horizontal movement along that crack. This behavior dissipates energy through friction at the cracked surface.

FEMA 356 provides an equation to determine the lateral strength of a pier for each of these failure modes. The two equations are as follows (ATC 2000):

$$\text{Bed-Joint Sliding: } Q_{CE} = V_{bjs} = v_{me} A_n \quad \textbf{(Eq. 3-3)}$$

$$\text{Rocking: } Q_{CE} = V_r = 0.9 \alpha P_E \left(\frac{L}{h_{eff}} \right) \quad \textbf{(Eq. 3-4)}$$

Where:

A_n = Area of net mortared/grouted section

h_{eff} = Effective pier height

L = Length of wall or pier

P_E = Expected axial compressive force due to gravity loads

v_{me} = Expected bed-joint sliding shear strength

V_{bjs} = Exp. shear strength of wall/pier based on bed-joint sliding shear strength

V_r = Strength of wall or pier based on rocking

α = Factor equal to 0.5 for fixed-free pier, 1.0 for fixed-fixed

Since the failures are ductile and do not pose a threat to collapse, the expected material properties are used (hence the subscript "E"). These properties are the mean results from component testing. It should be noted that bed-joint sliding capacity is only dependent on the shear strength of the bed joint, while rocking capacity is only dependent on the vertical load on the pier (P_E) and its aspect ratio (L/h_{eff}).

3.2.1.2. Force-Controlled Failures

Force-controlled failures tend to be brittle in nature and can often lead to sudden collapse. Masonry failures under vertical loads are always force-controlled as the blocks crush with little warning. For in-plane lateral loads on URM shear walls, the two main examples of a force-controlled failure are toe crushing and diagonal tension. These two mechanisms are illustrated in Figure 8 (a) and (d). Toe crushing happens when the base of a pier or wall crushes under the combined stress of a vertical load and overturning moment due to the lateral force. This type of failure tends to happen in elements with high initial vertical loads and high aspect ratios. The second type of force-controlled failure is diagonal tension cracks. This often happens in piers or spandrels with low aspect ratios and high shear loads. Large diagonal cracks develop between openings and usually start at the corner of an opening due to stress concentrations.

FEMA 356 provides an equation to determine the lateral strength of a shear wall for each of these failure modes. These equations are (ATC 2000):

$$\text{Diagonal Tension: } Q_{CL} = V_{dt} = f'_{dt} A_n \left(\frac{L}{h_{eff}} \right) \sqrt{1 + \frac{f_a}{f'_{dt}}} \quad \textbf{(Eq. 3-5)}$$

$$\text{Toe Crushing: } Q_{CL} = V_{tc} = \alpha P_L \left(\frac{L}{h_{eff}} \right) \left(1 - \frac{f_a}{0.7 f'_m} \right) \quad \textbf{(Eq. 3-6)}$$

Where:

f_a = Axial compressive stress due to gravity loads

f'_{dt} = Lower bound masonry diagonal tension strength

f'_m = Lower bound masonry compressive strength

P_L = Lower bound axial compressive force due to gravity loads

V_{dt} = Lower bound shear strength based on diagonal tension stress for wall/pier

V_{tc} = Lower bound shear strength based on toe compressive stress for wall/pier

To compensate for the potential for brittle failure, all material properties are specified as "lower bound" properties (hence the subscript "L"). For example, the lower bound masonry compressive strength is the mean compressive strength found through testing minus one standard deviation.

3.2.2. Out-of-Plane Failures

Out-of-plane failures can be very dangerous during seismic events for both load bearing/shear walls and facades. A wall that has failed out-of-plane clearly loses its in-plane stiffness and ability to hold vertical loads. Also, falling bricks from facades pose a great threat to people below. These failures tend to occur suddenly with little energy dissipation. Out-of-plane failures are largely dependent on the amount of rotation the wall experiences, which means factors like in-plane stiffness, type of diaphragm, diaphragm connection, and amount of vertical load have large effects on behavior.

In the FEMA 356 standard, the stiffness of out-of-plane walls is neglected in calculations. The linear and nonlinear procedures that are outlined in a later section are also not applicable when analyzing a URM wall out-of-plane. Instead, the standard recommends treating the walls as isolated elements spanning between floor levels or vertical columns. These elements are then subjected to out-of-plane inertia loads to determine the expected amount of cracking and if the wall will remain stable. The amount of cracking allowed in an out-of-plane wall depends on the desired performance level (ATC 2000).

3.2.3. Connection Failure

The effectiveness of the wall-diaphragm and façade-structure connections is critical to the behavior of a URM building. These connections are often deficient in old buildings leading to a weak link in the structural load path that could lead to premature failure. Façade-to-structure connections are often problematic since the façade will inevitably leak, leading to corrosion of the connecting elements. If these relieving angles and anchors are not replaced, there will be nothing restraining the bricks during a seismic event. As mentioned earlier, falling bricks is a leading cause of injury and death during earthquakes. Repairing façade connections is a relatively easy way to enhance building seismic performance.

3.3. Test Wall

The next step to compare retrofitting techniques using the FEMA 356 standard was to create a structure for analysis. As previously mentioned, a fictitious two-story brick URM building in Boston, MA, is being used for calculations. The building has 12" thick (three standard wythes) load bearing/shear walls that act as the main lateral force resisting elements. It also has a flexible wood diaphragm, which represents common construction practices in the area. An elevation of the front load bearing/shear wall of the building can be seen in Figure 9. Due to the relative thickness of the spandrel with respect to the piers, this wall is classified as strong spandrel-weak pier (coupled wall). This means that the capacity of the wall will be limited by the piers. The flexible diaphragm allows for this wall to be analyzed without considering the other in-plane lateral resisting elements, for example the rear wall. The weight and mass of the building is distributed through tributary area.

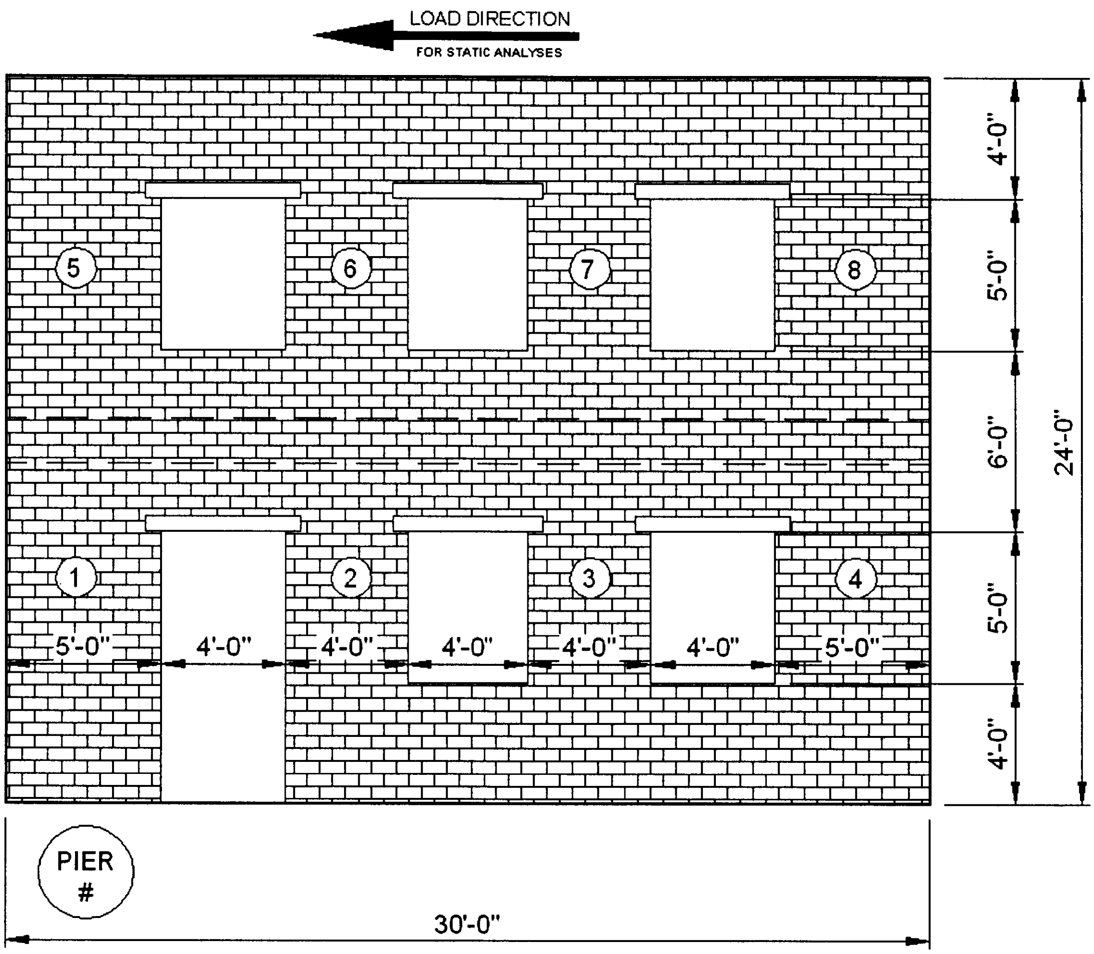

Figure 9 : Front elevation of test wall

3.3.1. Wall Properties

Analysis of the unretrofitted wall began by finding the component properties. Since the building is not real, material testing is clearly not an option. Default masonry properties were used instead as specified by FEMA 356. Table 3 shows the lower-bound values for different types of masonry. The test structure was deemed in "fair" condition with a common running bond lay-up. Expected material properties are calculated from these lower-bound properties by multiplying by a factor of 1.3.

	Masonry Condition[1]		
Property	**Good**	**Fair**	**Poor**
Compressive Strength (f'_m)	900 psi	600 psi	300 psi
Elastic Modulus in Compression	$550f'_m$	$550f'_m$	$550f'_m$
Flexural Tensile Strength[2]	20 psi	10 psi	0
Shear Strength[3]			
Masonry with a running bond lay-up	27 psi	20 psi	13 psi
Fully grouted masonry with a lay-up other than running bond	27 psi	20 psi	13 psi
Partially grouted or ungrouted masonry with a lay-up other than running bond	11 psi	8 psi	5 psi

Table 3 : Default lower-bound masonry properties (ATC 2000)

Using the component properties and the geometry of the wall, it was possible to calculate both the lower-bound and expected stiffness of each story. Each pier (eight in total) was modeled as a fixed-fixed element (coupled wall) with a stiffness calculated from equation 3-1. The stiffness of the four piers at each story can be added like springs in parallel and then each story is added like springs in series. The model can be seen in Figure 10 and the stiffness values are found in Table 4. Calculations are in Appendix B.

	Pier One	**Pier Two**	**Pier Three**	**Pier Four**	**Pier Five**	**Pier Six**	**Pier Seven**	**Pier Eight**
Lower-Bound Stiffness (k/in)	383	780	780	1131	1131	780	780	1131
Expected Stiffness (k/in)	498	1014	1014	1471	1471	1014	1014	1471

Table 4 : Stiffness properties for each pier. See Figure 9 for pier designation.

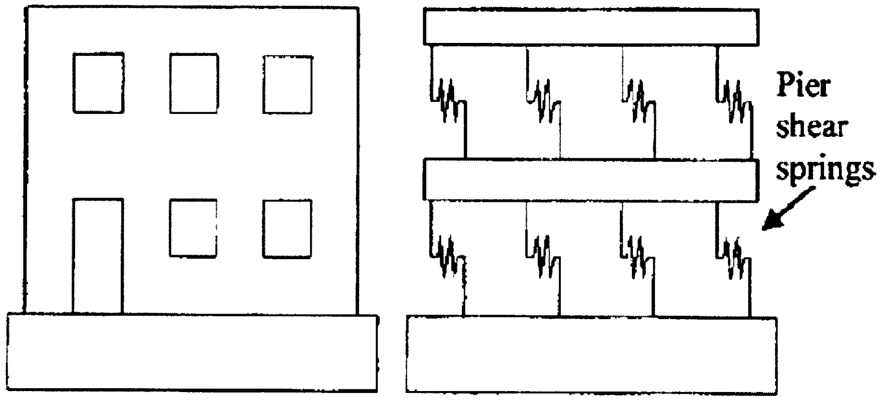

Figure 10 : Model of coupled wall with piers as shear springs (Yi 2006)

3.3.2. Wall Strength

The next step of the analysis was to determine the lateral strength of the wall and the limiting failure mechanism. The axial load on the wall was first calculated using 120 pcf as the unit weight of masonry and assuming 25 psf of both dead load and live load at each floor. Fifteen feet of tributary area was attributed to the wall for both stories. Using expected material properties for equations 3-3 and 3-4 and lower bound material properties for equations 3-5 and 3-6, the limiting failure mode and subsequent strength was determined for each of the eight piers. The calculations can be found in Appendix B and the results in Table 5.

		Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8
Q_{CE} (k)	**Bed-Joint Sliding**	16.10	12.88	12.88	16.10	10.40	8.32	8.32	10.40
	Rocking	9.08	10.45	10.45	16.34	6.09	3.90	3.90	6.09
Q_{CL} (k)	**Diagonal Tension**	14.10	13.48	13.48	21.07	12.18	7.79	7.79	12.18
	Toe Crushing	*7.40*	*7.07*	*7.07*	*11.04*	*3.80*	*2.43*	*2.43*	*3.80*

Table 5 : Pier strength values for each failure mode

It is important to note that all of the piers in this modestly loaded structure are limited by force-controlled failure mechanisms, more specifically toe crushing. This is clearly not good for the designing engineer as it means that the building will likely fail suddenly during a seismic event. This fact will be returned to later when discussing the linear and nonlinear analysis procedures.

4. Analysis Methods

The analysis of unreinforced masonry structures can be conducted in many ways. Most traditional methods have a force-based approach in which the strength capacity of a URM structure is limited to the onset of cracking. Designers believed that the masonry would always act in a brittle manner beyond this point and therefore reserve capacity could not be depended on. Many recent studies, however, have shown that URM structures can indeed dissipate significant energy beyond the elastic stage while still maintaining structural integrity (Griffith 2004). Most of this energy is dissipated though rocking and sliding mechanisms that do not become unstable until a certain limiting displacement is reached. This means that a performance-based design can be utilized for URM buildings with more accurate results than the traditional methods. FEMA 356 is an example of a code that uses performance-based principles in its analyses.

As previously mentioned, analyzing URM structures can become difficult due to the orthotropic nature of the components, the heterogeneous nature of elements, and the task of predicting crack patterns and nonlinear behavior. On top of this, a complete analysis should take into account P-delta effects, soil-structure interaction, multi-directional and vertical seismic effects, horizontal torsion, overturning moments, diaphragm action, and the effects of non-structural components (ATC 2000).

FEMA 356 specifies four procedures that can be used to analyze an existing building. These are (ATC 2000):

- Linear Static Procedure
- Linear Dynamic Procedure
- Nonlinear Static Procedure

- Nonlinear Dynamic Procedure

These analysis methods have varying degrees of complexity and give results with varying degrees of accuracy. The correct procedure to choose for a project depends on the accuracy needed for that particular building. The following sections explain the FEMA 356 design procedures in more detail.

4.1. Elastic Analyses

Elastic analyses assume linear behavior during a seismic event. This is clearly a stretch when considering URM buildings but the idea is to provide a quick estimate for the engineer to give him an idea as to what he is dealing with. FEMA 356 specifies two acceptable elastic analyses: the linear static procedure and the linear dynamic procedure. They are detailed here.

4.1.1. Linear Static Procedure (LSP)

The LSP uses a linearly elastic, static analysis to find the magnitude and distribution of seismic design forces, the corresponding internal forces, and the displacements. Assuming linear elastic stiffness and equivalent viscous damping values, a "pseudo-lateral load" is calculated from an empirical formula. The intention is that when the "pseudo-lateral load" is applied to the elastic model of the structure, it results in a displacement approximating the actual movement to be expected. If the building does indeed behave elastically during the seismic event, the calculated internal forces will be close to the actual forces. If the building behaves inelastically, as will probably be the case for URM buildings, the calculated internal forces will be greater than the actual forces (ATC 2000).

The first step in the LSP is to approximate the fundamental period of the structure. This can be done analytically, empirically, or using approximate equations according to the standard. An analytical model should only be used on buildings with well-defined framing systems and behavior. FEMA 356 specifies formulas to use for the empirical and approximate methods. A special equation is given to approximate the fundamental period for URM buildings with flexible diaphragms (ATC 2000):

$$T = (0.078\Delta_d)^{0.5} \quad \textbf{(Eq. 4-1)}$$

Where Δ_d is the maximum in-plane diaphragm displacement (inches). This equation assumes that the in-plane deflection of the masonry walls is negligible compared to that of the flexible diaphragm.

Once the period is determined, the next step is to calculate the pseudo-lateral load from the following equation (ATC 2000):

$$V = C_1 C_2 C_3 C_m S_a W \quad \textbf{(Eq. 4-2)}$$

Where:

V = Pseudo lateral load

C_1 = Modification factor relating expected inelastic displacements to the calculated elastic response.

C_2 = Modification factor for stiffness degradation and strength deterioration (1.0 for LSP)

C_3 = Modification factor to account for increased displacements due to P-Delta effects

C_m = Effective mass factor to account for higher mode mass participation (1.0 for URM)

S_a = Response spectrum acceleration at fundamental period and damping ratio of building (estimated at 5%)

W = Effective weight of the building

For URM buildings with flexible diaphragms and a fundamental period estimated from equation 4-1, the pseudo-lateral load is calculated for each span of the building and for each floor. It is then distributed to the vertical seismic-resisting elements (walls) according to tributary area. Forces in the diaphragm can then be calculated using these results.

The forces for each story determined from the pseudo lateral load are then compared to the story strengths to determine if they are acceptable. For elements that are limited by force-controlled failure modes, the governing equation is (ATC 2000):

$$\kappa Q_{CL} \geq Q_{UF} \quad \textbf{(Eq. 4-3)}$$

Where:

κ = Knowledge factor

Q_{CL} = Lower-bound strength of component

Q_{UF} = Force-controlled design action

The knowledge factor is obtained from FEMA 356 and depends on both the method used to determine component properties (testing vs. default) and the desired performance level. Table 6 shows the knowledge factor for different scenarios.

	Level of Knowledge							
Data	**Minimum**		**Usual**				**Comprehensive**	
Rehabilitation Objective	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP, LDP		All		All		All	
Testing	No Tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawings	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Construction Documents	Or Equivalent
Condition Assessment	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive
Material Properties	From Drawings or Default Values	From Default Values	From Drawings and Tests	From Usual Tests	From Drawings and Tests	From Usual Tests	From Documents and Tests	From Compre-hensive Tests
Knowledge Factor (κ)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

Table 6 : Knowledge factor according to acquired data (ATC 2000)

For elements that are limited by deformation-controlled mechanisms, the governing equation also takes into account the ability of the wall to resist lateral loading after yield. For these piers, the equation is as follows (ATC 2000):

$$m\kappa Q_{CE} \geq Q_{UD} \quad \textbf{(Eq. 4-4)}$$

Where:

m = Modification factor to account for expected ductility of failure mode

Q_{CE} = Expected strength of component

Q_{UD} = Deformation-controlled design action

The "m" factor is obtained from FEMA 356 and again depends on the failure mode (only for deformation-controlled mechanisms) and the performance level of the building. Table 7 shows the factors for URM walls according to limit state and performance level.

Limiting Behavioral Mode	*m*-factors				
	Performance Level				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Bed-Joint Sliding	1	3	4	6	8
Rocking	$1.5h_{eff}/L$ (not less than 1)	$3h_{eff}/L$ (not less than 1.5)	$4h_{eff}/L$ (not less than 2)	$6h_{eff}/L$ (not less than 3)	$8h_{eff}/L$ (not less than 4)

Table 7 : m-factors for URM walls (ATC 2000)

It should be noted that the LSP is not applicable for all buildings. The standard designates that the procedure should not be used for buildings with a fundamental period greater than 3.5 times T_s or for buildings with significant structural or geometrical irregularities. For these structures, the linear dynamic or nonlinear procedures should be used.

4.1.2. Linear Dynamic Procedure (LDP)

The linear dynamic procedure again assumes linear elastic stiffness and equivalent viscous damping values to model a structure. A modal spectral analysis that is not modified for nonlinear response is then used to find internal displacements and forces. As in the LSP, the idea is to approximate the actual displacements expected during an earthquake but produce conservative force values.

The first step in the LDP is to characterize the ground motion. This can either be done through a response spectrum or a more in depth ground acceleration time history analysis. For the response spectrum analysis, enough modes need to be included to total 90% of the participating mass of the building in each direction. Modal responses are then combined using the "square root sum of squares" rule or the "complete quadratic combination" rule to determine peak member forces, displacements, story shears, and base reactions. The time-history method requires a time-step by time-step evaluation of a building response using recorded ground motions (ATC 2000).

Forces and deformations obtained using the LDP should be modified using the C_1, C_2, and C_3 factors defined in the previous section. The design forces are then compared to the expected or lower-bound wall strengths using the same acceptance criteria as in the linear static procedure.

4.2. Inelastic Analyses

Inelastic analyses take into account the nonlinear behavior that a structure undergoes during a seismic event. This is much more accurate for URM buildings that are sure to exhibit this type of behavior post-cracking. FEMA 356 specifies two acceptable inelastic analyses: the nonlinear static procedure and the nonlinear dynamic procedure. They are detailed here.

4.2.1. Nonlinear Static Procedure (NSP)

The basis of the NSP is to incorporate the nonlinear load-deformation properties of a building into a mathematical model and then add incremental loading to that model until a target displacement is reached. This is sometimes called a "static pushover analysis." Since the nonlinear characteristics of the components are included in the model, the calculated forces at the target displacement should be accurate unlike in the linear procedures. The NSP model should include gravity loads on the components, should be discretized, and should include all primary and secondary lateral force resisting elements. A simplified version of the NSP is also allowed by FEMA 356 in which only primary elements are considered and the force-deformation properties of those elements are modeled as bilinear (ATC 2000).

The first step in the procedure is to designate a control node for the building. The standard states that this node should be at the center of mass at the roof of the structure. Lateral loads are then applied at diaphragm levels in proportion to the inertia forces in the structure. Two distributions should be considered for all NSP analyses: one that is proportional to the fundamental mode of the building or a story shear distribution and one that is either a uniform distribution or an adaptive load distribution that changes for nonlinear properties of the yielded structure.

The next step for the NSP is to generate nonlinear force-deformation relationships for each of the pier elements. A generalized force-deformation relationship is given in the standard and can be seen in Figure 11. These relationships are then used to develop a global force-displacement relationship for the building. An idealized bilinear curve is then fit over the actual building curve with the slope of the first section equal to an effective lateral stiffness, which is taken as the secant stiffness at 60% of the effective yield strength of the structure. This portion lasts until the effective yield strength of the building is reached. The second line has a slope of α which is a fraction of the effective lateral stiffness. This line ends when a target displacement is reached (ATC 2000).

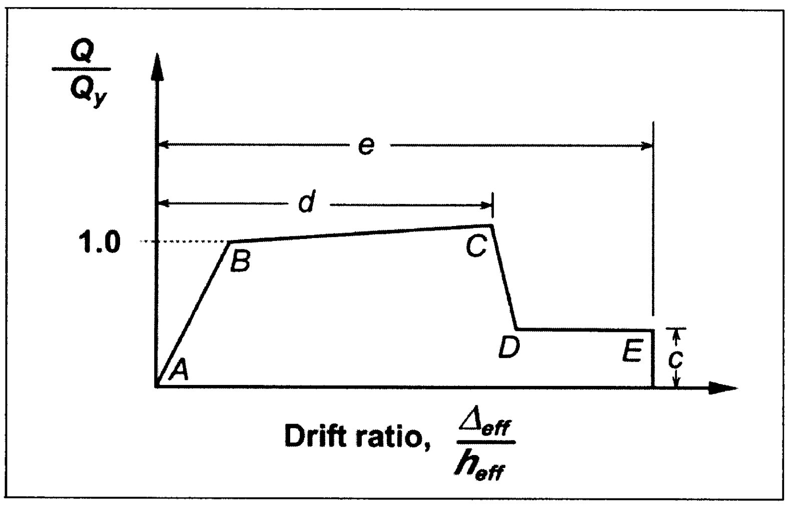

Figure 11 : Generalized force-deformation relationship for deformation controlled masonry elements (ATC 2000)

Once the idealized force-displacement relationship is determined, an effective fundamental period must be calculated for each orthogonal direction. The equation for this is as follows (ATC 2000):

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \quad \textbf{(Eq. 4-5)}$$

Where:

T_i = Elastic fundamental period calculated by elastic dynamic analysis

K_i = Elastic lateral stiffness

K_e = Effective lateral stiffness

FEMA 356 specifies an empirical formula to calculate the target displacement, δ_t. For URM buildings with flexible diaphragms, this target displacement must be calculated for each line of vertical seismic resisting elements with masses calculated by tributary area. The equation is (ATC 2000):

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \quad \textbf{(Eq. 4-6)}$$

Where:

C_0 = Factor to relate spectral displacement of equivalent SDOF system to the control node of the actual MDOF building

C_1, C_2, C_3 = Same factors as LSP

T_e = Effective fundamental period

S_a = Response spectrum acceleration

G = Acceleration of gravity

The forces and deformations obtained through the analyses are then modified to consider the effects of horizontal torsion and then compared to the acceptance criteria found in the standard.

4.2.2. Nonlinear Dynamic Procedure (NDP)

The nonlinear dynamic procedure involves creating a finite element model of a building that incorporates the nonlinear load-deformation properties of individual components and then subjecting that model to a ground motion time history. The procedure is similar to that of the NSP with the exception that time histories are used instead of spectral accelerations.

