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Evaluation and Analysis of the Performance of Masonry Infills During the Northridge Earthquake

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Evaluation and Analysis of the Performance of Masonry Infills During the Northridge Earthquake

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ABSTRACT

Observations were made of the behavior of masonry infills in structural frames during the Northridge earthquake, and an analytical technique was developed for analyzing infilled frame structures. Infills near the epicenter suffered significant damage, but in several cases contributed to the seismic resistance and life safety performance. Older infill buildings in downtown Los Angeles experienced intensity of shaking similar to that expected in central/eastern United States earthquakes. The infills experienced some cracking, but otherwise complemented the lateral resistance of the weak building frames. This suggests infill frame buildings in moderate seismic zones may provide at least life safety functions without the need for expensive retrofit. A developed analytical technique was used to analyze two buildings for which the observed behavior and records from the Northridge earthquake were available. The analytical technique was based on using a piecewise linear equivalent strut for the infill. Parameters for the strut were obtained by examining the results of a wide variety of experimental infill tests. The strut method is easy to incorporate in standard linear analyses, and converges quite rapidly. The strut method was applied to two structures that had records from the Northridge earthquake. Very favorable comparisons between the analytical method and observed response were obtained. Recommendations were made concerning evaluation of the vulnerability of infills to earthquakes, and the construction of infills.

ACKNOWLEDEGMENTS

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CHAPTER 1

INTRODUCTION

Several older buildings in the high seismic hazard areas of California have unreinforced masonry infills, with newer construction having reinforced masonry infills. Masonry infill construction remains very popular, particularly in the low to moderate seismic zones of the eastern/central United States. Masonry infills have received much attention recently, as evidenced by two recent workshops focusing on masonry infill construction (MMES, 1993; Abrams, 1994). Infill walls may have both beneficial and detrimental affects on the building 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 and the 1992 Cairo, Egypt earthquake. The infills provided a redundant load path for both horizontal and vertical loads.

The performance of infills during earthquakes as described in the literature is summarized in Chapter 2. Observations of infill performance during the Northridge earthquake is also discussed in Chapter 2. Several cases exist of buildings near the epicenter where the infills contributed to the life safety functions of the building. Buildings in downtown Los Angeles experienced ground shaking similar to a moderate earthquake, typical of the central/eastern United States. Cracking and damage were observed, but the buildings remained functional.

A method for analyzing infill structures under earthquake loads is developed in Chapter 3. The method is based on replacing the infill with an equivalent piecewise linear strut. The strut properties depend on the deformation of the infill, and the stiffness of the strut is related to various limit states of the panel. Based on a comprehensive review of the literature, methods for predicting diagonal cracking strength and corner crushing strength are developed. The corner crushing strength is a function of the thickness of the infill and the infill compressive strength, but only a very weak function of other parameters.

In Chapter 4, a reinforced concrete building with clay masonry infills is analyzed. This building was 7 km from the epicenter, and experienced peak horizontal ground motion of 0.94g. Although damaged, the building did perform life safety functions. The analytical method was able to predict location and magnitude of damage, and handle the nonlinearity and post-peak behavior of the infills.

A 13 story steel frame building with exterior clay masonry infills in downtown Los Angeles is analyzed in Chapter 5. This building experienced a peak horizontal motion of 0.18g, which is typical of the intensity expected in the large moderate seismic regions of the central/eastern United States. The building suffered only minor damage. The building was analyzed using our best judgement of material properties and modeling was based on the method developed in Chapter 3. The response of this building during the Northridge earthquake was available from instrumentation of the California Division of Mines and Geology. The predicted analytical results were compared to the recorded response. No attempt was made to "tune" the model to the measured results, but rather the analysis was conducted based on best estimates. Good agreement was obtained between the analytical and recorded results, particularly with respect to the natural frequencies.

Conclusions are drawn in Chapter 6 regarding construction of new masnory infills, seismic walkdown guidelines for infill structures, expected infill performance in moderate seismic zones, analysis of infill structures, experimental testing of infills, and critical areas of research.

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CHAPTER 2

OBSERVED INFILL BEHAVIOR DURING EARTHQUAKES

General Behavior

Several patterns of infill behavior are apparent when examining their behavior under seismic loading. Many aspects of infill behavior are beneficial. Jafarzadeh (1992) noted the reduction of intensity of damages of infilled frames in the Richter magnitude 7.6 1990 Iran earthquake. Other aspects of infill behavior may be detrimental. Two large infilled frame motels, including one that had been open only 18 days, suffered some of the most serious damage during the 1993 Guam earthquake (Swan and Harris, 1993). Recent observations of infill behavior are reviewed in the following.

Infills serve to stiffen structural framing, thus reducing the natural period of the structure. Typically the shortened period results in higher seismic forces for firm sites. The additional stiffness and strength, though, can serve to limit seismic drift, keep the building in the elastic range, and more than compensate for the increased seismic forces (Bruneau and Saatcioglu, 1994a). Miranda and Bertero (1989) suggested that perhaps the most important factor in the generally good performance of low-rise reinforced concrete frame structures in the 1985 Mexico City earthquake was the presence of masonry infills. The increased strength and stiffness generally kept the buildings in the elastic range. The shortened period also resulted in reduced inertial forces in the 1985 Mexico City earthquake (Klingner et al., 1987), although this may vary with different soil conditions at the site.

Similar observations of the beneficial performance of infills were made after the 1992 Egyptian earthquake (Adham, 1994; Elgamal et al., 1993). This earthquake generated maximum ground accelerations of approximately 0.20g. The intensity of shaking could be considered similar to what would be expected in moderate seismic zones of the eastern and central United States. Due to the soft soil conditions, the stiffening effects of the infills proved beneficial in this earthquake. Despite low-quality construction, and no consideration of seismic design, infilled frame buildings performed quite well. Damage was mainly due to special circumstances. For example, the one infilled frame building that collapsed was a 14 story building designed for 8 stories.

Typical in-plane behavior of infilled masonry structures is cracking around the infill perimeter, diagonal cracking of the infill, and finally compression failures along the diagonal. Often the compression failure will be in the loaded corners, and result in face shell spalling, and loss of masonry. Both in-plane corner crushing and diagonal tension failures of masonry infills have been observed after earthquakes (Malley, 1994). The failure of infills causes the lateral forces and energy to be shifted to the frame, and may result in substantial building damage or collapse (Amrhein et al., 1973; Adham, 1994). In the process of being damaged, the infills, even if unreinforced, dissipate a significant amount of seismic input energy. This has in many cases prevented collapse of the building (Stone et al., 1987). A detrimental side effect of the infill damage and failure is that the debris can block exits and create a hazard around the exterior of the building (Bruneau and Saatcioglu, 1994a).

Infills provide a redundant path for both lateral and gravity loads. The top stories of a building in Guam shifted nine inches, shattering the ground floor columns, and leaving them unable to carry gravity loads.. Part of the building was left supported by the masonry infills (Swan and Harris, 1993). Several buildings were observed in the 1992 Egypt earthquake where there was crushing of reinforced concrete corner columns. The masonry infill provided an alternate system for carrying part of the vertical loads after column crushing (Adham, 1994).

Since infills are generally not designed as structural elements, their effect is often ignored in the design phase. The placement of infills can lead to poor seismic configurations. Often infills are terminated at the lower level, causing a soft first story. This problem has been noted in several earthquakes. Thiruvengadam and Wason (1992) and Mallick (1984) describe buildings whose first floor completely collapsed due to the absence of infills or other lateral bracing, while the upper stories remained intact and fell as a rigid body, with no member overstressed. There was evidence of large sway deflections in the first floor prior to collapse. Infills may also be asymmetrically placed, leading to considerable torsion in the structure (Swan and Harris, 1993; Stone, et al., 1987; Saatcioglu and Bruneau, 1993).

Partial height infills can be quite detrimental to the performance of framed structures. Partial height infills can result either from initial construction, such as a wall with windows at the top, or can result from corner crushing, and loss of the top part of the infill. Partial height infills create short effective column heights which attract a high proportion of the load. The short columns often fail in shear, particularly if constructed of reinforced concrete (Berg and Hanson, 1973).

The 1992 Erzincan earthquake provided a good basis for observing infill behavior during a strong motion earthquake (Malley et al., 1993). Most of the commercial building stock was reinforced concrete frames with unreinforced masonry infills. Most of the masonry was hollow clay tile, with a few buildings with clay brick and concrete masonry infills. The one strong motion instrument recorded peak accelerations of 0.40g and 0.49g in the horizontal direction, and 0.25g in the vertical direction, with a 5-6 second duration of strong shaking (Malley, 1994). Typical problems with infills, such as soft first stories, and short effective column lengths from partial height infills were observed. In many cases, the infills were heavily damaged, but appeared to provide enough strength and ductility to perform life safety functions. Two hotels that collapsed were reported to have few infills and tall, open first stories.

The performance of hospitals during the 1992 Erzincan earthquake provides an interesting basis for the study of infills (Malley et al., 1993). The first floor of one wing of the Military Hospital which did not have infills collapsed. The upper four floors, which had infills, remained intact. The adjacent wing, which had infills on the first floor as well as the other floors, remained standing. Shear failures were observed in columns, indicating a significant contribution of the infills to the lateral resistance. At the Social Security hospital, a newer L-shaped wing completely collapsed. Torsion and force transfer problems at wide shallow beam connections contributed to the collapse. The original hospital suffered severe cracking to the longitudinal infills, but remained standing. At State Hospital, the newest building suffered little damage. The main building suffered severe damage to infills on the first floor, but remained standing. It was reported that this building suffered similar, but less severe, damage during the 1983 Erzurum earthquake and had been repaired. In summary, the infills were unable to provide sufficient seismic resistance for the hospitals to remain useable after the large earthquake. This severely hampered emergency medical care. The infills were able to prevent buildings from collapsing, thus providing life-safety functions and reducing the number of casualties.

Some of the most severe infill damage was experienced during the 1993 Guam earthquake (EERI, 1995). Although there were no ground motion records from this earthquake, maximum ground shaking was estimated to be between 0.15g and 0.25g. Two major motels (12 story and 4 story) consisting of reinforced concrete frames with reinforced concrete masonry infills were damaged to the point that they were subsequently demolished. Infill masonry damage consisted of large diagonal cracking, spalling of masonry face shells, and collapse of entire panels. In a smaller two-story structure, the concrete masonry faceshells of an infill spalled off leaving grout plugs hanging like icicles. Much of the damage is due to the infills, as they created poor seismic configurations. These include soft-stories where the infills are terminated, short effective column lengths from partial height infills, and torsion from the infill frame stiffness shifting the center of rigidity of the building. Infills in this earthquake appeared to do more harm than good. After creating problems due to their placement, the infills did at least perform life safety functions. One interesting aspect that was not discussed in the reconnaissance report was the behavior of infills with openings. A photo from one of the motels appears to show an infill with a door opening on one side. Diagonal/shear cracking appears to be present consistent with that observed in laboratory tests (Dawe and Seah, 1989).

Experimental testing has indicated that most infills have significant out-of-plane capacity due to arching (in-plane membrane forces) provided the boundary conditions are able to resist the in-plane thrust. Except for panels with high height to thickness ratios, infills should in general have acceptable out-of-plane behavior during seismic events. Very few out-of-plane infill failures have been reported during seismic events. For example, Thiruvengadam and Wason (1992) describe several buildings where unreinforced infills experienced diagonal cracking and "separation" (apparently cracking at the frame/infill interface) from the surrounding frame. However, in the three buildings described, no mention was made of out-of-plane damage or failures, and no restrictions were made on the subsequent use of the buildings.

Widespread out-of-plane damage to "thin" unreinforced masonry infills was noted in the 1990 Iran earthquake (Mehrain, 1990). A building with light steel sections that reduced the wall span for out-of-plane bending performed better, as well as structures with "thick" infills. This appears to confirm the importance of the height/thickness parameter in developing out-of-plane strength through arching.

The need for confinement of masonry infill panels was apparent in the 1992 Erzincan earthquake (Bruneau and Saatcioglu, 1994b; Malley, 1994). Widespread out-of-plane failures of gable end walls above the main roof slab were observed, while the infilled masonry panels below would remain stable. Most of the out-of-plane failures were due to prior in-plane damage. Damage around the periphery of the infill would cause gaps between the infill and the bounding frame, leading to out-of-plane movements, and possibly panel failures (Saatcioglu and Bruneau, 1993). Hollow clay tiles oriented with cells perpendicular to the wall at the perimeter formed a weak plane. Upon in-plane failure, the remaining panel failed out-of-plane as a unit (Malley, 1994). In some cases, in-plane failures would only cause partial out-of-plane failures, and a portion of the infill would remain intact (Bruneau and Saatcioglu, 1994b).

Northridge Earthquake Observations

Infill performance during the Northridge earthquake will be discussed in terms of three categories. First, near the epicenter, there were several buildings with double wythe reinforced clay brick infills. Second, also near the epicenter, several parking garages had infills, typically concrete masonry. Third, a number of older buildings in downtown Los Angeles with unreinforced infills (brick and clay tile) experienced some damage.

Several notable examples of double wythe reinforced clay brick infills existed near the epicenter. These were the Cal State Northridge Dormitory, and Buildings 3 and 40 at the Sepulveda Veterans Administration Hospital (TMS, 1994). Typical construction was a 9½" wall consisting of two 3½" clay brick wythes, and a 2½" grout space containing the reinforcing. The exterior wythe was typically outside the structural framing. Damage was observed in all these buildings. Part of the damage appeared to be typical infill damage, consisting of diagonal cracking and corner crushing. In several cases, the outer wythe appeared to have been inadequately tied to the interior wythe.

Building 40 at the Sepulveda VA hospital is a one story steam plant building with a reinforced concrete frame, Figure 2.1. Records at the base showed peak accelerations of 0.75g in the east-west direction, 0.94g in the north-south direction, and 0.45g in the vertical direction. This building was constructed in 1955. It appeared well designed, but some construction deficiencies were apparent. Concrete had spalled off of one concrete column near the base revealing about 15" of vertical reinforcing, Figure 2.2. No ties were present in this length, despite plans calling for #3 ties at 10". There was also a corner of the infill that had been damaged revealing the grout core, Figure 2.3. One place was observed in which there was a large void in the grout leaving some reinforcing with no grout around it. Nevertheless, this building survived the large ground shaking to which it was subjected. The building had visible damage at numerous locations but did not suffer collapse. Therefore, life safety was achieved and the building still has limited use. However, it will eventually be replaced. The infills had a positive beneficial effect on the performance of this structure.

Several parking garages in the Northridge area were observed which had reinforced concrete frames with concrete masonry infills. In one instance, these were separated from the concrete frame by a half-inch to one-inch joint filled with styrofoam, Figures 2.4-2.5. Obviously the intent was to isolate the infill from the frame, so that there would be no interaction. Isolating the infill leads to other problems. The infill must be adequately anchored out-of-plane, as arching cannot develop. If the gap is not large enough, the columns will engage the infill, and it will interact with the structure (Flanagan, 1994). Yanev and McNiven (1985) observed in dynamic testing that the buffeting that occurs when the frame engages the infill quickly destroys the infill. They tried using foam rubber as a filler in the gap to minimize the impact effects, but it had minimal effect. Thus, in designing isolated infills, the out-of-plane anchorage and the provision of a sufficient gap (½" may be inadequate since it only allows for approximately 0.5% drift) need to be considered.

Other parking garages were observed which had concrete masonry infills tightly fitted against the framing, Figure 2.6. These would then form part of the lateral load resisting system, and may have limited damage, or in some cases prevented collapse. Figure 2.7 shows an infill where there was cracking around the infill perimeter and some cracking in the concrete frame. One parking garage was observed which had out-of-plane supports at the top of a tightly fitted infill consisting of large steel angles anchored to the infill and underside of the beam. In terms of resisting out-of-plane motions, the angles were superfluous, as the arching capacity would greatly exceed the capacity of the angles. At most, only a nominal connection would be needed to insure the initiation of arching, and the prevention of walking of the infill. Cracking was noted around the anchors from in-plane movements. Dawe and Seah (1989) observed in experimental testing that ties between a steel column and a concrete masonry infill actually slightly reduced the in-plane capacity. The ties resulted in extensive off-diagonal random cracking of the infill, and prevented the compression diagonal from fully developing. Presumably, anchors at the top would have the same effect, and possibly more drastic. If the angles caused premature failure of the upper course of the infill, the behavior would be similar to an infill with a top gap. This leads to significantly lower capacities, and the increased likelihood of shear failures in the columns. It is thus believed that out-of-plane anchors of tightly fitted infills may do more harm than good, and are not necessary.

Several buildings in downtown Los Angeles had a primary lateral load resisting system of unreinforced infills. These buildings were constructed early in the 1900s and typically had clay brick or clay tile infills.. One notable example is the City Hall building which has steel framing with unreinforced clay tile infills. Ground motions in the downtown area were in the range of 0.15-0.20g during the Northridge earthquake. This is typical of the level of excitation expected in many of the large moderate seismic zones in the eastern and central United States. Damage was noted in many of the infills, primarily consisting of diagonal cracking or cracking along the infill boundary. Figures 2.8-2.9 show damage near building corners. Diagonal cracks are shown in Figures 2.10-2.11.

Despite the damage, the buildings remained useable and stable after the earthquake. Seismic analyses of unreinforced infill buildings in moderate eastern and central United States zones have indicated diagonal cracking, and in some cases the initiation of corner crushing, could be expected

(Flanagan et al., 1994). Thus, in a general qualitative sense, the behavior of the unreinforced infills was similar to that predicted by analyses.

The behavior of the infill buildings in downtown Los Angeles has significant impact on the eastern and central United States. A large building stock exists which has not been designed for any seismic load, but for which we know there is a risk of seismic activity. Many of these buildings only have a minimal lateral force resisting system apart from the unreinforced masonry infills. Adequately constructed infills should be capable not only of providing life safety functions, but should also in most cases have enough strength so that the buildings are useable after a moderate earthquake. Laboratory investigations have shown that repaired infills have close to the same strength as virgin infills (Flanagan, 1994), implying that infill buildings damaged during moderate earthquakes can be economically repaired. Thus, many of the infill buildings in the large moderate hazard seismic zones in the central and eastern United States need not be retrofitted. They have sufficient seismic strength as is.

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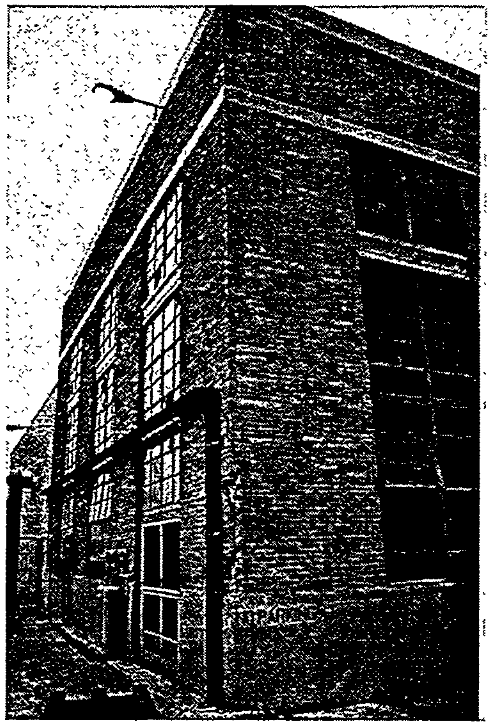

