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Shake Table Testing of Structural Clay Tile Infilled Frames

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SHAKE TABLE TESTING OF STRUCTURAL CLAY TILE INFILLED FRAMES

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ABSTRACT: Two steel frames with structural clay tile infills were tested under simulated seismic loads in both the out-of-plane and in-plane direction. Out-of-plane testing showed that infill panels separate from their bounding frame, and respond at their own natural frequency during a seismic excitation. Due to arching, the panels remain stable. In-plane seismic testing showed similar behavior patterns to previous static testing. The natural frequency was adequately predicted using a piecewise linear equivalent strut analytical method. The structure was then subjected to over one thousand cycles of loading using a sine sweep before failure.

KEYWORDS: Infills, In-Plane Behavior, Out-of-plane Behavior, Shake Table Testing, Structural Clay Tile

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INTRODUCTION

Infill construction is quite 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 and thus the seismic force, 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 beneficial effect on the structural behavior. They serve to increase the stiffness of flexible frame buildings, and provide a redundant load path for both the horizontal and vertical loads.

Many buildings were constructed using clay tile masonry in the early part of the century, and extending into the World War II era. Although scattered throughout the United States, clay tile was extensively used in the high seismic regions of San Francisco and Los Angeles. An extensive static testing program on clay tile infills was recently completed (Flanagan, 1994). This paper reports the results of dynamic shake table testing.

Test Set-up

A one-story, single bay system was constructed with two structural clay tile infilled steel frames spaced approximately 9' apart. An 8" concrete roof slab was bolted to the top of the steel frames for mass, and a truss bracing system is connected between the two frames for added stiffness and strength. The steel frame consisted of W10x30 columns and a W12x40 beam connected to form a nominal 8' high by 11' long wall. The beam to column connections were framed connections using double clip angles. The columns and beams were oriented so their strong axis resists in-plane motion. A three dimensional view of the test set-up for out-of-plane testing is shown in Figure 1.

The masonry infill panels were constru
cted using 12" x 12" x 8" structural clay tile. 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 using Type N mortar. The infill was bonded to the web of the columns and the bottom flange of the beam by snugly packed full mortar joints between the steel and masonry. There was no reinforcement placed in the masonry. Testing was performed on 2'x 4'x 8" clay tile prisms (Jones and Butala 1994). The gross compressive strengths were 0.72 ksi for the load perpendicular to the bed joint, and 1.02 ksi for the load parallel to the bed joint.

The specimen was tested on the United States Army Construction Engineering Research Laboratories (USACERL) biaxial shake table. The specimen was first tested such that the infills were subjected to out-of-plane motion. The entire specimen was then rotated so that the infills were subjected to in-plane motion. Details of the testing are in Gamble (1993) and Fowler (1994).

