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OAK RIDGE Y-12 PLANT

MARTIN MARIETTA

ANALYSIS OF DYNAMIC TESTING PERFORMED ON STRUCTURAL CLAY TILE INFILLED FRAMES

Prepared for the
Center for Natural Phenomena Engineering

December 18, 1994

Oak Ridge Y-12 Plant
Oak Ridge, Tennessee 37831
managed by
Martin Marietta Energy Systems, Inc.
for the
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MANAGED BY
MARTIN MARIETTA ENERGY SYSTEMS, INC.
FOR THE UNITED STATES
DEPARTMENT OF ENERGY
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**Central Engineering Services
Martin Marietta Energy Systems, Inc.**

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MASTER #1

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**Analysis of Dynamic Testing Performed on Structural
Clay Tile Infilled Frames**

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Joele Johnston Fowler

December, 1994

DEDICATION

This thesis is dedicated to my mom Audrey Fowler and the rest of my family for all the love and support they have given me throughout my educational pursuits.

}

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ABSTRACT

The behavior of two structural clay tile infilled frames subjected to dynamic loading is investigated. The testing was performed by USACERL using a biaxial shake table machine on which two framed infills were spaced nine feet apart and connected by steel trusses and an eight inch concrete roof slab. The infills were composed of structural clay tile block which were laid with the cores horizontal. The specimen was loaded in both the out-of-plane and in-plane directions using a site specific time history record. The testing focused on determining frame and panel load-deflection behavior, acceleration amplification, and frequency degradation characteristics.

The out-of-plane tests resulted in little degradation of frequency which means there was little loss of stiffness. There was no evidence of the infill "walking-out" of the steel frame; in fact the infill still had substantial stability after completion of the out-of-plane tests. As a result of the gradual increase in ground motion in the in-plane testing, the stiffness of the specimen gradually decreased. Strength and stiffness characteristics obtained from the dynamic testing were comparable to results and behavior seen in static tests. Degradation in the panel was much more rapid under the stronger ground motions which were produced during the sine sweep tests.

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Chapter 1

INTRODUCTION

1.1 General

The structural characteristics and behavior of unreinforced masonry infills have been an area of considerable research over the past couple of decades, and considerable current interest as evidenced by two recent workshops focusing on infill construction (MMES, 1993; Abrams, 1994). There is an advantage in building a steel frame structure with masonry infills in that the masonry adds significantly to the stiffness of the structure while the steel frame adds ductility to the system and confines the masonry panel. The behavior and stability of these types of structures under seismic conditions have been of particular interest.

Infill construction remains very popular, particularly in the low to moderate seismic zones of the eastern/central United States. Infill walls may have both beneficial and detrimental affects on the structural behavior under seismic loading. Infill walls increase the stiffness of a building, shift the structure's center of stiffness and affect the forces in the framing possibly causing premature failure of frame members. Infills can also have a positive influence on the structural behavior, as evidenced by the 1985 Mexico city earthquake (Klingner et al., 1987) and the 1992 Cairo, Egypt earthquake (Adham, 1994). Here, they served to

increase the stiffness of flexible frame buildings on soft soils, and provided a redundant load path for both the horizontal and vertical loads.

Many buildings were constructed using clay tile masonry during the World War II era. Clay tile was also extensively used in the high seismic regions of San Francisco and Los Angeles. The clay tile infills in these buildings were considered to be non-load bearing elements. At the time there was not enough information available on the load carrying capability of clay tile infills; therefore, engineers were unable to account for any added structural strength provided by the masonry panels.

1.2 Problem Statement

Static testing, in and out of plane, is presently being performed on clay tile infilled frames to obtain more information on the structural characteristics of these types of systems (Barclay, 1993; Flanagan, 1994). There is a need to correlate static tests results with the results obtained from dynamic testing in order to better understand the seismic performance of these types of structures.

1.3 Objectives

The main objectives of this study were to reduce and analyze data obtained from biaxial shake table tests which were performed by the United States Army Construction

Engineering Research Laboratories (USACERL) in August of 1993. Both in-plane and out-of-plane shake table tests were performed. Analyses were performed to determine frame and panel load-deformation behavior, along with acceleration amplification and frequency degradation characteristics.

Another objective of this study was to conduct a literature survey in which other shake table tests on infilled frames would be compared with the research performed by USACERL. This can be used to determine the characteristics of unreinforced infilled masonry under dynamic loads.

1.4 Scope

The focus of this study was on acceleration, frequency, and load-deflection behavior of the specimen under dynamic loading conditions with the panel oriented in both the out-of-plane and in-plane directions. Chapter 2 provides information on how the tests were set up and the types of instrumentation used. The testing procedures used along with specifics on how the tests were performed are given in chapter 3. Chapters 4, 5 and 6 present the analysis of data obtained from the out-of-plane, in-plane, and sine sweep tests respectively. Chapter 7 is a literature survey which compares this study to previous tests performed on clay tile masonry. Chapter 8 presents conclusions and comments on the results obtained from this study.

Chapter 2

TEST SETUP

2.1 Introduction

This chapter describes the procedures and preparations involved in setting up the specimen for testing. The preparations include the construction and design of the specimen frame, infill and roof slab, along with instrumentation locations (Gamble, 1993). Figure 2.1 shows a three dimensional view of the test setup. The structure consists of a one-story, single bay system constructed with two steel frames, infilled with structural clay tile, and spaced approximately 9' apart. An 8" concrete roof slab is bolted to the top of the steel frames, and a truss bracing system is connected between the two frames for added strength. Figure 2.2 shows the orientation of the specimens with respect to the reference frame and table motion, during both the in-plane and out-of-plane tests.

2.2 Steel Frame Design

The specimen frame shown in figure 2.3 consists of wide flange shapes which are connected to form a 8' high by 11' long wall (nominally). Both of the frames consist of two W10 x 30 columns and a W12 x 40 beam. The beam to column connections are framed connections using double clip angles.

Figure 2.1: Three dimensional view of test setup
(Gamble, 1993)

Figure 2.2: Plan view showing test orientations

Figure 2.3: Out-of-plane elevation view of bare frame

Slip critical, A325 bolts were used and were tightened using the turn of nut method. The columns were welded to 3/4" x 12" x 12" plates which were bolted on top of 12" x 8" x 1/2" steel tubing using a 1 1/4" diameter bolts. The tubing was then bolted to the shake table using 1 1/4" diameter bolts. Both the columns and beams were oriented so their strong axis resists in-plane motion. Figures 2.4a and 2.4b show the in-plane elevation view of the bare frame along with the steel truss system which was used to provide some additional out-of-plane stiffness. This bracing system was constructed using 3" x 3 1/4" angle iron and 5/16" connector plates. All the steel used met the requirements of ASTM A36 steel.

