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Infill Panel System for Seismic Strengthening of Flat-Plate Buildings

by

Francis Kam Humay

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE

Doctor of Philosophy

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AUGUST, 2000
ABSTRACT

Infill Panel System for Seismic Strengthening
of Flat-Plate Buildings

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Francis Kam Humay

Many existing flat-plate buildings are seismically deficient and pose a threat to life safety if subjected to ground motions of even low to moderate intensity. Failure in such structures is typically the result of punching failure at the slab-column connection. Because of this, performance-based retrofit procedures are needed to upgrade these non-ductile buildings. This investigation evaluated the use of lightweight pumice stone concrete (LWPSC) infill panels as a retrofit alternative for flat-plate buildings.

Six four-tenth-scale slab-column subassemblies were designed and detailed based on ACI 318-63 and current performance-based testing requirements. Except for one bare frame specimen, all the subassemblies were retrofitted with prefabricated LWPSC infill panels and subjected to quasi-static loading conforming to FEMA 273. The geometry of the individual units was governed by weight limitations for handling and erection. Among the variables studied were connections between the slabs and the infill wall and the addition of uniformly distributed perforations (circular and rectangular openings).

All of the retrofitted specimens had significant increases in both strength and stiffness over that of the bare frame. The behavior of the specimen with the infill panels not attached to the slabs was similar to that of a masonry wall without any connections to the
frame. Although diagonal tension cracks formed within the recessed region, ultimate failure of the infill did not occur. Instead, frame-wall interaction transmitted large concentrated shear forces into the column that eventually contributed to failure of the longitudinal tension splice.

The remaining subassemblies all had connections to the slabs and perforations within the wall. Specimens with circular holes experienced uniformly distributed cracking throughout the entire area of the infill wall. The chosen configuration, however, did not sufficiently weaken the wall, and shear failure of the column stopped the test. Because of its ductility and energy dissipation mechanism, the most promising infill panel configuration contained rectangular perforations. Two different reinforcement patterns were tested using rectangular openings. The addition of diagonal reinforcement between openings had the effect of increasing the yield strength of the wall as well as better maintaining post-yield deterioration.
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CHAPTER 1
INTRODUCTION

1.1 General

A large portion of existing reinforced concrete (RC) flat-plate buildings are seismically deficient and pose a risk of catastrophic failure if subjected to earthquakes of low to moderate intensity. Many of these buildings, particularly in the central and eastern parts of the United States, were only designed for gravity and wind loading and therefore do not have the lateral stiffness and proper connection detailing needed to withstand a seismic event. Experience from past earthquakes including the 1971 San Fernando [1], 1985 Mexico City [2] and others, clearly elucidates the potential for overwhelming damage and loss of life. The vulnerability of flat-plate buildings is mainly attributed to excessive lateral drift and punching shear failure at the slab-column connection. In many instances, the lack of continuous reinforcement (particularly bottom reinforcement) through the joint has resulted in the progressive collapse of floors during moderate earthquakes. Laboratory tests have demonstrated that slab-column connections in existing non-ductile flat-plate buildings exhibit insufficient lateral stiffness and have a limited moment-transfer capacity [3].

Because punching shear failure usually results in local collapse, and often triggers progressive collapse [4], non-ductile flat-plate buildings pose a substantial threat to life safety. It is, therefore, imperative to develop strategies for the retrofit of these potentially
hazardous structures. An appropriate and cost-effective retrofit design should satisfy an owner’s performance objectives by considering both the actual loading and inelastic response buildings experience in an earthquake. These goals are best achieved using performance-based design and analysis methodologies.

As non-ductile flat-plate buildings continue to age, the expedient development of performance-based seismic retrofit techniques becomes increasingly important. Results from past research reveal two possible retrofit solutions: strengthening of the connections and control of lateral drift. Luo and Durrani previously investigated schemes that involve increasing the connection capacity [5]. The second option can be achieved by the addition of stiffening elements such as shear walls, steel bracing or infill walls. Since the addition of shear walls or steel bracing may be architecturally intrusive and relatively expensive, the use of infill panels emerges as a more attractive solution. Previous experimental findings indicate the ability of infill panels to significantly improve the seismic performance of bare beam-column frames [6, 7, 8], but no test data exists for slab-column frames containing infills.