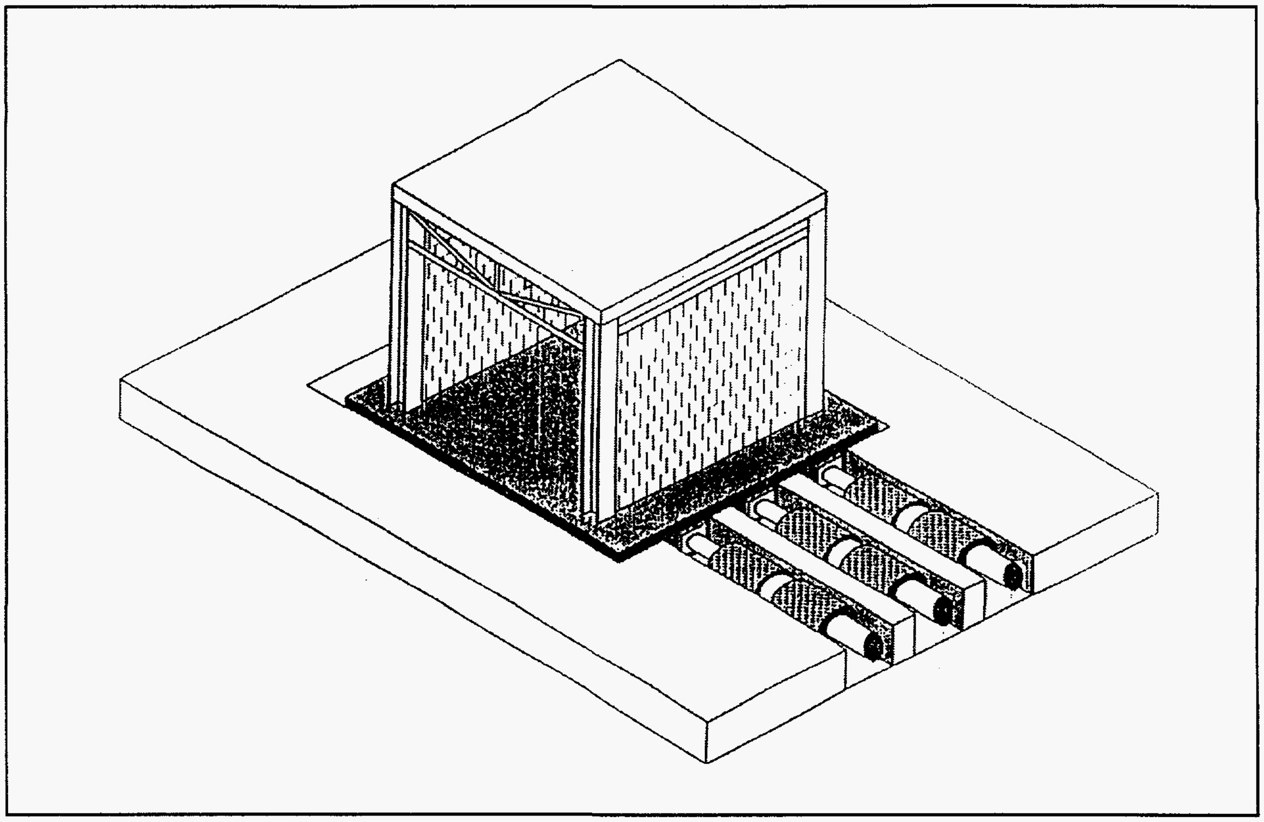

Figure 1: Three dimensional view of test setup (Gamble, 1993)

OUT-OF-PLANE TESTING

The structure was subjected to an artificially generated earthquake record that was representative of Oak Ridge, Tennessee. Peak acceleration for this record was 0.19g, with the strong motion lasting approximately ten seconds. The displacement record 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. Figure 2 shows the response spectra based on the actual table motion. The structure was actually subjected to three different loadings, these being at 1, 2 and 4 times the full scale value of the time history displacements.

Figure 3 shows the acceleration data between 2.5 seconds and 3.3 seconds for the third seismic test (4x full scale). The figure shows the amplification of motion in the structure. In all the seismic tests, there was an amplification of approximately three in the wall panel, and approximately 2.5 in the top concrete slab.