2.3 Masonry Panels

The masonry panels were constructed using 12" x 12" x 8" hollow clay tile blocks. Figure 2.5 shows the average dimensions of a typical 8" hollow clay tile used in these tests. The compressive strength of individual 8" clay tile units was determined by applying a gradual axial compressive load until failure (Flanagan et al., 1993a). The results of these tests yielded a gross compressive strength of 2.27 ksi for the cores oriented horizontally and 4.55 ksi for the cores oriented vertically. Testing was also performed on 2' x 4' x 8" clay tile prisms (Jones and Butala 1994). These prisms were loaded on the 2' face in axial compression until failure. The results of these tests are presented in Table

Figure 2.4a: In-plane elevation view of bare frame

Figure 2.4b: Truss detail

Figure 2.5: Average dimensions for a typical 8" hollow clay tile

2.1 which lists the ultimate load and the average compressive strengths, $f'm$.

The walls were constructed using single wythe construction with a running bond. The tile were laid with the cores being horizontal with approximately 1/2" head joints and 1/2" bed joints and using face shell bedding. A full layer of mortar was placed between the base beam surface and the first course of the wall panel. The infill was bonded to the web of the columns and the bottom flange

Table 2.1: Prism test results

Prism	Ultimate load (kips)	Gross comp. strength (ksi)	Net comp. strength (ksi)
8" cores vertical	190	1.02	3.82
8" cores horizontal	134	0.72	2.62

of the beam by snugly packed full mortar joints between the steel and masonry. There was no mortar placed in the gap between the flange of the columns and the masonry. There was no offset between the steel frame and the masonry, i.e. the masonry was laid at the frame centerline. Type N (ASTM C 270) mortar was used, but there was no steel reinforcement placed in the masonry.

2.4 Wall Rotation

The biaxial shake table being used for these tests could only provide horizontal acceleration in one direction. It was necessary to be able to shake the structure in both the out-of-plane and in-plane directions. To accomplish this the walls and columns were constructed on steel tube sections as shown in figure 2.4a. These tube sections were then bolted down to the shake table. The specimen was rotated between the out-of-plane and in-plane tests.

2.5 Out-of-Plane Instrumentation

After the specimen was constructed, various instrumentation and data collection devices were attached to the frames and panels to obtain data during the out-of-plane tests. Figure 2.6 shows the instrumentation and control schematic for the tests. The different types of measuring instruments used were linear variable displacement transducers (LVDTs), and absolute displacement transducers (ADs), strain gages and accelerometers. The locations of these instruments for walls #1 and #2 of the out-of-plane tests are shown in figures 2.7 and 2.8 respectively.

LVDTs were used to measure the relative displacement between the frame and the masonry panel. Figure 2.9 shows the mounting details for both the in-plane and out-of-plane tests. These are direct current (DC) powered units that were connected to Endevco model 4471.1A signal conditioners. The LVDTs were balanced by physically adjusting the sensing device until a zero output signal was produced.

The ADs were used to measure absolute displacements at the shake table and various locations of the structure. These transducers were mounted on a steel reference frame on the foundation, and the sensing elements were attached to the structure using stranded steel extension wires. The ADs were connected to a Endevco model 4471.3 signal conditioner which provided DC power and electrical balancing.

Figure 2.6: Instrumentation and control schematic

Figure 2.7: Out-of-plane test instrumentation, wall #1

Figure 2.8: Out-of-plane test instrumentation, wall #2

Figure 2.9: LVDT mounting details
(Gamble, 1993)

Strain gages were installed at various locations on the steel columns. These are 0.25" long by 0.12" wide grid gages which are temperature compensated for steel. The gages were mounted in pairs, on each side of the column flange. These gages were wired together in pairs in a half bridge configuration to yield an average reading of strain at each particular location on the column. The strain gages were connected to a Vishay model 2120 signal conditioner which provided power, balancing and signal amplification.

Accelerometers were used to measure accelerations in the frame and panel. These DC powered, Endevco model 7290-

10 and 7290-30 accelerometers were attached to Endevco model 4476.2 and 4476.2A signal conditioners which provided power, balancing and signal amplifications.

2.6 In-plane Instrumentation

After the out-of-plane tests were performed, the instrumentation was changed to provide readings in the in-plane direction. The same type of LVDTs, ADS, strain gages and accelerometers were used for the in-plane tests. The location of the instrumentation for wall #1 and wall #2 are shown in figures 2.10 and 2.11 respectively.

LVDTs were used to measure relative displacements between the panel and frame and also to measure panel displacements across the face of the walls.

The strain gages were placed on both the columns and the beam. The column gages were placed in the same locations on both flanges and wired together to give an average strain reading at each particular position on the column. The beam gages were placed on the top and bottom flanges and on each side of the flange. The two gages on the same flange were wired together to give an average strain in the beam.

Figure 2.10: In-plane test instrumentation, wall #1

Figure 2.11: In-plane test instrumentation, wall #2

Chapter 3

TESTING PROCEDURE

3.1 Introduction

Three different types of tests were performed on the specimen. The first set of tests were performed with the walls oriented out-of-plane with respect to the loading direction. The specimen was then rotated 90 degrees on the shake table so that the second set of tests could be performed with the walls oriented in-plane with respect to the loading direction. After the in-plane seismic tests were complete, two in-plane sine sweep tests were performed on the specimen.

3.2 Out-of-Plane Testing Procedure

A total of seven tests were performed on the structure in the out-of-plane direction as listed in table 3.1. Three of the tests were performed by exciting the model with an Oak Ridge, Tennessee, site specific seismic time history record as shown in figure 3.1. The displacement record from figure 3.1 was programmed into the biaxial shake table and, in turn, used to excite the structure. The acceleration time history was used to calculate a acceleration response spectra for the system (shown in Figure 3.2). Also shown in figure 3.2 is the actual acceleration response spectra obtained from each of the three seismic tests. As indicated

Figure 3.1: Seismic time history (Gamble, 1993)

Figure 3.2: Out-of-plane acceleration response spectra

Table 3.1: Out-of-plane testing sequence

Test	Test Level	Test Type
1	1.6% Full Scale	Low level random test
2	1 x Full Scale	Seismic time history test
3	2 x Full Scale	Seismic time history test
4	1.6% Full Scale	Low level random test
5	4 x Full Scale	Seismic time history test
6	1.6% Full Scale	Low level random test
7	1.6% Full Scale	Low level random test (all bolts retightened)

in Table 3.1, the seismic tests were performed at 1, 2 and 4 times the full scale value of the time history displacements.

