“Seismic Rehabilitation of Wood Diaphragms in Unreinforced Masonry Buildings”

by

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May 2007

This research was conducted at Texas A&M University and was supported by the Mid-America Earthquake Center through the Earthquake Engineering Research Centers Program of the National Science Foundation under NSF award number EEC-9701785 and the Department of Civil Engineering and the Texas Engineering Experiment Station at Texas A&M University. Any opinions, findings and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the National Science Foundation and the other sponsors.

ABSTRACT

This research focused on evaluating the seismic performance of existing and rehabilitated wood floor and roof diaphragms in typical pre-1950's, unreinforced masonry (URM) buildings found in the Central and Eastern portions of the United States. Specifically, there were two major objectives: (1) to assess the adequacy of current seismic guidelines for evaluating existing wood diaphragms in pre-1950's URM buildings and for designing necessary retrofits; and (2) to evaluate the effect of diaphragm retrofits, as designed by FEMA guidelines, on the overall response of URM structures.

This study utilized current guidelines and tools available to practicing engineers to evaluate wood diaphragms in two pre-1950s URM buildings for seismic demands and to design appropriate rehabilitations for these diaphragms. The linear static procedures from the FEMA 273 and FEMA 365 guidelines were used to evaluate the existing wood diaphragms of the case study buildings. This evaluation indicated that the existing diaphragms were not sufficient for the Life Safety performance level when subject to the demands of the 10% probability of exceedance in 50 years event in St. Louis, Missouri. Retrofit options were provided in the FEMA guidelines to upgrade the diaphragms to Life Safety performance.

A parametric study was also performed to evaluate the complete building response after the diaphragms of a URM prototype structure were retrofitted. The selected retrofit included increasing the in-plane strength of the diaphragm and improving the connection of the diaphragm to the URM walls. Various existing conditions of masonry were considered. It was found that retrofitting the diaphragms led to improved behavior for the diaphragms. However, stresses increased in other structural components, including the walls, due to a reduction in the building period and increased seismic demands.

ACKNOWLEDGMENTS

This research was conducted at Texas A&M University and was supported by the Mid-America Earthquake Center through the Earthquake Engineering Research Centers Program of the National Science Foundation under NSF award number EEC-9701785 and the Department of Civil Engineering and the Texas Engineering Experiment Station at Texas A&M University. Any opinions, findings and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the National Science Foundation and the other sponsors.

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1. INTRODUCTION

1.1 BACKGROUND AND PROBLEM STATEMENT

The purpose of this study is to evaluate the seismic performance of existing and rehabilitated wood floor and roof diaphragms in typical pre-1950's, unreinforced masonry (URM) buildings found in the Central and Eastern portions of the United States. Many structures in this area built prior to 1950 are two-story, rectangular buildings composed of URM exterior walls with wood floor and roof diaphragms. This type of construction is no longer permissible for zones of high seismicity due to its poor performance in past earthquakes. Current guidelines have challenging criteria for practicing engineers to seismically rehabilitate URM structures because guidance on how to achieve an acceptable retrofit is limited, particularly for the diaphragms. The evaluation of two case study buildings found in St. Louis, Missouri were carried out using the Federal Emergency Management Agency's (FEMA) guidelines for seismic rehabilitation of buildings.

The New Madrid seismic zone, located in the Central United States, has a moderately low level of public awareness for its seismic hazard because the recurrence of high intensity earthquakes is infrequent compared to the Western United States. However, the largest earthquakes in the continental United States occurred as a series of four events during late 1811 and early 1812, encompassing Northeast Arkansas and Southeast Missouri. Because of the potential for such an event to occur again, and the prevalence of URM structures that have not performed satisfactorily during past seismic events, it is important that seismic rehabilitation guidelines be evaluated.

The 1989 Loma Prieta earthquake in San Francisco, California drew attention to the poor performance of many URM structures. During this event, engineered buildings in the affected area performed predictably while retrofitted URM structures had an inconsistent pattern of success. URM buildings composed much of the more severe building damage overall. The life-threatening hazard posed by the potential collapse of URM buildings in an earthquake prompted the City of San Francisco to identify existing URM buildings and develop a risk reduction plan (EQE 1990).

Because most of these URM buildings were built prior to adoption of seismic code requirements, they were not adequately designed for earthquake excitation. Of the damage to URM buildings, much of the failure was the result of poor anchorage of the URM walls to the wood diaphragms or due to excessive in-plane flexibility of the floor diaphragms. The anchorage in some cases may be as little as the diaphragms connected to out-of-plane URM walls by sitting in a pocket in the wall. While this may be adequate for gravity loads, this connection will not successfully transfer lateral loading from the walls to the diaphgragm. Excessive flexibility of the floor can allow the out-of-plane walls to displace beyond their stability limit.

This study is one in a series of related studies directed by the Mid-America Earthquake (MAE) Center aiming for a long-term goal of mitigating the impact of earthquakes with a focus on the Central and Eastern United States. The MAE Center joins the Pacific Engineering Earthquake Research (PEER) Center and the Multidisciplinary Center for Earthquake Engineering Research (MCEER) as the three earthquake engineering research centers funded by the National Science Foundation. This particular study is a part of one MAE Center project, ST-8: Seismic Performance of Wood Floor and Roof Diaphragms. Peralta et al. (2002, 2003, 2004) documented the first phase of this project describing the experimental testing of unretrofitted and retrofitted wood diaphragms.

One group of studies directed by the MAE Center focuses on retrofitting essential facilities. Much of the existing building stock in the Central and Eastern portions of the United States is pre-1950s URM buildings, many of which are essential facilities. These structures need to remain operational after an earthquake event due to the emergency services these buildings must provide. Typical firehouses in St. Louis, Missouri were selected as case study buildings for this research due to the MAE Center's emphasis toward essential facilities.

This particular study focuses on the wood floor and roof diaphragms of pre-1950s URM buildings. With the observation that existing URM buildings can pose significant safety hazards during an earthquake, attention has been directed to the need for some form of seismic rehabilitation. This study considers retrofit of the diaphragms to limit in-plane deflection, thereby limiting damage to the out-of-plane masonry walls, and to do so by utilizing a simple retrofit design procedure currently available to industry. The goals of this research are to assess the adequacy of current seismic rehabilitation guidelines for URM structures with a focus on the diaphragms and to evaluate the effect of diaphragm retrofits, as designed by the FEMA guidelines, on the overall seismic response of URM structures.

There are two recent sets of guidelines for seismic rehabilitation maintained by the Federal Emergency Management Agency: the *NEHRP Guidelines for Seismic Rehabilitation of Buildings and Commentary* (FEMA 273 and 274) (ATC 1997a, b) and the more recent *NEHRP Prestandard and Commentary for Seismic Rehabilitation of Buildings* (FEMA 356) (ASCE 2000). These guidelines were used in this research to evaluate two case study buildings found in St. Louis, Missouri. Two URM firehouses were selected as case study buildings because of their typical, but relatively simple, layout and the obvious need for such essential facilities to survive an earthquake event.

1.2 RESEARCH OBJECTIVES

This research focuses on two major objectives: (1) assessing the adequacy of current seismic guidelines for evaluating existing wood diaphragms in pre-1950s URM buildings and for designing necessary retrofits; and (2) evaluating the effect of diaphragm retrofits, satisfying the FEMA guidelines, on the overall seismic response of URM structures.

The first objective is accomplished by applying the applicable performance-based evaluation procedures outlined in two sets of current seismic rehabilitation guidelines, FEMA 273 (ATC 1997a) and FEMA 356 (ASCE 2000), for two case study structures. The FEMA guidelines were developed for use by practicing engineers to design an acceptable retrofit for a specific seismic demand and performance level. These guidelines are easily available and at least some of the procedures are simple to use.

In both guidelines, FEMA 273 and FEMA 356, there are four analysis methods detailed: Linear Static Procedure (LSP), Nonlinear Static Procedure (NSP), Linear Dynamic Procedure (LDP), and Nonlinear Dynamic Procedure (NDP). For the purposes of this study, only the LSP has been selected. This procedure allows an evaluation of the performance of a diaphragm as a component. However, the information contained within each of these two documents pertaining to flexible diaphragms is limited. By stepping through a seismic evaluation and rehabilitation design for two case study buildings, these limitations can be demonstrated. The result of this effort is outlined in a comparison of these two relatively recent guidelines, which helps to define the differences between them.

The second objective is accomplished using a parametric study based on the conclusions of the FEMA 273 and FEMA 356 analyses. Because the approach used in the first part of this study is component based, the parametric study will evaluate how the rehabilitation of a single component affects the behavior of the system as a whole. Rehabilitating the diaphragm typically involves increasing the in-plane diaphragm strength and stiffness, which will change the behavior of the system, and may or may not have adverse effects on other building components. Various research studies have been conducted in the past on URM structures to observe the changing behavior of the system as a function of rehabilitating specific components (ABK 1985, Paquette et al. 2001, Tena-Colunga and Abrams 1996, and Yi et al. 2002). This parametric study is unique in that it demonstrates the changing behavior of the system utilizing the specific diaphragm retrofits designed according to the FEMA guidelines.

The analytical results of the parametric study were evaluated to observe how variations in the diaphragm stiffness and the adequacy of the connection between the wall and diaphragm affect the behavior of the system, rather than focusing solely on avoiding out-of-plane URM wall damage by limiting diaphragm in-plane deflections. Each parametric analysis physically represents a potential existing or retrofitted state of a prototype URM structure. The prototype structure is analyzed using a set of synthetic time histories developed to be representative of local soil conditions for St. Louis, Missouri for a 10% probability of exceedance in 50-year seismic event (Wen and Wu 2000).

1.3 SCOPE OF WORK

This report is organized in the following manner. Chapter 2 provides a summary of existing relevant literature. Each of the cited references provides background or influence to this study. Chapter 3 describes the case study buildings. The case study buildings are used in the subsequent analyses, therefore a thorough description of each building is necessary to validate the necessary assumptions made throughout the analysis. Chapter 4 discusses the possible alternatives for analyzing the case study buildings provided in the FEMA guidelines. A brief description of each alternative is provided with a corresponding explanation of the reasons for ultimately selecting the LSP. With the case study buildings described and the methodology for analysis selected, Chapter 5 presents an explanation of the LSP contained in both FEMA 273 and FEMA 356. This chapter first focuses on the analysis pertaining to Case Study Building 1, followed by Case Study Building 2. Any pertinent conclusions that can be made at these stages are included at the end of this section according to the respective case study building and guideline. Chapter 6 is devoted to explaining the parametric study. Initially, this section describes the parameters of the prototype used in the finite element modeling. The study varies critical parameters of three main structural components using values consistent with FEMA guidelines. The parameter variations are outlined in detail and accompanied by an explanation of each respective physical representation. Results and conclusions from the parametric study complete Chapter 6. Chapter 7 includes a summary of this research along with conclusions and recommendations for future research. The attached appendices contain calculations referred to within the main body of the report.

2. PREVIOUS RESEARCH

2.1 GENERAL

This chapter provides a review of relevant literature and research pertaining to this study.

2.2 MID-AMERICA EARTHQUAKE CENTER RESEARCH

2.2.1 General

The overall goal of the MAE Center is to create innovative solutions to mitigate impacts of earthquake events through system driven research. To accomplish this objective, the MAE Center conducts many related research studies that achieve a large overall common purpose. The studies are then organized and timed accordingly so that the deliverable from one study may feed into knowledge in another study. This allows the MAE Center as a whole to be able to achieve a larger goal through these more interdependent studies. One of these research programs, the Essential Facilities Program, focused primarily on URM structures and their critical components. The research described in this report is part of the group of research dedicated to URM structures and is a follow up investigation of previously conducted research at Texas A&M University.

2.2.2 Research Performed at Texas A&M University

This study is part of a MAE Center project at Texas A&M University (TAMU) that includes additional experimental and analytical studies of wood diaphragms. The related research focused on large-scale experimental testing of typical diaphragms for pre-1950's URM buildings in the Central and Eastern portions of the United States. Because much of the past damage in URM structures has been related to the poor connections of the diaphragms to the URM walls (EQE 1990) and diaphragm flexibility, the focus of this experimental work included several types of representative diaphragms and retrofits that strengthened the connections and stiffened and strengthened the diaphragm (Peralta et al. 2002, 2003, 2004).

Three basic types of diaphragms, typical of pre-1950's existing diaphragms in this region, were tested experimentally with plan dimensions of 3.66 m x 7.32 m (12 ft x 24 ft): (1) specimen MAE-1 had 1x4 in. (nominal) tongue and groove sheathing nailed to 2x10 in (nominal) joists in the long direction representative of a floor diaphragm; (2) specimen MAE-2 had 1x6 in. (nominal) staggered sheathing nailed to 2x10 in. (nominal) joists running in the short direction typical of a flat roof diaphragm; and (3) specimen MAE-3 was similar to MAE-2 but with a corner opening to represent a stairwell. MAE-1 utilized a replica star anchor, a wall-to-diaphragm anchor typical of the time period of concern (see Fig. 2.1), to attach the diaphragm to the rigid steel lateral support frames, whereas MAE-2 and MAE-3 had bolted connections representing anchors that connected the joists running parallel to the lateral support frames at 1.22 m (4 ft.) on center. This connection was also typical of pre-1950's construction.

FIG. 2.1 Prototype Star Anchor

Each of these diaphragms underwent displacement-controlled quasi-static reverse cyclic loading. The diaphragms representing the existing state were tested, retrofitted, and retested. MAE-1 was retrofitted twice, first with enhanced connections and perimeter strapping and secondly with a steel truss. MAE-2 was retrofitted three times: first with the steel truss similar to that used for MAE-1; secondly with an unblocked, unchorded plywood overlay; and lastly with a blocked, unchorded plywood overlay.

MAE-3 used the two plywood overlay retrofits, unblocked and blocked, like that used for MAE-2. Each of the plywood retrofits is a possible retrofit listed in the FEMA 273 and FEMA 356 guidelines.

The yield force, yield displacement, effective stiffness, and post-yield stiffness were measured for each diaphragm specimen. In addition, the predicted backbone curves from both FEMA guidelines were calculated for each specimen. The steel truss retrofit for diaphragms MAE-1 and MAE-2 improved the performance of the diaphragm the most, in terms of increased strength and stiffness. The blocked and unblocked plywood overlays did increase the strength and stiffness, although the blocked overlay gave a more significant increase in the stiffness. The FEMA predictions in all cases had consistent tendencies, but generally did not give an accurate prediction of the actual measured in-plane response for the diaphragm specimens. Generally, FEMA 273 overpredicted the stiffness and underpredicted yield displacement and deformation levels. The opposite was true for FEMA 356 where this method typically underpredicted stiffness and overpredicted the yield displacement and deformation levels.

2.2.3 Research Performed at Georgia Tech University

Kim and White (2002) developed a three-dimensional nonlinear model that can be applied for low-rise, URM structures with flexible diaphragms. The model was developed to provide a more realistic estimate of the structural response of URM structures under earthquake loadings, as compared to linear elastic models.

The model captures the diaphragm as a six degree-of-freedom element. The theory is based on a plate girder analogy with the diaphragm chords acting as the flange. It allows nails slip to be the major contributor in the diaphragm deformation. The model allows the user to alter material properties in necessary quadrants to account for weaknesses or openings in the diaphragm; thus, the material property of the diaphragm need not be uniform. The wall model employs a flexibility approach using finite elements for the in-plane stiffness of walls and ignores the out-of-plane stiffness.

Combining these two developed elements the overall 3-D model (see Fig. 2.2) has the ability to accurately predict building response and possible damage for this specific building type. The model uses the ABAQUS (HKS 1998) finite element software, which is not typically available in a design office.

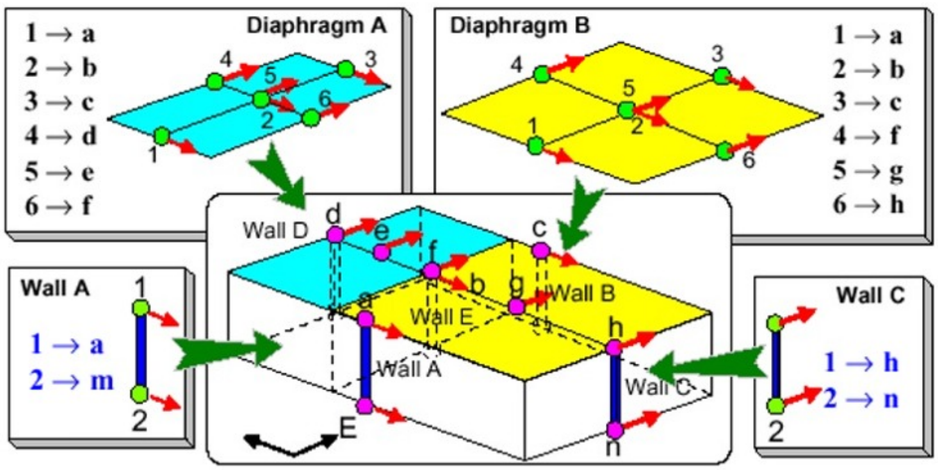


FIG. 2.2 3-D Lumped Mass Model (Kim and White 2002)

2.2.4 Additional MAE Center Studies in Unreinforced Masonry

There have been several projects dedicated to the evaluation of the performance of URM and their rehabilitation at both the component and system levels within the MAE Center.

At the component level, testing has shown that slender URM walls performed relatively well when loaded in the out-of-plane direction (Simsir et al. 2002). These tests showed that the walls could sustain gravity loads and remain stable under significant base excitation and lateral deformation. These results would support modifying the

FEMA recommendations to increase the permissible wall slenderness (height-to-thickness ratio) limitations. Separate tests took place on wall and pier specimens developed so that the strength and deformation behavior of the URM wall could be assessed both before and after rehabilitation (Erbay and Abrams 2001).

At the system level, a full-scale model of a typical URM building with wood floor and roof diaphragms found in the Eastern and Central portions of the United States was developed (Yi et al. 2002). The testing of this building will demonstrate the behavior of the system response according to performance-based design. Many of the input parameters on this building were derived from the various MAE Center projects previously mentioned. The results of the testing from this study were not published at the time this report was compiled.

2.3 RESEARCH PERFORMED BY CERL

The focus of the study funded by the United States Army Construction Engineering Research Laboratory (CERL) and Development Center was to identify and detail the wood diaphragm systems within the army's inventory of buildings and test a typical prototype for strength, stiffness and failure mechanisms (Cohen et al. 2002). The inventory study produced two prototypes of masonry buildings with flexible roof diaphragms that were tested at half-scale with dynamic shake table testing and compared with analytical predictions. Because the test specimens were half-scale, the ground motions were geometrically scaled down, accordingly.

There were generally two diaphragms studied: (1) a diagonally sheathed lumber diaphragm and (2) a corrugated metal deck diaphragm. The wood diaphragm of interest had 0.953 cm (0.375 in.) thick by 8.26 cm (3.25 in.) wide sheathing. The 1.91 cm x 14.0 cm (0.75 x 5.5 in.) joists were connected to the sheathing by 4d (nominal) nails. These material dimensions were roughly half scale dimensions of the prototype element.

These prototypes were studied analytically using an idealized two-degree-of-freedom finite element model in SAP2000 (CSI 1999). The material values were chosen to mimic the parameters that would have been selected in design practice. The testing of the half-scale specimens showed that the buildings remained elastic up to 0.5g. By refining the FEM model and keeping the peak ground acceleration below 0.5g during the analytical modeling, they were able to achieve results analytically that sufficiently approximated the results of the experiment. The refined FEM model accurately predicted the natural frequencies, the displacement and acceleration of both the systems and specimen, and cracking patterns. The model was refined based on critical parameters, which were selected based on the sensitivity of the model to those parameters. These modifications included decreasing the thickness of the masonry and the increasing the overall damping ratio. Acceptable results were produced using a two-degree-of-freedom system in predicting the diaphragm fundamental frequency, acceleration, and deflection.

The results from the experimental and analytical results showed that the system could not be idealized as a single degree of freedom system. The response spectrum analysis of a two-degree of freedom system did produce an acceptable system response.

2.4 OTHER STUDIES PERFORMED ON URM STRUCTURES

A study conducted after the 1989 Loma Prieta earthquake looked at the influence of flexible diaphragms on the seismic behavior of URM structures (Tena-Colunga and Abrams 1996). This study showed that diaphragm and shear wall accelerations have the potential to increase with increasing diaphragm flexibility. This study utilized three case study buildings of various typical URM building types and results were compared with the FEMA 273 guidelines. The Gilroy Firehouse was one of the case study buildings, and is similar to the structure type of interest for the study described in this report.

A recent study of typical URM buildings in North America used three wall specimens extracted from an existing building to observe the behavior of the existing and retrofitted walls (Paquette et al. 2001). One of these specimens was tested in an existing condition fashion with no retrofits while the other two were each retrofitted. The first retrofit followed what is often done in practice by anchoring the wall at mid-height. While this action did enhance the overall performance compared to the unretrofitted wall, it did not mitigate the displacement of the upper portion of the wall. Thus, this specimen failed sooner than expected. The second retrofit involved bonding fiberglass backing onto the back of the wall using epoxy with the intention of increasing out-of-plane wall stiffness. This specimen performed very well with almost no visible deflection and a significantly lower building period than that of the existing or first retrofitted specimens. Like the MAE Center testing, these tests demonstrated that each of the walls could sustain substantial out-of-plane acceleration. Furthermore, the testing showed that anchoring the walls at mid-height enhanced the performance and the addition of fiberglass to increase the out-of-plane stiffness of the wall was even more effective. Perhaps most importantly, the testing of these walls showed that they could be significantly affected by variation of boundary conditions. The need for anchoring URM walls at the intermediate floor is very important in older URM structures where wall to diaphragm anchors are likely to be weak or nonexistent.

3. CASE STUDY BUILDINGS

3.1 GENERAL

Two firehouses in St. Louis, Missouri that are typical of pre-1950's URM buildings found in the Central and Eastern portions of the United States were chosen as case study buildings for this research. Firehouses were selected due to the focus of the MAE Center research program on essential facilities. The required operability of these structures after an earthquake provides the potential to evaluate seismic performance with multiple levels of objectives. This study only considers the Life Safety objective, according to FEMA, but the same process conducted here can be applied to evaluate a higher level of performance. The office of the chief engineer for the Board of Public Services in St. Louis provided copies of original drawings for two local firehouses, as well as the drawings for any improvements made since their original construction. In addition to obtaining the drawings, a sight inspection and guided tour of each building was performed. Although firehouses have some characteristics specific to their function, such as large wall openings for overhead doors in the first story, the details of the structures have a number of similarities with the many URM structures in the Central and Eastern United States. Therefore, the case studies provide insight into the seismic performance and rehabilitation needs for other similar URM buildings.

3.2 CASE STUDY BUILDING 1

3.2.1 General Description

Case Study Building 1 is a small, two-story, URM firehouse located in St. Louis, Missouri, built in 1924 (Fig. 3.1). The length-to-width aspect ratio is 2.1:1.0 with plan dimensions of 9.20 m by 19.3 m (30.2 ft. by 63.3 ft.). The largest wall opening is associated with an overhead door located along the short dimension of the building. The first story is almost entirely open space used for fire engine parking and equipment storage. The lower story height is 4.42 m (14.5 ft.). The second story floor is 3.35 m (11 ft.) above the first story diaphragm. The top level in this building serves as space for a recreational room, personal lockers, and dormitory area. The exterior URM walls of this firehouse are 33.0 cm (13 in.) thick and made up of three wythes of clay brick.

FIG. 3.1 Case Study Building 1

3.2.2 Structural Details

Because the building was designed in 1924, the drawing labels for the beams and joists used in the building are not standard callouts that would be expected on modern drawing details. The joists and beams referenced here are the modern name equivalent for the components used in the floor system.

A wood truss system forms the pitched roof over the wood roof diaphragm. Because the truss prevents the diaphragm from behaving solely as a flexible diaphragm, the focus of the analysis dealing with this case study building is on the first floor wood diaphragm. The actual roofing material is composed of 7.62 cm (3 in.) thick slate. The existing diaphragm is 2.22 cm (0.875 in.) thick yellow pine single straight sheathing that runs across the 9.14 m (30 ft.) width of the building. The sheathing width is not provided. The floor layout is relatively simple: the beams, W18x55, span the width of the building and the wood joists, 2x10 (nominal) at 40.6 cm (16 in.) centers, span the distance between the beams across the longer, 18.3 m (60 ft.) building dimension (see Fig 3.2).

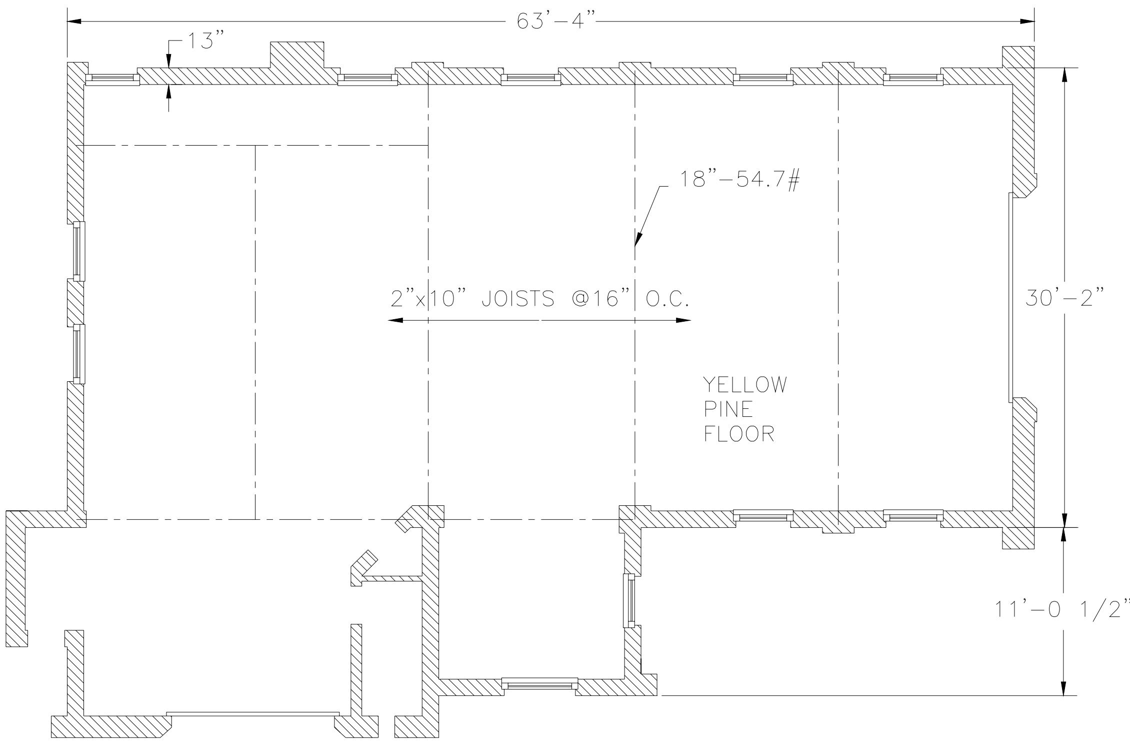


FIG. 3.2 Plan Layout of Case Study Building 1

The connection of the wood joists to the W18x55 is shown in Fig. 3.3. The metal ceiling for the first story is attached to 2x4 (nominal) boards, which are hung from the floor joists by pairs of 2x4 (nominal) nailers found on either side of the beams. The layout of the beams and joists are similar to more modern designs. The 2x10 (nominal) wood joists are attached to the top of the beam by steel strapping and gravity support is provided at mid-height of the W18x55 by two 3x4x $^{3}/_{8}$ in. angles. Both the 2x10 joists and the 2x4 boards are spaced at 40.6 cm (16 in.) on center.

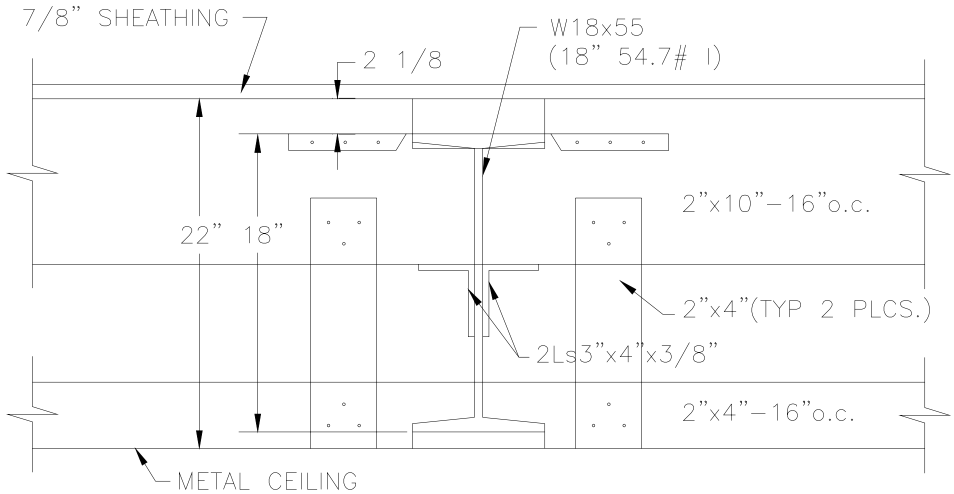


FIG. 3.3 Typical Joist-to-Beam Connection Detail

Fig. 3.4 describes the connection between either end of the W18x55 beams and the URM walls. The beams sit on a 30.5 cm (12 in.) square steel bearing plate located in a pocket in the URM wall. There is no information in the drawings to indicate that the connection is welded.

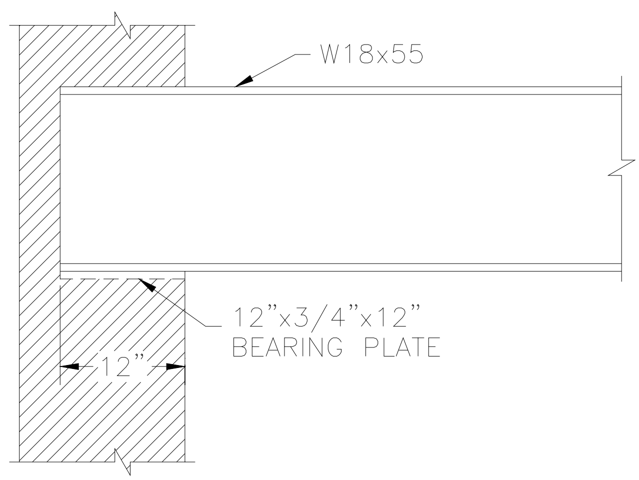


FIG 3.4 Beam-to-Wall Detail

3.3 CASE STUDY BUILDING 2

3.3.1 General Description

Case Study Building 2, shown in Fig. 3.5, is also a firehouse located in St. Louis, Missouri. This building was constructed in 1957 using the same general design and construction methods as pre-1950s URM firehouses in the city. However, some of the materials and structural components in the firehouse are more modern, including the use of a concrete slab at the floor level and steel bar joists at the roof level. Even with the changes, the similar layout of the newer case study building to the older one does not have a significant impact on the seismic behavior.

FIG. 3.5 Case Study Building 2

The firehouse is a two-story, URM building with a wood roof diaphragm. The first floor diaphragm is a thin 6.35 cm (2.5-in.) concrete slab supported by steel joists. The supports on this floor are identical to those on the roof level and are discussed in more detail later. The wood roof diaphragm is the primary focus of the analysis for this case study building.

The length-to-width aspect ratio of the building is approximately 1.9:1.0. The shorter side, 13.8 m (45.3 ft.), faces the main street and houses the fire engine entrance to the building. The longer dimension, 26.3 m (86.3 ft.), has a few openings on the first story along with a pedestrian walkway access through a door on the east side. The ground floor of the building is almost completely open space and is primarily used for parking fire engines and storing equipment. The upper floor has a few room divisions for dormitory, locker, recreational, and officers' rooms. The composite roof has a slight slope for drainage. A shaft for a small hose tower is located on the south side of the building.

The beams and joists used in the building are made of steel, which is more typical of URM buildings constructed after 1950, such as this one. However, due to the size and orientation of the beams and joists, the steel joists impact only the gravity load behavior of the floor, and do not influence the flexible behavior and stiffness of the wood sheathed roof diaphragm for in-plane loading. On the day of the site visit, there were ongoing, non-structural improvements being made to the building. The beams and joists supporting the roof sheathing were exposed, allowing the research team a chance to view these structural members.

3.3.2 Structural Details

Fig. 3.6 provides a plan layout and details for the roof of Case Study Building 2. Similar to Case Study Building 1, the drawing labels for the type of beams and joists used in the building are not standard callouts that would be expected on current structural design drawings. The joists and beam callouts referenced here are the modern name equivalent to the members used in the floor system. The drawings shown here are adapted from the original building drawings and do not necessarily provide all the information one might expect to find on more modern structural details.

The URM walls are three wythes of clay brick and approximately 31.8 cm (l2.5 in.) thick. The story heights are only slightly different from one another: the first story height is 38.1 m (15.8 ft.) and the second story height is 4.51 m (14.8 ft.).

The main beams span the short dimension of the building. The first floor beams range in size from W36x150 to W36x182, but the roof beams are all W27x94. Simple joists, W10x54 (SJ 102), are spaced 51 cm (20 in.) on centers and span the distance between the beams. Thus, the joists are parallel to the long direction of the building (see Fig. 3.6).

Fig. 3.7 shows a schematic diaphragm and photograph of the connection detail for the four beams that are connected into steel columns embedded in the supporting URM walls at the roof level. The beam is connected to the columns with two angles with an erection angle provided below the bottom flange.