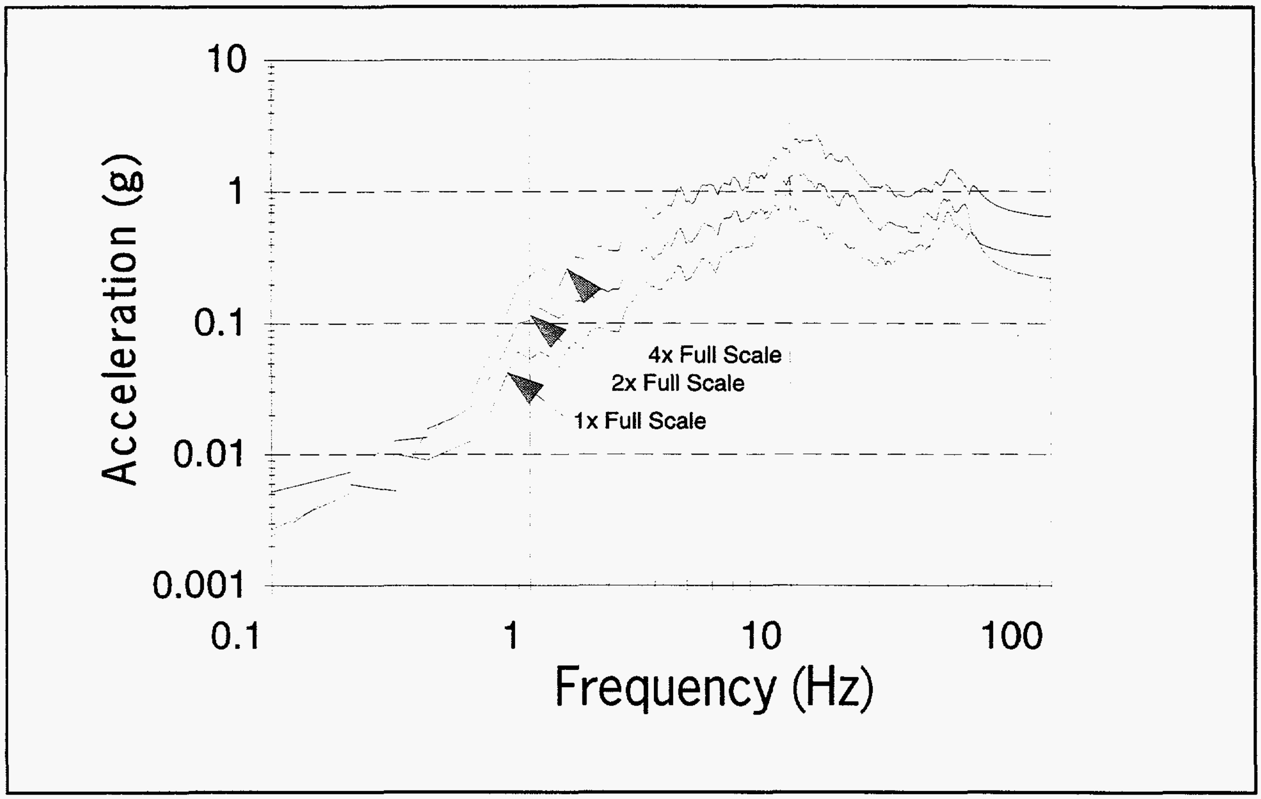

Figure 2: Out-of-plane acceleration response spectra

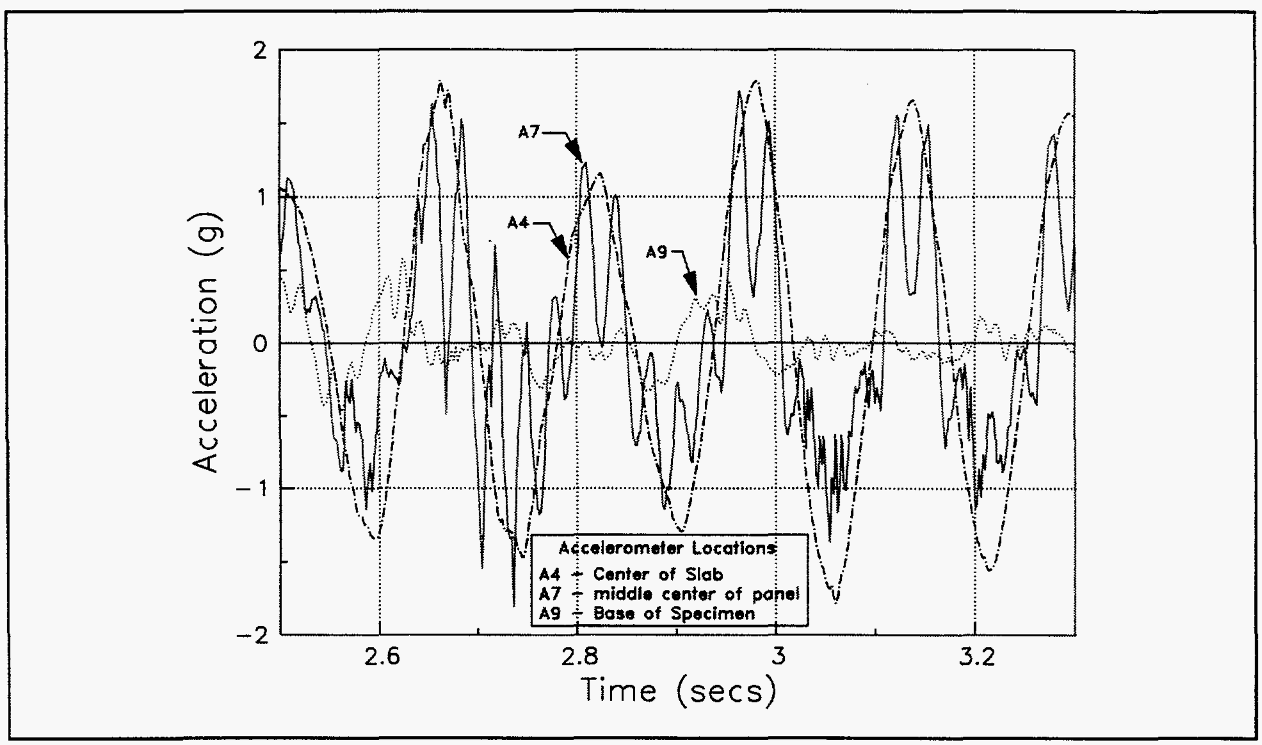

Figure 3: Accelerometer readings from the third seismic test

Figure 3 also indicates that the infill wall and the top slab were responding at different frequencies. This is more clearly seen in Figure 4, which is a plot of the transform functions between the slab base and the top slab, frame, and wall panel. The top slab and frame are responding at a frequency of 6.4 Hz. The infill wall panel was partially responding at the 6.4 Hz frequency, but the primary peak was at 31.7 Hz.

Figure 5 shows the maximum deformed shape of the columns and infill wall panel. The "kink" near the top of the columns is due to the truss framing in at this point. It is clear from this that the infill and the frame were moving separately. This was further confirmed from relative displacement measurements between the wall and the frame. The largest relative displacements were by far near the middle of the panel, but even these were small (0.058 inches for the third seismic test).

Based on the data, it is clear that the bond between the infill and the frame was broken. The bounding frame and the infill were responding separately, each at their own natural frequency. Infills subjected to out-of-plane earthquake motions thus are not expected to follow the frame motions throughout the height of the infill. However, the infill panel did remain stable, and there were no signs of any "walking" or loss of stability of the panel during the tests. Even during the third tests (peak acceleration of 0.76g), the panel remained stable. This is due to the arching of the panel. Geometrically, the panel is confined in the panel. Thus, although the infill panel responds at a different frequency from the frame, it is still believed that arching controls the actual capacity. Arching can give significant capacity to infills, generally on the order of several g (Gabrielsen and Kaplan, 1977; Mander et al, 1994).