The other four tests which were performed were low level random vibration tests. These tests were performed to measure the natural frequency of the structure after each of the seismic tests in order to observe the amount the frequency changed with each progressive test. These tests provided just enough vibration amplitude to produce a noticeable response in the specimen. Data were recorded at each of the locations shown for the out-of-plane configuration as discussed in chapter 2. A Fourier transfer function can be used on their data to obtain the natural frequencies of the structure. At the end of each out-of-plane tests, crack maps were drawn to document the progression of mortar cracking.

3.3 In-Plane Testing

A total of thirteen tests were performed with the structure oriented in the in-plane direction as listed in table 3.2. In the in-plane testing, six tests were conducted using the same site specific time history record that was used for the out-of-plane tests. Figure 3.3 shows the calculated response spectra along with the actual response spectra obtained for the six seismic tests. As table 3.2 indicates, these tests were performed at 1, 2, 4,

Table 3.2: In-plane testing sequence

Test	Test Level	Test Type
1	1.6% Full Scale	Low level random test
2	1 x Full Scale	Seismic time history test
3	1.6% Full Scale	Low level random test
4	2 x Full Scale	Seismic time history test
5	1.6% Full Scale	Low level random test
6	4 x Full Scale	Seismic time history test
7	1.6% Full Scale	Low level random test
8	8 x Full Scale	Seismic time history test
9	1.6% Full Scale	Low level random test
10	12 x Full Scale	Seismic time history test
11	1.6% Full Scale	Low level random test
12	16 x Full Scale	Seismic time history test
13	1.6% Full Scale	Low level random test

Figure 3.3: In-plane acceleration response spectra

8, 12 and 16 times the full scale time history record. Seven low level random vibration tests were performed to determine change in frequency of the structure as the testing progressed. Data were recorded during each of the tests at the instrumentation locations specified for the in-plane test as shown in chapter 2. At the end of each of the in-plane tests, crack maps were drawn.

3.4 Sine Sweep Tests

After all of the out-of-plane and in-plane tests were completed, two in-plane sine sweep tests were conducted in

order to fail the structure. During the sine sweep tests, the biaxial shake table was driven in a horizontal direction at a constant amplitude, sinusoidal acceleration. This acceleration was swept over a predetermined frequency range. For the first test the frequency started at 10 hertz and was increased up to 20 hertz at an amplitude of 1.0 g. For the second test the frequency started at 15 hertz and was to be decreased down to 5 hertz at an amplitude of 3.0 g. During the second test, the masonry panel began to fail and the test was stopped at a frequency of about 12.7 hertz.

Chapter 4

OUT-OF-PLANE TESTING

4.1 Introduction

This chapter discusses the data obtained from three out-of-plane tests which were conducted on the infilled frame structure. As discussed in chapter 3, the first seismic test involved exciting the structure with an input base displacement. In the second and third seismic tests, this base acceleration was increased by a factor of 2 and 4 respectively. The main reason for performing these dynamic tests was to determine loss of stability and load carrying capability of the infill. Degradation of the infill occurs with increasing load and causes a decrease in the stiffness of the structure. This usually occurs when the infill develops cracks as a result of the loading.

To help understand how this structure behaved in response to the dynamic loading, several things were analyzed. Acceleration behavior was observed along with behavior of relative displacements and absolute displacements. Also, information on column deformations is presented along with column bending stress levels. In the later part of the chapter, loss of stiffness as observed through a decrease in fundamental frequency of the structure, is analyzed.

4.2 Accelerometers

There were seven accelerometers located vertically along the center of the wall starting at the base, or shake table, and incremented at predetermined locations up the panel of the wall with the last one being located at the top of the slab. The locations of these accelerometers are shown in figure 4.1. An envelope of peak accelerations for each of the three out-of-plane tests is shown in figure 4.2. The maximum accelerations tend to increase with increasing wall height except at the top of the roof slab, where the accelerations drop off somewhat. Apparently the large mass of the roof slab did not respond to the input acceleration as drastically as the top of the wall. This drop off is amplified as the test scaling increases. The peak accelerations in the panel increased linearly with increased base displacement.

Figure 4.3 shows the instantaneous acceleration versus wall height. These two plots show what is happening at other accelerometer locations at the time when peak acceleration occurred at a selected accelerometer. One curve is shown for the accelerations occurring in conjunction with the peak acceleration of the roof slab at which time the wall appears to be responding in the first mode of the structure. Another curve is shown for the accelerations occurring in conjunction with the peak acceleration at the top of the panel at which time the wall

Figure 4.1: Accelerometer instrumentation locations

Figure 4.2: Peak accelerations vs. wall height

Figure 4.3: Instantaneous accelerations vs. wall height

appears to be responding in the second mode shape.

Figure 4.4 is an overplot from the third seismic test showing the acceleration data between 2.5 seconds and 3.3 seconds. These accelerations were recorded at three locations on the wall (a9 at specimen base, a7 at middle center of panel, and a4 at the center of roof slab). As the figure depicts, the accelerations are larger at the top of the wall. In observing the response of the structure to the input acceleration, the roof slab seems to be filtering out the higher frequencies and to be responding primarily at the fundamental frequency of the structure, whereas the rest of the structure is responding to higher frequencies, along with the fundamental frequency.

Figure 4.4: Overplots of accelerometer readings from the third seismic test

The amplification factors between the peak acceleration of the shake table and the three accelerometers located towards the top of the wall are shown in Table 4.1. The top of the wall apparently amplified the base acceleration by a factor of about 3.0.

4.3 Relative Displacements verses Absolute Displacements

Figure 4.5 shows the instrumentation locations for all of the relative and absolute displacement transducers being analyzed in this chapter. The maximum relative displacements were 0.018 inches for test #1, 0.030 inches for test #2, and 0.058 inches for test #3. All of the

Table 4.1: Acceleration amplifications

Test	Amplification		
	Slab (A4)	Beam (A3)	Panel (A8)
1	2.55	2.90	2.85
2	2.62	3.28	2.58
3	2.50	3.22	3.41

Figure 4.5: Instrumentation locations

maximums occurred in the relative displacement transducer located on the left side of the panel at a height of 39" (middle panel/column interface). The maximum relative

displacements recorded for the three tests performed were small in comparison to the maximum absolute displacements, which means that the infill and frame are basically moving together as a single rigid body. A larger relative displacement was observed between the panel-column interfaces than in the panel-beam and panel-base interfaces. Figure 4.6 shows a plot of the ratio of peak relative displacement to peak absolute displacement versus amount of base motion magnification for several locations. The plot shows that the ratio decreases with each successive test, meaning that the rate of increase of the relative displacements becomes smaller with respect to the rate of

Figure 4.6: Relative/absolute disp. ratio vs. base motion magnification

increase of the absolute displacements. This plot shows that the stability of the wall was being maintained even as the tests scaling increased.