4.3. *Test Structure Analysis*

The two-story test wall described in an earlier section was analyzed using the linear static, linear dynamic and nonlinear static procedures outlined above. The results of each were then compared to determine if the structure could resist the design seismic forces. A nonlinear dynamic analysis was not run because of the time required to create such a model and the unpredictable nature of masonry makes the task very difficult.

4.3.1. Results of Linear Static Procedure

A linear static analysis was conducted on the test wall according to the procedure outlined in FEMA 356. The calculations can be found in Appendix C. Since the building has a flexible diaphragm, the approximate period of the building was calculated from equation 4-1. The diaphragm was assumed to be a one inch thick wooden floor system with a modulus of elasticity of 1500 psi and spanning 30 feet. The approximate maximum deflection of the diaphragm under the applied inertia loads (assumed 25 psf) was then calculated and found to be around 2.8 inches. The use of the equation 4-1 is justified as the shear wall deflection is sure to be negligible compared to this value.

Using the approximate period and the response spectrums determined earlier, the pseudo-lateral load could be calculated for both the BSE-1 and BSE-2 earthquakes from equation 4-2. More detailed explanations on the coefficients can be found in the calculations in appendix C. The pseudo lateral load was then applied to each story and compared to the acceptance criteria for the wall according to equation 4-3. The results can be found in Table 8. Since toe crushing, a force controlled mechanism, limits the strength of the wall, no "m" ductility factors were used. For the BSE -1, the table shows that the design forces exceed the strength values ($Q_{UF} > \kappa Q_{CL}$) on the second floor and are near the limit on the first floor. For the BSE-2, the design values far exceed the strength of the wall. The linear static procedure shows that the building needs to be retrofitted to meet the performance requirements.

	BSE-1		BSE-2	
	First Floor	Second Floor	First Floor	Second Floor
Q_{UF} (k)	21.15	10.58	45.73	22.87
κQ_{CL} (k)	24.43	9.34	24.43	9.34

Table 8 : Results from linear static analysis

4.3.2. Results of Linear Dynamic Procedure

A linear dynamic analysis was also performed on the test wall. The wall was modeled as a two degree-of-freedom shear beam. The modal properties of the shear wall were found using the *MotionLAB* program "Shear Beam" provided by Jerome Connor. The same stiffness and mass properties calculated during the linear static procedure were inputted into the program as well as equivalent viscous damping values that resulted in 5% damping on the structure, as recommended by FEMA 356. The program then outputted the shape, period, and participation factor for each of the two modes. Using these results, the displacement and subsequent internal forces were calculated for each story. The spectral accelerations were found from the response spectrums plotted in Figure 4 and Figure 5. The calculations for this analysis can be found in Appendix D.

	BSE-1		BSE-2	
	First Floor	Second Floor	First Floor	Second Floor
Q_{UF} (k)	15.8	5.1	38.0	17.8
κQ_{CL} (k)	24.43	9.34	24.43	9.34

Table 9 : Results from linear dynamic analysis

Table 9 shows the results from the linear dynamic analysis. This analysis, which provides a better description of the actual behavior of the wall, shows that the building meets the acceptance criteria for the BSE-1 ($Q_{UF} < \kappa Q_{CL}$) but does not meet the criteria for the BSE-2. All of the design forces calculated are less than those from the linear static procedure as the LSP must provide a greater level of conservativeness due to the many assumptions it makes. However, the assumption that the system remains elastic, which is certainly a stretch, means the results for the LDP are still questionable.

4.3.3. Results of Nonlinear Static Procedure

A nonlinear static procedure (static pushover) analysis was conducted following the linear procedures. All calculations and graphs can be found in Appendix E. Force-deformation curves were created for each of the eight piers considering their failure modes. Since all piers were limited by toe crushing – an extremely brittle failure – the curves simply consisted of straight lines representing the elastic phase that ended at the point of toe crushing with no residual strength. These curves were combined to determine the stiffness of each floor and then each floor was added as springs in series.

Since the elements are force controlled, the wall was subjected to incrementally increasing lateral load according to two distributions. The first distribution considered an equal lateral force at each floor. The second distribution was in the shape of the fundamental mode as determined from the linear dynamic procedure. Once the force in a pier element had reached its limit, it was assumed to have failed and its stiffness fell to zero. The load was increased until all piers in one story had failed. The total displacement of the control node (at the second story) was then plotted against the total base shear to determine the force-displacement relationship of the structure and the effective stiffness. This relationship can be seen in Figure 12.

Using the performance level, the building period, the effective yield strength of the building, and the response spectrums, the target displacement for the building was calculated for both the BSE-1 and BSE-2 according to equation 4-6. The results can be found in Table 10. Due to the brittle nature of toe crushing, the building does not reach the target displacements for either of the design earthquakes. The shear in the second story causes a failure that effectively ends the analysis. This again shows that the building should be retrofitted to meet the seismic criteria.

	BSE-1	BSE-2
Target Displacement (in)	0.016	0.049
Second Story Shear (k)	9.5	9.5
Allowable Second Story Shear (k)	9.3	9.3

Table 10 : Target Displacements and shears for nonlinear static procedure

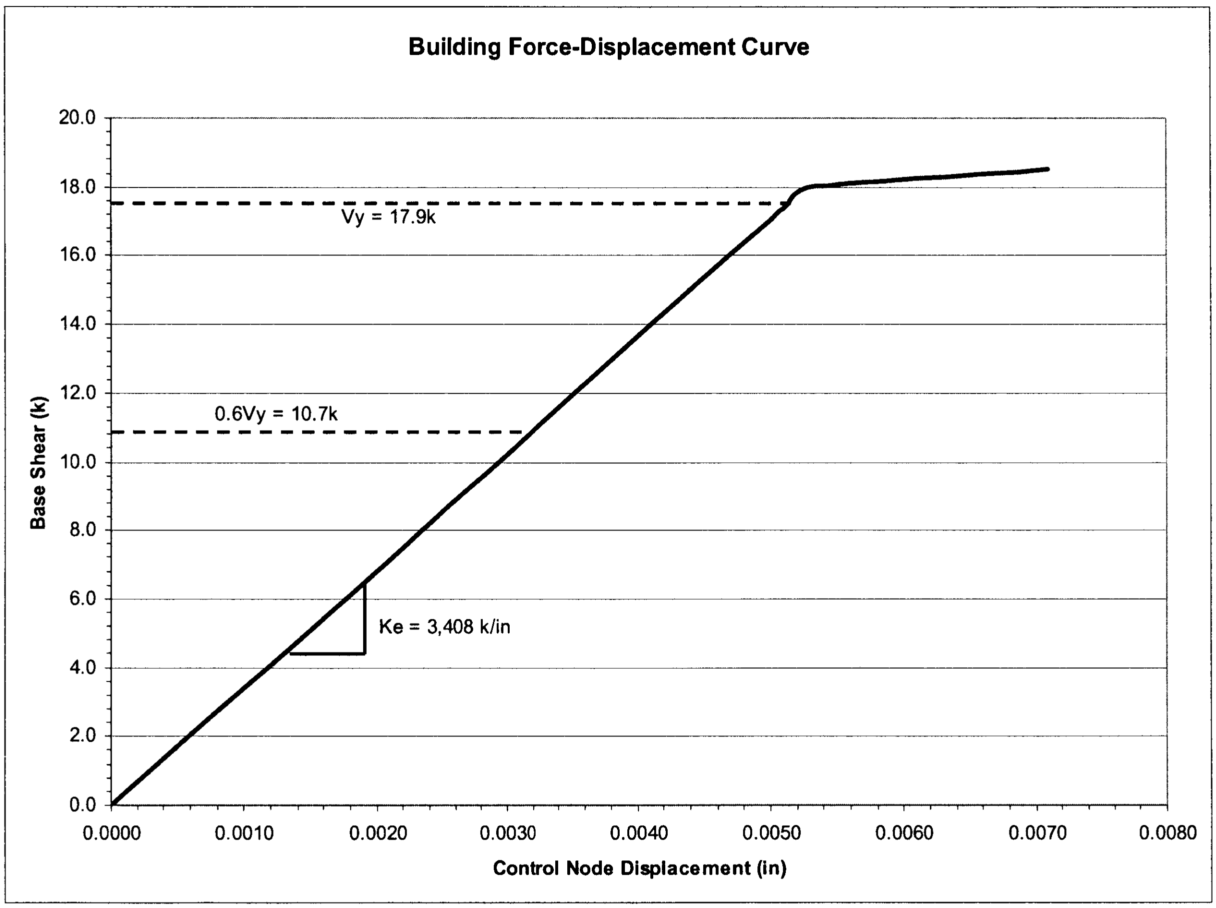

Figure 12: Building force-deformation curve from nonlinear static procedure

5. Analysis Modifications

Since FEMA 356 was published in November of 2000, a variety of experiments have been conducted to test the performance-based design procedures recommended in the standard. This is especially true for the masonry section since the behavior of masonry buildings has never been clearly understood. The methodology and equations are based on experiments that have been conducted on isolated masonry elements, like a pier or a wall, and not on full masonry buildings. It is not clear if extrapolating these tests to analyze a full building is accurate. The Mid-America Earthquake (MAE) Center recently conducted a research program to test the procedures of FEMA 356 that included the full-scale testing of a two-story masonry structure (Yi 2006). The experimental results were then compared with those found through the standard and changes were recommended. The following sections discuss some of these recommendations and how they could be applied to analyzing the test wall.

5.1. MAE Analysis Recommendations

The main focus of the MAE program was to examine the global behavior of masonry buildings and how it would affect the analyses. They did, however, recommend a few more fundamental changes to the FEMA 356 procedure. The most significant of these recommendations is the elimination of toe crushing as a separate failure mode. Due to the fact that toe crushing is almost always preceded by rocking, it is instead proposed that toe crushing become the ultimate condition of the rocking mechanism (Yi 2006). This led to changes in the force-drift relationship for deformation controlled masonry components that was explained in an earlier section. The sharp drops were eliminated from the graph and replaced with sloping lines. The residual strength plateau was also changed from 60% of the building yield strength to the toe crushing capacity of the pier. A schematic of the changes can be seen in Figure 13.

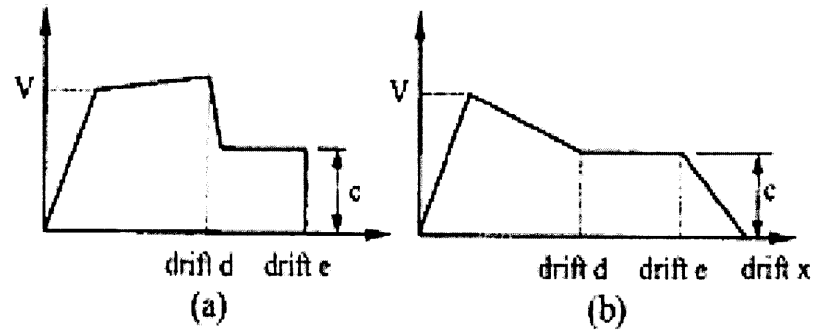

Figure 13 : (a) Force-drift relationship provided by FEMA 356 (b) Modified force-drift relationship (Yi 2006)

Another basic change recommended by the MAE was the definition of the effective pier height (h_{eff}). FEMA 356 defines effective height of a pier as the height of adjacent openings. The proposed changes define the effective height as the distance over which a compression strut will likely develop between two openings (Yi 2006). Figure 14 shows how this definition would be employed. It should be noted that with this definition, the effective height of a pier changes with the direction of loading.

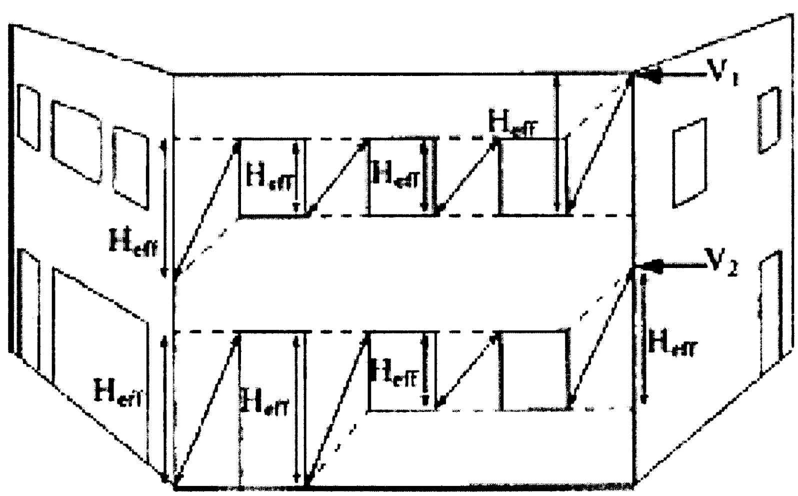

Figure 14 : Modified definition of effective pier height (Yi 2006)

As for the global behavior of a masonry structure, the MAE suggested three effects to include during analysis. These are overturning moment, global rocking, and flange participation. The overturning moment is important for buildings with the height equal to or greater than the base so that overturning moments have a large effect on vertical stress in the piers. The increase in vertical load caused by the moment could cause an exterior pier to fail in a brittle manner before expected. This effect need only be accounted for in coupled walls. Global rocking is a failure mechanism that results from a building with a large overturning moment. Again, this should only be accounted for in structures with coupled walls. Finally, flange effects occur when a portion of the out-of-plane walls helps the in-plane walls resist lateral loading. This can occur when a rocking mechanism forms in an exterior pier and a segment of the out-of-plane wall is lifted or compressed. This effect can significantly increase a structure's global resistance to lateral loads and its energy dissipation capacity (Yi 2006).

5.2. *Modified Test Structure Analysis*

With the MAE recommendations in mind, a modified analysis was conducted on the test wall. Since the global characteristic of the test building are not known, flange effects were not included in the modified analysis. Overturning moment was also neglected since the building has a relatively low height-base ratio. The change in the definition of pier height and the exclusion of toe crushing as a failure mode were included which led to significant changes in the analysis results. The effective pier height was changed according to the recommendations made by MAE with lateral loading assumed to act on the right side of the wall (furthest from the door). Toe crushing was also excluded as a failure mode which meant that the piers were all now limited by rocking. This is very important as the failure mechanism is now deformation-controlled.

The linear static procedure was conducted with the modified pier strengths. Calculations can be found in Appendix F. The results of the analysis can be found in Table 11. With the modified pier height and failure definitions, the building meets acceptable criteria for the BSE-1 but the second floor falls short in the BSE-2. All design loads are very close to the wall strengths.

	BSE-1		BSE-2	
	First Floor	Second Floor	First Floor	Second Floor
Q_{UD} (k)	21.15	10.58	73.65	36.82
$m\kappa$QCL (k)	24.72	11.02	74.15	33.07

Table 11 : Results from modified linear static procedure

A nonlinear static analysis was also conducted using the modified strengths. Since the piers are now deformation-controlled, new force-deformation relationships needed to be created to model this behavior. The recommendations of MAE were used to develop these curves, which can be found in Appendix F. The curves have an initial elastic portion which degrades into a softly downward sloping line after yield and the commencement of rocking. The line slopes down until it reaches the toe crushing capacity of the wall where it plateaus until a sharp drop-off after it reaches the deformation corresponding to ultimate crushing. These curves show much larger allowable deformations than the force-controlled curves due to the ductility of the rocking mechanism.

The force-deformation curves were then combined for each floor to develop the force-displacement relationship for the building. Since the building is now deformation controlled, incrementally increasing displacements were applied to each story instead of forces. The shear force on each story was then calculated using the stiffness of the piers. When the story reached a displacement corresponding to a yield point, the stiffness of the yielded pier would change. The displacement of the control node was then plotted against the corresponding base shear to determine the response of the structure. This graph can be found in Figure 15. Using the same procedure previously described, the effective stiffness and period of the building was determined and subsequently the target displacement. The results can be found in Table 12. While the structure will reach the target displacement for both design earthquakes, drift is too great to meet the performance level requirements. The acceptable drifts in Table 12 correspond to 0.1% for the immediate occupancy performance level (BSE-1) and 0.3% for the life-safety performance level (BSE-2) as were noted earlier. Additionally, the standard specifies that the base shear at the target displacement must be greater than 80% of the effective yield strength of the building. This criterion is not met during the BSE-2 event.

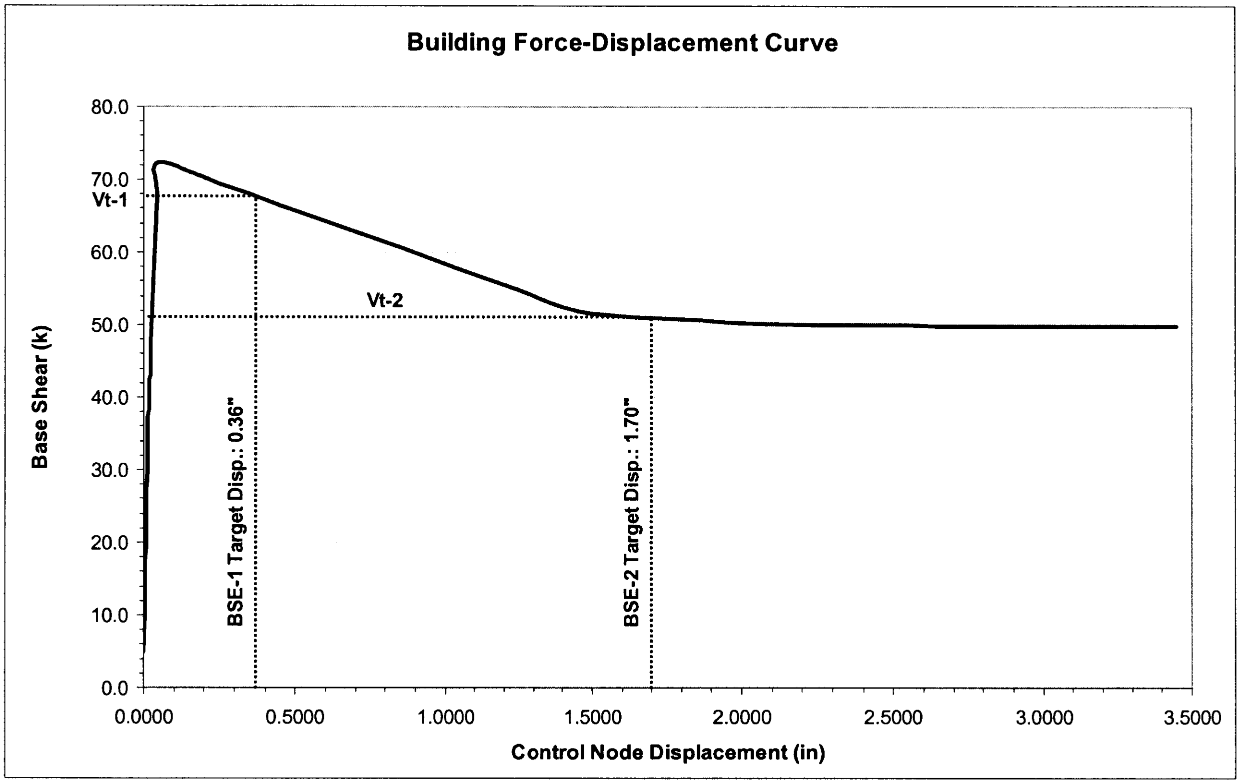

Figure 15: Building force-deformation curve for modified nonlinear static procedure

	BSE-1	BSE-2
Target Displacement (in)	0.36	1.70
Allowable Drift (in)	0.29	0.86

Table 12 : Target displacements for modified nonlinear static procedure

While the MAE modifications have a large effect on the analysis of the building, they do not change the final conclusion. The structure does not meet the performance level criteria for both seismic events and should be retrofitted to meet the code standards.

It is, however, very important that the failure mode was changed to rocking – a ductile mechanism. Even though the building still did not meet acceptance criteria in the modified analysis, it still had plenty of reserve capacity beyond the target displacement. Using the unmodified analysis method, the building collapsed well before the target strength was reached. Further research should be conducted to get a better idea as to which method provides the more accurate result.

6. Retrofit Techniques

Retrofitting unreinforced masonry buildings to meet today's seismic codes can be accomplished in a number of ways. These include:

- Local modifications (improving connections or component strength)
- Removal or lessening of structural irregularities (simplifying load paths or forcing ductile failures)
- Global structural stiffening (when lateral deflections are too great)
- Global structural strengthening (if there is a strength deficiency)
- Mass reduction (if building mass is excessive or flexibility high)
- Seismic isolation (decrease the seismic force entering structure)
- Supplemental energy dissipation (adding damping to structure)

The best rehabilitation technique for any particular project depends on the specifics of the building at hand. Some level of analysis is needed to determine if and why the building is deficient and what is the most cost-effective method to improve behavior. The following sections explain some popular techniques for retrofitting unreinforced masonry buildings.

6.1. Traditional Methods

Due to budget shortages and the cost of building downtime, it is often impractical to utilize innovative retrofitting techniques that the engineer or architect may desire. If this is the case, it is necessary to find "quick-fix" solutions that may or may not completely solve the long term problems of the building. These solutions usually involve repairing structural deficiencies to bring a building back to its original design strength or changing load paths to improve behavior. The following sections detail a few traditional repair techniques for URM buildings.

6.1.1. Infill Openings

A simple method to strengthening a shear wall in-plane is to infill unnecessary window and door openings. This prevents stress concentrations from forming at the corners of openings that initiate cracks. The important thing to consider when infilling an opening is to interlace the new units with the existing or to provide some type of shear connection between the two. This ensures that the existing wall works compositely with the new infill.

6.1.2. Enlarge Openings

Alternatively, it is also an option to enlarge openings by removing portions of masonry. This technique would be employed to increase the aspect ratio of a pier to alter the limit state from shear to flexure. This changes the mode of failure from brittle to ductile.

6.1.3. Increase Vertical Load

Adding vertical load to an unreinforced masonry wall will often improve its behavior for both in-plane and out-of-plane loading. The vertical load will help hold the masonry matrix together and will lead to larger friction forces after cracking. This retrofit can be accomplished by simply adding weight to a structure or by implementing post-tensioning rods or cables that apply vertical stress onto a wall element. This, of course, must be done carefully as the vertical load will increase the stress on the units and could lead to brittle crushing failures. The designer must also be sure to account for the loss of tension that will occur due to creep and shrinkage of the masonry.

6.1.4. Fortify Wall-Diaphragm Connection

A common problem with older URM buildings is insufficient or degraded wall-to-diaphragm connections. This connection is critical to the global behavior of the structure as the diaphragm both braces the wall and, in the case of rigid diaphragms, forces parallel walls to act together. A similar retrofit is to improve the veneer anchorage to the back-up wall or frame. This connection is often inadequate and the main cause behind out-of-plane façade failures.

6.1.5. Grout Injections/Repointing

Two of the most common retrofitting methods are grout injections and repointing. Grout injections involve filling cracks and voids with grout to restore the wall to its original strength. The important factor here is ensuring that the grout has similar strength, modulus, and thermal properties as the existing masonry. Repointing entails removing and replacing poor mortar to restore the wall to its original strength. The compressive strength of the mortar should be at least equal to the existing.

6.1.6. Shotcrete Overlay

An option for retrofitting URM elements is to cover the wall or pier with shotcrete (sprayed-on concrete). Shear connections must be provided between the existing wall and the shotcrete for the system to work compositely. If designed properly, the added steel reinforcing necessary for this retrofit will add a large amount of energy absorption capacity to the structure (Abrams 2007).

6.1.7. Grouted Cores

A grouted steel core can be added to an unreinforced wall to change the behavior to that of a reinforced wall. The grout must provide adequate connection between the reinforcement and the existing masonry to transfer the seismic forces. The anchorage of the core is also important to ensure that it can develop the full tensile strength of the wall. The strength and modulus properties of the grout must be compatible with the existing masonry. The added steel can vastly increase the ductility of the shear walls (Abrams 2007).

6.1.8. Added Bracing

An obvious method to improve the behavior of a structure is to incorporate new steel bracing elements to add stiffness. This can be employed if a building is deflecting too much. The added steel will also improve the ductility of the structure.

6.1.9. Steel Strips

Adding diagonal and vertical steel strips to the exterior of a masonry wall or pier will enhance seismic strength and ductility of URM walls. If anchored properly to the wall, the steel can behave like a truss under lateral loading (Taghdi 2000).

6.2. Fiber-Reinforced Polymer Methods

Fiber-reinforced polymers are fast becoming a commonly specified material in civil engineering projects, especially rehabilitations. Their high strength-to-weight and stiffness-to-weight ratios make FRP materials ideal for structural applications. This is especially true for seismic retrofits since a significant amount of stiffness and strength can be added to a structure while adding only negligible mass. A number of recent studies have tested the behavior of URM elements retrofitted with FRP to improve both in-plane and out-of-plane loading. The following sections describe some of the techniques used for masonry elements.

6.2.1. Full Wall Cover or X-Strips

The simplest and most tested FRP masonry rehabilitation technique is to cover an entire wall element with one or more sheets of FRP material. Studies have shown that this can be done on either one side or both sides of a wall without any noticeable out-of-plane effects. It is also possible to lay FRP strips onto a wall in an X-pattern to achieve a similar effect. Both of these systems enhance the in-plane stiffness and strength of the wall as well as the out-of-plane behavior. The FRP sheet, which is bonded to the wall with an epoxy coating, confines the masonry material and helps slow crack propagation. While there is no question that the FRP overlay increases strength and stiffness, some question remains as to the energy dissipation characteristics of the rehabbed system. A normal URM element will dissipate energy post-cracking through rocking and sliding along cracks. The FRP cover prevents this behavior and leaves the FRP-masonry bond failure as the main means of energy dissipation. This could lead to a more brittle failure than in the original scheme.