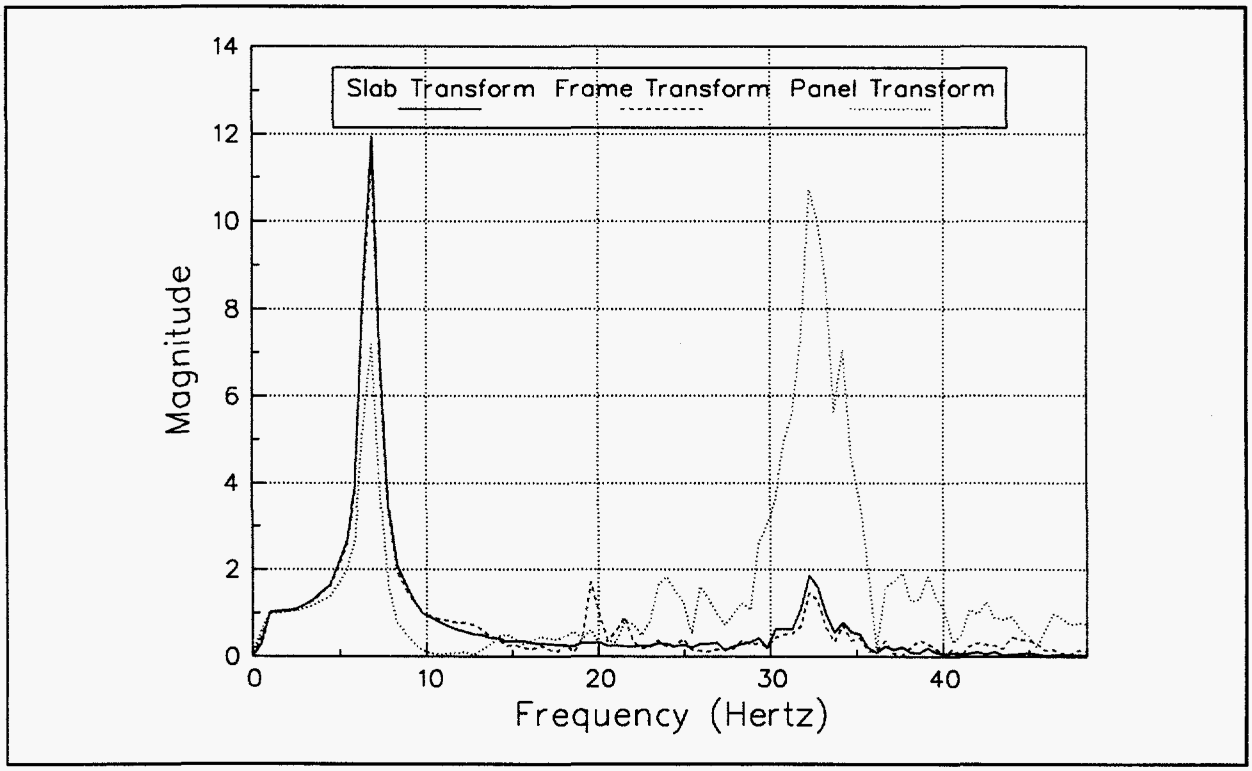

Figure 4: Transform functions from the third seismic test

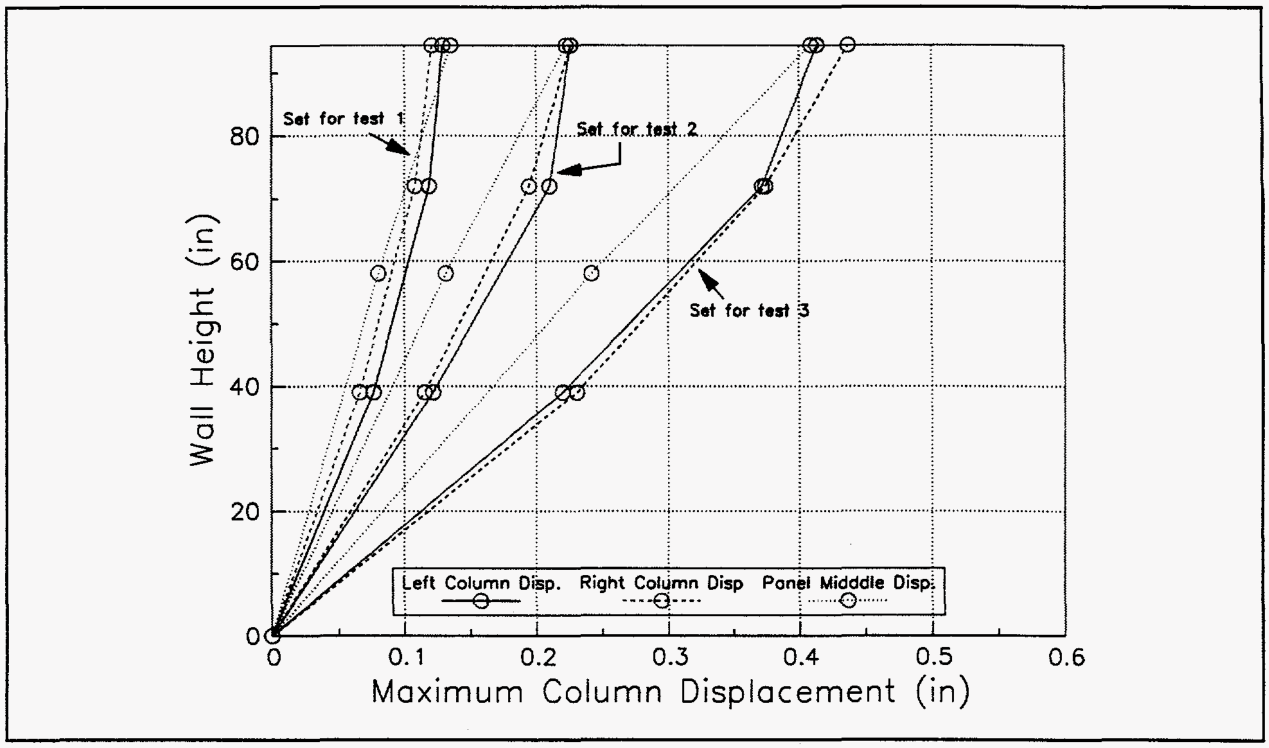

Figure 5: Maximum deformed shape of columns and panel

IN-PLANE SEISMIC TESTING

After completion of the out-of-plane tests, the specimen was rotated 90° on the shake table so that the infill walls could be loaded in the in-plane direction. Six seismic tests were performed with the structure oriented in the in-plane direction using the same site specific time history record that was used for the out-of-plane tests. These tests were performed at 1, 2, 4, 8, 12 and 16 times the full scale time history record. Seven low level random vibration white noise tests were performed to determine change in frequency of the structure as the testing progressed. A white noise test was performed at the beginning, and then following each seismic test. After completion of the tests, there was little damage to the structure. Some diagonal cracking through the mortar joints was noticed, but there were no tile failures.