After these first three out-of-plane tests, the infill was still intact, and the tests had not significantly affected the frame or infill, as they still seemed to be responding as a single unit or rigid body. There was no evidence to this point of any "walking out" (where the infill is no longer attached or confined within the steel framing and begins to move out of the framing system) of the infill.

4.4 Deformed Shape of Columns

The deformed shape of the structure was obtained using the data from the absolute displacement transducers which were placed at three specified locations on both the left and right columns, and at two locations on the middle of wall #1 (one on the beam and one at the center of the panel). Figure 4.7 shows the maximum displaced shape of both the left and right columns, and the center of wall #1, for all three tests. The maximum displacements that were used to obtain these curves occurred at the same time. The noticeable change occurring in the curve at about 72 inches in both the left and right column deformed shapes were caused by a truss bracing system which frames into the column at this location. The truss, which was provided to

Figure 4.7: Maximum deformed shape of columns and panel

add stiffness and stability to the structure, caused the column to bend outward at about 72 inches, while restricting the displacement at the top of the column, resulting in the deformed shape shown. Stresses, which are discussed later, were higher in this region due to the truss. The displacements occurring up the middle of the wall were not as large as the column displacements and, as the curves show, the middle deformations occurred in a linear fashion, unlike the bending shape which was present in the columns. This bending shape in the columns, leads to the maximum relative displacements occurring at the middle location.

4.5 Axial & Bending Stresses In Steel

By using the strain gage data that were recorded at heights of 6, 39, 66, and 94.5 inches (locations shown in figure 4.5), the axial and bending stresses in the steel were determined. The maximum bending stresses developed in the columns for the three tests were 4.21, 6.98, and 12.24 ksi, which occurred at the 66 inch point in the columns. All of these stresses fall well below the minimum yield stress of the steel which is 36.0 ksi, with the highest stress developed in the column (12.24 ksi) only 34% of yield. Since the strains developed in the columns were so small, there should be no loss in stiffness of the columns due to yielding during any of the out-of-plane tests. The axial stresses were much smaller in comparison to the bending stresses (between 10% and 15% of the bending stress), the highest reaching 1.56 ksi.

Figure 4.8 shows the moment envelopes obtained for each of the three tests. The larger bending moments in the columns at 66 inches were a result of the truss brace which frames in at approximately 70 inches. The truss stiffened the top part of the structure, i.e. it reduced the deflection at the top of the column and in turn caused larger bending stresses at the 66 inch point in the column.

Figure 4.8: Moment envelopes

4.6 Degradation In Frequency with Increasing Load

The data obtained from the accelerometers were used to analyze the degradation in frequency with increasing load. A Fourier transform was used to determine the frequency of sinusoidal components in accelerometer data, therefore giving a means for determining the magnitude of the fundamental frequency of the structure. The fundamental frequencies determined during the three tests were 6.45, 6.45, and 6.25 hertz, respectively. The frequency decreased with increasing seismic input, so it is evident that the structure was losing some stiffness; i.e. there was a loss

of rigidity between the infill and frame. This loss was probably due to cracking of the infill, which was observed to be occurring in increasing amount with each successive test.

Evidence of degradation also appears when analyzing the relative displacement data. As the load increased, the relative displacements between the infill and frame also increased, indicating that the infill was becoming looser within the frame as the load increased; therefore, the infill and frame were acting increasingly more independent of each other, resulting in a more flexible structure. There was a decrease in frequency of 3.2% and a decrease in stiffness of 6.1% as a result of performing the three out-of-plane tests.

Figure 4.9 shows curves obtained by using transform functions on accelerometer data recorded at the roof slab (A4), frame (A1) and the panel (A8), from seismic test #3. The roof slab and frame have a peak at about 6.4 hertz, the first natural frequency of the structure. The panel has the largest peak at a much higher frequency (about 31.7 Hz). This, along with difference in deflected shape of the panel and frame, implies that the structure was responding as two separate systems; the infill had separated from the frame. However, the infill remained stable and showed no evidence of losing stability.

Figure 4.9: Curves of transform functions from the third seismic test

4-11-11

Chapter 5

IN-PLANE TESTING

5.1 Introduction

The in-plane behavior of clay tile infills subjected to static cyclic racking forces has previously been researched and documented (Flanagan et al., 1993b). Substantial analysis has been performed on load-deflection behavior, column forces, panel deflection behavior, and relative displacements of statically loaded clay tile infills. From these tests, structural characteristics such as stiffness, load carrying capability and infill-frame interaction behavior can be obtained. To be able to use this information in evaluating structural performance under seismic load conditions, it is important to be able to draw comparisons between how the infill behaves under static and dynamic loading. The purpose of this chapter is to analyze data obtained from dynamic in-plane loading of clay tile infills, and to comment on similarities between the behavior of dynamically and statically loaded infills.

5.2 Peak Accelerations

Figures 5.1 and 5.2 show envelopes of peak accelerations for the left column of wall #1, center of wall #1 and center of wall #2 for various levels of base acceleration. The accelerometers and locations used to

Figure 5.1: Peak acceleration vs. wall height for 1 and 4 x full scale input displacement

Figure 5.2: Peak acceleration vs. wall height for 12 and 16 x full scale input displacement

obtain the curves for the left column and center of wall #1 are shown in figure 5.3. The location of the accelerometers used for the center of wall #2 were the same as those shown on wall #1. The reason for only two curves per set in figure 5.2 is because one of the accelerometers on wall #2 was defective.

The curves from the center of wall #1 and wall #2, show that the peak accelerations increase slightly when going vertically from the base to the roof slab. The two accelerometers in the left column show considerable amplification in peak accelerations. The large mass of the roof slab did not respond to the input base motion as drastically as did the column, and the difference in peak acceleration between the column and slab increased as the base motion intensity was magnified. The difference in peak accelerations between the center of the panels and the roof slab also increased as the base motion intensity was increased. This increase could be a result of the high peak accelerations in the columns racking against the infill, causing increased peak accelerations in the panel.