6.2.2. Reinforced Openings

An alternative method to a full overlay is to place FRP strips along the perimeter of window and door openings as well as placing intermittent vertical strips along the wall as illustrated in Figure 17. This procedure has mainly been tested to improve out-of-plane behavior and has been shown to improve strength and ductility up to five times for this case. The idea is that the strips will help prevent cracks from forming due to stress concentrations at the window corners and also change the behavior of the wall from an element spanning between two horizontal supports to multiple elements spanning between the vertical strips. To achieve this behavior, the proper anchorage and bond of the FRP strips is critical (Ghobarah 2004). A proper anchorage is illustrated in Figure 16. Reinforced openings will also improve in-plane behavior of a shear wall by making the pier elements behave like vertical reinforced beams.

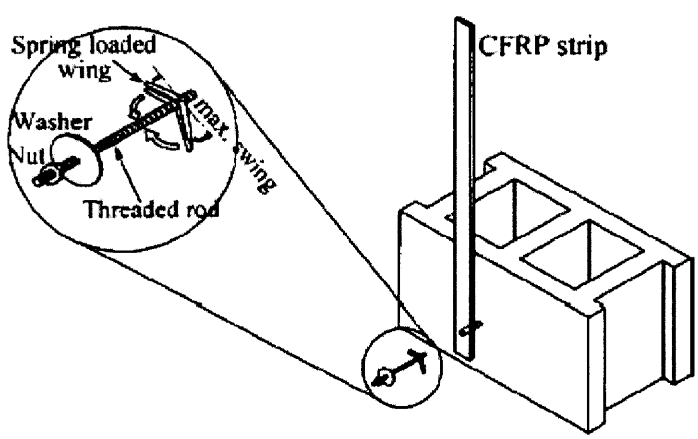

Figure 16 : Anchorage detail of FRP strip to masonry unit (Ghobarah 2004)

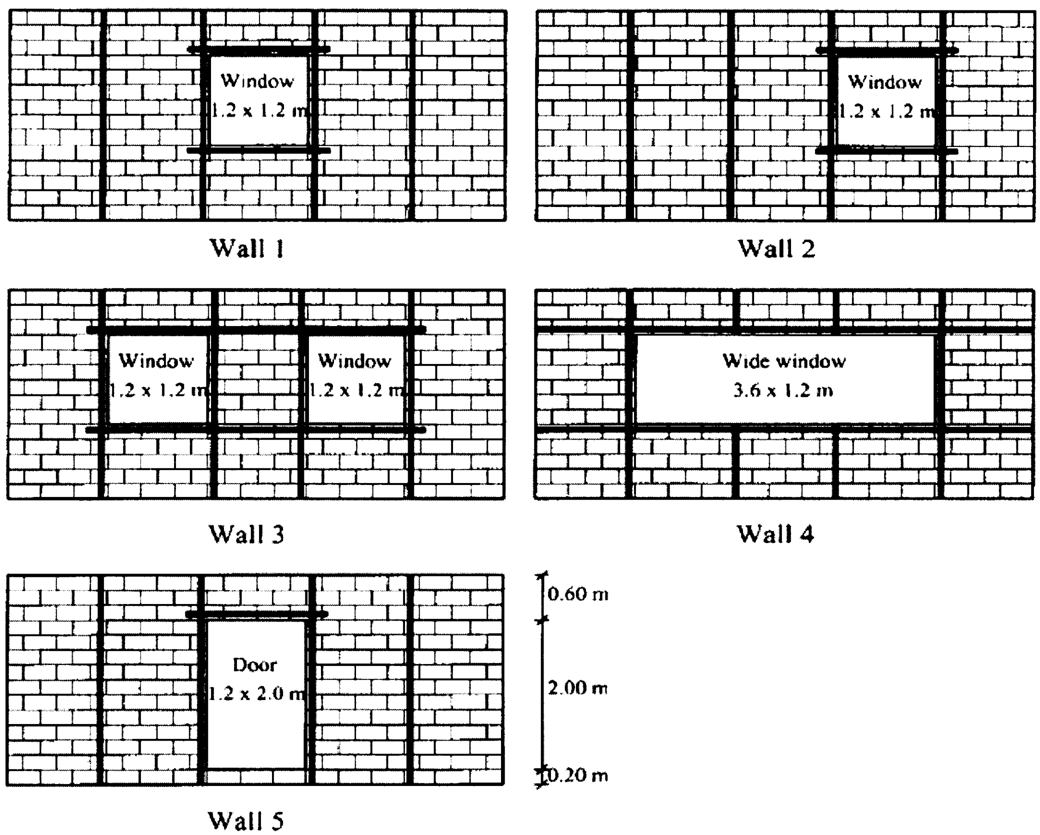

Figure 17 : Examples of walls retrofitted with FRP strips at openings (Ghobarah 2004)

6.2.3. Near-Surface Reinforcing

Although the above steel, concrete, and FRP rehabilitation techniques are structurally acceptable and could be utilized to bring most URM structures to code, they all share the common flaw that they drastically change the appearance of the structure. This makes a shotcrete overlay, steel bracing, or an FRP overlay entirely unacceptable for a historic building in which the masonry appearance must be preserved. To combat this problem, an unobtrusive FRP rehabilitation method has been created to help out-of-plane behavior of URM elements.

The technique involves placing thin carbon fiber composite cables (CFCC) horizontally into bed joints and vertically through head joints and bricks. The procedure involves cutting grooves into joints, drilling holes through brick units where necessary, placing epoxy into the grooves, placing the CFCC's, and repointing the masonry. This produces invisible "near-surface" FRP reinforcement in the wall. The method is illustrated in Figure 18. Studies have shown that walls retrofitted in this manner have shown significant increases in ultimate capacity, energy absorption, and deformability (Korany 2006).

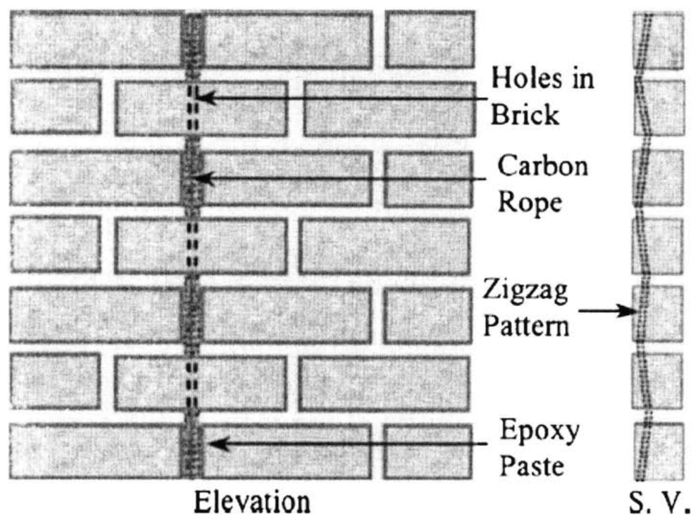

Figure 18 : Near-surface CFCC reinforcing technique (Korany 2006)

6.3. Base Isolation

An expensive but effective method of retrofitting masonry structures is base isolation. This is a difficult procedure for URM structures and involves placing bearings at the base of a structure to prevent ground accelerations from entering the building during an earthquake. This type of base isolation has been implemented for many retrofit projects on the West coast, including both the San Francisco and Los Angeles City Halls. Both of these buildings were constructed in the early 20th century and were considered historic structures. This meant that any rehabilitation could not change the exterior or interior appearance of the building. Considering the large budget for each project and the high seismic loads, base isolation was the only logical retrofit technique. The rehabilitation of the San Francisco City Hall cost a total of $293 million and the Los Angeles City Hall cost $273 million.

Alternative to bearings, a different isolation technique is possible for masonry buildings. Studies have shown that placing a layer of soft mortar between the foundation and the base of the shear wall can provide significant force reduction in the wall. Steel bars are also placed between the foundation and the shear wall to provide initial stiffness and energy dissipation during extreme loadings. This method is sometimes called a "reinforced cut-wall" and can be seen in Figure 19. A reinforced concrete beam is placed between the mortar and the existing masonry wall to provide adequate stiffness to the system. While this system has potential, it has not extensively used due to a poor understanding of the high fluctuations in mortar properties. More research needs to be conducted to develop standardized material strengths (Palazzo 2001).

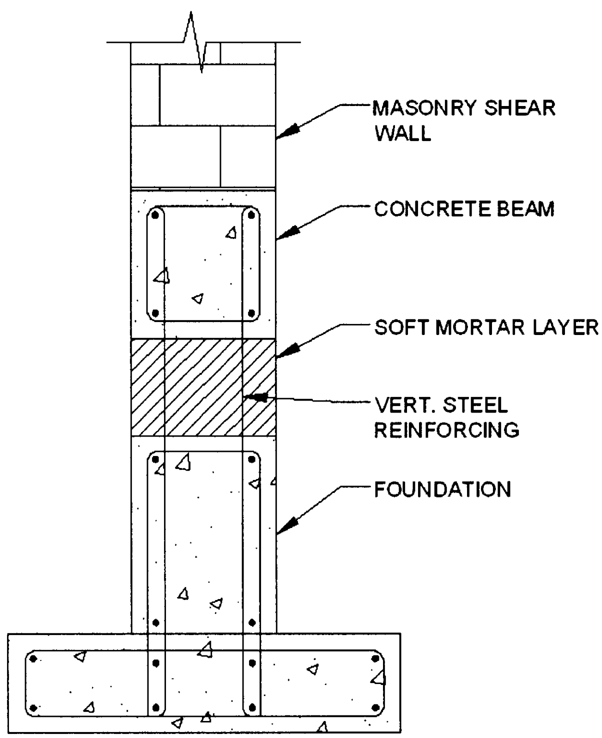

Figure 19 : Reinforced cut-wall base isolation system (Palazzo 2006)

6.4. *Analysis of Retrofitted Test Wall*

Two retrofit methods were chosen to analyze for the test wall: reinforced grouted cores and fiber-reinforced polymer strips at the perimeters of openings. These methods were chosen because they can enhance the behavior of the building without completely changing its structural system (like added bracing or shotcrete overlays). While the grouted cores can be difficult to install and temporarily disrupt the function of the building, the end result does not change the appearance of the building. This means that they are a possible retrofit option for historic structures or in the case that the owner wants to maintain the same masonry look to his building. The FRP strips behave in a similar manner to the cores, except that they are obviously applied to the exterior of the wall. They are much easier to install than the cores but they also take away from the appearance of the masonry wall, although less so than a complete FRP overlay.

To calculate the strength of the retrofitted structure, both systems were assumed to act as reinforced shear walls. The procedure is similar to a URM wall as the piers are still considered the limiting element under lateral load except the failure modes are different. FEMA 356 designates two types of failure mechanisms for reinforced masonry piers: flexure and shear. The piers are basically considered vertical beam elements with fixed-fixed (coupled) or fixed-free (uncoupled) end conditions. According to the standard, a flexural failure is deformation controlled, as it involves yielding steel, while a shear failure is force controlled. With this in mind, a linear static analysis was conducted on both retrofitted buildings.

6.4.1. Analysis for Reinforced Grouted Cores

The amount of steel specified for the retrofit was (4)-#4 bars at the edge of each of the piers. This would provide reinforcing for both lateral directions. The amount of steel was determined by approximating the minimum amount of reinforcing needed for an equivalent concrete beam. This would ensure that the steel would yield before the masonry crushed so that the failure would be ductile. It must be noted that when installing these cores, it is critical to anchor the rebar sufficiently so that the full tensile strength can be developed. Then steel was assumed to have a yield strength of 60ksi and a typical modulus of 29,000ksi. See Figure 20 for a schematic of the retrofitted wall.

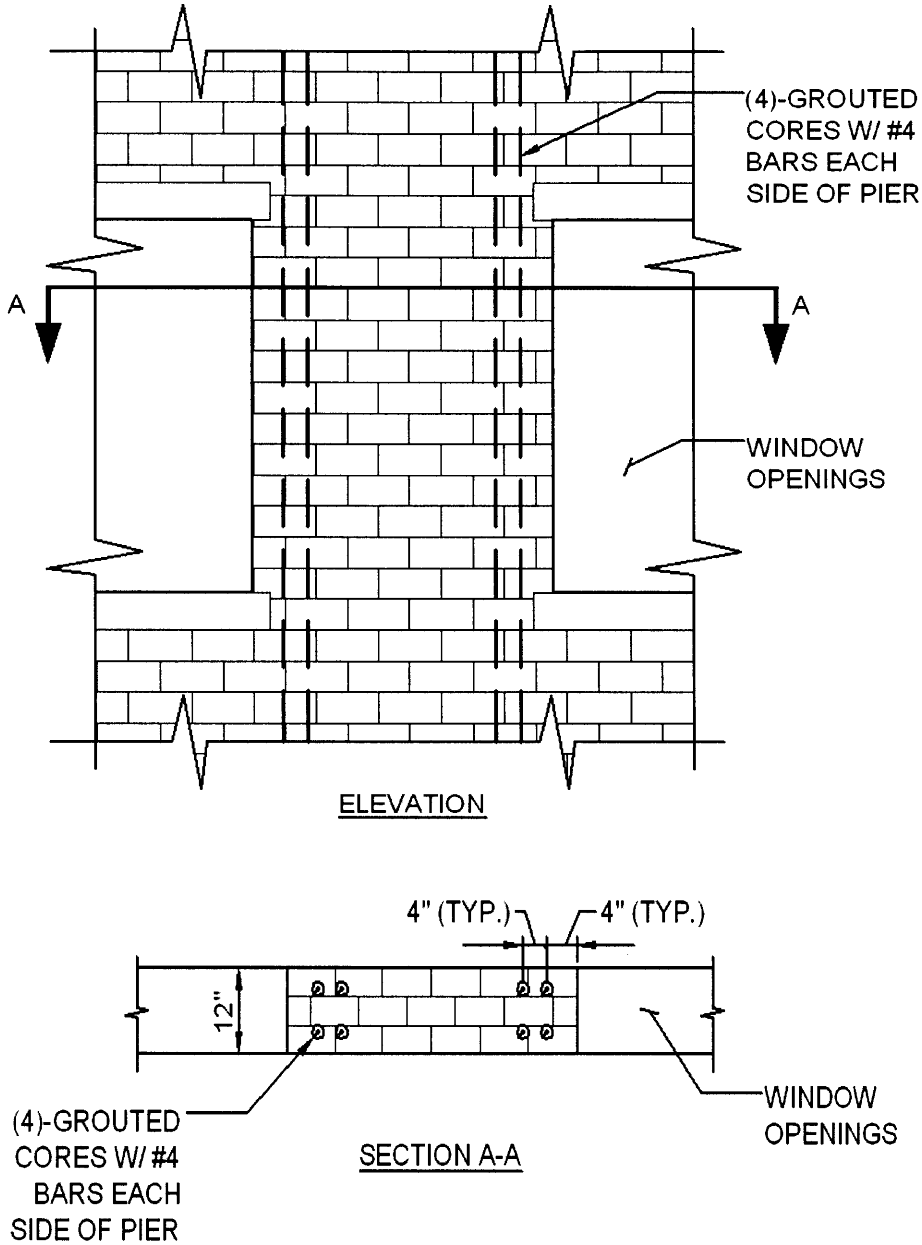

Figure 20 : Reinforced grouted core retrofit

Using the procedures outlined in FEMA 356, the strengths of each of the piers was calculated. The standard recommends using an equivalent rectangular stress block, very similar to that used for concrete, to determine the flexural strength of the piers. The shear strength of the masonry is calculated from the following equation (ATC 2000):

$$V_{mL} = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right] A_n \sqrt{f'_m} + 0.25P_L \quad \textbf{(Eq. 6-1)}$$

Where:

M = Moment on the masonry section

V = Shear on the masonry section

d_v = Wall length in direction of shear force

A_n = Area of net mortared/grouted section

f'_m = Lower bound compressive strength of masonry

P_L = Lower bound vertical load on pier

All of the piers were found to be limited by flexure, meaning they were deformation controlled. A linear static analysis was then used to determine if the walls met the acceptance criteria for the design earthquakes. The analysis was conducted according to the procedures of the current FEMA 356, not the modified version explained earlier. All of the piers easily met the criteria for both seismic events. The results can be found in Table 13.

	BSE-1		**BSE-2**	
	First Floor	Second Floor	First Floor	Second Floor
Q_{UD} (k)	21.15	10.58	73.65	36.82
mκQCL (k)	103.70	106.36	311.09	319.08

Table 13 : Results of linear static analysis for grouted reinforced core retrofitted wall

6.4.2. Analysis for FRP Strips

The analysis for the FRP strips was carried out with similar assumptions used in the grouted core wall, except with the FRP acting as the tension element instead of the steel. The properties of the steel were replaced with those of FRP – the tension strength was assumed to be 45ksi and the modulus 2,800ksi. Two layers of FRP were used on each edge of the openings and on each side of the wall. The strips were determined to be 12" wide to provide an equivalent amount as the steel reinforcing used in the grouted core wall. This meant that a total area of 0.47 square inches of FRP was used for calculations. See Figure 21 for a schematic of the retrofitted wall.

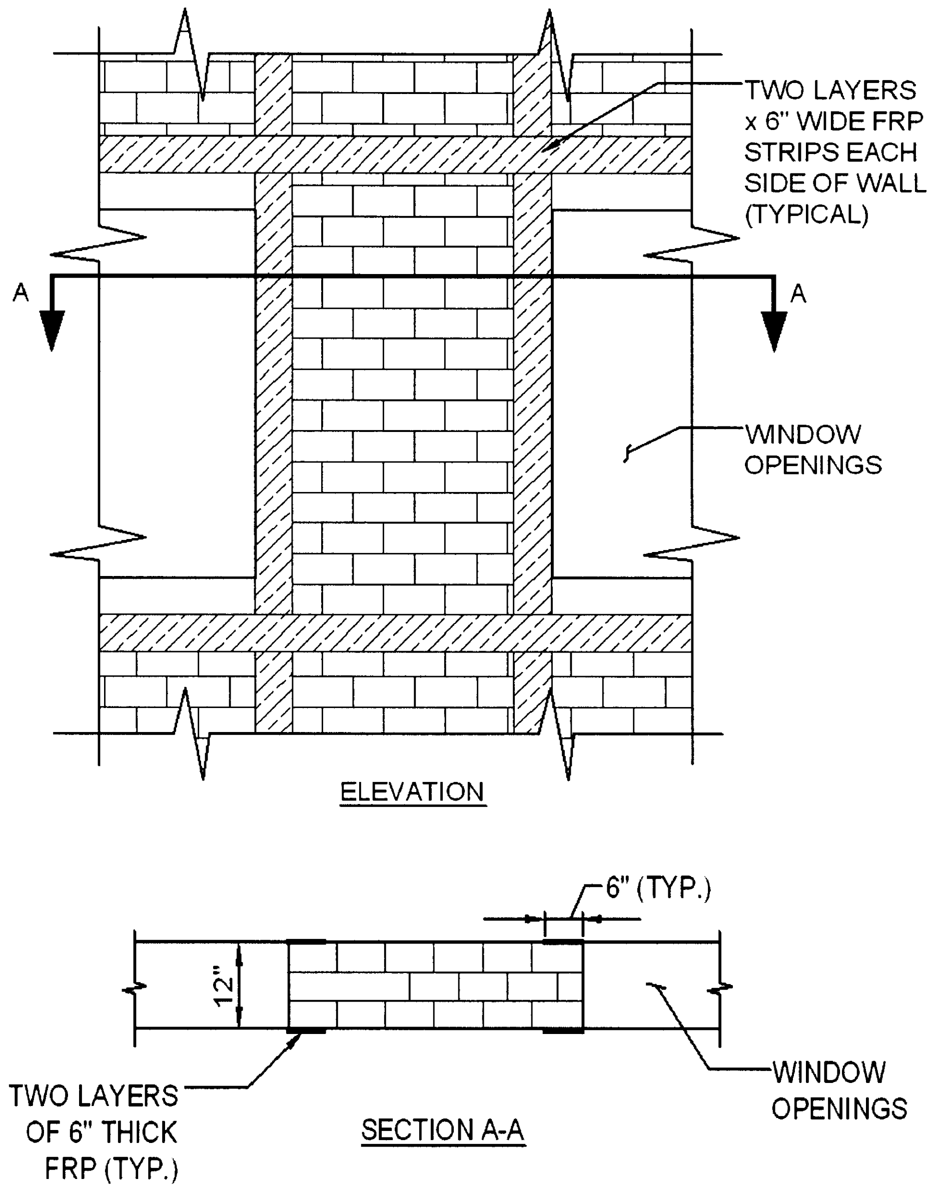

Figure 21 : FRP retrofit method

After the strength values were calculated, it was again determined that each pier failed in flexure. However, even though the failure mechanism is the same as in the grouted core wall, for FRP the flexural failure cannot be considered ductile and therefore cannot be labeled as deformation controlled. Unlike in a steel reinforced wall where the steel yields and proceeds to dissipate energy in its plastic phase, the FRP has a brittle failure with the only energy dissipation is through tearing and bond rupture. This can be a scary thought for an engineer. This means that when comparing the strength of the retrofitted walls to the design values, the flexural failure of the piers must be considered force controlled. With this in mind, the calculated wall strengths were compared to the design forces. The results can be found in Table 14. The wall meets criteria for BSE-1, but does not pass for BSE-2. This result can be directly attributed to the fact that the FRP fails in a brittle manner. If the flexural failure was ductile, the wall strength would be multiplied by the m-factor of three for the Life-Safety performance level specified for BSE-2. This would make the wall well within the acceptable criteria.

	BSE-1		**BSE-2**	
	First Floor	Second Floor	First Floor	Second Floor
Q_{UD} (k)	21.15	10.58	73.65	36.82
κQ_{CL} (k)	47.44	48.65	47.44	48.65

Table 14 : Results of linear static analysis for FRP retrofitted wall

7. Conclusions

The performance-based analysis and retrofit procedures outlined in FEMA 356 for the rehabilitation of existing unreinforced masonry buildings have been illustrated and used on a fictional brick shear wall. A linear static, linear dynamic, and nonlinear static analysis was carried out to determine the strength of the existing wall. These strengths were then compared to seismic loads calculated from response spectrums for the region. Two retrofit methods were then reviewed and analyzed using the same design procedures. Additionally, a modified analysis of the FEMA procedure was conducted according to recommended changes from the Mid-America Earthquake Center.

A comparison of the analysis results shows the difficulty in predicting the behavior of unreinforced masonry buildings. The results for the unretrofitted wall generally showed that the wall came close to meeting acceptance criteria for the 10%/50 year seismic event (BSE-1) but fell short during the 2%/50 year quake (BSE-2). The pier failure mode was also critical when calculating the capacity of the walls, as the performance-based code rewarded deformation controlled elements by including a ductility factor. Force controlled elements do not get this benefit so their estimated capacities lean to the conservative side. It is important to note that the MAE did not agree with the FEMA 356 definition of toe crushing as a limiting force controlled failure mode for URM walls. All piers in the test wall were limited by this failure mechanism. If the MAE recommendations are adopted, the failure modes would have all changed to rocking and the wall would have gained significant capacity after yield. More research must be conducted to determine which method provides a better picture of masonry wall behavior.

The ductility of the retrofitting method must also be taken into account during design. While the reinforced grouted core retrofit and the FRP strips retrofit both provided similar strength capacities, the grouted core wall benefited from the ductility factor since the flexural failure of the reinforcing dissipated significant energy through steel yielding. The FRP strips, however, fail quickly through de-bonding or tearing and therefore must be considered force controlled. This meant that although the FRP provided similar strength in comparison to the steel, it did not meet the acceptance criteria for the BSE-2 while the steel method did easily. So although the use of FRP is growing in these types of retrofit projects because of its strength-to-weight ratio and ease of application, the designer must keep in mind that the use of the polymers are not rewarded by codes because of their lack of energy dissipation during extreme events.

Although more research is needed, the performance-based methodology of FEMA 356 provides a good basis for the difficult task of analyzing URM structures for seismic loading. The duty falls on the practicing engineer as to which analysis method is appropriate for a specific project. Limited information and resources will often restrict the amount of analysis and retrofitting that can be performed on a building. Historical significance, budget concerns, occupancy difficulties, and negligence will often complicate and delay necessary rehabilitation projects. The longer these projects go unperformed, the greater the risk that the vulnerable buildings will experience a seismic event before being repaired.