The results of the in-plane testing were compared to previous static in-plane testing of similar panels (Barclay, 1993; Flanagan, 1994). For comparison, a "load"-deflection curve was developed, where the "load" for the dynamic testing was obtained as the lumped mass at the top of the structure multiplied by the acceleration recorded on the top concrete slab. Load-deflection loops for the maximum deformations in each test are shown in Figure 6. Figure 7 compares the load-deflection loop for the last seismic test (16x full scale) to that obtained from static testing.

Peak loads in the dynamic testing and static testing were similar, although there was much more damage to the infill at this point in the static testing. 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

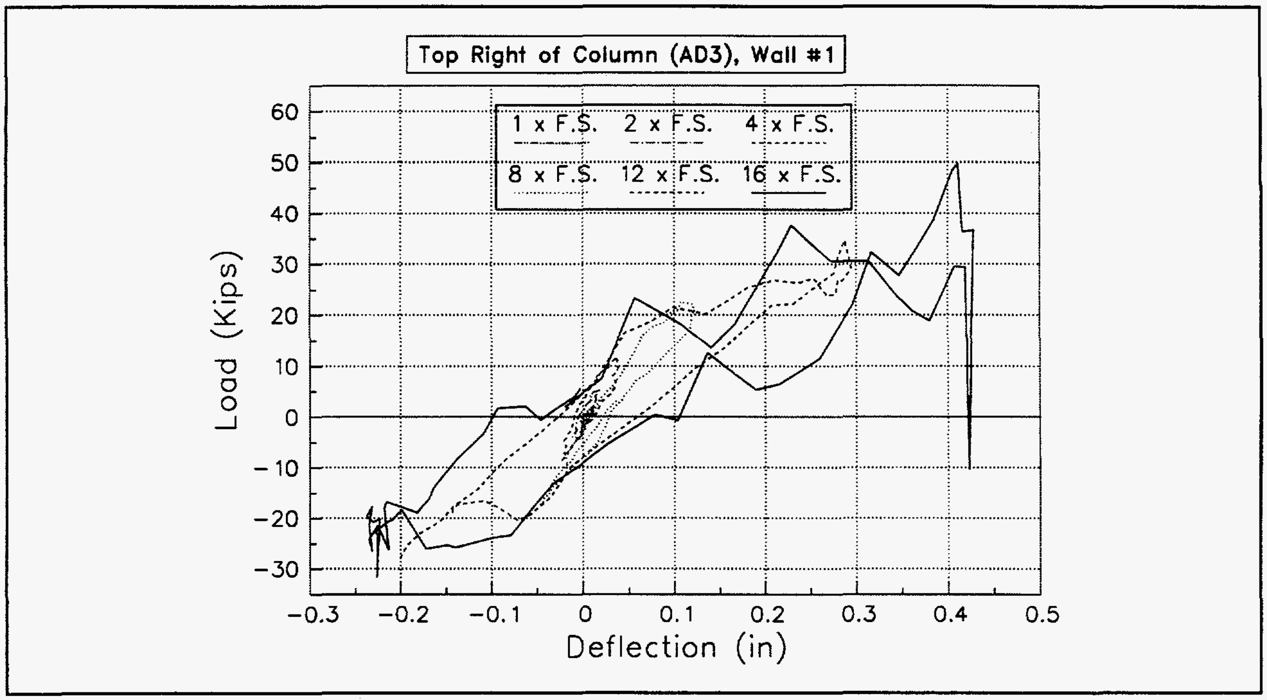

Figure 6: Load-deflection curve for seismic testing

used for the dynamic testing was a little stronger than those used in the static testing. The characteristic pinching of infill behavior was not as prevalent in the dynamic testing. Perhaps it would be more pronounced at higher excitation levels, or perhaps the vertical load from the top slab is causing a stiffer behavior.