Figure 5.4 is an overplot from the sixth in-plane seismic test showing the acceleration data between 2.3 seconds and 2.95 seconds. The accelerometer readings were recorded at three locations on the wall (A7 at center of the roof slab, A5 at middle center of panel, and A3 at specimen base). In observing the response of the structure to the

Figure 5.3: Accelerometer locations

Figure 5.4: Overplots of accelerometer readings from the sixth in-plane seismic test

input displacement, the roof slab seemed to be filtering out the higher frequencies and responding primarily to the fundamental frequency of the structure, whereas the rest of the structure was responding to higher frequencies, along with the fundamental frequency. The panel was seeing higher frequencies than the roof slab. As explained before, this could be a result of the columns, racking against the infill.

5.3 Load-Deflection

Figure 5.5 shows the load-deflection curves calculated at the top of the right column (AD3) of wall #1. These were taken from absolute displacement recordings from each of the six in-plane tests. These load-deflection loops show the maximum recorded load for each test. In all the load-deflection plots, the load pertains to the roof slab force which was obtained by multiplying the lumped mass (taken as 9.5 kips) by the accelerations recorded at the top of the roof slab (A7). The maximum load taken by the structure was 48.4 kips. The slope of the load-deflection curves decreased with each progressive test, indicating that the structure was losing stiffness. Figure 5.6 shows the load-deflection curve for the top of the left column of wall #2. These two figures are similar in reference to load magnitudes, and stiffness characteristics although there

Figure 5:5: Load-deflection curve for top right of wall #1

Figure 5:6: Load-deflection curve for top left of wall #2

does not seem to be much energy dissipated in the last curve of figure 5.6.

The load-deflection behavior observed in static testing seems similar in stiffness characteristics and in ultimate load which was 46.0 kips. The ultimate load from the dynamic tests was expected to be a little higher since prism testing and unit block tile tests had shown that the block used for the dynamic testing was a little stronger than those used in the static testing.

Figure 5.7 shows the maximum load-deflection curves from the dynamic and static tests. In the static tests, there was a flat portion of low stiffness at the center of the load deflection curves. This pinching behavior has been attributed (Flanagan, 1994) to opening and closing of the interface gap between the column and infill and plastic deformations of the masonry. There was no pinching apparent in the dynamic tests and, therefore, the deflections were not as large as those observed in the static tests.

5.4 Column Axial Forces and Bending Stresses

Figure 5.8 shows the Load-Axial Column Force curve obtained from strain gages located at the top of the right column of wall #1 (see figure 5.9 for strain gage locations). As the force acted to the left (corresponding to figure 5.10) and the load taken by the structure increased, the axial tensile force in the column increased.

Figure 5.7: Static and dynamic load-deflection curve overplots

Figure 5.8: Load-axial column force curve

Figure 5.9: Instrumentation, wall #1

Figure 5.10: Column force free body diagrams

The tensile force in the column is almost as high as the load taken by the structure. As the load was reversed and acted to the right, the column initially went into compression. As the load increased, the column force reversed and went back into tension, which is consistent with what was observed in the statically tested walls (Flanagan, 1994).

Apparently the infill was acting as an equivalent strut. As the load acted to the left, a compressive strut formed (see figure 5.10a) diagonally (approximately 50 degrees) in the infill. This compressive strut produced an upward force at the corner of the column and beam, causing a tensile force in the column. As the load initially acted to the right, the compression strut formed as shown in figure 5.10b, and the column was placed in compression in reaction to the moment from the load. As the loads increase during the cycle, high lateral strains develop normal to the compression strut (see figure 5.10c) causing an upward force at the column beam interfaces, resulting in a tensile force in the top of the column. Lateral strains as high as the longitudinal strains were recorded in the uniaxial prism tests (Boyd, 1993).

Figure 5.11 shows the Load-Bending stress curve obtained at the top of the right column, from wall #1. The bending stresses were small until the last test in which a stress of 3.25 ksi was reached. These bending stresses were

Figure 5.11: Load-bending stress curve

of the same magnitude of the axial stresses which were observed to be 3.05 ksi. No apparent pattern existed with the bending moments.

5.5 Relative Displacement

Figure 5.12 shows the load-deflection curves at the top left of the panel (RD1 in figure 5.9). The curves for the higher intensity tests show a negative displacement during both the tensile and compressive cycles. This may be a result of the panel's shifting during the tests and becoming permanently offset. In portions of the curves the load increased with little change in relative displacement,

Figure 5.12: Load-relative displacement (from RD1) curve

indicating that the panel and frame were deflecting or moving together. There are no portions in the curve that would indicate the presence of a gap between the infill and frame.

Figure 5.13 shows the load-deflection curves at the top right of the panel (RD2 in figure 5.9). These relative displacements occur in both negative and positive fashions. This curve shows that there was a gap that had developed between the panel and the frame, evidenced by portions of the curve where there is an increase or decrease in deflection, but little increase or decrease in load.

Because the behavior and response of the panel during dynamic loading are much more erratic, it is difficult to

Figure 5.13: Load-relative displacement (from RD2) curve

draw comparisons between these curves and the data obtained during the static tests. The static tests generally showed the presence of gaps between the frame and infill as the curves would show an increase in deflection with no increase in load. Eventually, after measurable deflection, the load would begin to increase (Barclay, 1993).

5.6 Panel Load-Strain

Figures 5.14 and 5.15 show the load-strain curves for the panel horizontal and panel vertical transducers (PD3 and PD4 in figure 5.9) respectively. In these figures it appears that there is little deformation occurring in the panel during the low intensity tests. Apparently there had

Figure 5.14: Load-panel horizontal strain curve

Figure 5.15: Load-panel vertical strain curve

been little to no cracking and the panel was responding elastically. In the last two seismic tests, the strain in the panels was increasing during both the compressive and tension cycles in a way similar to what was observed in the static test results. This similarity indicated that, due to significant cracking and panel degradation, the panel was expanding both laterally and longitudinally within the frame during loading. These curves suggest that significant cracking of the panel did not occur until the fifth seismic test (12 x F.S.). The maximum recorded horizontal and vertical panels strains were 0.00025 and 0.00077 respectively. In the static test, strains of 0.00137 and 0.00363 (Barclay, 1993) were recorded at the horizontal and vertical panel locations. These static strains are significantly higher than those observed in the dynamic tests. This is due to the larger vertical load being applied by the roof slab which is confining the panel and resulting in lower strains in the dynamic tests. There was no roof slab present in the static tests therefore the strains were higher.