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Appendix A

Seismic Calculations

Appendix A - Seismic Calculations

Location: Boston, MA
Site Class: D

Notes:
1. Seismic calculations performed according to Section 1-6 of FEMA-356
2. Spectral accelerations obtained from USGS earthquake hazard maps
3. Basic Safety Earthquake 1 (BSE-1) is based on 10%/50 year seismic event
4. Basic Safety Earthquake 2 (BSE-2) is based on 2%/50 year seismic event
5. S_s is the short period response acceleration (0.2 s)
6. S_1 is the long period response acceleration (1 s)
7. Values obtained for 5% damped response

BSE-1	
S_s (%g):	0.090
S_1 (%g):	0.025
F_a:	1.6
F_v:	2.4
S_{xs}:	0.144
S_{x1}:	0.060
Damping (% Critical):	5
B_s:	1.0
B_1:	1.0
T_s:	0.417
T_0:	0.083

BSE-2	
S_s (%g):	0.280
S_1 (%g):	0.090
F_a:	1.6
F_v:	2.4
S_{xs}:	0.448
S_{x1}:	0.216
Damping (% Critical):	5
B_s:	1.0
B_1:	1.0
T_s:	0.482
T_0:	0.096

Table 1-4: Fa as a Function of Site Class and Short-Period Spectral Response Ss

	S_s				
Site Class	**≤0.25**	**0.50**	**0.75**	**1.00**	**≥1.25**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	-	-	-	-	-

Table 1-5: Fv as a Function of Site Class and One Second Spectral Response Ss

	S_v				
Site Class	**≤0.1**	**0.2**	**0.3**	**0.4**	**≥0.5**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	2.5	1.7	1.2	0.9	0.9
F	-	-	-	-	-

Table 1-6: Damping Coefficients as a Function of Effective Damping

Effective viscous damping	B_s	B_1
≤2	0.8	0.8
5	1.0	1.0
10	1.3	1.2
20	1.8	1.5
30	2.3	1.7
40	2.7	1.9
≥50	3.0	2.0

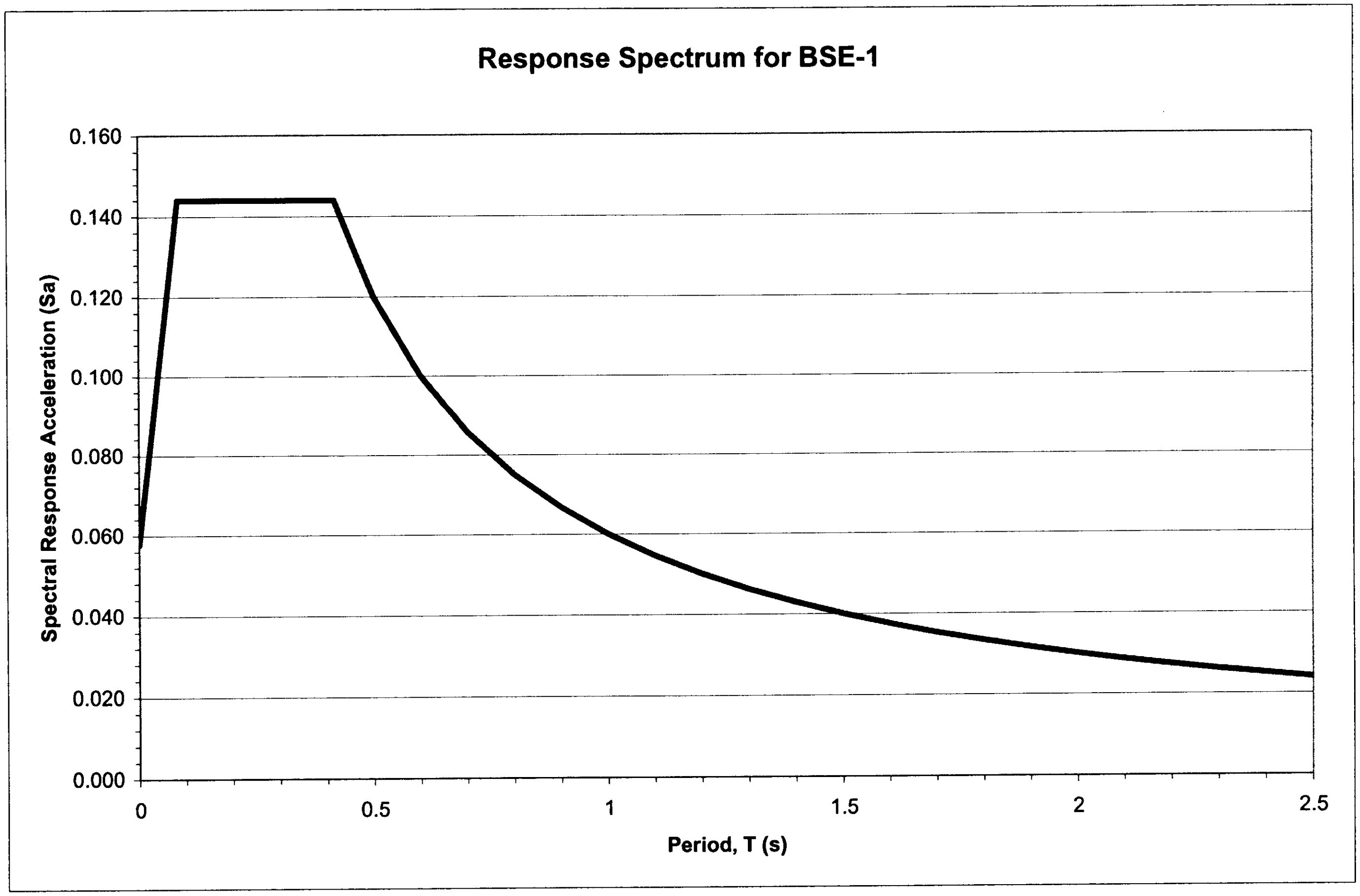
Appendix A - Seismic Calculations

BSE-1:

T	Sa
0	0.058
0.01	0.068
0.02	0.078
0.03	0.089
0.04	0.099
0.05	0.109
0.06	0.120
0.07	0.130
0.08	0.141
0.083	0.144

T	Sa
0.083	0.144
0.417	0.144

T	Sa
0.417	0.144
0.50	0.120
0.60	0.100
0.70	0.086
0.80	0.075
0.90	0.067
1.00	0.060
1.10	0.055
1.20	0.050
1.30	0.046
1.40	0.043
1.50	0.040
1.60	0.038
1.70	0.035
1.80	0.033
1.90	0.032
2.00	0.030
2.10	0.029
2.20	0.027
2.30	0.026
2.40	0.025
2.50	0.024



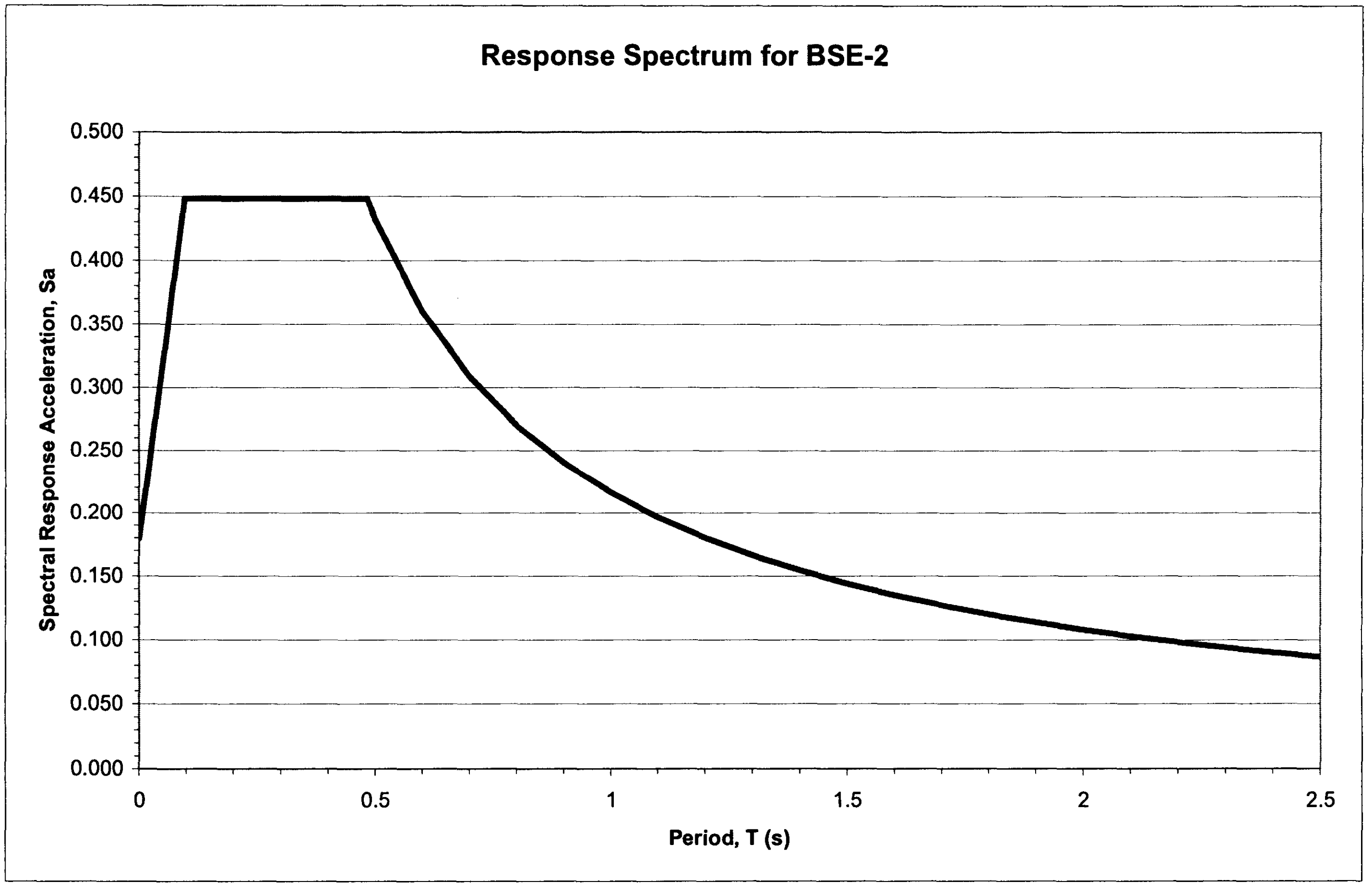
Appendix A - Seismic Calculations

BSE-2:

T	Sa
0	0.179
0.01	0.207
0.02	0.235
0.03	0.263
0.04	0.291
0.05	0.319
0.06	0.346
0.07	0.374
0.08	0.402
0.09	0.430
0.096	0.448

T	Sa
0.096	0.448
0.482	0.448

T	Sa
0.482	0.448
0.50	0.432
0.60	0.360
0.70	0.309
0.80	0.270
0.90	0.240
1.00	0.216
1.10	0.196
1.20	0.180
1.30	0.166
1.40	0.154
1.50	0.144
1.60	0.135
1.70	0.127
1.80	0.120
1.90	0.114
2.00	0.108
2.10	0.103
2.20	0.098
2.30	0.094
2.40	0.090
2.50	0.086



Appendix B

Wall Properties

Appendix B - Wall Properties

Default Component Properties

Lower-Bound Masonry Properties

Property	Good	Fair	Poor
Compressive Strength (f'_m) (psi)	900	600	300
Elastic Modulus in Comp. (E_m) (psi)	495000	330000	165000
Flexural Tensile Strength (psi)	20	10	0
Shear Strength			
Shear Modulus (G_m) (psi)	198000	132000	66000
Running bond lay-up (psi)	27	20	13
Fully Grouted w/o running bond (psi)	27	20	13
Partial/ungrouted w/o running bond (psi)	11	8	5

Expected Strength Masonry Properties

Property	Good	Fair	Poor
Compressive Strength (f'_m) (psi)	1170	780	390
Elastic Modulus in Comp. (E_m) (psi)	643500	429000	214500
Flexural Tensile Strength (psi)	26	13	0
Shear Strength			
Shear Modulus (G_m) (psi)	257400	171600	85800
Running bond lay-up (psi)	35.1	26	16.9
Fully Grouted w/o running bond (psi)	35.1	26	16.9
Partial/ungrouted w/o running bond (psi)	14.3	10.4	6.5

Appendix B - Wall Properties

Wall Stiffness

Wall Thickness (in):	12
Masonry Wt. (pcf):	130
Number of Stories:	2
Masonry Condition:	fair

Model DOF: 2

Component Properties:		
	Lower Bound	**Expected**
f'_m (psi):	600	780
E_m (psi):	330000	429000
G_m (psi):	132000	171600

First Story

Story Height (ft):	12		
No. of Opngs:	3		
No. of Piers:	4		
Pier One:	Length (ft):	5	
	h_{eff} (ft):	9	
	I_g (in^4):	216000	
	A_v (in^2):	720	
		L.B.	**Exp.**
	k (k/in):	383	498
Pier Two:	Length (ft):	4	
	h_{eff} (ft):	5	
	I_g (in^4):	110592	
	A_v (in^2):	576	
		L.B.	**Exp.**
	k (k/in):	780	1014
Pier Three:	Length (ft):	4	
	h_{eff} (ft):	5	
	I_g (in^4):	110592	
	A_v (in^2):	576	
		L.B.	**Exp.**
	k (k/in):	780	1014
Pier Four:	Length (ft):	5	
	h_{eff} (ft):	5	
	I_g (in^4):	216000	
	A_v (in^2):	720	
		L.B.	**Exp.**
	k (k/in):	1131	1471
		L.B.	**Exp.**
Total Story One Stiffness (k/in):		**3074**	**3997**

Second Story

Story Height (ft):	12		
No. of Opngs:	3		
No. of Piers:	4		
Pier Five:	Length (ft):	5	
	h_{eff} (ft):	5	
	I_g (in^4):	216000	
	A_v (in^2):	720	
		L.B.	**Exp.**
	k (k/in):	1131	1471
Pier Six:	Length (ft):	4	
	h_{eff} (ft):	5	
	I_g (in^4):	110592	
	A_v (in^2):	576	
		L.B.	**Exp.**
	k (k/in):	780	1014
Pier Seven:	Length (ft):	4	
	h_{eff} (ft):	5	
	I_g (in^4):	110592	
	A_v (in^2):	576	
		L.B.	**Exp.**
	k (k/in):	780	1014
Pier Eight:	Length (ft):	5	
	h_{eff} (ft):	5	
	I_g (in^4):	216000	
	A_v (in^2):	720	
		L.B.	**Exp.**
	k (k/in):	1131	1471
		L.B.	**Exp.**
Total Story Two Stiffness (k/in):		**3822**	**4969**

Appendix B - Wall Properties

Wall Strength

Wall Thickness (in): 12
Masonry Wt. (pcf): 120
Number of Stories: 2

Masonry Condition: fair
Running Bond?: yes

Model DOF: 2

Component Properties:			
	Lower Bound	Expected	
f'_m:	600	780	psi
E_m:	330,000	429,000	psi
G_m:	132,000	171,600	psi
v_{te}:	20	26	psi

Gravity Loads			
First Story		Second Story	
Ht. of masonry above (ft):	15	Ht. of masonry above (ft):	4
Dead Load from Masonry (plf):	1800	Dead Load from Masonry (plf):	480
Dead Load from Building (plf):	375	Dead Load from Building (plf):	375
Live/Snow Load from Bldg. (plf):	375	Live/Snow Load from Bldg. (plf):	375
Q_G (Factored Additive) (plf):	3630	Q_G (Factored Additive) (plf):	1353
Q_G (Factored Counteractive) (plf):	2295	Q_G (Factored Counteractive) (plf):	769.5

Appendix B - Wall Properties

Wall Strength

Expected Strength

First Story

Pier One:	Length (ft):	5.0		
	h_{eff} (ft):	9.0		
	A_n (in^2):	720		
	v_{te} (psi):	26		
	P_{CE} (lbs):	18,150		
	v_{me} (psi):	22.35		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	16.10	
		V_r (k):	9.08	**Controls!**
Pier Two:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	v_{te} (psi):	26		
	P_{CE} (lbs):	14,520		
	v_{me} (psi):	22.35		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	12.88	
		V_r (k):	10.45	**Controls!**
Pier Three:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	v_{te} (psi):	26		
	P_{CE} (lbs):	14,520		
	v_{me} (psi):	22.35		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	12.88	
		V_r (k):	10.45	**Controls!**
Pier Four:	Length (ft):	5.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	720		
	v_{te} (psi):	26		
	P_{CE} (lbs):	18,150		
	v_{me} (psi):	22.35		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	16.10	**Controls!**
		V_r (k):	16.34	
Exp. Story Strength (Q_{CE}):		V_{bjs} (k):	57.94	
		V_r (k):	46.32	**Controls!**

Second Story

Pier Five:	Length (ft):	5.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	720		
	v_{te} (psi):	26		
	P_{CE} (lbs):	6,765		
	v_{me} (psi):	14.45		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	10.40	
		V_r (k):	6.09	**Controls!**
Pier Six:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	v_{te} (psi):	26		
	P_{CE} (lbs):	5,412		
	v_{me} (psi):	14.45		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	8.32	
		V_r (k):	3.90	**Controls!**
Pier Seven:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	v_{te} (psi):	26		
	P_{CE} (lbs):	5,412		
	v_{me} (psi):	14.45		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	8.32	
		V_r (k):	3.90	**Controls!**
Pier Eight:	Length (ft):	5.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	720		
	v_{te} (psi):	26		
	P_{CE} (lbs):	6,765		
	v_{me} (psi):	14.45		
	α:	1.0		
	Q_{CE}:	V_{bjs} (k):	10.40	
		V_r (k):	6.09	**Controls!**
Exp. Story Strength (Q_{CE}):		V_{bjs} (k):	37.45	
		V_r (k):	19.97	**Controls!**

Appendix B - Wall Properties

Wall Strength

Lower Bound Strength

First Story

Pier One:	Length (ft):	5.0		
	h_{eff} (ft):	9.0		
	A_n (in^2):	720		
	f'_a (psi):	16		
	P_{CL} (lbs):	11,475		
	f'_{dt} (psi):	22.35		
	α:	1.0		
	L/h_{eff}:	0.67		
	Q_{CL}:	V_{dt} (k):	14.11	
		V_{tc} (k):	7.40	**Controls!**
		P_{CL} (k):	293.8	
Pier Two:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	f'_a (psi):	16		
	P_{CL} (lbs):	9,180		
	f'_{dt} (psi):	22.35		
	α:	1.0		
	L/h_{eff}:	0.8		
	Q_{CL}:	V_{dt} (k):	13.48	
		V_{tc} (k):	7.07	**Controls!**
		P_{CL} (k):	235.0	
Pier Three:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	f'_a (psi):	16		
	P_{CL} (lbs):	9,180		
	f'_{dt} (psi):	22.35		
	α:	1.0		
	L/h_{eff}:	0.8		
	Q_{CL}:	V_{dt} (k):	13.48	
		V_{tc} (k):	7.07	**Controls!**
		P_{CL} (k):	235.0	
Pier Four:	Length (ft):	5.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	720		
	f'_a (psi):	16		
	P_{CL} (lbs):	11,475		
	f'_{dt} (psi):	22.35		
	α:	1.0		
	L/h_{eff}:	1		
	Q_{CL}:	V_{dt} (k):	21.07	
		V_{tc} (k):	11.04	**Controls!**
		P_{CL} (k):	293.8	
L.B. Story Strength (Q_{CL}):		V_{dt} (k):	62.14	
		V_{tc} (k):	32.57	**Controls!**

Second Story

Pier Five:	Length (ft):	5.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	720		
	f'_a (psi):	5		
	P_{CL} (lbs):	3,848		
	f'_{dt} (psi):	14.45		
	α:	1.0		
	L/h_{eff}:	1		
	Q_{CL}:	V_{dt} (k):	12.18	
		V_{tc} (k):	3.80	**Controls!**
		P_{CL} (k):	293.8	
Pier Six:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	f'_a (psi):	5		
	P_{CL} (lbs):	3,078		
	f'_{dt} (psi):	14.45		
	α:	1.0		
	L/h_{eff}:	0.8		
	Q_{CL}:	V_{dt} (k):	7.79	
		V_{tc} (k):	2.43	**Controls!**
		P_{CL} (k):	235.0	
Pier Seven:	Length (ft):	4.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	576		
	f'_a (psi):	5		
	P_{CL} (lbs):	3,078		
	f'_{dt} (psi):	14.45		
	α:	1.0		
	L/h_{eff}:	0.8		
	Q_{CL}:	V_{dt} (k):	7.79	
		V_{tc} (k):	2.43	**Controls!**
		P_{CL} (k):	235.0	
Pier Eight:	Length (ft):	5.0		
	h_{eff} (ft):	5.0		
	A_n (in^2):	720		
	f'_a (psi):	5		
	P_{CL} (lbs):	3,848		
	f'_{dt} (psi):	14.45		
	α:	1.0		
	L/h_{eff}:	1		
	Q_{CL}:	V_{dt} (k):	12.18	
		V_{tc} (k):	3.80	**Controls!**
		P_{CL} (k):	293.8	
L.B. Story Strength (Q_{CL}):		V_{dt} (k):	39.93	
		V_{tc} (k):	12.46	**Controls!**

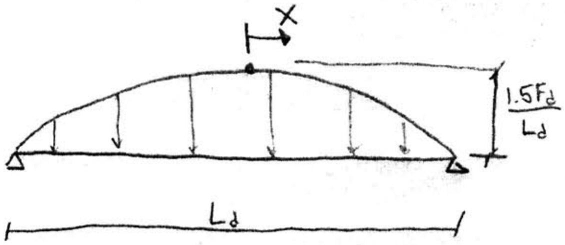
Appendix C

Linear and Nonlinear Analyses

APPENDIX C

CALCULATE DIAPHRAGM DEFLECTION

According to FEMA, applied inertia load $f_d = \frac{1.5F_d}{L_d}\left[1-\left(\frac{2x}{L_d}\right)^2\right]$



f_d = Inertia Load per Foot
F_d = Total inertia load on diaphragm
x = Distance from centerline of diaphragm
L_d = Distance between lateral supports

$$f_d = \frac{1.5F_d}{L_d}\left[1-\left(\frac{2x}{L_d}\right)^2\right] = \frac{1.5F_d}{L_d} - \frac{6F_d x^2}{L_d^3}$$

$$V = \frac{1.5F_d x}{L_d} - \frac{2F_d x^3}{L_d^3} + C_1 \qquad \rightarrow V(x=0)=0\ ,\ C_1=0$$

$$M = \frac{1.5F_d x^2}{2L_d} - \frac{F_d x^4}{2L_d^3} + C_2 \qquad \rightarrow M(x=\tfrac{L}{2})=0 \qquad C_2 = -\frac{5F_d L_d}{32}$$

$$\frac{M}{EI} = \frac{d^2\Delta}{dx^2} = \frac{1}{EI}\left[\frac{3F_d x^2}{4L_d} - \frac{F_d x^4}{2L_d^3} - \frac{5F_d L_d}{32}\right]$$

$$\frac{d\Delta}{dx} = \frac{1}{EI}\left[\frac{F_d x^3}{4L_d} - \frac{F_d x^5}{10L_d^3} - \frac{5F_d L_d x}{32} + C_3\right] \qquad \rightarrow \Delta/x = 0 \text{ at } x=0 \rightarrow C_3 = 0$$

$$\Delta = \frac{1}{EI}\left[\frac{F_d x^4}{16L_d} - \frac{F_d x^6}{60L_d^3} - \frac{5F_d L_d x^2}{64} + C_4\right]$$

$$\Delta(x=\tfrac{L}{2}) = 0 \rightarrow C_4 = 0.01589F_d L_d^3$$

$$\Delta = \frac{1}{EI}\left[\frac{F_d x^4}{16L_d} - \frac{F_d x^1}{60L_d^3} - \frac{5F_d L_d x^2}{64} + 0.01589F_d L_d^3\right]$$

$$\Delta_{max} = \Delta(0) = \frac{0.01589F_d L_d^3}{EI}$$

Appendix C - Linear Static Procedure

Determine Period

Diaphragm Span (ft):	30	
Diaphragm Length (ft):	30	
Diaphragm Tk. (in):	1.00	
Diaphragm Mod. (psi):	1500	
Diaphragm I (in^4):	3,888,000	
Floor Dead Load (psf):	25	
Inertial Diaphragm Force (lbs):	22,500	
Max. Diaphragm Deflection (in):	2.86	
Approximate Period T (s):	0.47	
Trib. Weight of Building	(k)	(kg)
Floor One:	83.25	37,747
Floor Two:	83.25	37,747

Calculate Spectral Acceleration:

BSE-1:		BSE-2:	
S_a :	0.127	S_a :	0.448

Calculate Pseudo Lateral Load:

BSE-1		BSE-2	
Factors:		**Factors:**	
C_1:	1.00	C_1:	1.01
C_2:	1.00	C_2:	1.00
C_3:	1.00	C_3:	1.00
C_m:	1.00	C_m:	1.00
Pseudo Lateral Load (k):		**Pseudo Lateral Load (k):**	
Floor One (k):	10.58	Floor One (k):	37.30
Floor Two (k):	10.58	Floor Two (k):	37.30

Appendix C - Linear Static Procedure

Design Forces:

BSE-1		BSE-2	
Deformation Controlled		**Deformation Controlled**	
Story Shear:		**Story Shear:**	
Q_{UD}: Floor Two (k):	10.58	Q_{UD}: Floor Two (k):	37.30
Q_{UD}: Floor One (k):	21.15	Q_{UD}: Floor One (k):	74.59
Force Controlled		**Force Controlled**	
J factor:	1.0		1.6
Story Shear:		**Story Shear:**	
Q_{UF}: Floor Two (k):	10.58	Q_{UF}: Floor Two (k):	22.87
Q_{UF}: Floor One (k):	21.15	Q_{UF}: Floor One (k):	45.73

Acceptance Criteria

BSE-1			BSE-2		
Deformation Controlled			**Deformation Controlled**		
Limit State:	Rocking		Limit State:	Rocking	
Performance Level:	IO		Performance Level:	LS	
Knowledge Factor (κ):	0.75		Knowledge Factor (κ):	0.75	
m factor:	1		m factor:	3	
$m\kappa Q_{CE}$:			$m\kappa Q_{CE}$:		
Floor Two (k):	14.98	**GOOD!**	Floor Two (k):	44.93	**GOOD!**
Floor One (k):	34.74	**GOOD!**	Floor One (k):	104.22	**GOOD!**
Force Controlled			**Force Controlled**		
Limit State:	Toe Crushing		Limit State:	Toe Crushing	
Knowledge factor (κ):	0.75		Knowledge factor (κ):	0.75	
κQ_{CL}:			κQ_{CL}:		
Floor Two (k):	9.34	**NO GOOD!**	Floor Two (k):	9.34	**NO GOOD!**
Floor One (k):	24.43	**GOOD!**	Floor One (k):	24.43	**NO GOOD!**