Other aspects of the specimen behavior were similar to the static tests. Longitudinal strains along the diagonal reached a maximum of 0.00135, compared to longitudinal strains in the static test of 0.00145 (Barclay, 1993). These were similar to strains recorded at failure in prism compression tests (Boyd, 1993). Little vertical or horizontal strain was observed in the panel during the first four seismic tests. Apparently there had 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. Due to significant cracking and panel degradation, the panel was expanding both laterally and longitudinally within the frame during loading.

Column axial forces were also similar to that observed in static testing. A tension force was developed in the "windward" column almost as high as the load taken by the structure. This supports the common idealization in which the infill is considered as an equivalent strut bracing the frame. Interestingly, a tension force of about one-third that in the "windward" column was developed in the top of the "leeward" column (this was also observed in the static testing). As the loads increase during the cycle, high lateral strains develop normal to the compression strut causing an upward force at the column and beam interfaces, resulting in a tensile force in the top of the column.

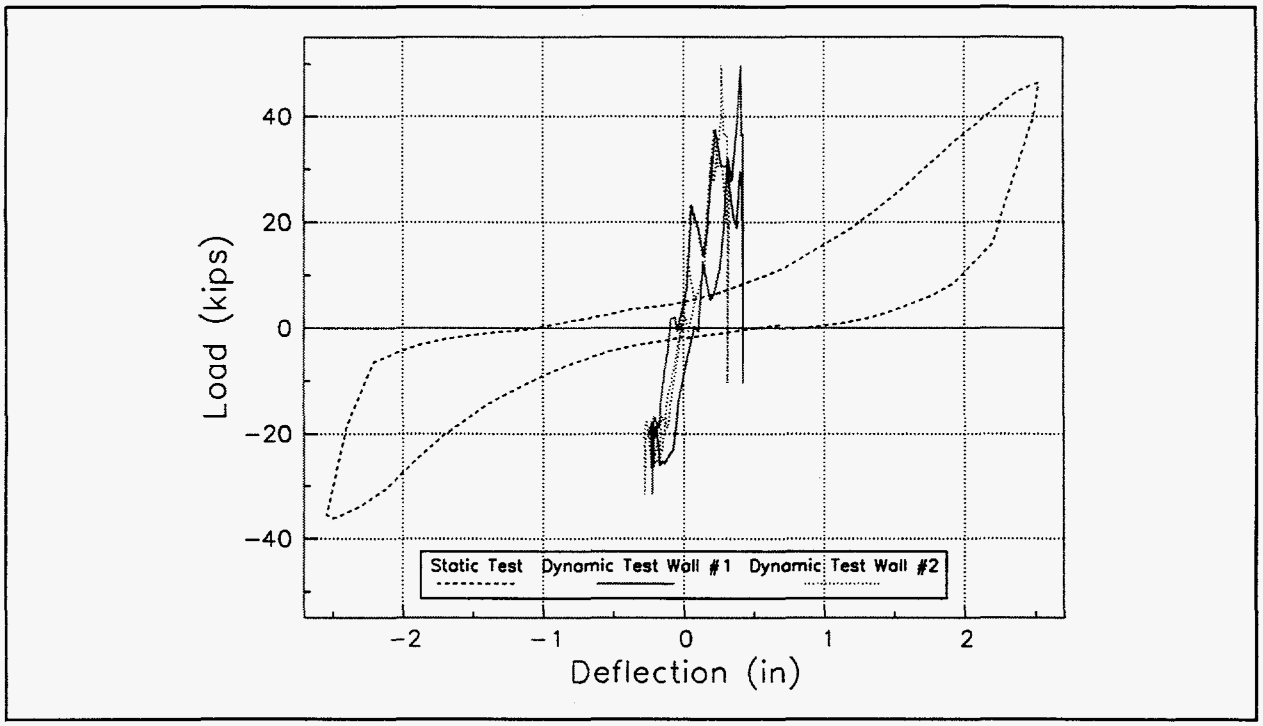

Figure 7: Static and dynamic load-deflection curve overplots

A high peak acceleration of approximately 13g was noticed in the columns during the last seismic test (16x full scale, peak ground acceleration of 3g). The peak acceleration in the infill was only about 60% of that of the column. A peak acceleration of approximately 6g was noticed in the top slab, or an amplification of approximately 2. Apparently the high peak accelerations in the column were due to the impact of the infill racking against the column after interface cracking. Due to the short duration impulse nature of the racking, these accelerations are not necessarily representative of actual column forces.