The load-strain curves for the panel diagonals (PD1 and PD2 in figure 5.9) are shown in figures 5.16 and 5.17. During the tension cycle, there was an increase in the panel diagonal strains with increasing load. During the compressive cycle, there was a decrease in the panel diagonal strains with increase in load. The curves show

Figure 5.16: Load-panel diagonal strain (from PD1) curve

Figure 5.17: Load-panel diagonal strain (from PD2) curve

that the panel stiffness was decreasing with each progressive test. The maximum longitudinal and lateral strains recorded in the panel were 0.00135 and 0.00175 respectively. This compares to the maximum longitudinal and lateral strains of 0.00145 and 0.0046 (Barclay, 1993) which were observed in the static tests. Lateral strains as high as the longitudinal strains were also recorded in uniaxial prism tests (Boyd, 1993). The load-strain loops obtained in the static tests showed more plastic deformation of the masonry than did those shown in the dynamic load-strain loops.

The panel diagonal load-strain curves show that large lateral strains were being developed in the panel during the tests. The axial tensile forces in the columns discussed earlier in the chapter were attributed to these high lateral strains.

5.7 Degradation in Frequency with Increasing Load

As discussed in the out-of-plane chapter, the data obtained from the accelerometers were used to analyze the degradation in frequency with increasing load. A frequency transform was used to determine the magnitude of the fundamental frequency of the structure. Table 5.1 shows the fundamental frequency of the structure after each of the in-plane tests. A frequency transfer function was also used on accelerometer data obtained by performing low frequency,

Table 5.1: Results of frequency analysis

Seismically Loaded Tests		White Noise Test	
Test	Frequency Transform	Test	Frequency Transform
4	16.6	Before Test 4	16.0
5	15.1	After Test 4	16.7
6	15.6	After Test 5	15.8
7	13.2	After Test 6	16.2
8	10.3	After Test 7	15.0
9	8.8	After Test 8	13.3
-	-	After Test 9	14.3

white noise tests. The frequencies obtained in each of the analyses were somewhat different indicating that the structure responded at a different frequency under the low level white noise test than under the higher intensity seismic test.

The frequency decreased with increasing seismic input, indicating that the structure is losing stiffness; i.e. there is a loss of rigidity between the panel and frame and within the panel. The frequency transform analysis yielded a 47% decrease in frequency. Since stiffness is proportional to the square of the frequency, this meant there was a 72% decrease in stiffness. From the white noise analysis, there was a 11.0% decrease in frequency and a 21% decrease in stiffness.

Evidence of panel degradation also appears when analyzing the load-deflection and load-strain curves. As the load increased, the relative displacements between the frame and panel increased and also strain levels in the panel are increasing. The infill interface was deteriorating as the load increased, and the infill was losing stiffness due to cracking.

Chapter 6

SINE SWEEP TESTS

6.1 Introduction

After the out-of-plane and in-plane tests were completed, two sine sweep tests were performed on the structure. The objective of the sine sweep tests was to locate and excite the structure at its fundamental frequency, causing the walls to fail. The first sine sweep test was performed at an amplitude of 1.0 g, and the frequency started at 10 hertz and was increased up to 20 hertz. After the first sine sweep test was performed, the structure had not failed; hence, the second sine sweep test was performed. This test had an amplitude of 3.0 g, and the frequency started at 15 Hz and was decreased down to 5 Hz. The test was stopped at approximately 12.7 Hz when several tile fell from the upper course of the wall.

The sine sweep tests subjected the structure to a large number of loadings. Eight hundred fifty cycles of loading were performed on the structure during the 1.0 g sine sweep test and three hundred twenty cycles were performed during the 3.0 g sine sweep test. This greatly exceeds what would be expected in a typical seismic event. The loading could have significantly affected the characteristics of the structure and may have caused a fatigue failure.

6.2 Magnification analysis

Figure 6.1 is a plot of acceleration magnification in the roof slab versus frequency for the 1.0 g sine sweep test. It was obtained by isolating portions of the sine sweep test such that the frequency was relatively constant. This plot shows the down slope of the dynamic magnification curve for the structure. As shown in the frequency analysis section of the in-plane testing chapter, at low excitation levels (average excitation around 0.1 to 0.2g) the fundamental frequency is around 15 Hz. At higher levels of excitation (an average excitation of 1.0 g), the frequency drops down to around 9 Hz. The peak for the curve in figure 6.1 presumably falls somewhere below 10 hertz, indicating that the first sine sweep test missed the fundamental frequency of the structure. A small peak is observed in the 14 to 15 Hz range, but with only small magnifications.

Figure 6.2 shows a plot of acceleration magnification in the roof slab verses frequency for the 3.0 g sine sweep test. In this test, tile began to break and fall out of the top course of the panel at about 13 hertz. At this time, the lab technician stopped the test; therefore, the fundamental frequency of the structure was never reached. The breaking and falling out of tile could be attributed to excessive loading or a fatigue failure, as discussed earlier.

Figure 6.1: Acceleration magnification vs. frequency from sine sweep test with an input acceleration of 1.0 g

Figure 6.2: Acceleration magnification vs. frequency from sine sweep test with an input acceleration of 3.0 g

6.3 Load-deflection

Figure 6.3 shows the load-deflection curves calculated at the top of the right column (AD3) of wall #1 and the top of the left column (AD6) of wall #2. The deflections were taken from absolute displacement transducers and the load was obtained (as discussed in the in-plane chapter) by multiplying a lumped mass by the acceleration in the slab. These are the load-deflection curves obtained from the 1.0 g sine sweep test, at a frequency of 11 hertz. Figure 6.4 shows the load-deflection curves obtained from the 1.0 g sine sweep test, at a frequency of 14 hertz. The load has dropped off somewhat due to less magnification in the table.

Figure 6.5 shows the load-deflection curves obtained from the 3g sine sweep test, at a frequency of 13 hertz. The structure had lost some stiffness since the 1.0 g sine sweep test. Figure 6.6 shows the load-deflection curves obtained when the test was stopped (while the structure was in free vibration). The curves are plotted at four different times during the free vibration phase. There is a steady decrease in these load-deflection curves over this 1.7 second period.

The maximum loads recorded initially in the 1.0 g sine sweep test were around 22 kips, it reduced to 9.5 kips as the test progressed. The maximum loads recorded in the 3.0 g sine sweep test were around 38 kips for all cycles. In comparing these to the maximum load levels observed in the

Figure 6.3: Load-deflection curves from 1.0 g sine sweep test at a frequency of 11 hertz

Figure 6.4: Load-deflection curves from 1.0 g sine sweep test at a frequency of 14 hertz

Figure 6.5: Load-deflection curves from 3.0 g sine sweep test at a frequency of 13 hertz

Figure 6.6: Load-deflection curves from the end of the 3.0 g sine sweep test taken during free vibration

in-plane tests and in static testing (around 48 and 46 kips respectively), it appears that a little larger sine sweep load would essentially fail the panel. The 3.0 g sine sweep test was subjecting the panel to a large number of cycles at about 80% of its capacity.