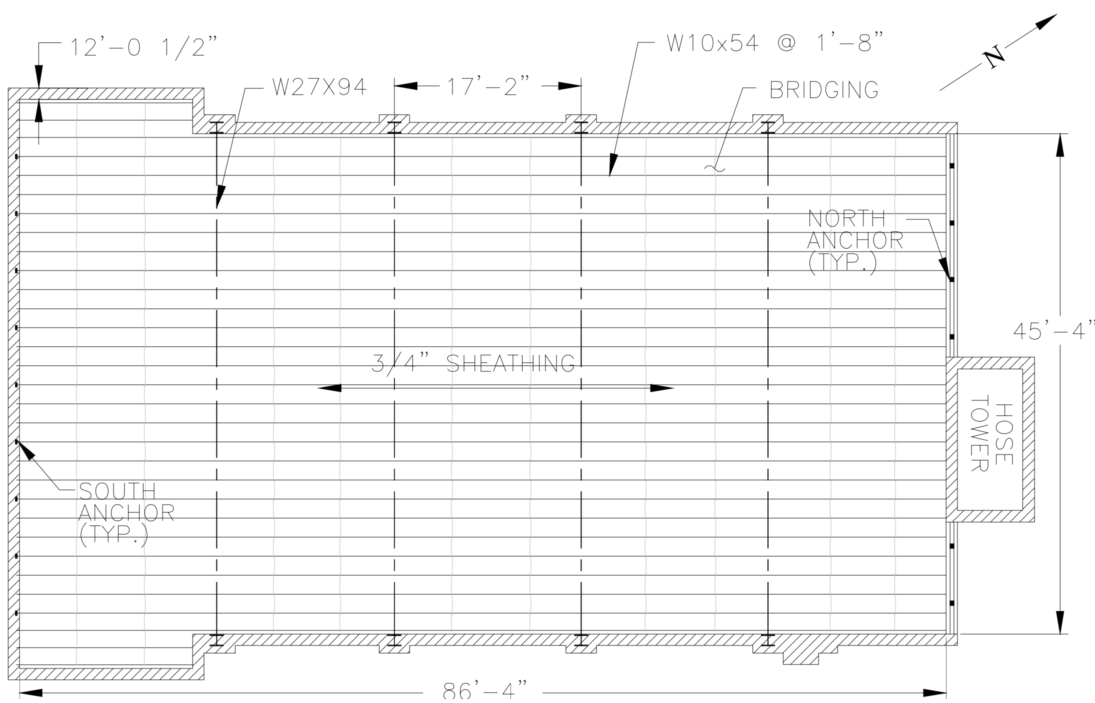
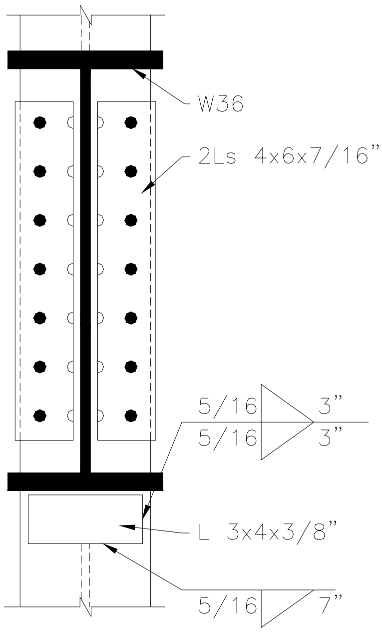


FIG. 3.6 Plan Layout of Case Study Building 2 (Roof Level)

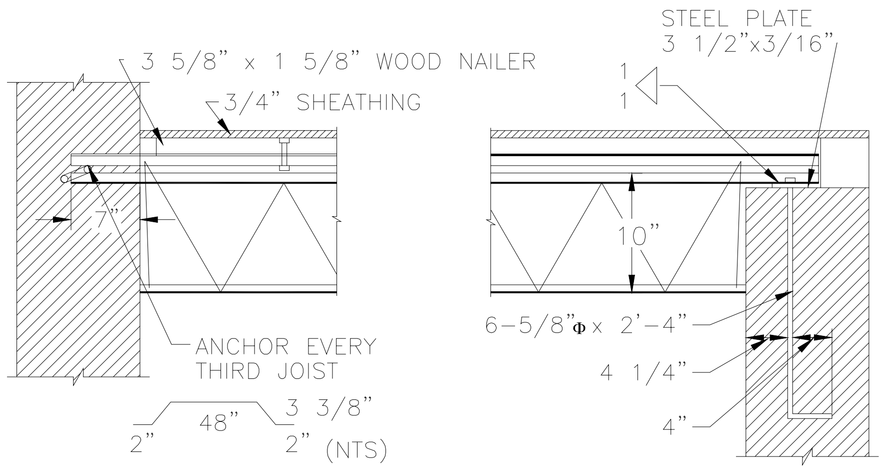


(a) Schematic Diagram **(b) Photograph**

FIG. 3.7 Masonry Wall-to-Beam Connection at Roof Level

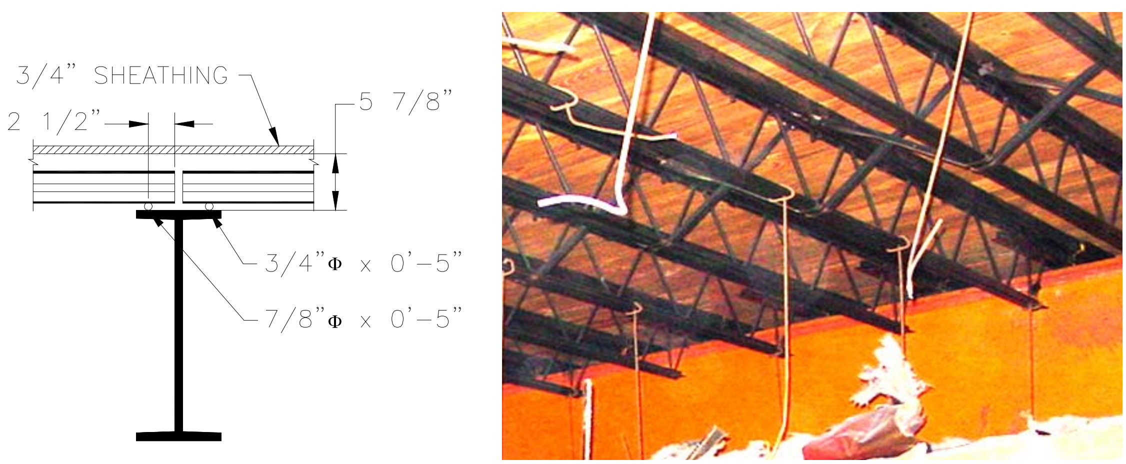
The typical connection of the joist wall anchors is shown in Fig. 3.8. On the South wall, every third joist is anchored to the masonry wall according to the detail shown in Fig. 3.8a. The small anchor shown appears to be a bent bar that is clamped into place from the weight of the material above it. The north wall anchors are formed by a steel plate welded to the joist and anchored with a 1.59 cm (0.625 in.) diameter bar embedded 71.1 cm (28 in.) into the masonry wall.

The roof diaphragm is 1.91 cm (0.75 in.) thick, single straight sheathing connected to the joists by a 2x4 in. (nominal) nailer that is attached to the top of the joists with screws (Fig. 3.9). The width of the sheathing boards is not mentioned. The detail showing the bearing of the steel joists on the supporting steel beams is shown in Fig. 3.9. The joists are supported on the beam using two bars with different diameters, one bar with 1.91 cm (0.75 in.) diameter and one with 2.22 cm (0.875 in.), which permits slight sloping of the roof for drainage.



(a) South Wall Anchor **(b) North Wall Anchor**

FIG. 3.8 Joist-to-Wall Detail



(a) Schematic Detail **(b) Photograph**

FIG. 3.9 Typical Beam Bearing Details for Roof Joists

Each of the case study buildings are evaluated using the selected analysis procedure provided by FEMA 273 and FEMA 356 and the results are described in detail in the following chapters.

4. FEMA SEISMIC REHABILITATION GUIDELINES

4.1 GENERAL

To assess the adequacy of the wood diaphragms in the case study buildings, two seismic rehabilitation guidelines were selected: the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* (FEMA 273) (ATC 1997a), and the more recent *NEHRP Prestandard and Commentary for Seismic Rehabilitation of Buildings* (FEMA 356) (ASCE 2000). These guidelines provide analytical procedures and guidelines for the seismic rehabilitation of existing buildings. There are four analysis procedures provided in both FEMA 273 and FEMA 356: (1) the Linear Static Procedure (LSP); (2) the Nonlinear Static Procedure (NSP); (3) the Linear Dynamic Procedure (LDP); and (4) the Nonlinear Dynamic Procedure (NDP).

The scope of the case study building evaluation is limited to applying the FEMA guidelines to the wood diaphragms. In general, FEMA 356 is a revised and updated version of FEMA 273. FEMA 273 is accompanied by a companion document containing the relevant commentary, FEMA 274, whereas FEMA 356 is a combined standard and commentary. The two sets of guidelines contain few, but potentially critical, differences for the evaluation of existing buildings. FEMA 356 contains a few more specific discussions for URM buildings, which will be described later in more detail.

The following sections briefly discuss the four analysis procedures provided in FEMA 273 and FEMA 356 in the context of applying these methods to the case study building diaphragms. However, the LSP is the only procedure that was used in this study to evaluate the case study buildings. This procedure permits a component evaluation of the diaphragms without requiring a URM wall model, as discussed in the following sections.

4.2 LINEAR STATIC PROCEDURE (LSP)

The LSP analysis determines the elastic structural response for an equivalent static lateral force distribution. The maximum predicted base shear force for a specified demand displacement is used to determine the pseudo-lateral load, which is distributed over the height of the building for the analysis. The actual strength of a structure is over-predicted in the nonlinear range of behavior by assuming the building will behave elastically (see Fig. 4.1). Therefore, member forces determined for the maximum demand may exceed the actual strength. The LSP accounts for the discrepancy between actual member strength and computed member forces through a ductility factor used in the member-level evaluation.

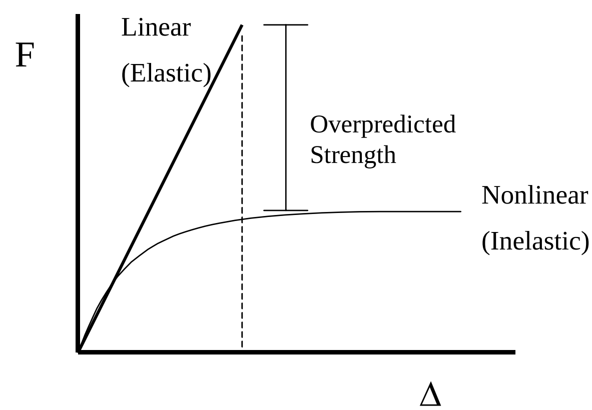


FIG. 4.1 Comparison of Linear and Nonlinear Force versus Displacement Relationship

The LSP may be applied to both case study buildings and is relatively simple to use. It was selected for this study to examine the adequacy of using simplified methods in evaluating the existing diaphragms and for selecting a sufficient diaphragm retrofit. Chapter 5 describes the details of the LSP for both FEMA 273 and FEMA 356 and provides the analytical results for this procedure when applied to the diaphragms of the case study buildings.

4.3 NONLINEAR STATIC PROCEDURE

4.3.1 General

The NSP is the second of the two static methods available in the FEMA guidelines. The NSP involves computing the member forces for a structure at a target lateral displacement. The target displacement is intended to be equivalent to the maximum displacement during the design earthquake when considering inelastic material behavior. The fundamental idea is to monotonically increase loading on a representative building model, using a nonlinear push-over analysis, until the predetermined target displacement has been reached. The corresponding internal forces and deformations are determined. The computed forces are thought to provide reasonable approximations of the internal forces that would develop during the design earthquake.

4.3.2 Applicability of NSP Analysis

According to both FEMA guidelines, nonlinear analysis procedures should be applied when the linear procedures are deemed inapplicable. Additionally, the NSP is permitted when higher mode effects are not significant. Higher mode effects are classified as significant when the story shear in any story which is required to obtain 90% mass participation exceeds the story shear in the first mode by more than 130%. If higher mode effects are significant, the NSP is still applicable if the LDP is used as a supplement. The higher mode effects in URM structures similar to the case study buildings were investigated using SAP 2000 (CSI 1999) with the model described in Chapter 7. The story shears in the analysis do not exceed the limitations for the NSP, therefore the higher mode effects were deemed insignificant and the NSP is applicable for the case study buildings.

The NSP is permitted for the following rehabilitation objectives: (1) local modification of existing components, (2) removal or lessening existing irregularities, (3) global structural stiffening, (4) mass reduction, or (5) seismic isolation. The selected rehabilitation objective for the case study buildings is to locally modify the existing diaphragm as a structural component. Thus, the nonlinear static procedure is applicable for the two case study buildings because the rehabilitation objective is permitted and higher mode effects are not significant.

4.3.3 Description of NSP Analysis

As stated previously, the NSP is based on deforming the structure to a target displacement. The target displacement, δ_t, requires the specification of a control node in the building of interest. The control node, by definition, is located at the center of mass at the roof level of the building, excluding penthouses. Additionally, if the building contains multiple flexible diaphragms, a control node and target displacement should be determined for each line of vertical seismic framing. Lateral forces are applied monotonically until this control node exceeds the target displacement. The manner in which the lateral forces may be applied are also described in the FEMA guidelines.

Determining the target displacement is an iterative procedure. Many of the factors in the calculation of the target displacement, δ_t, are derived from the results of the nonlinear push-over curve developed for a particular building. The sequence shown in Eqs. 4.1 through 4.3 (ATC 1997a) is the FEMA 273 recommended procedure for determining δ_t, which is established by the following relationship. FEMA 356 has slight differences in some of the equations below but undergoes the same iterative process.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{4.1}$$

where:

T_e = Effective fundamental period of the building (s)
C_0 = Factor relating spectral and roof displacement (1.2 for a two-story building)
C_1 = Factor relating expected maximum inelastic displacements to displacements calculated for linear elastic response, based on the relationship between T_e, R and T_o

C_2 = Factor representing effect of hysteresis shape on maximum displacement (1.1 for Life Safety and $T > T_o$)
C_3 = Factor to represent increased displacements due to dynamic P-Δ effects (Eq. 4.2)
S_a = Response spectrum acceleration at the effective fundamental period (g)
T_0 = Characteristic period of response spectrum(s)
R = Ratio of elastic strength demand to yield strength coefficient (Eq. 4.3)

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \tag{4.2}$$

where:
α = Ratio of post-yield stiffness to effective elastic stiffness (Fig. 4.1)

$$R = \frac{S_a}{V_y / W} \frac{1}{C_0} \tag{4.3}$$

where:
V_y = Yield strength calculated using results of NSP based on Fig 4.1, (N/m) (lb/ft.)
W = Total dead load and anticipated live load (N) (kips)

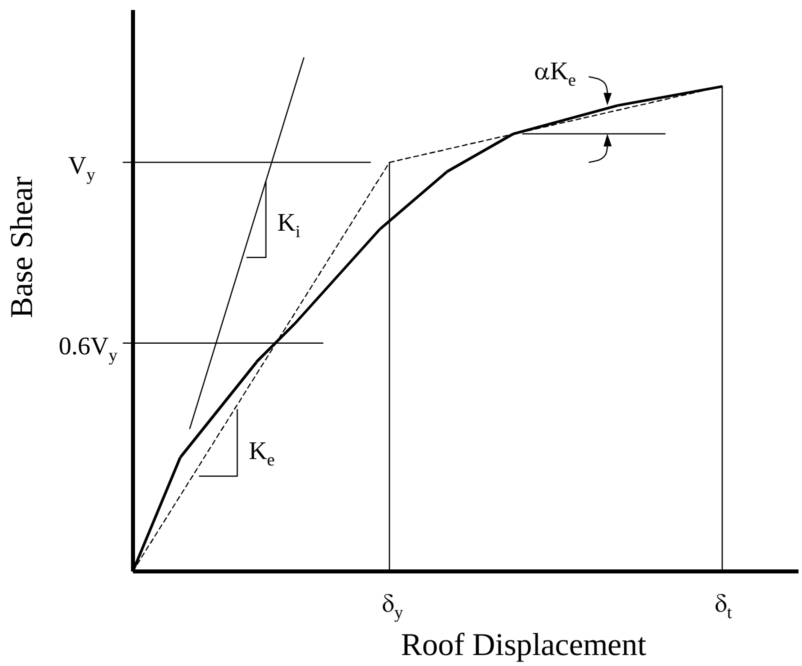


FIG. 4.2 Effective and Elastic Stiffness Relationship (Adapted from ATC 1997a)

The terms C_1 and C_3 utilize the effective fundamental period of the building, T_e, which is a function of the elastic fundamental period and the ratio of the elastic lateral stiffness and the effective lateral stiffness. The effective lateral stiffness, K_e, is found from the results of the NSP (see Fig. 4.2). This sequence of calculations requires an iterative computation of the target displacement, δ_t. The values for stiffness are found from the results of a nonlinear static (push-over) analysis of the building with the prescribed lateral load distribution. Definition of a nonlinear analytical model to describe these building properties makes this analysis difficult for a URM building, because appropriate nonlinear models are not well quantified in the literature for such structures.

4.3.4 Application of NSP Analysis to Case Study Buildings

An accurate structural model is necessary to perform the necessary iterations for the NSP analysis because this procedure is heavily dependent on accurate iterative results. Although a reasonably accurate building model can be developed for a reinforced concrete or steel building, based on current knowledge and analytical tools, this is not the case for URM structures. A simple model can be simulated similar to that of the spring model shown previously in Fig. 2.2. However, creating a finite-element model containing nonlinear properties that accurately predict damage mechanisms in the URM walls is a complex task and would require simplifying assumptions. Thus, a model may be produced to approximate the performance that a URM structure may have under monotonic lateral loading, but the possible inconsistencies that can come from individuals making modeling assumptions based on limited information makes this procedure less desirable for evaluating URM structures. Because the focus of this study was on the diaphragm components, the necessity of modeling the entire structure using a nonlinear model to determine the target displacement made the NSP less desirable for this evaluation.

4.4 DYNAMIC PROCEDURES

The LDP is developed based on the same premise for predicting strength and displacement criteria as the LSP, but instead utilizes a time-history analysis to calculate the response of the building. As in the LSP, the outcome of this analysis are representative displacements for the building under the design earthquake, but the internal forces may be overestimated because nonlinear behavior is not included. Like the NSP, the LDP requires an accurate analytical representation for a URM building, although elastic models may be used for the LDP. While an elastic model can be created, this representation is limited in that the nonlinear structural behavior is not included.

The NDP is applicable for the same building types as the NSP and utilizes time history analysis for the response computation like the LDP. Ultimately, the NDP contains the same limitation as the NSP. It is difficult to develop a model of an URM building that accurately predicts the dynamic structural response into the nonlinear range of behavior. Therefore, the LSP was chosen for use in evaluating the case study building wood diaphragms as components of the structural system. This approach uses a relatively simple modeling and analytical procedure to determine the adequacy of the diaphragms and to assess the need for rehabilitation.

5. LINEAR STATIC PROCEDURE

5.1 GENERAL

The Linear Static Procedure (LSP) uses a pseudo-lateral load applied over the height of the building to approximate the maximum displacements during a design earthquake using an elastic analysis. If a building behaves elastically during an earthquake, then the actual demands that develop may be predicted by an elastic analysis. If the design earthquake causes the building to behave inelastically, then the elastic analysis over-predicts the force demands but is assumed to give a reasonable estimate of the lateral displacements. The LSPs described in both the FEMA 273 and FEMA 356 guidelines were used to evaluate the wood diaphragms in the case study buildings. Detailed calculations for both case study buildings, including calculations for retrofit options, are provided in Appendices A thru E.

5.2 LINEAR STATIC PROCEDURE USING FEMA 273

5.2.1 Applicability of Linear Procedures

The LSP contained in FEMA 273 is applicable for building rehabilitation as long as the building of interest meets the demand-to-capacity ratio (DCR) requirements. These requirements state the LSP may be used for any building as long as the demand, as calculated by the linear procedure, is no more than twice the expected strength of the component, regardless of regularity. This comparison is made for each component in the rehabilitated building. If all components meet the criteria, then any of the linear procedures are applicable. It is important to note that these ratios are only used to determine the applicability of these procedures and not to determine the acceptability of a component's behavior. If the DCR exceeds 2.0, the linear procedures no longer apply if any of the following irregularities are present: in-plane or out-of-plane discontinuities in any primary element of the lateral-force-resisting system, severe weak story irregularity, or torsional strength irregularity.

Additional criteria must be met for the LSP to be applicable: (1) the total building height must be less than or equal to 30.5 m (100 ft.), (2) the ratio of the horizontal dimension from one story to the next must be less than 1.4, (3) the lateral drift along any side of the structure may not exceed 150% of the average story drift, and (4) the building must have an orthogonal lateral force resisting system. However, the required demand of a component cannot be determined until the LSP analysis is complete. Therefore, the applicability of the FEMA 273 LSP analysis can only be determined at the end of the analysis for this procedure.

5.2.2 Details of Linear Static Procedure

FEMA 273 suggests that the building under consideration satisfy the performance objectives of a specific seismic demand. The event must meet the following criteria: a BSE-1 earthquake event with a magnitude equal to the *smaller* of a 10% probability of exceedance in 50-years, or two-thirds of the maximum considered event, which is defined as 2% probability of exceedance in 50-years, evaluated for a Life Safety Performance Level. For St. Louis, the earthquake that satisfies these criteria is the 10% in 50-years earthquake. The St. Louis region most closely fits in Site Class C for the soil conditions typically found in St. Louis. FEMA defines Class C soils as very dense soils and soft rock. These designations are used to adjust the mapped spectral response acceleration parameter. Taking all of this into consideration provides adequate information to determine the short period, S_{XS}, and one-second, S_{X1}, design response spectrum parameters (see Table 5.1).

TABLE 5.1 Design Response Acceleration Parameters for Case Study Buildings

Case Study Building	S_{XS} (g)	S_{X1} (g)	S_a (g)
Building 1	0.207	0.090	0.145
Building 2	0.207	0.090	0.207

The last term shown in Table 5.1 is the spectral response acceleration, S_a. This parameter is the acceleration at which the building is excited for the natural frequency of interest. The forces that develop in the building, which are based on the value of this parameter, are calculated by means of a static procedure. S_a is taken from the general response spectrum provided in the guidelines. The spectrum from FEMA 273 is shown in Fig. 5.1.

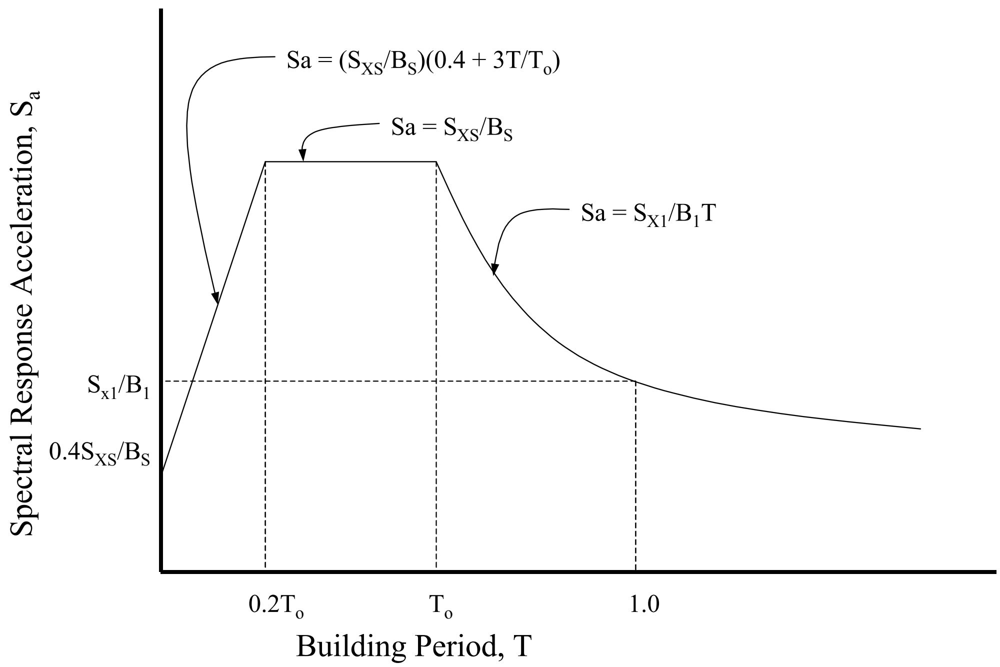


FIG. 5.1 General Response Spectrum for FEMA 273 (Adapted from ATC 1997a)

The pseudo-lateral load for the LSP analysis is based on the building weight, the spectral response parameter, and a series of constants (ATC 1997a) (see Eq. 5.1), and is represented by static loading distributed over the height of the building. These coefficients are dependent on the fundamental period of the building, performance level, framing type, and P-Δ effects.

$$V = C_1 C_2 C_3 S_a W \tag{5.1}$$

where:

V = Pseudo lateral load equal to the total base shear (N) (kips)
S_a = Response spectral acceleration at the fundamental period and damping ratio of the building in the direction under consideration (m/s^2) (ft/s^2)
W = Total dead load and anticipated live load (N) (kips)
C_1 = Modification factor relating maximum inelastic displacements to those calculated for linear elastic response
1.5 for $T < 0.10$ s
1.0 for $T \geq T_o$
C_2 = Modification factor accounting for stiffness and strength degradation on maximum displacement
1.1 for Framing Type 1, $T > T_o$, Life Safety Performance Level
C_3 = Modification factor representing increased displacements due to P-Δ effects, 1.0 for $\theta < 0.1$
θ = Indicative of stability of a structure under gravity loads and lateral deflection induced by earthquakes
T = Fundamental period of the building (s)
T_o = Characteristic period of the response spectrum (s)

For simplicity, and to use the two case study buildings to represent generic URM structures, the wall openings in both buildings were ignored in the determination of the pseudo-lateral load. For these two case study buildings, P-Δ effects were not significant. Therefore, the corresponding constant, C_3, was set to 1.0 for both cases. Notice that C_1 compares the fundamental building response to the characteristic period of the response spectrum, T_o. FEMA 273 provides an equation to estimate the fundamental building period for a one-story building with a single span flexible diaphragm given in Eq. 5.2 (ATC 1997a). The estimated period is dependent on the in-plane wall and diaphragm displacement created by a lateral load equal to the weight tributary to the diaphragm.

$$T = \left(0.1\Delta_W + \Delta_d\right)^{0.5} \tag{5.2}$$

where:

T = Fundamental building period (s)
Δ_w = In-plane wall displacement due to a lateral load equal to the weight tributary to the diaphragm (cm) (in.)
Δ_d = Diaphragm midspan displacement due to a lateral load equal to the weight tributary to the diaphragm (cm) (in.)

FEMA 273 also provides an expression to estimate diaphragm displacement, shown here as Eq. 5.3. (ATC 1997a). The equation is the same for all types of sheathing, but the guidelines provide different shear stiffness values for the various types of sheathing. In both case study buildings, the existing diaphragm is composed of single straight wood sheathing. FEMA 273 assigns a diaphragm shear stiffness of 35,000 N/cm (200,000 lb/in.) to this type of sheathing.

$$\Delta = \frac{\nu L^4}{G_d b^3} \tag{5.3}$$

where:

Δ = Calculated diaphragm deflection (cm) (in.)
ν = Maximum shear in direction under consideration (kg/m) (lb/ft.)
G_d = Diaphragm shear stiffness (kg/cm) (lb/in.)
L = Diaphragm span between shear walls (m) (ft.)
b = Diaphragm width (m) (ft.)

The shear stiffness, G_d, is actually the in-plane stiffness of the floor diaphragm equal to the shear modulus times the thickness, t, of the diaphragm (Isoda et al., 2002).

Using Eqs. 5.1 through 5.3, along with SAP 2000 to calculate an estimated in-plane stiffness for the in-plane walls, the fundamental building period and pseudo-lateral load were found for both case study buildings. A summary of these calculations is shown for both case study buildings in Table 5.2.

TABLE 5.2 Summary of LSP Design Parameters for Case Study Buildings

Case Study Building	W kN (kips)	G_d kN/m (lb./ft.)	T (s)	T_o (s)	V kN (kips)
1	1,790 (403)	2,920 (200,000)	0.622	0.436	344 (77.4)
2	3,300 (741)	2,920 (200,000)	0.430	0.436	618 (139)

Once these design parameters have been determined, a series of equations provided by FEMA 273 are used to calculate the demand for the diaphragm at each level. Generally, these equations distribute the pseudo-lateral load based on the mass distribution over the building height. Initially, the procedure determines the load applied at each floor level based on the building weight and the height of the floor from the base of the building as shown in Eqs. 5.4 and 5.5 (ATC 1997a).

$$F_x = C_{VX} V \tag{5.4}$$

$$C_{VX} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{5.5}$$

where:

F_x = Lateral load applied at any floor level (N) (kips)
C_{vx} = Vertical distribution factor
w_i = Portion of total building weight, W, located on or assigned to floor level i (N) (kips)
w_x = Portion of total building weight, W, located on or assigned to floor level x (N) (kips)
h_i = Height from base to floor level i (m) (ft.)
h_x = Height from base to floor level x (m) (ft.)
k = 1.0 for $T \leq 0.5$ (sec)
2.0 for $T > 2.5$ (sec) (linear interpolation used between)
C_1, C_2 and C_3 are as described for Eq. 5.1

The force on each diaphragm is then found with the constants used previously to determine the pseudo lateral load, along with the weight distribution of the building at each floor level as given in Eq. 5.6 (ATC 1997a).

$$F_{px} = \frac{1}{C_1 C_2 C_3} \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \tag{5.6}$$

where:

F_{px} = Total diaphragm force at level x (N) (kips)

The diaphragm force is used to evaluate the flexibility of the diaphragms and to check diaphragm strength. The total diaphragm force can then be used once more in Eq. 5.3 to determine the midspan lateral displacement of the diaphragm. Table 5.3 shows a summary of the diaphragm forces for each case study building.

TABLE 5.3 LSP Diaphragm Demands

Case Study Building	Force Applied to Diaphragm kN (kips)
1	143 (32.2)
2	193 (43.6)

As described in Section 5.2.1, the applicability of the LSP to the case study building diaphragms could not be determined until the analysis was complete. Table 5.4 shows that the DCR for each case study building diaphragm exceeds 2.0. However, the buildings do not have irregularities and so the LSP can still be used.

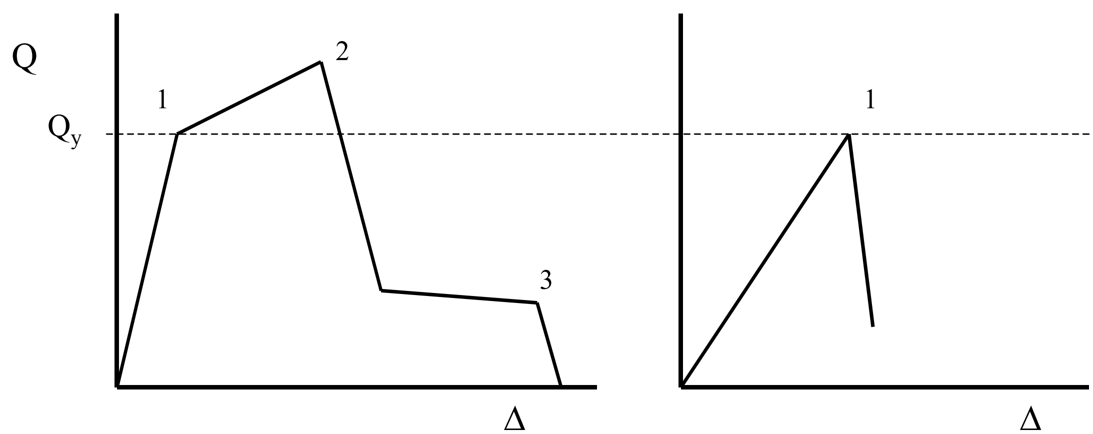
TABLE 5.4 FEMA 273 LSP Diaphragm DCRs

Case Study Building No.	DCR
1	7.28
2	8.01

5.2.3 FEMA 273 Linear Static Analysis Acceptance Criteria

The demands previously identified are all determined with the intention of satisfying deformation-controlled or force-controlled criteria, as outlined by FEMA 273. A deformation-controlled element is typically a ductile element characterized by an elastic and inelastic range of behavior. The strength in the inelastic range, points 1 to 3 on Fig. 5.2, may be less than that of the peak strength, but be at least significant. If the inelastic range of an element is large enough, the element is considered deformation-controlled (see Fig. 5.2a). A force-controlled element is more likely to be a brittle element that has primarily an elastic range of strength exhibited by the component (see Fig. 5.2b).

Flexible wood diaphragms are ductile elements that are more likely to be deformation-controlled. Out-of-plane URM walls are more brittle elements and are more likely to be force-controlled. However, both deformation and force-controlled criteria are provided for wood diaphragms and URM walls.



(a) Deformation-Controlled Behavior **(b) Force-Controlled Behavior**

FIG. 5.2 Component Behavior Relationships (Adapted from ATC 1997a)

5.2.3.1 Force-Controlled Actions

The acceptance criterion for force-controlled actions using the linear procedures compares the value of the design action with the strength of a component in the linear elastic range of behavior. The design action, Q_{UF}, is determined based on gravity and earthquake demands, but the earthquake demand is reduced by the series of constants used in calculating the pseudo-lateral load (see Eq. 5.7).

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} \tag{5.7}$$

where:

Q_{UF} = Design actions due to gravity loads and earthquake loads (N) (kips)
Q_E = Action due to design earthquake loads calculated using forces in Eq. 5.6 (N) (kips)
Q_G = Action due to design gravity loads when they counteract or are additive to seismic loads (N) (kips)
C_1, C_2, C_3 as defined in Eq. 5.1

The design action Q_{UF} must be less than a knowledge factor times the lower-bound strength, Q_{CL}, of the component of concern, as shown in Eq. 5.8. For this study, a minimum level of knowledge was assumed in selecting the knowledge factor.

$$\kappa Q_{CL} \geq Q_{UF} \tag{5.8}$$

where:

κ = Knowledge factor (0.75 for minimum level)
Q_{CL} = Lower-bound strength of a component or element at the deformation level under consideration (N) (kips)

5.2.3.2 Deformation-Controlled Actions

The criteria according to FEMA 273 for deformation-controlled actions using the linear procedures are as follows: the demands imposed on the diaphragm, Q_{UD} calculated according to Eq. 5.9, must be less than the expected strength of the diaphragms, Q_{CE}, multiplied by two factors to account for knowledge and ductility as shown in Eq. 5.10. FEMA 273 provides values for both of these factors.

$$Q_{UD} = Q_G \pm Q_E \tag{5.9}$$

where:

Q_{UD} = Design action due to gravity loads and earthquake loads (N) (kips)
Q_E, Q_G as defined in Eq. 5.7

$$m\kappa Q_{CE} \geq Q_{UD} \tag{5.10}$$

where:

m = Demand modifier to account for expected ductility (1.5 for unchorded, single straight sheathing, Life Safety, and length-to-width aspect ratio less than 2.0)
Q_{CE} = Expected strength of a component at the deformation level under consideration equal to the yield capacity per unit length of the diaphragm times the width (N) (kips)

For the case study buildings, again the minimal level of knowledge, κ = 0.75, was considered in evaluating the acceptance criteria for a conservative analysis. The demand modifier, *m*, to account for ductility, is dependent on the type of sheathing, the length-to-width aspect ratio, and the required performance level of the diaphragm. Table 5.5 provides a summary of the force-controlled and deformation-controlled criteria for both case study building diaphragms. As demonstrated by this procedure, the existing diaphragms in both case study buildings do not meet the FEMA 273 requirements as either a force-controlled or deformation-controlled element.