Appendix C

Linear Dynamic Analysis

Use MotionLAB - SHEARBEAM Software provided by Jerome Connor

Assume shear wall is 2-DOF system [Shear Beam]

Input: $m_1 = 37{,}747\ kg$ $\quad k_1 = 5.383 \times 10^8\ N/m$ $\quad C_1 = 450{,}700\ kg\text{-}s$

$m_2 = 37{,}747\ kg$ $\quad k_2 = 8.702 \times 10^8\ N/m$ $\quad C_2 = 573{,}000\ kg\text{-}s$

- Damping was assumed to be 5% according to FEMA 356 procedure
- Properties same as used for LSP

SHEARBEAM Output:
$$\Phi_1 = \begin{bmatrix} 0.74 \\ 1 \end{bmatrix} \qquad \Phi_2 = \begin{bmatrix} 1 \\ -0.74 \end{bmatrix}$$

$$\Gamma_1 = 1.125 \qquad \Gamma_2 = 0.17$$

$$T_1 = 0.081s\ \left(\omega_1 = 77.6\tfrac{rad}{s}\right) \quad T_2 = 0.027s\ \left(\omega_2 = 232.7\tfrac{rad}{s}\right)$$

$$\xi_1 = 0.032 \qquad \xi_2 = 0.080$$

Use Spectral Accelerations: $\quad$ BSE-1 → $S_s = 0.144g = 1.41\ m/s^2$

$$S_d = \frac{S_a}{\omega_1^2} = \frac{1.41}{77.6^2} = 2.34 \times 10^{-4}\ m$$

$$q_{max} = \Gamma \cdot S_{dmax} = 1.125 \times 2.34 \times 10^{-4}\ m = 2.63 \times 10^{-4}\ m$$

For shear building $U = q \cdot \Phi$ $\qquad U = 2.63 \times 10^{-4}\ m \begin{bmatrix} 0.74 \\ 1 \end{bmatrix} = \begin{bmatrix} 1.95 \times 10^{-4} \\ 2.63 \times 10^{-4} \end{bmatrix} m$ [BSE-1]

→ Second mode response is negligible

BSE-2: $S_s = 0.448g = 4.39\ m/s^2$

$$S_d = \frac{4.39}{77.6^2} = 7.30 \times 10^{-4}\ m$$

$$q_{max} = 1.125 \times 7.30 \times 10^{-4} = 8.21 \times 10^{-4}$$

$U = q_{max}\,\Phi = \begin{bmatrix} 6.08 \times 10^{-4} \\ 8.21 \times 10^{-4} \end{bmatrix} m$ [BSE-2]

Convert to inches: $U_{BSE\text{-}1} = \begin{bmatrix} 0.0077 \\ 0.010 \end{bmatrix}$ inches $\qquad U_{BSE\text{-}2} = \begin{bmatrix} 0.024 \\ 0.032 \end{bmatrix}$ inches

Since failure modes are force-controlled, find force in each story

BSE-1:
- Story 1: $F_1 = k_1 u_1 = 1584\ k/in \cdot 0.01 = \boxed{15.8^k}$
- Story 2: $F_2 = k_2 u_2 = 2222\ k/in \cdot [0.01 - 0.0077] = \boxed{5.1^k}$

BSE-2:
- $F_1 = \boxed{38.0^k}$
- $F_2 = \boxed{17.8^k}$

Create Force-Drift Curves for First Floor Piers

Pier One

					Mode	Strength
heff (ft):	9.00		**Failure Mode:**	**DC (Q_{CE}):**	Rocking	9.08
L (ft):	5.00			**FC (Q_{CL}):**	Toe Crushing	7.40

	Expected	Lower-Bound
Elastic Stiffness (k/in):	498	383

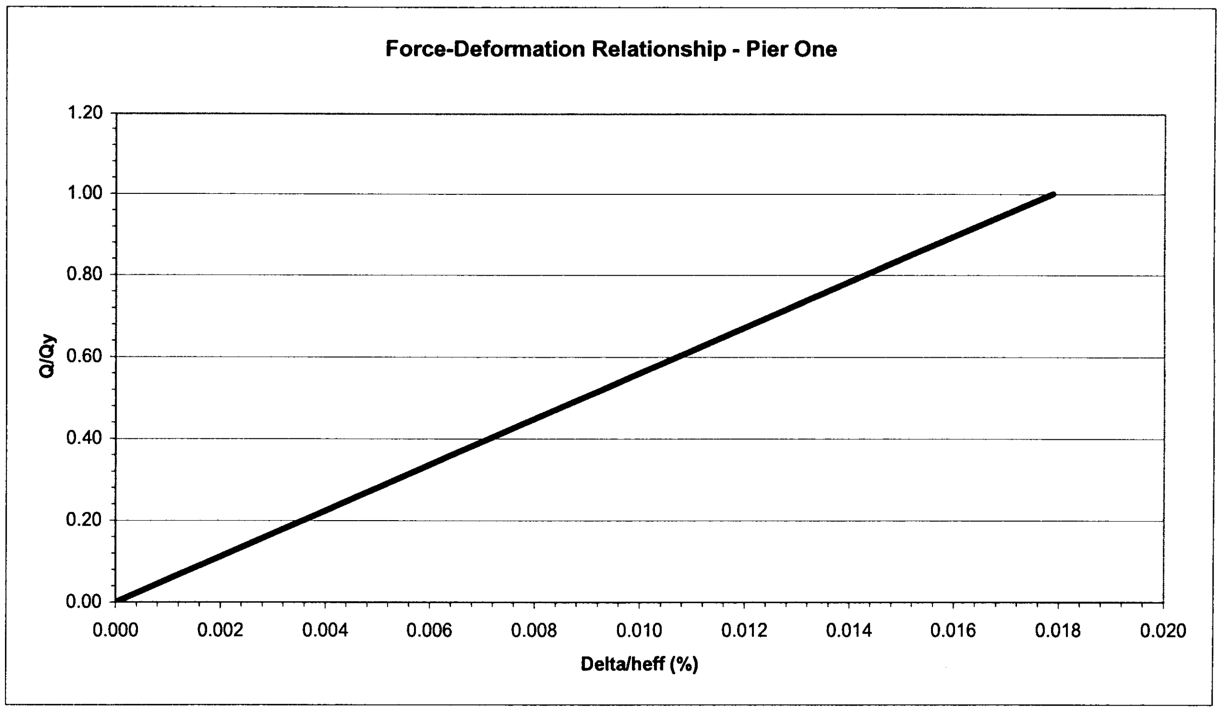
DC or FC?	**FC**	**FORCE CONTROLLED**
Knowledge Factor:		**0.75**
Reduced Strength:		**5.55**

c (%):	0
d (%):	0
e (%):	0

Drift at yield (in):	0.019
Δ_{eff}/h_{eff} (%):	0.018

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.018	1.00
0.017	0.94
0.016	0.89
0.015	0.83
0.014	0.78
0.013	0.72
0.012	0.66
0.011	0.61
0.010	0.55
0.009	0.50
0.008	0.44
0.007	0.38
0.006	0.33
0.005	0.27
0.004	0.22
0	0.00



Create Force-Drift Curves for First Floor Piers

Pier Two

heff (ft):	5.00			**Mode**	**Strength**
L (ft):	4.00	**Failure Mode:**	**DC (Q_{CE}):**	Rocking	10.45
			FC (Q_{CL}):	Toe Crushing	7.07

	Expected	Lower-Bound
Elastic Stiffness (k/in):	1014	780

DC or FC? **FC** **FORCE CONTROLLED**

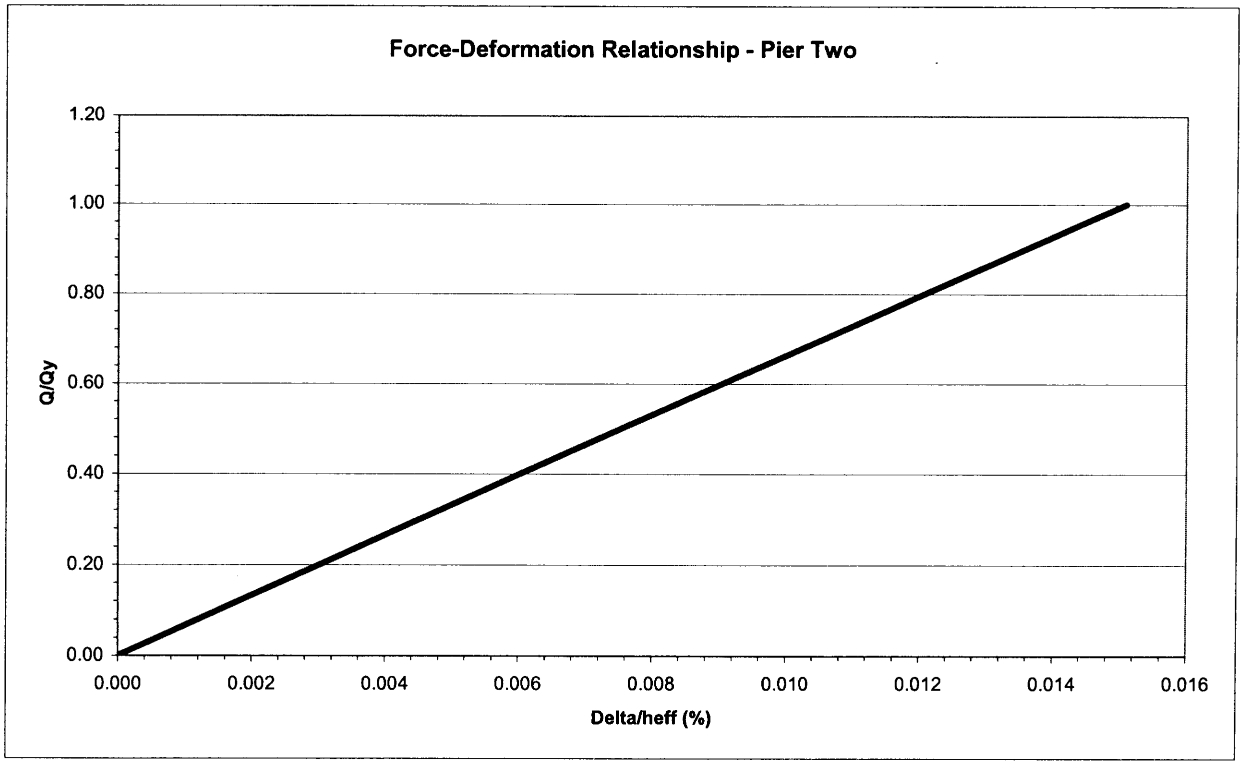
Knowledge Factor: **0.75**

Reduced Strength: **5.30**

c (%):	0
d (%):	0
e (%):	0
Drift at yield (in):	0.009
Δ_{eff}/h_{eff} (%):	0.015

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.015	1.00
0.014	0.93
0.013	0.87
0.012	0.80
0.011	0.74
0.010	0.67
0.009	0.60
0.008	0.54
0.007	0.47
0.006	0.40
0.005	0.34
0.004	0.27
0.003	0.21
0.002	0.14
0	0.00



Create Force-Drift Curves for First Floor Piers

Pier Three

heff (ft):	5.00		**Failure Mode:**		**Mode**	**Strength**
L (ft):	4.00			**DC (Q_{CE}):**	Rocking	10.45
				FC (Q_{CL}):	Toe Crushing	7.07

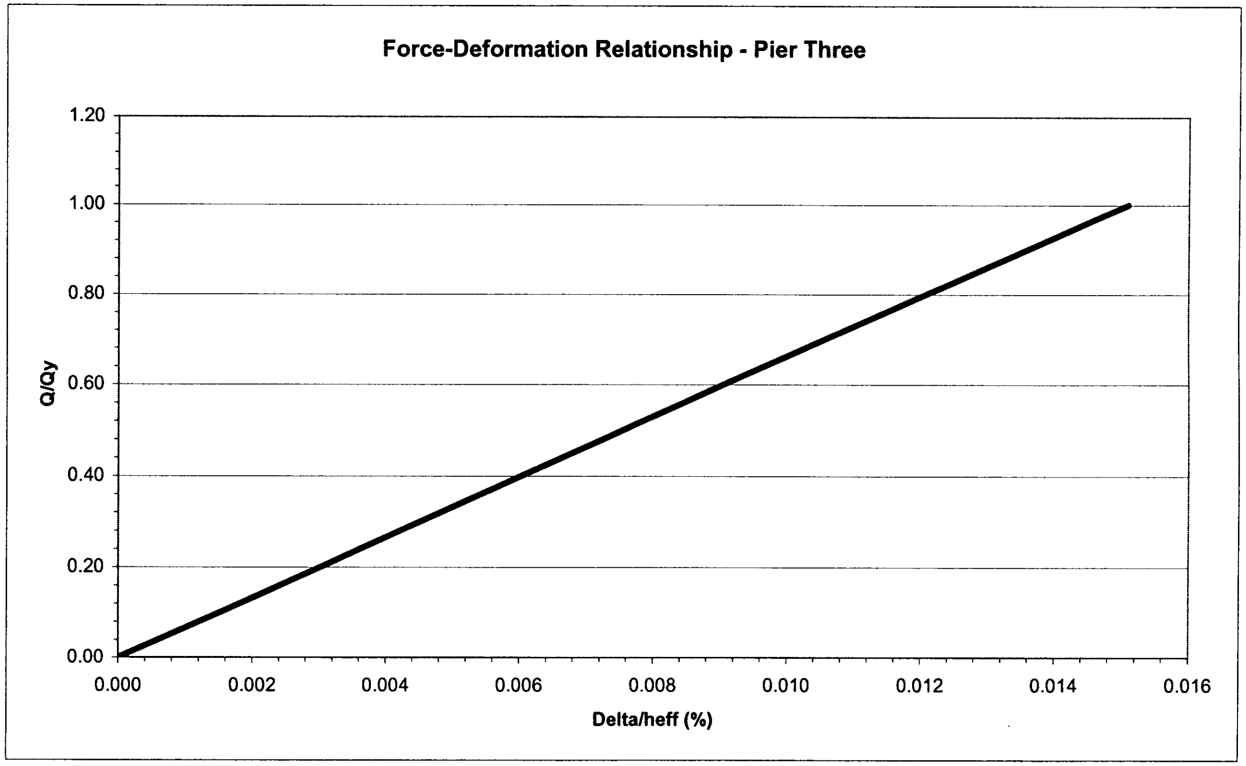
	Expected	Lower-Bound
Elastic Stiffness (k/in):	1014	780

DC or FC?	**FC**	**FORCE CONTROLLED**
Knowledge Factor:	**0.75**	
Reduced Strength:	**5.30**	

c (%):	0
d (%):	0
e (%):	0
Drift at yield (in):	0.009
Δ_{eff}/h_{eff} (%):	0.015

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.015	1.00
0.014	0.93
0.013	0.87
0.012	0.80
0.011	0.74
0.010	0.67
0.009	0.60
0.008	0.54
0.007	0.47
0.006	0.40
0.005	0.34
0.004	0.27
0.003	0.21
0.002	0.14
0	0.00



Create Force-Drift Curves for First Floor Piers

Pier Four

heff (ft):	5.00	Failure Mode:		**Mode**	**Strength**
L (ft):	5.00		**DC (Q_{CE}):**	BJS	16.10
			FC (Q_{CL}):	Toe Crushing	11.04

	Expected	Lower-Bound
Elastic Stiffness (k/in):	1471	1131

DC or FC? **FC** **FORCE CONTROLLED**

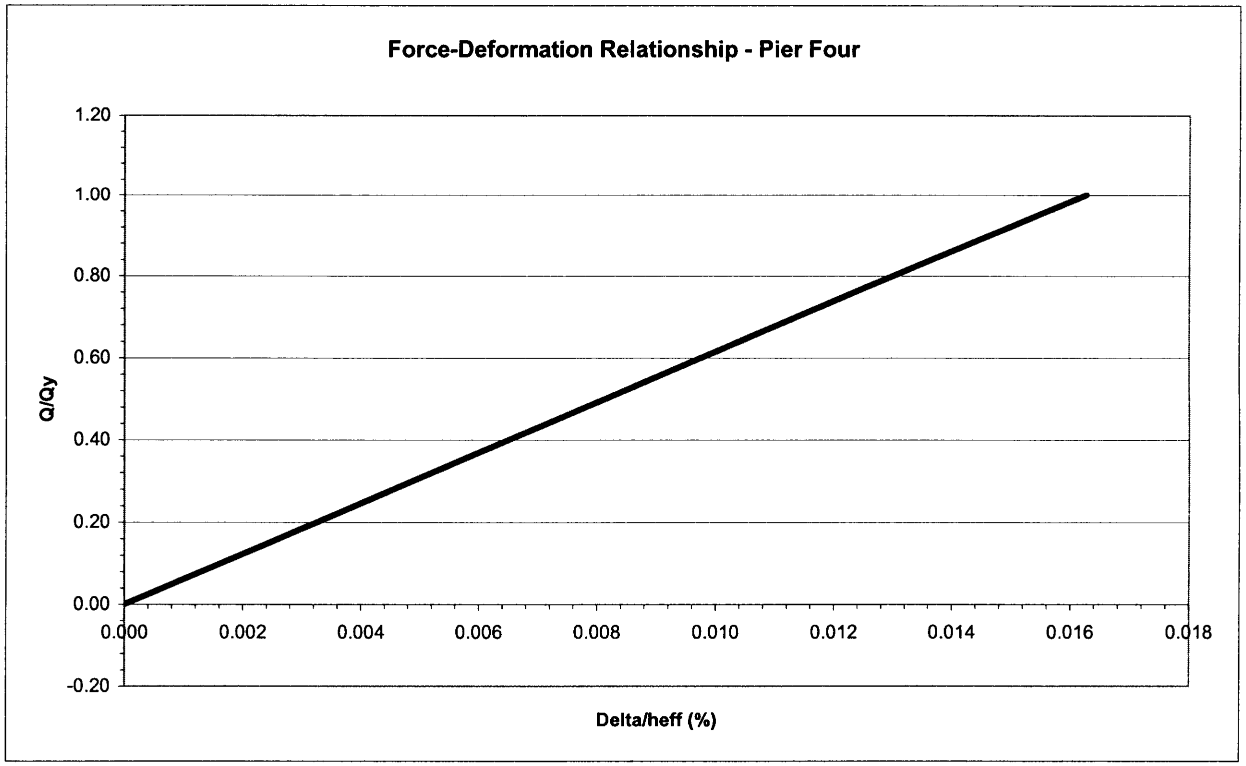
Knowledge Factor: **0.75**

Reduced Strength: **8.28**

c (%):	0
d (%):	0
e (%):	0
Drift at yield (in):	0.010
Δ_{eff}/h_{eff} (%):	0.016

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.016	1.00
0.015	0.94
0.014	0.88
0.013	0.82
0.012	0.75
0.011	0.69
0.010	0.63
0.009	0.57
0.008	0.51
0.007	0.45
0.006	0.39
0.005	0.32
0.004	0.26
0.003	0.20
0.002	0.14
0	0.00



Create Force-Drift Curves for Second Floor Piers

Pier Five

heff (ft):	5.00			
L (ft):	5.00			

		Mode	Strength
Failure Mode:	**DC (Q_{CE}):**	Rocking	6.09
	FC (Q_{CL}):	Toe Crushing	3.80

	Expected	Lower-Bound
Elastic Stiffness (k/in):	1471	1131

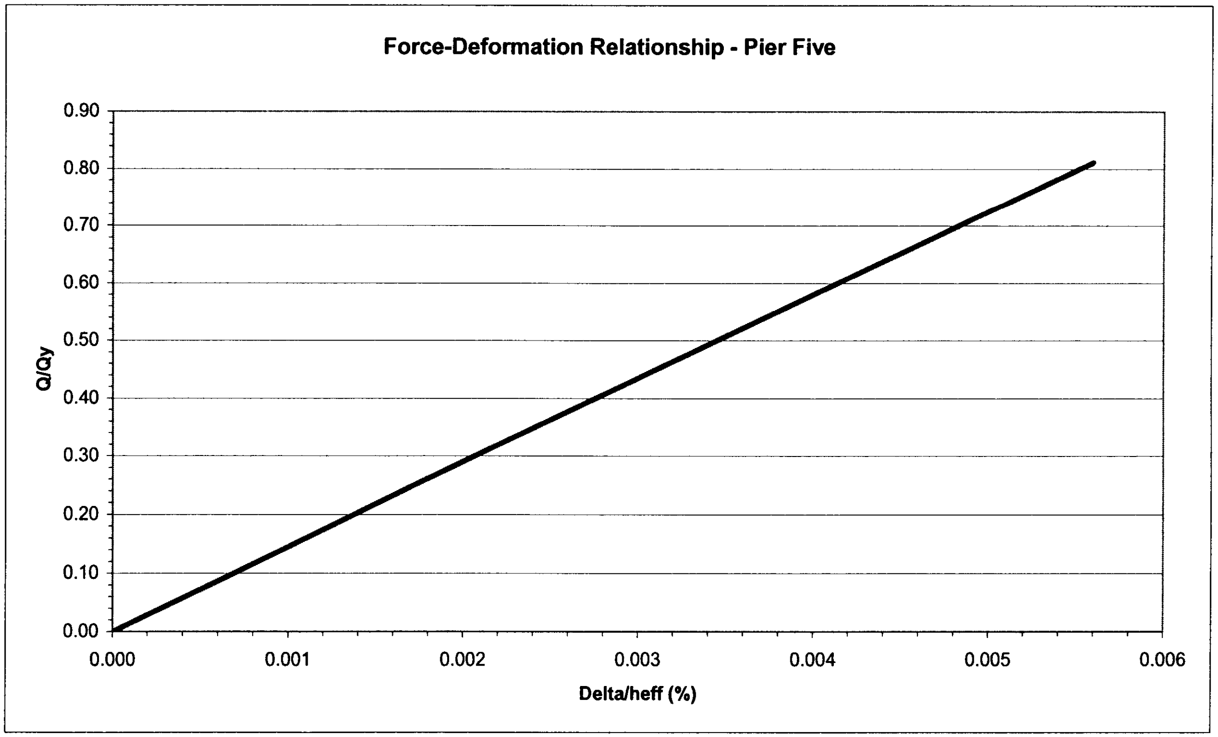
DC or FC?	**FC**	**FORCE CONTROLLED**
Knowledge Factor:		**0.75**
Strength:		**2.85**

c (%):	0
d (%):	0
e (%):	0

Drift at yield (in):	0.003
Δ_{eff}/h_{eff} (%):	0.006

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.006	0.81
0.005	0.67
0.004	0.52
0.003	0.38
0.002	0.23
0.001	0.09
0.000	0.00



Create Force-Drift Curves for Second Floor Piers

Pier Six

heff (ft):	5.00	**Failure Mode:**		**Mode**	**Strength**
L (ft):	4.00		**DC (Q_{CE}):**	Rocking	3.90
			FC (Q_{CL}):	Toe Crushing	2.43

	Expected	Lower-Bound
Elastic Stiffness (k/in):	1014	780

DC or FC? **FC** **FORCE CONTROLLED**

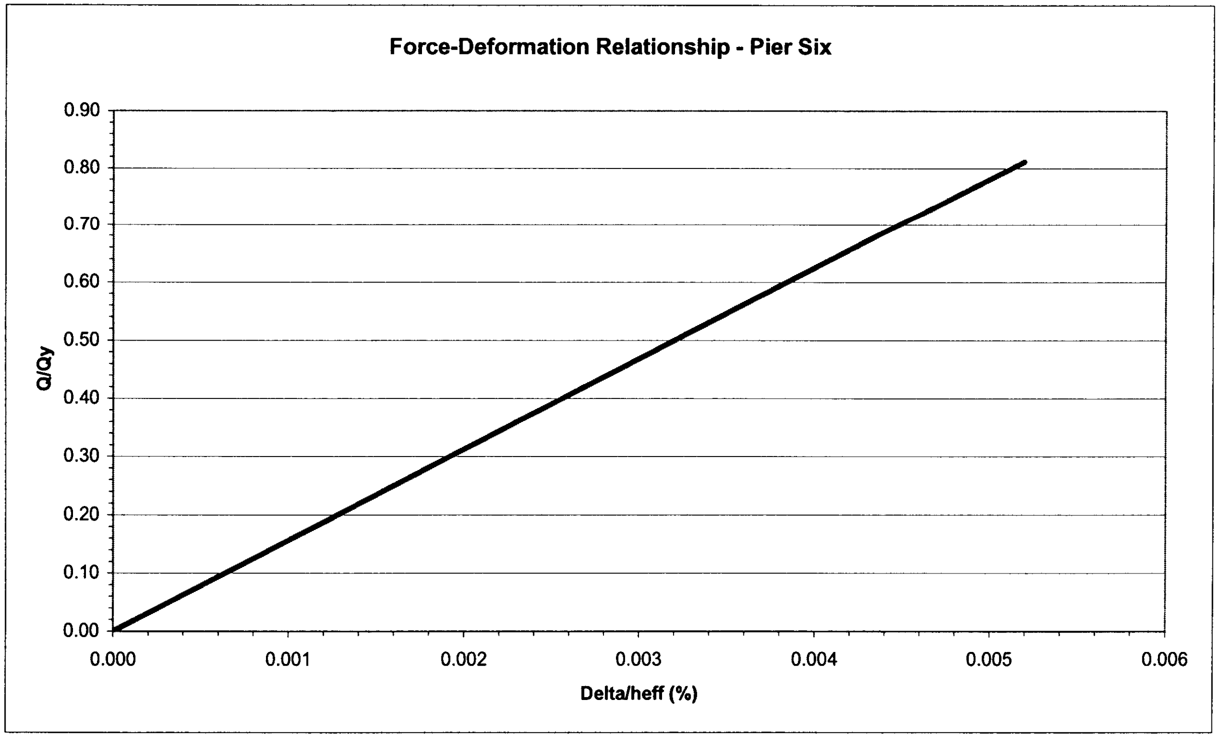
Knowledge Factor: **0.75**

Strength: **1.82**

c (%):	0
d (%):	0
e (%):	0
Drift at yield (in):	0.003
Δ_{eff}/h_{eff} (%):	0.005

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.005	0.81
0.004	0.65
0.003	0.50
0.002	0.34
0.001	0.19
0.000	0.03
0.000	0.00