Figure 2.1: Sepulveda VA Hospital
Building 40

Figure 2.2: Column Reinforcement

Figure 2.3: Infill Grout Core

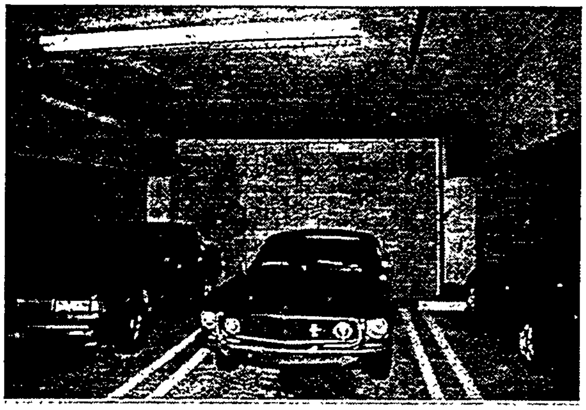

Figure 2.4: Parking Garage

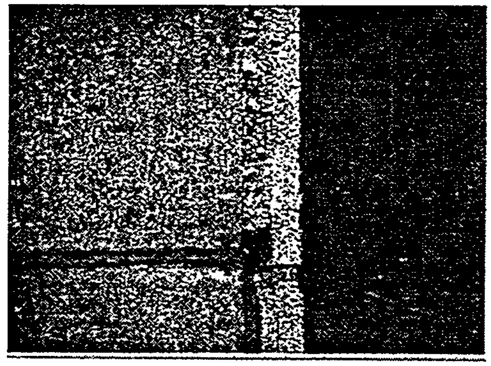

Figure 2.5: Isolated Panel

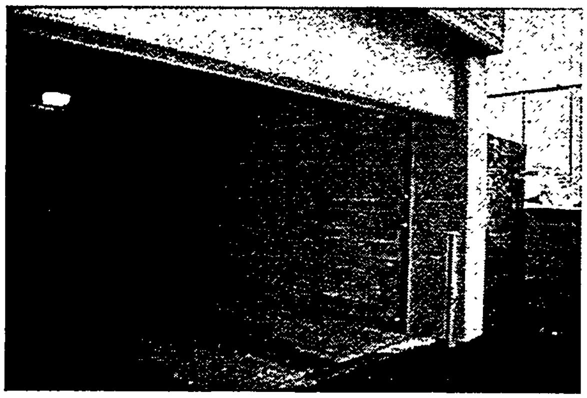

Figure 2.6: Parking Garage

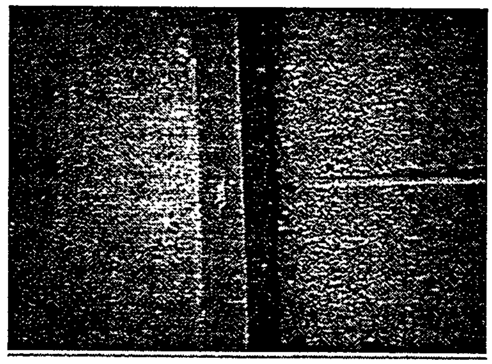

Figure 2.7: Infill Contact

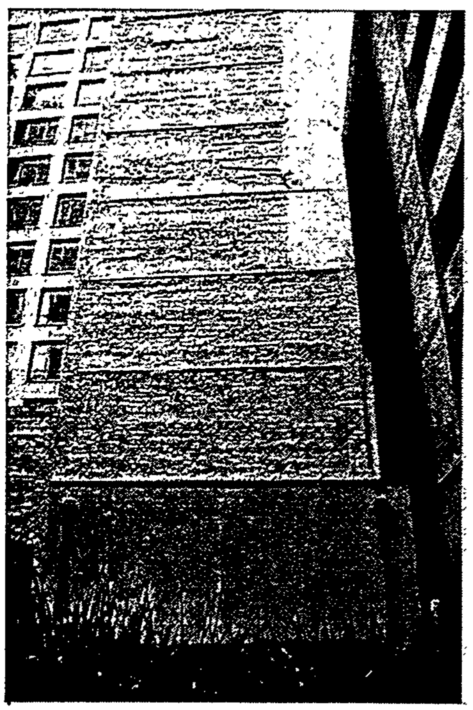

Figure 2.8: Los Angeles Building

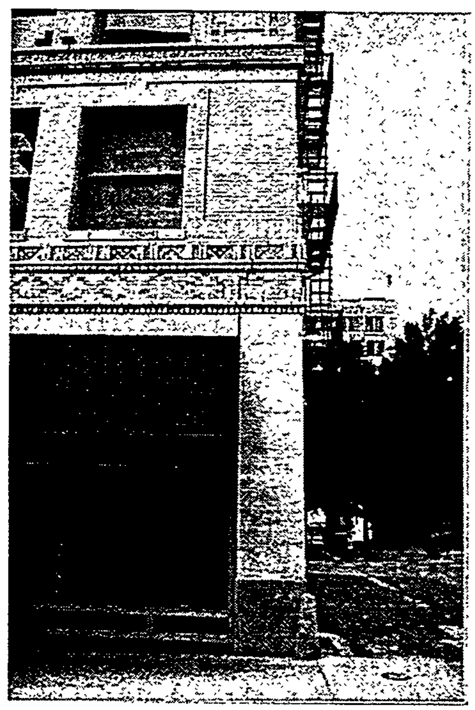

Figure 2.9: Pasadena Building

Figure 2.10: Pomona Building

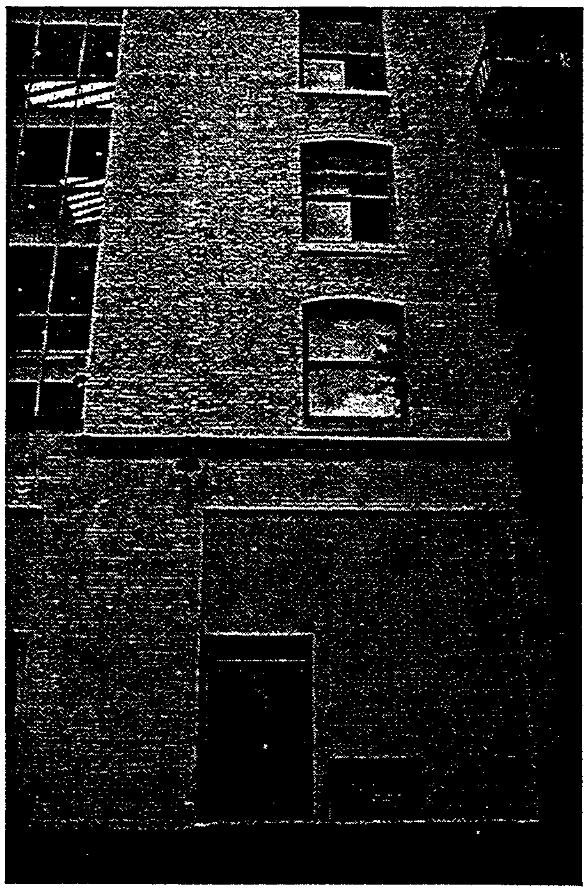

Figure 2.11: Pasadena Building

CHAPTER 3

EQUIVALENT STRUT ANALYTICAL METHOD

Introduction

Building analysis techniques for infill structures need to be computationally efficient, and readily integrated into existing analysis techniques and methods. A piecewise linear strut is chosen for modeling the in-plane behavior. The material properties of the strut are kept constant while the area of the strut is varied based on the lateral drift. The decrease in strut area with increasing drift is also qualitatively related to expected damage levels of the infill, such as diagonal cracking, and corner crushing. Several iterations of a linear analysis are required, but convergence is quite rapid (Flanagan et al., 1994).

The replacing of the infill with the strut enables large, complex three-dimensional analyses to be conducted. The strut adequately captures the global behavior of the structure, such as stiffness, natural frequencies, and torsion. The method thus accommodates standard techniques for dynamic analysis (either equivalent static forces or response spectra methods) while incorporating the nonlinear behavior of masonry infills in an effective manner.

The strut formulation used is similar to that developed by Stafford-Smith and Carter (1969). The width of the equivalent strut, w, is

$$w=\frac{\alpha}{\cos\theta} \tag{1}$$

in which α is the length of column bearing on the infill and θ is the angle of the diagonal with respect to the horizontal. The length of column bearing, α, is determined as

$$\alpha=\frac{\pi}{C\lambda} \tag{2}$$

in which C is an indicator of the limit state of the infill and varies with the in-plane drift displacement, and λ is a parameter relating the infill stiffness and the frame stiffness. The parameter λ is determined as

$$\lambda=\sqrt[4]{\frac{E_m t\sin 2\theta}{4E_c I_c h'}} \tag{3}$$

in which E_m is the gross elastic modulus of the masonry, E_cI_c is the flexural rigidity of the columns, t is the infill gross thickness, and h′ is the infill height.

The capacity of the infill is limited by both by a displacement and a force based criterion. For softer, more flexible frames, the displacement criteria will control, while the force criteria will control for stiffer frames.

Suggested values for the parameter C for various types of infill material and both concrete and steel frames are given in the following sections. Various modeling issues are also discussed.

Stiffness Characteristics for Different Materials/Frame Types

Structural Clay Tile in Steel Frames

The development of the analytical approach outlined in the previous section was initially made for steel frames with structural clay tile infills (Flanagan, 1994). It was the result of a large, comprehensive testing program. The clay tile used for the tests were 12"x12" units with horizontal coring. Both 8" and two-wythe 13" walls were tested. Tests were conducted with varying frame stiffness, varying aspect ratios, varying sizes, with corner openings, and with the infills offset from the framing. The criteria is currently being used by the U.S. Department of Energy for seismic evaluation of their facilities. The criteria has undergone several peer reviews, and been refined as a result of these reviews. Table 3.1 gives the value of C for varying in-plane drift displacements.

Table 3.1. Values of C for Steel Frames with Structural Clay Tile Infilling

C	Displacement (in)	Typical Infill Damage
5	0.0-0.05	None
7	0.05-0.2	Diagonal Mortar Joint Cracking
11	0.2-0.4	Off Diagonal Mortar Joint Cracking
14	0.4-0.6	Banded Diagonal Mortar Joint Cracking
16	0.6-0.8	Corner Mortar Crushing and Tile Cracking
18	0.8-1.0	Tile Faceshell Splitting (Primarily Corner Regions)

Concrete Masonry in Steel Frames

One of the most comprehensive set of tests conducted on steel frames with concrete masonry infills is summarized in Dawe and Seah (1989). A total of thirty-four tests were performed, examining such aspects as openings, reinforcement, top gaps, and interface conditions. Of the thirty-four tests, fifteen were analyzed that were considered to be basically "standard" frames. Most of tests were limited by deflection to preserve the frame for future tests. Thus, the true ultimate behavior, and the post-peak behavior was not always obtainable. However, the one test that was carried well past ultimate demonstrated a gradual reduction in load carrying capacity, indicating significant ductility and energy absorption capability.

Values of C verses the in-plane displacement are shown in Figure 3.1 Figure 3.2 shows the results of a piecewise linear regression. For ease of use in application to building analysis, suggested values of C that would be applicable over a given displacement range are given in Table 3.2. Typical expected damage for different displacements is also given in the table.

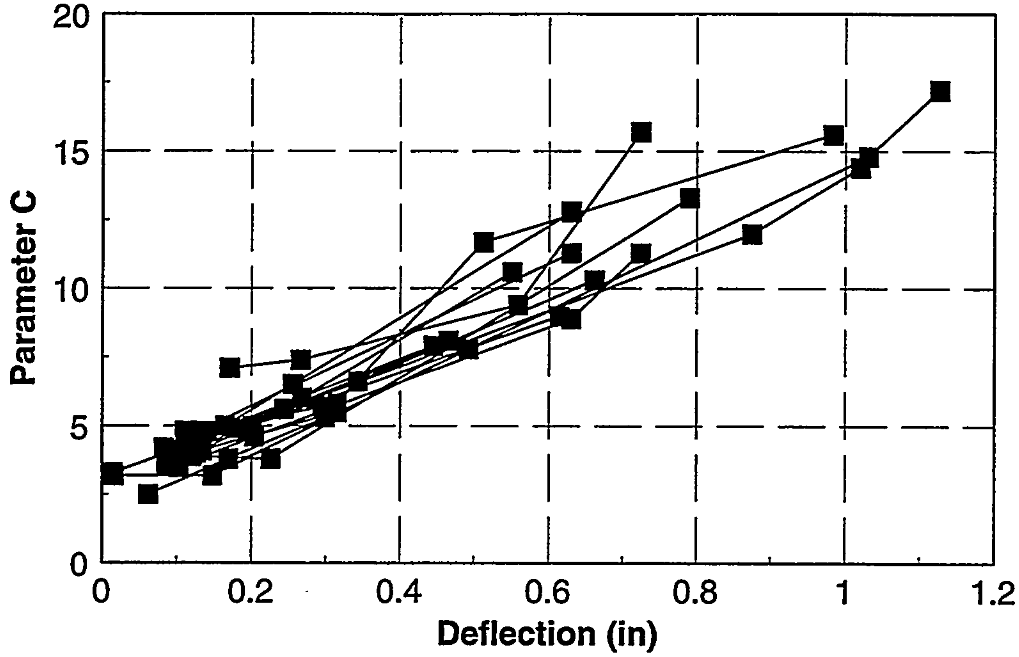

Figure 3.1 Parameter C versus Deflection for Steel Frames with Concrete Masonry Infills

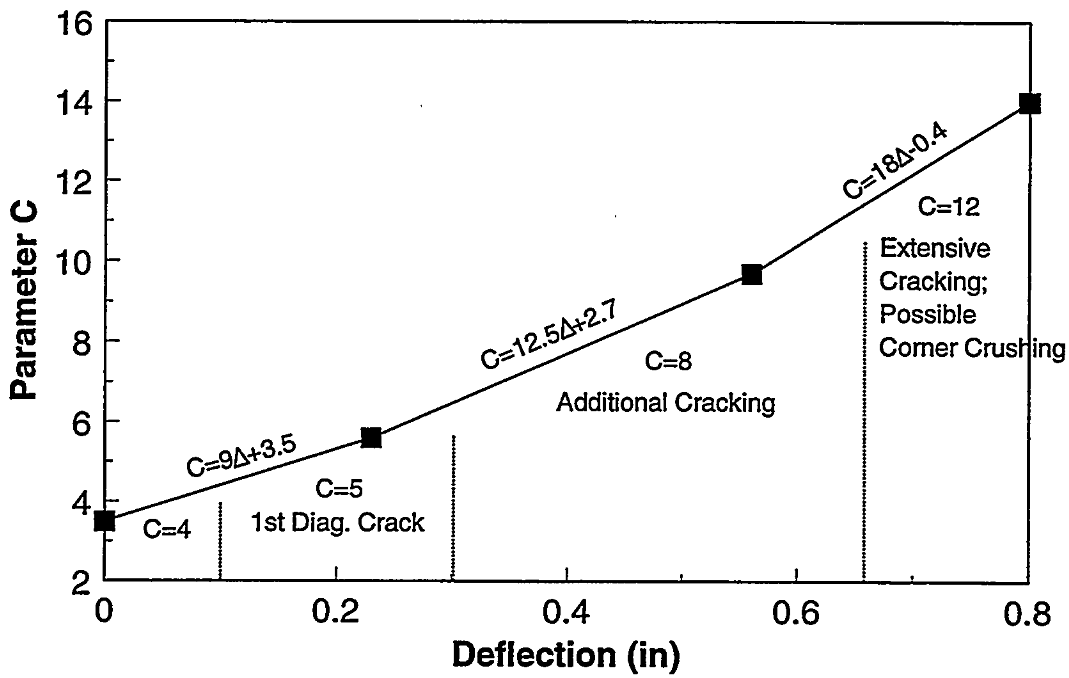

Figure 3.2 Summary of Parameter C versus Deflection for Steel Frames with Concrete Masonry Infills

Table 3.2. Values of C for Steel Frames with Concrete Masonry Infilling

C	Displacement (in)	Typical Infill Damage
4	0.0-0.1	None
5	0.1-0.3	Diagonal Mortar Joint Cracking
8	0.3-0.65	Off Diagonal Mortar Joint Cracking
12	0.65-0.8	Extensive Random Cracking; Possible Corner Crushing

Concrete Masonry in Concrete Frames

Mehrabi et al. (1994) reported on a series of half-scale tests of concrete masonry infills in reinforced concrete frames. Both solid and hollow masonry units were used, with all the masonry being unreinforced. Twelve tests were conducted on single bay frames, and two tests on two bay frames. The tests showed significant post-peak strength, and a significant amount of ductility.

Figure 3.3 illustrates the dependence of C on the in-plane displacement for concrete frames with concrete masonry infilling. Figure 3.4 is a summary of a piecewise linear regression of C versus the in-plane displacement. There is a slight difference between solid and hollow concrete masonry. However, the difference is small enough that for most practical analyses it can be neglected. Table 3.3 gives recommended C values and corresponding expected damage. This table would be applicable for both solid and hollow concrete masonry.

Table 3.3. Values of C for Concrete Frames with Concrete Masonry Infilling

C	Displacement (in)	Typical Infill Damage
2	0.0-0.1	None
4	0.1-0.25	Diagonal/Sliding Mortar Joint Cracking
8	0.25-0.45	Off Diagonal Mortar Joint Cracking; Bed Joint Sliding; Corner Crushing

Brick Masonry in Concrete Frames

Benjamin and Williams (1958) tested concrete frames with clay brick infills. Most of the tests were small scale, and appeared to fail primarily through diagonal cracking and sliding. Based on the one full scale test, approximate values of C would be 2 for a deflection of 0.1", C=4 for a deflection of 0.15", C=8 for a deflection of 0.25", and C=10 for a deflection of 0.35". Although care needs to be exercised in extrapolating from one test, it appears that values of C are similar for concrete masonry and clay brick masonry in concrete frames.

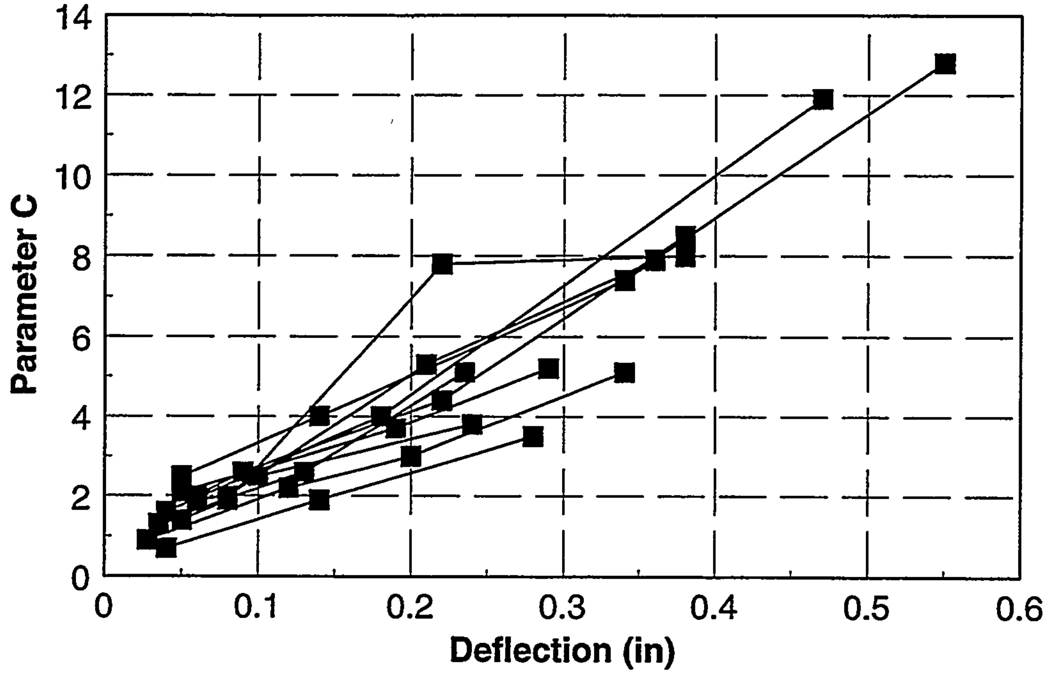

Figure 3.3 Parameter C versus Deflection for Concrete Frames with Concrete Masonry Infills

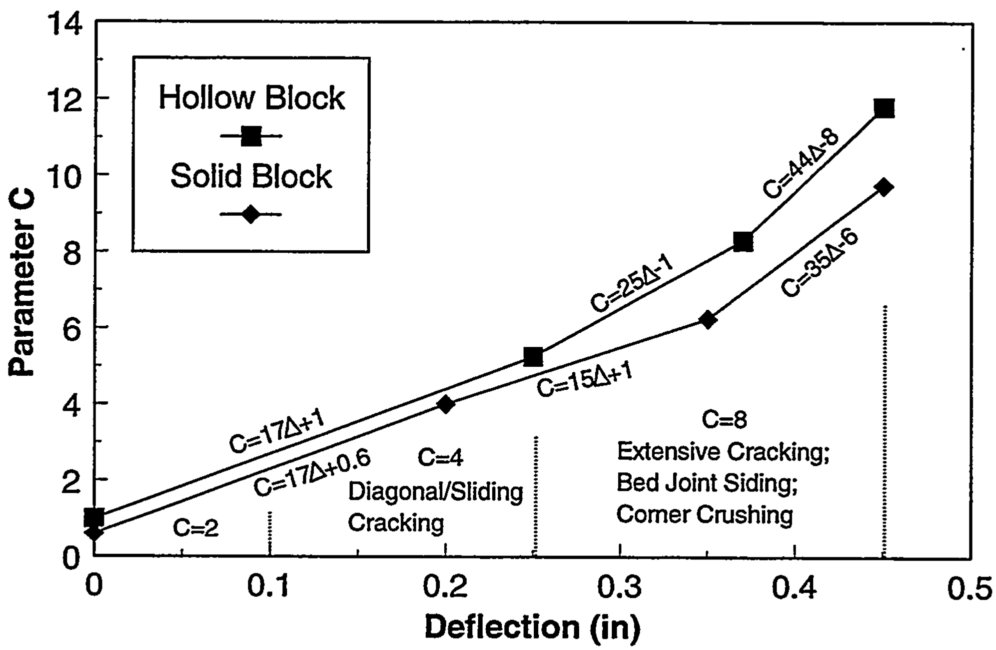

Figure 3.4 Summary of Parameter C versus Deflection for Concrete Frames with Concrete Masonry Infills

Tests with Flexible Steel Frames

Several test results are available with quite flexible steel frames. Dawe and Seah (1989) tested a frame with true pins at the connections. The stiffness was considerably lower than frames with moment connections. Benjamin and Williams (1958) tested brick infills in a pinned steel frame. Values of C for these tests ranged from 10-100. These tests were disregarded in the development of the stiffness formulation.

Hendry and Liauw (1994) and El-Ouali et al. (1991) tested steel frames with various types of masonry infill. Instead of using a fairly stiff base, such as a laboratory strong floor, they used a steel beam identical to the top beam. The rectangular frame was supported only at the bottom corners. This resulted in a much softer response. Values of C obtained were again on the order 10 to almost 100. Due to the flexibility, these tests are also not used in the development of stiffness formulation.

Clearly, the flexibility of the frame can drastically affect the stiffness. It is felt that a truly pinned frame is not representative of typical construction. Some nominal moment capacity will be present, even with shear connections. It appears that even a nominal amount of moment will stiffen the infill-frame response (Flanagan, 1994). The stiffness of the supporting base for the infill also apparently has a large affect on the system stiffness. It is felt that a fairly rigid base, such as a laboratory strong floor is more representative of field conditions. Typically the critical infills are at the base of the structure. For upper levels, the floor slab will add some additional stiffness, as well as the infill on the floor below. Thus, the tests that showed quite flexible results, with large values of C, are not included in the development of the stiffness formulation.