TABLE 5.5 LSP Diaphragm Acceptance Criteria for Case Study Buildings

Case Study Building	***m***	**κ**	**Deformation-Controlled**		**Force-Controlled**	
			$m\kappa Q_{CE}$ **kN (kips)**	Q_{UD} **kN (kips)**	κQ_{CL} **kN (kips)**	Q_{UF} **kN (kips)**
1	1.5	0.75	18.1 (4.07)	117 (26.4)	23.4 (5.70)	89.0 (20.0)
2	1.5	0.75	27.2 (6.12)	194 (43.6)	34.6 (7.77)	177 (39.9)

It is important to note that the guidelines further clarify that the deformation limitations of the diaphragm are dependent upon the out-of-plane limitations of the masonry.

> *Deformation acceptance criteria will largely depend on the allowable deformations for other structural and nonstructural components and elements that are laterally supported by the diaphragm. (ATC 1997a, Sec. 8.5.2.3).*

The deformation acceptance criterion for out-of-plane, unreinforced masonry in FEMA 273 is based solely on a wall height-to-thickness (*h*/*t*) ratio shown in Table 5.6 (ATC 1997a). These *h/t* ratios are used to ensure dynamic stability of the out-of-plane URM walls during seismic excitation.

TABLE 5.6 Permissible *h/t* Ratios for URM Out-of-Plane Walls (ATC 1997a)

Wall Types	$S_{X1} \leq 0.24g$	$0.24g < S_{X1} \leq 0.37g$	$0.37g < S_{X1} \leq 0.5g$
Walls of one-story buildings	20	16	13
First-story wall of multistory building	20	18	15
Walls in top story of multistory building	14	14	9
All other walls	20	16	13

Case Study Building 1 has an *h*/*t* ratio of 13.4 and Case Study Building 2 has a *h*/*t* ratio of 15.8 and they both fall into the first column. Table 5.6 is applicable for Life Safety and Collapse Prevention performance levels. FEMA defines three conditions for existing masonry: good, fair and poor. The deformation limitations based on the h/t ratio do not take into account the condition of the masonry, but does permit cracking of the walls as long as the cracked wall segments remain stable. Out-of-plane masonry walls are force-controlled elements. The tensile strength of the masonry should exceed the required demands for the out-of-plane walls. In this analysis, the stiffness of the out-of-plane walls is ignored per FEMA requirements, causing the analysis to rely on the *h*/*t* limitations of Table 5.6.

Both case study buildings contain URM walls that meet the *h*/*t* ratio criteria. Table 5.7 shows a comparison of the *h*/*t* ratio for each Case Study Building with the corresponding maximum *h*/*t* ratio from Table 5.6. As shown, the upper wall of Case Study Building 2 is only marginally greater than the limiting value and this slight exceedance was assumed to be negligible in this analysis.

TABLE 5.7 Comparison of *h*/*t* Ratio Limitations

Wall Types	$S_{X1} \leq 0.24g$	Case Study Building 1	Case Study Building 2
First-story wall of multistory building	20	13.4	15.8
Walls in top story of multistory building	14	10.2	14.8

5.2.4 Identification of Acceptable Diaphragm Retrofit

Table 5.5 demonstrates that for each case study building, the existing diaphragm retrofit is unsatisfactory for the Life Safety performance level for a design earthquake of 10% probability of exceedance in 50 years. Chapter 8 of FEMA 273 contains material parameters to evaluate seven different types of retrofit possibilities: double straight sheathing, single diagonal sheathing, diagonal sheathing with straight sheathing, double diagonal sheathing, wood structural panel sheathing, wood structural panel overlays on straight and diagonal sheathing, and wood structural panel overlays on existing wood structural panels. Each of these possibilities can be evaluated as chorded and unchorded. A chord is a component along the edge of the diaphragm designed to resist lateral tension and compression due to in-plane bending of the diaphragm. Discarding any options that would require the removal of the existing diaphragm, four possible retrofits remained and they were examined both as unchorded and chorded. The remaining eight possible retrofits were as follows: unchorded and chorded double straight sheathing, unchorded and chorded diagonal sheathing with straight sheathing, unchorded and chorded blocked panel overlay, and unchorded and chorded unblocked panel overlay.

The Linear Static Procedure and acceptance criteria evaluation described in Section 5.2.2 was performed for each of the possible retrofits listed above. Tables 5.8 and 5.9 summarize the critical parameters from each analysis for both case study buildings. The force values for the three retrofit options that meet the deformation acceptance criterion are marked using bold type.

TABLE 5.8 FEMA 273 LSP Diaphragm Retrofits for Case Study Building 1

Retrofit		Yield Capacity N/m (lb/ft.)	G_d N/cm (lb/in.)	Building Period (s)	$m\kappa Q_{CE}$ kN (kips)	Q_{UD} kN (kips)
Sheathing	Double Straight Unchorded	5,830 (400)	1,230,000 (700,000)	0.343	60.4 (13.6)	167 (37.5)
Sheathing	Double Straight Chorded	8,760 (600)	2,630,000 (1,500,000)	0.237	121 (27.2)	167 (37.5)
Sheathing	Diagonal with Straight Unchorded	9,120 (625)	1,580,000 (900,000)	0.303	126 (28.3)	167 (37.5)
Sheathing	Diagonal with Straight Chorded	13,100 (900)	3,150,000 (1,800,000)	0.217	**226 (50.9)**	**167 (37.5)**
Plywood Overlays	Panel, Unblocked, Unchorded	4,380 (300)	876,000 (500,000)	0.401	60.4 (13.6)	167 (37.6)
Plywood Overlays	Panel, Unblocked, Chorded	6,570 (450)	1,580,000 (900,000)	0.300	113 (25.5)	167 (37.6)
Plywood Overlays	Panel, Blocked Unchorded	9,810 (672)	1,230,000 (700,000)	0.340	**169 (38.0)**	**167 (37.6)**
Plywood Overlays	Panel, Blocked Chorded	14,000 (960)	3,160,000 (1,800,000)	0.215	**290 (65.2)**	**167 (37.6)**

TABLE 5.9 FEMA 273 LSP Diaphragm Retrofits for Case Study Building 2

	Retrofit	Yield Capacity N/m (lb/ft)	G_d N/cm (lb/in.)	Building Period (s)	$m\kappa Q_{CE}$ kN (kips)	Q_{UD} kN (kips)
Sheathing	Double Straight, Unchorded	5,840 (400)	1,230,000 (700,000)	0.261	90.7 (20.4)	203 (45.6)
	Double Straight, Chorded	8,760 (600)	2,630,000 (1,500,000)	0.194	181 (40.8)	203 (45.6)
	Diagonal with Straight, Unchorded	9,120 (625)	1,580,000 (900,000)	0.236	189 (42.5)	203 (45.6)
	Diagonal with Straight, Chorded	13,100 (900)	3,150,000 (1,800,000)	0.182	**340 (76.5)**	**203 (45.6)**
Plywood Overlays	Panel, Unblocked, Unchorded	4,380 (300)	875,650 (500,000)	0.298	90.7 (20.4)	200 (44.9)
	Panel, Unblocked, Chorded	6,570 (450)	1,580,000 (900,000)	0.232	170 (38.3)	200 (44.9)
	Panel, Blocked Unchorded	9,810 (672)	1,230,000 (700,000)	0.258	**254 (57.1)**	**200 (44.9)**
	Panel, Blocked Chorded	14,000 (960)	3,150,000 (1,800,000)	0.179	**436 (97.9)**	**200 (44.9)**

5.2.5 Discussion of FEMA 273 LSP Results

The building period for the various retrofit options ranged from 0.215 to 0.401 seconds for Building 1 and 0.179 to 0.298 seconds for Building 2. It is interesting to note that the demand for both buildings using the FEMA 273 LSP remained essentially the same for each building, regardless of the variations in the period.

According to the results of the LSP analysis, there are sheathing and plywood overlay retrofits that are acceptable for each case study building. For each building, either of the blocked, plywood retrofits or the chorded diagonal sheathing overlay meets the required demands according to this analysis. The selected retrofit would depend on the reason for the rehabilitation. If aesthetics were a concern and the structural floor was to be exposed, the diagonal sheathing may be the desired choice. However, in many cases the plywood would be chosen because it tends to be more economical, quicker to install, and would displace the inhabitants of the building for less time.

5.3 LINEAR STATIC PROCEDURE USING FEMA 356

5.3.1 General

The LSP described in FEMA 356 is similar to that of FEMA 273. FEMA 356 is actually an update of the FEMA 273 guidelines. The following sections will briefly outline the LSP contained in FEMA 356 and highlight the major differences between the two.

5.3.2 Applicability of Linear Procedures

The FEMA 356 requirements for applicability of linear procedures are identical to those outlined in FEMA 273. The limiting DCR is the same (2.0), and the same structural irregularities must not be present for buildings with components that surpass the DCR limit. There are, however, a few differences for determining the applicability of utilizing the LSP. The four stipulations listed previously in Section 5.2.1 have been modified in FEMA 356, as follows: (1) the fundamental period must be less than 3.5 times characteristic period of the building, (2) the ratio of the horizontal dimensions from one story to the next may not exceed 1.4, (3) the building may not contain a definable severe torsional stiffness irregularity, (4) the drift along any side of the structure can not exceed 150% of the average story drift, and (5) the building must have an orthogonal lateral force resisting system. Again, the required demand of a component cannot be determined until the LSP analysis is complete. Therefore, the applicability of the FEMA 356 LSP analysis can only be determined after the procedure has been applied.

5.3.3 Details of the Linear Static Procedure

FEMA 356 develops the methodology for the LSP in the same manner as FEMA 273. The response acceleration parameters are found from the same maps and adjusted by the same factors for local soil conditions. Thus, the design acceleration parameters are the same.

There is a slight difference in the value of the spectral response acceleration for FEMA 356 (see Fig. 5.3) as compared to FEMA 273. For both guidelines, the building period is greater than T_s, or T_o in the case of FEMA 273. However, because the fundamental building period is calculated differently for the two guidelines, the spectral response acceleration is not the same. Thus, S_a for Building 1 is 0.415g and S_a for Building 2 is 0.207g. The variation of the building period calculation is discussed in the following section.

The seismic demand is again equivalent to a pseudo-lateral load based on the spectral response parameter, building weight, and a series of constants (see Eq. 5.11) (ASCE 2000). However, FEMA 356 includes an additional factor, C_m, to account for higher mode mass participation. This additional term does not affect the analysis of the case study buildings. Again, P-Δ effects represented by C_3 are not significant for the case study buildings.

$$V = C_1 C_2 C_3 C_m S_a W \tag{5.11}$$

where:

C_1, C_3, S_a, W	=	Defined for Eq. 5.1
C_2	=	Modification factor to represent effects of pinched hysteretic behavior, stiffness degradation and strength deterioration on maximum displacement response = 1.0 for linear procedure
C_m	=	Effective mass factor = 1.0 for two-story building

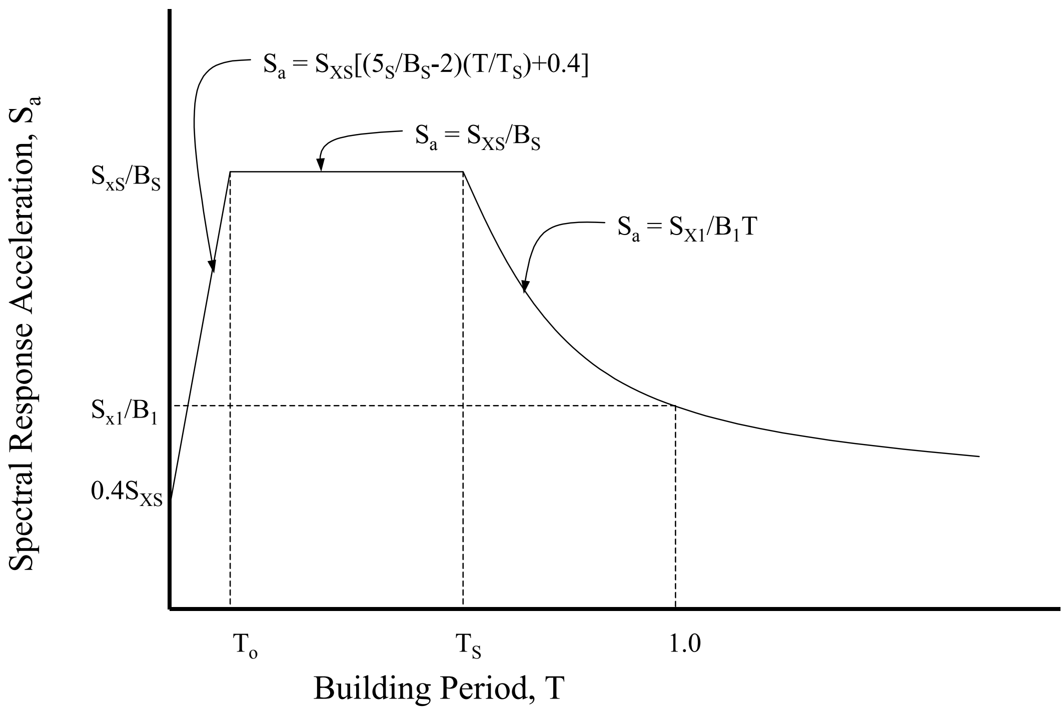


FIG. 5.3 General Response Spectrum for FEMA 356 (Adapted from ASCE 2000)

At this point the slight variations between the two guidelines begin to emerge because the pseudo-lateral load varies due to the difference in the spectral response acceleration. The modification factor C_1, relates maximum inelastic displacements to the displacements calculated for linear elastic response. This is estimated using a comparison of the fundamental building period to the characteristic period of the response spectrum. The characteristic period is determined from the mapped acceleration parameters and is identical between the two guidelines. However, FEMA 356 provides a procedure to estimate the fundamental building period that is more specific for the buildings in this study (see Eq 5.12) (ASCE 2000). This equation does not take the in-plane wall displacement into consideration and can be used for buildings with flexible diaphragms, up to six stories in height.

$$T = \left(0.078\Delta_d\right)^{0.5} \tag{5.12}$$

where:

T = Fundamental building period (s)
Δ_d = Diaphragm midspan displacement due to a lateral load equal to the weight tributary to the diaphragm (cm) (in.)

FEMA 356 also provides an expression for the estimation of the diaphragm displacement for use in Eq. 5.12. Unlike Eq. 5.3, which takes the aspect ratio of the floor into account, the FEMA 356 equation to estimate the diaphragm midspan displacement only considers the diaphragm span, as follows (ASCE 2000).

$$\Delta_y = \frac{v_y L}{2G_d} \tag{5.13}$$

where:

Δ_y = Calculated diaphragm deflection at yield (cm) (in.)
v_y = Shear at yield in the direction under consideration (kg/m) (lb/ft.)
L = Diaphragm span between shear walls (m) (ft.)
G_d = Diaphragm shear stiffness = 3,500 N/cm (2,000 lb/in.)

Additionally, the diaphragm shear stiffness is expressed as a value with a different order of magnitude than that used in FEMA 273. In both case study buildings, the stiffness for the single straight sheathing composing the existing diaphragm is designated as 3,500 N/cm (2,000 lb/in.). Because of these differences, the estimated fundamental building period for the same building varies between guidelines.

FEMA 356 uses the same procedure described by Eqs. 5.4 and 5.5 to distribute the pseudo-lateral load to the separate floors and then to the diaphragms. However, when estimating the distribution of forces to the diaphragm specifically, FEMA 273 removes the constants that were used to calculate the pseudo lateral load (C_1, C_2, and C_3), but FEMA 356 does not, as shown in Eq. 5.14 (ASCE 2000).

$$F_{px} = \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \tag{5.14}$$

where:

F_{px} = Total diaphragm force at level x (kg) (kips)
F_i = Lateral load applied at any floor level (N) (kips)
w_i = Portion of seismic building weight W located on or assigned to floor level i (kg) (kips)
w_x = Portion of seismic building weight W located on or assigned to floor level x (kg) (kips)

FEMA 356 follows by specifying that if using Eq. 5.12 to calculate the building period, this method of distributing the pseudo-lateral load is only applicable if the diaphragm deflection is less than 15.2 cm (6 in.).

The anticipated diaphragm displacement is then estimated utilizing this predicted force, F_{px}, and Eq. 5.9. Table 5.10 shows the fundamental building periods (T), response spectral accelerations at the building period (S_a), diaphragm shear stiffness (G_d), pseudo-lateral loads (V), and the diaphragm force (F_{px}) for each case study building. These calculations resulted in different values compared to FEMA 273.

TABLE 5.10 LSP Demands Predicted Using FEMA 356

Case Study Building	***T*** (s)	***S_a*** (g)	***G_d*** N/m (lb/ft)	***V*** kN (kips)	***F_{px}*** kN (kips)
1	1.44	0.063	29,200 (2,000)	112 (25.3)	61.8 (13.9)
2	1.14	0.079	29,200 (2,000)	261 (58.6)	73.9 (16.6)

Table 5.11 shows that the DCRs are greater than 2.0 for the wood diaphragms in both case study buildings. However, the buildings meet the additional requirements for

regularity when the demand capacity ratio is exceeded, so the FEMA 356 LSP may be used**.**

TABLE 5.11 FEMA 356 LSP –Diaphragm Demand-to-Capacity Ratios

Case Study Building	DCR
1	3.14
2	3.06

5.3.4 FEMA 356 Linear Static Analysis Acceptance Criteria

FEMA 356 also provides acceptance criteria for both force- and deformation-controlled actions. These terms have been defined in Section 5.2.3.

5.3.4.1 Force-Controlled Actions

Assessment of force-controlled actions in FEMA 356 is identical to the FEMA 273 procedure. Again, this criterion does not allow nonlinear behavior of the material and utilizes the lower-bound strength of the diaphragm, Q_{CL}, as a means of comparison to demand values. However, because of differences in determining the seismic demand for the diaphragm (see Eq. 5.14), the design action, Q_{UF}, is considerably smaller when using FEMA 356 versus FEMA 273.

5.3.4.2 Deformation-Controlled Actions

The demands calculated by FEMA 356 must also satisfy deformation-controlled actions based on the strength and ductility of the component. The same factors accounting for level of knowledge of the existing building and ductility utilized in FEMA 273 are combined with the expected strength, Q_{CE}, and compared with the predicted demands, Q_{UD}. The ductility factors, *m*, are identical to those listed in FEMA 273 and are representative of the type of sheathing retrofit. The minimal level of knowledge was assumed again for these case study buildings for a conservative analysis.

Table 5.12 shows the inadequacy of the existing diaphragms in both case study buildings according to the FEMA 356 deformation-controlled and force-controlled acceptance criteria, where in both cases the demand exceeds the corresponding strength value.

TABLE 5.12 LSP Diaphragm Acceptance Criteria for Case Study Buildings

Case Study Building	m	κ	Deformation-Controlled		Force-Controlled	
			$m\kappa Q_{CE}$ kN (kips)	Q_{UD} kN (kips)	κQ_{CL} kN (kips)	Q_{UF} kN (kips)
1	1.5	0.75	18.1 (4.07)	50.6 (11.4)	25.4 (5.70)	50.6 (11.4)
2	1.5	0.75	27.2 (6.12)	73.9 (16.6)	34.6 (7.77)	49.3 (11.1)

In addition to these acceptance criteria, FEMA 356 also points out that the allowable deformation of the diaphragm is heavily dependent on the allowable deformation of other structural or non-structural components. One such component is the out-of-plane URM walls. The same height-to-thickness ratio criteria for the out-of-plane wall dynamic stability check provided in FEMA 273 is given in FEMA 356 (see Table 5.6). This table is applicable for design for Life Safety and Collapse Prevention performance levels only. This does not take into account the condition of the masonry, although cracking of the walls is permitted.

5.3.5 Identification of Acceptable Diaphragm Retrofit

The results of the LSP using FEMA 356 also show that the existing diaphragms in both case study buildings fail to meet acceptable criteria. However, demands determined using the FEMA 356 procedure are considerably less than for FEMA 273. As in FEMA 273, the FEMA 356 guidelines contain parameters for use in evaluating different types of retrofits. Using the same method discussed in Section 5.2.3, the potential retrofits were narrowed down to eight possibilities. The LSP and evaluation of acceptance criteria described in Section 5.3.2 was performed for each of the potential retrofits. Tables 5.13 and 5.14 summarize the critical parameters from each retrofit analysis for both case study buildings.

TABLE 5.13 FEMA 356 LSP Diaphragm Retrofits for Case Study Building 1

	Retrofit	**Yield Capacity** N/m (lb/ft)	G_d N/cm (lb/in.)	T (s)	$m\kappa Q_{CE}$ kN (kips)	Q_{UD} kN (kips)
Sheathing	Double Straight, Unchorded	5,840 (400)	12,300 (7,000)	0.789	60.4 (13.6)	92.3 (20.8)
Sheathing	Double Straight, Chorded	8,760 (600)	26,300 (15,000)	0.539	121 (27.2)	136 (30.4)
Sheathing	Diagonal with Straight, Unchorded	9,120 (625)	15,800 (9,000)	0.696	**126 (28.3)**	**105 (23.5)**
Sheathing	Diagonal with Straight, Chorded	13,100 (900)	31,500 (18,000)	0.492	**226 (50.9)**	**148 (33.3)**
Plywood Overlays	Panel, Unblocked, Unchorded	4,380 (300)	8,760 (5,000)	0.925	60.4 (13.6)	78.8 (17.7)
Plywood Overlays	Panel, Unblocked, Chorded	6,570 (450)	15,800 (9,000)	0.689	**113 (25.5)**	**106 (23.8)**
Plywood Overlays	Panel, Blocked Unchorded	9,810 (672)	12,300 (7,000)	0.782	**169 (38.0)**	**93.3 (20.97)**
Plywood Overlays	Panel, Blocked Chorded	14,000 (960)	31,500 (18,000)	0.487	**290 (65.2)**	**150 (33.6)**

TABLE 5.14 FEMA 356 LSP Diaphragm Retrofits for Case Study Building 2

	Retrofit	Yield Capacity N/m (lb/ft)	G_d N/cm (lb/in.)	T (s)	$M\kappa Q_{CE}$ kN (kips)	Q_{UD} kN (kips)
Sheathing	Double Straight Unchorded	5,840 (400)	12,300 (7,000)	0.647	90.7 (20.4)	137 (30.7)
	Double Straight Chorded	8,760 (600)	26,300 (15,000)	0.442	181 (40.8)	200 (45.0)
	Diagonal with Straight Unchorded	9,120 (625)	15,800 (9,000)	0.571	**189 (42.5)**	**155 (34.9)**
	Diagonal with Straight Chorded	13,100 (900)	31,500 (18,000)	0.404	**340 (76.5)**	**170 (38.2)**
Plywood Overlays	Panel, Unblocked, Unchorded	4,380 (300)	8,760 (5,000)	0.752	90.7 (20.4)	116 (26.1)
	Panel, Unblocked, Chorded	6,570 (450)	15,800 (9,000)	0.561	**170 (38.3)**	**156 (35.0)**
	Panel, Blocked Unchorded	9,810 (672)	12,300 (7,000)	0.636	**254 (57.1)**	**137 (30.9)**
	Panel, Blocked Chorded	14,000 (960)	31,500 (18,000)	0.396	**436 (97.9)**	**168 (37.9)**

5.3.6 Discussion of FEMA 356 LSP Results

Despite changes in the period of the building due to the varying diaphragm retrofit, ultimately the demand imposed on the building remained the same using FEMA 273 procedure. However, in FEMA 356 the demand varies depending on the period of the building. As the period decreases with the addition of the stiffer diaphragm retrofits, the demand on the building increases using the FEMA 356 procedure.

According to the results, there are sheathing and plywood overlay retrofits that are acceptable for each case study building. In each building, either of the blocked, plywood retrofits or the diagonal sheathing overlay meets the required demands according to this analysis. The selected retrofit would depend on the reason for the rehabilitation. If aesthetics were a concern, the diagonal sheathing may be the desired choice. However, in many cases the plywood overlay would be chosen because it tends to be more economical and would displace the inhabitants of the building for less time.

5.4 SUMMARY OF LSP RESULTS

As demonstrated by the analyses utilized for the two case study buildings, the FEMA 273 and FEMA 356 guidelines separately draw similar conclusions for the selection of retrofits, where the objective is to meet the requirements of the Life Safety performance level for a 10% in 50 years earthquake demand. The evaluation based on FEMA 356 led to a larger selection of suitable alternatives than for the FEMA 273 evaluation. The retrofit that will be used for the remaining parametric study will be the chorded, blocked plywood, because all analyses have this retrofit in common and it will provide the most significant difference in the variation in performance from the existing state of the diaphragm. This retrofit is a typical retrofit in such buildings because of its strength and relative economic feasibility. In reality, if the addition of a diaphragm chord and blocking were not needed and a different retrofit was acceptable, the most economical rehabilitation method will be chosen.

It is important to note that both sets of guidelines consistently determine the strength of the diaphragm component. It is only the calculation of the demand on the diaphragm that differs. As shown in Fig. 5.4, FEMA 273 more than doubles the demand prediction of FEMA 356. Similar relationships are true for other building parameters (see Figs. 5.5 and 5.6). In Case Study Building 1, FEMA 273 gives larger values for the force applied to the diaphragm and the base shear by factors of 2.3 and 3.1, respectively. However, the FEMA 356 estimations of both the diaphragm midspan displacement and the building period exceed that of FEMA 273 by a factor of 2.3. In Case Study Building 2, the same relationships are true, except FEMA 273 gives larger values for the first two parameters by factors of 2.6 and 2.4, respectively, while the FEMA 356 predictions for the building period and the diaphragm midspan displacement exceed FEMA 273 by approximately 2.7.

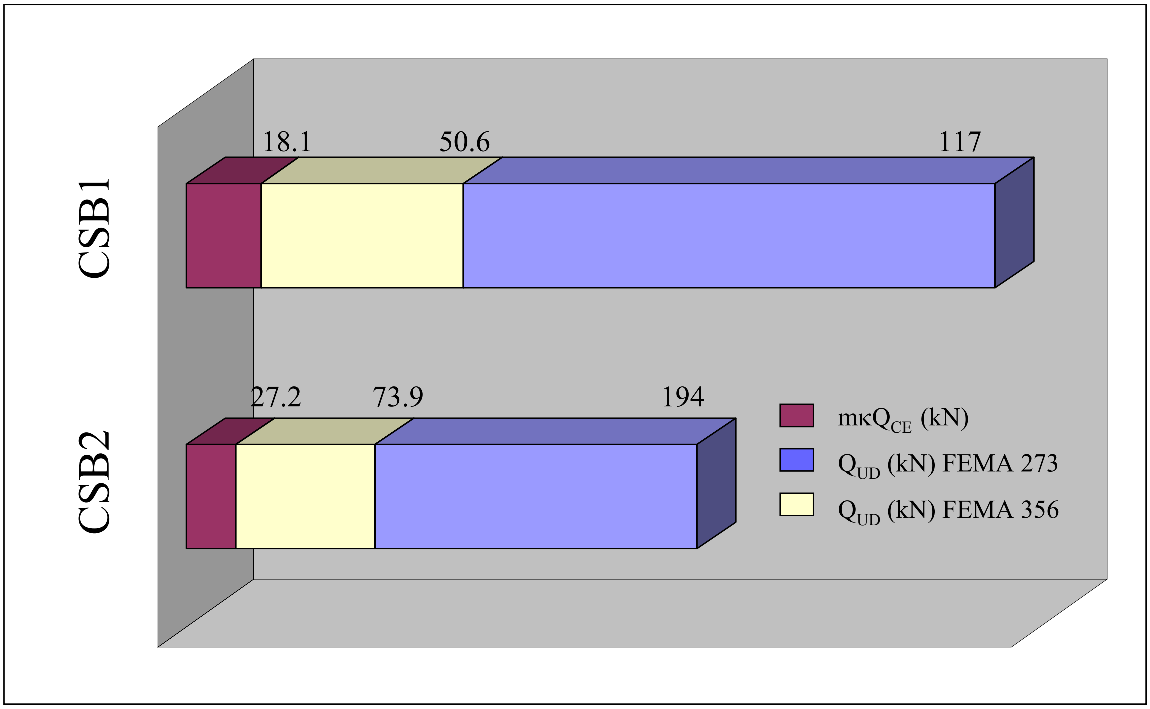


FIG. 5.4 Comparison of FEMA 273 and FEMA 356 Demands

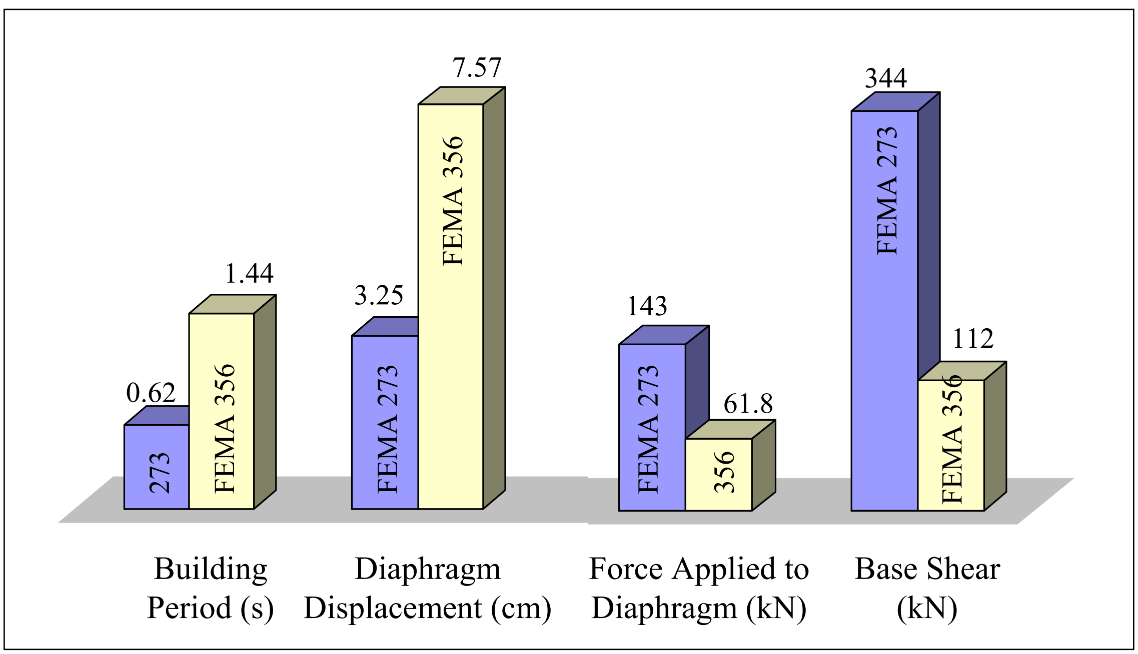


FIG 5.5 Comparison of FEMA 273 and FEMA 356 Predictions for Case Study Building 1

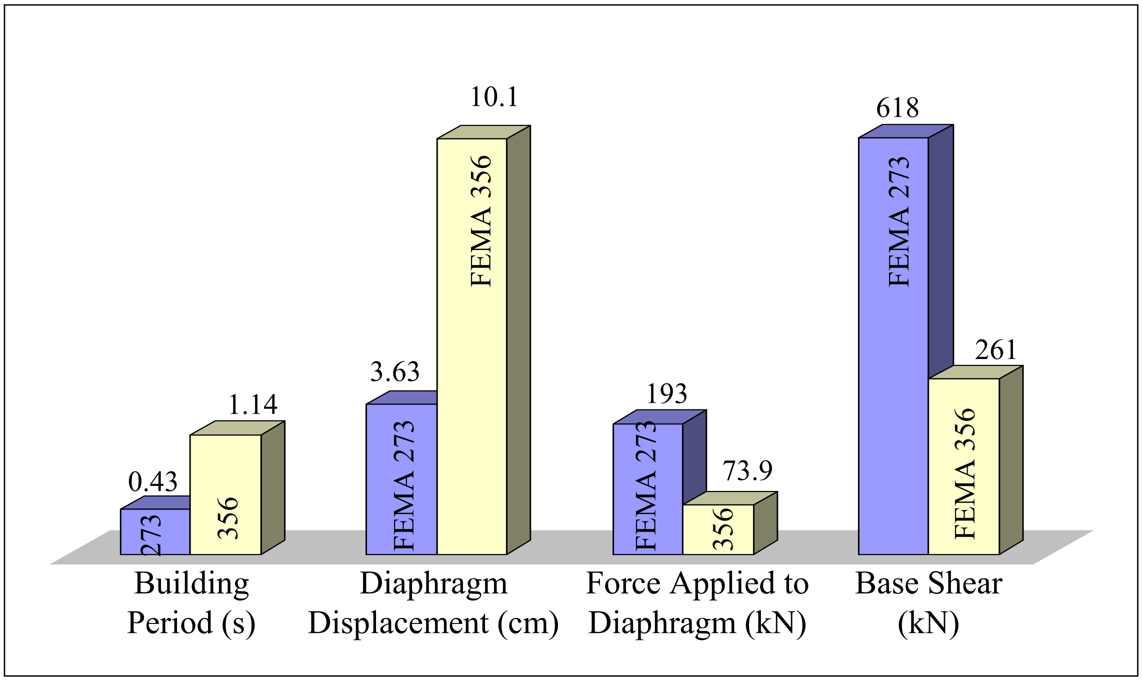


FIG 5.6 Comparison of FEMA 273 and FEMA 356 Predictions for Case Study Building 2

6. PARAMETRIC STUDY

6.1 GENERAL

The primary objective of the parametric study is to evaluate the seismic response of the URM building system to changing structural parameters. This is accomplished using a parametric study based on the conclusions of the completed rehabilitation analyses. Because the approach used in the first part of this study is focused on the diaphragm only, the parametric study evaluates the effect of rehabilitating the diaphragm on the behavior of the structural system as a whole. Rehabilitating the diaphragm typically involves increasing the in-plane diaphragm strength and stiffness and increasing the quality and number of the wall-to-diaphragm connections. Assessing the impact of the retrofit on the performance of the building system is necessary to ensure the retrofit has no adverse effects on other structural components.

The parameters in this portion of the study will utilize a prototype that represents both an existing and retrofitted typical URM building with material values chosen according to the recommendations in the FEMA 273 and FEMA 356 guidelines, rather than focusing on theoretical limitations. In general, two models are evaluated using the elastic dynamic analysis routine contained in SAP 2000 (CSI 1999): one model representing a typical URM building in its existing state and one model representing a typical URM building with a typical diaphragm retrofit based on the FEMA guidelines.

The selected retrofit used in this procedure corresponds to the results of the Linear Static Procedure described in Chapter 5. Although several types of retrofits were acceptable according to the FEMA guidelines, the selected retrofit for the parametric study is the chorded, blocked plywood overlay. This retrofit is the strongest and stiffest of the acceptable retrofits and is expected to have the most significant impact on the building response.