A retrofit strategy for flat-plate buildings was developed by experimentally testing four-tenth scale slab-column subassemblies retrofitted with different configurations of lightweight infill panels. In order to generate data that was applicable to performance-based rehabilitation, all of the tests conformed to the requirements in section 2.13 of FEMA 273 [9]. Fully reversed cycles of increasing displacement were used to simulate earthquake ground motion.
1.2 Objectives and Scope

The overall goal of this research was to develop a performance-based retrofit strategy for seismically deficient flat-plate buildings using lightweight, prefabricated infill panels. In order to achieve this goal, three main objectives were defined. The first objective was to choose an infill wall material and develop a panel layout compatible with non-ductile flat-plate frames. Secondly, it was decided to experimentally quantify the behavior of slab-column subassemblies retrofitted with the proposed system. The test set-up and procedures were designed to provide the information needed to analyze a new retrofit system within the framework of performance-based design. The final objective was to produce lateral force-deformation pushover curves, acceptance criteria and general recommendations for the performance-based retrofit of flat-plate structures.

The major tasks comprising the scope of this work were arranged as follows:

- Investigation of various promising lightweight materials and the selection of the most suitable alternative based on material tests, engineering properties, ease of construction and cost.

- Design and construction of four 0.4-scale slab-column subassemblies that accurately represented actual construction details, support conditions and loading conditions.

- Design and construction of infill panels and retrofitted subassemblies.

- Quasi-static testing of a bare frame specimen and retrofitted subassemblies.

- Developing lateral force-deformation pushover curves from the experimental data.
• Developing acceptance criteria that couples performance levels with permissible values of drift limitation, component strength demand or inelastic demand.

• Providing recommendations for the performance-based retrofit of flat-plate buildings using the proposed system.
CHAPTER 2
BACKGROUND

2.1 Flat-Plate Buildings

2.1.1 Behavior and Structural Weaknesses

There are a number of deficiencies associated with non-ductile, flat-plate buildings located in seismic regions. The primary problem is that existing flat-plate structures have low lateral stiffness, and may be prone to excessive lateral drift under seismic excitation. Without additional lateral force resisting elements (i.e. shear walls or braced frames), slab-column connections must resist both gravity and lateral loading (unbalanced moments). The low lateral stiffness of flat-plate buildings can be attributed to the sole reliance on "inherently flexible" slab-column connections to carry both vertical and lateral forces. Even if a flat-plate system can accommodate large deflections without collapse, damage to non-structural elements of the building can be extreme. Further aggravating this condition, it has been shown that the horizontal stiffness significantly decreases under cyclic loading, thus increasing drift. In tests of two-bay slab-column subassemblies, Durrani, Du and Luo found that approximately 50% of the initial lateral stiffness was lost during the first 1.0% drift [3]. Other important properties that control the behavior of these structures have been observed through this type of testing.

Slab-column connections of flat-plate buildings have a low energy-dissipation capacity. Tests of individual non-ductile, slab-column connections and multi-connection
subassemblies reveal lateral load-deflection relationships comprised of severely pinched hysteresis loops (Durrani, Du & Luo, and Dovich & Wight) [3, 10]. Since the area enclosed within a particular hysteresis loop represents the amount of energy dissipated during one complete cycle of loading, any pinching in these curves indicates an inability for the system to dissipate energy. Mechanisms for dissipating energy in flat-plate structures include: yielding of slab reinforcement, opening of flexural cracks, slippage of slab reinforcing bars, local crushing of concrete and friction along major cracks. In addition, experimental research has also found that the lateral displacement ductility of reinforced flat-plate interior connections is low by comparison with values often considered marginally acceptable in seismic design. Data collected from various researchers and analyzed by Pan and Moehle suggests a trend in the amount of displacement ductility available in a slab-column connection [11]. A primary variable affecting the displacement ductility of flat-plate connections is the magnitude of the gravity load shear carried by the slab. As the gravity shear ratio increases the displacement ductility decreases.

As previously mentioned, slab-column connections must be able to resist both gravity loads and unbalanced moment. Any unbalanced moment is transferred through a combination of eccentric shear along the critical perimeter and flexure of the slab. Typical slab detailing of older flat-plate structures, however, did not require the bottom reinforcement to be continuous through the column. Connections were solely designed to resist negative moment, without any consideration of possible moment reversals during earthquakes. The positive moment transfer capacity of these slabs is, therefore, limited to the cracking strength of the slab (Durrani, Du & Luo) [3]. If the shear stress due to both
gravity loading and unbalanced moment exceeds the shear capacity of the slab, the connection is susceptible to punching shear failure. Punching shear is a brittle failure mechanism that may ultimately result in the progressive collapse of floor slabs. In addition to experimental data, observations from past earthquakes have provided the most conclusive evidence of the vulnerability of flat-plate structures. During the 1985 earthquake in Mexico City where flat-plate construction is widely used, 41% of collapse or serious damage occurred in buildings of this type [2].