Ultimate Strength of Infill

There are two primary failure modes for infills, diagonal cracking and corner crushing. In addition, there are several secondary limit states, as indicated in Tables 3.1-3.3. The load-deflection behavior of a typical infill is shown in Figure 3.5

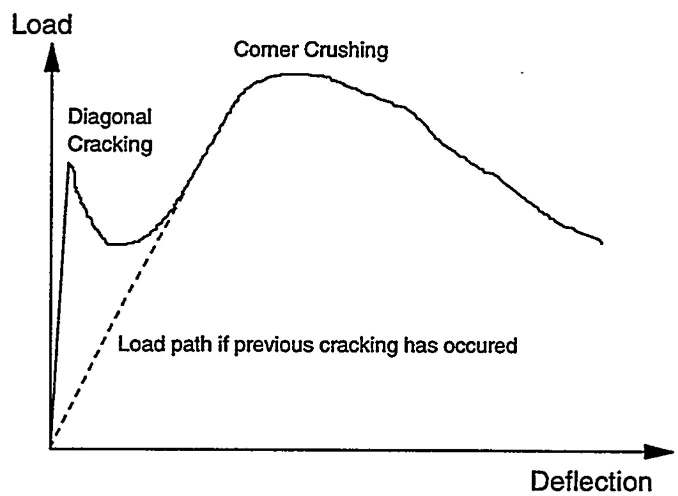

Figure 3.5 Typical Behavior of an Infill

The behavior of an infill is analogous to a reinforced concrete beam. Prior to diagonal cracking, the intact infill tends to be quite stiff. This corresponds to the uncracked behavior of a reinforced concrete beam. Considerable stiffness is lost after diagonal cracking, and there may be a drop in load capacity immediately after diagonal cracking, analogous to cracking of the beam. Due to the confinement of the infill, it will continue to carry load until either corner crushing occurs, or there is a failure in the bounding frame. Depending on the properties of the infill, the ultimate load in corner crushing may be greater than or less than the diagonal cracking load. This corresponds conceptually to the behavior of a reinforced concrete beam with greater than minimum reinforcing and less than minimum reinforcing, respectively.

Once the infill has cracked, its behavior is much softer, following the dashed line in Figure 3.5. Cracking could occur from the cyclic load caused by a seismic event. Prior cracking can occur from shrinkage or moisture expansion movements, thermal movements, or out-of-plane loads. For most existing infills, particularly in older buildings, the infill may often have minor cracking prior to an earthquake. Thus, it is felt that the stiffness used for the infill should reflect the stiffness after diagonal cracking. Methods for estimating the cracking load and corner crushing load for infills are given in the following sections.

Diagonal Cracking Load

Diagonal cracking appears to be related to the panel size, and potentially the square root of the compressive strength, f'_m. Table 3.4 lists diagonal cracking stresses for various types of infills. The diagonal cracking stress is based on the net bedded area.

Table 3.4 Diagonal Cracking Stresses for Infills

Infill Masonry Type	Diagonal Cracking Stress	Value of f'_m used
Structural Clay Tile	$\sqrt{f'_m}$	Parallel to bed joint
Concrete Masonry		Perpendicular to bed joint
Steel Frames	$3\sqrt{f'_m}$	
Concrete Frames	$5\sqrt{f'_m}$	
Clay Masonry		Perpendicular to bed joint
Steel Frames	$2.3\sqrt{f'_m}$	
Concrete Frames	$3.2\sqrt{f'_m}$	

Values for structural clay tile were determined from the tests by Flanagan (1994). Perhaps these are lower than for other types of masonry since the bed joints were solid. Also, there was significant horizontal coring and only face shell mortar in the head joints. The full bedded area (gross area) was used in determining the diagonal cracking strength. However, the value of f'_m for loading parallel to the bed joint was used to reflect the large horizontal coring of the clay tile.

The value for concrete masonry in steel frames was determined from the tests by Dawe and Seah (1989). The coefficient of variation for this result was 20%. Applying this to the tests of Hendry and Liauw (1994) gave an estimated cracking load of 52 kips. The actual cracking load was 58-63 kips, or about 15% higher than predicted.

A note is in order on the tests by Dawe and Seah (1989). The prism strengths reported seemed to be high, being about twice that of Mehrabi et al. (1994), and about twice that found in much of the United States. Indeed Dawe and Seah's (1989) prism strengths were approximately equal to the unit strength. The prism strength values reported by Dawe and Seah (1989) were used in all calculations. However, if the prism strength were actually lower, the coefficient would tend to be higher for determining the cracking stress.

The value for concrete masonry in concrete frames was determined from the tests by Mehrabi (1994). The coefficient of variation was 35% for this value. The net frame load (system load minus bare frame load at same deformation) was used since the concrete frame was carrying approximately 10% of the load. The value of $5\sqrt{f'_m}$ is high compared to tests reported by Angel et al. (1994). For their two frames with concrete masonry infills, the cracking strength was $1.3\sqrt{f'_m}$ and $2.9\sqrt{f'_m}$. Angel et al. (1994) observed diagonal cracking at very low displacements (0.03" and 0.02").

For steel frames with clay masonry infills, the test results of Hendry and Liauw (1994) were used. This was a series of eight tests, and the coefficient of variation was 46%. For concrete frames with clay masonry, the results of Benjamin and Williams (1958) were used. No reported value of f'_m was given, so a value of 3000 psi was assumed. Note that the constant would increase to $3.5\sqrt{f'_m}$ is the prism strength were only 2500 psi. The coefficient of variation was 22%.

El-Ouali et al. (1991) tested five steel frames, one with a clay brick masonry infill, two with concrete brick infills, and two with sand and lime brick infills. The diagonal cracking stresses were considerably lower than what was seen in other tests. The average was $1.2\sqrt{f'_m}$, with a coefficient of variation of 30%.

It is difficult to draw many general conclusions given the variety of testing procedures and conditions. It does appear, though, that there is significant uncertainty in diagonal cracking stresses (coefficient of variation on the order of 30-40%). It also appears that the cracking stress of clay masonry is slightly lower than concrete masonry, when related to $\sqrt{f'_m}$. A general value of $3\sqrt{f'_m}$ for the cracking stress of an infill appears to be reasonably appropriate. As indicated in the discussion on general behavior of an infill, diagonal cracking may not be as important a limit state as corner crushing.

Corner Crushing

The ultimate limit state of a masonry infill will be corner crushing. The term corner crushing is used somewhat generically, to be consistent with much of the previous literature . Although crushing of the masonry will often occur in one of the upper corners, it may occur elsewhere in the panel. Corner crushing will be used to describe the failure of the masonry through crushing, spalling of faceshells, or any failure that results in loss of masonry.

Although many formulations have been proposed for determining the capacity of infills under corner crushing, few seem to consistently predict the results of experimental data. Most seem to place an overemphasis on frame properties. A regression of structural clay tile infills tested under a wide range of frame stiffnesses and sizes indicated the capacity being proportional to $\lambda^{-0.132}$. Recalling that λ is proportional to the fourth root of various infill properties (E_m, I_c, sin 2θ), the strength would then be proportional to the 0.033 root of the infill property. In other words, an order of magnitude change of a stiffness property would only cause an 8% change in strength; two orders of magnitude change of a stiffness property would only cause a 16% change in the strength. The small effect of infill properties on the ultimate capacity was partially independently confirmed by Mehrabi et al. (1994). Tests with different aspect ratios showed little difference in ultimate strength.

Given the small effect of infill parameters on strength, it is perhaps reasonable to consider the corner crushing strength of an infill to be a constant times f'_m and the infill thickness. Although not entirely correct, this would significantly simplify the analysis process. Due to the biaxial nature of the infill stress state, the value of f'_m used should be some composite of the strength parallel and perpendicular to the bed joint. One proposed suggestion is to use the geometric mean of the strength parallel and perpendicular to the bed joint (EQE, 1995). For many infills, the prism strength will only be known, or estimated, for loading perpendicular to the bed joint. Thus, it is proposed that in general the strength of an infill due to corner crushing be taken as a constant times the prism strength perpendicular to the bed joint. The constant would account for the biaxial stress state. The infill strength would be determined as

$$P_u = K f'_m \tag{4}$$

in which P_u is the ultimate corner crushing strength (kips), K is a constant, and f'_m is the prism strength (psi).

Although it would be desirable to have an analytical method for obtaining the constant K, none appears to be available at present. Empirical values are given in the following Table 3.5 for various types of masonry.

Table 3.5 Values of K for Different Types of Masonry

Type of Masonry	Constant, K
Structural Clay Tile	
8" wall	0.087
13" wall	0.12
Concrete Masonry	
Steel Frames	$0.0065t_{eq}$
Concrete Frames	$0.010t_{eq}$
Clay Masonry	
Steel Frames	$0.0080t_{eq}$

Values for structural clay tile were obtained from the tests reported in Flanagan (1994). Due to the unique and large coring of structural clay tile, as well as the construction of the two-wythe 13" walls, the prism strength used is the gross prism strength for the load parallel to the bed joint. Structural clay tile significantly different in geometric characteristics would require testing to confirm these values. It is interesting to note that if the gross infill thickness is used for t_{eq} for structural clay tile (consistent with using the gross prism strength), then the constant K averages $0.010t_{eq}$, which is fairly consistent with the other types of masonry.

The value for concrete masonry in steel frames was obtained from analyzing the data in Dawe and Seah (1989). There was coefficient of variation of 18%. The prism strength used is the typical prism strength, the net prism strength for the load perpendicular to the bed joints. The value of t_{eq}

is the equivalent net thickness of the masonry, obtained by dividing the cross-section area of the unit masonry by the length. Typically, t_{eq} is approximately 4" for standard hollow 8" block.

To verify the results for concrete masonry in steel frames, the method was applied to the tests of Hendry and Liauw (1994) and El-Ouali (1991). For Hendry and Liauw (1994), the method predicted an ultimate load of 42 kips. Neglecting the tests from Hendry and Liauw (1994) that had concreted corners, and using 49 kips instead of 58 kips for the first test (49 kips appeared to be the corner crushing load from the load-deflection curve), the average ultimate experimental load was 45 kips. Based on the two tests with concrete brick (different infill thicknesses) reported by El-Ouali (1991), the constant should be $0.010t_{eq}$. As noted earlier, the prism strengths reported by Dawe and Seah (1989) seem high, which indicates that perhaps the constant K derived from their tests is low. It is interesting that the results of El-Ouali et al. (1991) match those for concrete frames. Perhaps there is little difference between concrete and steel frames. However, using $K=0.0065t_{eq}$ should give a conservative estimate of the strength of steel frames with concrete masonry infills.

The value for concrete masonry in concrete frames was obtained from the tests by Mehrabi et al. (1994). Their tests included both hollow and solid block. The coefficient of variation was 24%.

The value for clay masonry in steel frames was obtained from analyzing the results of the tests by Hendry and Liauw (1994). Perhaps most significant about their tests is that coefficient of variation of the ultimate load for eight tests was only 4.3%. Although slightly higher, this value is reasonably consistent with that obtained for concrete masonry. For the infill with clay masonry that El-Ouali et al. (1991) tested, the constant K would be $0.0036t_{eq}$. The low value is perhaps due to not loading the specimen until corner crushing really occurred. For the two tests with sand and lime bricks, the constant K averaged $0.0089t_{eq}$, or reasonably consistent with Hendry and Liauw (1994).

The full size test of Benjamin and Williams (1958) with a concrete frame and brick infill resulted in a constant K of $0.0059t_{eq}$, based on an assumed f'_m of 3000 psi. Interestingly enough, essentially the same constant, $0.0062t_{eq}$, is obtained from their test with a steel frame. If the prism strength were actually 2500 psi, the constant would be approximately $0.0075t_{eq}$, or reasonably consistent with Hendry and Liauw (1994).

As with diagonal cracking, it is difficult to draw many general conclusions given the variety of testing procedures and conditions. There does appear to be less variation in the corner crushing strength than the diagonal cracking strength. A general value of $0.0080t_{eq}$ for the corner crushing strength of an infill appears to be reasonably appropriate, irrespective of the type of masonry and the type of framing. Note that the constant of 0.008 corresponds to an effective contact length of the infill of 8". In other words, if the stress distribution between the infill and the bounding column were uniform (and no friction at the top), the stress block would have a depth of 8" at ultimate.

Effects of Additional Parameters on Stiffness and Strength

The strength and stiffness of the infill obtained using the formulations described in the preceding sections will be for a solid panel tightly fitted against the bounding frame. The strength and stiffness will need to be modified for other cases, as discussed in the following sections.

Height of Infill

The height of the of the infill appears to have little effect on the behavior (Flanagan, 1994). Thus, inter-story displacement levels are used in determining the parameter C. Non-dimensional inter-story drift ratios (inter-story displacement/height) should not be used.

Masonry Modulus of Elasticity

The development of the parameter λ was based on beam on elastic foundation theory, considering the column to be supported by the infill. This suggests that the appropriate modulus of elasticity to be used for the masonry should be that for loading parallel to the bed joints. Typically, only the modulus of elasticity perpendicular to the bed joints is known.

For brickwork, Dhanasekar et al. (1982) suggest that there is no significant difference in the elastic modulus normal or parallel to the bed joints.

Tests by Hegemier et al. (1978) and Hamid and Drysdale (1980) suggest that for grouted concrete masonry, the modulus parallel to the bed joints is slightly less than the modulus perpendicular to the bed joints. However, the difference is small enough, that grouted concrete masonry can be considered to be isotropic. Hamid and Drysdale (1980) show a larger decrease in the modulus for ungrouted concrete masonry. At lower levels of stress, the modulus parallel to the bed joints is approximately 70% of the modulus perpendicular to the bed joints. However, at higher stress levels, the secant modulus parallel and perpendicular to the bed joints becomes nearly equal. For simplicity, it is suggested that the standard modulus, that perpendicular to the bed joint, be used.

For other masonry materials, such as structural clay tile, the difference can be quite pronounced between the two modulii. Insufficient information exists to provide any generalizations.

Offset Infills

Infills may be offset from the column centerlines, and a portion of the infill may be outside the bounding frame. Only the portion of the infill enclosed by the bounding frame should be included in determining the area of the equivalent strut and the capacity. Eccentricity of the infill from the column centerline does not appear to be a significant problem in terms of inducing out-of-plane forces in the infill or twisting of the columns (Flanagan, 1994).

Non-symmetric Infills

The stiffness and strength of many infills will be different depending on the direction of loading. This could be due to different column sizes, non-symmetric openings, etc. This would cause the behavior to be direction dependent, and thus nonlinear. For linear analyses, it is suggested that the stiffness and strength of non-symmetric infills be taken as the average in each direction. Thus, for example, if an infill was enclosed by different column sizes, the average moment of inertia would be used in Equation 3.3 for determining the parameter λ.

Stiff Columns

For very stiff columns, the strut formulation predicts significantly high contact lengths. It is suggested that the contact length, α, be limited to 20% of the infill height, h'. This avoids unreasonably high stiffnesses of the infill.

Concrete Frame Moment of Inertia

For concrete frames, it is suggested that the moment of inertia for both beams and columns be taken as one-half the gross moment of inertia. This is based on several common approximations, and results of tests.

Stafford-Smith and Coull (1991) suggest using 50% of the gross moment of inertia for beams and 80% of the gross moment of inertia of columns for frame analysis. It is felt that the column moment of inertia can also be decreased to 50% of the gross value for infilled frames due to the infill bearing on the column. The column is similar to a beam, in that a distributed load is being applied to it from the infill.

The commentary to the ACI 318 Code (1989) suggests several approximations for the stiffness of concrete members depending on the analysis condition. These are summarized in table 3.6. Using the commentary suggestion of section 10.10.1 with $E_s/E_c = 8$, and ρ_t=0.03, results in $0.5E_cI_g$ for the stiffness.

Table 3.6 Recommended Stiffness Values for Reinforced Concrete Frames

Commentary Section	Stiffness	Comments
8.6.1	Gross EI all members, or ½ gross EI for beams and full gross EI columns	Braced Frame
10.10.1	$E_cI_g(0.2+1.2\rho_tE_s/E_c)$ columns $0.5E_cI_g$ beams	Design of compression members
10.11.2	$0.5I_g$ flexural members I_g compression members	Effective length of compression members
10.11.5.2	$E_cI_g/2.5$	Conservative value for Euler buckling determination

Possibly the best evidence for the use of $0.5I_g$ is the modeling of the bare frame reinforced concrete tests conducted by Mehrabi et al. (1994). Using $0.45I_g$ for the columns and $0.58I_g$ for the beam matched the stiffness of the bare frame at about one-half its capacity, or at a deflection of the bare frame on the order of when the maximum infill force occurs. The use of $0.5I_g$ thus seems to be a reasonable estimate for the concrete frame stiffness.

Pilasters

Infilled steel frames will often have pilasters built around them, in many cases for fireproofing. The pilaster will add some stiffness to the structure, but it is difficult to determine the actual amount. Estimates of the effective contribution of the pilaster have ranged from 0-10% of the gross area. It is suggested that 10% of the gross area tightly fitted well constructed pilasters be used in obtaining the initial stiffness of the column.

Vertical Loads

Mehrabi (1994) indicated that vertical loads only have a slight effect on the stiffness and strength of the infill. Vertical loads applied to the beam have more of an effect than vertical loads in the columns. Stafford-Smith (1968), based on model tests with micro-concrete infilling, suggested that vertical loads up to about half the vertical failure load will increase horizontal strength and stiffness. Vertical loads above half the vertical failure load will cause the failure mode to change to that for vertical loads, and the horizontal strength will decrease.

The distribution of vertical loads in an infill is difficult to obtain. Flat-jack testing of structural clay tile infills has shown a wide variation in vertical load, even within a single infill panel. The interaction of the infill with the frame under vertical loads is quite complex, being influenced by construction, creep, shrinkage, moisture expansion of clay masonry, etc. Given the complexity of vertical load distribution, and the small effect of vertical loads in the typical load range, it is suggested that vertical loads not be considered when evaluating the lateral stiffness or strength of an infill.

Joint Reinforcement

Many infills, particularly those of concrete masonry, will have joint reinforcement. Dawe and Seah (1989) has shown that the joint reinforcement has little effect on either the ultimate strength or the stiffness of a concrete masonry infill. A similar observation was made by Hendry and Liauw (1994) for clay brick masonry infills. Joint reinforcement is beneficial in controlling the size and distribution of the cracks.

Panel to Column Ties

Panel to column ties are often provided for out-of-plane anchorage of the infill. The ties do not appear to effect either the strength or the stiffness of an infill (Dawe and Seah, 1989; Hendry and Liauw, 1994). The need for the ties for out-of-plane loads is questionable, since the primary resisting mechanism is arching. Given that the panel to column ties do not effect in-plane behavior, and are not necessary for tightly fitted infills under out-of-plane loading, it is recommended that they be eliminated.

Related to column ties is the integrity of the infill at the column line. Dawe and Seah (1989) tested concrete masonry infills against steel columns under weak axis bending. There was no difference between infills butted against the column web, and those that also had mortar packed between the column flanges. The additional mortar did not increase stiffness or ultimate strength.

Bond Beams

Dawe and Seah (1989) tested an infill with bond beams at the third-points of the panel height. The bond beams did not effect the ultimate strength, or the initial stages of loading. In the mid-range of loading, the panel was much stiffer, and the behavior approached elastic-perfectly plastic type behavior. This could be accounted for by using the initial value of C until ultimate load is reached. For concrete masonry in steel frames, a value of C=4 would be used until the ultimate load in the infill is reached. This load would be maintained until the maximum deflection given for the particular masonry (e.g., 0.8" for concrete masonry and steel frames).

Vertical Bars and Partially Grouted Infills

Dawe and Seah (1989) tested an infill with vertical bars grouted the entire length of the compression diagonal. No effect was observed on either the stiffness or ultimate strength.

Hendry and Liauw (1994) tested steel frames with concrete masonry infilling with no vertical reinforcement, 0.4% vertical reinforcement, and 0.8% vertical reinforcement. No substantial effect in either stiffness or strength was observed. The failure mode was corner crushing, which is not affected by the reinforcement. Replacing the upper corner block(s) with concrete slightly improved the strength of the infills.

Based on these results, it is suggested that vertical reinforcement not be considered in determining the strength or stiffness of an infill. A potential secondary effect of vertical reinforcing is the grouted cells. If all cells were grouted, this would change the net thickness of the infill.

Effects of Prior Loading and/or Repair

Flanagan (1994) tested several infill specimens under in-plane loading after they had been subjected to either imposed out-of-plane drift loading or uniform out-of-plane loading using an air bag. He also tested a specimen that had been repaired after complete in-plane testing. Primarily, the top two courses of masonry were replaced, and the rest of the infill was tuck-pointed. In all these cases, the ultimate load, and the displacement at the ultimate load, was essentially the same as a virgin infill. In the early stages of loading, the stiffness was less than that of a virgin infill. Thus, the only change to the strut formulation for substantially cracked or repaired infills would be a reduction in stiffness in the early stages of loading. Perhaps the initial C value should be doubled, and other C values be increased using a linear interpolation, with the final C value remaining the same. For example, for a damaged structural clay tile infill, the values of C in Table 3.1 may be changed to 10, 12, 14, 15, 16, and 18 for the respective deformation levels.

Column Gaps

Based on a test with an approximately 1" gap between the column and infill (Flanagan, 1994), there was no reduction in ultimate load, but it occurred at a higher level of deformation. The infill was carrying some load before the gap closed due to transfer of forces through friction at the top interface, particularly at higher displacements. After closure of the gap, the infill did appear to be stiffer than would be predicted based on subtracting the gap distance from the deformation, and comparing to a tightly fitted frame. As a simple approximation, it is suggested that the following deformation be used for column gaps when determining the parameter C.

$$\Delta_C = \Delta_{frame} - 0.75\Delta_{gap} \tag{5}$$

in which Δ_C is the deformation to be used in determining the parameter C, Δ_{frame} is the inter-story deformation of the building frame, and Δ_{gap} is the column gap.

A test with gaps only at the bottom of the column showed no significant difference in behavior from a test with no gaps. Thus, column gaps should only be considered if they occur in the top part, perhaps the top third, of the column.

Top Gap

A test with a 20mm top gap between the infill and bounding frame was performed by Dawe and Seah (1989). The top gap drastically reduced the ultimate load and stiffness. It is suggested that an infill with a top gap have the stiffness reduced by 50%, and the ultimate strength reduced by 60%. One also needs to be cautious of potential shear failures in the columns when there are top gaps. The entire infill shear will be transferred through the column, whereas a significant portion of the shear is transferred through friction in the standard infill.