This parametric study permits an assessment of the building system's behavior, specifically observing the changing response of the building due to rehabilitating the diaphragm according to the FEMA 273 and FEMA 356 criteria. The goal of the parametric study is to observe the changing behavior of the system by increasing the strengths or stiffnesses of critical components. The behavior of the building will be observed as it is analytically subjected to the selected set of synthetic ground motion records developed by Wen and Wu (2000) for St. Louis, Missouri for a 10% probability of exceedance in 50 years seismic event with representative soil conditions. As discussed in Section 5.2.2, the FEMA 273 and FEMA 356 guidelines suggest that achieving the Basic Safety Objective for seismic rehabilitation in this region is based on the demands imposed by a seismic event having this probability of exceedance evaluated for a Life Safety Performance Level. Note that an additional requirement is that Collapse Prevention performance is ensured for the 2% in 50 years event. The scope of this study did not include an evaluation of this second performance objective.

6.2 DESCRIPTION OF PARAMETRIC BUILDING MODELS

In physical appearance, the existing and retrofitted prototype models are identical and differ only in the material properties. Both the existing and retrofitted models consist of a rectangular building with URM walls and wood floor and roof diaphragms, closely approximating a typical URM building found in the Central and Eastern portion of the United States. The walls and floors were developed using a three-dimensional finite element mesh containing 38.1 cm (15 in.) square shell elements with the representative material properties suggested by the FEMA 273 and FEMA 356 guidelines.

6.2.1 Existing Building Model Description

6.2.1.1 General

The existing building model contains characteristics typical of either case study building in its current state. This prototype is 26.3 m (86.3 ft.) long by 13.7 m (45 ft.)

wide, an aspect ratio of 1.9:1.0, and 9.14 m (30 ft.) tall. The first floor height is at 4.57 m (15 ft.) (see Fig. 6.1). The major components include the URM walls, wood diaphragms, and wall-to-diaphragm connections. The base has pinned conditions along all four walls to represent the known rocking behavior observed for URM walls in past earthquakes.

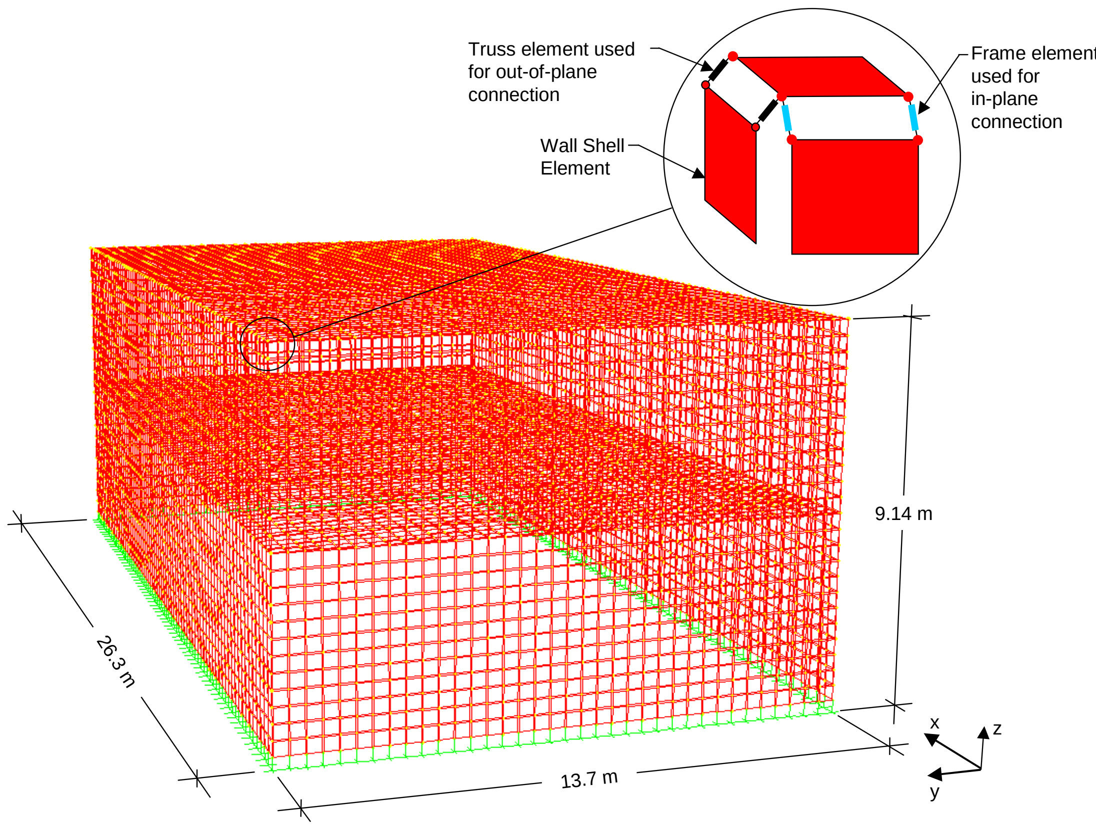


FIG. 6.1 SAP Model of URM Prototype Building

6.2.1.2 URM Walls

The walls are composed of unreinforced clay masonry, as is typical of materials and construction practices of the early twentieth century. The model utilizes a masonry weight of 7.97 x 10^{-4} kg/cm^3 (2.88 x 10^{-5} k/in.^{3}) for 30.5 cm (12 in.) thick, clay masonry (TMS 2001). FEMA 273 and FEMA 356 categorize the possible states for the condition of untested existing masonry into three conditions: good, fair and poor. The guidelines also list corresponding default values for the compressive strength according to the applicable existing state.

For the baseline existing building model, the condition of the masonry is assumed to be in "good" condition. Thus, the default compressive strength is 6,210 kPa (900 psi) with an elastic modulus of 34.1 x 10^5 kPa (4.95 x 10^5 psi) based on the relationship shown in Eq. 6.1 (ATC 1997a).

$$E = 550 * f \tag{6.1}$$

where:

E	=	Elastic modulus = 34.1 x 10^5 kPa (495 ksi) for "good" masonry
f	=	Compression strength for various masonry conditions
		good = 6,210 kPa (900 psi)
		fair = 4,140 kPa (600 psi)
		poor = 2,070 kPa (300 psi)

The existing building model was evaluated using all three different masonry conditions: good, fair, and poor. The effect of decreasing the elastic modulus can also represent various conditions of cracking. In reality, the wall has large openings that are not represented on the prototype and this reduction in modulus could be considered to better represent the actual strength of walls with openings.

6.2.1.3 Diaphragms

FEMA 273 and FEMA 356 provide a single value to represent the modulus of rigidity for single, straight sheathing, instead of providing a shear modulus. As discussed in Chapter 5, the modulus of rigidity, G_d, is the in-plane stiffness for diaphragms equal to the shear modulus of the diaphragm times the diaphragm thickness. The suggested value for the modulus of rigidity for single straight sheathing is 36,000 kg/cm (200 k/in.). This value, as provided by FEMA 273 and FEMA 356, should more accurately represent the shear stiffness of the wood sheathed flooring system rather than using a generic material value for wood.

The modulus of rigidity provided by FEMA 273 and FEMA 356 is utilized along with the thickness of the diaphragm to define the modulus of elasticity for the finite elements representing the diaphragm in the prototype existing building model. The diaphragm element is a 38.1 cm (15 in.) square shell that is 1.27 cm (0.5 in.) thick. The modulus of elasticity is then calculated according to Eq. 6.2 to be, E = 66.2 x 10^5 kPa (960 k/in.^{2}), and incorporated in the material parameters of the shell elements used to model the diaphragms in the existing building model.

$$E = \left(\frac{2G_d\left(1+\upsilon\right)}{t} \right) \tag{6.2}$$

where:

E = Modulus of elasticity of sheathing (kPa) (k/in.^{2})
G_d = Modulus of rigidity of sheathing (kPa) (k/in.^{2})
ν = Poisson's ratio for wood = 0.2
t = Thickness of finite element = 1.27 cm (0.5 in.)

The weight of the sheathing used in the model is an average value representing the total weight of the flooring system including the sheathing, joists and beams as calculated for the LSP. The total weight was applied uniformly over the area of the floor. The weight of the entire flooring system, composed of sheathing, joists and beams, was thereby taken into account rather than the weight of only the sheathing.

$$W_w = \frac{W_f}{\left(w_d * d_d * t_d\right)} \tag{6.3}$$

where:

W_w = Unit weight of sheathing as used for prototype (3.73 x 10^{-2} N/cm^3) (1.375 x 10^{-4} k/in.^{3})
W_f = Total weight of flooring system = 172 kN (38.7 k)
w_d = Actual width of diaphragm = 13.7 m (540 in.)
l_d = Actual length of diaphragm = 26.3 m (1035 in.)
t_d = Thickness of diaphragm used for prototype = 1.27 cm (0.5 in.)

6.2.1.4 Diaphragm-to-Wall Connections

The final component of concern is the out-of-plane diaphragm-to-wall connection. In existing buildings, this connection is provided by what is known as a “star” or “government” anchor or in some cases, no anchor at all. The star anchor, shown in Fig. 2.1, was replicated in the previous experimental research performed at Texas A&M University (Peralta et al. 2002). Again, the star anchor is a flexible connection, in terms of lateral load transfer, typical of the Central and Eastern portions of the United States during early twentieth century construction. In the model, this anchor is represented by a relatively weak axial spring connecting the diaphragm and out-of-plane wall at four points along the length of the diaphragm (described in Chapter 3). In both case study buildings, the diaphragm appears to be connected to the out-of-plane walls at the location of the beams spaced approximately at every 5.18 m (17 ft.). The star anchor does not represent the connection of the beam to the out-of-plane wall, only the connection of the diaphragm to the out-of-plane wall. The joists in the first case study building were supported by a pocket in the out-of-plane wall, which relies on the friction of the connection for the lateral force resistance.

The star anchor is represented in the model by a small truss element connecting the diaphragm to the out-of-plane wall at the relevant locations. Like the star anchor, the truss element transfers only axial load from one adjoining component to the other. The stiffness of the truss element in the model is defined by the modulus of elasticity (MOE). The MOE of these connections in the early twentieth century is not documented. However, FEMA 273 and FEMA 356 recommend a structural steel tensile strength of 55 ksi for the years 1909 to 1923 in the United States. While using a reduced MOE of steel to model the slip of the anchors was considered, no information was available to accurately represent this behavior. Thus, the typical MOE for steel of 2.00×10^8 kPa (2.90×10^4 ksi) was used for the star anchors in the finite element models.

In the prototype model, the finite element mesh that composes the URM walls and the diaphragm has a line of nodes along the edge of the diaphragm adjacent to the out-of-plane wall and identical line of nodes along the top of the out-of-plane wall. The nodes are in identical locations along the x-horizontal direction and are separated by a very small distance, 1.72 cm (0.5 in.), in the y-horizontal direction. The wall and diaphragm are connected at the aforementioned star anchor locations by a very short 0.5 in. diameter truss element. The truss element provides axial stiffness to the connection and has no horizontal rotational stiffness. Hence, the connection does not have the capability to transfer moment to the out-of plane walls from the diaphragm. The same is true for the diaphragm-to-wall connections at the first floor level.

The diaphragm-to-wall connections along the in-plane walls are similar to that of the out-of-plane wall connection. The girders connecting the diaphragm to the in-plane wall sit in a pocket in the URM wall, giving it the capacity to transfer shear as well as axial loads. Because of this, the modeled connection of the diaphragm to the in-plane wall utilizes a 0.5 in. diameter frame element that transferred shear and axial loads, but without the capacity to transfer moment. The nodes along the edge of the wall and along the edge of the diaphragm are in identical locations along the y-horizontal direction but are separated by a small distance, 1.72 cm (0.5 in.) in the x-horizontal direction. However, the corresponding nodes along the diaphragm and wall are connected along the full length with a frame element at each node. Like the out-of-plane wall connection, the strength of the connection was unknown. Thus, the normal MOE of steel 2.00 x 10^8 kPa (2.90 x 10^4 ksi) was used

6.2.2 Retrofitted Building Model Description

6.2.2.1 General

The retrofit model of the building is generally the same form as the existing building model, with increased values for the modulus of the diaphragm. The three-dimensional finite element mesh of the retrofitted building model is identical to that shown in Fig. 6.1.

6.2.2.2 URM Walls

As in the existing building model, the retrofitted building model was modeled using all three existing masonry default conditions: good, fair, and poor. Again, the decreased MOE values for the wall elements can represent a "cracked" condition or the presence of openings.

6.2.2.3 Diaphragm

In the prototype model containing a wood floor and roof diaphragm, both diaphragms were modified to represent retrofitted conditions in this portion of the analysis. The retrofitted diaphragm is no longer simply single, straight sheathing. According to the results of the linear static procedure, an acceptable retrofit is a blocked, chorded plywood overlay. The corresponding recommended value for the modulus of rigidity is 3,150 kN/cm (1,800 k/in.); therefore the corresponding modulus of elasticity is 7,560 kN/cm (4,320 k/in.) from Eq. 6.2. The finite element representing this retrofitted diaphragm was increased to 2.54 cm (1 in.) thick because the effect of overlaying the plywood also increases the thickness of the floor system. The weight per unit volume was modified to account for the new thickness of 2.54 cm (1 in.) and the unit weight was increased to 2.14 x 10^{-2} N/cm^3 (7.87 x 10^{-5} k/in.^{3}).

6.2.2.4 Diaphragm-to-Wall Connections

In current practice, it is common to improve the connection between the floor and roof diaphragms to the out-of-plane walls. The strength of the connection is designed so that the retrofitted connection is no longer the weakest component of the lateral system. After design, the strength of the connection would exceed the flexural capacity of the out-of-plane wall and the shear strength of the diaphragm. Additionally, the connection improvements would consist of connecting the diaphragm to the out-of-plane wall at closer spacings.

Because the connection in the unretrofitted model utilized the typical steel MOE in current practice, the retrofitted connection was modeled in the same way. The MOE used to represent the steel of the retrofitted connection is also 2.00 x 10^8 kPa (2.90 x 10^4 ksi). Therefore, in the retrofit prototype, the retrofitted connection is only different from the unretrofitted connection in the reduced spacing between connections.

The retrofitted connection was modeled as a truss element located at every third joist along the out-of-plane wall, typically about every 0.91 m (3 ft.) in the model. For flexible diaphragms, FEMA 273 and FEMA 356 require that the walls should be anchored to the diaphragms at least every 2.44 m (8 ft.), and continuously connected with diaphragm crossties. The frequency of connecting every third joist is taken from common retrofit practices for these types of structures. Both FEMA guidelines consider these anchors force-controlled elements.

6.2.3 Summary of Parametric Models

Table 6.1 summarizes each case evaluated in the parametric study for both the existing and retrofitted building.

TABLE 6.1 Summary of Material Properties for Parametric Study

Masonry Condition	**Diaphragm and Connection Condition**	**Material Type**	**Unit Weight** **N/cm^3** **(k/in.^{3})**	**Elastic Modulus** **kPa** **(k/in.^{2})**
Good	Single Straight Sheathing Existing Anchors	Masonry	$7.82\text{x}10^{-3}$ ($2.88\text{x}10^{-5}$)	$3.41\text{x}10^{6}$ (495)
		Wood	$3.73\text{x}10^{-2}$ ($1.38\text{x}10^{-4}$)	$3.31\text{x}10^{6}$ (480)
		Spring	N/A	$2.00\text{x}10^{8}$ ($2.9\text{x}10^{4}$)
	Chorded Blocked Plywood Overlay Retrofit Anchors	Masonry	$7.82\text{x}10^{-3}$ ($2.88\text{x}10^{-5}$)	$3.41\text{x}10^{6}$ (495)
		Wood	$2.14\text{x}10^{-2}$ ($7.87\text{x}10^{-5}$)	$2.98\text{x}10^{7}$ (4320)
		Spring	N/A	$2.00\text{x}10^{8}$ ($2.9\text{x}10^{4}$)
Fair	Single Straight Sheathing Existing Anchors	Masonry	$7.82\text{x}10^{-3}$ ($2.88\text{x}10^{-5}$)	$2.28\text{x}10^{6}$ (330)
		Wood	$3.73\text{x}10^{-2}$ ($1.38\text{x}10^{-4}$)	$3.31\text{x}10^{6}$ (480)
		Spring	N/A	$2.00\text{x}10^{8}$ ($2.9\text{x}10^{4}$)
	Chorded Blocked Plywood Overlay Retrofit Anchors	Masonry	$7.82\text{x}10^{-3}$ ($2.88\text{x}10^{-5}$)	$2.28\text{x}10^{6}$ (330)
		Wood	$2.14\text{x}10^{-2}$ ($7.87\text{x}10^{-5}$)	$2.98\text{x}10^{7}$ (4320)
		Spring	N/A	$2.00\text{x}10^{8}$ ($2.9\text{x}10^{4}$)
Poor	Single Straight Sheathing Existing Anchors	Masonry	$7.82\text{x}10^{-3}$ ($2.88\text{x}10^{-5}$)	$1.14\text{x}10^{6}$ (165)
		Wood	$3.73\text{x}10^{-2}$ ($1.38\text{x}10^{-4}$)	$3.31\text{x}10^{6}$ (480)
		Spring	N//A	$2.00\text{x}10^{8}$ ($2.9\text{x}10^{4}$)
	Chorded Blocked Plywood Overlay Retrofit Anchors	Masonry	$7.82\text{x}10^{-3}$ ($2.88\text{x}10^{-5}$)	$1.14\text{x}10^{6}$ (165)
		Wood	$2.14\text{x}10^{-2}$ ($7.87\text{x}10^{-5}$)	$2.98\text{x}10^{7}$ (4320)
		Spring	N/A	$2.00\text{x}10^{8}$ ($2.9\text{x}10^{4}$)

N/A = Not Applicable

6.3 TIME HISTORY ANALYSIS

6.3.1 General

As stated earlier, the ground motions used in the analysis are taken from a series of synthetic ground motions developed for St. Louis, Missouri both for regional soil and rock conditions (Wen and Wu 2000). This study utilizes the set of ten ground motions synthesized for the regional soil conditions for a 10% probability of exceedance in 50 years. FEMA 273 and FEMA 356 recommend that when using more than seven sets of ground motions, that the average of each response parameter should be used in evaluating structural performance. These ground motions were developed with the intent that the median value of the response parameter of interest would provide the best reflection of the actual response of the system for the given event. In this case, the median response is determined by averaging the natural logarithm of the maximum response parameters from each ground motion record and then determining the median using an exponential function (see Eq. 6.5).

$$median = e^{\frac{1}{10}\sum_{i=1}^{n}\ln(x)} \tag{6.5}$$

where:

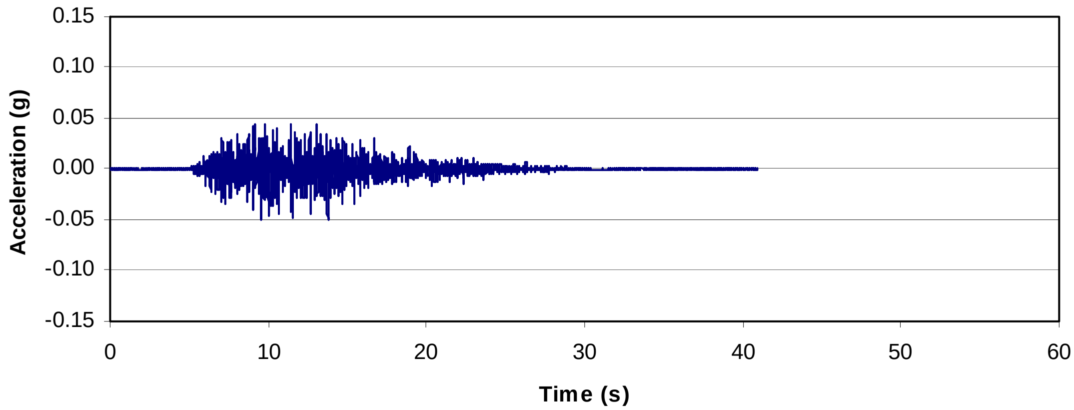
i = Ground motion record number.
x = Response parameter of interest (typically a maximum value from a time history analysis).
n = Total number of ground motion records = 10.

Table 6.2 shows a summary of the ground motion records used in this analysis, listing the major characteristics of each: duration, peak ground acceleration (PGA), magnitude, focal depth, distance from epicenter, and the deviation from median attenuation, ε. Fig. 6.2 depicts the ground motion for each time history record by showing the graphs of acceleration versus time for each record. The complete time history records were used in the analysis, however only the first sixty seconds of the record is shown in Fig. 6.2. The acceleration values beyond this time point are negligible. The accelerations are provided as a fraction of the acceleration of gravity (g).

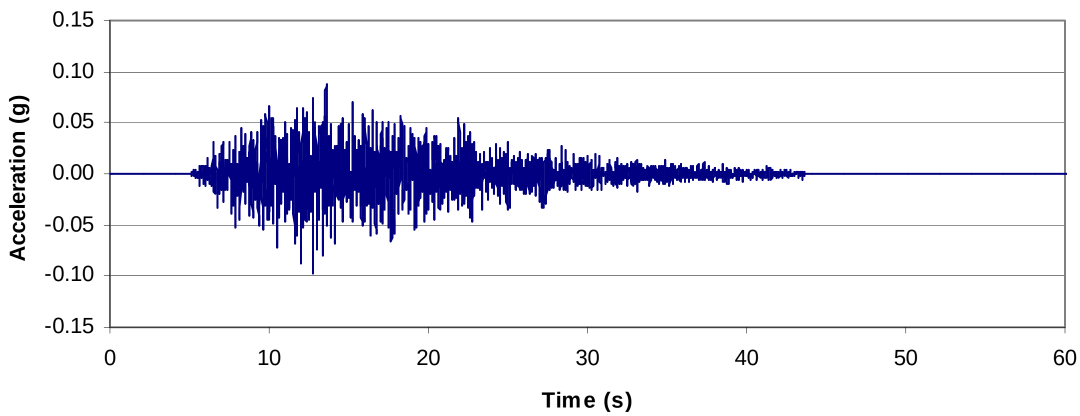
Both the record number referenced in this study and the label used by Wen and Wu (2000) are provided for each record.

Table 6.2 Time History Parameters, 10% in 50 years event for St. Louis, Missouri (Adapted from Wen and Wu 2000)

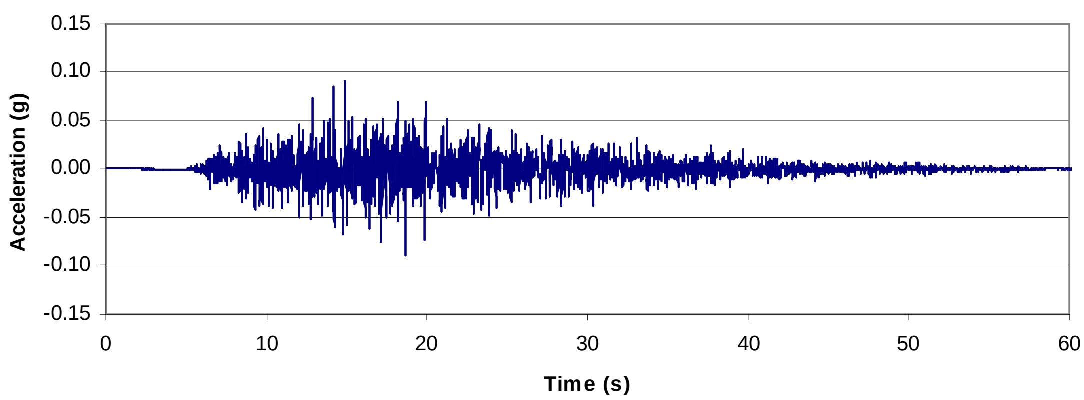
Record No.	Duration (s)	PGA cm/s^2 ($in./s^2$)	Moment Magnitude	Focal Depth km (mi.)	Epicentral Distance km (mi.)	ε
1	40.95	43.1 (17.0)	6.0	2.7 (1.68)	76.4 (47.5)	0.90
2	81.91	86.9 (34.2)	6.8	9.3 (5.78)	201.5 (125.2)	0.44
3	81.91	89.3 (34.1)	7.2	4.4 (2.73)	237.5 (147.5)	0.07
4	40.95	85.2 (33.6)	6.3	9.8 (6.09)	252.1 (156.6)	1.71
5	40.95	127 (49.9)	5.5	2.9 (1.80)	123.1 (76.5)	1.81
6	40.95	101 (39.6)	6.2	7.7 (4.78)	207.6 (128.9)	1.68
7	81.91	89.5 (35.3)	6.9	1.7 (1.06)	193.7 (120.3)	0.35
8	40.95	116 (45.6)	6.2	27.6 (17.14)	174.5 (108.4)	1.40
9	40.95	92.8 (36.5)	6.2	6.5 (4.04)	221.3 (137.5)	1.72
10	81.91	75.1 (29.6)	6.9	2.7 (1.68)	237.2 (147.3)	0.81



(a) Time History Record 1 (l10_01s)

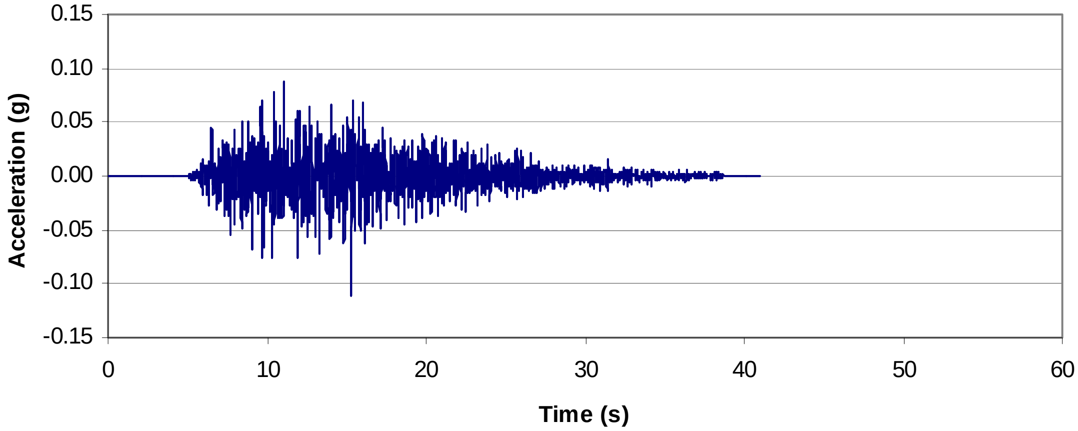


(b) Time History Record 2 (l10_02s)

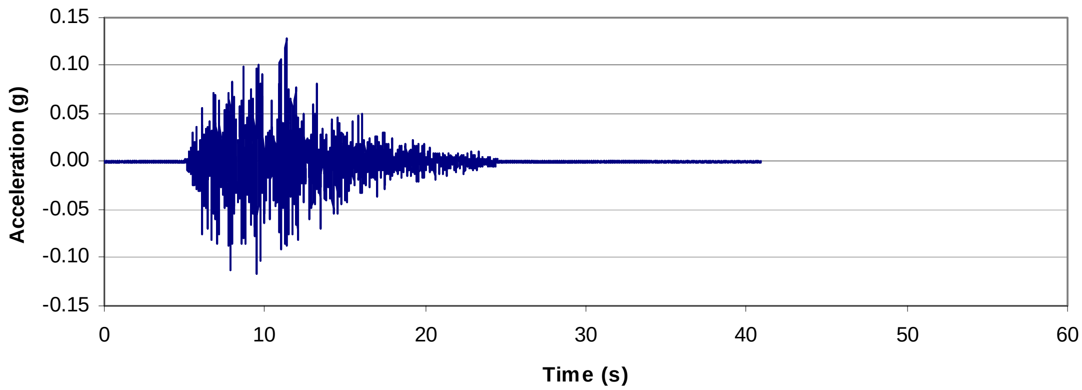


(c) Time History Record 3 (l10_03s)

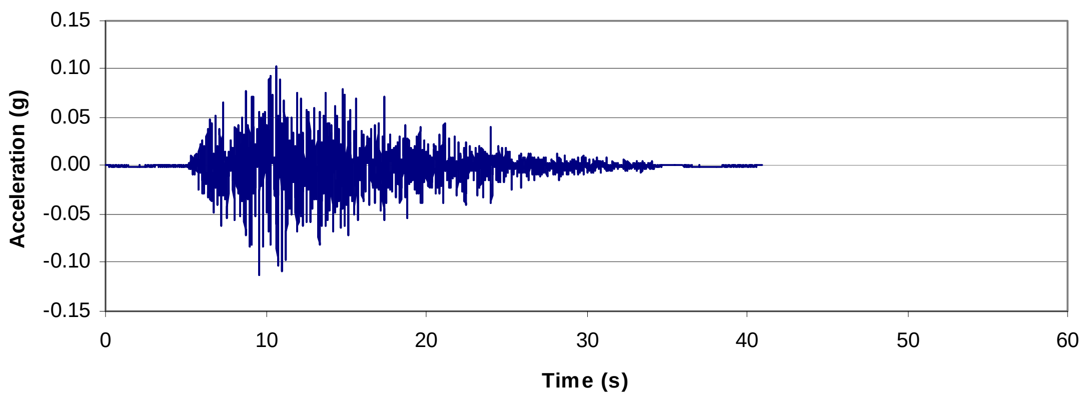
FIG 6.2 Time History Records Used for Parametric Study



(d) Time History Record 4 (l10_04s)

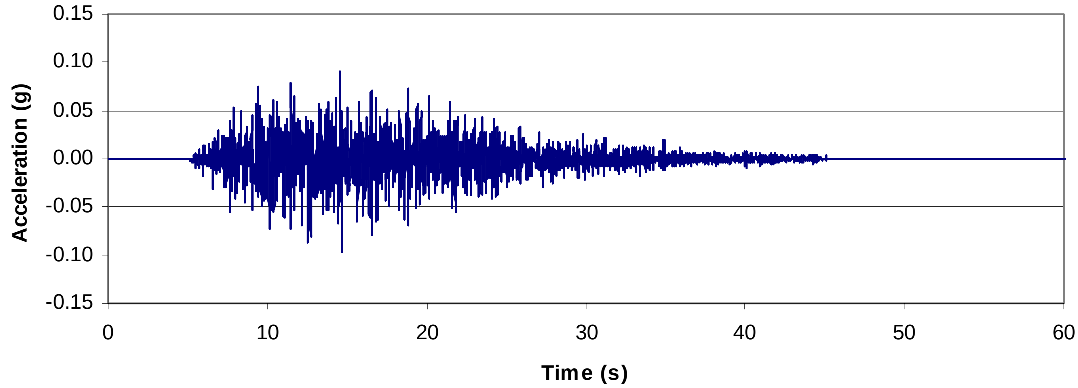


(e) Time History Record 5 (l10_05s)

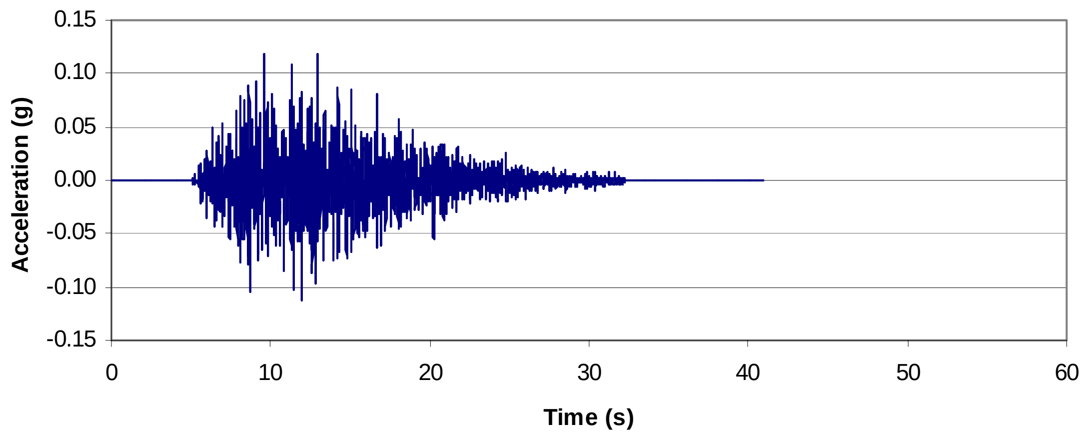


(f) Time History Record 6 (l10_06s)

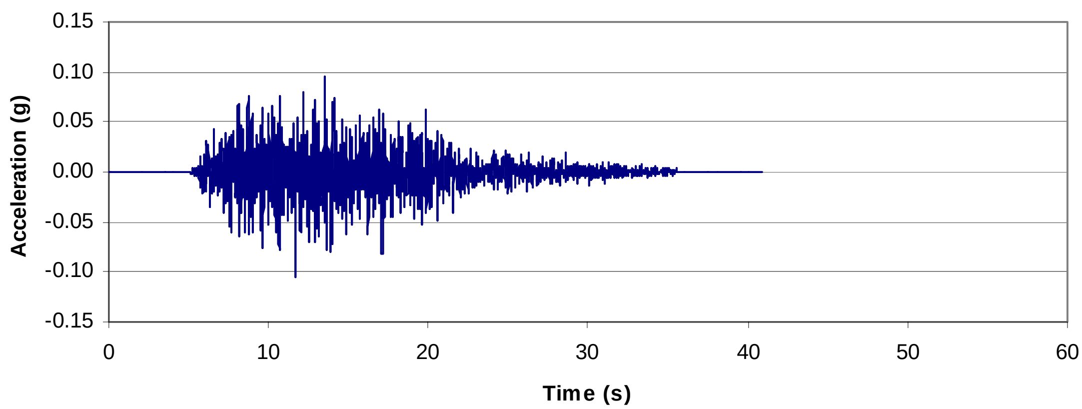
FIG. 6.2 Time History Records Used for Parametric Study (cont.)



(g) Time History Record 7 (l10_07s)

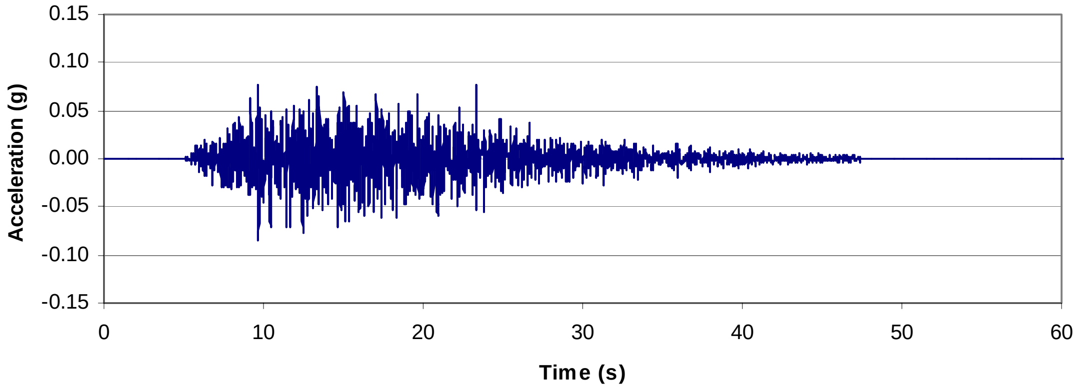


(h) Time History Record 8 (l10_08s)



(i) Time History Record 9 (l10_09s)

FIG. 6.2 Time History Records Used for Parametric Study (cont.)