2.1.2 Non-Ductile Reinforcing Details

The basic design approach and detail requirements for flat-plate buildings remained practically unchanged in the ACI 318-41 [12], ACI 318-56 [13] and ACI 318-63 [14] building codes. Working stress methods were used to design the slab-column connections for punching shear and the effect of unbalanced moment was neglected. The reinforcing detail at an interior slab-column connection did not require the slab bottom bars to be continuous through the column. Instead, fifty percent of the positive slab reinforcement was terminated 3" from the column centerline, and the remaining bars stopped at 0.125L (L = span length) from the support centerline in the column strip and at 0.15L in the middle strip (Fig. 2.1(a)).

If flat-plate buildings are subjected to lateral loading, however, the slab-column connection must also resist an unbalanced bending moment. Research has shown that both gravity loading and unbalanced bending moments contribute to the total shear stress acting on the critical section of a slab-column connection. As a result, the capacity to resist gravity loads is diminished as the unbalanced moment is increased. Excessive shear
demands can ultimately result in the development of a punching shear failure mechanism (Fig. 2.1(b)). As shown in Fig. 2.1(b), the top reinforcement is pulled through the shallow concrete cover and, therefore, cannot suspend the slab after such a failure. Because of a steeper angle of inclination, however, continuous bottom bars have been found to provide an efficient method of support. Hawkins and Mitchell found that the residual shear capacity provided by continuous bottom bars could be calculated as $A_s' f_{ys} n$ [4]. In this expression $A_s' = \text{the area of a bottom bar, } f_{ys} = \text{yield strength in shear (50\% of the yield strength in tension)}$ and $n = \text{the total number of bottom bars that are continuous through the column.}$

The columns of existing flat-plate frames also possess non-ductile detailing that may lead to column failure and catastrophic collapse. In gravity load designed buildings the column reinforcement splices are typically designed for compressive forces. Because earthquake ground motions can induce large tensile forces in a structure's columns, the inability of the reinforcement splice to transfer these forces may result in a tension failure of the column. Another deficiency in older column detailing is the large spacing of column ties. This excessive spacing does not conform to today's stricter seismic code provisions that are based on providing confinement for the concrete and lateral support for the longitudinal bars. As a result, the column's low shear resistance may ultimately trigger a brittle failure.

2.1.3 Punching Failure vs. Lateral Drift Ratio

The analysis of published experimental data reveals that the amount of gravity load carried by a slab is the primary variable affecting both the strength and ductility of slab-
column connections. As illustrated in Fig. 2.2 [15], the available lateral drift capacity of interior slab-column connections is highly sensitive to changes in the gravity shear ratio, $V_g/V_o$, in the region from 0.2 to 0.5. In this ratio, $V_g$ represents the applied shear due to gravity loads and $V_o$, where $V_o = 4\sqrt{f'_c} b_o d$ (as defined by ACI 318-89) [16], is the theoretical punching shear strength in the absence of an unbalanced moment. Pan and Moehle have extensively studied flat-plate buildings subjected to seismic loadings [11]. Based on their findings, slab-column connections can exhibit some minimal ductility and a drift capacity of at least 1.5% only if the percentage of shear due to gravity loading is less than approximately 40% of the punching shear capacity. Furthermore, according to ACI 352, the maximum allowable shear stress on the critical section of a slab-column connection in conjunction with inelastic moment transfer should not exceed $1.6\sqrt{f'_c} \times 4\sqrt{f'_c}$ [17]. Since the gravity shear ratio in existing buildings is already established, punching failure of the interior connections can be avoided by controlling the interstory lateral drift.

### 2.1.4 Seismic Retrofit

According to Sugano, there are three aims for seismic rehabilitation: (1) to recover original structural performance after an earthquake, (2) to upgrade original structural performance prior to experiencing any damage and (3) to reduce seismic response (typically by using base isolation, energy dissipating devices or reducing the seismic mass) [18]. Since RC flat-plate buildings are vulnerable to severe damage under moderate intensity earthquakes, any substantial seismic event would most likely render such a structure irreparable. The main objective of a flat-plate retrofit scheme should,
therefore, be to upgrade original structural performance. The third option, although technically sound, would probably be too cost prohibitive for most flat-plate construction.

One method of minimizing the hazard associated with this type of construction is to control the lateral drift and/or reduce the amount of applied gravity load on the floor slabs, as previously illustrated in Fig. 2.2. Because of their excessive flexibility, it is common for engineers to design flat-plate retrofit schemes, such as the addition of new shear walls or braced frames, capable of independently resisting the entire lateral load. This approach, however, may be uneconomical for building owners and highly disruptive to the functioning and aesthetics of a building. The addition of infill walls is a more cost-effective and less intrusive solution that a number of researchers have previously investigated for non-ductile beam-column frames. Although much can be learned from these past studies, new research is required to develop a reliable infill panel retrofit technique for flat-plate systems.