Similar to a top gap is the prevention of significant shear transfer at the top interface through the use of a polyethylene sheet. Dawe and Seah showed a slight decrease in stiffness and strength for this case. If the infill is tight, but there is a bond break between the infill and the frame (e.g., flashing material), it is suggested that the stiffness and strength be reduced by 20%.

Corner Openings

A test with a 2'x2' opening (25% of infill height, 20% of infill length) in one corner was conducted by Flanagan (1994). This is typical of mechanical openings that might be found in industrial plants. The capacity when the strut was intersected by the opening was about 35% of a solid infill, with the stiffness being about one-half that of a solid infill. In the other direction, the capacity was about 75% that of a solid infill, with the stiffness about that of a solid infill.

Any opening in the loaded corner will be critical, even if it is just one masonry unit. For a non-symmetric case (opening on only one side), an average strength and stiffness can be used. It is suggested that 75% of the solid infill stiffness be used, and 50% of the solid infill strength. For the symmetric case (openings on both sides), a large reduction in stiffness and capacity would occur. It is suggested that 50% of the solid infill stiffness be used, and 35% of the solid infill strength be used.

No experimental results are available for openings in the lower corners. Based on fact that gaps at the bottom of the column do not have any significant effect on the behavior suggests that openings in the bottom corners may not be as critical. For openings in the bottom corners, it is suggested that average between a solid infill and one with openings at the top be used.

Doorway Openings

Dawe and Seah (1989) tested several infills with doorway openings that were 79% of the height of the infill and 22% of the length of the infill. Some of the openings were centered, and some offset to the side. The openings reduced the stiffness by 50% and the strength by 40%. Although the offset opening had a greater effect in one direction of loading than the other, the average effect was similar to a central opening. The inclusion of jamb steel on either side of the opening did not alter the stiffness or strength.

Wood (1958) reported that a 4½" infill with a doorway opening (no further description given) had a net infill strength of 55% of a solid infill. It would appear conservative to take the strength of an infill with a doorway opening as 50% of the strength of a solid infill, and to take the stiffness as 50% of the solid panel stiffness.

Window Openings

Benjamin and Williams (1958) tested an infill with a central window opening that was approximately one-ninth the area of the infill. The stiffness was about half that of the solid panel stiffness, and the strength was about 55% of that of the solid panel. Similar to a doorway opening, a 50% reduction in strength and stiffness appears to be reasonable, and slightly conservative for a large central window opening.

Dynamic Properties

Damping

Infilled frames are generally not specifically mentioned in tables of recommended damping values for various types of structural systems. Damping values can be inferred from similar types of construction. Some of the damping values that have been suggested in the literature are given in Table 3.7.

Several damping values have been reported from testing. Benedetti and Benzoni (1984) measured damping of 4% before cracking of the infill, and 12% after cracking. Bertero and Brokken (1984) reported similar values, with 2% damping before cracking and 12% damping after cracking.

For typical seismic analyses, a damping value of 10% is suggested for infilled frames. This assumes that the infill has cracked, and there is some damage. Prior to cracking, the damping will be less, with a value of 4% being suggested.

Table 3.7 Recommended Damping Values

Form of Construction	Damping Value		Reference
	Service	Ultimate	
Concrete frames with concrete or masonry shear walls		10	Dowrick (1987)
Concrete and masonry shear wall buildings		10	
Reinforced Concrete	5	10	Army (1986)
Masonry Shear Walls	7	12	
RC frame, some internal walls	4	12	Stafford-Smith and Coull (1991)
RC all forms, many internal walls	5	16	
Steel frame, many internal walls	4	15	

Natural Frequency Evaluation

As the strut formulation implies, the stiffness decreases with increasing deformation. This can have an effect on the behavior of the structure, as well as dynamic testing. This was apparent in a shake table test of a steel frame with a structural clay tile infill (Fowler, 1994). After subjecting the specimen to several seismic time histories, the natural frequency was measured using a low-level white noise excitation, and was determined to be between 13 and 14 Hz. A calculated frequency using C=5 was 12.3 Hz, while for C=4 the calculated frequency was 13.7 Hz. Thus, the structure was quite stiff at low levels of excitation, even after experiencing some cracking. Under the last seismic excitation of the specimen, there was approximately a 0.3" displacement, which would indicate a value of C=11. Using C=11 resulted in a calculated frequency of 8.4 Hz, while the measured frequency under the seismic excitation was 8.8 Hz.

This nonlinear phenomenon has several implications. The first concerns low level vibration tests to determine the natural frequency of an infill building. Due to the nonlinear nature of infilled frames, the frequency determined during a low level test will be significantly higher than the frequency at which the structure responds during an actual earthquake. Using the frequency calculated or measured from low level vibrations will, however, in general be conservative. For most seismic response spectra, the response decreases with increase in natural period, or decrease in natural frequency. The second implication is that for a true determination of the behavior, it is necessary to consider the frequency at the appropriate level of displacement, that is, the reduced natural frequency at higher levels of relative displacement. The developed analytical method with the increasing C, and hence softening of the system, will capture this behavior.

Post-Peak Behavior

Postpeak testing of infills indicates significant capacity well beyond the displacement at which the peak load occurs. Figure 3.6 shows load-deflection curves for two structural clay tile infills. Flanagan (1994) recommended that the in-plane strength be reduced to 75% of the peak strength at an in-plane drift of 1.5 times the displacement at peak. This is perhaps appropriate for structural clay tile, which tends to be more brittle than other masonry materials, and is more difficult to confine after cracking due to the large coring.

Figure 3.7 shows the load-deflection curve for a steel frame with concrete masonry. A significant amount of ductility is observed, with approximately 50% of the capacity still remaining at a displacement of eight times the displacement at peak.

Load-deflection curves for concrete masonry in concrete frames are shown in Figure 3.8. Curves for both hollow masonry and solid masonry are shown. Average displacements at peak load for the concrete frames were 0.45".

Infills clearly have a significant capacity beyond peak, and a significant amount of ductility and energy absorption capability. For large earthquakes, the additional capacity beyond peak needs to be accounted for. Displacements at 1.5-2 times the displacement at peak may serve as a practical limit in predicting repairable damage levels of the masonry. For life safety functions, displacements even up to ten times the displacement at peak may be tolerable.

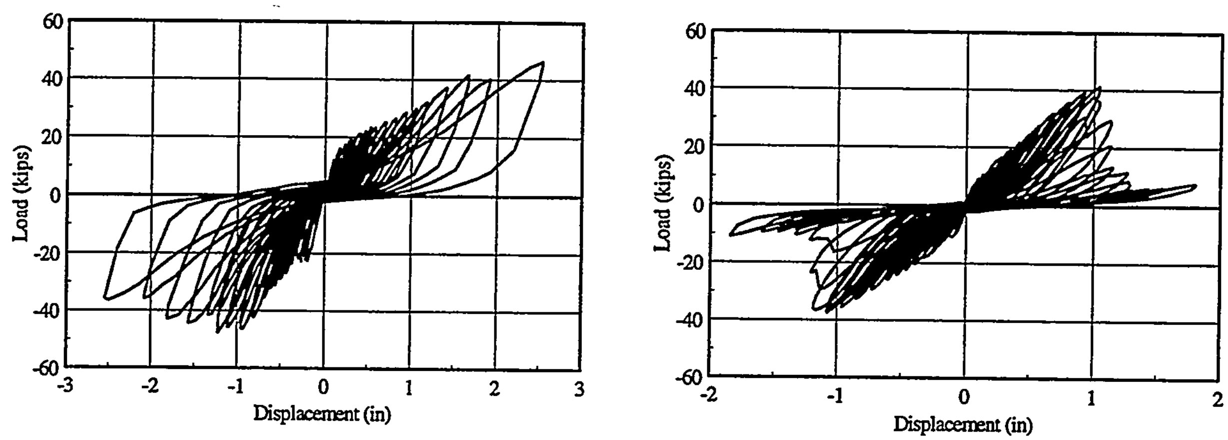

Figure 3.6 Load-Deflection Curves for Structural Clay Tile Infilled Frames (Flanagan, 1994)

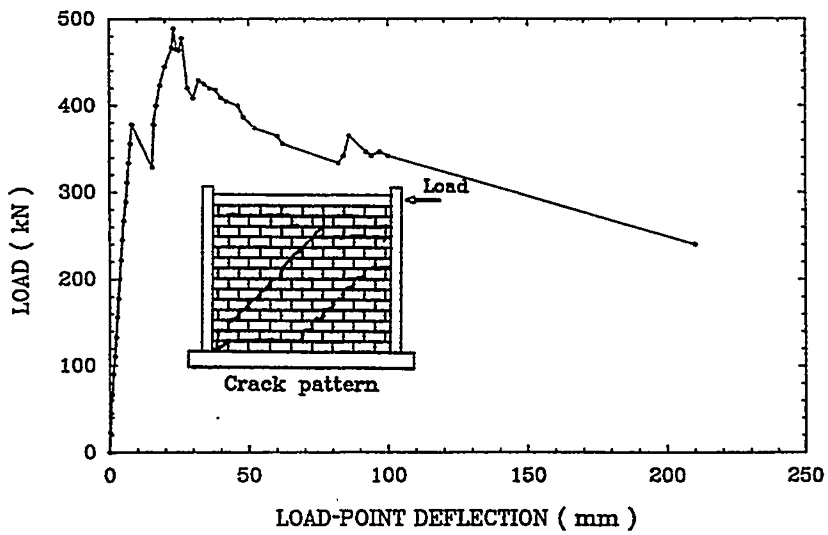

Figure 3.7 Load-Deflection Curve for Steel Frame with Concrete Masonry Infill (Dawe and Seah, 1989)

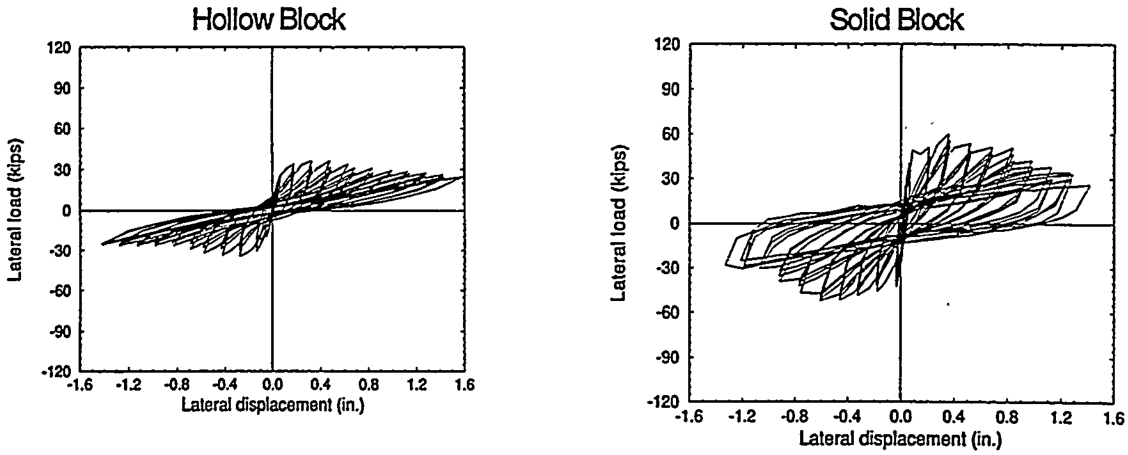

Figure 3.8 Load-Deflection Curves for Hollow and Solid Concrete Masonry in Concrete Frames (Mehrabi et al., 1994)

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CHAPTER 4

SEPULVEDA HOSPITAL BUILDING 40

Building Description

The Sepulveda Veterans Administration Hospital complex is located approximately 7 km from the epicenter of the Northridge earthquake. Building 40 is a boiler house building built in 1955. The structure is approximately 47' in the North/South direction by 82' in the East/West direction, and approximately 33' tall. The structural system consists of a reinforced concrete frame with multi-wythe clay masonry infills. Being a mechanical building, there is little structural framing in the interior. There is a platform and mezzanine at two different levels that covers part of the floor, but did not provide any significant lateral load resistance. Thus, the primary lateral resistance is the exterior infills. The total weight of the building is just under 1700 kips.

A plan view of the building is shown in Figure 4.1, with elevations in Figure 4.2. Note that significant openings exist in all the sides, except for the west side.

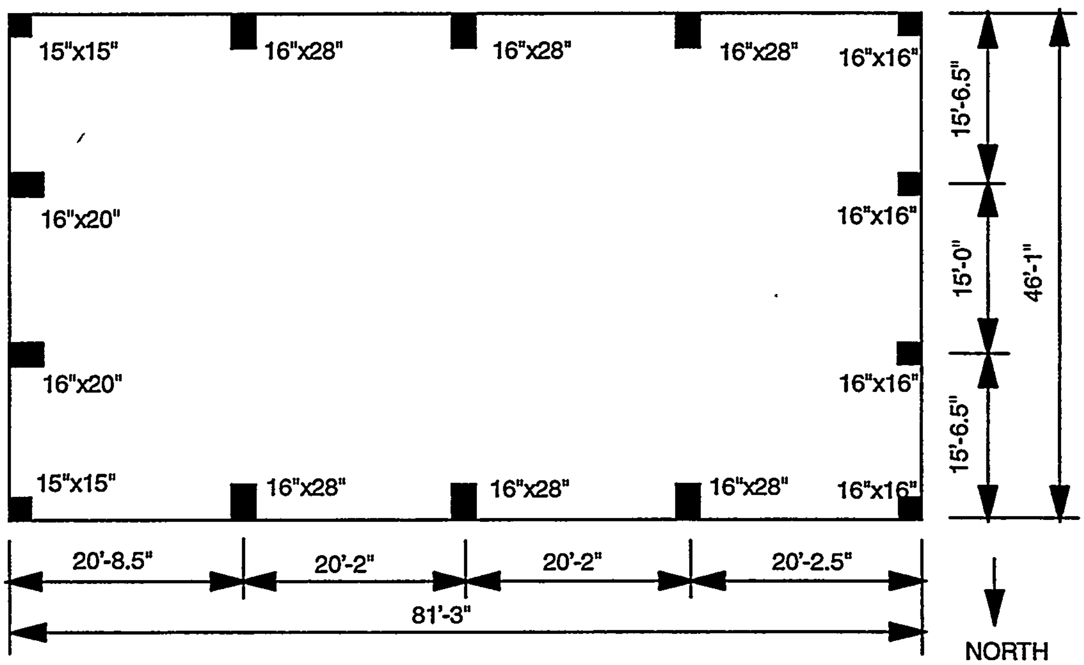

Figure 4.1 Plan View of Building 40

The exterior of Building 40 is reinforced three-wythe 13" clay masonry walls. Part of the walls extend beyond the perimeter of the column, with part of the walls being within the columns, and acting as infills. The typical cross-section of a wall is shown in Figure 4.3.

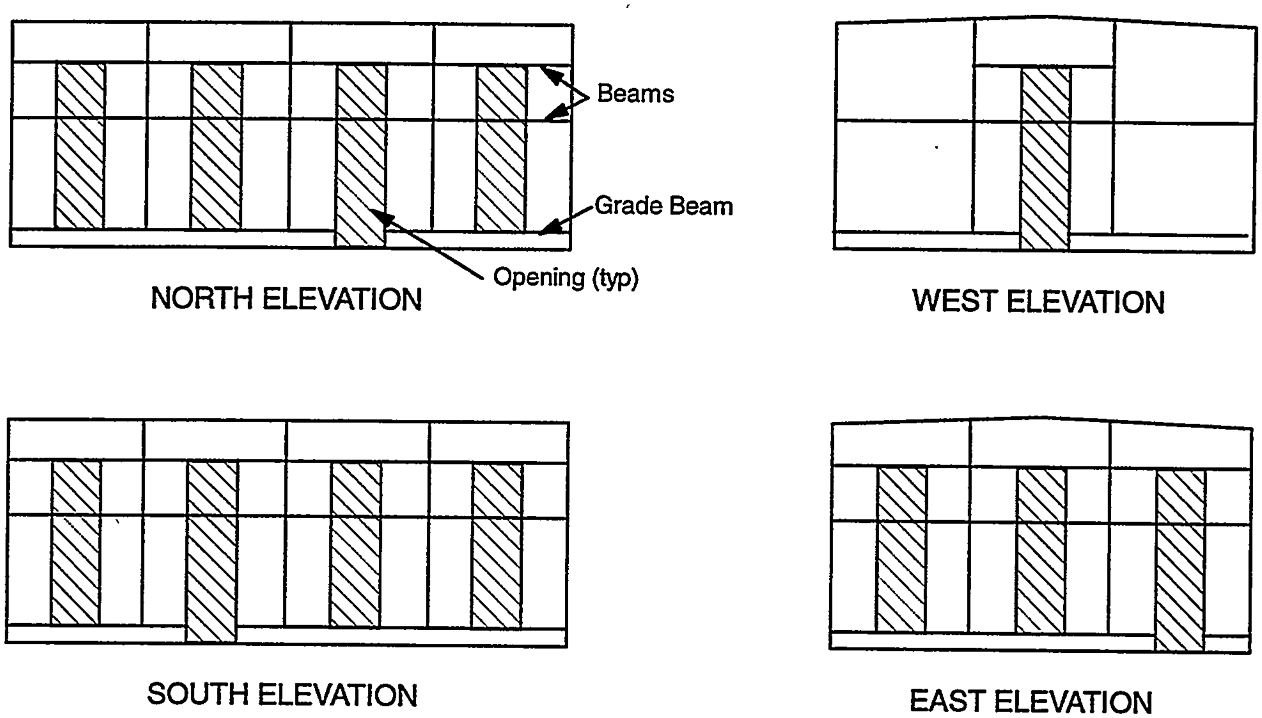

Figure 4.2 Elevations of Building 40

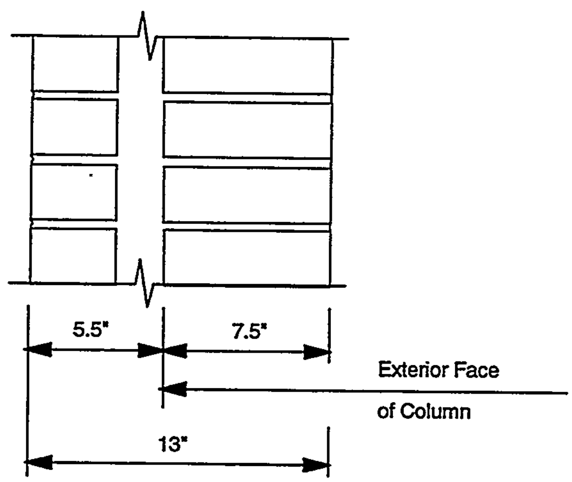

Figure 4.3 Typical Cross-section thru Wall

Building Model

A three-dimensional model of the structure was constructed using ABAQUS, version 5.4. No soil-structure interaction affects were considered, and the columns were assumed to be fixed at their base. Soil-structure interaction effects will affect total displacements more than relative displacements. Since infill behavior is governed by relative displacements, soil-structure interaction was neglected.

One-half of the gross moment of inertia was used for the concrete frame members to account for cracking. The full area and moment of inertia were used for the concrete roof, which was modeled using horizontal plate elements. Concrete strength was assumed to be 2500 psi, with a modulus of elasticity of 3030 ksi.

A masonry prism strength, f'_m, of 4000 psi was assumed. This was based on data shown in SCPI (1969), which indicated that an average prism compression strength is at least 4000 psi. A modulus of elasticity of 2000 ksi was assumed. This was based on a mean value of the modulus of elasticity of 1960 ksi for solid bricks (Atkinson and Yan, 1990). The mean value was rounded to 2000 ksi, which is also the code value (MSJC, 1992) for 8000 psi bricks with Type N mortar. Poisson's ratio was assumed to be 0.25.

An equivalent strut, as proposed in Chapter 3, was used to model the infill. Only the portion of the wall within the column boundaries (7.5") was considered as being effective. For locations where there were openings, the equivalent strut was assumed to have half the stiffness and half the strength of a solid infill. Unfortunately, there is little experimental data for obtaining values of C, the parameter governing the stiffness of the equivalent strut, for concrete framing with clay masonry infills. Therefore, parameters of C that were used were those for concrete framing with concrete masonry infills (Table 3.3).

Even though the building was primarily open (there was a small platform and mezzanine which were included in the model), there were two spandrel beams in all except the west side. Equivalent struts were used between the spandrel beams. Thus, in elevation, there were in general three equivalent struts in the vertical direction.

Ultimate strength of the infill was determined using Equation 3.4, with K=0.008. This resulted in a horizontal component of the capacity of 0.008(7.5)(4000)=240 kips for each infill. Due to the ductility of infills, and in particular the ductility of reinforced infills, an elastic-plastic behavior for the infill was assumed. That is, after the horizontal force in an infill exceeded 240 kips, the infill was assumed to be able to continue to carry this load with increasing deflection.

A response spectra modal analysis of the structure was conducted using base motions recorded at the site. The two horizontal components of the earthquake were considered, but not the vertical component. The response from different modes and different directions were combined using square-root-of-the-sum-of-the-squares.

Due to the stiffness change and possible failure of the infills, an iterative analysis was conducted. Whenever a force exceeded the strut capacity, the area of the strut was reduced in the next iteration so as to lower the force being taken by that panel. The reduction in area was determined by a ratio of the allowable force in the strut divided by the force from the analysis.

Ground Motion

A strong-motion instrument was located in the north-east corner at the base of the building. Peak accelerations were 0.75g in the east-west direction, 0.94g in the north-south direction, and 0.45g in the vertical direction. For analysis purpose only the two horizontal components were considered. Response spectra for 10% damping, as developed from the earthquake records, are shown in Figure 4.4. Ten percent damping was used since the ultimate strength of the building was approached. The significant damage would act to absorb much energy. The spectra is enveloped and broadened to obtain a "design spectra" for use in the analysis of Building 40.