(j) Time History Record 10 (l10_10s)

FIG 6.2 Time History Records Used for Parametric Study (cont.)

6.3.2 Discussion of Method to Report Results

For the purpose of clearly describing the results of the parametric study, a drawing of the prototype building, typical of any of the masonry conditions evaluated, is provided in Fig. 6.3. The locations where parameters are reported are shown with corresponding letter designations. Any parameters reported for the lower diaphragm are designated with the same letter shown in Fig. 6.3 for the roof diaphragm, but with the subscript "L". The first floor level is indicated by the dashed line on Fig. 6.3. Table 6.3 provides a description of each location shown in Fig. 6.3.

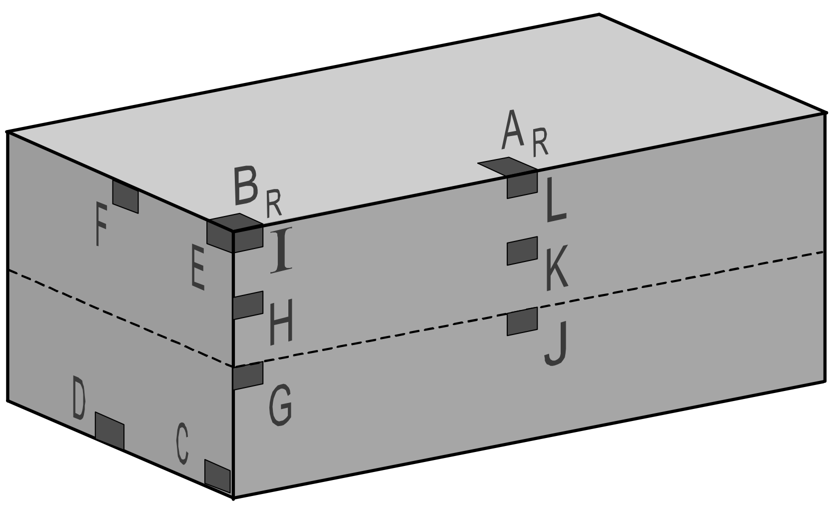


FIG 6.3 Demonstration of Reported Locations for Prototype

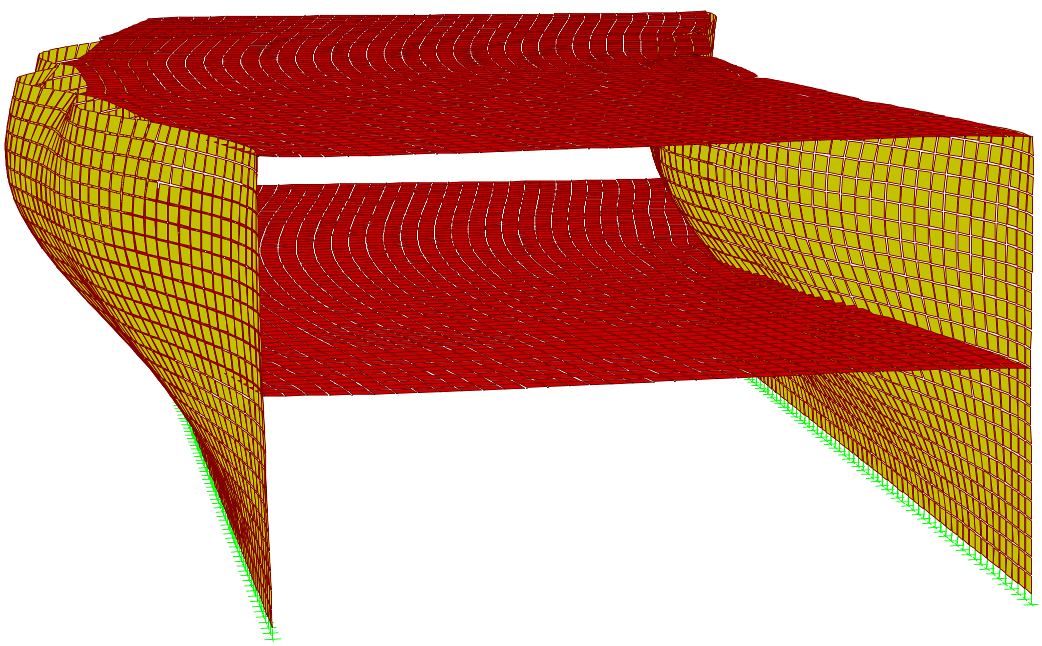
Table 6.3 Description of Reported Locations Shown in Fig. 6.3

Letter Designation	Description
A_R	Outside edge of roof diaphragm at the midspan
B_R	Corner of roof diaphragm adjacent to the in-plane and out-of-plane walls
A_L	Same location as A_R but on the first floor diaphragm
B_L	Same location as B_R but on the first floor diaphragm
C	Bottom corner of in-plane wall adjacent to the out-of-plane wall
D	Center of in-plane wall at the base
E	Upper corner of in-plane wall adjacent to the out-of-plane wall at roof level
F	Center of top edge of in-plane wall adjacent to the roof diaphragm
G	Outside edge of out-of-plane wall adjacent to the in-plane wall at floor level 1
H	Outside edge of out-of-plane wall adjacent to the in-plane wall at mid-height between floor level 1 and roof level
I	Upper corner of out-of-plane wall adjacent to the in-plane wall and the roof diaphragm
J	Center of the out-of-plane wall at floor level 1
K	Center of the out-of-plane wall at mid-height between floor level 1 and roof level
L	Center of out-of-plane wall adjacent to the roof diaphragm

6.3.3 Discussion of Fundamental Mode Shapes

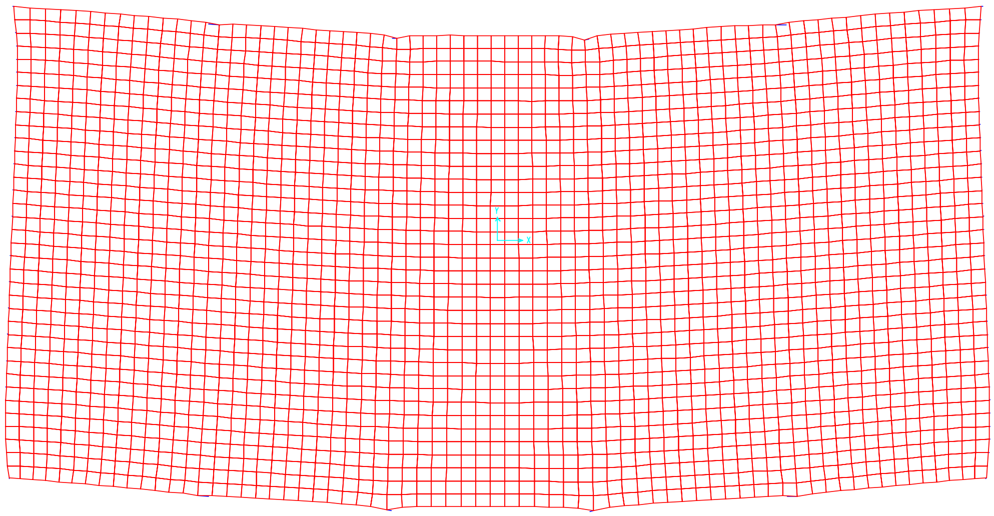
The building periods are presented in Section 6.3.4. Figs. 6.4 and 6.5 show the fundamental mode shape for the unretrofitted and retrofitted buildings, respectively. The deformations are exaggerated in the figures so that the mode shapes can be visualized. Therefore, the figures provide only the relative displacement for the fundamental mode shape.

Fig. 6.4(a) shows the fundamental mode shape for the three-dimensional unretrofitted prototype building model. These graphics show that in the unretrofitted building, the out-of-plane walls pull away from the diaphragm between the connection locations. Because of this, there is visible displacement between corresponding nodes on the out-of-plane wall and the diaphragm. Fig. 6.4b shows the diaphragm flexing and the out-of-plane walls pulling away from the diaphragm between anchor locations. The small nodes that appear unattached are actually the top nodes of the out-of-plane walls (see Fig. 6.4b). The roof diaphragm flexes with the greatest displacement at midspan. However, the out-of-plane walls displace beyond the diaphragm in the first mode. The deformation of the out-of-plane walls, shown in Fig. 6.4c, suggests that the walls endure substantial activity between floor levels and is representative of all evaluated masonry conditions. The out-of-plane wall deflects similar to a cantilevered beam above the first floor level. The following paragraphs discuss the computed displacements and stresses in the walls and diaphragms.

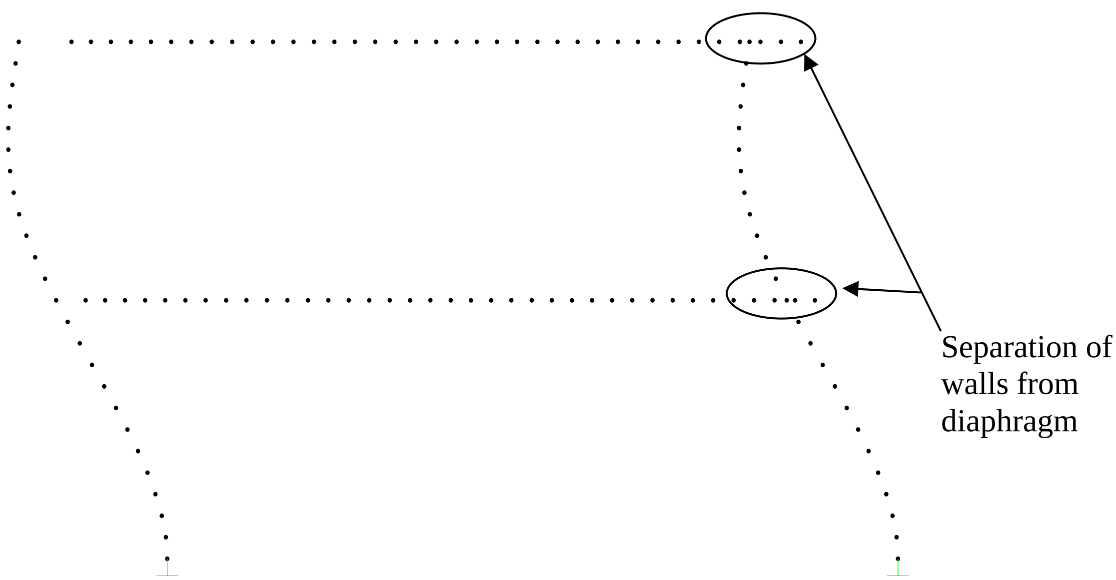


(a) 3-D Unretrofitted Prototype Fundamental Mode Shape

FIG. 6.4 Unretrofitted Prototype Fundamental Mode Shape



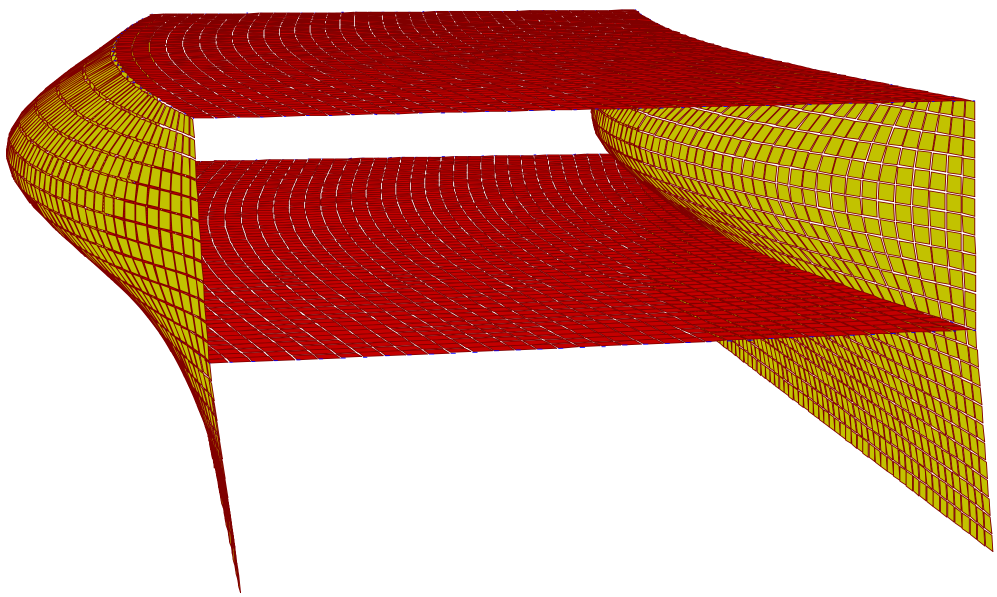
(b) Unretrofitted Roof Diaphragm – First Mode Shape (Plan View)



(c) Out-of-Plane Wall - First Mode Shape (Elevation View)

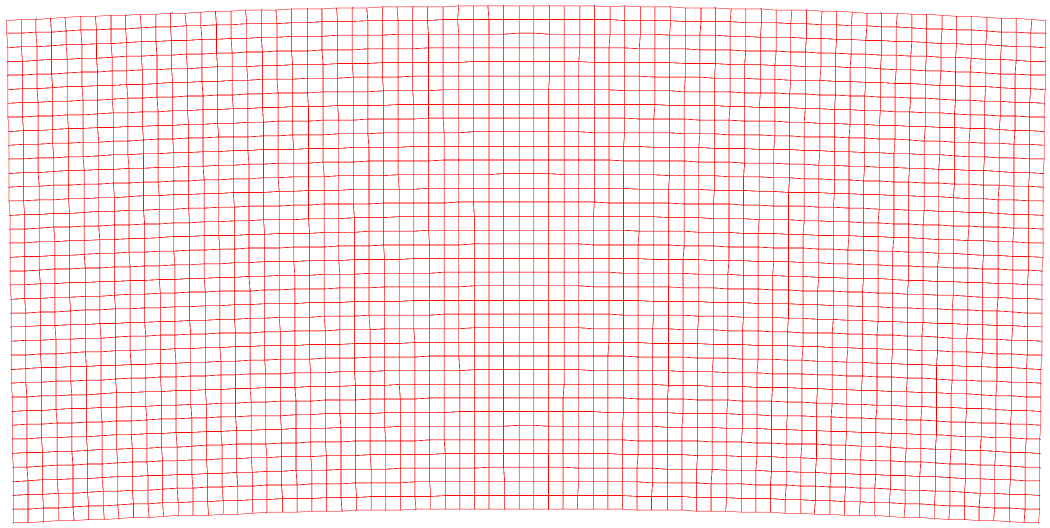
FIG. 6.4 Unretrofitted Prototype Model Fundamental Mode Shape (cont.)

Fig. 6.5a shows the overall fundamental mode shape of the retrofitted building model typical for any of the masonry conditions evaluated. As shown in the plan view of the out-of-plane wall-to-diaphragm connection, the retrofitted prototype shows the tendency of the out-of-plane walls to deform with the diaphragm because they are connected more closely along the entire length (see Fig 6.5b). Thus, as the connection spacing is reduced, the walls and the diaphragm move together more uniformly. In the retrofitted building, the mode shape of the out-of-plane walls shows significant bending between floors levels. Contrary to the cantilevered behavior of the out-of-plane wall in the unretrofitted mode shape, the retrofitted connection causes the out-of-plane wall to deform similar to a simply supported beam. For the retrofitted prototype, the out-of-plane walls deformation suggests the walls are more restrained at each diaphragm level (see Fig. 6.5c).



(a) 3-D Retrofitted Prototype Fundamental Mode Shape

FIG. 6.5 Retrofitted Prototype Fundamental Mode Shape



(b) Retrofitted Roof Diaphragm – First Mode Shape (Plan View)

(a) Out-of-Plane Wall - First Mode Shape (Elevation View)

FIG. 6.5 Retrofitted Prototype Model Fundamental Mode Shape (cont.)

6.3.4 Comparison of Major Building Response Parameters

6.3.4.1 General

Table 6.4 provides a summary of the major building parameters, and this information is shown visually in Fig. 6.6. As expected, the change in the building period between the unretrofitted and retrofitted case has the same trend for each wall condition. The addition of a stiffer diaphragm and closer spacing of retrofitted connections decreases the fundamental period of the building. The period of the retrofitted building ranges from 55% to 63% of the fundamental period for the corresponding unretrofitted structure. As the condition of the masonry is degraded from good to fair to poor, the period increases due to the decrease in the URM wall stiffness.

Table 6.4 Summary of Building Response Parameters (Median Maximum Values)

Parameter	Prototype Condition					
	Good Masonry		Fair Masonry		Poor Masonry	
	UR	R	UR	R	UR	R
Building Period (s)	0.557	0.305	0.603	0.342	0.706	0.447
Base Shear, kN (kips)	668 (150)	1,474 (331)	570 (51)	1,230 (276)	475 (107)	769 (173)
Max Building Drift, (%)	0.0606	0.0308	0.0567	0.0326	0.0594	0.0331
Diaphragm Disp. (at F), cm (in.)	0.503 (0.198)	0.229 (0.090)	0.426 (0.168)	0.224 (0.088)	0.386 (0.152)	0.232 (0.091)
Diaphragm Accel. (at A_R) (g)	0.147	0.151	0.124	0.139	0.0979	0.0961
In-Plane Wall Accel. (at F), (g)	0.0102	0.0522	0.0128	0.0616	0.0192	0.0559

UR = Unretrofitted, R = Retrofitted
See Fig. 6.3 for key to reported locations.

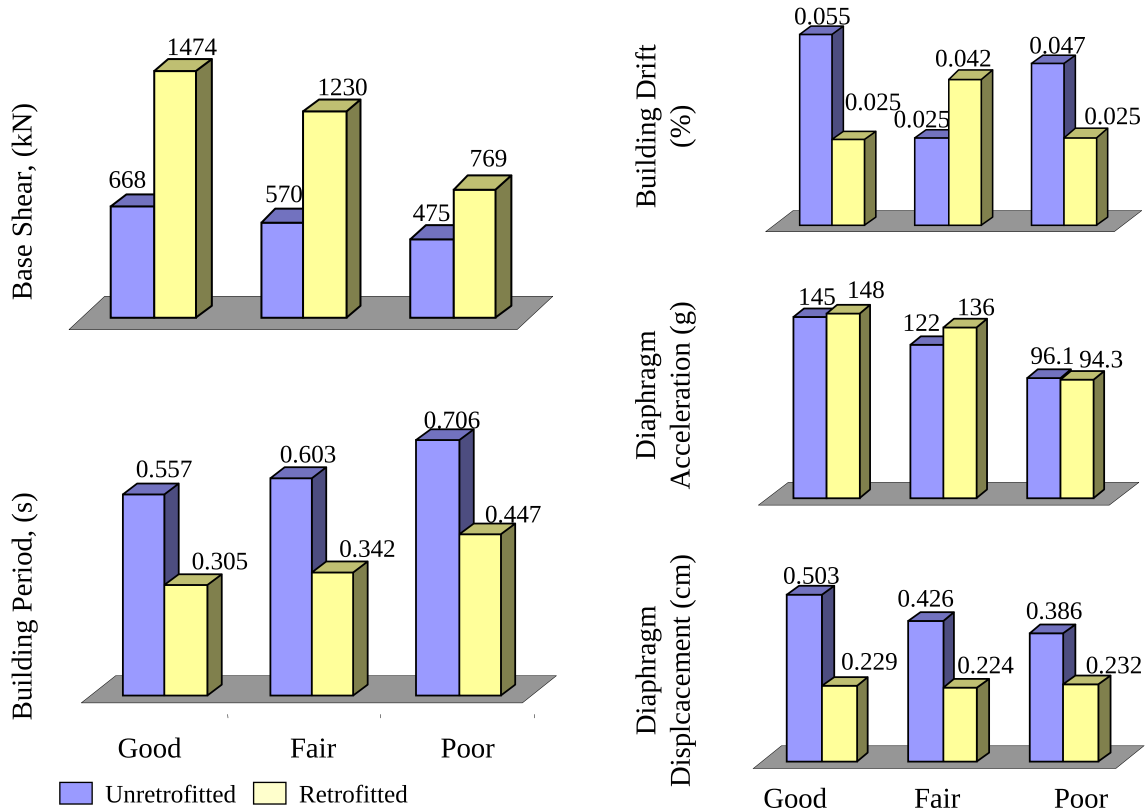


FIG. 6.6 Summary of Building Response Parameters

6.3.4.2 Fundamental Period

As the building is retrofitted and stiffened, the fundamental period of the building significantly decreases and the base shear in the direction of loading increases substantially (see Table 6.4). The base shear for the retrofitted building varies from 1.6 to 2.2 times the base shear for the corresponding unretrofitted structure. The dead weight of the structure, *W*, is 3,300 kN (742 kips). The base shear values range from 15% to 20% of *W* for the unretrofitted case and 23% to 45% of *W* for the retrofitted case. The increase in the base shear is more considerable in magnitude than intuitively expected for a retrofit only involving the diaphragm and diaphragm-to-wall connections. However, the base shear increase is warranted because the base shear is dependent upon the period of the building, which for the retrofitted cases, was reduced to almost 50% of the corresponding unretrofitted value.

6.3.4.3 Building Drift

The maximum drift, found as the ratio of the roof diaphragm midspan displacement to the building height, is the largest for the unretrofitted prototype structure with a “good” masonry condition. The drift tends to decrease with decreasing masonry conditions and with the addition of the diaphragm retrofit as demonstrated in Table 6.4. However, the drift is reduced and approximately equal in the retrofitted models, regardless of the masonry condition. The buildings with reduced masonry properties are less stiff and so they also attract less force. This helps to explain the reduced displacements for the “fair” and “poor” retrofitted masonry conditions.

6.3.4.4 Roof Diaphragm Displacement

As the building diaphragms are stiffened, the midspan deflection of the roof diaphragm of the building decreases. The diaphragm displacement reported in Table 6.4 is the maximum roof diaphragm displacement. It is again interesting to note that the deflections in the retrofitted (R) condition are approximately equal regardless of the condition of the masonry. However, all of the deflections of the diaphragm are relatively small. Again, this response is expected, and reiterates the success of the plywood overlay in effectively stiffening the floor and minimizing additional displacements imposed on the out-of-plane URM walls.

6.3.4.5 Roof Diaphragm Acceleration

The acceleration in the diaphragm increases slightly with the added retrofit (see Table 6.4), except in the case of the “poor” condition of masonry. As the masonry condition deteriorates, the acceleration imposed on the diaphragm tends to decrease. The impact on the diaphragm acceleration due to retrofit is most pronounced for the “fair” masonry condition, increasing by approximately 12% compared to the unretrofitted condition. The “good” masonry has twice the diaphragm acceleration of the “poor” masonry prototype for the unretrofitted condition.

6.3.4.6 In-Plane Wall Acceleration

The acceleration in the in-plane walls increases considerably with the stiffening of the diaphragm (see Table 6.4). This is most notable for the “good” masonry condition, where the in-plane wall acceleration in the retrofitted case is five times that of the unretrofitted case. However, all three masonry conditions show substantial increases in the wall acceleration with diaphragm retrofit. This response concurs with the conclusions of a study focused on in-plane URM walls that took place at University of Illinois at Urbana-Champaign and described in Section 2.3 (Simsir et al. 2002).

6.3.4.7 Comparison of Median and Average Response

As described in Section 6.3.1, the maximum values of base shear, roof diaphragm displacement, diaphragm acceleration, and in-plane wall acceleration reported in Table 6.4 are the median values associated with the ten synthetic ground motion records for the 10% in 50-year event. Table 6.5 shows the average, median and absolute maximum base shear values from the time histories analyses. The median base shear is the recommended value for use in structural performance assessments by FEMA 273 and FEMA 356. In the case of these ground motions, the trend is that the average values are consistently higher than the median value. The absolute maximum is approximately 10% to 13% higher than the median base shear regardless of the retrofit or masonry condition.

Table 6.5 Comparison of Average, Median, and Absolute Maximum Base Shear Values

Parameter	Prototype Condition					
	Good Masonry		Fair Masonry		Poor Masonry	
	UR	R	UR	R	UR	R
Average Base Shear, kN (kips)	671 (151)	1,490 (336)	575 (129)	1,250 (280)	480 (108)	777 (174)
Median Base Shear, kN (kips)	668 (150)	1,470 (331)	570 (128)	1,230 (276)	475 (107)	769 (173)
Absolute Max. Base Shear, kN (kips)	750 (169)	1,810 (407)	694 (156)	1,590 (356)	669 (150)	1,010 (226)

UR = Unretrofitted, R = Retrofitted

6.3.5 Deformation Response of URM Prototype

Table 6.6 describes the potentially critical areas of deformation in the building. The values shown in the table are the maximum deformations at each area, reported as the median value resulting from the maximum response for each of the ten time history analyses. The stresses shown in the table are the tensile stresses due to out-of-plane bending. They are compared with the allowable tensile stresses according to FEMA 273 and FEMA 356. These deformations are summarized in Fig. 6.7.

Table 6.6 Maximum Deformations of URM Prototype (Median Values)

Parameter		Prototype Condition					
		Good Masonry		Fair Masonry		Poor Masonry	
		UR	R	UR	R	UR	R
Roof Level	Diaphragm (at A_R), cm (in.)	0.503 (0.198)	0.229 (0.090)	0.426 (0.168)	0.224 (0.088)	0.386 (0.152)	0.232 (0.091)
	Out-of-Plane Wall (at L), cm (in.)	0.780 (0.307)	0.239 (0.094)	0.721 (0.284)	0.240 (0.095)	0.804 (0.317)	0.237 (0.093)
	In-Plane Wall (at E), cm (in.)	0.0367 (0.0144)	0.0801 (0.0315)	0.0463 (0.0182)	0.1034 (0.0407)	0.0782 (0.0308)	0.133 (0.0524)
Mid-height	Out-of-Plane Wall (at K), cm (in.)	0.824 (0.324)	0.530 (0.209)	0.825 (0.325)	0.598 (0.236)	0.970 (0.382)	0.670 (0.264)
Floor Level	Diaphragm (at A_L), cm (in.)	0.433 (0.170)	0.272 (0.107)	0.420 (0.165)	0.246 (0.097)	0.434 (0.171)	0.168 (0.066)
	Out-of-Plane Wall (at J), cm (in.)	0.582 (0.229)	0.284 (0.112)	0.579 (0.228)	0.254 (0.100)	0.668 (0.263)	0.217 (0.085)

UR = Unretrofitted, R = Retrofitted
See Fig. 6.3 for key to reported locations.

As the building deforms, in-plane walls move very little. The diaphragms deform similar to a beam in bending transferring the lateral forces into the out-of-plane walls. The out-of-plane walls attempt to absorb the force in out-of-plane bending, of which there is little capacity, and this causes significant stress and displacement in the walls. However, as the building is retrofitted with improved connections and stiffened diaphragms, the uniformity of the structure's deformation improves significantly. The out-of-plane masonry walls pulled away from the diaphragm in the unretrofitted cases. In the retrofitted cases, the out-of-plane walls were more restrained and displaced one-third of the original displacement in the unretrofitted cases at both floor levels.

However, the most displacement in either case occurred at the mid-height of the wall between the first floor and the roof level. This is the case for all the conditions of masonry that were evaluated.

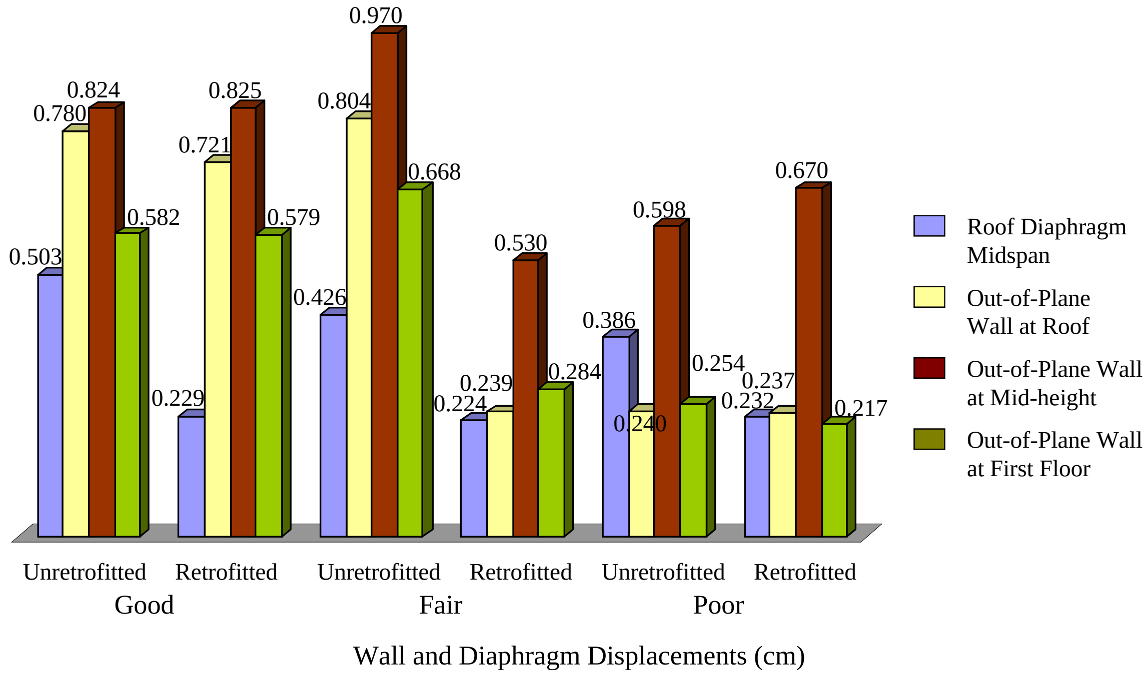


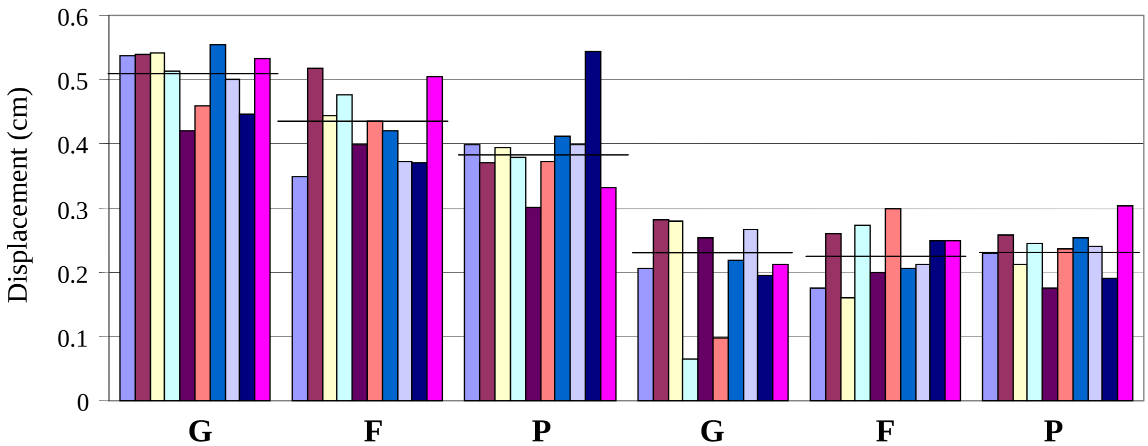
FIG. 6.7 Summary of Maximum Deformation in URM Prototype

As mentioned earlier, the top of the out-of-plane wall displaces more than the midspan of the diaphragm at the roof level. In the unretrofitted prototype, the displacement of the out-of-plane wall exceeds the midspan displacement of the corresponding roof or first floor diaphragm by a factor ranging from 1.5 to 2.0. However, in the retrofitted prototypes, the displacement between the out-of-plane wall and the diaphragm midspan are significantly reduced and almost equal at the roof and first floor level.

In the unretrofitted case, the displacement of out-of-plane wall at the roof level is greater than at the floor level. In the retrofitted condition, the opposite is true and the larger displacement is at the first floor level. However, the addition of the diaphragm retrofit causes the displacement of the out-of-plane wall at the roof and floor level to become approximately equal. With more uniform movement of these two components, the likelihood of the walls to repeatedly pound against the diaphragm during an earthquake is lessened.

The largest deformation in both the unretrofitted and retrofitted prototype structures, for all masonry conditions, occurs at the mid-height of the out-of-plane wall between the first floor level and the roof level (position K). Refer to Fig. 6.3 for locations and Figs. 6.4a and 6.5a for a comparison of the unretrofitted and retrofitted mode shapes, respectively. The displacement at the mid-height for the retrofitted conditions is approximately the same for the three masonry conditions. This is true again for the unretrofitted conditions, increasing slightly as the masonry conditions deteriorate. The deformation of the mid-height of the out-of-plane wall is about twice that of the out-of-plane wall deformation at the first floor level (position J) in the unretrofitted buildings and three times the deformation in the retrofitted buildings. In the unretrofitted prototype, the first floor diaphragm deforms almost twice as much as the roof diaphragm for all masonry conditions. However, when comparing the displacement of the wall at mid-height with the roof diaphragm displacement, the deformation of the out-of-plane wall at mid-height is only slightly larger than the roof in the unretrofitted case, but more than twice that of the roof in the retrofitted case.

Fig. 6.8 demonstrates the relative behavior of the maximum midspan displacement of the diaphragm for each ground motion record, by building type and masonry condition. The response values are shown in numerical order based on the ground motion name. The median value for each building type is shown by a horizontal line. The median value decreases significantly as the masonry condition deteriorates for the unretrofitted cases. In the retrofitted cases, the median maximum displacement is essentially the same for all three conditions of masonry.



(a) Unretrofitted Prototype **(b) Retrofitted Prototype**

FIG. 6.8 Maximum Diaphragm Displacement for All Time History Records

6.3.6 Stresses Developed in URM Prototype

6.3.6.1 General

The stresses discussed in the following sections, and shown on Table 6.7, are the maximum stresses occurring in the area of concern. This model does not interpret the effect that the wall pounding against the diaphragm would have in the unretrofitted buildings. However, the significant separation of the two components implies that there could be a significant impact as the walls pound against the outside edge of the diaphragm. In addition, a separation larger than the bearing length for a joist could lead to collapse of the diaphragm.

6.3.6.2 Diaphragms

The largest stresses that developed in the diaphragm occurred in the corners adjacent to the in-plane wall, and were highest in the first floor diaphragm for the retrofitted case and at the roof diaphragm for the unretrofitted case. Typically, the stress in this location on the first floor diaphragm was twice the stress at the roof. As the diaphragm was strengthened, the stress in the corner of the roof diaphragm reduced to almost half of the stress value in the unretrofitted building with "good" masonry, and less than one-third for the "poor" masonry condition (see Table 6.7). Unlike the variation in the roof stress, the stress at the first floor level increased when it was retrofitted. The stress in the first floor is not as high as the roof stress in the unretrofitted condition, and not as low as the roof in the retrofitted condition. So while the stress does change, the change is not as substantial. In the center portion of the diaphragm at both floors, the stress reduces in the retrofitted cases. However, the amount of variation at each floor level is quite different. The stress in the retrofitted case is less than 25% of the unretrofitted at the first floor level and 50% at the roof level.