Individual slab-column connections can also be retrofitted to protect against progressive collapse once punching failure has occurred. Luo and Durrani proposed using an external steel capital that was attached to the column directly underneath the slab [5]. Tests conducted on interior slab-column connections retrofitted in this manner demonstrated the feasibility of this retrofit technique. It was found that an external steel capital could prevent progressive collapse following a punching failure if two conditions were satisfied. First, the dowels that transfer shear from slab to column must be able to sustain the whole shear load. Second, the steel capital must extend a distance larger than $2.5d$ (where $d$ is the effective thickness of slab) from the face of the column.
2.2 Infilled Frames

2.2.1 General

Many existing buildings, in areas of every seismic risk level, have concrete block or brick masonry infills designed as non-load bearing partition walls. Unless completely isolated from the surrounding frame, these infills can alter the seismic behavior of a building considerably. Although masonry panels typically increase the strength, stiffness and ductility of a structure, it is a common misconception that they must always be beneficial to seismic performance [19]. Because masonry infills can significantly alter a structure’s intended response, seismic forces may be attracted to portions of the building that have not been designed to resist them. Large concentrated shear forces can be imparted to the bounding frame’s columns and beams due to the complex interaction between infill and frame. In addition, brittle failures of unreinforced infills have resulted in shedding of masonry into streets or stairwells, with great hazard to life.

Various possible failure mechanisms exist for masonry infilled frames [19]. For frames of high aspect ratio the failure may be similar to that of a slender cantilever shear wall yielding in flexure. Within the masonry, three mechanisms: sliding shear failure, diagonal tension cracking and compression of the diagonal strut can occur. Flexural or shear failure of the bounding columns may also govern. Most probably a combination of these modes will lead to the eventual failure of the system.

Since seismic retrofit has become an increasingly critical issue, infill panels are being used to repair damaged buildings and upgrade existing ones. In particular, the rehabilitation of RC buildings has been researched using various infill materials and
panel configurations. Experimental testing has been conducted with cast-in-place (CIP) concrete, precast (PC) concrete (using both single and multiple panels), masonry, steel panels and shotcrete. Results from these programs and the eventual success of a given infill panel system is not only dependent on the choice of material, but also the connection detailing between the new and existing elements. Dowels, epoxy, wedge anchors and shear keys have all been used to connect CIP concrete to original construction. PC concrete panels have been attached using both welded and bolted connections. For retrofit purposes, mechanical anchorage systems have also been developed for masonry infills. Based on a compilation of experimental results, Sugano compared the typical load-displacement behavior of frames strengthened with various construction techniques. The main findings of the investigation were summarized as follows: (1) CIP infill walls exhibited nearly the same strength as monolithic walls, (2) multiple PC panels provided improved ductility over bare frames, but much lower strength than solid walls, (3) concrete block and brick masonry also significantly increased strength [18].

2.2.2 Experimental Tests of Infilled Beam-Column Frames

During the past forty years a considerable number of experimental investigations on the strength and stiffness of infilled frames have been carried out. These tests have included both RC and steel frames with various infill panel materials and configurations. The majority of these tests were conducted with single-story single-bay frames subjected to either monotonic or cyclic loading. The earliest infill panel tests occurred in the 1950's. During the period from 1951 to 1956, Benjamin and Williams tested one-story
RC and steel frames infilled with plain concrete, reinforced concrete and unreinforced brick masonry [20, 21]. Variables considered in the study included: loading, materials, panel design, frame design and method of construction. The load-deflection relationship under monotonic loading was established and used in developing simple methods for predicting the stiffness and ultimate strength of the infilled frames. In the 1960’s, Holmes considered steel frames with non-structural concrete and clay brick infill panels [22]. He found that the in-plane stiffness of an infilled frame depended on both the relative geometry and the mechanical properties of the frame and infill. At the same time, Stafford Smith was experimenting with diagonally loaded models composed of mortar infills surrounded by mild steel bars [23, 24]. It was shown that size, thickness, proportions, material properties and length and distribution of load on the corner affected the stiffness of the system. All of the aforementioned specimens were tested until failure under monotonic loading. Since those early investigations, however, a considerable number of researchers have examined infilled frame behavior under cyclic loading. Cyclic loading is capable of more accurately representing the demands imposed on a structure by actual ground motions.