Results

Due to the nonlinear nature of the equivalent struts for the infills, an iterative analysis was required. The initial run set C=8 for all struts. Struts whose capacities were exceeded then had their

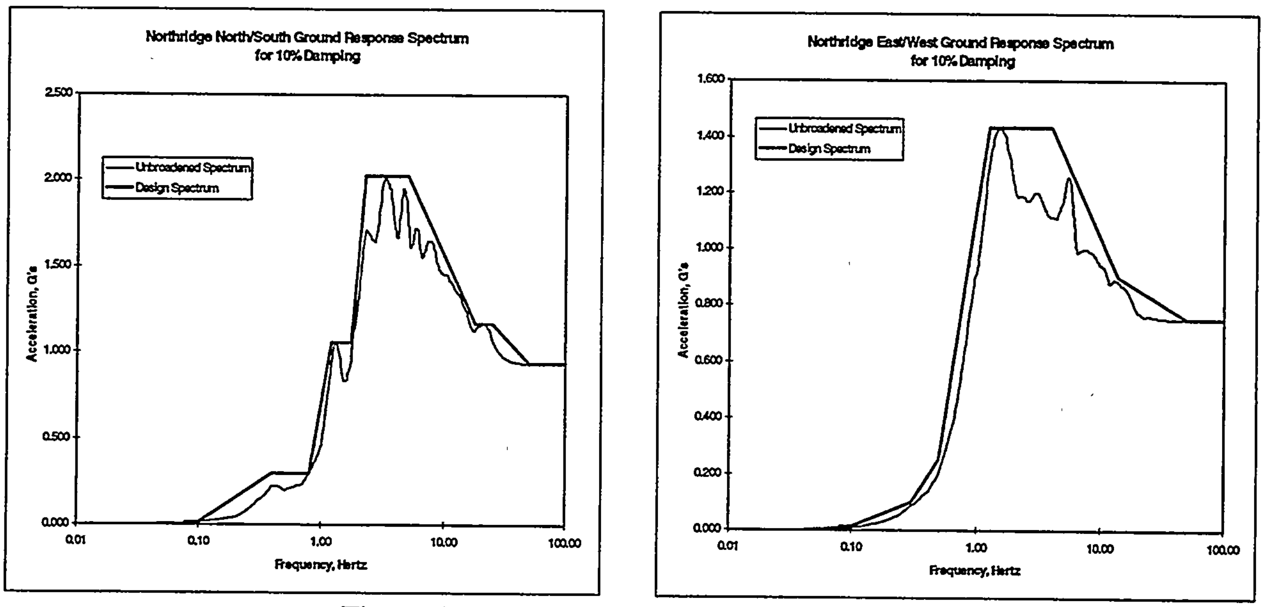

Figure 4.4 Response Spectra for Building 40

areas reduced, as discussed in the section on building modeling. The iterative process was continued until the strut stiffness and forces converged. This required eight iterations for the present problem. Two of the iterations will be discussed, the first and the last.

The first analysis was conducted with C=8 for all struts. Primary natural frequencies were 3.7 Hz in the North-South direction and 4.1 Hz in the East-West direction. The maximum strut forces were in the lower struts on the east side, and were about three times as great as the infill capacity. By scaling the results, this building would have been able to withstand an earthquake of about 0.32g without exceeding the capacity of any infill. Some minor cracking would be expected at this level, but it would be easily repairable. Torsion was evident in the building as the deformations on the east side were about 40% greater than on the west side. The infills were creating an unsymmetrical structure.

At a 0.32g level earthquake, the displacement at the top of the east side would be about 0.58". A value of C=8 would have been appropriate for the lower stories, but not the upper stories. An iterative analysis was not conducted with different values of C, since the results would not change drastically. It thus appears that this building could withstand a moderate earthquake with little damage.

After eight iterations, the results converged for the actual earthquake. The three primary modes of the structure are listed in Table 4.1. A total of 200 modes were included in the analysis, which resulted in accounting for at least 93% of the mass in both the North-South direction and East-West direction.

Table 4.1 Primary Frequencies of Building 40

Mode	Frequency (Hz)	Participating Mass (Percentage)			Description
		North-South	East-West	Rotational	
1	2.0	52	0.4	4	North-South translational
2	2.3	0.3	82	12	East-West translational
4	3.9	25	0	56	Torsional

The displaced shape of the structure under the earthquake loading is shown in Figure 4.5. The maximum roof displacements in the North-South direction are 5.0" on the east side, and 1.3" on the west side. The torsion is even more evident under this higher loading. In the East-West direction, the roof maximum displacements are 3.3" on the south side, and 3.4" on the north side.

The highest accelerations were in the northeast corner of the building. The analysis indicated an east-west acceleration of 1.9g (2.5 amplification) and a north-south acceleration of 2.1g (2.2 amplification). At the opposite, or southwest corner, the accelerations were 1.8g in the east-west direction, and 1.8g in the north-south direction.

The results can also be viewed in terms of strut/infill behavior. There is a band of solid infills around the top of the structure, which adds a significant amount of stiffness. The drift in these infills was on the order of 0.1-0.2", with C generally being 4. Possibly some minor cracking would exist in these infills. Drifts at the middle level varied between 0.4-0.8". Thus, these infills were reaching their capacity, and corner crushing was starting to occur. As expected, the largest drifts were in the lowest level, with the drift on the east side being approximately 4.1". All of the lower level infills had reached their capacity, and with the large drifts that some experienced, quite visible damage would be expected.

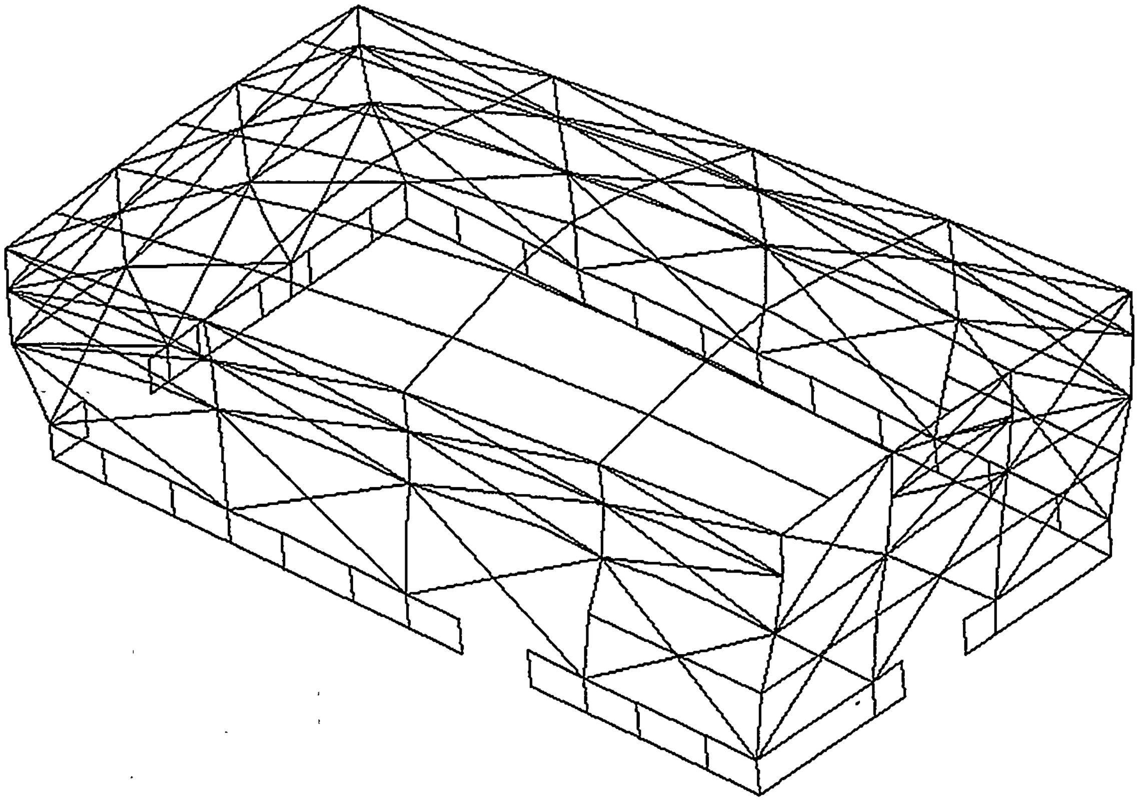

Figure 4.5 Deformed Shape of Building 40

Critical Discussion

Unfortunately, there was no instrumentation other than at the base. Thus, comparisons with field performance can only be made in a qualitative sense. The maximum horizontal displacement of 5" on the east side appears to be of the right magnitude. The displacement decreases to 1.3" on the west side. EERI (1995) describes mechanical damage which indicated about 4" of movement near the top of the structure. There was about 4" of relative axial motion between pipe hangers and the supported chilled water lines. A 12" steam line moved about 4" laterally at a roller support. Thus, the calculated displacements are of the order of what can be inferred from field observations.

The structural damage to the building was described in Chapter 2. Probably the worst damage was at the northeast corner, Figures 2.1, 2.3. This corresponds to the location of highest displacements and accelerations from the analysis. The damage was indicative of high infill corner forces, which was also predicted by the analysis. Little damage was apparent in the higher levels of the building. Thus, the analysis predicted the type of damage, as well as locations of high damage.

The equivalent strut methodology provided a computationally tractable means for analyzing this infilled frame structure to displacements well past the infill capacities. A response spectra analysis was sufficient to capture the actual behavior and damage to the structure. The equivalent strut procedure can be used for the efficient analysis of buildings subjected to large earthquakes.

The results can also be qualitatively verified on the basis of the 1971 San Fernando earthquake. The Sepulvada VA Hospital was approximately 14 km from the epicenter of the San Fernando earthquake. No ground records were available, but peak ground accelerations were estimated to be 0.4g (Agbabian Associates, 1971). Some minor cracking of the reinforced concrete frame was evident in this earthquake, but there was no significant damage to the clay masonry infills. As noted above, the equivalent strut methodology predicted corner crushing would commence at approximately 0.32g. Although difficult to draw a rigorous comparison (for example, the response spectra of the San Fernando and Northridge earthquake are not identical), the proposed equivalent strut method gave a reasonable prediction of the behavior during the San Fernando earthquake, and perhaps was slightly conservative.

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CHAPTER 5

DOWNTOWN LOS ANGELES BUILDING

Building Description

A 13 story office building located in downtown Los Angeles which was 32 km (20 miles) from the epicenter of the Northridge earthquake was analyzed. The structure was built in the 1913 time frame. It is a steel frame with clay brick and clay tile infills. In the 1969 time frame, there was a major renovation. Practically all of the interior masonry partitions were removed, and these were replaced by steel stud/gypsum board partitions. Although this reduced the mass of the structure, it also significantly reduced the lateral capacity of the structure. Several years later, there were further modifications to the building. The exterior was restored to the original condition, with several large precast cladding panels installed in 1969 being removed.

A plan view of the building is shown in Figure 5.1. The building is located on a street corner, and the basement extends under the sidewalk on the north and east sides. On the second through the thirteenth floor there is a setback on the west side, creating a U-shaped building. The framing of the building is quite irregular. The framing shown in Figure 5.1 is typical of the third through the thirteenth floor. There are some changes in column locations and framing in the lower floors. There is also a small penthouse on the south side of the structure.

Elevations of the structure are shown in Figure 5.2. The first two floors on the East face (the front of the building) are essentially open. The top floors have large window openings, which comprise about 36% of the infill panel area. Floors three through thirteen will be considered to be infills with openings. The north face, which is a side of the building the faces a street, is similar to the front face. The bottom two floors are open, with floors three through thirteen having infills with large openings (approximately 35% of the area). The south face of the building has solid infills on the lower two floors. The third and fourth floor have some solid infills; above the fourth floor, all infills have openings. However, the openings are smaller than on the north and east face, averaging about 16% of the infill panel area. The west face, or back of the building, has solid infills up through the seventh floor. Originally it appears that this building abutted and was built against another building on the west side. Apparently the abutting building was of height equal to seven stories of this building. The abutting building has since been demolished. Above the seventh floor, there are some openings. A few of the openings were filled in during the 1969 renovation, and these are treated as solid infills. As with the south face, the openings are smaller in size, approximately 23% of the infill panel area on the south wing and 28% on the north wing. The north, east, and south faces in the alcove, or setback, all have large openings, being approximately 50% of the infill area on the east and south faces, and 40% of the infill area on the north face.

The floors of the building are similar to a one-way concrete joist system, with the "pans" being formed with structural clay tiles. The total weight of the structure is 27500 kips, which amounts to an approximate average floor weight of 220 psf. Much of the mass is due to the heavy exterior cladding.

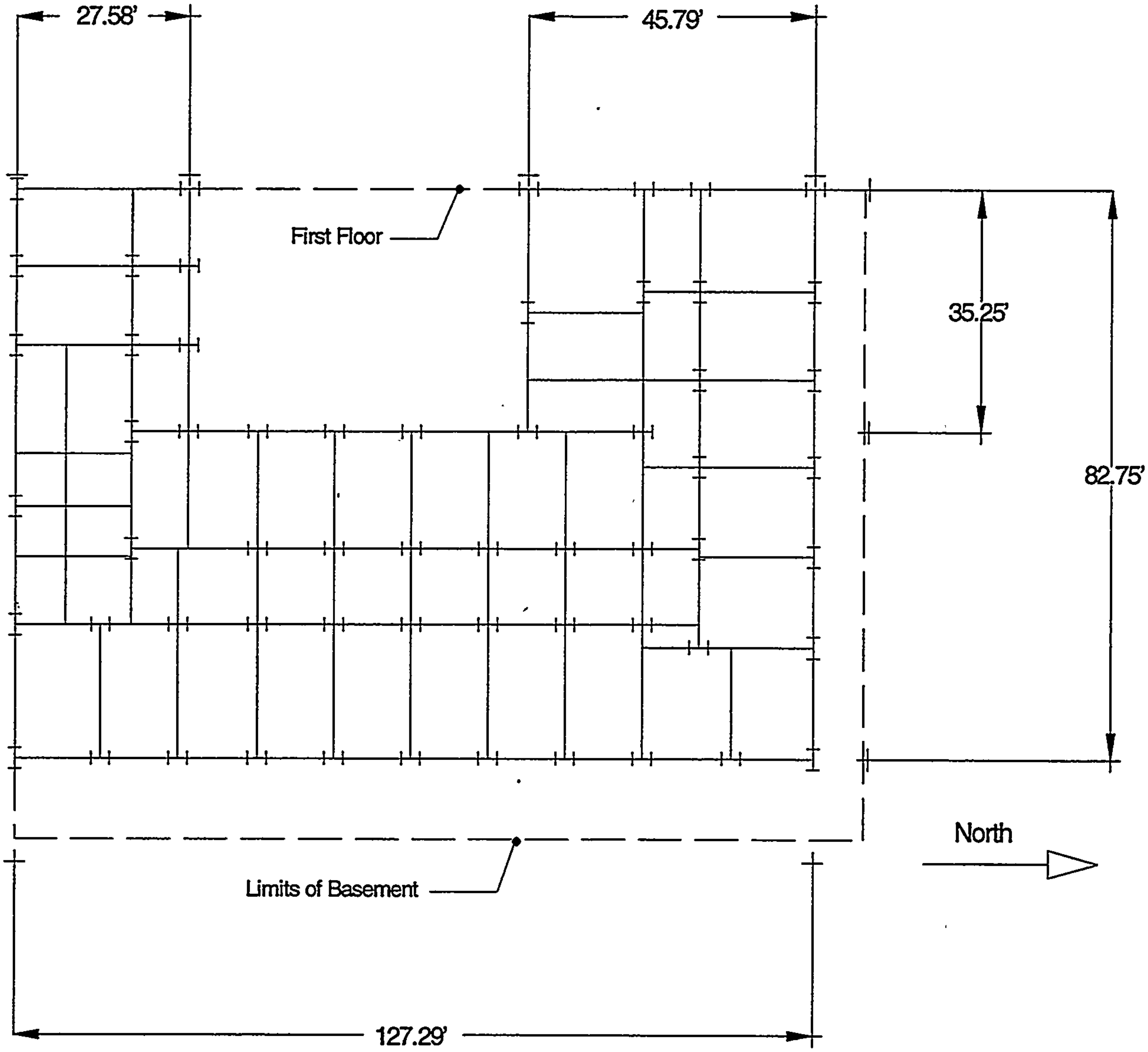


Figure 5.1 Plan View of Los Angeles Building

Qualitative Observed Response

The investigators did not have access to the interior of the building to observe any damage as the result of the Northridge earthquake. The exterior of the building showed some minor damage. Primarily this was vertical cracking on the back side at the southwest side. Some minor diagonal cracking was also observed in these infills. Figures 2.8 and 5.3 show some of the damage to the structure. Note the cracking in the lower infills in Figure 5.3.

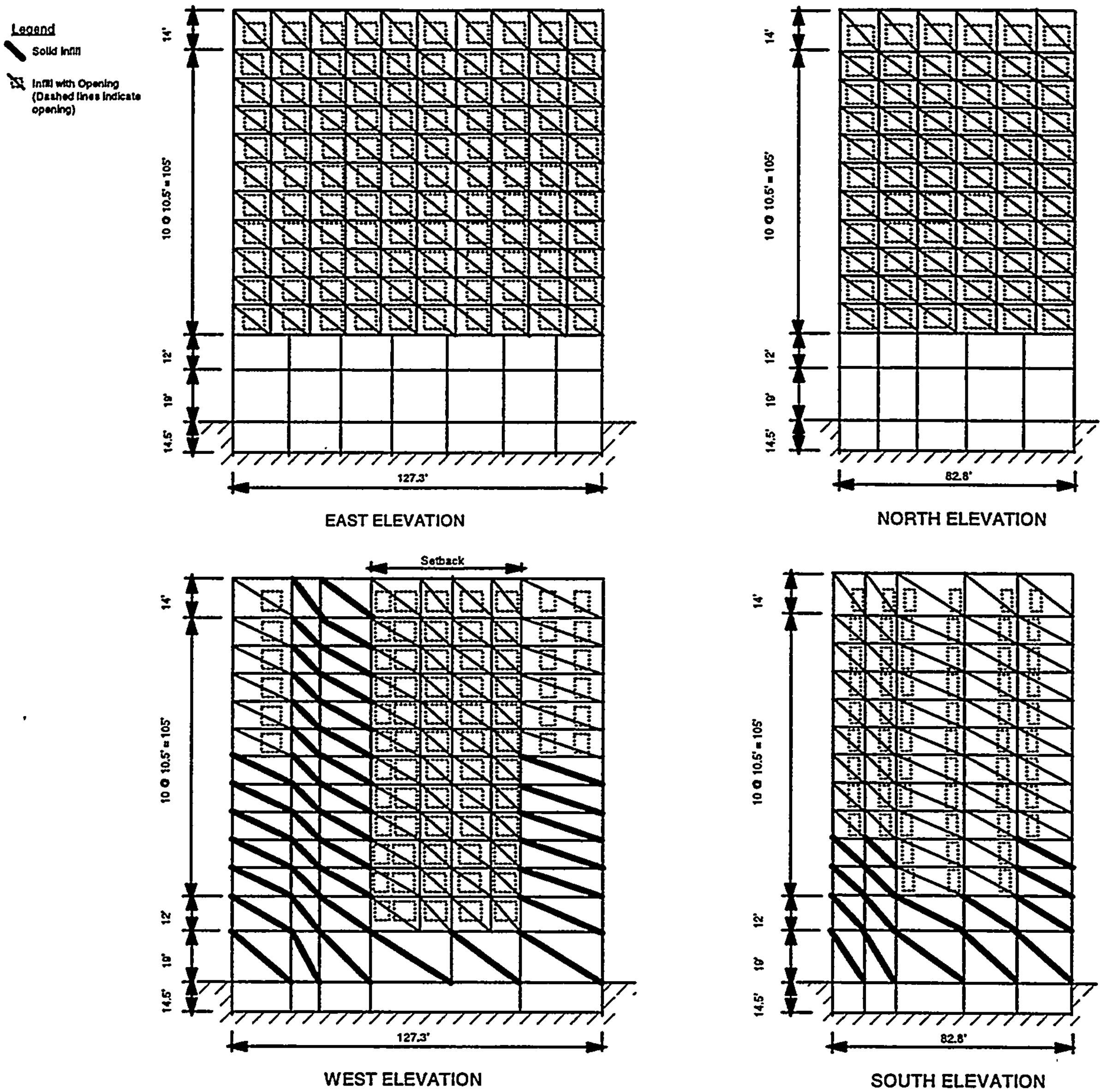


Figure 5.2 Building Elevations

Recorded Response

This building was instrumented by the California Division of Mines and Geology under the California Strong Motion Instrumentation Program (CSMIP). Locations of the sensors are shown in Figure 5.4. Figures 5.5-5.7 show response spectra for the horizontal motion as recorded in the basement (CDMG, 1995). Strong motion shaking lasted for about 8-10 seconds. Maximum total accelerations and displacements recorded by the instrumentation are given in Table 5.1.