Because the stress in the diaphragm generally decreases once the building is retrofitted, the critical areas where the addition of a retrofit could create a possible weakness are the out-of-plane walls and the diaphragm–to-wall connection. Either of these elements could potentially attract more stress than it had prior to the retrofit. The stresses that develop along the outside edge of the diaphragm place demands on the anchors that are closer to the strength of the anchors.

Table 6.7 Maximum Stresses in the Floor and Roof Diaphragms

Parameter		Prototype Condition					
		Good Masonry		Fair Masonry		Poor Masonry	
		UR	R	UR	R	UR	R
Roof	At B_R, kPa (psi)	7,710 (1,120)	4,450 (645)	6,490 (942)	2,500 (363)	5,470 (793)	1,580 (229)
	At A_R, kPa (psi)	1,460 (212)	700 (101)	1,190 (172)	610 (88.4)	1,020 (148)	762 (111)
Level 1	At B_L, kPa (psi)	5,300 (768)	6,980 (1,010)	5,330 (773)	3,980 (578)	5,780 (838)	2,580 (375)
	At A_L, kPa (psi)	2,810 (408)	661 (95.8)	1,120 (162)	565 (81.9)	1,160 (168)	391 (56.7)

UR = Unretrofitted, R = Retrofitted
See Fig. 6.3 for key to reported locations.

6.3.6.3 In-Plane URM Walls

The stresses of concern in the in-plane walls are that of shear stress at the base of the wall where they are at a maximum. As shown in Table 6.8, the stresses in the in-plane walls increase when the diaphragms are retrofitted. For the “good” masonry condition, the stress corresponding to the retrofitted condition is 2.3 times the stress in the unretrofitted condition at the center of the in-plane wall and 2.5 times the unretrofitted condition at the corners. While not as substantial in the “poor” masonry condition, this stress increase is consistent at both the corner and center of the in-plane wall.

It is interesting to note that the stresses along the base of the wall vary more in the unretrofitted case than in the retrofitted case (see Table 6.8). Only one condition causes the shear stress to slightly exceed the allowable shear stress as provided by FEMA 273 and FEMA 356. However, the stresses due to retrofitting the diaphragm much more closely approach the allowable stresses than for the unretrofitted building.

Table 6.8 Maximum Stress in In-Plane URM Wall

Parameter	Prototype Condition					
	Good Masonry		Fair Masonry		Poor Masonry	
	UR	R	UR	R	UR	R
In-Plane Wall (at C), kPa (psi)	73.3 (10.6)	166 (24.1)	62.2 (9.02)	146 (21.2)	55.0 (7.98)	88.8 (12.9)
In-Plane Wall (at D), kPa (psi)	46.1 (6.69)	117 (16.9)	40.1 (5.81)	102 (14.7)	35.4 (5.13)	65.3 (9.47)
Shear Strength, kPa (psi)	186 (27.0)	186 (27.0)	138 (20.0)	138 (20.0)	89.6 (13.0)	89.6 (13.0)

6.3.6.4 Out-of-Plane URM Walls

From the analyses, the highest stresses in the out-of-plane wall occur in the center of the wall at mid-height between the first floor level and the roof diaphragm. The tensile strength of URM according to FEMA 273 and FEMA 356 is based on the existing masonry condition and is shown in Table 6.10. The stresses reported in this table are the median values of the maximum stresses in the out-of-plane walls. These values exceed the tensile stress in all of the reported locations along the out-of-plane wall, but most severely at mid-height (location K). Thus, the most critical areas of concern are central portions of the out-of-plane walls at mid-height. These large stresses coincide with the area where the deformation is the largest in the walls. In the unretrofitted building, the stress at mid-height is about 1.8 times the stress at the first floor level for the two weaker conditions of masonry and about 4.6 times the stress along the top of the wall near the roof diaphragm for the good masonry condition. The stress at the mid-height is between 1.1 to 2.0 times the stress in the roof location in the unretrofitted condition. However, in the retrofitted building, the stress at the mid-height increases substantially and is three and five times the stress on the roof location, increasing with deteriorating masonry condition.

When the building is retrofitted the stresses increase at the first level and midheight and decrease at the roof level, with the exception of the good condition of masonry which decreases slightly (see Table 6.9). The stress in the central portion (locations L, K and J) of the wall is significantly higher than the stress along the edge of the wall adjacent to the in-plane wall (locations I, H and G). The stresses at L, K and J are anywhere from 2.2 to 4.9 times the stress at the corresponding locations at I, H and G. In the case of the good condition of masonry, the stress at location K is 2.2 times that of location L and 4.6 times that of location J. However, in the retrofitted condition, the stress is approximately 3 times the stress at both locations L and J. Note that the stress at the first level (J) actually increased once the building was retrofitted. The out-of-plane and in-plane wall stresses are summarized in Fig. 6.9. The stress in the center of the wall at the two diaphragm locations is actually less than at the edge of the out-of-plane wall, but does not vary significantly.

Table 6.9 Maximum Stress in Out-of-Plane URM Wall

Out of Plane Wall Location		Prototype Condition					
		Good Masonry		Fair Masonry		Poor Masonry	
		UR	R	UR	R	UR	R
Roof	At I, kPa (psi)	480 (69.6)	259 (37.6)	302 (43.8)	142 (20.6)	173 (25.1)	79.2 (11.5)
Roof	At L, kPa (psi)	432 (62.7)	255 (37.0)	334 (48.4)	213 (30.9)	267 (38.8)	122 (17.7)
Mid-height	At H, kPa (psi)	193 (28.0)	262 (37.9)	148 (21.4)	188 (27.3)	96.6 (14.0)	123 (17.8)
Mid-height	At K, kPa (psi)	944 (137)	781 (113)	324 (47.0)	658 (95.4)	251 (36.5)	401 (58.2)
Level 1	At G, kPa (psi)	301 (43.7)	358 (51.9)	233 (33.8)	215 (31.1)	216 (31.3)	120 (17.5)
Level 1	At J, kPa (psi)	204 (29.6)	279 (40.4)	168 (24.4)	271 (39.3)	142 (20.6)	212 (30.8)
Tensile Strength, kPa (psi)		140 (20)	140 (20)	69 (10)	69 (10)	0 (0)	0 (0)

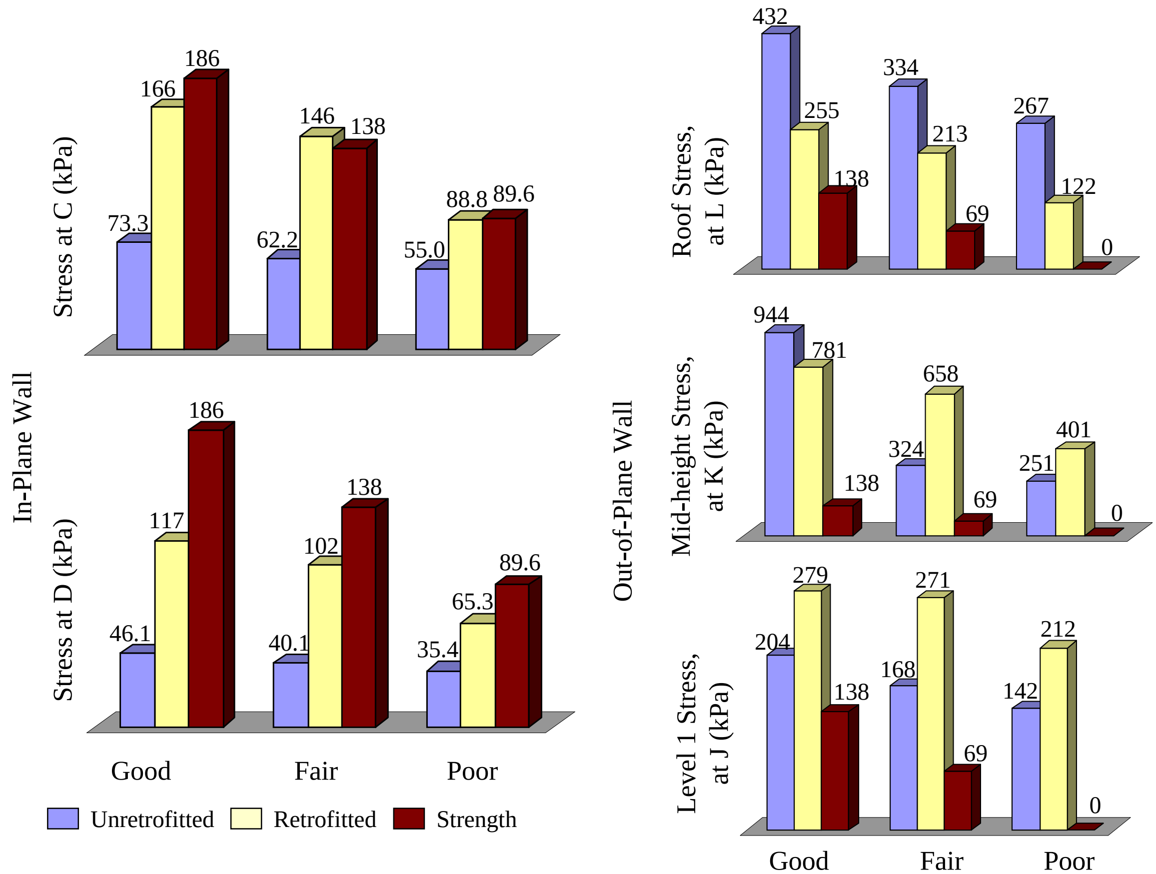


FIG. 6.9 Summary of Critical In-Plane and Out-of-Plane Wall Stresses

The suggested design tensile strength values for masonry are shown in Table 6.10. These values are suggested as default strengths by the FEMA guidelines and are provided by masonry condition: masonry in “good” condition is 138 kPa (20.0 psi), “fair” condition is 69.0 kPa (10.0 psi) and “poor” condition is 0 kPa (0.0 psi). These limiting values are exceeded at all locations in the out-of-plane wall: at the first floor level (locations G and J), the mid-height between the first floor and the roof diaphragm (locations H and K), and the roof level (locations G and J). As discussed above, the strength is the most severely exceeded at the mid-height level by a maximum factor of 6.4. At the first floor level for any condition of masonry and at the midheight level for the fair and poor condition of masonry, the stresses all increase significantly when the diaphragm and connections are retrofitted. The stresses at the other reported locations in the out-of-plane wall decrease as the diaphragm and connections are retrofitted, but still far exceed the allowable stress for URM.

The suggested design shear strength of masonry is shown in Table 6.8. These values are suggested as default strengths by the FEMA guidelines and provided by masonry condition: masonry in “good” condition is 186 kPa (27.0 psi), “fair” condition is 138 kPa (20.0 psi) and “poor” condition is 89.6 kPa (13.0 psi). The allowable stress is not exceeded at either location in the in-plane walls with the exception of the “fair” condition of masonry at location C. However, in the retrofitted prototype, the in-plane stress approaches the allowable stress. At location C in the in-plane wall, the demand comes within 10% of the allowable stress.

6.2.6.5 Connections

As discussed in Section 6.2.1.4, the strength of the retrofitted connection would be designed such that it is not a weak link that would fail first. Because the modeling included strong connections that were not necessarily based on the strength of an actual retrofitted connection, the parametric results for the connections are not specifically discussed here. However, the above results demonstrate the benefit of adding retrofit connections at a relatively close spacing.

6.4 SUMMARY OF PARAMETRIC STUDY RESULTS

The parametric study demonstrates that retrofitting the diaphragms changes the response of the building system. The performance of the diaphragm is improved in that the deflections are minimized at both the roof and first floor level and the stresses are decreased at each level, with one exception that does not exceed the diaphragm strength. Although the stress demands on the in-plane walls did not exceed the allowable stress, the stresses increased significantly and approach the allowable strength. Had the study evaluated a more intense earthquake, such as less frequent seismic event, it is likely that these results would be different.

The diaphragm retrofits mitigated the displacement of the out-of-plane walls, which is expected to improve their stability. However, the out-of-plane wall stresses in the unretrofitted model exceeded the allowable stress, and the situation was worsened with a retrofitted diaphragm and connection. The areas that already had high stresses showed substantially increased stresses for the retrofitted case. The central portion of the walls, which had the highest stress and the most severe displacement in the unretrofitted cases, had amplified values for the retrofitted case. In both the unretrofitted and retrofitted cases, the stresses exceed the allowable tensile strength for out-of-plane bending. The diaphragm retrofits also led to increased stresses in the in-plane walls, along with increased base shear forces. Except for one case, the in-plane wall stresses remained within the allowable shear stress limits. Based on these observations, it is not recommended to retrofit the diaphragm without a structure-specific study on the redistribution of lateral forces to the out-of-plane walls, along with an evaluation of the impact of the retrofit on the in-plane walls and the foundation.

6.5 DISCUSSION

It should be noted that the results of this study are specific to the parameters used for the analysis. Several of these parameters are discussed below to highlight their importance to the outcome of the LSP analysis and evaluation.

The accelerations used for the LSP are affected by two primary components: the soil type and local ground motions. Soil type C, as defined by FEMA 273 and FEMA 356, requiring no soil amplification factors, was used for this study to not overestimate the expected damage to the case study structures. While this soil type is appropriate for much of the St. Louis area, the Mid-America region commonly has type D soils and in some locations type E soils. The short period spectral acceleration values change from 1.2 for type C soil to 1.6 and 2.5 for type D and E soil, respectively; while the one second period spectral acceleration values increase from 1.7

for type C to 2.4 and 3.5 for type D and E, respectively. FEMA 273 and FEMA 356 specify that the demands governing the rehabilitation design criteria should be based on the larger of the following: 1) the smaller of 10% in 50 years earthquake or two-thirds of a 2% in 50 years earthquake evaluated considering the Life Safety Performance Level or 2) the 2% in 50 years earthquake considering the Collapse Prevention Performance Level. The demands for this study were based on ground motion records representing an earthquake that would have a 10% probability of exceedance in 50 years (10% in 50 years). Depending on local soil conditions and building type, the seismic demand may be greater than that considered in this study.

Typical large openings present in many URM buildings were not included in the analytical models for this study. The presence of these openings would greatly affect the structures ability to redistribute lateral forces when the building is retrofitted. It is noted that wall openings could have a significant impact on the structural performance.

The parameters appropriate for this study demonstrated that with a solid wall model and relatively low seismic demands, the out-of-plane wall performance was not satisfactory. Therefore, it is anticipated that similar structures in other parts of Mid-America could have even more significant vulnerabilities when evaluated using FEMA 273 and 356.

7. CONCLUSIONS

7.1 SUMMARY

This research study focused on evaluating the seismic performance of existing and rehabilitated wood floor and roof diaphragms in typical pre-1950s, unreinforced masonry (URM) buildings found in the Central and Eastern portions of the United States. Specifically there were two major objectives: (1) to assess the adequacy of current seismic guidelines for evaluating existing wood diaphragms in pre-1950s URM buildings and for designing necessary retrofits; and (2) to evaluate the effect of diaphragm retrofits, as designed by FEMA guidelines, on the overall response of URM structures.

The first objective was accomplished by utilizing two case study buildings located in St. Louis, Missouri, and evaluating them according to current seismic rehabilitation guidelines, FEMA 273 and FEMA 356. Each of the four analysis procedures provided in these guidelines was considered. However, only the Linear Static Procedure (LSP) was applicable for evaluating diaphragms in typical URM structures as components. Both FEMA 273 and FEMA 356 were used to allow an evaluation of the consistency of the results between the two guidelines. The procedures produced the recommendations shown in Table 7.1 for a satisfactory diaphragm retrofit, with the bolded retrofits being the most likely selected retrofits in practice because they are the most economic choices. If other constraints, such as aesthetics, were a concern, a different retrofit from the selection may be chosen. Intermediate steps in the FEMA 356 LSP gave significantly different values than FEMA 273, but ultimately the two guidelines gave the same retrofit solutions, with two more retrofit possibilities provided by FEMA 356.

TABLE 7.1 FEMA Recommended Diaphragm Retrofits

FEMA 273		**FEMA 356**	
Case Study Building 1	**Case Study Building 2**	**Case Study Building 1**	**Case Study Building 2**
-	-	Diagonal Sheathing with Straight Unchorded Sheathing	Diagonal Sheathing with Straight Unchorded Sheathing
Diagonal Sheathing with Straight Chorded Sheathing	Diagonal Sheathing with Straight Chorded Sheathing	Diagonal Sheathing with Straight Chorded Sheathing	Diagonal Sheathing with Straight Chorded Sheathing
-	-	**Unblocked, Unchorded Plywood Panel Overlay**	**Unblocked, Unchorded Plywood Panel Overlay**
Blocked, Unchorded Plywood Panel Overlay	**Blocked, Unchorded Plywood Panel Overlay**	Blocked, Unchorded Plywood Panel Overlay	Blocked, Unchorded Plywood Panel Overlay
Panel Blocked, Chorded Plywood Overlay	Panel Blocked, Chorded Plywood Overlay	Panel Blocked, Chorded Plywood Overlay	Panel Blocked, Chorded Plywood Overlay

The second objective was accomplished by defining a URM prototype building based on typical pre-1950s URM structures to be analyzed using the SAP 2000 finite element analysis program. A diaphragm retrofit based on the FEMA 273 and FEMA 356 LSP recommendations was used to create a retrofitted prototype. The selected retrofit was the blocked, chorded plywood panel overlay because this retrofit would have the most significant change from the existing single straight-sheathed diaphragm in terms of an increase in in-plane strength and stiffness. Structural response parameters for the unretrofitted and retrofitted structures were compared for three conditions of existing masonry (“good,” “fair,” and “poor”) under the demands for a 10% probability of exceedance in 50 years seismic event based on synthetic ground motions developed for St. Louis, Missouri utilizing representative soil conditions (Wen and Wu 2000). The response of each building was observed for the following components: the wood floor and roof diaphragms, the out-of-plane walls, and the in-plane walls. Each response was compared to applicable strength and deformation criteria.

7.2 CONCLUSIONS

The following conclusions were drawn from this study.

1. Three of the four analysis procedures provided in FEMA 273 and FEMA 356 were not desirable for the purposes of evaluating and selecting a rehabilitation approach for the diaphragm as a component in existing URM structures with the documentation provided. The Nonlinear Static Procedure, the Linear Dynamic Procedure, and the Nonlinear Dynamic Procedure all required analytical modeling of the entire structure as part of these procedures. Creating a finite-element model containing nonlinear properties that accurately predicts damage mechanisms in the URM walls is a significant task, because this behavior is not well understood, and such a model would require simplifying assumptions. While considering the system behavior is important, the focus of this work was on the diaphragm components and so this approach was not taken for the first phase of this study that focused on case study buildings.

2. Both FEMA 273 and FEMA 356 permit the possibility of rehabilitating the diaphragm without retrofitting the out-of-plane walls. The out-of-plane wall acceptance criteria consisted of height-to-thickness limits for the walls that depend on the wall location, building safety objective, and spectral response parameter. The existing condition of the masonry and the diaphragm stiffness is not taken into consideration.

3. Using the LSP from either FEMA 273 or FEMA 356, the existing diaphragms were not acceptable according to the provided acceptance criteria; hence both

case study buildings require a retrofit. The LSP from each of the guidelines permit three of the same diaphragm retrofits for the case study buildings. However, FEMA 356 had two additional retrofits that met the acceptance criteria for Life Safety performance. The LSP from FEMA 273 and FEMA 356 predict the same strength for the diaphragm, but the demand from FEMA 273 is twice that of the value from FEMA 356. Along these same lines, FEMA 273 gives larger values for the diaphragm forces and the base shear by factors of 2.3 and 3.1, respectively, as compared to FEMA 356. However, the FEMA 356 estimation of the diaphragm midspan displacement and the building period is more than double the corresponding FEMA 273 values due to differences in the equations used to estimate these quantities.

4. The parametric study gave the following observations for the general building response parameters for each masonry condition. The stiffening of the structure from the diaphragm retrofit caused the building period to decrease with a corresponding increase in the base shear. Both the displacement and acceleration for the roof diaphragm decreased when it was retrofitted. However, the 10% in 50 years seismic event used in the evaluation did not impose demands that were large enough to give significant displacements for either the unretrofitted or retrofitted cases. In addition, the building drift also decreased, but was not substantial even in the unretrofitted case. The acceleration in the in-plane walls increased substantially when the diaphragm was retrofitted.

5. For the parametric study, the most significant change in the structural response took place in the central portion of the out-of-plane walls. The results of the analysis showed the following:

    - The tensile stresses caused by out-of-plane bending in the out-of-plane walls exceed the allowable tensile stress for the unretrofitted case and more than double when the diaphragms are retrofitted. This is generally true for all reported locations on the out-of-plane wall.

- The deformed first mode shape of the out-of-plane walls changed significantly from the unretrofitted model to the retrofitted model. In the unretrofitted model, the out-of-plane wall arched away from the building between diaphragm levels as though it were cantilevering from the base. In the retrofitted building, the out-of-plane wall deformed as though it were a two-span beam, supported laterally at each floor level.

6. The stresses at the reported locations of the in-plane wall did not exceed the allowable shear strength. However, for the retrofitted condition, the stresses increased significantly and approached the allowable strength.

7. According to the FEMA 273 and FEMA 356 LSP recommendations, the diaphragm can be retrofitted as long as it meets certain criteria for deformation-controlled elements, and acceptable retrofits are given for the rehabilitation. In evaluating a single component, the LSP allows for the rehabilitation of the wood diaphragms without the rehabilitation of the walls because these case study buildings met the acceptable out-of-plane wall height-to-thickness ratios. A complete evaluation according to the FEMA guidelines would include all components. The results of the parametric study show that the addition of a diaphragm retrofit causes more severe stresses in the out-of-plane walls than with the existing diaphragm, with a potentially hazardous effect. These stresses cannot be sustained in the out-of-plane walls without some form of rehabilitation. Therefore, a diaphragm retrofit should be accompanied by an evaluation of the remaining structural components and those components should also be retrofitted, if necessary, to ensure adequate seismic performance of the complete structure.

7.3 RECOMMENDATIONS FOR FUTURE WORK

The following recommendations for future research would provide additional information necessary for the further development of guidelines for the seismic rehabilitation of wood floor and roof diaphragms in existing URM structures, along with a better understanding of the behavior of pre-1950s URM buildings. The suggested analytical work should be complemented by experimental studies.

1. The FEMA 273 and FEMA 356 guidelines provide an important first step in giving guidance for seismic rehabilitation of buildings, including URM structures. More information should be provided in the guidelines to provide guidance for modeling URM structures to reduce error due to oversimplification. This may include development of an accessible analytical model that more accurately predicts the behavior of URM walls and wood diaphragms or additional specific guidance for modeling these components as part of the structural system.

2. Nonlinear modeling of similar URM structures with wood floor and roof diaphragms using time history analyses would provide an improved understanding of the effect the diaphragm retrofit has on the response of the system into the inelastic range of behavior.

3. An additional parametric study is suggested for a higher intensity earthquake, the 2% in 50 years seismic event, evaluated for the Collapse Prevention Performance Level, again using representative soil conditions. A complete parametric study should evaluate the effect of wall and diaphragm openings, plan aspect ratio, and building height on the structural response of typical URM buildings. For these conditions, the use of a nonlinear model will be even more critical.

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APPENDIX A

LSP CALCULATIONS FOR EXISTING CASE STUDY BUILDING 1

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)		Floor 2 (Kips)	
Weight Floor 1	21.28			
Weight Floor 2			79.32	
IP Walls (W_{IPX})	41.50		17.90	
OOP Walls (W_{OOPx})	87.13		37.59	
Superimposed	(0.02 Ksf)	38.21	(0.01 Ksf)	19.11
Total Dead (W_{Fx})	188.12		153.91	
Live (IBC)	(0.04 Ksf)	76.42	(0.012 Ksf)	22.93
Snow (ASCE 7)			(0.022 Ksf)	42.03

2.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

2.6.1.4 Adjustment for Site Class C

From Table 2-13 — From Table 2-14

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_a S_s =$ 20.72 % (Eq 2-4)

$S_{x1} = F_v S_1 =$ 9.04 % (Eq 2-5)

2.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 2-15. For $\beta = 5\%$,

$B_s =$ 1.0 — $T_o = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 2-10)

$B_1 =$ 1.0 — $T_o =$ 0.4364 sec

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < 0.2T_o$	$S_a = (S_{XS}/B_X)*(0.4+3T/T_o)$	(Eq 2-8)
$0.2T_o < T < T_o$	$S_a = S_{XS}/B_S$	(Fig 2-1)
$T_o < T$	$S_a = S_{X1}/B_1T$	(Eq 2-9)

2.11.2 P-Δ Effects

$\theta = P\delta/(Vh)$ (Eq 2-14)

Ex. $P_1 = W_{F1} + (10\ \text{psf})*w*d$
$V =$ Force at each floor from 3.3.1.3(b)
$\delta =$ Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
$P_1 =$	361.1	Kips	$P_2 =$	153.9	Kips
$\delta =$	0.107	ft	$\delta =$	0.004	ft
$V =$	31.1	Kips	$V =$	46.3	Kips
$h =$	14.50	ft	$h =$	25.50	ft

$\theta_1 =$ 0.085549
$\theta_2 =$ 0.000581

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1} =$ 1.0

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-2)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-3)

where:	$Q_D =$	0.00	= dead load effect
	$Q_L =$	0.00	= live load effect (25% design live load)
	$Q_S =$	0.00	= 70% full snow load, not < 20% snow load

None of the above loads create lateral response

$Q_G =$ 0.00 Kips

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.2 Modeling and Analysis Considerations
Method 3 - Period Determination -
One story building with single span flexible diaphragm

$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5}$ (Eq 3-5)

Δ_w from SAP Analysis of wall relative to ground

$\Delta_w =$ 0.023 in

$\Delta_w =$ 0.023 in

$\Delta_d =$ 4.940 in

Δ_d found using Eq 8-5 and lateral load equal to the weight tributary to the diaphragm

T = 0.6226 sec

$\Delta_d = vd^4/(G_d w^3)$ (Eq 8-5)

$vy = ((h_1+h_2)/2*(d/2)*120\ \text{lb/ft}^3) + W_{F1}/2)/w$

vy = 1,686 lb/ft

3.3.1.3 Determination of Actions & Deformations

A) Psuedo Lateral Load

$V = C_1C_2C_3S_aW$ (Eq 3-6)

Ex. $W = W_{F1} + S + (10\text{psf})*w*d$

V = 77.31 Kips

$C_1 =$ 1.0
$C_2 =$ 1.1
$C_3 =$ 1.20
$S_a =$ 0.1453
(interpolated according to 2.6.1.5)
W = 403.2 Kips

B) Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-7)

$C_{VX} = w_x h_x^k/(\Sigma w_i h_i^k)$ (Eq 3-8)

where:

$w_1 =$	188.12	Kips	$w_2 =$	153.91	Kips
$h_1 =$	14.500	ft	$h_2 =$	25.500	ft
k =	1.061286		k =	1.0612856	
$\Sigma w_i h_i^k =$	7,999.8		$\Sigma w_i h_i^k =$	7,999.8	
$Cv_1 =$	0.4017		$Cv_2 =$	0.5983	

$F_1 =$ 31.05 Kips
$F_2 =$ 46.25 Kips

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

C) Horizontal Distribution of Seismic Forces
Distributed according to distribution of mass at floor level

D) Floor Diaphragms

$F_{PX} = (C_1C_2C_3)^{-1} * \Sigma(F_i * (w_x/\Sigma w_i))$ (Eq 3-9)
w_i = portion of total building
weight assigned to floor level i

$C_1C_2C_3 =$ 1.320
$F_1 =$ 31.05 Kips
$F_2 =$ 46.25 Kips
$w_1 =$ 188.12 Kips
$w_2 =$ 153.91 Kips
$\Sigma w_i =$ 342.03 Kips

$F_{p1} =$ 32.21 Kips
$F_{p2} =$ 26.35 Kips

E) Determination of Deformations
8.5.2 Single Straight Sheathed Diaphragms

$\Delta = vd^4/(G_d w^3)$ (Eq 8-5)
$v_2 = F_{p2}*1000/d/2$

$\Delta_d =$ 1.280 in

$G_d =$ 200,000 lb/in
$d =$ 63.33 ft
$w =$ 30.17 ft
$v_2 =$ 436.82 lb/ft

3.4.2 Acceptance Criteria

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-14)
where:
$Q_G =$ 0.00 = action due to design gravity loads (Kips)
$Q_E =$ 26.35 = action due to design eq loads (Kips)
$Q_{UD} =$ 26.35 Kips

3.4.2.1.B Force Controlled Actions

$Q_{UF} = Q_G +/- Q_E/(C_1C_2C_3)$ (Eq 3-16)
$Q_{UF} =$ 19.97 Kips

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2.2.A Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-18)

Q_{CE} = Yeild Capacity*w/1000

where:

m =	1.5	= single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-1
κ =	0.75	= knowledge factor (REF 4 2.7.2)
Yield Capacity =	120	lb/ft
Q_{CE} =	3.62	= expected strength (Kips)
$m\kappa Q_{CE}$ =	4.07	Kips
Q_{UD} =	26.35	Kips

Does it satisfy criteria??? No

8.5.2.3 Deformation Acceptance Criteria

Criteria will largely depend on the allowable deformations for other structural and non-structural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of the diaphragm.

3.4.2.2.B. Acceptance Criteria for Force-Controlled Actions

k QCL >= QUF

k =	0.75	
QCL =	7.6	
k QCL =	5.7	Kips
QUF =	19.97	Kips

Does this satify criteria??? No

2.9.1.1 Demand Capacity Ratio

QUD/QCE = 7.2803101

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)		Floor 2 (Kips)	
Weight Floor 1		21.28		
Weight Floor 2				79.32
IP Walls (W_{IPX})		41.50		17.90
OOP Walls (W_{OOPX})		87.13		37.59
Superimposed	(0.02 Ksf)	38.21	(0.01 Ksf)	19.11
Total Dead (W_{FX})		188.12		153.91
Live (IBC)	(0.04 Ksf)	76.42	(0.012 Ksf)	22.93
Snow (ASCE 7)			(0.022 Ksf)	42.03

1.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

1.6.1.4 Adjustment for Site Class C

From Table 1-4 — From Table 1-5

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_a S_s =$ 20.724 % (Eq 1-4)

$S_{x1} = F_v S_1 =$ 9.044 % (Eq 1-5)

1.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 1-6. For β = 5%,

$B_s =$ 1.0 — $T_S = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 1-11)

$B_1 =$ 1.0 — $T_S =$ 0.4364 sec

$T_o = 0.2 * T_S =$ 0.0873 sec

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < T_o$	$S_a = S_{XS}((5/B_S - 2)*(T/T_S) + 0.4)$	(Eq 1-8)
$T_o < T < T_S$	$S_a = S_{XS}/B_S$	(Eq 1-9)
$T_S < T$	$S_a = S_{X1}/B_1T$	(Eq 1-10)

2.6.7.1 Out of Plane Anchorage to Diaphragms

$F_p = \chi S_{XS} W$ Wall anchor connections shall be considered force-controlled. F_p not less than minimum of 400 lb/ft or $400S_{XS}$

where:

$\chi =$ 1.2 Table 2-4 based on Life Safety

$S_{XS} =$ 0.20724

$W =$ 1.62 = wall weight tributary to the anchor (K)

$F_p =$ 0.402 K/ft

3.0 Analysis Procedures

3.2.4 Classification of Diaphragms

Flexible when the maximum horizonal deformation of the diaphragm along its length is morethan twice the average interstory drift of the vertical lateral-force-resisting elements of the storyimmediately below the diaphragm.

Max horizontal deformation = 2.984
Average interstory drift = 0.003

Is the diaphragm flexible? Yes

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.2.5.1.1 Linear Procedures

$\theta = P\delta/(Vh)$ (Eq 3-2)

Ex. $P_1 = W_{F1} + (10psf) * w * d$
V = Force at each floor from 3.3.1.3(b)
δ = Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
P =	361.1	Kips	P =	153.9	Kips
δ =	0.124	ft	δ =	0.005	ft
V =	8.8	Kips	V =	16.5	Kips
h =	14.50	ft	h =	25.50	ft

$\theta_1 =$ 0.352636
$\theta_2 =$ 0.001898

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1} =$ 1.000

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-3)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-4)
where:

Q_D =	0.00	= dead load effect (Kips)
Q_L =	0.00	= live load effect (25% design live load) (Kips)
Q_S =	0.00	= 70% full snow load, greater than 20% full design snow load.
		None of the above loads create lateral response
Q_G =	0.00	Kips

3.3.1.2 Period Determination
Method 3 - URM Buildings with single span flexible diaphragms, < 6 stories

$T = (0.078\Delta_d)^{0.5}$ (Eq 3-9) Where Δ_d found using Eq 8-3 and lateral load equal to the weight tributary to the diaphragm

$\Delta_d =$ 26.690 in

$\Delta_y = v_y d/(2G_d)$

$vy = ((h_1+h_2)*d)/2*120\ lb/ft^3) + W_{F1}/2)/w$

T = 1.4429 sec

vy = 1,686 K/ft

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.3 Determination of Actions & Deformations

3.3.1.3.1 Psuedo Lateral Load

$V = C_1C_2C_3C_mS_aW$

Ex. $W = W_{F1} + S + (10psf)*w*d$

$V =$ 25.27 Kips

$C_1 =$ 1.0
$C_2 =$ 1.0
$C_3 =$ 1.0
$C_m =$ 1.0
$S_a =$ 0.0627
(interpolated according to 1.6.1.5.1)
$W =$ 403.2 Kips

3.3.1.3.2 Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-11)
$C_{VX} = w_xh_x^k/(\Sigma w_ih_i^k)$ (Eq 3-12)

where:

$w_1 =$	188.12	Kips	$w_2 =$	153.91	Kips
$h_1 =$	14.50	ft	$h_2 =$	25.50	ft
$k =$	1.47143		$k =$	1.47143	
$\Sigma w_ih_i^k =$	27690.00		$\Sigma w_ih_i^k =$	27690.00	
$Cv_1 =$	0.3475		$Cv_2 =$	0.6525	

Ex. $w_1 = W_{F1}$

$F_1 =$ 8.78 Kips
$F_2 =$ 16.49 Kips

3.3.1.3.3 Horizontal Distribution of Seismic Forces

Distributed according to distribution of mass at floor level; however, none is given to out-of-plane walls.

3.3.1.3.4 Floor Diaphragms

$F_{px} = \Sigma F_i(w_x/\Sigma w_i)$ (Eq 3-13)

w_i = portion of total building weight assigned to floor level i

$F_1 =$ 8.78 Kips
$F_2 =$ 16.49 Kips
$w_1 =$ 188.12 Kips
$w_2 =$ 153.91 Kips

$\Sigma w_i =$ 342.03 Kips

$F_{p1} =$ 13.90 Kips
$F_{p2} =$ 11.37 Kips

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms

1. For each span of the building and at each level, calculate period from Eq 3-9.
2. Using Eq 3-10, calculate pseudo lateral load for each span.
3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic-resisting elements using tributary loads.
4. Diaphragm forces for evaluation of diaphragms shall be determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shape.
5. Diaphragm deflection shall not exceed 6 inches for this method of distributuion of pseudo lateral loads to be applicable.