The most common test configuration consisted of single-story single-bay subassemblies infilled with masonry and loaded in plane. One extensive program of this type was initiated by Zarnic in 1983 and included cyclic tests on 28 reduced-scale RC specimens [25]. The main goal of the study was to define the inelastic behavior of these structures when subjected to simulated earthquake loading. The effects of openings and different repair techniques were also investigated. More recently Mehrabi et al. tested twelve half-scale specimens with RC frames and masonry infills [26]. In this study the
variable parameters included the relative strength of infill to that of the bounding frame, the panel aspect ratio, the distribution of vertical loads and the lateral load history. It was demonstrated that the failure mechanism of an infilled frame is greatly dependent on the relative strength of the infill to the frame. In addition, similar testing has been carried out with confining frames constructed of structural steel. Dawe and Seah tested 28 large-scale specimens under monotonic lateral loading [27]. Of all the parameters studied, they concluded that the interface condition between bounding frame and infill panel most significantly affected strength and overall behavior. Mander et al. [28] studied brick infilled steel frames with and without retrofit. The researchers found that unreinforced brick infills behave in a ductile manner under in-plane loading. Such infills, however, were vulnerable to fallout under horizontal loads perpendicular to the wall.

The out-of-plane behavior of masonry infilled frames has also been the focus of a number of studies. Dawe and Seah tested nine large-scale concrete masonry panels with steel enclosing frames [29]. An air bag was used to distribute a uniform pressure to the face of the infill. It was found that the out-of-plane loading was mainly resisted by flexural action in the elastic range and by arching action after cracking of the panel. Similarly, Angel et al. tested the out-of-plane seismic strength of masonry infilled panels, but considered panels that were previously cracked by in-plane lateral forces [30].

In general, the strength and stiffness of an infilled frame was found to be much higher than that of a bare frame. Most researchers also agree that infills can dissipate a large amount of energy when they are subjected to seismic reversals. Because of these qualities, infill panels can be an effective solution for the seismic retrofit of non-ductile frames.
2.2.3 Retrofit of Non-Ductile Frames Using Infill Panels.

Because of the beneficial qualities of infill panels, many researchers have used them as a retrofit alternative for non-ductile frames. A number of studies have investigated the use of CIP or PC concrete infills with promising results. Kahn tested five half-scale RC frames infilled with CIP concrete and PC concrete panels [6]. Of the proposed schemes, Kahn concluded that multiple vertical PC panels exhibited the most promise for seismic rehabilitation. This system increased the stiffness and lateral load carrying capacity of the frame while providing greater ductility and cyclic load capacity than any other configuration. Bertero and Brokken investigated lightweight concrete panels within a series of experiments on 3-1/2 story, 1-1/2 bay structures [7]. Their results, however, indicated that the best retrofit solution in terms of strength, stiffness and energy dissipation capacity was the use of solid brick panels with exterior welded wire fabric anchored to the frame. In addition, multiple PC concrete panels were used by Frosch et al. to retrofit a non-ductile RC frame [8]. Two rows of PC panels were connected by reinforced closure strips and attached to the existing frame with steel pipes (shear lugs). Although the final wall was composed of multiple units, it behaved as if it were monolithic.

If large openings are required, infilling the entire bay may not be a suitable retrofit option. For such cases the use of wing walls can be a more attractive solution. Testing programs focusing on the use of wing walls have not received the same attention as fully filled frames, but some experimental results do exist. Higashi and Kokusho investigated single-bay, single-story RC frames retrofitted with CIP and PC wing walls [31]. Frames strengthened with CIP wing walls exhibited strengths and rigidities similar to those of
monolithic columns and wing walls. The PC wing walls did not significantly increase strength and stiffness, but they did limit any strength reduction due to cyclic loading and increased the deformation capacity. A continuation of this study conducted by Higashi et al. considered similar retrofit techniques installed in single-bay, three-story frames [32]. Work done by Roach at the University of Texas at Austin used RC piers to strengthen a non-ductile RC frame [33]. The addition of the piers increased the seismic capacity of the columns and changed the failure mode of the frame from column shear to flexural hinging of the beams.

Although infill panels can greatly enhance a structure’s seismic performance, if not designed properly they may have a detrimental impact on the existing surrounding frame. The addition of infill panels increases a building’s mass, and as a result the inertial forces imparted to the structure are also increased. If the extra dead load is substantial, problems may arise with the existing capacity of the foundation. Furthermore, infill panels will stiffen a structure and ultimately alter the load path and failure mechanisms of the infilled frame. Because of large stirrup spacing and lap splices designed for compression, columns in “weak” (non-ductile) frames are not designed to resist high concentrated shear forces or large overturning moments. These deficiencies, which may cause column failure and catastrophic collapse, can, and have been, overcome in a variety of manners.