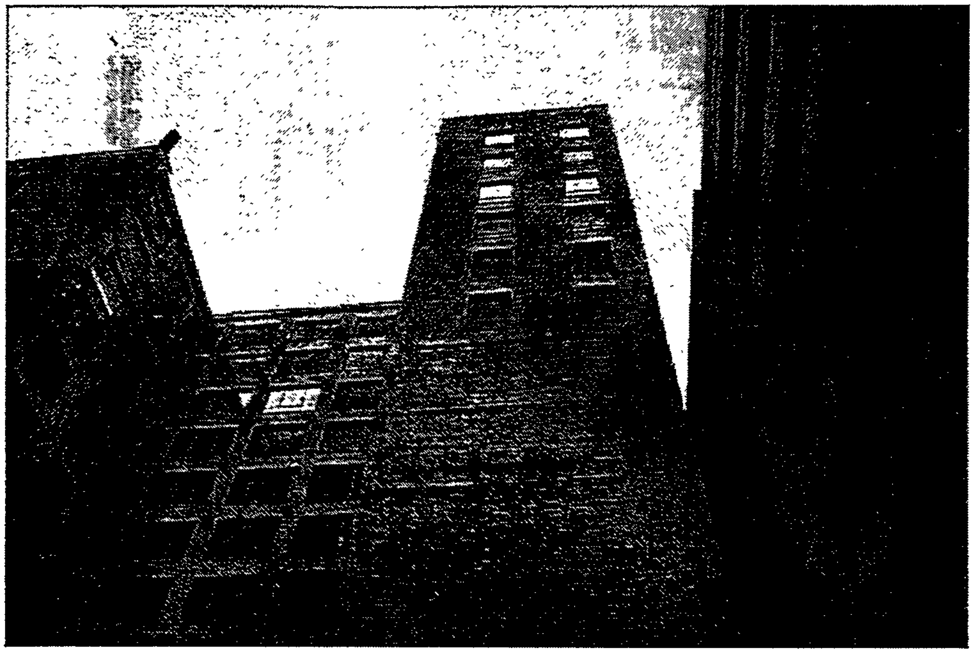


Figure 5.3 Damage to Los Angeles Building

Table 5.1 Maximum Total Accelerations and Displacements from Recorded Response

Channel	Location	Direction	Max. Acc. (g)	Max. Disp. (in)	Comments
1	N Basement	Up	0.062	0.60	
2	N Basement	N/S	0.181	1.07	
3	N Basement	E/W	0.174	1.33	
4	S Basement	E/W	0.156	1.33	
5	W 2nd Floor	N/S	0.255	1.06	Large spike in acc.
6	E 2nd Floor	N/S	0.166	1.17	
7	N 2nd Floor	E/W	0.174	2.07	
8	S 2nd Floor	E/W	0.175	1.39	
9	E 8th Floor	N/S	0.220	1.90	
10	N 8th Floor	E/W	0.222	3.61	
11	S 8th Floor	E/W	0.196	2.10	
12	W Roof	N/S	0.249	4.02	
13	E Roof	N/S	0.268	2.70	
14	N Roof	E/W	0.372	5.35	
15	S Roof	E/W	0.222	3.19	

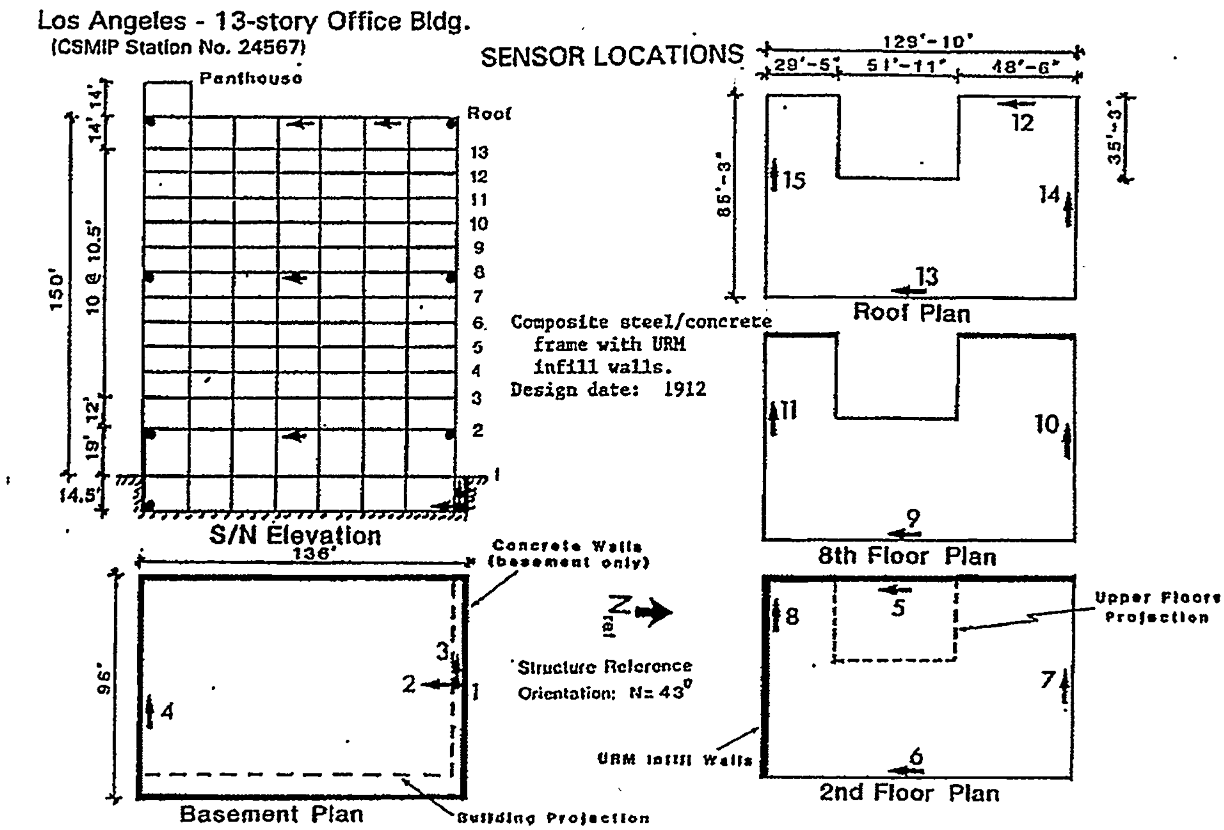


Figure 5.4 Strong Motion Instrumentaion Locations (CDMG, 1995)

To obtain the natural frequency of the building, a transform analysis was conducted on the recorded accelerations. Essentially, the energy at a given frequency in the response was divided by the energy at the same frequency in the excitation. To maintain accuracy in the transform function, a rather coarse frequency interval of approximately 0.1 Hz had to be used. Since the building had a rather low natural frequency, only an approximate value could be obtained from the transform analysis.

The transform functions are shown in Figures 5.8-5.10, the N/S transform function in Figure 5.8, the E/W transform function in Figure 5.9, and the torsional transform function in Figure 5.10. The range of the first natural frequency is shown in Table 5.2 for various transforms.

The primary natural frequency appears to be in the 0.4-0.5Hz range in both directions. There appears to be significant torsional response as base motion in the E/W direction induces response in the N/S direction and vice versa. The torsional response was anticipated since the south and west sides are stiffer than the other sides.

NORTHRIDGE EARTHQUAKE JANUARY 17, 1994 04:31 PST
LOS ANGELES - 13-STORY OFFICE BLDG.
CHN 2: 180 DEG (BASEMENT: N. WALL)
ACCELEROGRAM BANDPASS-FILTERED WITH RAMPS AT .10-.20 TO 46.0-50.0 HZ.
24567-C1111-94019.03 062895.1446-QN94A567

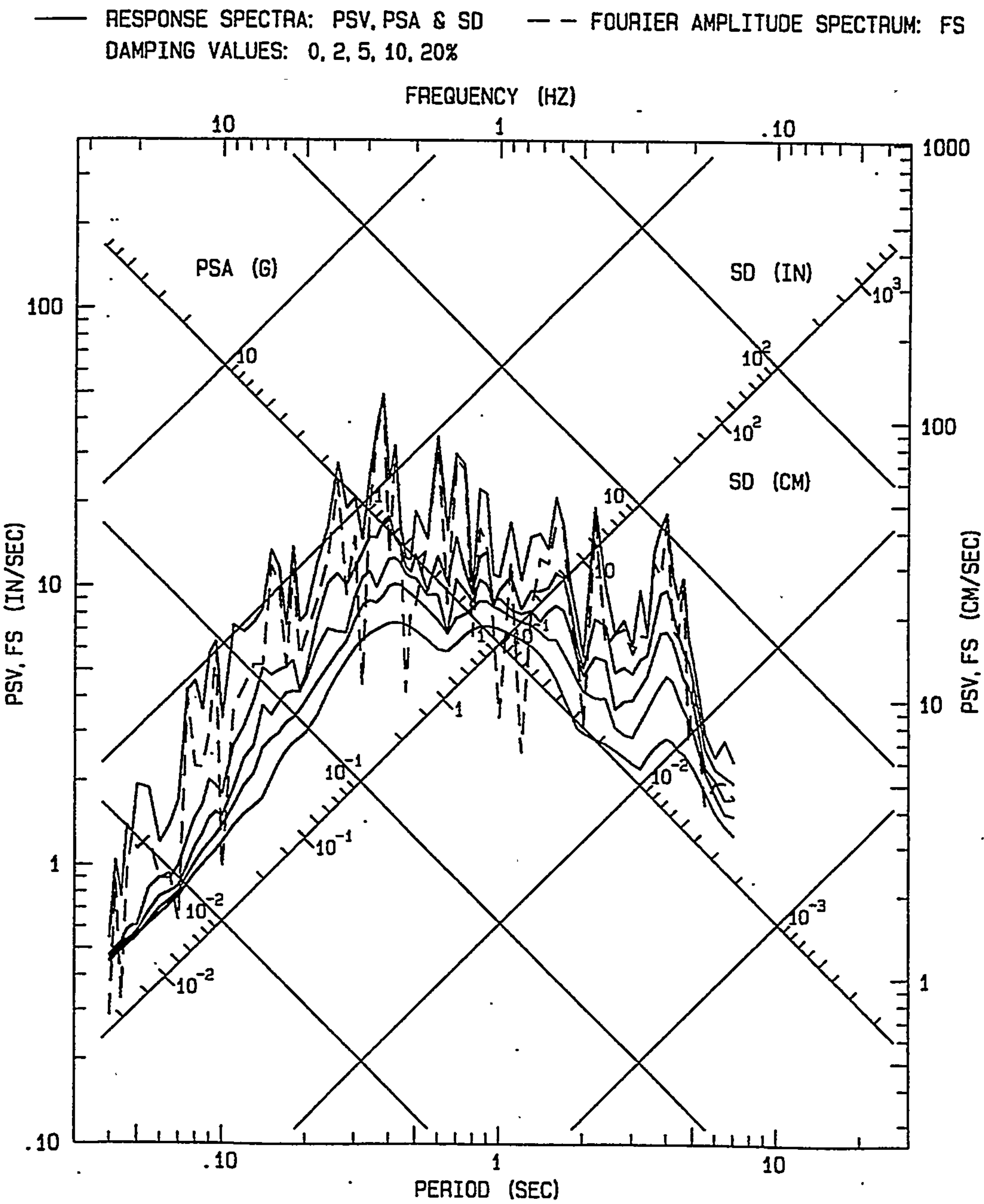


Figure 5.5 Station 2 N/S Spectra (CDMG, 1995)

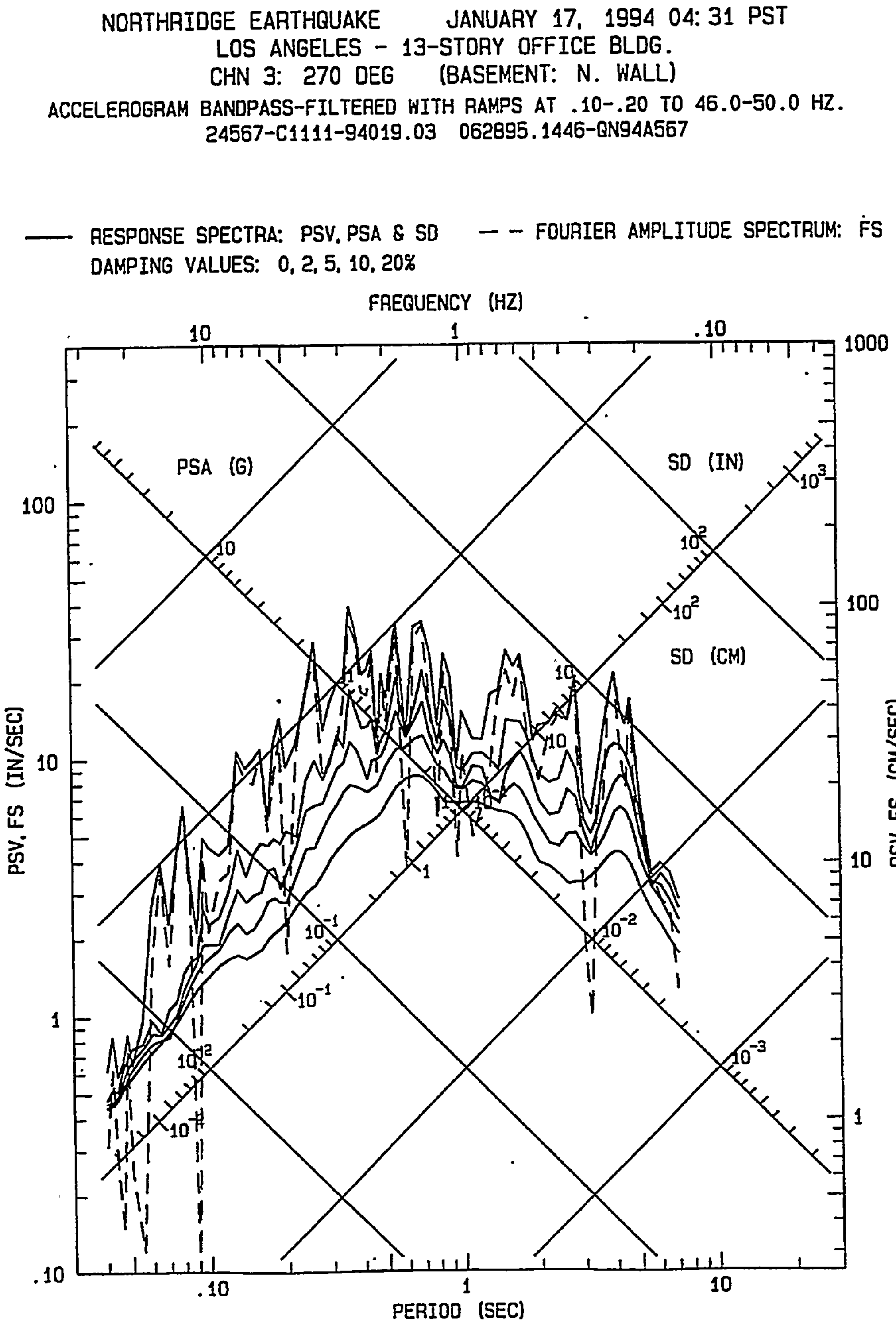


Figure 5.6 Station 3 E/W on North Wall Spectra (CDMG, 1995)

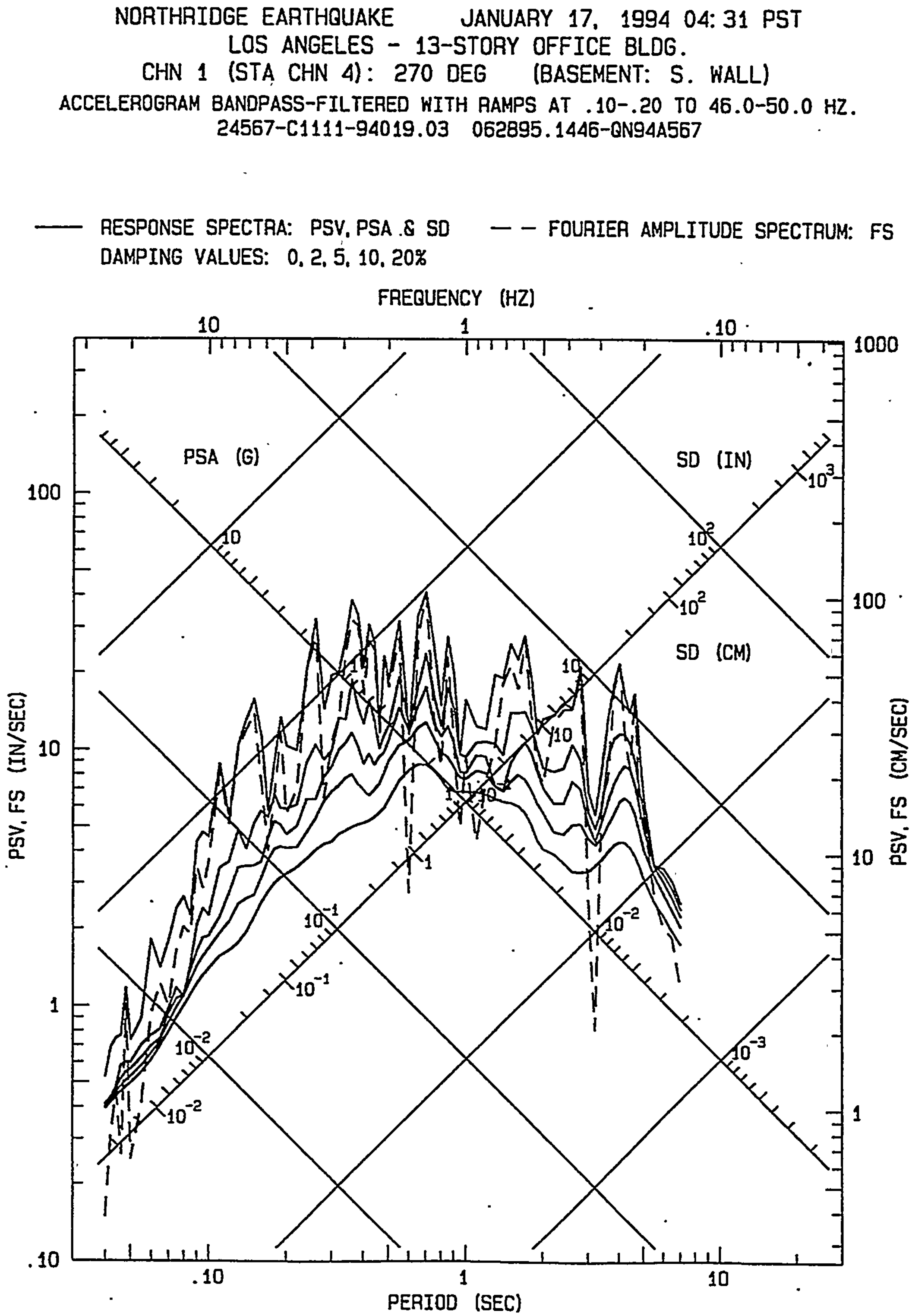


Figure 5.7 Station 4 E/W on South Wall Spectra (CDMG, 1995)

Table 5.2 First Natural Frequency from Transform Functions

Transform Function	Input Acceleration Location	Output Acceleration Location	First Natural Frequency (Hertz)
North-South	2	9	0.3-0.4
	2	12	0.4-0.5
	2	13	0.4-0.5
East-West	3	10	0.4
	3	14	0.4-0.5
	4	11	0.4-0.6
	4	15	0.4-0.7
Cross East-West	3	15	0.4-0.7
	4	14	0.4-0.5
Torsion	2	14	0.4-0.6
	2	15	0.4-0.6
	3	12	0.4-0.5
	3	13	0.4
	4	12	0.4-0.6
	4	13	0.4

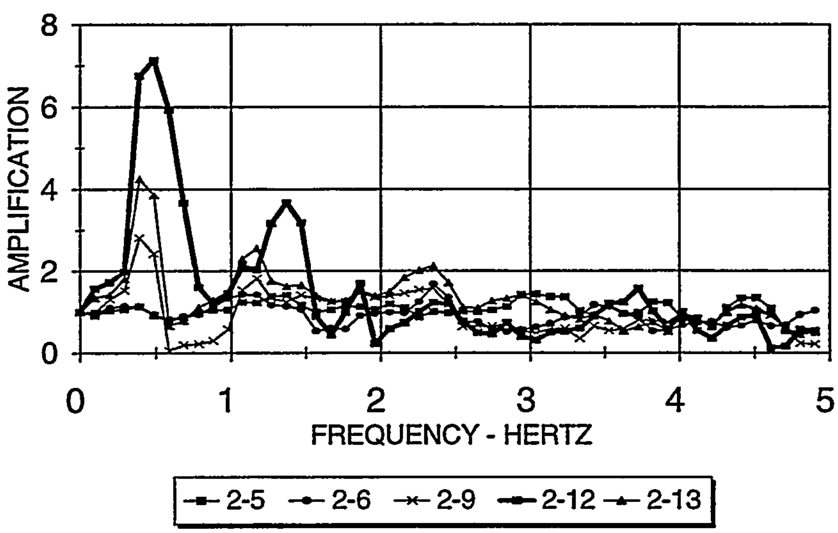


Figure 5.8 North-South Transfer Function

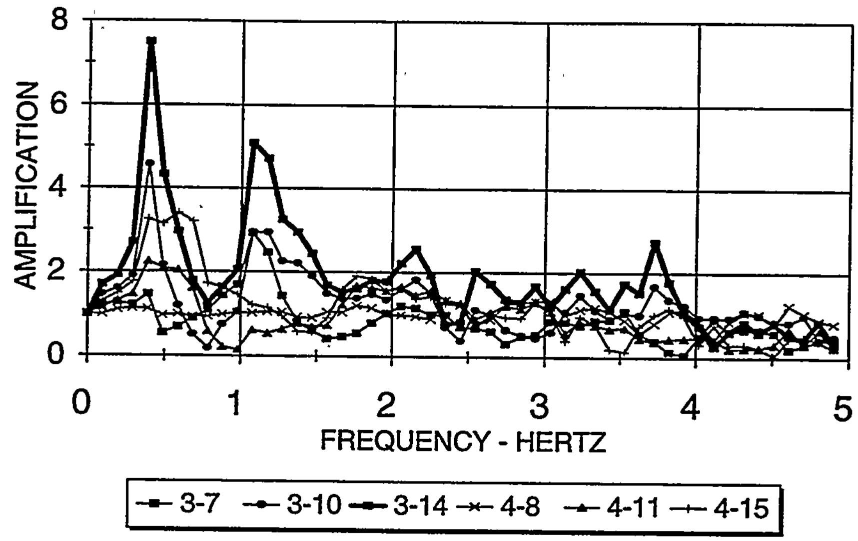


Figure 5.9 East-West Transfer Function

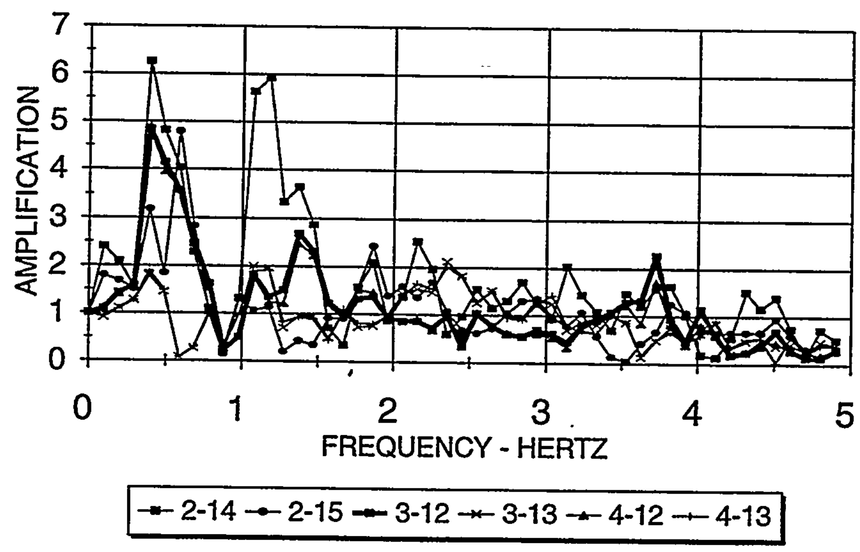


Figure 5.10 Torsional Transfer Function

The measured natural frequency can be compared to that which might be estimated using empirical formulas in the ASCE 7 code. Estimating the natural period as 0.1N, where N is the number of stories results in a predicted natural frequency of 0.77 Hz. The natural period can also be estimated as:

$$T = C_T h_n^{3/4} \tag{5.1}$$

in which T is the natural period, C_T is a coefficient, and h_n is the building height in feet. Using a value of C_T of 0.02 (that recommended for all other buildings, which infills appear to fall into) results in an estimated frequency of 1.2 Hz. A value of C_T of 0.050 results in an estimated natural frequency of 0.47 Hz, close to that observed.