Create Force-Drift Curves for Second Floor Piers

Pier Seven

heff (ft):	5.00		Failure Mode:		**Mode**	**Strength**
L (ft):	4.00			**DC (Q_{CE}):**	Rocking	3.90
				FC (Q_{CL}):	Toe Crushing	2.43

	Expected	Lower-Bound
Elastic Stiffness (k/in):	1014	780

DC or FC? **FC** **FORCE CONTROLLED**

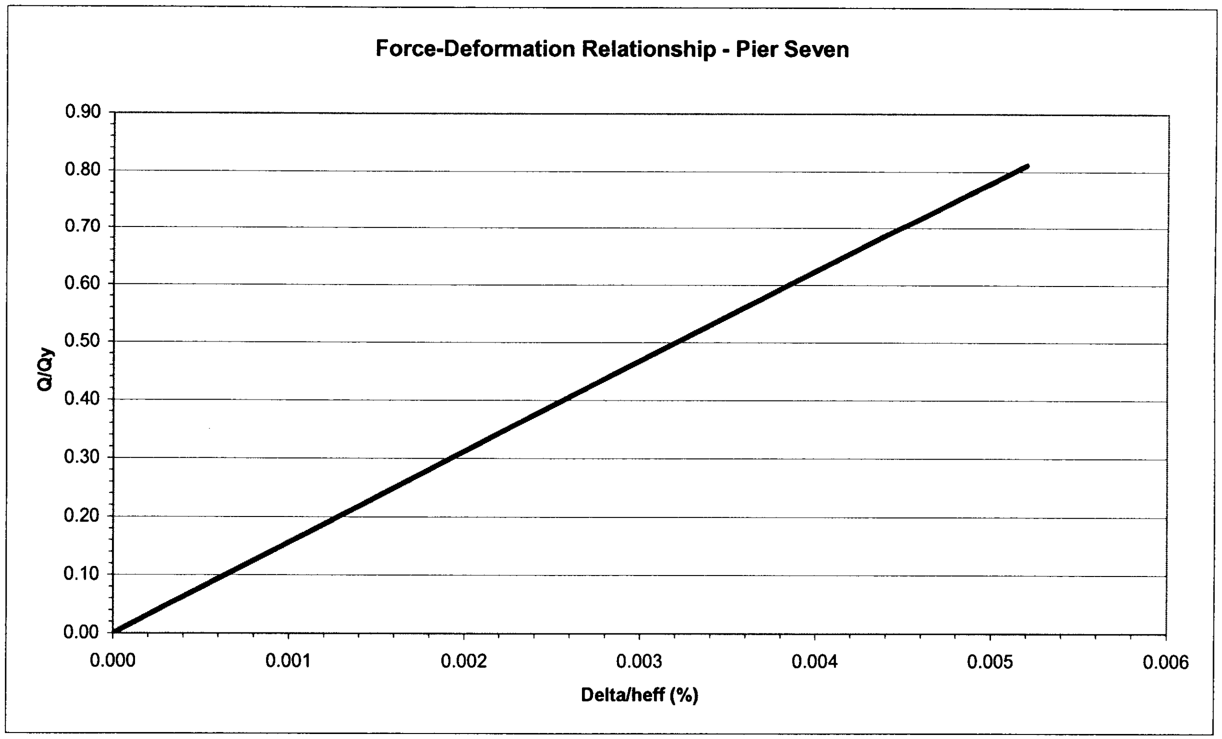
Knowledge Factor: **0.75**

Strength: **1.82**

c (%):	0
d (%):	0
e (%):	0
Drift at yield (in):	0.003
Δ_{eff}/h_{eff} (%):	0.005

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.005	0.81
0.004	0.65
0.003	0.50
0.002	0.34
0.001	0.19
0.000	0.03
0.000	0.00



Create Force-Drift Curves for Second Floor Piers

Pier Eight

heff (ft):	5.00		**Failure Mode:**		**Mode**	**Strength**
L (ft):	5.00			**DC (Q_{CE}):**	Rocking	6.09
				FC (Q_{CL}):	Toe Crushing	3.80

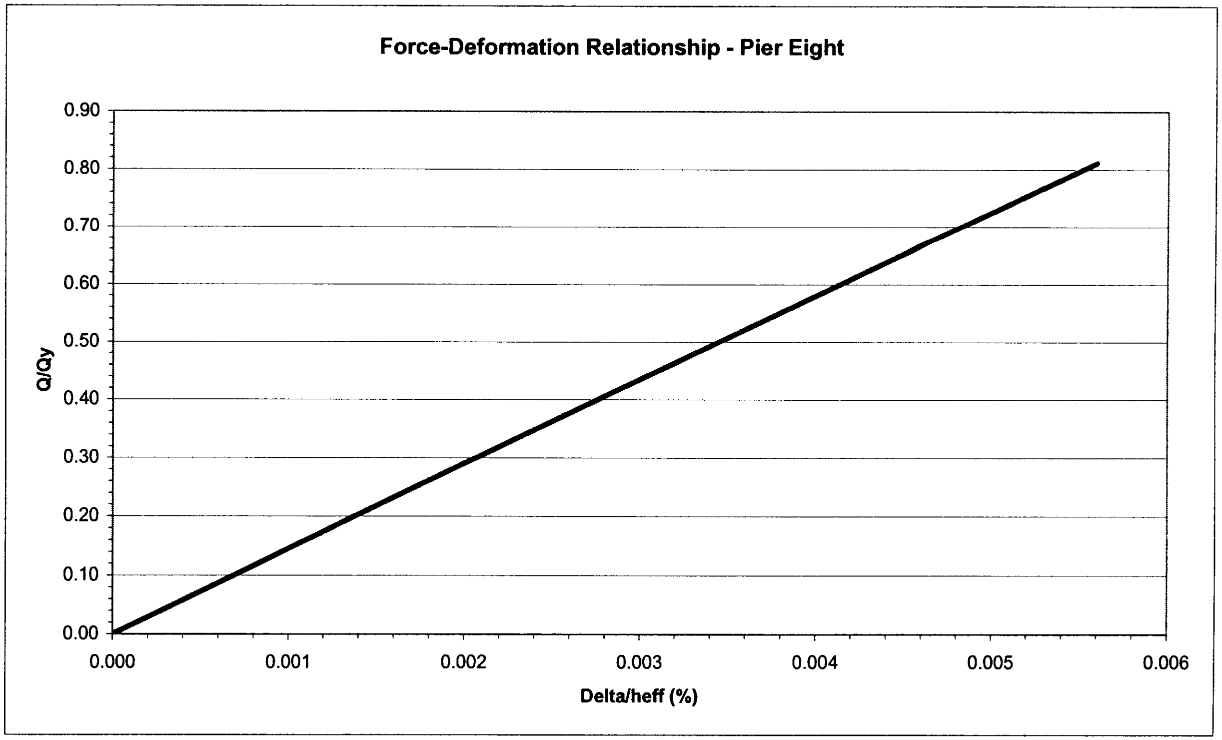
	Expected	Lower-Bound
Elastic Stiffness (k/in):	1471	1131

DC or FC?	**FC**	**FORCE CONTROLLED**
Knowledge Factor:		**0.75**
Strength:		**2.85**

c (%):	0
d (%):	0
e (%):	0
Drift at yield (in):	0.003
Δ_{eff}/h_{eff} (%):	0.006

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.006	0.81
0.005	0.67
0.004	0.52
0.003	0.38
0.002	0.23
0.001	0.09
0.000	0.00



Create Building Force-Deformation Relationship

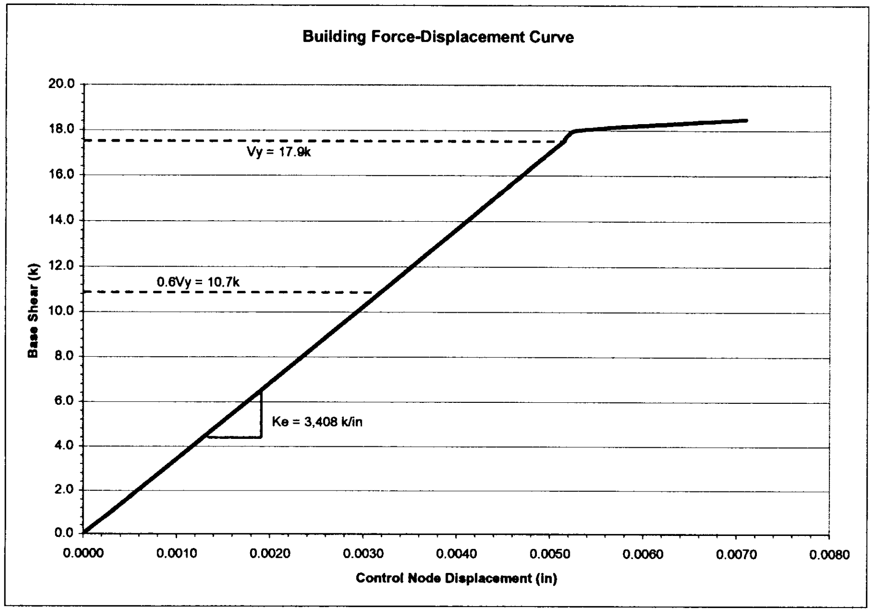
Pier Strengths:				
	Pier One:	5.55	Pier Five:	2.85
	Pier Two:	5.30	Pier Six:	1.82
	Pier Three:	5.30	Pier Seven:	1.82
	Pier Four:	8.28	Pier Eight:	2.85

Uniform lateral load distribution:

V_2 (k)	K_5 (k/in)	K_6 (k/in)	K_7 (k/in)	K_8 (k/in)	K_{second} Total (k/in)	F_5 (k)	F_6 (k)	F_7 (k)	F_8 (k)	U_{2rel} (in)	V_1 (k)	V_{total} (k)
0.00	1131	780	780	1131	3822	0.00	0.00	0.00	0.00	0.0000	0.00	0.0
0.50	1131	780	780	1131	3822	0.15	0.10	0.10	0.15	0.0001	0.50	1.0
1.00	1131	780	780	1131	3822	0.30	0.20	0.20	0.30	0.0003	1.00	2.0
1.50	1131	780	780	1131	3822	0.44	0.31	0.31	0.44	0.0004	1.50	3.0
2.00	1131	780	780	1131	3822	0.59	0.41	0.41	0.59	0.0005	2.00	4.0
2.50	1131	780	780	1131	3822	0.74	0.51	0.51	0.74	0.0007	2.50	5.0
3.00	1131	780	780	1131	3822	0.89	0.61	0.61	0.89	0.0008	3.00	6.0
3.50	1131	780	780	1131	3822	1.04	0.71	0.71	1.04	0.0009	3.50	7.0
4.00	1131	780	780	1131	3822	1.18	0.82	0.82	1.18	0.0010	4.00	8.0
4.50	1131	780	780	1131	3822	1.33	0.92	0.92	1.33	0.0012	4.50	9.0
5.00	1131	780	780	1131	3822	1.48	1.02	1.02	1.48	0.0013	5.00	10.0
5.50	1131	780	780	1131	3822	1.63	1.12	1.12	1.63	0.0014	5.50	11.0
6.00	1131	780	780	1131	3822	1.78	1.22	1.22	1.78	0.0016	6.00	12.0
6.50	1131	780	780	1131	3822	1.92	1.33	1.33	1.92	0.0017	6.50	13.0
7.00	1131	780	780	1131	3822	2.07	1.43	1.43	2.07	0.0018	7.00	14.0
7.50	1131	780	780	1131	3822	2.22	1.53	1.53	2.22	0.0020	7.50	15.0
8.00	1131	780	780	1131	3822	2.37	1.63	1.63	2.37	0.0021	8.00	16.0
8.50	1131	780	780	1131	3822	2.52	1.73	1.73	2.52	0.0022	8.50	17.0
8.75	1131	780	780	1131	3822	2.59	1.79	1.79	2.59	0.0023	8.75	17.5
9.00	1131	780	780	1131	3822	2.66	[redacted]	[redacted]	2.66	0.0024	9.00	18.0
9.25	1131	0	0	1131	2263	[redacted]	0.00	0.00	[redacted]	0.0041	9.25	18.5

V_2 (k)	K_1 (k/in)	K_2 (k/in)	K_3 (k/in)	K_4 (k/in)	K_{first} Total (k/in)	F_1 (k)	F_2 (k)	F_3 (k)	F_4 (k)	U_1 (in)	U_{2Total} (in)	Building Drift (%)
0.00	383	780	780	1131	3074	0.00	0.00	0.00	0.00	0.0000	0.0000	0.0000
0.50	383	780	780	1131	3074	0.12	0.25	0.25	0.37	0.0002	0.0003	0.0001
1.00	383	780	780	1131	3074	0.25	0.51	0.51	0.74	0.0003	0.0006	0.0002
1.50	383	780	780	1131	3074	0.37	0.76	0.76	1.10	0.0005	0.0009	0.0003
2.00	383	780	780	1131	3074	0.50	1.01	1.01	1.47	0.0007	0.0012	0.0004
2.50	383	780	780	1131	3074	0.62	1.27	1.27	1.84	0.0008	0.0015	0.0005
3.00	383	780	780	1131	3074	0.75	1.52	1.52	2.21	0.0010	0.0018	0.0006
3.50	383	780	780	1131	3074	0.87	1.78	1.78	2.58	0.0011	0.0021	0.0007
4.00	383	780	780	1131	3074	1.00	2.03	2.03	2.94	0.0013	0.0023	0.0008
4.50	383	780	780	1131	3074	1.12	2.28	2.28	3.31	0.0015	0.0026	0.0009
5.00	383	780	780	1131	3074	1.25	2.54	2.54	3.68	0.0016	0.0029	0.0010
5.50	383	780	780	1131	3074	1.37	2.79	2.79	4.05	0.0018	0.0032	0.0011
6.00	383	780	780	1131	3074	1.50	3.04	3.04	4.42	0.0020	0.0035	0.0012
6.50	383	780	780	1131	3074	1.62	3.30	3.30	4.78	0.0021	0.0038	0.0013
7.00	383	780	780	1131	3074	1.75	3.55	3.55	5.15	0.0023	0.0041	0.0014
7.50	383	780	780	1131	3074	1.87	3.80	3.80	5.52	0.0024	0.0044	0.0015
8.00	383	780	780	1131	3074	1.99	4.06	4.06	5.89	0.0026	0.0047	0.0016
8.50	383	780	780	1131	3074	2.12	4.31	4.31	6.26	0.0028	0.0050	0.0017
8.75	383	780	780	1131	3074	2.18	4.44	4.44	6.44	0.0028	0.0051	0.0018
9.00	383	780	780	1131	3074	2.24	4.57	4.57	6.62	0.0029	0.0053	0.0018
9.25	383	780	780	1131	3074	2.31	4.69	4.69	6.81	0.0030	0.0071	0.0025

Create Building Force-Deformation Relationship



Vy (k):	17.90
0.6Vy (k):	10.74
K_i (k/in):	3408
K_e (k/in):	3408
T_i (s):	0.081
T_e (s):	0.081

Calculate Target Displacement δ_t:

BSE-1		BSE-2	
PL:	IO	PL:	LS
C_0:	1.15	C_0:	1.15
R:	1.32	R:	3.77
C_1:	1.50	C_1:	1.5
C_2:	1.0	C_2:	1.1
C_3:	1.0	C_3:	1.0
S_a:	0.142	S_a:	0.405
δ_t (in):	0.0157	δ_t (in):	0.049

Base Shear at Target Displacement:

V (k):	53.45	V (k):	168.17

Appendix C - Nonlinear Static Procedure

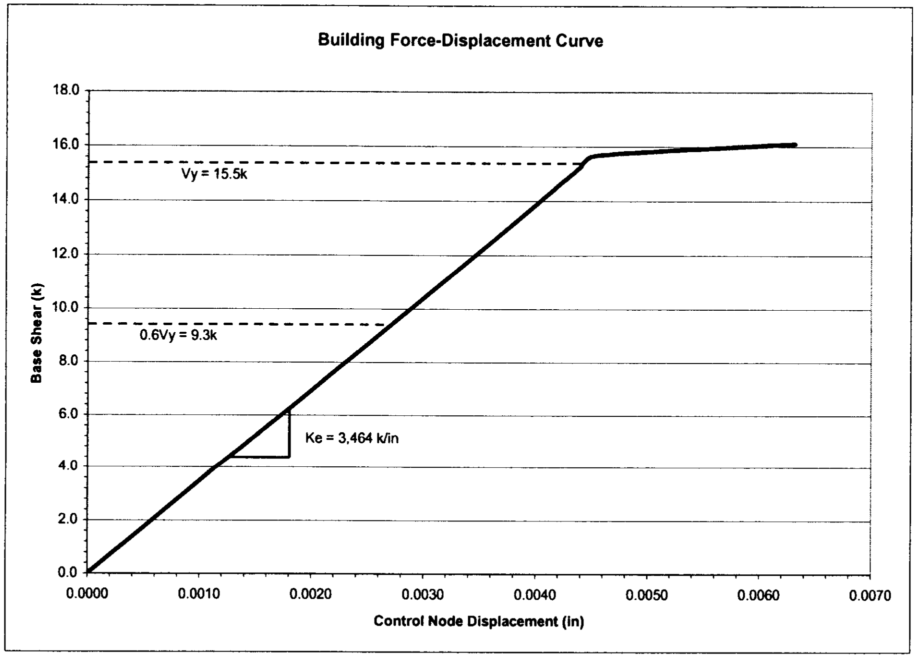
Create Building Force-Deformation Relationship

Pier Strengths:				
	Pier One:	5.55	Pier Five:	2.85
	Pier Two:	5.30	Pier Six:	1.82
	Pier Three:	5.30	Pier Seven:	1.82
	Pier Four:	8.28	Pier Eight:	2.85

Uniform lateral load distribution:

V_2 (k)	K_5 (k/in)	K_6 (k/in)	K_7 (k/in)	K_8 (k/in)	K_{second} Total (k/in)	F_5 (k)	F_6 (k)	F_7 (k)	F_8 (k)	U_{2rel} (in)	V_1 (k)	V_{total} (k)	K_1 (k/in)	K_2 (k/in)	K_3 (k/in)	K_4 (k/in)	K_{first} Total (k/in)	F_1 (k)	F_2 (k)	F_3 (k)	F_4 (k)	U_1 (in)	U_{2Total} (in)	Building Drift (%)
0.00	1131	780	780	1131	3822	0.00	0.00	0.00	0.00	0.0000	0.00	0.0	383	780	780	1131	3074	0.00	0.00	0.00	0.00	0.0000	0.0000	0.0000
0.50	1131	780	780	1131	3822	0.15	0.10	0.10	0.15	0.0001	0.50	1.0	383	780	780	1131	3074	0.12	0.25	0.25	0.37	0.0002	0.0003	0.0001
1.00	1131	780	780	1131	3822	0.30	0.20	0.20	0.30	0.0003	1.00	2.0	383	780	780	1131	3074	0.25	0.51	0.51	0.74	0.0003	0.0006	0.0002
1.50	1131	780	780	1131	3822	0.44	0.31	0.31	0.44	0.0004	1.50	3.0	383	780	780	1131	3074	0.37	0.76	0.76	1.10	0.0005	0.0009	0.0003
2.00	1131	780	780	1131	3822	0.59	0.41	0.41	0.59	0.0005	2.00	4.0	383	780	780	1131	3074	0.50	1.01	1.01	1.47	0.0007	0.0012	0.0004
2.50	1131	780	780	1131	3822	0.74	0.51	0.51	0.74	0.0007	2.50	5.0	383	780	780	1131	3074	0.62	1.27	1.27	1.84	0.0008	0.0015	0.0005
3.00	1131	780	780	1131	3822	0.89	0.61	0.61	0.89	0.0008	3.00	6.0	383	780	780	1131	3074	0.75	1.52	1.52	2.21	0.0010	0.0018	0.0006
3.50	1131	780	780	1131	3822	1.04	0.71	0.71	1.04	0.0009	3.50	7.0	383	780	780	1131	3074	0.87	1.78	1.78	2.58	0.0011	0.0021	0.0007
4.00	1131	780	780	1131	3822	1.18	0.82	0.82	1.18	0.0010	4.00	8.0	383	780	780	1131	3074	1.00	2.03	2.03	2.94	0.0013	0.0023	0.0008
4.50	1131	780	780	1131	3822	1.33	0.92	0.92	1.33	0.0012	4.50	9.0	383	780	780	1131	3074	1.12	2.28	2.28	3.31	0.0015	0.0026	0.0009
5.00	1131	780	780	1131	3822	1.48	1.02	1.02	1.48	0.0013	5.00	10.0	383	780	780	1131	3074	1.25	2.54	2.54	3.68	0.0016	0.0029	0.0010
5.50	1131	780	780	1131	3822	1.63	1.12	1.12	1.63	0.0014	5.50	11.0	383	780	780	1131	3074	1.37	2.79	2.79	4.05	0.0018	0.0032	0.0011
6.00	1131	780	780	1131	3822	1.78	1.22	1.22	1.78	0.0016	6.00	12.0	383	780	780	1131	3074	1.50	3.04	3.04	4.42	0.0020	0.0035	0.0012
6.50	1131	780	780	1131	3822	1.92	1.33	1.33	1.92	0.0017	6.50	13.0	383	780	780	1131	3074	1.62	3.30	3.30	4.78	0.0021	0.0038	0.0013
7.00	1131	780	780	1131	3822	2.07	1.43	1.43	2.07	0.0018	7.00	14.0	383	780	780	1131	3074	1.75	3.55	3.55	5.15	0.0023	0.0041	0.0014
7.50	1131	780	780	1131	3822	2.22	1.53	1.53	2.22	0.0020	7.50	15.0	383	780	780	1131	3074	1.87	3.80	3.80	5.52	0.0024	0.0044	0.0015
8.00	1131	780	780	1131	3822	2.37	1.63	1.63	2.37	0.0021	8.00	16.0	383	780	780	1131	3074	1.99	4.06	4.06	5.89	0.0026	0.0047	0.0016
8.50	1131	780	780	1131	3822	2.52	1.73	1.73	2.52	0.0022	8.50	17.0	383	780	780	1131	3074	2.12	4.31	4.31	6.26	0.0028	0.0050	0.0017
8.75	1131	780	780	1131	3822	2.59	1.79	1.79	2.59	0.0023	8.75	17.5	383	780	780	1131	3074	2.18	4.44	4.44	6.44	0.0028	0.0051	0.0018
9.00	1131	780	780	1131	3822	2.66	1.84	1.84	2.66	0.0024	9.00	18.0	383	780	780	1131	3074	2.24	4.57	4.57	6.62	0.0029	0.0053	0.0018
9.25	1131	0	0	1131	2263	4.63	0.00	0.00	4.63	0.0041	9.25	18.5	383	780	780	1131	3074	2.31	4.69	4.69	6.81	0.0030	0.0071	0.0025

Create Building Force-Deformation Relationship



Vy (k):	15.50
0.6Vy (k):	9.30
K_i (k/in):	3464
K_e (k/in):	3464
T_i (s):	0.470
T_e (s):	0.470

Calculate Target Displacement δ_t:

BSE-1		BSE-2	
PL:	IO	PL:	LS
C_0:	1.15	C_0:	1.15
R:	1.37	R:	4.81
C_1:	1.00	C_1:	1.0
C_2:	1.0	C_2:	1.1
C_3:	1.0	C_3:	1.0
S_a:	0.128	S_a:	0.448
δ_t (in):	0.3174	δ_t (in):	1.250

Base Shear at Target Displacement:

V1 (k):	No Good	V1 (k):	No Good

Appendix D

Modified Linear and Nonlinear Analyses

Appendix D - Modified Linear Static Analysis

Determine Period

Diaphragm Span (ft):	30	
Diaphragm Length (ft):	30	
Diaphragm Tk. (in):	1.00	
Diaphragm Mod. (psi):	1500	
Diaphragm I (in^4):	3,888,000	
Floor Dead Load (psf):	25	
Inertial Diaphragm Force (lbs):	22,500	
Max. Diaphragm Deflection (in):	2.86	
Approximate Period T (s):	0.47	
Trib. Weight of Building	(k)	(kg)
Floor One:	83.25	37,747
Floor Two:	83.25	37,747

Calculate Spectral Acceleration:			
BSE-1:		BSE-2:	
S_a :	0.127	S_a :	0.448

Calculate Pseudo Lateral Load:			
BSE-1		**BSE-2**	
Factors:		**Factors:**	
C_1:	1.00	C_1:	1.01
C_2:	1.00	C_2:	1.00
C_3:	1.00	C_3:	1.00
C_m:	1.00	C_m:	1.00
Pseudo Lateral Load (k):		**Pseudo Lateral Load (k):**	
Floor One (k):	10.58	Floor One (k):	37.30
Floor Two (k):	10.58	Floor Two (k):	37.30

Appendix D - Modified Linear Static Analysis

Design Forces:

BSE-1		BSE-2	
Deformation Controlled		**Deformation Controlled**	
Story Shear:		**Story Shear:**	
Q_{UD}: Floor Two (k):	10.58	Q_{UD}: Floor Two (k):	37.30
Q_{UD}: Floor One (k):	21.15	Q_{UD}: Floor One (k):	74.59
Q_{UD}: Base (k):	21.15	Q_{UD}: Base (k):	74.59
Force Controlled		**Force Controlled**	
J factor:	1.0		1.5
Story Shear:		**Story Shear:**	
Q_{UF}: Floor Two (k):	10.58	Q_{UF}: Floor Two (k):	24.55
Q_{UF}: Floor One (k):	21.15	Q_{UF}: Floor One (k):	49.10
Q_{UF}: Base (k):	21.15	Q_{UF}: Base (k):	49.10

Acceptance Criteria

BSE-1			BSE-2		
Deformation Controlled			**Deformation Controlled**		
Limit State:	Rocking		Limit State:	Rocking	
Performance Level:	IO		Performance Level:	LS	
Knowledge Factor (κ):	0.75		Knowledge Factor (κ):	0.75	
m factor:	1		m factor:	3	
$m\kappa Q_{CE}$:			$m\kappa Q_{CE}$:		
Floor Two (k):	11.02	**GOOD!**	Floor Two (k):	33.07	**GOOD!**
Floor One (k):	24.72	**GOOD!**	Floor One (k):	74.15	**GOOD!**
Force Controlled			**Force Controlled**		
Limit State:	Diag. Tension		Limit State:	Diag. Tension	
Knowledge factor (κ):	0.75		Knowledge factor (κ):	0.75	
κQ_{CL}:			κQ_{CL}:		
Floor Two (k):	31.87	**GOOD!**	Floor Two (k):	31.87	**GOOD!**
Floor One (k):	22.04	**GOOD!**	Floor One (k):	31.87	**NO GOOD!**

Create Force-Drift Curves for First Floor Piers

Pier One

heff (ft):	10.30	**Failure Mode:**		**Mode**	**Strength**
L (ft):	5.00		**DC (Q_{CE}):**	Rocking	7.93
			FC (Q_{CL}):	Diag. Tension	10.23

	Expected	Lower-Bound
Elastic Stiffness (k/in):	371	285

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Reduced Strength: **7.93**

c (%):	0.68
d (%):	0.82
e (%):	1.65
x (%):	1.67
Drift at yield (in):	0.021
Δ_{eff}/h_{eff} (%):	0.017

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.017	1.00
0.016	0.94
0.015	0.88
0.014	0.83
0.013	0.77
0.012	0.71
0.011	0.65
0.010	0.60
0.009	0.54
0.008	0.48
0.007	0.42
0.006	0.36
0.005	0.31
0.004	0.25
0.003	0.19
0	0.00

Inelastic BC:

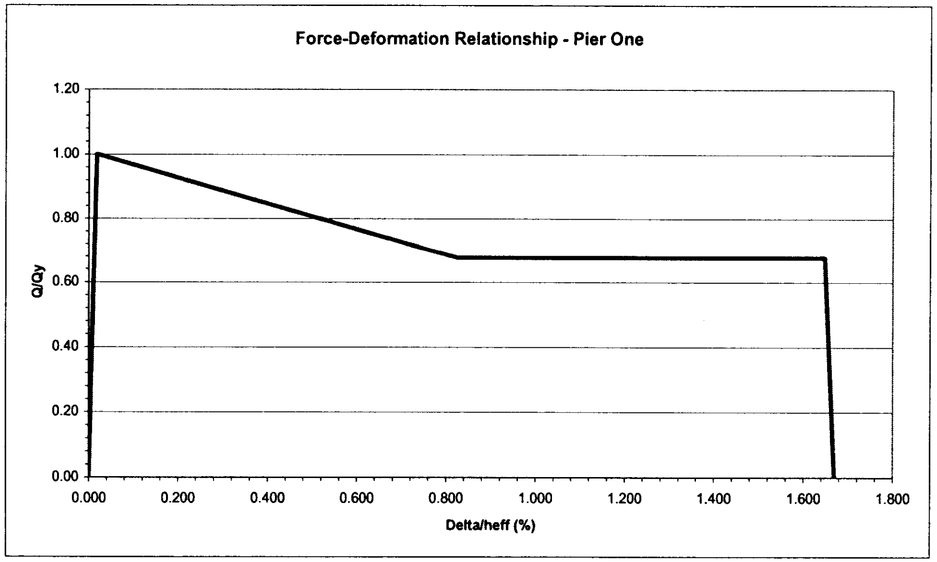
0.017	1.00
0.82	0.68
slope (k/in):	-2.58

Inelastic CD:

0.82	0.68
1.65	0.68

Inelastic CD:

1.65	0.68
1.67	0.00



Create Force-Drift Curves for First Floor Piers

Pier Two

heff (ft):	9.85	**Failure Mode:**		**Mode**	**Strength**
L (ft):	4.00		**DC (Q_{CE}):**	Rocking	5.31
			FC (Q_{CL}):	Diag. Tension	6.84

	Expected	Lower-Bound
Elastic Stiffness (k/in):	244	188

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Reduced Strength: **5.31**

c (%):	0.68
d (%):	0.98
e (%):	1.97
x (%):	1.99

Drift at yield (in):	0.022
Δ_{eff}/h_{eff} (%):	0.018

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.018	1.00
0.017	0.95
0.016	0.89
0.015	0.84
0.014	0.78
0.013	0.73
0.012	0.67
0.011	0.62
0.010	0.57
0.009	0.51
0.008	0.46
0.007	0.40
0.006	0.35
0.005	0.29
0.004	0.24
0	0.00

Inelastic BC:

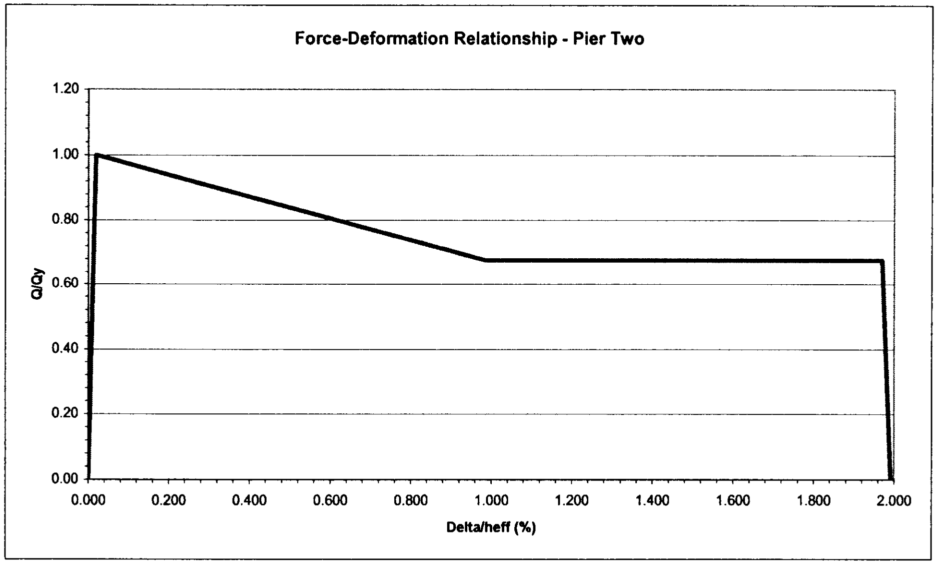
0.018	1.00
0.98	0.68
slope (k/in):	-1.51

Inelastic CD:

0.98	0.68
1.97	0.68

Inelastic CD:

1.97	0.68
1.99	0.00



Create Force-Drift Curves for First Floor Piers

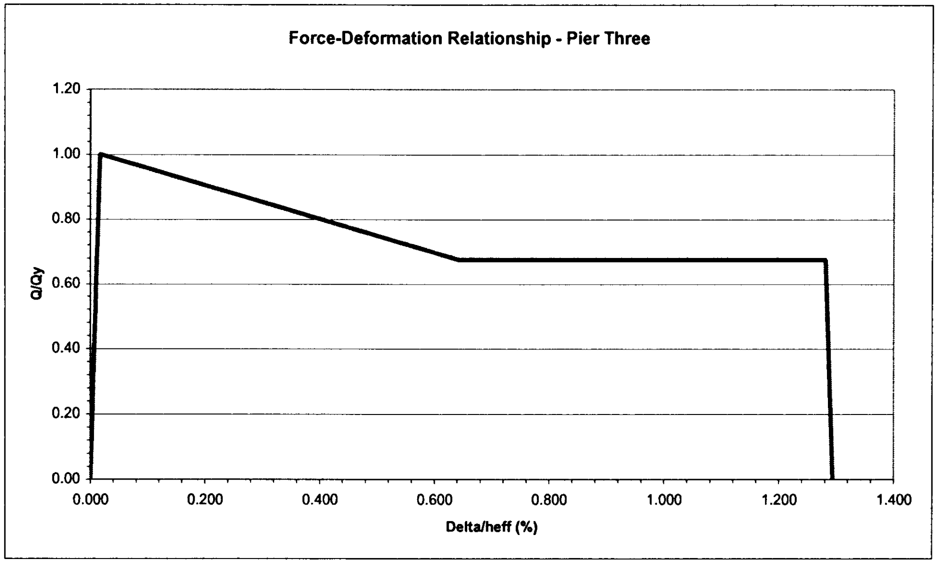
Pier Three

					Mode	**Strength**
heff (ft):	6.40		**Failure Mode:**	**DC (Q_{CE}):**	Rocking	8.16
L (ft):	4.00			**FC (Q_{CL}):**	Diag. Tension	10.53

	Expected	Lower-Bound
Elastic Stiffness (k/in):	635	489

DC or FC?	**DC**	**DEFORMATION CONTROLLED**	
	Knowledge Factor:	**0.75**	
	Reduced Strength:	**8.16**	

c (%):	0.68
d (%):	0.64
e (%):	1.28
x (%):	1.29
Drift at yield (in):	0.013
Δ_{eff}/h_{eff} (%):	0.017



Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.017	1.00
0.016	0.94
0.015	0.88
0.014	0.82
0.013	0.76
0.012	0.70
0.011	0.64
0.010	0.58
0.009	0.52
0.008	0.46
0.007	0.40
0.006	0.34
0.005	0.28
0.004	0.22
0.003	0.16
0	0.00

Inelastic BC:

0.017	1.00
0.64	0.68
slope (k/in):	-5.52

Inelastic CD:

0.64	0.68
1.28	0.68

Inelastic CD:

1.28	0.68
1.29	0.00

Create Force-Drift Curves for First Floor Piers

Pier Four

heff (ft):	7.07			**Mode**	**Strength**
L (ft):	5.00	**Failure Mode:**	**DC (Q_{CE}):**	Rocking	11.55
			FC (Q_{CL}):	Diag. Tension	14.90

	Expected	Lower-Bound
Elastic Stiffness (k/in):	809	622

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Reduced Strength: **11.55**

c (%):	0.68
d (%):	0.57
e (%):	1.13
x (%):	1.15
Drift at yield (in):	0.014
Δ_{eff}/h_{eff} (%):	0.017

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.017	1.00
0.016	0.94
0.015	0.88
0.014	0.82
0.013	0.76
0.012	0.70
0.011	0.64
0.010	0.58
0.009	0.52
0.008	0.47
0.007	0.41
0.006	0.35
0.005	0.29
0.004	0.23
0.003	0.17
0	0.00

Inelastic BC:

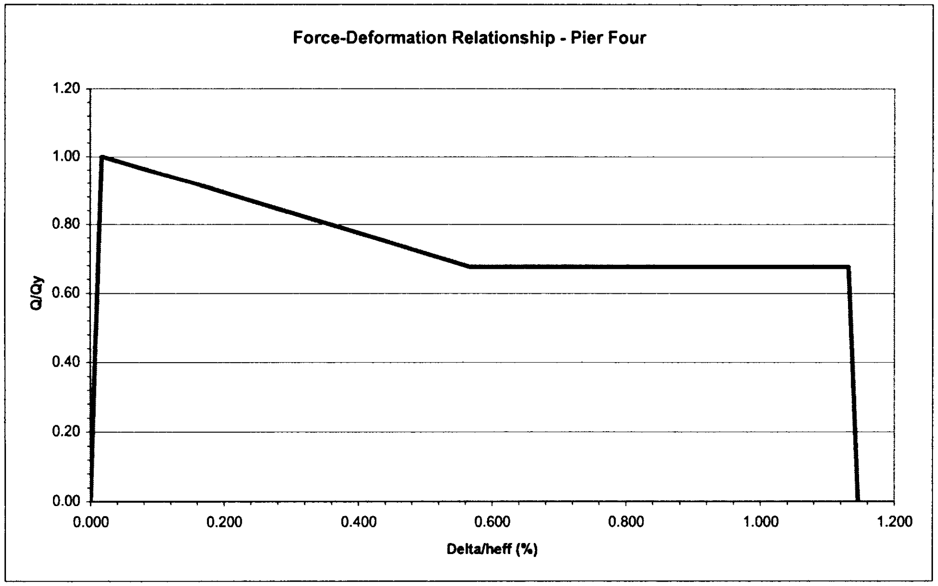
0.017	1.00
0.57	0.68
slope (k/in):	-8.04

Inelastic CD:

0.57	0.68
1.13	0.68

Inelastic CD:

1.13	0.68
1.15	0.00



Create Force-Drift Curves for Second Floor Piers

Pier Five

					Mode	Strength
heff (ft):	7.07		Failure Mode:	DC (Q_{CE}):	Rocking	4.31
L (ft):	5.00			FC (Q_{CL}):	Diag. Tension	8.61

	Expected	Lower-Bound
Elastic Stiffness (k/in):	809	622

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Strength: **4.31**

c (%):	0.624
d (%):	0.566
e (%):	1.131
x (%):	1.137

Drift at yield (in):	0.005
Δ_{eff}/h_{eff} (%):	0.006

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.006	1.00
0.005	0.84
0.004	0.68
0.003	0.52
0.002	0.36
0.001	0.20
0.000	0.04
0.000	0.00

Inelastic (BC):

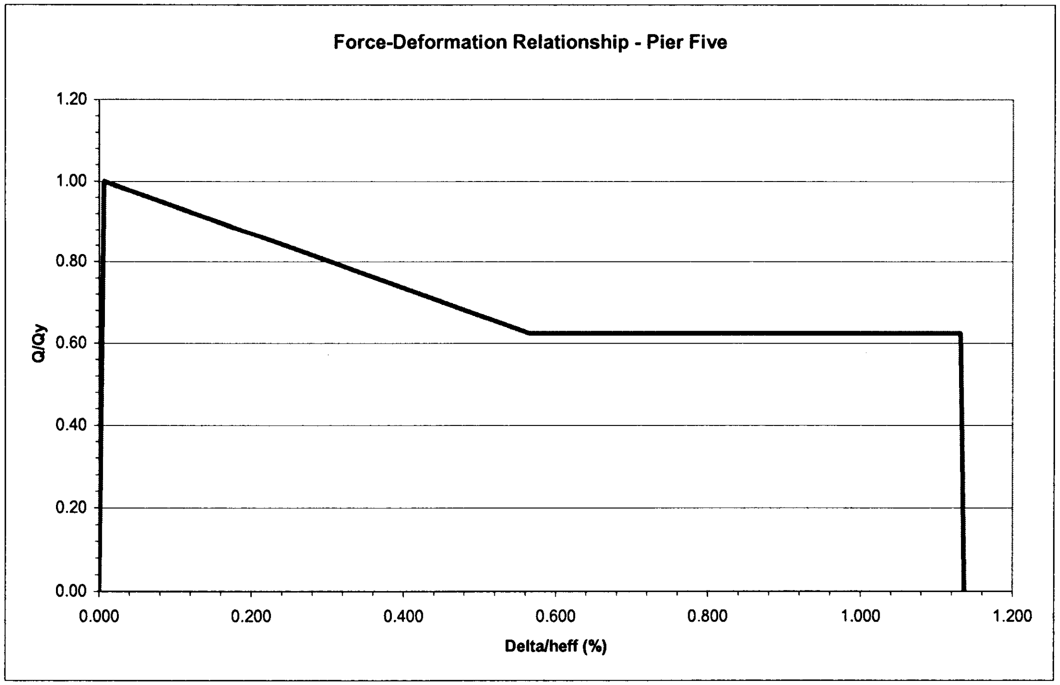
0.006	1.000
0.566	0.624
slope (k/in):	-3.41

Inelastic (CD):

0.566	0.624
1.131	0.624

Inelastic (DE):

1.13	0.624
1.14	0.000



Create Force-Drift Curves for Second Floor Piers

Pier Six

heff (ft):	6.40	**Failure Mode:**		**Mode**	**Strength**
			DC (Q_{CE}):	Rocking	3.04
L (ft):	4.00		**FC (Q_{CL}):**	Diag. Tension	6.08

	Expected	Lower-Bound
Elastic Stiffness (k/in):	635	489

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Strength: **3.04**

c (%):	0.624
d (%):	0.640
e (%):	1.281
x (%):	1.285

Drift at yield (in):	0.005
Δ_{eff}/h_{eff} (%):	0.006

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.006	1.00
0.005	0.84
0.004	0.68
0.003	0.52
0.002	0.36
0.001	0.20
0.000	0.00

Inelastic (BC):

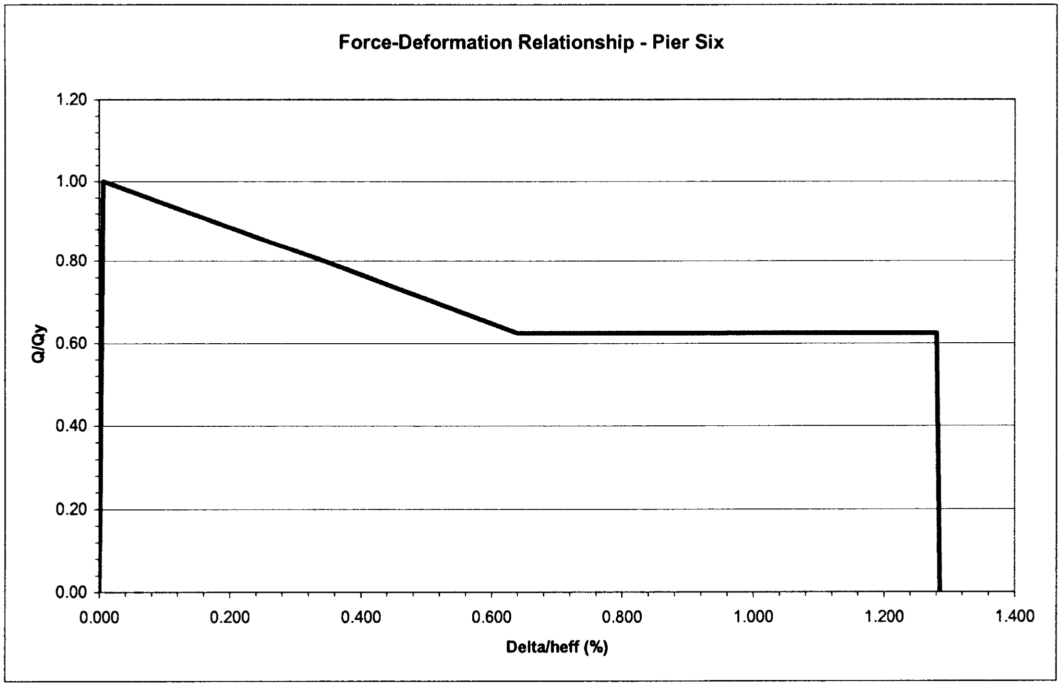
0.006	1.000
0.640	0.624
slope (k/in):	-2.35

Inelastic (CD):

0.640	0.624
1.281	0.624

Inelastic (DE):

1.28	0.624
1.29	0.000



Create Force-Drift Curves for Second Floor Piers

Pier Seven

heff (ft):	6.40	Failure Mode:		**Mode**	**Strength**
L (ft):	4.00		DC (Q_{CE}):	Rocking	3.04
			FC (Q_{CL}):	Diag. Tension	6.08

	Expected	Lower-Bound
Elastic Stiffness (k/in):	635	489

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Strength: **3.04**

c (%):	0.624
d (%):	0.640
e (%):	1.281
x (%):	1.285
Drift at yield (in):	0.005
Δ_{eff}/h_{eff} (%):	0.006

Elastic:

Δ_{eff}/h_{eff} (%):	Q/Qy
0.006	1.00
0.005	0.84
0.004	0.68
0.003	0.52
0.002	0.36
0.001	0.20
0.000	0.04
0.000	0.00

Inelastic (BC):

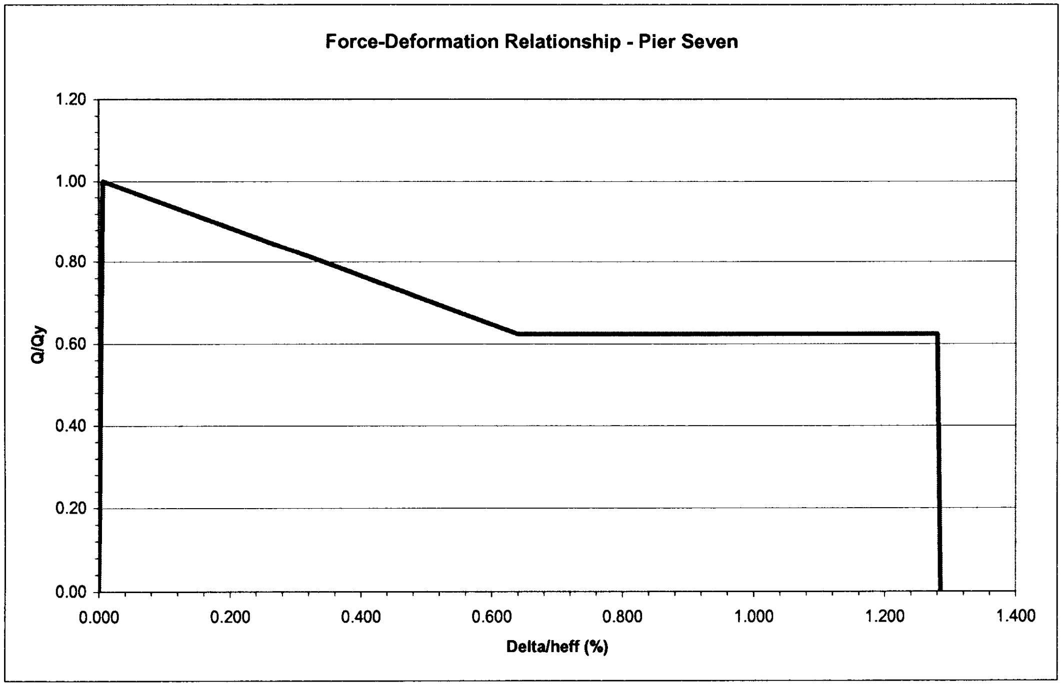
0.006	1.000
0.640	0.624
slope (k/in):	-2.35

Inelastic (CD):

0.640	0.624
1.281	0.624

Inelastic (DE):

1.28	0.624
1.29	0.000



Create Force-Drift Curves for Second Floor Piers

Pier Eight

heff (ft):	7.07				**Mode**	**Strength**
L (ft):	5.00		**Failure Mode:**	**DC (Q_{CE}):**	Rocking	4.31
				FC (Q_{CL}):	Diag Tension	8.61

	Expected	Lower-Bound
Elastic Stiffness (k/in):	809	622

DC or FC? **DC** **DEFORMATION CONTROLLED**

Knowledge Factor: **0.75**

Strength: **4.31**

c (%):	0.624
d (%):	0.566
e (%):	1.131
x (%):	1.137
Drift at yield (in):	0.005
Δ_{eff}/h_{eff} (%):	0.006

Elastic:

Δ_{eff}/h_{eff} (%):	**Q/Qy**
0.006	1.00
0.005	0.84
0.004	0.68
0.003	0.52
0.002	0.36
0.001	0.20
0.000	0.04
0.000	0.00

Inelastic (BC):

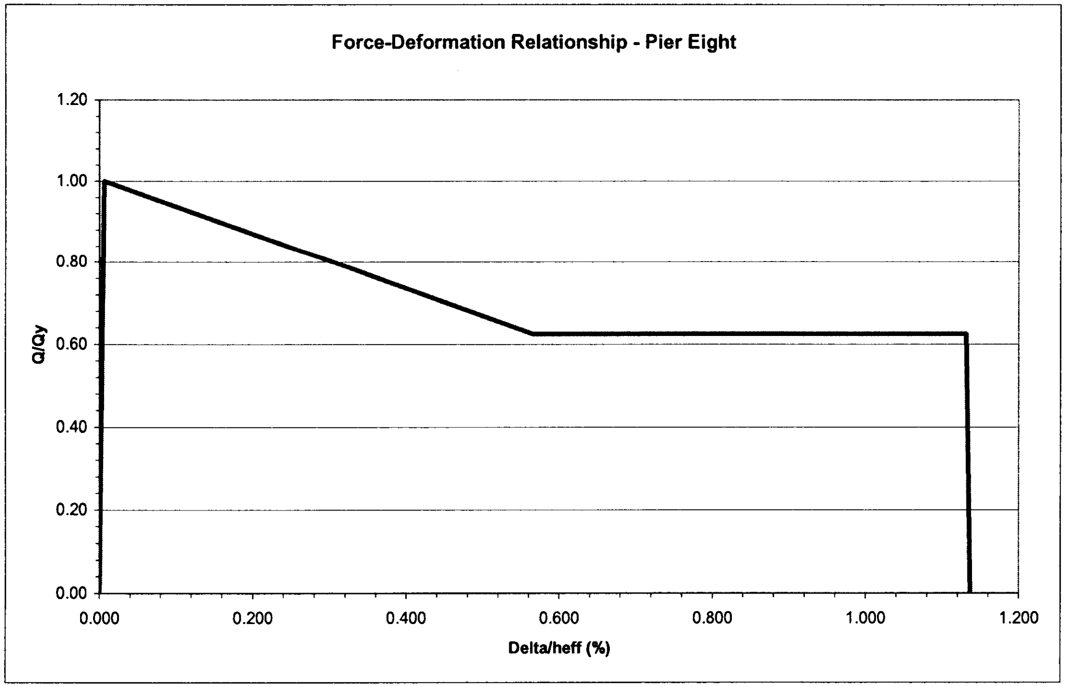
0.006	1.000
0.566	0.624
slope (k/in):	-3.41