Kahn addressed the former problem by connecting the panel to the surrounding frame only at the top and bottom, leaving a gap between panel and column [6]. The separation between panel and column improved the structure’s overall ductility. Contact between column and infill was eliminated, thereby preventing the occurrence of a column shear failure. Increasing the tensile capacity of the existing columns, on the other hand,
controlled the detrimental effects of introducing a large overturning moment. Frosch et al. accomplished this by providing a post-tensioning anchorage system adjacent to the interior face of each column [8]. The extra tensile capacity of the proposed scheme allowed the retrofitted frame to behave in an acceptable manner. When designing an infill panel system for retrofit it is, therefore, imperative to consider all of the possible adverse and existing frame deficiencies.

2.2.4 Connection of Infill to Existing Frame

The boundary conditions between infill and bounding frame are an extremely important factor in the overall behavior of the retrofitted system. Some infill materials such as non-structural masonry may have no positive connection. The eventual interaction between infill and frame in this case is governed in large part by how tightly the panel is anchored. On the other hand, structural elements such as PC panels may require mechanical connections. Connections of this type can significantly affect the behavior of an infilled frame by modifying the load path of the seismic shear forces. For this reason, the location and structural adequacy of connections are essential variables in the design of retrofit applications. Economics, practicality and material composition are also equally important considerations.

By attaching vertical PC panels only to the top and bottom of the bounding frame (Fig. 2.3), Kahn forced the behavior of the infill to parallel that of a deep fixed-ended beam loaded in shear through the connections [6]. To eliminate the transfer of large concentrated shear forces into the column, a gap between the infill and column was provided. A welded connection was selected to eliminate the accuracy needed in aligning
bolted connections. Although requiring more precision in the field, a number of bolted connections [32, 34] have also been developed (Fig. 2.3). Two other promising schemes, one using external welded wire mesh to reinforce brick infills [7] and the other providing shear lugs to transfer shear between multiple PC panels [8], are also shown in Fig. 2.3.

2.2.5 Analytical Models for Infilled Frames

The in-plane stiffness and strength of infilled frames has been evaluated using a number of different analytical techniques: shear beam models, equivalent strut procedures and finite element methods. The shear beam model is a simple approach based on elementary strength of materials. In this method, the total lateral deflection of an infilled frame is calculated as the sum of the lateral deflection due to shear and that due to flexure (Eqs. (2.1) & (2.2)). The stiffness, $K$, can then be predicted by the expression in Eq. (2.3). Since this relationship predicts the elastic behavior of an infilled frame, it is only valid until the onset of cracking in the panel.

\[ \delta_{total} = \delta_{shear} + \delta_{flexure} \]  
\[ \delta_{total} = \frac{PH}{A'G} + \frac{PH^3}{12EI} \]  
\[ K = \frac{P}{\delta_{total}} = \frac{1}{\left( \frac{H}{A'G} + \frac{H^3}{12EI} \right)} \]  

where: $P =$ applied horizontal load  
$H =$ height of specimen
\[ A' = \text{shear area of structure} \]
\[ I = \text{moment of inertia} \]
\[ E = \text{elastic modulus} \]
\[ G = \text{shear modulus of elasticity} \]

Benjamin and Williams used this method to predict the pre-cracking behavior of a series of test specimen with different frame configurations and infill materials [20]. To determine the load at which cracking initiated, \( R_{cr} \), they recommended using the average shear stress in the panel as an index (\( R_{cr} = 0.1 f'_c A' \)). Furthermore, an empirical relationship was developed in order to calculate the ultimate load and deflection. By incorporating these results, theoretical load-deflection diagrams were approximated by three straight-line sections representing the uncracked, cracked and post ultimate regions.

The most common procedure for analytically expressing the behavior of infilled frames is the equivalent diagonal strut method. This method replaces the infill panel by diagonal struts spanning directly between opposite corners of the frame. The struts have the same thickness as the original panel, but an effective width, \( w \), less than that of the full panel (Fig. 2.4). Both Holmes and Stafford Smith performed early studies in this area while working simultaneously along parallel lines. Holmes proposed an effective width, \( w \), which was one-third of the length of the diagonal, \( r_{inf} \)[22]. He calculated the ultimate lateral load (that which produces failure in the infill) by considering the forces in the frame and infill separately and equating the shortening of the frame diagonal to the deformation of the equivalent strut. Similarly, Stafford Smith also transformed the infill panel into equivalent diagonal struts. Unlike Holmes, however, a beam on elastic foundation analogy was used along with experimental results to develop an analytical
expression for the effective width of the equivalent strut (Eqs. (2.4) & (2.5)) [24]. The relative stiffness of the frame to infill, the stress-strain relationship of the materials and the load level were all critical factors in determining this value. It can be seen from Eqs. (2.4) & (2.5) that the stiffer the frame is compared to the infill, the greater the contact length, \( \alpha \). The overall stiffness of the system was determined by considering the strain energies resulting from axial force in the tension column, compression of the diagonal strut and flexure of the frame [23]. Since these initial investigations, various other researchers have refined the equivalent strut approach.