The building thus appears to be quite flexible. Using standard code equations would significantly underestimate the true natural period of the building. This would in general lead to lower seismic forces in an earthquake load analysis. Probably much of the flexibility is due to the removal of the interior partitions in the 1969 renovation. Although the removal of the clay tile partitions decreased the mass, it eliminated much of the seismic resistance of the building.

Building Model

A three-dimensional model of the structure was constructed using GTSTRUDL, which is shown in Figure 5.11. No soil-structure interaction affects were considered, and the columns were assumed fixed against translation, but free to rotate, at their base. Soil-structure interaction effects will affect total displacements more than relative displacements. Since infill behavior is governed by relative displacements, soil-structure interaction was neglected.

The basic model also considered the building to be fixed against translation at the ground level. Due to the presence of large concrete walls around most of the perimeter of the basement, the basement was essentially considered to be rigid. The effect of this support condition was to make the columns at the ground floor level to be fixed against displacement, but have some rotational restraint due to the basement columns. The effect of fixing the ground floor against displacement was ascertained through a parametric study, to be discussed later.

Most of the steel columns were encased by masonry, particularly at the exterior. The masonry would form a pilaster around the column. Although testing has shown that this masonry pilaster probably contributes some to the stiffness, it was not included in the calculations. The moment of inertia used was that of the steel column.

The floor diaphragm was considered to be flexible. This was based on a relatively thin section (3 in) throughout most of the floor. The penthouse on the south side was not included in the analysis in terms of the member framing, but the mass was included.

An equivalent strut, as proposed in Chapter 3, was used to model the infill. Only the portion of the wall within the column boundaries (6 in) was considered as being effective. The only location of the infills was on the exterior faces of the building. For locations where there were openings, the equivalent strut was assumed to have half the stiffness and half the strength of a solid infill. Unfortunately, there is little experimental data for obtaining values of C, the parameter governing the stiffness of the equivalent strut, for steel framing with clay masonry infills. Although most of the exterior was brick, the parameters of C that were used were those for steel framing with structural clay tile masonry infills (Table 3.1). Values of C for concrete masonry in steel frames (Table 3.2) were also tried in the parametric study.

A masonry prism strength, f'_m, of 2000 psi and a modulus of elasticity of 1600 ksi was assumed for the clay masonry. These values are slightly lower than that used for Sepulveda Building 40 due to age of the structure. Ultimate strength of the infill was determined using Equation 3.4, with K=0.008. This resulted in a horizontal component of the capacity of 0.008(6)(2000)=96 kips for each infill.

A response spectra modal analysis of the structure was conducted using the base motions recorded in the basement. The two horizontal components (N/S and E/W) of the earthquake were considered, but not the vertical component. The response spectra generated from the recorded motion on the north side of the structure (station 3) was used for the E/W direction. The spectra used in the analyses were smoothed and are shown in Figure 5.12, with station 2 being the N/S direction, and station 3 the E/W direction. Damping was assumed to be 10%. The response from different modes and different directions were combined using square-root-of-the-sum-of-the-squares.

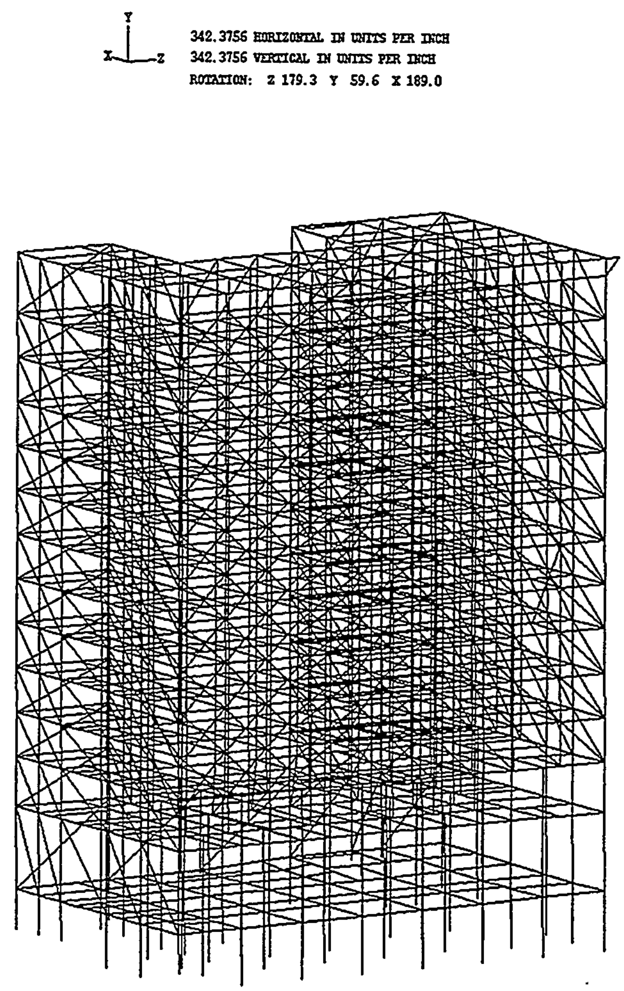


Figure 5.11 Building Model

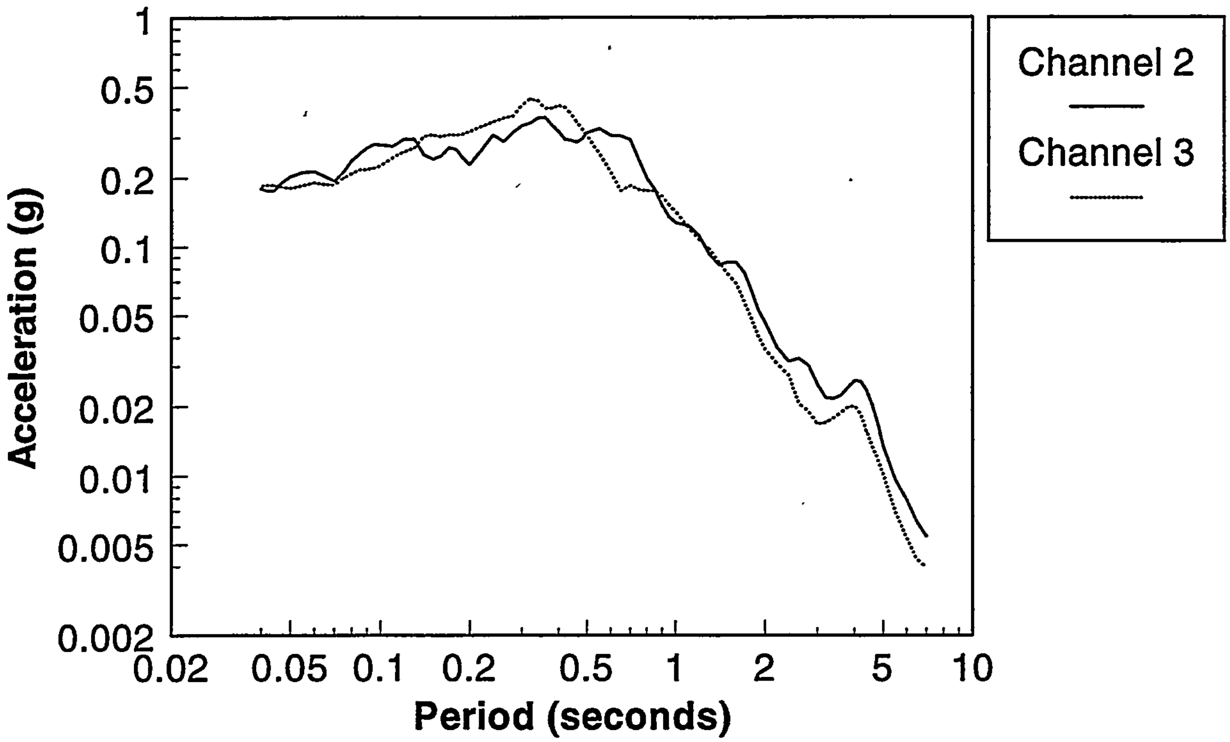


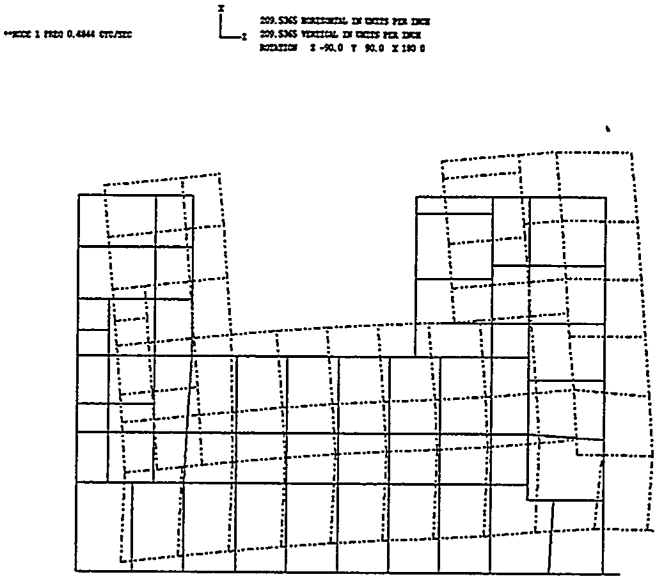
Figure 5.12 Response Spectra

Results

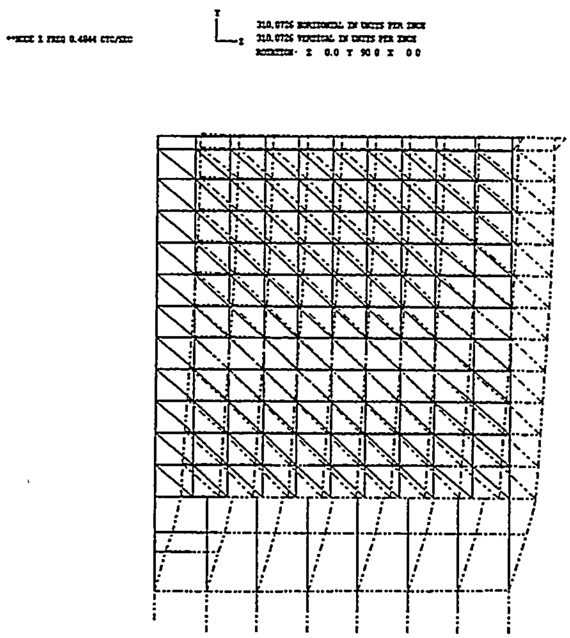
Due to the nonlinear nature of the equivalent struts for the infills, an iterative analysis linear was required. However, there was rapid convergence, within only one iteration being required.

Table 5.3 lists the first ten natural frequencies and the mass participation. A total of 80 frequencies were calculated, which resulted in 92.1% mass in the E/W direction, and 90.9% mass in the N/S direction. The first two mode shapes are shown in Figure 5.13 and 5.14. These figures show the displaced shape of the roof and the east and south side.

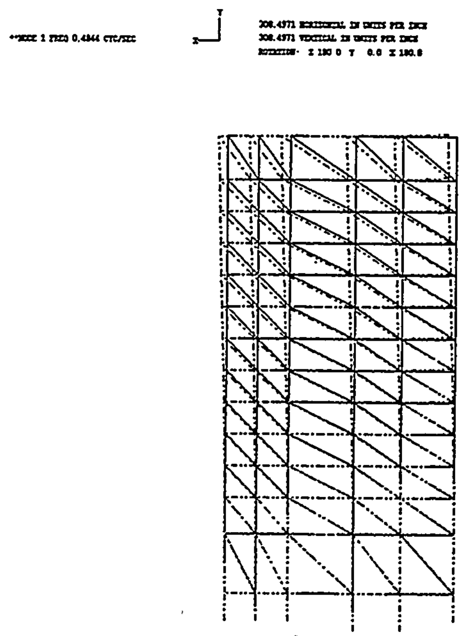
The two primary natural frequencies of the building had significant mass participation in each of the directions. These frequencies agree quite well with what was obtained from the analysis of the recorded response, which showed a primary frequency in each direction of 0.4-0.6 Hz. From the mass participation, it is also evident that there is significant coupling in the different directions, and a three-dimensional model is required. The analytical model, as well as the observed response, shows a secondary frequency at approximately 1.3 Hz in each of the orthogonal directions. Based on the calculated versus observed frequencies, the model appears to be reasonably accurate.



Roof Plan



East Elevation



South Elevation

Figure 5.13 Mode 1

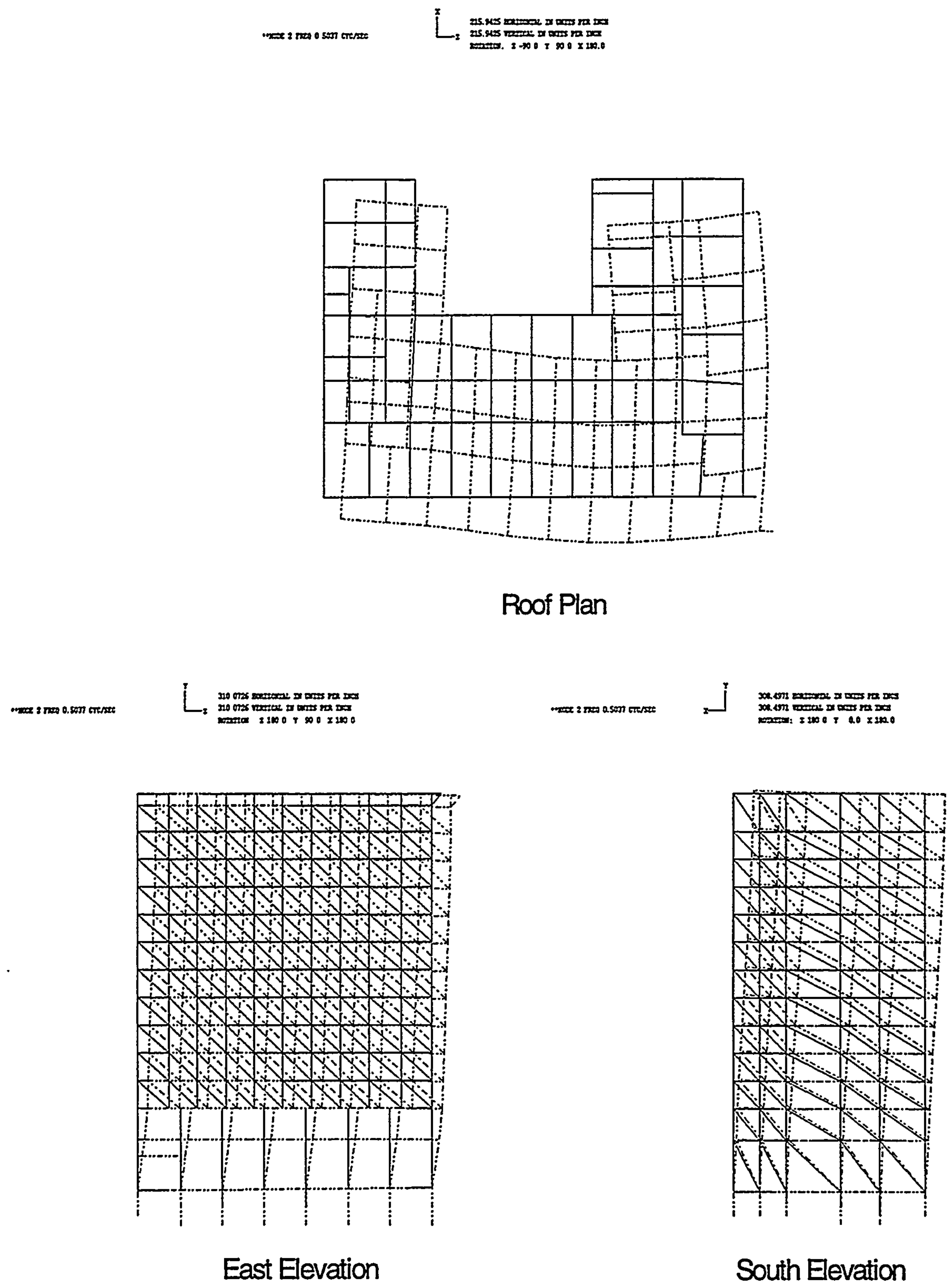


Figure 5.14 Mode 2

Table 5.3 Natural Frequencies and Mass Participation

Mode	Natural Frequency (Hz)	% Mass N/S Direction	% Mass E/W Direction
1	0.48	49.1	21.9
2	0.50	25.5	47.7
3	0.66	1.19	5.33
4	0.97	0.11	0
5	1.04	0.71	1.14
6	1.28	0.01	7.35
7	1.30	6.86	0.002
8	1.49	0.14	0.06
9	1.54	0.002	0.58
10	1.64	0.09	0.26

The calculated and recorded total accelerations at the top of the structure are shown in Figure 5.15. The calculated and recorded displacements at the top of the structure are shown in Figure 5.16. It should be noted that the calculated displacements are relative displacements, while the measured displacements are total displacements, and thus the two are not directly comparable. Although of the correct magnitude, both the calculated accelerations and calculated displacements are slightly lower than the measured values.

There are several possible explanations for differences between the calculated frequencies and measured frequencies. The value of the modulus of elasticity used for the masonry could have been in error. No data on the actual masonry properties was available. It is also possible that the values of C used were not correct. The effect of using C values for concrete masonry in steel frames instead of structural clay tile in steel frames is examined in a later parametric study. Another explanation for any differences is the support conditions. These were difficult to determine. The effect of support condition at the base, and a discussion on the modeling of the support condition, is given in conjunction with the parametric study. Finally, almost all the infills had some sort of openings. The effect of openings on infill behavior is not well understood, particularly for the large openings on many faces of the building.

For most of the structure, the value of C was 7. At a few places near the top of the structure, a C value of 5 was used. At several places near the base of the structure, a C value of 11 was used. Places where C=11 was used were the second floor on the south side, the first through fourth floors on the back west side, the second through fourth floors on the north side of the alcove, and the second floor of the east side of the alcove. This is reasonably consistent with the observed damage. For values of C=7, diagonal mortar joint cracking is expected. The small amount of cracking at this level would be difficult to see after an earthquake. Also, for a building this age, the cracking had probably already occurred from shrinkage, thermal movements, and other loadings. For C=11, off diagonal mortar joint cracking was observed in laboratory tests. This is consistent with the cracking observed on the back face (west side) of the structure. No displacements were of sufficient magnitude to cause the initiation of corner crushing.