Inelastic (CD):

0.566	0.624
1.131	0.624

Inelastic (DE):

1.13	0.624
1.14	0.000



Appendix D - Modified Nonlinear Static Analysis

Create Building Force-Deformation Relationship

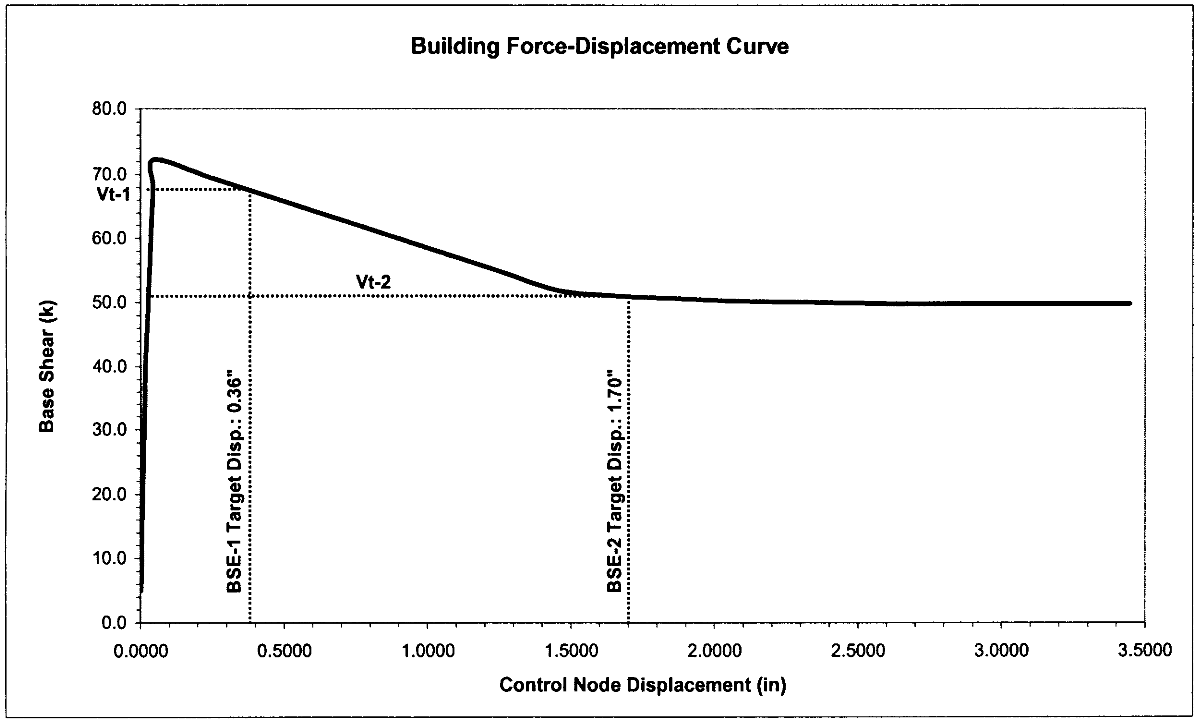
Pier Deformation Capacities:

	Pier One	Pier Two	Pier Three	Pier Four	Pier Five	Pier Six	Pier Seven	Pier Eight
AB	0.025	0.026	0.024	0.024	0.009	0.009	0.009	0.009
BC	1.186	1.418	0.922	0.815	0.815	0.922	0.922	0.815
CD	2.372	2.837	1.844	1.629	1.629	1.844	1.844	1.629
DE	2.403	2.868	1.863	1.650	1.637	1.851	1.851	1.637

Lateral load distribution proportional to fundamental mode

U_2 (in)	K_5 (k/in)	K_6 (k/in)	K_7 (k/in)	K_8 (k/in)	K_{second} Total (k/in)	DeltaV2 (k)	V_2 (k)	U_1 (in)	U_{2Total} (in)	Building Drift (%)	K_1 (k/in)	K_2 (k/in)	K_3 (k/in)	K_4 (k/in)	K_{first} Total (k/in)	Delta V1 (k)	V_1 (k)	V_{total} (k)
0.001	809	635	635	809	2888	2.89	2.89	0.001	0.0017	0.0006	371	244	635	809	2059	1.52	1.52	4.4
0.002	809	635	635	809	2888	2.89	5.78	0.001	0.0035	0.0012	371	244	635	809	2059	1.52	3.05	8.8
0.003	809	635	635	809	2888	2.89	8.67	0.002	0.0052	0.0018	371	244	635	809	2059	1.52	4.57	13.2
0.004	809	635	635	809	2888	2.89	11.55	0.003	0.0070	0.0024	371	244	635	809	2059	1.52	6.10	17.6
0.005	809	635	635	809	2888	2.89	14.44	0.004	0.0087	0.0030	371	244	635	809	2059	1.52	7.62	22.1
0.006	809	635	635	809	2888	2.89	17.33	0.004	0.0104	0.0036	371	244	635	809	2059	1.52	9.14	26.5
0.007	809	635	635	809	2888	2.89	20.22	0.005	0.0122	0.0042	371	244	635	809	2059	1.52	10.67	30.9
0.008	809	635	635	809	2888	2.89	23.11	0.006	0.0139	0.0048	371	244	635	809	2059	1.52	12.19	35.3
0.009	-3.41	-2.35	-2.35	-3.41	-11.52	-0.01	23.10	0.007	0.0157	0.0054	371	244	635	809	2059	1.52	13.71	36.8
0.010	-3.41	-2.35	-2.35	-3.41	-11.52	-0.01	23.08	0.007	0.0174	0.0060	371	244	635	809	2059	1.52	15.24	38.3
0.012	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	23.06	0.009	0.0209	0.0073	371	244	635	809	2059	3.05	18.29	41.3
0.014	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	23.04	0.010	0.0244	0.0085	371	244	635	809	2059	3.05	21.33	44.4
0.016	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	23.02	0.012	0.0278	0.0097	371	244	635	809	2059	3.05	24.38	47.4
0.018	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.99	0.013	0.0313	0.0109	371	244	635	809	2059	3.05	27.43	50.4
0.020	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.97	0.015	0.0348	0.0121	371	244	635	809	2059	3.05	30.48	53.4
0.022	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.95	0.016	0.0383	0.0133	371	244	635	809	2059	3.05	33.53	56.5
0.024	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.92	0.018	0.0418	0.0145	371	244	635	809	2059	3.05	36.57	59.5
0.030	-3.41	-2.35	-2.35	-3.41	-11.52	-0.07	22.85	0.022	0.0522	0.0181	371	244	635	809	2059	9.14	45.72	68.6
0.124	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	21.77	0.092	0.2158	0.0749	-2.58	-1.51	-5.52	-8.04	-18	-1.31	35.27	57.0
0.224	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	20.62	0.166	0.3898	0.1353	-2.58	-1.51	-5.52	-8.04	-18	-1.31	33.96	54.6
0.324	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	19.47	0.240	0.5638	0.1958	-2.58	-1.51	-5.52	-8.04	-18	-1.31	32.65	52.1
0.424	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	18.31	0.314	0.7378	0.2562	-2.58	-1.51	-5.52	-8.04	-18	-1.31	31.35	49.7
0.524	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	17.16	0.388	0.9118	0.3166	-2.58	-1.51	-5.52	-8.04	-18	-1.31	30.04	47.2
0.624	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	16.01	0.462	1.0858	0.3770	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	28.74	44.7
0.724	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	14.86	0.536	1.2598	0.4374	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	27.43	42.3
0.824	0.00	-2.35	-2.35	0.00	-4.70	-0.47	14.39	0.610	1.4338	0.4978	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	26.12	40.5
0.924	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.684	1.6078	0.5583	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	24.82	39.2
1.024	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.758	1.7818	0.6187	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	23.51	37.9
1.124	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.832	1.9558	0.6791	-2.58	-1.51	-5.52	0.00	-9.61	-0.71	22.80	37.2
1.224	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.906	2.1298	0.7395	-2.58	-1.51	0.00	0.00	-4.09	-0.30	22.50	36.9
1.324	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.980	2.3038	0.7999	-2.58	-1.51	0.00	0.00	-4.09	-0.30	22.19	36.6
1.424	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.054	2.4778	0.8603	-2.58	-1.51	0.00	0.00	-4.09	-0.30	21.89	36.3
1.524	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.128	2.6518	0.9208	0.00	-1.51	0.00	0.00	-1.51	-0.11	21.78	36.2
1.624	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.202	2.8258	0.9812	0.00	-1.51	0.00	0.00	-1.51	-0.11	21.67	36.1
1.724	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.276	2.9998	1.0416	0.00	0.00	0.00	0.00	0.00	0.00	21.67	36.1

Create Building Force-Deformation Relationship



Vy (k):	72.00
0.6Vy (k):	43.20
K_i (k/in):	2474
K_e (k/in):	1894
T_i (s):	0.470
T_e (s):	0.537

Calculate Target Displacement δ_t:

BSE-1		**BSE-2**	
PL:	IO	PL:	LS
C_0:	1.15	C_0:	1.15
R:	0.26	R:	0.93
C_1:	1.00	C_1:	1.0
C_2:	1.0	C_2:	1.3
C_3:	1.0	C_3:	1.0
S_a:	0.112	S_a:	0.402
δ_t (in):	0.3628	δ_t (in):	1.698

Base Shear at Target Displacement:

Vt (k):	68.0	Vt (k):	51.0
0.8Vy (k):	57.6	0.8Vy (k):	57.6

Appendix D - Modified Nonlinear Static Analysis

Create Building Force-Deformation Relationship

Pier Deformation Capacities:

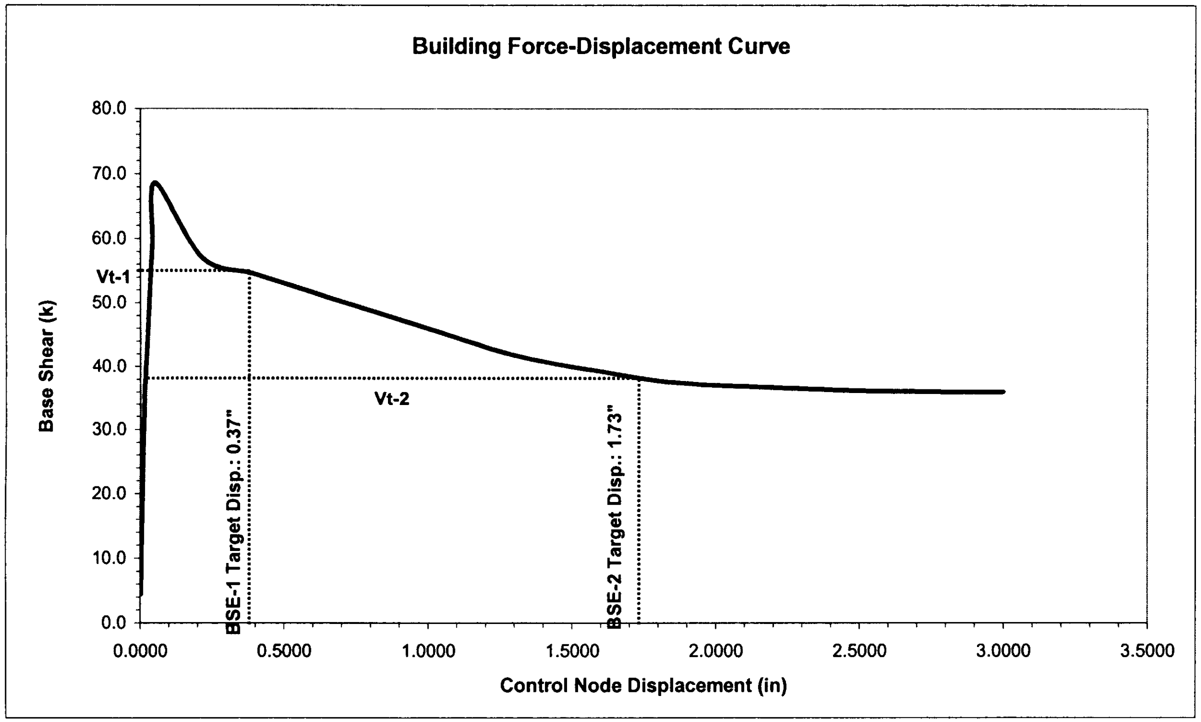
	Pier One	Pier Two	Pier Three	Pier Four	Pier Five	Pier Six	Pier Seven	Pier Eight
AB	0.025	0.026	0.024	0.024	0.009	0.009	0.009	0.009
BC	1.186	1.418	0.922	0.815	0.815	0.922	0.922	0.815
CD	2.372	2.837	1.844	1.629	1.629	1.844	1.844	1.629
DE	2.403	2.868	1.863	1.650	1.637	1.851	1.851	1.637

Lateral load distribution proportional to fundamental mode

U_2 (in)	K_5 (k/in)	K_6 (k/in)	K_7 (k/in)	K_8 (k/in)	K_{second} Total (k/in)	DeltaV2 (k)	V_2 (k)	U_1 (in)	U_{2Total} (in)	Building Drift (%)	K_1 (k/in)	K_2 (k/in)	K_3 (k/in)	K_4 (k/in)	K_{first} Total (k/in)	Delta V1 (k)	V_1 (k)	V_{total} (k)
0.001	809	635	635	809	2888	2.89	2.89	0.001	0.0017	0.0006	371	244	635	809	2059	1.52	1.52	4.4
0.002	809	635	635	809	2888	2.89	5.78	0.001	0.0035	0.0012	371	244	635	809	2059	1.52	3.05	8.8
0.003	809	635	635	809	2888	2.89	8.67	0.002	0.0052	0.0018	371	244	635	809	2059	1.52	4.57	13.2
0.004	809	635	635	809	2888	2.89	11.55	0.003	0.0070	0.0024	371	244	635	809	2059	1.52	6.10	17.6
0.005	809	635	635	809	2888	2.89	14.44	0.004	0.0087	0.0030	371	244	635	809	2059	1.52	7.62	22.1
0.006	809	635	635	809	2888	2.89	17.33	0.004	0.0104	0.0036	371	244	635	809	2059	1.52	9.14	26.5
0.007	809	635	635	809	2888	2.89	20.22	0.005	0.0122	0.0042	371	244	635	809	2059	1.52	10.67	30.9
0.008	809	635	635	809	2888	2.89	23.11	0.006	0.0139	0.0048	371	244	635	809	2059	1.52	12.19	35.3
0.009	-3.41	-2.35	-2.35	-3.41	-11.52	-0.01	23.10	0.007	0.0157	0.0054	371	244	635	809	2059	1.52	13.71	36.8
0.010	-3.41	-2.35	-2.35	-3.41	-11.52	-0.01	23.08	0.007	0.0174	0.0060	371	244	635	809	2059	1.52	15.24	38.3
0.012	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	23.06	0.009	0.0209	0.0073	371	244	635	809	2059	3.05	18.29	41.3
0.014	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	23.04	0.010	0.0244	0.0085	371	244	635	809	2059	3.05	21.33	44.4
0.016	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	23.02	0.012	0.0278	0.0097	371	244	635	809	2059	3.05	24.38	47.4
0.018	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.99	0.013	0.0313	0.0109	371	244	635	809	2059	3.05	27.43	50.4
0.020	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.97	0.015	0.0348	0.0121	371	244	635	809	2059	3.05	30.48	53.4
0.022	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.95	0.016	0.0383	0.0133	371	244	635	809	2059	3.05	33.53	56.5
0.024	-3.41	-2.35	-2.35	-3.41	-11.52	-0.02	22.92	0.018	0.0418	0.0145	371	244	635	809	2059	3.05	36.57	59.5
0.030	-3.41	-2.35	-2.35	-3.41	-11.52	-0.07	22.85	0.022	0.0522	0.0181	371	244	635	809	2059	9.14	45.72	68.6
0.124	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	21.77	0.092	0.2158	0.0749	-2.58	-1.51	-5.52	-8.04	-18	-1.31	35.27	57.0
0.224	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	20.62	0.166	0.3898	0.1353	-2.58	-1.51	-5.52	-8.04	-18	-1.31	33.96	54.6
0.324	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	19.47	0.240	0.5638	0.1958	-2.58	-1.51	-5.52	-8.04	-18	-1.31	32.65	52.1
0.424	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	18.31	0.314	0.7378	0.2562	-2.58	-1.51	-5.52	-8.04	-18	-1.31	31.35	49.7
0.524	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	17.16	0.388	0.9118	0.3166	-2.58	-1.51	-5.52	-8.04	-18	-1.31	30.04	47.2
0.624	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	16.01	0.462	1.0858	0.3770	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	28.74	44.7
0.724	-3.41	-2.35	-2.35	-3.41	-11.52	-1.15	14.86	0.536	1.2598	0.4374	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	27.43	42.3
0.824	0.00	-2.35	-2.35	0.00	-4.70	-0.47	14.39	0.610	1.4338	0.4978	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	26.12	40.5
0.924	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.684	1.6078	0.5583	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	24.82	39.2
1.024	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.758	1.7818	0.6187	-2.58	-1.51	-5.52	-8.04	-17.65	-1.31	23.51	37.9
1.124	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.832	1.9558	0.6791	-2.58	-1.51	-5.52	0.00	-9.61	-0.71	22.80	37.2
1.224	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.906	2.1298	0.7395	-2.58	-1.51	0.00	0.00	-4.09	-0.30	22.50	36.9
1.324	0.00	0.00	0.00	0.00	0.00	0.00	14.39	0.980	2.3038	0.7999	-2.58	-1.51	0.00	0.00	-4.09	-0.30	22.19	36.6
1.424	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.054	2.4778	0.8603	-2.58	-1.51	0.00	0.00	-4.09	-0.30	21.89	36.3
1.524	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.128	2.6518	0.9208	0.00	-1.51	0.00	0.00	-1.51	-0.11	21.78	36.2
1.624	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.202	2.8258	0.9812	0.00	-1.51	0.00	0.00	-1.51	-0.11	21.67	36.1
1.724	0.00	0.00	0.00	0.00	0.00	0.00	14.39	1.276	2.9998	1.0416	0.00	0.00	0.00	0.00	0.00	0.00	21.67	36.1

Appendix D - Modified Nonlinear Static Analysis

Create Building Force-Deformation Relationship



Vy (k):	68.00
0.6Vy (k):	40.80
K_i (k/in):	2536
K_e (k/in):	1869
T_i (s):	0.470
T_e (s):	0.547

Calculate Target Displacement δ_t:

BSE-1		BSE-2	
PL:	IO	PL:	LS
C_0:	1.15	C_0:	1.15
R:	0.27	R:	0.97
C_1:	1.00	C_1:	1.0
C_2:	1.0	C_2:	1.3
C_3:	1.0	C_3:	1.0
S_a:	0.110	S_a:	0.395
δ_t (in):	0.3697	δ_t (in):	1.730

Base Shear at Target Displacement:

Vt (k):	55.0	Vt (k):	38.0
0.8Vy (k):	54.4	0.8Vy (k):	54.4

Appendix E

Analysis of Retrofitted Walls

Appendix E - Steel Retrofitted Analysis

Determine Period

Diaphragm Span (ft):	30	
Diaphragm Length (ft):	30	
Diaphragm Tk. (in):	1.00	
Diaphragm Mod. (psi):	1500	
Diaphragm I (in^4):	3,888,000	
Floor Dead Load (psf):	25	
Inertial Diaphragm Force (lbs):	22,500	
Max. Diaphragm Deflection (in):	2.86	
Approximate Period T (s):	0.47	
Trib. Weight of Building	(k)	(kg)
Floor One:	83.25	37,747
Floor Two:	83.25	37,747

Calculate Spectral Acceleration:			
BSE-1:		BSE-2:	
S_a :	0.127	S_a :	0.448

Calculate Pseudo Lateral Load:			
BSE-1		BSE-2	
Factors:		**Factors:**	
C_1:	1.00	C_1:	1.01
C_2:	1.00	C_2:	1.00
C_3:	1.00	C_3:	1.00
C_m:	1.00	C_m:	1.00
Pseudo Lateral Load (k):		**Pseudo Lateral Load (k):**	
Floor One (k):	10.58	Floor One (k):	37.30
Floor Two (k):	10.58	Floor Two (k):	37.30

Appendix E - Steel Retrofitted Analysis

Design Forces:

BSE-1			BSE-2		
Deformation Controlled			**Deformation Controlled**		
Story Shear:			**Story Shear:**		
Q_{UD}: Floor Two (k):	10.58		Q_{UD}: Floor Two (k):	37.30	
Q_{UD}: Floor One (k):	21.15		Q_{UD}: Floor One (k):	74.59	
Force Controlled			**Force Controlled**		
J factor:	1.0			1.0	
Story Shear:			**Story Shear:**		
Q_{UF}: Floor Two (k):	10.58		Q_{UF}: Floor Two (k):	36.82	
Q_{UF}: Floor One (k):	21.15		Q_{UF}: Floor One (k):	73.65	

Acceptance Criteria

BSE-1			BSE-2		
Deformation Controlled			**Deformation Controlled**		
Limit State:	Flexure		Limit State:	Flexure	
Performance Level:	IO		Performance Level:	LS	
Knowledge Factor (κ):	0.75		Knowledge Factor (κ):	0.75	
m factor:	1		m factor:	3	
$m\kappa Q_{CE}$:			$m\kappa Q_{CE}$:		
Floor Two (k):	106.36	**GOOD!**	Floor Two (k):	319.08	**GOOD!**
Floor One (k):	103.70	**GOOD!**	Floor One (k):	311.09	**GOOD!**
Force Controlled			**Force Controlled**		
Limit State:	Shear		Limit State:	Shear	
Knowledge factor (κ):	0.75		Knowledge factor (κ):	0.75	
κQ_{CL}:			κQ_{CL}:		
Floor Two (k):	193.07	**GOOD!**	Floor Two (k):	193.07	**GOOD!**
Floor One (k):	198.22	**GOOD!**	Floor One (k):	198.22	**GOOD!**

Appendix E - FRP Retrofitted Analysis

Determine Period

Diaphragm Span (ft):	30
Diaphragm Length (ft):	30
Diaphragm Tk. (in):	1.00
Diaphragm Mod. (psi):	1500
Diaphragm I (in^4):	3,888,000
Floor Dead Load (psf):	25
Inertial Diaphragm Force (lbs):	22,500
Max. Diaphragm Deflection (in):	2.86
Approximate Period T (s):	0.47

Trib. Weight of Building	(k)	(kg)
Floor One:	83.25	37,747
Floor Two:	83.25	37,747

Calculate Spectral Acceleration:			
BSE-1:		BSE-2:	
S_a :	0.127	S_a :	0.448

Calculate Pseudo Lateral Load:			
BSE-1		**BSE-2**	
Factors:		**Factors:**	
C_1:	1.00	C_1:	1.01
C_2:	1.00	C_2:	1.00
C_3:	1.00	C_3:	1.00
C_m:	1.00	C_m:	1.00
Pseudo Lateral Load (k):		**Pseudo Lateral Load (k):**	
Floor One (k):	10.58	Floor One (k):	37.30
Floor Two (k):	10.58	Floor Two (k):	37.30

Appendix E - FRP Retrofitted Analysis

Design Forces:

BSE-1			BSE-2		
Deformation Controlled			**Deformation Controlled**		
Story Shear:			**Story Shear:**		
Q_{UD}: Floor Two (k):	10.58		Q_{UD}: Floor Two (k):	37.30	
Q_{UD}: Floor One (k):	21.15		Q_{UD}: Floor One (k):	74.59	
Q_{UD}: Base (k):	21.15		Q_{UD}: Base (k):	74.59	
Force Controlled			**Force Controlled**		
J factor:	1.0			1.0	
Story Shear:			**Story Shear:**		
Q_{UF}: Floor Two (k):	10.58		Q_{UF}: Floor Two (k):	36.82	
Q_{UF}: Floor One (k):	21.15		Q_{UF}: Floor One (k):	73.65	
Q_{UF}: Base (k):	21.15		Q_{UF}: Base (k):	73.65	

Acceptance Criteria

BSE-1			BSE-2		
Deformation Controlled			**Deformation Controlled**		
Limit State:	Flexure		Limit State:	Flexure	
Performance Level:	IO		Performance Level:	LS	
Knowledge Factor (κ):	0.75		Knowledge Factor (κ):	0.75	
m factor:	1		m factor:	1	
$m\kappa Q_{CE}$:			$m\kappa Q_{CE}$:		
Floor Two (k):	48.65	**GOOD!**	Floor Two (k):	48.65	**GOOD!**
Floor One (k):	47.44	**GOOD!**	Floor One (k):	47.44	**NO GOOD!**
Force Controlled			**Force Controlled**		
Limit State:	Shear		Limit State:	Shear	
Knowledge factor (κ):	0.75		Knowledge factor (κ):	0.75	
κQ_{CL}:			κQ_{CL}:		
Floor Two (k):	193.07	**GOOD!**	Floor Two (k):	193.07	**GOOD!**
Floor One (k):	198.22	**GOOD!**	Floor One (k):	198.22	**GOOD!**