\[
\alpha = \frac{\pi}{2\lambda} \quad \text{Eq. (2.4)}
\]

\[
\lambda = \sqrt[4]{\frac{E_{\text{inf}} t_{\text{inf}} \sin 2\theta}{4E_c I_{\text{col}} h_{\text{inf}}}} \quad \text{Eq. (2.5)}
\]

where: \( E_{\text{inf}} \) = elastic modulus of infill
\( t_{\text{inf}} \) = thickness of infill
\( h_{\text{inf}} \) = height of infill
\( E_c \) = elastic modulus of frame
\( I_{\text{col}} \) = moment of inertia of column
\( \theta \) = slope of diagonal

Based on an extensive series of tests on model frames and a few full-scale specimens, Mainstone developed empirical relationships for the strength and stiffness of infilled frames [35]. A framework was suggested, in terms of equivalent diagonal struts, for estimating the cracking strength, ultimate strength and initial stiffness. These three values were each defined by a dimensionless parameter, \( w'_{ec}/r_{\text{inf}} \), \( w'_{ec}/r_{\text{inf}} \) and \( w'_{ek}/r_{\text{inf}} \),
respectively (where \( w'_{ct} \), \( w'_{ce} \) and \( w'_{ek} \) denoted the widths of uniformly stressed struts of length \( r_{inf} \) which would have the same stiffness or strength as the infills). Similar to Smith, the measure of relative stiffness between infill and frame was determined with the dimensionless parameter, \( \lambda \) (Eq. (2.5)).

A number of researchers have developed analytical hysteresis models to describe the complete load-displacement history of cyclically loaded infilled frames. Mosalam et al. defined a generic hysteresis loop for an infilled frame based on five experimentally determined physical parameters [36]. Madan et al. integrated the equivalent strut approach with a smooth hysteretic model for masonry infilled frames [37]. This model considered both strength and stiffness degradation and pinching. Although more computationally intensive, detailed finite element models have been proposed for masonry infilled frames. Mehrabi and Shing [38] developed a smeared crack finite element model that included the interface behavior of mortar joints. Analysis showed that these models could capture failure mechanisms and maximum lateral resistance. Furthermore, Mosalam [39] was able to predict complicated crack patterns of frames with walls that had openings.

2.3 Performance-Based Seismic Retrofit Philosophy

The most widely used code in highly seismic regions of the United States is the Uniform Building Code (UBC) [40], which is based in large part on the Structural Engineers Association of California (SEAOC) blue book [41]. The UBC design methodology is based on a single performance design objective, life safety. Serviceability limit states are only indirectly satisfied through drift limitations. Since there are no
requirements to determine the amount or type of damage that will occur due to a given earthquake, the actual performance of a structure is unknown. In addition, present day building codes are written for new construction and are not capable of handling the unique challenges of retrofit work.

An increasing number of papers have been written emphasizing the need for the development and implementation of performance-based seismic design [42, 43]. It is now widely acknowledged that seismic design is not a one-step process with simple rules universal to all structures. Furthermore, traditional code-prescribed static lateral force procedures do not adequately model the true loading or inelastic demands imposed on a structure. Because of these shortcomings, the current philosophy is to base seismic design and retrofit on performance objectives established by design professionals interacting closely with building owners. As illustrated in SEAOC's performance objective matrix (Fig. 2.5), a seismic performance objective couples a structure's earthquake performance level with its earthquake design level, or possible hazard. Recent publications that outline Performance-Based procedures for seismic retrofit of RC buildings include ATC-40, *Seismic Evaluation and Retrofit of Concrete Buildings* [44] and FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* [9].
CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Prototype Structure

A prototype structure, representative of existing non-ductile flat-plate buildings constructed without seismic provisions, was designed and detailed for gravity loading based on ACI 318-63 [14]. The full-scale structure was assumed to be an office building with a column-to-column spacing of 20' in both directions and a story-height of 10'. Given the occupancy of the structure, a superimposed live load of 50 psf was required. In addition to the dead weight of the floor slab, an extra dead load allowance of 20 psf was included for movable partitions and 10 psf was added for ceiling and mechanical equipment. The total uniformly distributed dead and live load was 192.5 psf (assuming a 9" thick floor slab). The empirical method of ACI 318-63 [14] was subsequently used to design the floor slab. In addition, square columns, 20" x 20", were chosen because they were representative of a typical size for this type of construction.