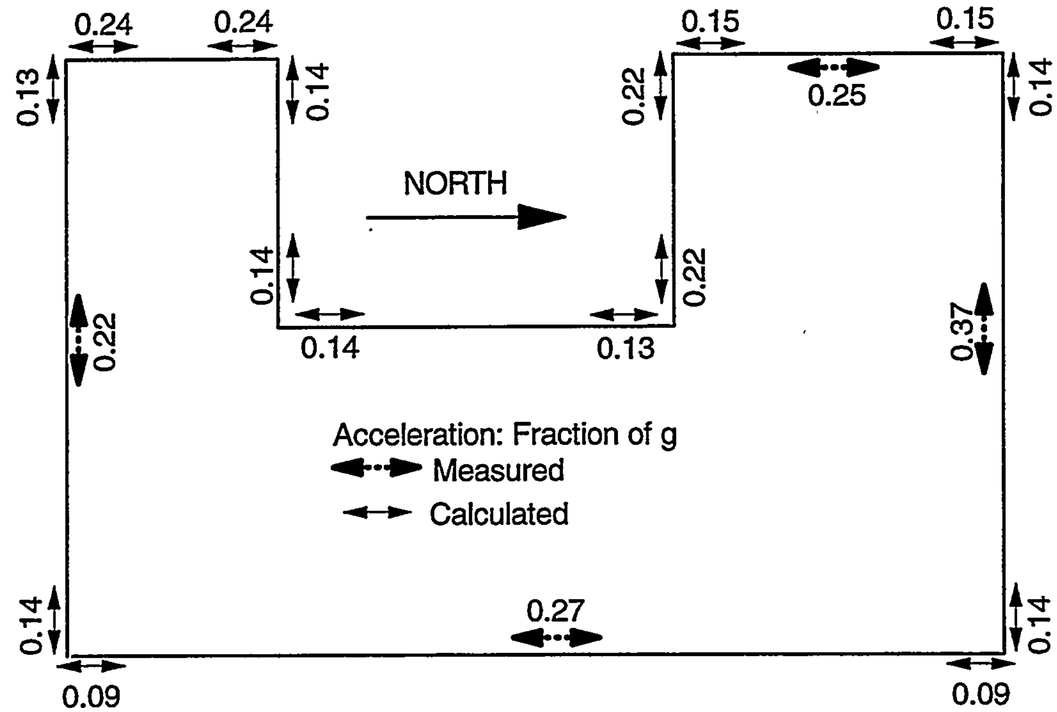


Figure 5.15 Measured and Calculated Accelerations at the Roof

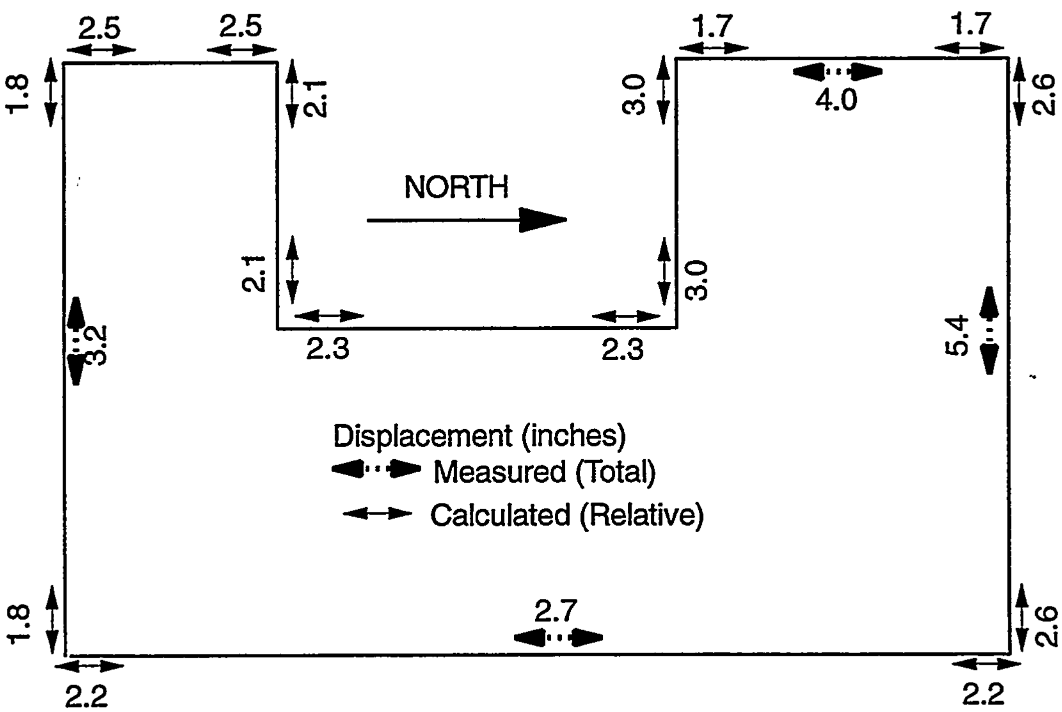


Figure 5.16 Measured and Calculated Displacements at the Roof

With the exception of a few struts on the lower floors of the north and east side of the alcove, no strut force exceeded the expected capacity. This is again consistent with the observed response. Given the uncertainty in obtaining both the ultimate load of an infill, and the uncertainty in the masonry strength, no modifications were made to the model where the strut capacity was exceeded.

To examine the effect of various assumptions, a parametric study was conducted, Table 5.4. Only the natural frequencies were extracted in the parametric study.

Table 5.4 Natural Frequencies from Parametric Study

Masonry Properties	Supports	Openings	Mode	Frequency (Hertz)	% E/W Mass	% N/S Mass
Clay Tile	In at Ground Level	50% everywhere	1 2	0.48 0.50	22 48	49 26
Concrete Masonry	In at Ground Level	50% everywhere	1 2	0.51 0.54	28 40	41 33
Clay Tile	None at Ground Level	50% everywhere	1 2	0.37 0.44	5 74	64 8
Clay Tile	In at Ground Level	50% S and E faces; 25% all others	1 2	0.45 0.47	21 46	46 26
Clay Tile	In at Ground Level	50% S and E faces; none at all others	1 2	0.32 0.40	13 43	39 23
No Struts	In at Ground Level	N/A	1 2 3	0.29 0.34 0.36	0.3 36 36	50 12 8

The first values listed in Table 5.4 are those that were obtained from the base run, as discussed above. As indicated in Chapter 3, there is some evidence that concrete masonry and clay masonry have similar stiffness (in terms of C values) when used as infill material. Thus, C values of concrete masonry were tried. For most of the infills, this C changed the C value from 7 to 5, almost a 30% decrease. The added stiffness did increase the natural frequency slightly, but less than 10%. The basic behavior remained the same, as evidenced by the percentage mass participation in the modes. It thus appears that any reasonable value of C will be sufficient to obtain the behavior and a plausible estimate of the response of the structure. Of perhaps particular note is that, as indicated in Chapter 3, the C values used were developed from tests in which the base of the infill was quite rigid. Tests in which the base of the infill was quite flexible indicated C values of 10 up to 100. From the building analysis conducted herein, it does not appear that these large C values are appropriate for use in actual building evaluation.

The next analysis examined the effect of the support conditions at the base. The restraints against displacement at the ground floor were removed. This did have a significant effect on the response. It shows the need to properly ascertain the actual support conditions. Unfortunately, this is often difficult to do. In the present structure, the column baseplates were at several different elevations. On the west side, the columns were supported by a concrete wall, and the baseplates were almost at the ground level. It was unclear from the plans to what extent the structure was tied to walls under the sidewalks. From the instrumented response, it appears that there was little relative motion and/or amplification of response between the basement level and the second floor. The transfer function also showed that there was little change in the frequency response between the basement and the second level. This gives a basis for the supports at the ground level. Instead of supports against displacement at the ground level, springs could be used. This springs could be tuned to match the recorded response. Since it was desired to analyze structure as if the recorded response were not available, the use and tuning of springs was not done.

The openings on the north side, east side, and in the alcove were quite large. No infill testing has really been performed with openings of this size. The extent to which these infills participate in the lateral behavior is unknown. It could well be argued that their participation is less than the 50% that normally seems to be the case for openings. Therefore, two other reductions of the infill stiffness were used. In the first, a 75% reduction (or 25% of the solid infill stiffness) was used for the large openings. In the second, no infills were considered except on the north and west side. Based on comparing the calculated frequencies to the measured frequencies, it appears that even with the large openings, there is still some participation from the infills. The frequencies obtained from only considering infills on the north and west side appear to be too low, or the stiffness is too soft. Although it is difficult to tell whether 50% or 25% stiffness should be used for the large openings, it is erroneous to ignore the contribution of the infills with large openings.

Finally, the structure was analyzed as just a frame structure with no infills. The frequencies were quite low, significantly lower than those measured. This shows the effect of infills, even infills with large openings, on the structural behavior. To properly analyze a structure under seismic loading, infills need to be considered.

Critical Discussion

The analyzed building was quite complicated in terms of geometry, the alcove setback in the second through thirteenth floor, and the stiffness irregularities in both plan and elevation. Nonetheless, the use of the proposed equivalent strut model enabled natural frequencies to be obtained which were in agreement with those recorded during the Northridge earthquake. For the moderate level of shaking that this building experienced, the piecewise linear equivalent strut method was quite efficient, requiring only one iteration.

There are two areas where the analysis is perhaps slightly suspect. The first area is the support conditions at the base. For this structure they were quite complicated. The support conditions are an area of uncertainty for all buildings, not just infill buildings, and thus is outside the scope of this research. The second area is the behavior of infills with large openings. Only a limited amount of testing has been done on infills with openings, and many of the tests are not representative of actual buildings. There is the need for additional information on infill behavior with large openings typical of mid-rise commercial construction.

This building was subjected to a level of shaking typical of the large moderate seismic zones in the central/eastern United States. Despite having several very poor seismic details, it performed quite well. The performance was typical of that of many of the older buildings in downtown Los Angeles. Infills can be quite beneficial in moderate seismic zones. In this case, they provided sufficient lateral stability to the structure so that only minor, repairable damage occurred. The building remained open and usable after the Northridge earthquake.

References

ASCE 7 (1993). Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers Standard 7-93.

CDMG (1995). Processed CSMIP Strong-Motion Records from the Northridge, California Earthquake of January 17, 1994. California Division of Mines and Geology Strong Motion Instrumentation Program, Report OSMS 95-01T.

CHAPTER 6

CONCLUSIONS

Conclusions from this research are drawn in several areas, including construction of new infills, seismic walkdown guidelines for infill structures, expected infill performance, analysis of infill structures, experimental testing of infills, and critical areas of research. Detailed conclusions and recommendations concerning infills are provided in the following.

Construction of New Infills

New infills are either isolated from the bounding frame, or snugly confined in the bounding frame. There are certain problems with both types of construction. Recommendations for both types of construction are given in the following.

Isolated Infills. Isolated infills need to have a sufficient gap so that seismic drifts can take place without the frame contacting the infill. In the instance of a seismic hazard exposure group I building, the gap required might be as much as 0.025 times the story height (ASCE, 1993). For a story height of 10', this would be a 3" gap. Thus, a typical gap of 1 inch may not be sufficient. Unforeseen damage and load distribution may occur if the infill is allowed to come in contact with the frame during an earthquake.

Isolated infills need to have out-of-plane anchorage. This detail is quite difficult, as there needs to be free in-plane movement. Any restraint against in-plane movement will cause significant force transfer. This can lead to premature connection and localized masonry failure, which may result in loss of out-of-plane support. Thus, isolated infills require careful detailing to allow for in-plane slip.

Given the difficulty of detailing and constructing isolated infills, tightly fitted infill construction may be more practical. In moderate seismic zones tightly fitted infills may enhance the lateral force resistance of building structures. Thus, tightly fitted infills have the beneficial aspects of improved seismic behavior and economic construction.

Tightly Fitted Infills. Tightly fitted infills can greatly enhance the lateral stiffness and capacity of the structure. The infill behavior needs to be considered in the design and analysis of the structure so that poor seismic details, such as torsional irregularities and partial height infills, are not inadvertently designed into the structure. With infills generally not included in building codes, most consulting engineers are not familiar with infill behavior, analysis and design. For inclusion of infills in routine building analysis, a tractable analytical methods is necessary. This research has led to the development of such a method.

The researchers have observed the construction of tightly fitted infills that used some details that added to the expense of the structure without adding any beneficial behavior, and in some cases detrimental behavior. Figure 6.1 shows a typical infill construction. The infill was anchored to the columns using dovetail anchors. Presumably this is done to anchor the panel against out-of-plane motions. The primary out-of-plane resisting mechanism is arching, for which the anchors do not contribute. The anchors can lead to premature in-plane cracking, which can reduce the panel capacity. It is therefore recommended that these anchors be eliminated. A similar situation exists at the top of the infill. For example, Figure 6.1 illustrates a steel plate embedded into the underside of the concrete floor slab, an angle field welded to the plate after completion of the masonry wall, and an anchor installed to anchor the wall to the frame. As with the side anchors, this does little to enhance the out-of-plane stability of the wall. The anchors can also lead to localized failures of the infill under in-plane loading, which can cause a premature out-of-plane failure, or a partial height infill and premature shear failures of the columns. Again, it is recommended that these anchors be eliminated. A tightly constructed infill has in general a large factor of safety against out-of-plane loads due to arching, and can add to the in-plane stiffness and strength.

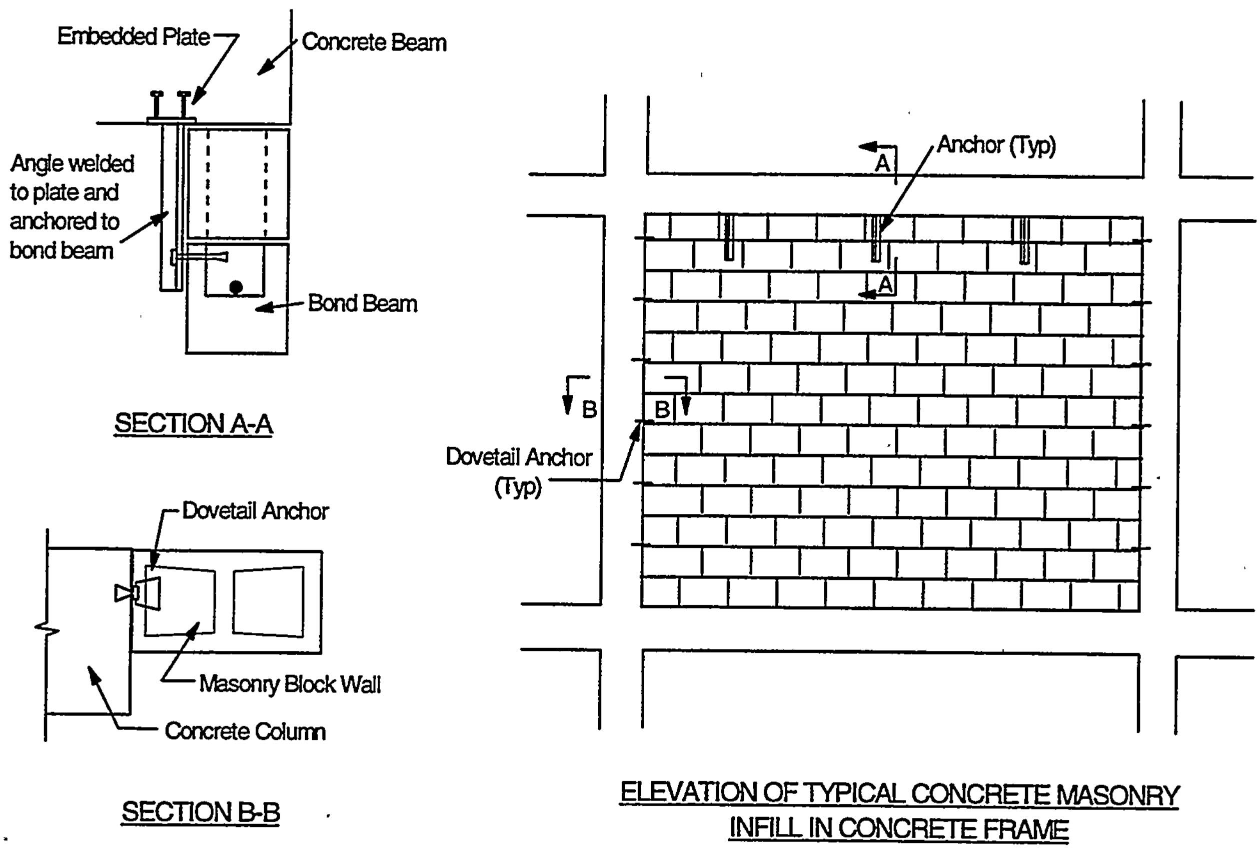


Figure 6.1 Typical Infill Construction

Seismic Walkdown Guidelines for Infill Structures

The primary focus regarding infills during a seismic walkdown of an existing facility should be the interface between the infill and the bounding frame. If the interface is tight, then arching can develop and the infill will have out-of-plane stability except in cases of extremely large height to thickness ratios. If the infill is tight against the frame, particularly in the upper loaded corners, then it can be expected to have significant in-plane seismic resistance.

Partial height infills often lead to premature shear failure in the columns. Thus, partial height infills should be carefully noted, and steps taken to guard against column failure. Options include removing the infill, isolating the infill (with out-of-plane restraint), filling in the opening, or strengthening the column against shear failure.

Other aspects of the infill construction that should be noted in a seismic walkdown include the amount of infill thickness enclosed by the bounding frame, the type and construction of the masonry, the size and location of openings, any structural cracking or damage of the infill, and any significant deterioration of the mortar or the units.

Expected Infill Performance

Many infill structures exist in the large moderate seismic zones of the central/eastern United States, and infill structures continue to be built on a regular basis. Although high magnitude earthquakes are rare in these regions, life safety functions must be considered. Based on observed and calculated behavior, it is expected that the infills will be very beneficial to the seismic performance. Tightly fitted infills should have sufficient strength to provide the life safety functions, and in many cases should limit damage to the extent that building remains open and useable after a moderate earthquake. Only minimal repairs would be expected. Infills may protect much of the building stock against moderate earthquakes without any further retrofit or seismic rehabilitation.

Although infills have at times proven to be beneficial in high seismic zones, detrimental behavior has also been observed. The initiation of corner crushing may cause parts of the masonry infill to fall out, creating a hazard to occupants and those outside the building. Corner crushing may also lead to high shear forces in the columns, and thus column failure. Torsional irregularities due to arbitrarily placed infills will be more detrimental in high seismic zones.

Analysis of Infill Structures

The equivalent strut methodology is a tractable means of incorporating infill response into large three-dimensional models of structures. The piecewise linear equivalent strut methodology adequately captures the nonlinear global, or macro, behavior of the infill. The method developed herein provides rapid convergence, and gives an indication of the expected infill damage based on the amount of drift.

In-plane capacity of the infill is dominated by a corner crushing limit state and is primarily a function of the thickness and compressive strength of the confined infill. A simple method was developed for predicting the capacity. The method was surprisingly consistent for very diverse infill tests. It is noted that there are other failure modes of infilled frames with the most critical being a shear failure of a column. Additional work is needed to define the shear loads in columns and the other forces in the bounding frame.

Experimental Testing of Infills

Infilled frames have been tested in a wide variety of ways, including concrete and steel frames, different types of masonry, differing support configurations, and many other different details. Although this leads to data on many different cases, it does make it difficult to synthesize the results, and develop a general understanding of infill behavior. As a result of this research, the following recommendations are made concerning the in-plane testing of infills.

Adequacy of Single Bay - Single Story Frame Testing. Most of the infill testing has been on single bay, single story frames. Although testing of multi-bay and multi-story frames is appealing, it appears that the results of single bay, single story frames can be extrapolated to large, complex structures. Thus, the continued testing of single bay, single story frames is encouraged for economic reasons. The coordination of single-bay, single-story testing with larger size specimens might be important for system integration. Care needs to exercised in the application of loads to multi-bay, multi-story structures so that they are truly representative of the load distribution in actual infill structures.

Base Support Conditions. Much of the infill testing has been performed with the base of the infill bearing on strong, stiff reaction floors or beams. Other infill testing has been performed where a rectangular frame was constructed, and only supported at the corners. This allows for bending of the base beam. These tests have resulted in much lower values of stiffness than for testing on a strong floor. From the analysis of actual structural performance, it appears that stiffnesses from frames supported only at the corners are too low. It is recommended that infill testing continue to be conducted on strong, stiff floors or beams. The stiffness obtained from these tests does appear to be appropriate for general structures. Due to diaphragm floor action, and in many cases infills continuous throughout the height of the structure, it appears testing on a strong, stiff floor results in realistic values of stiffness.

Definition of Failure. Some of the infill testing that has been performed has been at low displacement levels. The test is stopped after cracking. Infills have significant ductility and load carrying capacity well beyond cracking, and even beyond corner crushing and loss of some masonry. The ductility of the infill can greatly enhance the seismic performance. It is imperative that infill testing capture the post-cracking and post-peak behavior of the infill frame system. Thus, infills need to be tested to high displacement levels. It is also desirable that cyclic testing be conducted as opposed to monotonic testing.

Critical Areas of Research

There are still many aspects of masonry infill behavior that are not well understood. However, the research needs must be prioritized. It is recommended that future infill research focus on several critical areas, as outlined in the following.

Test Series of Clay and Concrete Masonry in Both Steel and Concrete Frames. Researchers performing infill frame testing have almost exclusively used one frame material (steel or concrete), and one type of masonry (clay brick, concrete masonry, structural clay tile). Due to differences and idiosyncrasies of testing, it is difficult to compare the results of the different tests. An experimental program that would perform identical testing of different types of masonry in both steel and concrete frames would be extremely valuable in developing general infill analytical methods.

Testing of Infills with Typical Openings. Most of the infill testing has been performed on solid infills. Only a limited amount of infill testing with openings has been performed, and much of this has not captured some of the typical configurations. It is recommended that an infill test series be conducted that focuses on typical openings.

Openings in commercial structures can be quite large (35-50% of the infill area). The opening is often centrally located in the panel. There may be two openings, with a masonry pier between the openings. Often the opening will extend almost to the column. Many older structures of this type exist in both high and moderate seismic zones, and infill construction with large openings continues in moderate seismic zones.

The openings in industrial facilities are typically of two types. One type is a smaller opening for mechanical equipment, piping, and ductwork. These openings will often be in very critical areas of the infill panel, such as the upper loaded corners. Another type of opening in industrial settings is ground floor infills with large overhead door openings for loading docks.

Significant testing of infills with openings is needed, but with openings that are typical of actual construction.

Analysis Methods for Infilled Frames. Research needs to be continued in the refinement of the piecewise equivalent linear strut model for global analysis of infilled frame structures. Additional research needs to be conducted with regard to the individual panel behavior, and in particular the transfer of forces between the frame and the infill panel. Tractable methods need to be obtained for determining the force distribution within the frame members as a result of the infill. The development of nonlinear finite element methods needs to continue. The methods need to be validated with actual tests and comparison to performance of actual structures.

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