Table 1 shows the fundamental frequency of the structure obtained using a frequency transfer function on the response during the seismic tests and the low-level white noise tests. There is a noticeable difference between the two frequencies, particularly at the higher levels of excitation. This indicates the nonlinear nature of the response of infills. The implications are two-fold. First, low-level tests do not necessarily give an accurate picture of the natural frequency of an infill structure. Second, the stiffness of the structure is a function of the excitation level. Any analysis technique, such as using an equivalent strut, needs to account for this nonlinear behavior.

The observed natural frequency was compared to the predicted natural frequency using a piecewise linear strut formulation that had been developed based on static testing (Flanagan et al, 1994). The width of the equivalent strut is a reciprocal function of a parameter C, where the value of the parameter C is based on the deflection of the infill. For deflections similar to those seen in the white noise test, a value of C=5 is recommended. Using this results in a calculated natural frequency of 12.3 Hz, which is only slightly lower than that observed. Based on the observed deflections during the last seismic test, a value of C=11 is recommended. This results in a calculated frequency of 8.8

Hz, which agrees very well with the observed value of 8.8 Hz. Thus, the shake table tests validate the developed analytical model, and show the need for a nonlinear analysis.

Table 1: Results of frequency analysis

Seismic Tests		White Noise Test	
Test	Natural Frequency (Hz)	Test	Natural Frequency (Hz)
1x	16.6	Before 1x	16.0
2x	15.1	After 1x	16.7
4x	15.6	After 2x	15.8
8x	13.2	After 4x	16.2
12x	10.3	After 8x	15.0
16x	8.8	After 12x	13.3
-	-	After 16x	14.3

SINE SWEEP TESTS

After the out-of-plane and in-plane tests were completed, two in-plane sine sweep tests were performed on the structure. The first sine sweep test was performed at an amplitude of 1.0 g; 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 failed and 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" type failure.

The sine sweep tests were conducted to excite the structure at its natural frequency. Based on the low level white noise tests, the natural frequency was erroneously thought to be in the 10-20 Hz range. Actually, the structure's natural frequency was lower than this, and the structure was being excited at a frequency greater than its natural frequency during the first sine seep test. The amplification of the ground motion was approximately 2.2 at the beginning of the test. This decreased quickly, and the amplification was approximately 1.2 for the remainder of the test. The "load" on the structure at the beginning of the test was 22 kips, and dropped to 9.5 kips.

In an attempt to find the natural frequency, the second sine sweep test was started at 15 Hz, and the frequency gradually decreased. The lab technician stopped the test when tile began to fall out, but the natural frequency was not yet reached. Based on observations from the static testing, the infill still had significant capacity, and could have withstood further shaking. There was essentially no amplification of the ground motion in this test, since the excitation was significantly above the natural frequency. The "load" during the 3g sine sweep test was 38 kips. Thus, the structure was subjected to over 300 cycles of loading at approximately 80% of the static panel capacity. This far exceeds the loading that would be expected from any realistic seismic event.

CONCLUSIONS

Out-of-plane testing showed that infill panels do separate from their bounding frame, and respond at their own natural frequency during a seismic excitation. Due to arching, the panels remain stable. Infill panels generally perform quite well in the out-of-plane direction.

In-plane seismic testing showed similar behavior patterns to previous static testing. The natural frequency was adequately predicted using an equivalent strut analytical method. Nonlinear, or piecewise linear, methods are necessary as the strut stiffness degrades with increasing deflection. As evidenced by the sine sweep tests, infills are able to withstand significant shaking at near the ultimate capacity prior to failure. Infills can be expected to enhance the seismic resistance of otherwise laterally weak structures.

ACKNOWLEDGEMENTS

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