Chapter 7

LITERATURE SURVEY

7.1 Introduction

This chapter reviews other research which has been conducted in the areas of static and dynamic loading of masonry infilled frames. Although this thesis focused on analyzing dynamic loading of structural clay tile infilled frames, there is also pertinent and comparable information which has been obtained from other types of research and testing performed on masonry infilled frames. This literature survey is divided into two sections. The first pertains to research in the area of statically loaded masonry infills. The second investigates other research involving dynamic testing of masonry infills.

7.2 Static testing

Significant studies on the behavior and strength of masonry infills have been performed over the last four decades. The majority of these tests have been conducted on single bay, single story, single wythe masonry panels confined by either reinforced concrete frames or steel frames. Different physical parameters such as aspect ratio, slenderness ratio, and type of mortar and masonry infills, have been used throughout the various tests. Different loading conditions have also been used. Some of these

include loading in-plane to failure, out-of-plane to failure, and in-plane to cracking then out-of-plane to failure.

Various types of physical parameters and loading conditions were used on tests performed on hollow clay tile infilled steel frames during an experimental program conducted at the University of Tennessee (Flanagan, 1994). Thirteen in-plane tests, eight out-of-plane tests, and one bi-directional test were performed during this program.

The conditions that were varied during the in-plane testing included varying frame stiffness, aspect ratio, single or double wythe construction, offset panel, corner openings, and mortarless column/infill interfaces. Two of the tested specimens were repaired then retested. Some of the conclusions obtained from these different tests were:

- 1) Strength and stiffness of the system is increased significantly with the addition of the infill.
- 2) Changing the steel frame stiffness significantly effected the stiffness of the infilled frame system but had little effect on the ultimate capacity of the infilled frame system.

- 3) Increasing the aspect ratios of the infill increased the system stiffness, ultimate load, post-ultimate load and post-ultimate stiffness.
- 4) Double wythe and single wythe panels exhibited similar structural characteristics.
- 5) Mortarless infill/column interfaces resulted in a softer load-deflection behavior and a marginally smaller ultimate strength.
- 6) The testing of a repaired specimen resulted in a softer load-deflection behavior and an ultimate strength capacity of about 80% of the undamaged infill capacity. This type of behavior seems to be consistent with those results obtained from ferrocement overlays also.

Two types of out-of-plane tests were performed. One subjected specimens to simulated induced inter-story drift and the other applied a uniform lateral pressure on the infill by using an air bag.

The infill panels that were subjected to inter-story drift were loaded cyclically out-of-plane at either the top or midheight level of the columns. These tests showed there was only a small amount of degradation in the strength

and stiffness of the panels as a result of these out-of-plane drift displacements.

Three air bag tests were performed on a 4", 8" and a 13" double wythe panel in which the specimens were loaded cyclically until ultimate capacity was reached. Results of these tests showed that arching action (which developed as a result of the panel deflection at the middle portion) provided the strength for the panels.

Two other tests were performed in which the specimen was loaded cyclically in-plane and then loaded out-of-plane to failure by an air bag. These tests showed that the initial in-plane loading reduced the out-of-plane ultimate capacity by approximately 15%.

Angel et al. (1994) performed experimental and analytical research on single bay, single story reinforced concrete infilled frames. The tests included both concrete and clay block infills which were subjected to cyclic in-plane loading (to a displacement equal to twice the initial cracking displacement) and then loaded out-of-plane with an air bag. The physical parameters that were varied in the tests were the masonry and mortar type and the slenderness ratio of the infill panels. Analytical models were also developed to predict the strength and behavior of the panels under these type testing situations.

Some of Angel and Abrams' conclusions regarding the in-plane and out-of-plane behavior of these types of infills were:

- 1) After cracking occurs in the infill, the lateral stiffness of the panel decreases greatly.
- 2) Lateral stiffness is directly proportional to the compressive strength of the mortar.
- 3) Analytical models based on equivalent strut theories adequately predict the lateral stiffness of the infill, up to cracking.
- 4) Slenderness ratio greatly effects the out-of-plane strength.
- 5) Repeated loadings within the elastic region of the panel does not decrease the stiffness of the specimen.
- 6) For slender panels, initial in-plane cracking reduces the out-of-plane strength by as much as a factor of two.

There have been other similar types of tests performed but with different physical parameters from those discussed above (e.g. Dawe, 1988a). Dawe performed twenty-eight in-plane tests on large-scale concrete masonry infilled steel frame specimens. The parameters varied were column/panel interface conditions, beam/panel interface conditions, and the presence of panel openings. Some conclusions from these tests were:

- 1) Fully packed mortar joints between the panel and columns results in a higher initial stiffness but has little effect on ultimate loads. Reduction in the amount of mortar between the panel and columns reduces the cracking load and ultimate strength of the panel.
- 2) Strength and stiffness of infilled frames can be improved by using reinforcement.
- 3) A gap between the top beam and panel significantly decreases the cracking and ultimate load capacity of the specimen.
- 4) Door or window openings in the panels reduce the cracking load but may not significantly effect

the ultimate capacity if the opening is not located close to the loaded side of the panel.

Some results and conclusions from these static tests seem to substantiate results obtained in Chapter 4 (Dynamic Out-of-Plane Testing). For instance, the static tests by Flanagan (1994) suggested substantial out-of-plane stability. There is no evidence of the infill walking-out or falling-out of the frame, which was confirmed in the dynamic tests. Also as discussed in the static testing, once the infills crack the lateral stiffness decreases greatly as the infill and frame no longer act compositely. Evidence of this can be seen in the dynamic tests as the deflected shapes of the frame and panel are not the same and the frame and panel do not respond at the same natural frequency (i.e. the frame and panel are not acting together). However, as with the static tests the infill greatly increased the strength and controlled the lateral drift.

7.3 Dynamic testing

Although not as voluminous as static testings, there has been some research and experimental programs which have involved the dynamic testing of masonry infilled framed systems. These tests have investigated the dynamic response of masonry infills loaded in both the in-plane and

out-of-plane direction. Unlike static testing there does not seem to be a large number of analytical models developed to predict infill behavior under dynamic loading.