8.6.3.1 Single Straight Sheathing - Determination of Deformations

$\Delta_y = v_y d/(2G_d)$ (Eq 8-3)

$G_d =$ 2,000 lb/in
$d =$ 63.33 ft
$v_{y2} =$ 188.48 lb/ft

$\Delta_d =$ 2.984 in

** not to exceed 6 in

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2 Acceptance Criteria for Linear Procedures

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-18)

where:

$Q_G =$	0.00	= action due to design gravity loads (Kips)
$Q_E =$	11.37	= action due to design eq loads (Kips)
$Q_{UD} =$	11.37	Kips

3.4.2.2.A Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-20)

Q_{CE} = Yield Capacity*w/1000

where:

m =	1.5	= single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-3
κ =	0.75	= knowledge factor
Yield Capacity =	120	lb/ft
$Q_{CE} =$	3.62	= expected strength (Kips)
$m\kappa Q_{CE} =$	4.07	Kips
$Q_{UD} =$	11.37	Kips

Does it satisfy criteria??? **No**

CASE STUDY BUILDING 1
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

8.5.2.3 Deformation Acceptance Criteria

Criteria will largely depend on the allowable deformations for other structural and non-structural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of diaphragm.

3.4.2.2.B. Acceptance Criteria for Force-Controlled Actions

k QCL >= QUF

k =	0.75	
QCL =	7.6	
k QCL =	5.7	Kips
QUF =	11.37	Kips

Does this satify criteria??? No

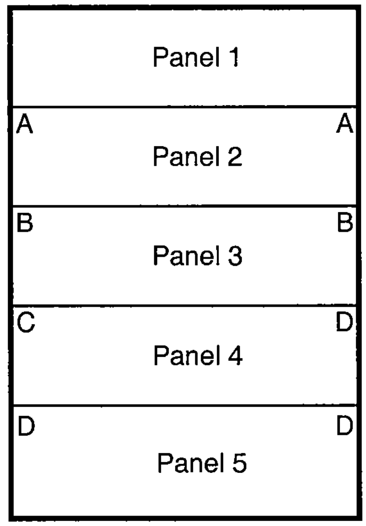
2.4.1.1 Demand Capacity Ratio

QUD/QCE 3.141339

CASE STUDY BUILDING 1
BUILDING WEIGHT CALCULATIONS

Overall Dimensions:

Height (h_1):	14.500	ft
Height (h_2):	11.000	ft
Width (w):	30.167	ft
Depth (d):	63.333	ft
Panel 1:	11.229	ft
Panel 2:	11.333	ft
Panel 3:	13.542	ft
Panel 4:	13.313	ft
Panel 5:	11.417	ft



Aspect Ratio: 2.10

First Elevated Floor (Second Floor):

Beams:

Ibeams

AA	15"-42.9#I =	42.9	lb/ft
BB	18"-54.7#I =	54.7	lb/ft
CC	15"-42.9#I =	42.9	lb/ft
DD	18"-54.7#I =	54.7	lb/ft

Weight of I Beams = (AA+BB+CC+DD) * w
Weight of I Beams = 5,888.53 lb

Angles (2Ls 3" x 4" x 3/8")
Weight per 2L = 17 lb/ft
Weight of Angles = 4,136.67 lb

Total Beam Weight = 10,025.20 lb

Joists (2"x10"):

Weight of Wood (W_{wood})= 0.01986 lb/in^3
Weight per Joist ($W_{perJoist}$) = 4.8 lb/ft
Number of Joists = 45.0
Weight of Joists (W_{joist}) = 6,470.75 lb

Sheathing (7/8"):
Thickness of Sheathing (T_{sheath}) = 0.88 in
$W_{sheath} = T_{sheath} * w * d * W_{wood}$
Weight of Sheathing (W_{sheath}) = 4,781.17 lb

Total Weight of 1st Floor = 21.28 Kips

CASE STUDY BUILDING 1
BUILDING WEIGHT CALCULATIONS

Floor 2 (Roof)

Rafters:

Joist Size	Number	Length (ft)	Weight (lb)
2" x 8"	13.5	37.58	1,934.79
2" x 12"	13.5	49.20	3,799.22
1" x 6"	13.5	36.80	710.42
2" x 8"	42	30.17	4,831.49
2" x 10"	42	41.16	8,240.23
1" x 6"	42	31.56	1,895.49
2" x 14"	1	74.33	496.05
	Total Rafter Weight (Kips)=		**21.91**

Slate:

Slate Area (A_{slate}) = 4,611.12 ft^2

Weight Slate (W_{slate}) = 49.8 lb/ft^3

Thickness Slate (T_s) = 3 in

$$W_{roof} = A_{slate}*T_s*W_{slate}$$

Weight of Roof (W_{roof}) = 57.41 Kips

Total Weight of Roof = 79.32 Kips

CASE STUDY BUILDING 1
BUILDING WEIGHT CALCULATIONS

Walls:

Masonry Weight (W_m) = 49.8 lb/ft^3
Thickness (T_m) = 13 in

Portion of Walls contributing to Floor 1

In-Plane
$W_{IP1} = 1/2*(h_1 + h_2)*(2*w)*T_m*W_m$
W_{IP1} = 41.50 Kips

Out-of-Plane
$W_{OOP1} = 1/2*(h_1 + h_2)*(2*d)*T_m*W_m$
W_{OOP1} = 87.13 Kips

Portion of Walls contributing to Floor 2

In-Plane
$W_{IP2} = 1/2*(h_2)*(2*w)*T_m*W_m$
W_{IP2} = 17.90 Kips

Out-of-Plane
$W_{OOP2} = 1/2*(h_2)*(2*d)*T_m*W_m$
W_{OOP2} = 37.59 Kips

APPENDIX B

SAMPLE LSP CALCULATION FOR RETROFITTED CASE STUDY BUILDING 1

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)		Floor 2 (Kips)	
Weight Floor 1		21.28		
Retrofit		4.78		
Weight Floor 2				79.32
IP Walls (W_{IPX})		41.50		17.90
OOP Walls (W_{OOPx})		87.13		37.59
Superimposed	(0.02 Ksf)	38.21	(0.01 Ksf)	19.11
Total Dead (W_{Fx})		192.90		153.91
Live (IBC)	(0.04 Ksf)	76.42	(0.012 Ksf)	22.93
Snow (ASCE 7)			(0.022 Ksf)	42.03

2.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

2.6.1.4 Adjustment for Site Class C

From Table 2-13 — From Table 2-14

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_a S_s =$ 20.72 % (Eq 2-4)

$S_{x1} = F_v S_1 =$ 9.04 % (Eq 2-5)

2.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 2-1

For β = 5%,

$B_s =$ 1.0 — $T_o = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 2-10)

$B_1 =$ 1.0 — $T_o =$ 0.4364 sec

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < 0.2T_o$	$S_a = (S_{XS}/B_X)*(0.4+3T/T_o)$	(Eq 2-8)
$0.2T_o < T < T_o$	$S_a = S_{XS}/B_S$	(Fig 2-1)
$T_o < T$	$S_a = S_{X1}/B_1T$	(Eq 2-9)

2.11.2 P-Δ Effects

$\theta = P\delta/(Vh)$ (Eq 2-14)

Ex. $P_1 = W_{F1} + (10\ psf)*w*d$
V = Force at each floor from 3.3.1.3(b)
δ = Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
P_1 =	365.9	Kips	P_2 =	153.9	Kips
δ =	0.017	ft	δ =	0.001	ft
V =	41.7	Kips	V =	58.5	Kips
h =	14.50	ft	h =	25.50	ft

θ_1 = 0.010213
θ_2 = 7.3E-05

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1}$ = 1.0

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-2)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-3)

where:	Q_D =	0.00	= dead load effect
	Q_L =	0.00	= live load effect (25% design live load)
	Q_S =	0.00	= 70% full snow load, not less than 20% full desig snow load

Q_G = 0.00 Kips None of the above loads create lateral response

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.2 Modeling and Analysis Considerations

Method 3 - Period Determination - One story building with single span flexible diaphra

$T = (0.1\Delta_w + 0.078\Delta_d)^0$ (Eq 3-5)

Δ_w from SAP Analysis of wall relative to ground

$\Delta_w =$ 0.023 in

Δ_d found using Eq 8-5 and lateral load equal to the weight tributary to the diaphragm

$\Delta_w =$ 0.023 in

$\Delta_d =$ 0.575 in

$\Delta_d = vd^4/(G_d w^3)$ (Eq 8-5)

$vy = ((h_1+h_2)/2*(d/2)*120\ \text{lb/ft}^3) + W_{F1}/2)/w$

T = 0.2171 sec

vy = 1,765 lb/ft

3.3.1.3 Determination of Actions & Deformations

A) Psuedo Lateral Load

$V = C_1C_2C_3S_aW$ (Eq 3-6)

Ex. $W = W_{F1} + S + (10\text{psf})*w*d$

V = 100.19 Kips

$C_1 =$ 1.0

$C_2 =$ 1.2

$C_3 =$ 1.00

$S_a =$ 0.21

(interpolated according to 2.6.1.5)

W = 407.9 Kips

B) Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-7)

$C_{VX} = w_x h_x^k/(\Sigma w_i h_i^k)$ (Eq 3-8)

Ex. $w_1 = W_{F1}$

where:

$w_1 =$	192.90	Kips	$w_2 =$	153.91	Kips
$h_1 =$	14.500	ft	$h_2 =$	25.500	ft
k =	1		k =	1	
$\Sigma w_i h_i^k =$	6,721.7		$\Sigma w_i h_i^k =$	6,721.7	
$Cv_1 =$	0.4161		$Cv_2 =$	0.5839	

$F_1 =$ 41.69 Kips

$F_2 =$ 58.50 Kips

C) Horizontal Distribution of Seismic Forces

Distributed according to distribution of mass at floor level

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

D) Floor Diaphragms

$F_{PX} = (C_1C_2C_3)^{-1} * \Sigma(F_i * (w_x/\Sigma w_i))$ (Eq 3-9)

w_i = portion of total building weight assigned to floor level i

$C_1C_2C_3$ =	1.185	
F_1 =	41.69	Kips
F_2 =	58.50	Kips
w_1 =	192.90	Kips
w_2 =	153.91	Kips
Σw_i =	346.81	Kips

F_{p1} =	47.02	Kips
F_{p2} =	37.52	Kips

E) Determination of Deformations

8.5.2 Single Straight Sheathed Diaphragms

$\Delta = vd^4/(G_dw^3)$ (Eq 8-5)

$v_2 = F_{p2}*1000/d/2$

G_d =	1,800,000	lb/in
d =	63.33	ft
w =	30.17	ft
v_2 =	621.86	lb/ft

Δ_d =	0.202	in

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2 Acceptance Criteria

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-14)

where:

$Q_G =$	0.00	= action due to design gravity loads (Kips)
$Q_E =$	37.52	= action due to design earthquake loads (Kips)
$Q_{UD} =$	37.52	Kips

3.4.2.1.B Force Controlled Actions

$Q_{UF} = Q_G +/- Q_E/(C_1C_2C_3)$ (Eq 3-16)

$Q_{UF} =$ 31.66 Kips

3.4.2.2.A Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-18)

$Q_{CE} =$ Yeild Capacity*w/1000

where:

m =	3.0	= single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-1
$\kappa =$	0.75	= knowledge factor (REF 4 2.7.2)
Yield Capacity =	960	lb/ft
$Q_{CE} =$	28.96	= expected strength (Kips)
$m\kappa Q_{CE} =$	65.16	Kips
$Q_{UD} =$	37.52	Kips

Does it satisfy criteria??? **Yes**

2.9.1.1 Demand Capacity Ratio

QUD/QCE = 1.30

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)		Floor 2 (Kips)	
Weight Floor 1		21.28		
Retrofit		4.78		
Weight Floor 2				79.32
IP Walls (W_{IPX})		41.50		17.90
OOP Walls (W_{OOPX})		87.13		37.59
Superimposed	(0.02 Ksf)	38.21	(0.01 Ksf)	19.11
Total Dead (W_{FX})		192.90		153.91
Live (IBC)	(0.04 Ksf)	76.42	(0.012 Ksf)	22.93
Snow (ASCE 7)			(0.022 Ksf)	42.03

1.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

1.6.1.4 Adjustment for Site Class C

From Table 1-4 — From Table 1-5

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_a S_s =$ 20.724 % (Eq 1-4)

$S_{x1} = F_v S_1 =$ 9.044 % (Eq 1-5)

1.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 1-6

For $\beta = 5\%$,

$B_s =$ 1.0 — $T_S = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 1-11)

$B_1 =$ 1.0 — $T_S =$ 0.4364 sec

$T_o = 0.2 * T_S =$ 0.0873 sec

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < T_o$	$S_a = S_{XS}((5/B_S - 2)*(T/T_S) + 0.4)$	(Eq 1-8)
$T_o < T < T_S$	$S_a = S_{XS}/B_S$	(Eq 1-9)
$T_S < T$	$S_a = S_{X1}/B_1T$	(Eq 1-10)

2.6.7.1 Out of Plane Anchorage to Diaphragms

$F_p = \chi S_{XS} W$ Wall anchor connections shall be considered force-controlled

F_p not less than minimum of 400 lb/ft or $400S_{XS}$

where:

χ =	1.2	Table 2-4 based on Life Safety
S_{XS} =	0.20724	
W =	1.62	= wall weight trib to the anchor (K)
F_p =	0.402	K/ft

3.0 Analysis Procedures

3.2.4 Classification of Diaphragms

Flexible when the maximum horizonal deformation of the diaphragm along its length is more than twice the average interstory drift of the vertical lateral-force-resisting elements of the story immediately below the diaphragm.

Max horizontal deformation =	1.556
Average interstory drift =	0.003
Is the diaphragm flexible?	**Yes**

3.2.5.1.1 Linear Procedures

$\theta = P\delta/(Vh)$ (Eq 3-2)

Ex. $P_1 = W_{F1}$ + (10psf) *w*d

V = Force at each floor from 3.3.1.3(b)

δ = Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
P =	365.9	Kips	P =	153.9	Kips
δ =	0.130	ft	δ =	0.005	ft
V =	18.5	Kips	V =	28.2	Kips
h =	14.50	ft	h =	25.50	ft

θ_1 = 0.176514

θ_2 = 0.001164

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1} =$ 1.364

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1\ (Q_D + Q_L + Q_S)$ (Eq 3-3)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-4)

where:

$Q_D =$	0.00	= dead load effect (Kips)
$Q_L =$	0.00	= live load effect (25% design live load) (Kips)
$Q_S =$	0.00	= 70% full snow load, greater than 20% full design snow load
		None of the above loads create lateral response
$Q_G =$	0.00	Kips

3.3.1.2 Period Determination

Method 3 - URM Buildings with single span flexible diaphragms, less than 6 stories

$T = (0.078\Delta_d)^{0.5}$ (Eq 3-9) Where Δ_d found using Eq 8-3 and lateral load equal to the weight tributary to the diaphragm

$\Delta_d =$ 7.984 in $\Delta_y = v_y d/(2G_d)$

$vy = ((h_1+h_2)*d)/2*120\ lb/ft^3) + W_{F1}/2)/w$

$T =$ 0.7892 sec $vy =$ 1,765 K/ft

3.3.1.3 Determination of Actions & Deformations

3.3.1.3.1 Psuedo Lateral Load

$V = C_1C_2C_3C_mS_aW$

Ex. $W = W_{F1} + S + (10psf)*w*d$

$V =$ 46.75 Kips

$C_1 =$	1.0
$C_2 =$	1.0
$C_3 =$	1.0
$C_m =$	1.0
$S_a =$	0.1146

(interpolated according to 1.6.1.5.1)

$W =$ 407.9 Kips

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.3.2 Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-11)

$C_{VX} = w_x h_x^k/(\Sigma w_i h_i^k)$ (Eq 3-12)

Ex. $w_1 = W_{F1}$

where:

$w_1 =$	192.90	Kips	$w_2 =$	153.91	Kips
$h_1 =$	14.50	ft	$h_2 =$	25.50	ft
$k =$	1.14458		$k =$	1.1445805	
$\Sigma w_i h_i^k =$	10385.78		$\Sigma w_i h_i^k =$	10385.78	
$Cv_1 =$	0.3964		$Cv_2 =$	0.6036	

$F_1 =$	18.53	Kips
$F_2 =$	28.22	Kips

3.3.1.3.3 Horizontal Distribution of Seismic Forces

Distributed according to distribution of mass at floor level; however, none is given to out-of-plane walls.

3.3.1.3.4 Floor Diaphragms

$F_{px} = \Sigma F_i(w_x/\Sigma w_i)$ (Eq 3-13)

w_i = portion of total building weight assigned to floor level i

$F_1 =$	18.53	Kips
$F_2 =$	28.22	Kips
$w_1 =$	192.90	Kips
$w_2 =$	153.91	Kips
$\Sigma w_i =$	346.81	Kips

$F_{p1} =$	26.00	Kips
$F_{p2} =$	20.75	Kips

3.3.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms

1. For each span of the building and at each level, calculate period from Eq 3-9.
2. Using Eq 3-10, calculate pseudo lateral load for each span.
3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic-resisting elements using tributary loads.
4. Diaphragm forces for evaluation of diaphragms shall be determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shape.
5. Diaphragm deflection shall not exceed 6 inches for this method of distributuion of pseudo lateral loads to be applicable.

CASE STUDY BUILDING 1 - RETROFIT
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

8.6.3.1 Single Straight Sheathing - Determination of Deformations

$\Delta_y = \nu_y d/(2G_d)$ (Eq 8-3)

G_d = 7,000 lb/in
d = 63.33 ft
ν_{y2} = 343.89 lb/ft

Δ_d = 1.556 in

** not to exceed 6 in

3.4.2 Acceptance Criteria for Linear Procedures

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G$ +/- Q_E (Eq 3-18)

where:

Q_G =	0.00	= action due to design gravity loads (Kips)
Q_E =	20.75	= action due to design earthquake loads (Kips)
Q_{UD} =	20.75	Kips

3.4.2.2.A Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-20)

Q_{CE} = Yield Capacity*w/1000

where:

m =	1.5	= single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-3
κ =	0.75	= knowledge factor
Yield Capacity =	400	lb/ft
Q_{CE} =	12.07	= expected strength (Kips)

$m\kappa Q_{CE}$ =	13.58	Kips
Q_{UD} =	20.75	Kips

Does it satisfy criteria??? **No**

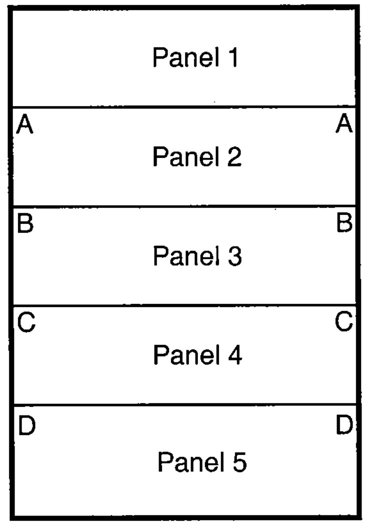
2.4.1.1 Demand Capacity Ratio

QUD/QCE = 1.72

CASE STUDY BUILDING 1 - RETROFIT
BUILDING WEIGHT CALCULATIONS

Overall Dimensions:

Height (h_1):	14.500	ft
Height (h_2):	11.000	ft
Width (w):	30.167	ft
Depth (d):	63.333	ft
Panel 1:	11.229	ft
Panel 2:	11.333	ft
Panel 3:	13.542	ft
Panel 4:	13.313	ft
Panel 5:	11.417	ft



Aspect Ratio: 2.10

First Elevated Floor (Second Floor):

Beams:

Ibeams

AA	15"-42.9#I =	42.9	lb/ft
BB	18"-54.7#I =	54.7	lb/ft
CC	15"-42.9#I =	42.9	lb/ft
DD	18"-54.7#I =	54.7	lb/ft

Weight of I Beams = (AA+BB+CC+DD) * w
Weight of I Beams = 5,888.53 lb

Angles (2Ls 3" x 4" x 3/8")
Weight per 2L = 17 lb/ft
Weight of Angles = 4,136.67 lb

Total Beam Weight = 10,025.20 lb

Joists (2"x10"):

Weight of Wood (W_{wood})= 0.01986 lb/in^3
Weight per Joist ($W_{perJoist}$) = 4.8 lb/ft
Number of Joists = 45.0
Weight of Joists (W_{joist}) = 6,470.75 lb

Sheathing (7/8"):
Thickness of Sheathing (T_{sheath}) = 0.88 in

$$W_{sheath} = T_{sheath} * w * d * W_{wood}$$

Weight of Sheathing (W_{sheath}) = 4,781.17 lb

Total Weight of 1st Floor (without retrofit)= 21.28 Kips

CASE STUDY BUILDING 1 - RETROFIT
BUILDING WEIGHT CALCULATIONS

Weight of Sheathing Retrofit

Sheathing (Wsheath) = 4.78 Kips

Weight of Plywood Retrofit

Thickness of Plywood (T_{ply}): 0.5 in

$W_{ply} = T_{ply} * w * d * W_{wood}$

Plywood Overlay (W_{ply}) = 2.73 Kips

Total Weight of 1st Floor retrofit = 4.78 Kips

Floor 2 (Roof)

Rafters:

Joist Size	Number	Length (ft)	Weight (lb)
2" x 8"	13.5	37.58	1,934.79
2" x 12"	13.5	49.20	3,799.22
1" x 6"	13.5	36.80	710.42
2" x 8"	42	30.17	4,831.49
2" x 10"	42	41.16	8,240.23
1" x 6"	42	31.56	1,895.49
2" x 14"	1	74.33	496.05
Total Rafter Weight (Kips)=			**21.91**

Slate:

Slate Area (A_{slate}) = 4,611.12 ft^2

Weight Slate (W_{slate}) = 49.8 lb/ft^3

Thickness Slate (T_s) = 3 in

$W_{roof} = A_{slate} * T_s * W_{slate}$

Weight of Roof (W_{roof}) = 57.41 Kips

Total Weight of Roof = 79.32 Kips

CASE STUDY BUILDING 1 - RETROFIT
BUILDING WEIGHT CALCULATIONS

Walls:

Masonry Weight (W_m) = 49.8 lb/ft^3
Thickness (T_m) = 13 in

Portion of Walls contributing to Floor 1

In-Plane
$W_{IP1} = 1/2*(h_1 + h_2)*(2*w)*T_m*W_m$
W_{IP1} = 41.50 Kips

Out-of-Plane
$W_{OOP1} = 1/2*(h_1 + h_2)*(2*d)*T_m*W_m$
W_{OOP1} = 87.13 Kips

Portion of Walls contributing to Floor 2

In-Plane
$W_{IP2} = 1/2*(h_2)*(2*w)*T_m*W_m$
W_{IP2} = 17.90 Kips

Out-of-Plane
$W_{OOP2} = 1/2*(h_2)*(2*d)*T_m*W_m$
W_{OOP2} = 37.59 Kips

APPENDIX C

LSP CALCULATIONS FOR EXISTING CASE STUDY BUILDING 2

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)		Floor 2 (Kips)	
Beams	29.56		17.05	
Joists	10.97		12.45	
Concrete	122.31		-	
Wood Sheathing	-		9.24	
IP Walls (W_{IPX})	69.05		33.37	
OOP Walls (W_{OOPX})	131.49		63.55	
Superimposed	(0.02 Ksf)	78.28	(0.01 Ksf)	39.14
Total Dead (W_{Fx})	441.65		174.80	
Live	(0.04 Ksf)	156.55	(0.012 Ksf)	46.97
Snow			(0.022 Ksf)	86.10

2.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

2.6.1.4 Adjustment for Site Class C

From Table 2-13 — From Table 2-14

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_aS_s =$ 20.72 % (Eq 2-4)

$S_{x1} = F_vS_1 =$ 9.04 % (Eq 2-5)

2.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 2-15. For $\beta = 5\%$,

$B_s =$ 1.0 — $T_o = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 2-10)

$B_1 =$ 1.0 — $T_o =$ 0.4364 sec

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < 0.2T_o$	$S_a = (S_{XS}/B_X)*(0.4+3T/T_o)$	(Eq 2-8)
$0.2T_o < T < T_o$	$S_a = S_{XS}/B_S$	(Fig 2-1)
$T_o < T$	$S_a = S_{X1}/B_1T$	(Eq 2-9)

2.11.2 P-Δ Effects

$\theta = P\delta/(Vh)$ (Eq 2-14)

Ex. $P_1 = W_{F1} + (10\text{ psf})*w*d$
$V =$ Force at each floor from 3.3.1.3(b)
$\delta =$ Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
$P_1 =$	655.6	Kips	$P_2 =$	174.8	Kips
$\delta =$	0.001	ft	$\delta =$	0.064	ft
$V =$	95.5	Kips	$V =$	73.2	Kips
$h =$	15.81	ft	$h =$	30.60	ft

$\theta_1 =$ 0.000217
$\theta_2 =$ 0.004992

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1} =$ 1.0

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-2)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-3)

where:

$Q_D =$	0.00	= dead load effect
$Q_L =$	0.00	= live load effect (25% design live load)
$Q_S =$	0.00	= 70% full snow load, not < 20% full design snow load

$Q_G =$ 0.00 Kips
None of the above loads create lateral response

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.2 Modeling and Analysis Considerations
Method 3 - Period Determination -
One story building with single span flexible diaphragm

$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5}$ (Eq 3-5)

Δ_w from SAP Analysis of wall relative to ground
$\Delta_w =$ 0.105 in
Δ_d found using Eq 8-5 and lateral load equal to the weight tributary to the diaphragm
$\Delta_d = vd^4/(G_d w^3)$ (Eq 8-5)

$\Delta_w =$ 0.105 in
$\Delta_d =$ 2.319 in

$vy = ((h_2/2)*(d/2)*120\ lb/ft^3) + W_{F2}/2)/w$
vy = 778 lb/ft

T = 0.4375 sec

3.3.1.3 Determination of Actions & Deformations
A) Pseudo Lateral Load

$V = C_1C_2C_3S_aW$ (Eq 3-6)
Ex. $W = W_{F1} + S + (10psf)*w*d$

$C_1 =$ 1.0
$C_2 =$ 1.1
$C_3 =$ 1.00
$S_a =$ 0.2067
(interpolated according to 2.6.1.5)
W = 741.7 Kips

V = 168.67 Kips

B) Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-7)
$C_{VX} = w_xh_x^k/(\Sigma w_ih_i^k)$ (Eq 3-8)

where:

$w_1 =$	441.65	Kips	$w_2 =$	174.80	Kips
$h_1 =$	15.813	ft	$h_2 =$	30.604	ft
$k =$	1		$k =$	1	
$\Sigma w_ih_i^k =$	12,333.1		$\Sigma w_ih_i^k =$	12,333.1	
$Cv_1 =$	0.5662		$Cv_2 =$	0.4338	

$F_1 =$ 95.51 Kips
$F_2 =$ 73.16 Kips

C) Horizontal Distribution of Seismic Forces
Distributed according to distribution of mass at floor level

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

D) Floor Diaphragms

$F_{PX} = (C_1C_2C_3)^{-1} * \Sigma(F_i * (w_x/\Sigma w_i))$ (Eq 3-9)

w_i = portion of total building weight assigned to floor level i

$C_1C_2C_3 =$	1.100	
$F_1 =$	95.51	Kips
$F_2 =$	73.16	Kips
$w_1 =$	441.65	Kips
$w_2 =$	174.80	Kips
$\Sigma w_i =$	616.45	Kips

$F_{p1} =$ 109.86 Kips

$F_{p2} =$ 43.48 Kips

E) Determination of Deformations

8.5.2 Single Straight Sheathed Diaphragms

$\Delta = vd^4/(G_d w^3)$ (Eq 8-5)

$v_2 = F_{p2} * 1000/d/2$

$G_d =$	200,000	lb/in
$d =$	86.33	ft
$w =$	45.33	ft
$v_2 =$	479.56	lb/ft

$\Delta_d =$ 1.430 in

SAP Analysis using a beam with floor dimensions, appropriate material properties and a distributed lateral load, F_{p1}, provides the following midspan displacement:

$\Delta_{cf} =$ 0.0060 in

3.4.2 Acceptance Criteria

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-14)

where:

$Q_G =$ 0.00 = action due to design gravity loads (Kips)

$Q_E =$ 43.48 = action due to design earthquake loads (Kips)

$Q_{UD} =$ 43.48 Kips

3.4.2.1.B Force Controlled Actions

$Q_{UF} = Q_G +/- Q_E/(J\ C_1C_2C_3)$ (Eq 3-16)

$J =$ 1.21

$Q_{UF} =$ 32.74 Kips

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2.2.A Acceptance Criteria for Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-18)

Q_{CE} = Yield Capacity*w/1000

where:

m =	1.5	= single straight sheathing, unchorded L/b<2.0, Life Safety (REF 4) Table 8-1
κ =	0.75	= knowledge factor (REF 4 2.7.2)
Yield Capacity =	120	lb/ft
Q_{CE} =	5.44	= expected strength (Kips)
$m\kappa Q_{CE}$ =	6.12	Kips
Q_{UD} =	43.48	Kips

Does it satisfy criteria??? **No**

8.5.2.3 Deformation Acceptance Criteria

Criteria will largely depend on the allowable deformations for other structural and non-structural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of the diaphragm.

3.4.2.2.B. Acceptance Criteria for Force-Controlled Actions

$\kappa Q_{CL} >= Q_{UF}$

k =	0.75	
Q_{CL} =	10.36	
κQ_{CL} =	7.77	Kips
Q_{UF} =	32.74	Kips

Does this satify criteria??? **No**

2.9.1.1 Demand Capacity Ratio

QUD/QCE = 7.99

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)		Floor 2 (Kips)	
Beams		29.56		17.05
Joists		10.97		12.45
Concrete		122.31		-
Wood Sheathing		-		9.24
IP Walls (W_{IPX})		69.05		33.37
OOP Walls (W_{OOPx})		131.49		63.55
Superimposed	(0.02 Ksf)	78.28	(0.01 Ksf)	39.14
Total Dead (W_{FX})		441.65		174.80
Live (IBC)	(0.04 Ksf)	156.55	(0.012 Ksf)	46.97
Snow (ASCE 7)			(0.022 Ksf)	86.10

1.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered

$S_s =$ 55.00 % =>

$S_1 =$ 17.72 % =>

2/3 BSE 2 Max Considered

$S_s =$ 36.67 %

$S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

1.6.1.4 Adjustment for Site Class C

From Table 1-4 From Table 1-5

$F_a =$ 1.2 $F_v =$ 1.7

$S_{XS} = F_a S_s =$ 20.724 % (Eq 1-4)

$S_{X1} = F_v S_1 =$ 9.044 % (Eq 1-5)

1.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 1-6. For $\beta = 5\%$,

$B_s =$ 1.0 $T_S = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 1-11)

$B_1 =$ 1.0 $T_S =$ 0.4364 sec

$T_o = 0.2 * T_S =$ 0.0873 sec

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < T_o$	$S_a = S_{XS}((5/B_S - 2)*(T/T_S) + 0.4)$	(Eq 1-8)
$T_o < T < T_S$	$S_a = S_{XS}/B_S$	(Eq 1-9)
$T_S < T$	$S_a = S_{X1}/B_1T$	(Eq 1-10)

2.6.7.1 Out of Plane Anchorage to Diaphragms

$F_p = \chi S_{XS} W$ Wall anchor connections shall be considered force-controlled. F_p not less than minimum of 400 lb/ft or 400SXS,

where:

$\chi =$ 1.2 Table 2-4 based on Life Safety

$S_{XS} =$ 0.20724

$W =$ 1.20 = wall weight tributary to the anchor (K)

$F_p =$ 0.892 K

3.0 Analysis Procedures

3.2.4 Classification of Diaphragms

Flexible when the maximum horizontal deformation of the diaphragm along its length is more than twice the average interstory drift of the vertical lateral-force-resisting elements of the story immediately below the diaphragm.

Max horizontal deformation = 3.957

Average interstory drift = 0.003

Is the diaphragm flexible? Yes

3.2.5.1.1 Linear Procedures

$\theta = P\delta/(Vh)$ (Eq 3-2)

$P_1 = W_{F1} + (10psf) *w*d$

$V =$ Force at each floor from 3.3.1.3(b)

$\delta =$ Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
P =	655.6	Kips	P =	174.8	Kips
δ =	0.000	ft	δ =	0.165	ft
V =	30.1	Kips	V =	28.5	Kips
h =	15.81	ft	h =	30.60	ft

$\theta_1 =$ 6.89E-06

$\theta_2 =$ 0.033018

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1} =$ 1.000

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-3)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-4)

where:

where:	$Q_D =$	0.00	= dead load effect (Kips)
	$Q_L =$	0.00	= live load effect (25% design live load) (Kips)
	$Q_S =$	0.00	= 70% full snow load, greater than 20% full design snow load

None of the above loads create lateral response

$Q_G =$ 0.00 Kips

3.3.1.2 Period Determination

Method 3 - URM Buildings with single span flexible diaphragms,< 6 stories

$T = (0.078\Delta_d)^{0.5}$ (Eq 3-9) Where Δ_d found using Eq 8-3 and lateral load equal to the weight tributary to the diaphragm

$\Delta_d =$ 16.786 in $_y = v_y d/(2G_d)$

$vy = ((h_2/2)*(d/2)*120\ lb/ft^3) + W_{F2}/2)/w$

T = 1.1442 sec vy = 778 K/ft

3.3.1.3 Determination of Actions & Deformations

3.3.1.3.1 Pseudo Lateral Load

$V = C_1C_2C_3C_mS_aW$

Ex. $W = W_{F1} + S + (10psf)*w*d$

V = 58.62 Kips

$C_1 =$ 1.0
$C_2 =$ 1.0
$C_3 =$ 1.0
$C_m =$ 1.0
$S_a =$ 0.0790
(interpolated according to 1.6.1.5.1)
W = 741.7 Kips

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.3.2 Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-11)

$C_{VX} = w_x h_x^k/(\Sigma w_i h_i^k)$ (Eq 3-12)

where:

			Ex. $w_1 = W_{F1}$		
$w_1 =$	441.65	Kips	$w_2 =$	174.80	Kips
$h_1 =$	15.81	ft	$h_2 =$	30.60	ft
$k =$	1.322122		$k =$	1.3221217	
$\Sigma w_i h_i^k =$	33097.75		$\Sigma w_i h_i^k =$	33097.75	
$Cv_1 =$	0.5135		$Cv_2 =$	0.4865	
$F_1 =$	30.10	Kips			
$F_2 =$	28.52	Kips			

3.3.1.3.3 Horizontal Distribution of Seismic Forces

Distributed according to distribution of mass at floor level; however, none is given to out-of-plane walls.

3.3.1.3.4 Floor Diaphragms

$F_{px} = \Sigma F_i(w_x/\Sigma w_i)$ (Eq 3-13)

w_i = portion of total building weight assigned to floor level i

$F_1 =$	30.10	Kips
$F_2 =$	28.52	Kips
$w_1 =$	441.65	Kips
$w_2 =$	174.80	Kips
$\Sigma w_i =$	616.45	Kips

$F_{p1} =$	42.00	Kips	Concrete Diaphragm
$F_{p2} =$	16.62	Kips	Wood Diaphragm

3.3.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms

1. For each span of the building and at each level, calculate period from Eq 3-9.
2. Using Eq 3-10, calculate pseudo lateral load for each span.
3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic-resisting elements using tributary loads.
4. Diaphragm forces for evaluation of diaphragms shall be determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shape.
5. Diaphragm deflection shall not exceed 6 inches for this method of distribution of pseudo lateral loads to be applicable.

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

8.6.3.1 Single Straight Sheathing - Determination of Deformations

$\Delta_y = \nu_y d/(2G_d)$ (Eq 8-3)

		G_d =	2,000 lb/in
		d =	86.33 ft
Δ_d =	3.957 in	ν_{y2} =	183.34 lb/ft

** not to exceed 6 in

SAP Analysis using a beam with floor dimensions, appropriate material properties and a distributed lateral load, F_{p1}, provides the following midspan displacement:

Δ_{cf} = 0.0001 in

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2 Acceptance Criteria for Linear Procedures

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-18)

where:

$Q_G =$	0.00	= action due to design gravity loads (Kips)
$Q_E =$	16.62	= action due to design eq loads (Kips)
$Q_{UD} =$	16.62	Kips

3.4.2.2.1 Acceptance Criteria for Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-20)

$Q_{CE} =$ Yield Capacity*w/1000

where:

m =	1.5	= single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-3
$\kappa =$	0.75	= knowledge factor
Yield Capacity =	120	lb/ft
$Q_{CE} =$	5.44	= expected strength (Kips)

$m\kappa Q_{CE} =$	6.12	Kips
$Q_{UD} =$	16.62	Kips

Does it satisfy criteria??? **No**

3.4.2.1.2 Force Controlled Actions

$Q_{UF} = Q_G +/- Q_E / (C_1C_2C_3J)$

J =	1.5	(moderate seismicity)
$C_1C_2C_3 =$	1.0	
$Q_E =$	16.62	

$Q_{UF} =$ 11.08 Kips

CASE STUDY BUILDING 2
ANALYZED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2.2.2 Acceptance Criteria for Force-Controlled Actions

$\kappa\, Q_{CL} >= Q_{UF}$

$k =$ 0.75
$Q_{CL} =$ 10.36

$\kappa\, Q_{CL} =$ 7.77 Kips
$Q_{UF} =$ 11.08 Kips

Does this satify criteria??? No

8.5.2.3 Criteria will largely depend on the allowable deformations for other structural and non-structuralcomponents and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of diaphragm.

Applicability of LSP for Case Study Building 1

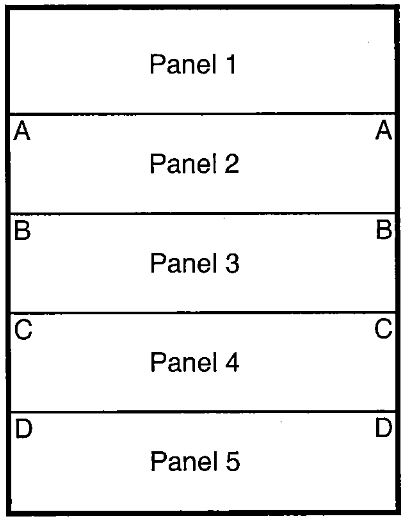
2.4.1.1 Demand Capacity Ratio

$Q_{UD}/Q_{CE} =$ 3.06

CASE STUDY BUILDING 2
BUILDING WEIGHT CALCULATIONS

Overall Dimensions:

Height (h_1):	15.813	ft
Height (h_2):	14.792	ft
Width (w):	45.333	ft
Depth (d):	86.333	ft
Panel 1:	17.583	ft
Panel 2:	17.167	ft
Panel 3:	17.167	ft
Panel 4:	16.333	ft
Panel 5:	18.083	ft



Aspect Ratio: 1.90

Floor 1:

Beams:

AA	182	lb/ft
BB	170	lb/ft
CC	150	lb/ft
DD	150	lb/ft

Weight of Beams = (AA+BB+CC+DD)*w
Weight of Beams = 29,557.33 lb

Weight of Joists (124,126,102,104) = 5.0 lb/ft (REF 5)

Weight of Joists

	Type	Number	Length (ft)	Weight (lb)
Panel 1	126	13	228.58	1142.92
	124	11	193.42	967.08
Panel 2	124	14	240.33	1201.67
	126	13	223.17	1115.83
Panel 3	124	10	171.67	858.33
	126	15	257.50	1287.50
Panel 4	124	24	392.00	1960.00
	126	0	0.00	0.00
Panel 5	124	25	452.08	2260.42
	126	2	36.17	180.83
			Total Weight of Joists =	10974.58

Concrete Slab:

Thickness =	2.5	in
Area =	3913.8	ft^2
Weight =	150	lb/ft^3
Total Weight of Slab =	122.31	Kips
Total Weight of Floor 1 =	**162.84**	**Kips**

CASE STUDY BUILDING 2
BUILDING WEIGHT CALCULATIONS

Floor 2:

Beams

AA	94	lb/ft
BB	94	lb/ft
CC	94	lb/ft
DD	94	lb/ft
Weight of Beams =	17,045.33	lb

Weight of Joists

	Type	Number	Length (ft)	Weight (lb)
Panel 1	102	28	492.33	2,461.67
	104	0	0.00	0.00
Panel 2	102	26	446.33	2,231.67
	104	2	34.33	171.67
Panel 3	102	28	480.67	2,403.33
	104	0	0.00	0.00
Panel 4	102	26	424.67	2,123.33
	104	2	32.67	163.33
Panel 5	102	32	578.67	2,893.33
	104	0	0.00	0.00
			Total Weight of Joists (lb) =	12,448.33

Sheathing:

Thickness (T_s):* 0.75 in *dim. different on layout & detail

Nailers: - taken from detail dim

Width (W_N):* 3.625 in

Depth (D_N):* 1.625 in

$L_N = d * 12$

Approx. Length (L_N): 1,036.0 in

Weight of Southern
Pine Wood (W_{wood}): 0.01986 lb/in^3 (REF 1)

$W_{sheath} = W_{wood} * T_s * w * d$

Weight of Sheathing (W_{sheath}) = 8,395.05 lb

$W_{nailer} = 7 * (W_{wood} * W_N * D_N * L_N)$

Weight of Nailers (W_{nailer}) = 848.44 lb

Total Wood Weight = 9,243.50 lb

Total Weight of Floor 2 = 38.737 Kips

CASE STUDY BUILDING 2
BUILDING WEIGHT CALCULATIONS

Walls:

Masonry Weight (W_m) = 49.8 lb/ft^3
Thickness (T_m) = 12 in

Portion of Walls contributing to Floor 1

In-Plane
$W_{IP1} = 1/2*(h_1 + h_2)*(2*w)*T_m*W_m$
W_{IP1} = 69.05 Kips

Out-of-Plane
$W_{OOP1} = 1/2*(h_1 + h_2)*(2*d)*T_m*W_m$
W_{OOP1} = 131.49 Kips

Portion of Walls contributing to Floor 2

In-Plane
$W_{IP2} = 1/2*(h_2)*(2*w)*T_m*W_m$
W_{IP2} = 33.37 Kips

Out-of-Plane
$W_{OOP2} = 1/2*(h_2)*(2*d)*T_m*W_m$
W_{OOP2} = 63.55 Kips

APPENDIX D

SAMPLE LSP CALCULATION FOR RETROFITTED CASE STUDY BUILDING 2

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)	Floor 2 (Kips)
Beams	29.56	17.05
Joists	10.97	12.45
Concrete	122.31	-
Wood Sheathing	-	9.24
Retrofit	-	5.60
IP Walls (W_{IPX})	69.09	33.39
OOP Walls (W_{OOPX})	131.58	63.60
Superimposed	(0.02 Ksf) 78.28	(0.01 Ksf) 39.14
Total Dead (W_{Fx})	441.78	180.46
Live	(0.04 Ksf) 156.55	(0.012 Ksf) 46.97
Snow		(0.022 Ksf) 86.10

2.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

2.6.1.4 Adjustment for Site Class C

From Table 2-13 — From Table 2-14

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_aS_s =$ 20.72 % (Eq 2-4)

$S_{x1} = F_vS_1 =$ 9.04 % (Eq 2-5)

2.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 2-15

For β = 5%,

$B_s =$ 1.0 — $T_o = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 2-10)

$B_1 =$ 1.0 — $T_o =$ 0.4364 sec

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < 0.2T_o$	$S_a = (S_{XS}/B_X)*(0.4+3T/T_o)$	(Eq 2-8)
$0.2T_o < T < T_o$	$S_a = S_{XS}/B_S$	(Fig 2-1)
$T_o < T$	$S_a = S_{X1}/B_1T$	(Eq 2-9)

2.11.2 P-Δ Effects

$\theta = P\delta/(Vh)$ (Eq 2-14)

Ex. $P_1 = W_{F1} + (10\text{ psf})*w*d$
$V =$ Force at each floor from 3.3.1.3(b)
$\delta =$ Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
$P_1 =$	661.4	Kips	$P_2 =$	180.5	Kips
$\delta =$	0.001	ft	$\delta =$	0.007	ft
$V =$	107.1	Kips	$V =$	84.7	Kips
$h =$	15.81	ft	$h =$	14.79	ft

$\theta_1 =$ 0.000195
$\theta_2 =$ 0.000985

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1} =$ 1.0

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-2)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-3)

where:	$Q_D =$	0.00	= dead load effect
	$Q_L =$	0.00	= live load effect (25% design live load)
	$Q_S =$	0.00	= 70% full snow load, not less than 20% full design snow load

None of the above loads create lateral response

$Q_G =$ 0.00 Kips

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.2 Modeling and Analysis Considerations

Method 3 - Period Determination -
One story building with single span flexible diaphragm

Δ_w from SAP Analysis of wall rel. to ground

$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5}$ (Eq 3-5)

$\Delta_w =$ 0.105 in

Δ_d found using Eq 8-5 and lateral load equal to the weight tributary to the diaphragm

$\Delta_w =$ 0.105 in

$\Delta_d =$ 0.278 in

$_1 = vd^4/(G_d w^3)$ (Eq 8-5)

$vy = ((h_2/2)*(d/2)*120\ lb/ft^3) + W_{F2}/2)/w$

$T =$ 0.1794 sec

$vy =$ 840 lb/ft

3.3.1.3 Determination of Actions & Deformations

A) Pseudo Lateral Load

$V = C_1C_2C_3S_aW$ (Eq 3-6)

Ex. $W = W_{F1} + S + (10psf)*w*d$

$V =$ 191.83 Kips

$C_1 =$ 1.0
$C_2 =$ 1.3
$C_3 =$ 1.00
$S_a =$ 0.2072
(interpolated according to 2.6.1.5)
$W =$ 747.5 Kips

B) Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-7)

$C_{VX} = w_x h_x^k/(\Sigma w_i h_i^k)$ (Eq 3-8)

Ex. $w_1 = W_{F1}$

where:

$w_1 =$	441.78	Kips	$w_2 =$	180.46	Kips
$h_1 =$	15.813	ft	$h_2 =$	30.604	ft
$k =$	1		$k =$	1	
$\Sigma w_i h_i^k =$	12,508.6		$\Sigma w_i h_i^k =$	12,508.6	
$Cv_1 =$	0.5585		$Cv_2 =$	0.4415	

$F_1 =$ 107.13 Kips

$F_2 =$ 84.70 Kips

C) Horizontal Distribution of Seismic Forces

Distributed according to distribution of mass at floor level

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

D) Floor Diaphragms

$F_{PX} = (C_1C_2C_3)^{-1}*\Sigma(F_i*(w_x/\Sigma w_i))$ (Eq 3-9)

w_i = portion of total building weight assigned to floor level i

$C_1C_2C_3$ =	1.238	
F_1 =	107.13	Kips
F_2 =	84.70	Kips
w_1 =	441.78	Kips
w_2 =	180.46	Kips
Σw_i =	622.25	Kips

F_{p1} = 109.98 Kips
F_{p2} = 44.93 Kips

E) Determination of Deformations
8.5.2 Single Straight Sheathed Diaphragms

$\Delta = vd^4/(G_dw^3)$ (Eq 8-5)
$v_2 = F_{p2}*1000/d/2$

G_d =	1,800,000	lb/in
d =	86.33	ft
w =	45.33	ft
v_2 =	495.51	lb/ft

Δ_d = 0.164 in

SAP Analysis using a beam with floor dimensions, appropriate material properties and a distributed lateral load, F_{p1}, provides the following midspan displacement:

Δ_{cf} = 0.0060 in

3.4.2 Acceptance Criteria

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-14)

where:

Q_G =	0.00	= action due to design gravity loads (Kips)
Q_E =	44.93	= action due to design earthquake loads (Kips)
Q_{UD} =	44.93	Kips

3.4.2.1.B Force Controlled Actions

$Q_{UF} = Q_G +/- Q_E/(C_1C_2C_3)$ (Eq 3-16)

Q_{UF} = 36.28 Kips

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 273
NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.4.2.2.A Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-18)

Q_{CE} = Yield Capacity*w/1000

where:

m = 3.0 = single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-1

κ = 0.75 = knowledge factor (REF 4 2.7.2)

Yield Capacity = 960 lb/ft

Q_{CE} = 43.52 = expected strength (Kips)

$m\kappa Q_{CE}$ = 97.92 Kips

Q_{UD} = 44.93 Kips

Does it satisfy criteria??? **Yes**

8.5.2.3 Deformation Acceptance Criteria

Criteria will largely depend on the allowable deformations for other structural and non-structural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of the diaphragm.

2.9.1.1 Demand Capacity Ratio

QUD/QCE = 1.03

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Load Summary

	Floor 1 (Kips)	Floor 2 (Kips)
Beams	29.56	17.05
Joists	10.97	12.45
Concrete	122.31	-
Wood Sheathing	-	9.24
Retrofit	-	5.60
IP Walls (W_{IPX})	69.09	33.39
OOP Walls (W_{OOPx})	131.58	63.60
Superimposed	(0.02 Ksf) 78.28	(0.01 Ksf) 39.14
Total Dead (W_{FX})	441.78	180.46
Live (IBC)	(0.04 Ksf) 156.55	(0.012 Ksf) 46.97
Snow (ASCE 7)		(0.022 Ksf) 86.10

1.6.1.2 Response Acceleration Parameters (FEMA Maps)

a) 10% / 50 year (BSE 1)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

b) BSE 2 Max Considered — 2/3 BSE 2 Max Considered

$S_s =$ 55.00 % => $S_s =$ 36.67 %

$S_1 =$ 17.72 % => $S_1 =$ 11.81 %

Use Smaller of (a) or 2/3(b)

$S_s =$ 17.27 %

$S_1 =$ 5.32 %

1.6.1.4 Adjustment for Site Class C

From Table 1-4 — From Table 1-5

$F_a =$ 1.2 — $F_v =$ 1.7

$S_{xs} = F_aS_s =$ 20.724 % (Eq 1-4)

$S_{x1} = F_vS_1 =$ 9.044 % (Eq 1-5)

1.6.1.5 General Response Spectrum

Damping coefficient, B_1 & B_S, given as a function of effective damping, β, in Table 1-6

For $\beta = 5\%$,

$B_s =$ 1.0 — $T_S = (S_{X1}B_S)/(S_{XS}B_1)$ (Eq 1-11)

$B_1 =$ 1.0 — $T_S =$ 0.4364 sec

$T_o = 0.2 * T_S =$ 0.0873 sec

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

Spectral Response Acceleration, S_a:

$0 < T < T_o$	$S_a = S_{XS}((5/B_S - 2)*(T/T_S) + 0.4)$	(Eq 1-8)
$T_o < T < T_S$	$S_a = S_{XS}/B_S$	(Eq 1-9)
$T_S < T$	$S_a = S_{X1}/B_1T$	(Eq 1-10)

2.6.7.1 Out of Plane Anchorage to Diaphragms

$F_p = \chi S_{XS} W$ Wall anchor connections shall be considered force-controlled
F_p not less than minimum of 400 lb/ft or $400S_{XS}$

where:

χ =	1.2	Table 2-4 based on Life Safety
S_{XS} =	0.20724	
W =	2.18	= wall weight tributary to the anchor (K)
F_p =	0.541	K/ft

3.0 Analysis Procedures

3.2.4 Classification of Diaphragms

Flexible when the maximum horizonal deformation of the diaphragm along its length is more than twice the average interstory drift of the vertical lateral-force-resisting elements of the story immediately below the diaphragm.

Max horizontal deformation =	1.001
Average interstory drift =	0.003
Is the diaphragm flexible?	**Yes**

3.2.5.1.1 Linear Procedures

$\theta = P\delta/(Vh)$ (Eq 3-2)

Ex. $P_1 = W_{F1}$ + (10psf) *w*d
V = Force at each floor from 3.3.1.3(b)
δ = Lateral drift in story i at center of rigidity

Floor 1:			Floor 2:		
P =	661.4	Kips	P =	180.5	Kips
δ =	0.001	ft	δ =	0.042	ft
V =	72.9	Kips	V =	57.6	Kips
h =	15.81	ft	h =	14.79	ft

θ_1 = 0.000286906

θ_2 = 0.008840592

$C_3 = 1+5(\theta_{max} - 0.1)*T^{-1}$ = 1.000

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.2.8 Component Gravity Loads and Load Combinations

When gravity & seismic loads are additive,
$Q_G = 1.1(Q_D + Q_L + Q_S)$ (Eq 3-3)

When effects of gravity counteract seismic loads,
$Q_G = 0.9Q_D$ (Eq 3-4)
where:

$Q_D =$	0.00	= dead load effect (Kips)
$Q_L =$	0.00	= live load effect (25% design live load) (Kips)
$Q_S =$	0.00	= 70% full snow load, greater than 20% full design snow lo:

None of the above loads create lateral response

$Q_G =$ 0.00 Kips

3.3.1.2 Period Determination

Method 3 - URM Buildings with single span flexible diaphragms, less than 6 stories

$T = (0.078\Delta_d)^{0.5}$ (Eq 3-9) Where Δ_d found using Eq 8-3 and lateral load equal to the weight tributary to the diaphragm

$\Delta_d =$ 2.014 in

$\Delta_y = v_y d/(2G_d)$

$vy = ((h_2/2)*(d/2)*120\ lb/ft^3) + W_{F2}/2)/w$

$T =$ 0.3963 sec

$vy =$ 840 K/ft

3.3.1.3 Determination of Actions & Deformations

3.3.1.3.1 Pseudo Lateral Load

$V = C_1C_2C_3C_mS_aW$

Ex. $W = W_{F1} + S + (10psf)*w*d$

$V =$ 130.52 Kips

$C_1 =$ 0.8
$C_2 =$ 1.0
$C_3 =$ 1.0
$C_m =$ 1.0
$S_a =$ 0.2072
(interpolated according to 1.6.1.5.1)
$W =$ 747.5 Kips

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

3.3.1.3.2 Vertical Distribution of Seismic Forces

$F_x = C_{vx}V$ (Eq 3-11)

$C_{VX} = w_x h_x^k / (\Sigma w_i h_i^k)$ (Eq 3-12)

Ex. $w_1 = W_{F1}$

where:

$w_1 =$	441.78	Kips	$w_2 =$	180.46	Kips
$h_1 =$	15.81	ft	$h_2 =$	30.60	ft
$k =$	1		$k =$	1	
$\Sigma w_i h_i^k =$	12508.56		$\Sigma w_i h_i^k =$	12508.56	
$Cv_1 =$	0.5585		$Cv_2 =$	0.4415	
$F_1 =$	72.89	Kips			
$F_2 =$	57.63	Kips			

3.3.1.3.3 Horizontal Distribution of Seismic Forces

Distributed according to distribution of mass at floor level; however, none is given to out-of-plane walls.

3.3.1.3.4 Floor Diaphragms

$F_{px} = \Sigma F_i(w_x/\Sigma w_i)$ (Eq 3-13)

w_i = portion of total building weight assigned to floor level i

$F_1 =$	72.89	Kips
$F_2 =$	57.63	Kips
$w_1 =$	441.78	Kips
$w_2 =$	180.46	Kips
$\Sigma w_i =$	622.25	Kips

$F_{p1} =$	92.67	Kips
$F_{p2} =$	37.85	Kips

3.3.1.3.5 Distribution of Seismic Forces for Unreinforced Masonry Buildings with Flexible Diaphragms

1. For each span of the building and at each level, calculate period from Eq 3-9.
2. Using Eq 3-10, calculate pseudo lateral load for each span.
3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic elements using tributary loads.
4. Diaphragm forces for evaluation of diaphragms shall be determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shap
5. Diaphragm deflection shall not exceed 6 inches for this method of distribution of pseudo lateral loads to be applicable.

CASE STUDY BUILDING 2 - RETROFIT
REHABILITATED ACCORDING TO FEMA 356
PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS

8.6.3.1 Single Straight Sheathing - Determination of Deformations

$\Delta_y = \nu_y d/(2G_d)$ (Eq 8-3)

$G_d =$ 18,000 lb/in
$d =$ 86.33 ft
$\nu_{y2} =$ 417.50 lb/ft

$\Delta_d =$ 1.001 in

** not to exceed 6 in

SAP Analysis using a beam with floor dimensions, appropriate material properties and a distributed lateral load, F_{p1}, provides the following midspan displacement:

$\Delta_{cf} =$ 0.0060 in

3.4.2 Acceptance Criteria for Linear Procedures

3.4.2.1.A Design -- Deformation-Controlled Actions

$Q_{UD} = Q_G +/- Q_E$ (Eq 3-18)

where:

$Q_G =$ 0.00 = action due to design gravity loads (Kips)
$Q_E =$ 37.85 = action due to design earthquake loads (Kips)
$Q_{UD} =$ 37.85 Kips

3.4.2.2.A Deformation Controlled Actions

$m\kappa Q_{CE} >= Q_{UD}$ (Eq 3-20)

$Q_{CE} =$ Yield Capacity*w/1000

where:

m = 3.0 = single straight sheating, unchorded L/b<2.0, Life Safety (REF 4) Table 8-3
$\kappa =$ 0.75 = knowledge factor
Yield Capacity = 960 lb/ft
$Q_{CE} =$ 43.52 = expected strength (Kips)

$m\kappa Q_{CE} =$ 97.92 Kips
$Q_{UD} =$ 37.85 Kips

Does it satisfy criteria??? **Yes**

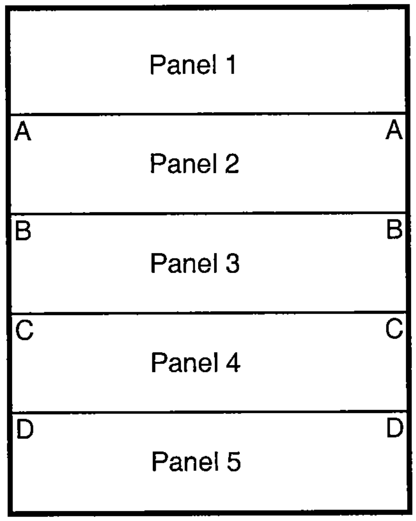
2.4.1.1 Demand Capacity Ratio

QUD/QCE = 0.87

CASE STUDY BUILDING 2 - RETROFIT
BUILDING WEIGHT CALCULATIONS

Overall Dimensions:

Height (h_1):	15.813	ft
Height (h_2):	14.792	ft
Width (w):	45.333	ft
Depth (d):	86.333	ft
Panel 1:	17.583	ft
Panel 2:	17.167	ft
Panel 3:	17.167	ft
Panel 4:	16.333	ft
Panel 5:	18.083	ft
Aspect Ratio:	1.90	



Floor 1:

Beams:

AA	182	lb/ft
BB	170	lb/ft
CC	150	lb/ft
DD	150	lb/ft

Weight of Beams = (AA+BB+CC+DD)*w
Weight of Beams = 29,557.33 lb

Weight of Joists (124,126,102,104) = 5.0 lb/ft (REF 5)

Weight of Joists

	Type	Number	Length (ft)	Weight (lb)
Panel 1	126	13	228.58	1142.92
	124	11	193.42	967.08
Panel 2	124	14	240.33	1201.67
	126	13	223.17	1115.83
Panel 3	124	10	171.67	858.33
	126	15	257.50	1287.50
Panel 4	124	24	392.00	1960.00
	126	0	0.00	0.00
Panel 5	124	25	452.08	2260.42
	126	2	36.17	180.83
			Total Weight of Joists =	10974.58

Concrete Slab:

Thickness =	2.5	in
Area =	3913.8	ft^2
Weight =	150	lb/ft^3
Total Weight of Slab =	122.31	Kips

Total Weight of Floor 1 = 162.84 Kips

CASE STUDY BUILDING 2 - RETROFIT
BUILDING WEIGHT CALCULATIONS

Floor 2:

Beams

AA	94	lb/ft
BB	94	lb/ft
CC	94	lb/ft
DD	94	lb/ft
Weight of Beams =	17,045.33	lb

Weight of Joists

	Type	Number	Length (ft)	Weight (lb)
Panel 1	102	28	492.33	2,461.67
	104	0	0.00	0.00
Panel 2	102	26	446.33	2,231.67
	104	2	34.33	171.67
Panel 3	102	28	480.67	2,403.33
	104	0	0.00	0.00
Panel 4	102	26	424.67	2,123.33
	104	2	32.67	163.33
Panel 5	102	32	578.67	2,893.33
	104	0	0.00	0.00
			Total Weight of Joists (lb) =	12,448.33

Sheathing:

Thickness (T_s):* 0.75 in *dim different on layout & detail

Nailers: - taken from detail dim

Width (W_N):* 3.625 in

Depth (D_N):* 1.625 in

$L_N = d * 12$

Approx Length (L_N): 1,036.0 in

Weight of Southern Pine Wood (W_{wood}): 0.01986 lb/in^3 (REF 1)

$W_{sheath} = W_{wood} * T_s * w * d$

Weight of Sheathing (W_{sheath}) = 8,395.05 lb

$W_{nailer} = 7 * (W_{wood} * W_N * D_N * L_N)$

Weight of Nailers (W_{nailer}) = 848.44 lb

Total Wood Weight = 9,243.50 lb

Total Weight of Floor 2 without retrofit = 38.74 Kips

CASE STUDY BUILDING 2 - RETROFIT
BUILDING WEIGHT CALCULATIONS

Weight of Sheathing Retrofit

Sheathing (W_{sheath}) = 8.40 Kips

Weight of Plywood Retrofit

Thickness of Plywood (T_{ply}): 0.5 in

$W_{ply} = T_{ply} * w * d * W_{wood}$

Plywood Overlay (W_{ply}) = 5.60 Kips

Total Weight of Floor 2 retrofit = 5.60 Kips

Walls:

Masonry Weight (W_m) = 49.8 lb/ft^3

Thickness (T_m) = 12 in

Portion of Walls contributing to Floor 1

In-Plane

$W_{IP1} = 1/2*(h_1 + h_2)*(2*w)*T_m*W_m$

W_{IP1} = 69.09 Kips

Out-of-Plane

$W_{OOP1} = 1/2*(h_1 + h_2)*(2*d)*T_m*W_m$

W_{OOP1} = 131.58 Kips

Portion of Walls contributing to Floor 2

In-Plane

$W_{IP2} = 1/2*(h_2)*2*w*T_m*W_m$

W_{IP2} = 33.39 Kips

Out-of-Plane

$W_{OOP2} = 1/2*(h_2)*(2*d)*T_m*W_m$

W_{OOP2} = 63.60 Kips

APPENDIX E

SUMMARY OF ALL RETROFIT CALCULATIONS FOR CASE STUDY BUILDINGS

CASE STUDY BUILDING 1
FEMA 273 RETROFIT COMPARISON

Sheathing Retrofits

	Existing: Single Straight Sheathing	Double Straight Unchorded:	Double Straight Chorded:	Diagonal w/ Straight Unchorded	Diagonal w/ Straight Chorded
weight of retrofit		equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing
m	1.5	1.5	2.0	2.0	2.5
Yield Capacity (lb/ft)	120	400	600	625	900
Gd (lb/in)	200,000	700,000	1,500,000	900,000	1,800,000
Building Period (sec)	0.4297	0.2614	0.1936	0.2356	0.1816
Actual, mKQ_{CE} (Kips)	6.12	20.4	40.8	42.5	76.5
Demand, Q_{UD} (Kips)	43.58	45.59	45.59	45.59	45.59

Plywood Retrofits

	Existing: Single Straight Sheathing	Panel Overlay unblocked unchorded	Panel Overlay, unblocked, chorded	Panel Overlay, blocked, unchorded	Panel Overlay, blocked, chorded
weight of retrofit		weight of plywood	weight of plywood	weight of plywood	weight of plywood
m	1.5	2.0	2.5	2.5	3.0
Yield Capacity (lb/ft)	120	300	450	672	960
Gd (lb/in)	200,000	500,000	900,000	700,000	1,800,000
Building Period (sec)	1.1429	0.2977	0.2322	0.2575	0.1794
Actual, mKQ_{CE} (Kips)	6.12	20.4	38.25	57.12	97.92
Demand, Q_{UD} (Kips)	43.58	44.93	44.93	44.93	44.93

CASE STUDY BUILDING 1
FEMA 356 RETROFIT COMPARISON

Sheathing Retrofits

	Existing: Single Straight Sheathing	Double Straight Unchorded:	Double Straight Chorded:	Diagonal w/ Straight Unchorded	Diagonal w/ Straight Chorded
weight of retrofit		equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing
m	1.5	1.5	2.0	2.0	2.5
Yield Capacity (lb/ft)	120	400	600	625	900
Gd (lb/in)	2,000	7,000	15,000	9,000	18,000
Δ_d	2.984	1.556	1.063	1.372	0.970
Base Shear	25.27	46.75	68.44	53.01	74.97
Building Period (sec)	1.1442	0.7892	0.5391	0.696	0.4921
Actual, mKQ_{CE} (Kips)	4.07	13.58	27.15	28.28	50.91
Demand, Q_{UD} (Kips)	11.37	20.75	30.37	23.53	33.27

Plywood Retrofits

	Existing: Single Straight Sheathing	Panel Overlay unblocked unchorded	Panel Overlay, unblocked, chorded	Panel Overlay, blocked, unchorded	Panel Overlay, blocked, chorded
weight of retrofit		weight of plywood	weight of plywood	weight of plywood	weight of plywood
m	1.5	2.0	2.5	2.5	3.0
Yield Capacity (lb/ft)	120	300	450	672	960
Gd (lb/in)	2,000	5,000	9,000	7,000	18,000
Δd	2.98	1.86	1.39	1.57	0.98
Base Shear	25.27	39.70	53.26	46.97	75.32
Building Period (sec)	1.1442	0.9247	0.6892	0.7815	0.4874
Actual, mKQ_{CE} (Kips)	4.07	13.58	25.45	38.01	65.16
Demand, Q_{UD} (Kips)	11.37	17.72	23.78	20.97	33.63

CASE STUDY BUILDING 2
FEMA 273 RETROFIT COMPARISON

Sheathing Retrofits

	Existing: Single Straight Sheathing	Double Straight Unchorded:	Double Straight Chorded:	Diagonal w/ Straight Unchorded	Diagonal w/ Straight Chorded
weight of retrofit		equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing
m	1.5	1.5	2.0	2.0	2.5
Yield Capacity (lb/ft)	120	400	600	625	900
Gd (lb/in)	200,000	700,000	1,500,000	900,000	1,800,000
Building Period (sec)	0.4297	0.2614	0.1936	0.2356	0.1816
Actual, mKQ_{CE} (Kips)	6.12	20.4	40.8	42.5	76.5
Demand, Q_{UD} (Kips)	43.58	45.59	45.59	45.59	45.59

Plywood Retrofits

	Existing: Single Straight Sheathing	Panel Overlay unblocked unchorded	Panel Overlay, unblocked, chorded	Panel Overlay, blocked, unchorded	Panel Overlay, blocked, chorded
weight of retrofit		weight of plywood	weight of plywood	weight of plywood	weight of plywood
m	1.5	2.0	2.5	2.5	3.0
Yield Capacity (lb/ft)	120	300	450	672	960
Gd (lb/in)	200,000	500,000	900,000	700,000	1,800,000
Building Period (sec)	1.1429	0.2977	0.2322	0.2575	0.1794
Actual, mKQ_{CE} (Kips)	6.12	20.4	38.25	57.12	97.92
Demand, Q_{UD} (Kips)	43.58	44.93	44.93	44.93	44.93

CASE STUDY BUILDING 2
FEMA 356 RETROFIT COMPARISON

Sheathing Retrofits

	Existing: Single Straight Sheathing	Double Straight Unchorded:	Double Straight Chorded:	Diagonal w/ Straight Unchorded	Diagonal w/ Straight Chorded
weight of retrofit		equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing	equals weight of original sheathing
m	1.5	1.5	2.0	2.0	2.5
Yield Capacity (lb/ft)	120	400	600	625	900
Gd (lb/in)	2,000	7,000	15,000	9,000	18,000
Δ_d	3.957	2.091	1.428	1.844	1.010
Base Shear	58.62	104.86	153.50	118.90	130.25
Building Period (sec)	1.1442	0.6471	0.4421	0.5707	0.4035
Actual, mKQ_{CE} (Kips)	6.12	20.4	40.8	42.5	76.5
Demand, Q_{UD} (Kips)	16.62	30.74	45.01	34.86	38.19

Plywood Retrofits

	Existing: Single Straight Sheathing	Panel Overlay unblocked unchorded	Panel Overlay, unblocked, chorded	Panel Overlay, blocked, unchorded	Panel Overlay, blocked, chorded
weight of retrofit		weight of plywood	weight of plywood	weight of plywood	weight of plywood
m	1.5	2.0	2.5	2.5	3.0
Yield Capacity (lb/ft)	120	300	450	672	960
Gd (lb/in)	2,000	5,000	9,000	7,000	18,000
Δd	3.96	2.48	1.85	2.10	1.00
Base Shear	58.62	89.90	120.62	106.37	130.52
Building Period (sec)	1.1442	0.752	0.5605	0.6355	0.3963
Actual, mKQ_{CE} (Kips)	6.12	20.4	38.25	57.12	97.92
Demand, Q_{UD} (Kips)	16.62	26.07	34.98	30.85	37.85