Out-of-plane dynamic testing was conducted by Blondet, Mayes and Kelly (1990) on six full-scale structural clay tile walls which were not infilled into steel or reinforced concrete framing. The purpose of this study was to study how much vertical grouting and reinforcement, and vertical loading effects the earthquake response of such type of structures. The structure was excited at both the top and the bottom. The excitation was larger at the top to account for response amplification at the top. Some of the main observations that were made are:

- 1) The wall response was larger than both the top and the bottom excitations.
- 2) The amount of reinforcement significantly influences the stiffness and response at all input levels.
- 3) Vertical load increases stiffness and reduces amplitude of response.

Some of these conclusions are consistent with results obtained from the out-of-plane analysis given in chapter 4. Similar to these tests, during the dynamic out-of-plane

analysis it was determined that the response was amplified greater at the top of the panel than at the top of the specimen. In comparing the load-deflection plots from In-Plane Testing (Chapter 5) with the load-deflection plots obtained from static tests, the dynamic tests give a stiffer structure. This could be attributed to the vertical slab load which was present during the dynamic tests but was not present in the static tests.

Dynamic out-of-plane testing was also conducted by Mander et al (1994). These experiments were performed on three story, three bay steel frame specimens which had a brick masonry infill placed in the center bay of all three - stories. Two specimens were tested, one was undamaged whereas the other had been previously tested in-plane. The first specimen was tested with a 15 to 1 hertz sine sweep excitation. At 5 hertz the maximum response acceleration of 10 g was observed under a 0.3 g input. Mander et al (1994) noted that the structure was difficult to fail and it took extra shaking before the structure became unstable. The second specimen which had already been tested in-plane was also tested out-of-plane with the 15 to 1 hertz sine sweep excitation. The maximum response acceleration during this test was 5 g. Some loss of strength resulted due to the specimen being previously damaged in the in-plane direction. The results of these tests show that the out-of-plane strength of masonry infill panels is very substantial due to

in-plane arching. There was no evidence of loss of stability or walking except at quite high and continuous levels of excitation.

Dawe et al. (1988b) performed in-plane shake table tests on one bay, one story frames infilled with brick masonry. Eight framed walls and two bare frame walls were tested in this study. The purpose of their research was to investigate strength characteristics, degradation and its effect on behavior, the effect of frame stiffness and roof slab rigidity on the response of the system, and the influence of ground motion amplification on the response of the system. The following observations were made:

- 1) The use of stiff frames and rigid slabs increases the strength of the system. The stiff frames cause extensive damage to the panels.
- 2) The panels significantly increase the strength and stiffness of the system.
- 3) Under weak to moderate ground motions, panel degradation is gradual. Under strong ground motions the panel degradation is very rapid.

Number 3 above is similar to results seen in chapter 6 in that during the 1.0 g sine sweep test the panel

degradation was gradual but when the 3.0 g sine sweep test was performed, the panel degradation was very rapid.

Numbers 1 and 2 above were verified in the static tests.

Three analytical models were used (Dawe et al., 1988b) on the tests discussed above. These included a single degree of freedom model, a compressional strut model and a braced frame model. From this study, the braced frame model predicted the response during the linear and initial stages of the nonlinear response, and appeared to be the most adequate of the three.

Chapter 8

SUMMARY AND CONCLUSIONS

8.1 Research Summary

The behavior of two structural clay tile infilled steel frames subjected to in-plane and out-of-plane dynamic loading was investigated to determine the structures' strength and stiffness characteristics. The main focus of this study was on frame and panel load-deflection behavior, along with acceleration amplification and frequency degradation behavior. The dynamic testing was performed on a biaxial shake table machine and the structure was composed of two framed infills, spaced nine feet apart and connected by two steel trusses and an eight inch concrete roof slab. After the out-of-plane and in-plane phases of the testing were completed, two in-plane sine sweep tests were conducted to fail the structure.

A literature review was conducted to correlate and compare the results from this study with the results and conclusions obtained from other experimental research which has been conducted in the areas of masonry infill testing.

8.2 Conclusions

The following conclusions are based on the observations made by analyzing data obtained from the out-of-plane, in-plane, and sine sweep tests which were conducted.

8.3 Out-of-Plane Behavior

The areas of interest in the out-of-plane direction were acceleration amplifications, relative and absolute displacements, column deformations, and frequency degradation.

The panels of the specimen had a larger acceleration amplification than either the roof slab or beam. The largest acceleration response occurred in the panels near the top of the walls in which an input amplification factor of about three was calculated. The relative and absolute displacement transducers indicated very little relative movement between the panel and frame. This suggests that the panel and frame were essentially moving together as a single unit, although frequency analysis indicated that the panel and frame were responding at two separate natural frequencies. The deformed shapes show that the largest relative displacement between the panel and frame occurred midway up the columns. The truss system which connects the two walls added considerable stiffness to the top of the specimen. This caused the maximum relative displacement to occur at midheight of the columns and may have caused the frame and panel to respond at two different frequencies.

As a result of the out-of-plane testing, there was a small drop in the structures stiffness but there was no evidence of the panel's "walking-out" of the frame. The

panel was still very stable after the completion of the out-of-plane testing.

8.4 In-Plane Behavior

The in-plane analysis also involved looking at acceleration amplification, relative and absolute displacements, and frequency degradation. Also, load-deflection behavior was investigated.

An analysis of acceleration data indicates that there was considerably larger acceleration amplification in the columns than in the roof slab. This might be attributed to the large mass of the roof slab not responding to the input base motion as drastically as did the column. The load-deflection analysis indicated that the stiffness of the structure decreased with each progressive in-plane test.

In comparison to static tests, higher stiffness was seen in the dynamically tested structure which could be attributed to the vertical load being applied by the slab. Ultimate strength characteristics were very similar to those observed from static in-plane tests.

Load-column force analysis indicated the formation of a diagonal compressive strut during loading which caused high lateral strains in the panels. In observing panel load-strain curves, it became evident that significant cracking of the panel did not occur until the fifth seismic test (12 x F.S.). Frequency degradation occurred with increasing

seismic input, indicating that the panel was cracking and losing stiffness.

The sine sweep tests that were performed failed to locate the natural frequency of the structure which was much lower than initially thought. The panels were eventually failed by using an input acceleration of 3.0 g although it is possible that the panels failed due to fatigue.

During all of these out-of-plane and in-plane tests, there was no evidence of the panels "walking out". Up until the last sine sweep test, the infill remained stable and still provided some load resistance. Therefore, infills can be expected to enhance the seismic resistance of otherwise laterally weak structures.

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