The standard design bay was located at the interior of the building. The design compressive strength for the concrete, $f'_{c}$, was 3000 psi and the yield strength of the slab reinforcement, $f_{y}$, was 40,000 psi (40 Grade). The thickness of the floor slab was chosen based on criteria for both two-way shear and wide-beam shear. Only gravity loads were considered in calculating the maximum design moments at the critical sections of the slab. At the supports and at the center of the span for both the column strip and middle
strip the required ratio of reinforcement area to gross concrete area was determined. These values would later be used to determine the amount of reinforcement needed in the reduced-scale model structure. All of the bar cut-off points and detailing requirements of ACI 318-63 [14] were followed.

The columns of the prototype structure, on the other hand, were not designed for any specific loading. Instead, the longitudinal reinforcement was proportioned based on a reasonable ratio of reinforcement area to gross concrete area. The value of this ratio was then checked for compliance with the minimum and maximum values of 0.01 and 0.08 respectively. The spacing of column ties and sizing of splice lengths was all based on ACI 318-63 [14].

3.2 Model Structure

Because of size constraints imposed by the available testing equipment at Rice University, a full-scale structural subassembly could not be tested. For this reason, the prototype system was scaled down to the maximum size the reaction frame could adequately accommodate. After finalizing the configuration of the slab-column subassembly, a four-tenth-scale model was ultimately selected. Dimensions of the model were geometrically similar to those of the prototype. A length scale factor of 0.4 was applied in all directions to determine the model’s final proportions. Application of this modeling rule resulted in a structure that had 8' column-to-column spacing in each direction and a 4' story height. The reduced-scale columns were 8'' x 8'' and the floor slabs were 3.6'' thick (the actual thickness of the slab was rounded to 3.5'').
The slab reinforcement was initially designed using the dimensions and loading of the prototype structure. The static moment, or numerical sum of the positive and negative bending moments in a rectangular panel, was calculated as \( M_o = 0.09WLF\left(1 - \frac{2c}{3L}\right)^2 \) where \( W \) = the total dead and live load on the panel, \( L \) = span length from center to center of supports, \( c \) = effective support size, and \( F = 1.15 - \frac{c}{L} \) but not less than 1. This total moment was then distributed throughout the interior panel according to the percentages shown in Table 3.1.

### Table 3.1

**Moments in Interior Panels (ACI 318-63)**

<table>
<thead>
<tr>
<th>Strip</th>
<th>Moments in Slabs Without Drop Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support</td>
</tr>
<tr>
<td>Column Strip</td>
<td>-0.46(M_o)</td>
</tr>
<tr>
<td>Middle Strip</td>
<td>-0.16(M_o)</td>
</tr>
</tbody>
</table>

The amount of flexural reinforcement at each critical section of the model structure was determined by providing the same reinforcement ratio, \( \rho_g \) (ratio of area of steel to gross concrete area), as the prototype structure. The basis for designing the model structure in this manner stems from the fact that the reinforcement ratio of the prototype and model must maintain structural similitude. In addition to the amount of reinforcement required to resist the applied moment demand, the ACI code specifies a minimum quantity of steel for temperature and shrinkage (\( \rho_g = 0.0020 \)) and also a maximum bar spacing (max spacing = 2\(t\), where \( t \) = thickness of slab). Originally, it was intended to
provide #2 deformed bars (40 Grade) as the reinforcement throughout the floor slabs. Due to a lack of availability of #2 deformed bars, however, the initial layout of reinforcement had to be altered.

The final design of the slab reinforcement consisted of a combination of #3 deformed bars (40 Grade) and #2 smooth bars. At each critical section a sufficient number of #3 bars were provided in order to maintain the proper $\rho_s$ value. In the case that this amount of reinforcement violated the maximum spacing requirement of 7", additional #2 smooth bars were added to the section. #2 smooth bars were supplied instead of additional #3 bars so that the flexural capacity of the slab was not significantly increased while the spacing limitations were satisfied. All of the reinforcement consisted of straight bars detailed according to ACI 318-63 [14]. The reinforcing detail at the interior slab-column connections did not require the slab bottom bars to be continuous through the column. Instead, fifty percent of the positive slab reinforcement was terminated 1 1/4" (0.4 x 3") from the column centerline, and the remaining bars stopped at 0.125$L$ ($L = \text{span length}$) from support centerline in the column strip and at 0.15$L$ in the middle strip (Table 3.2).

The longitudinal column reinforcement for the model structure was initially proportioned based on selecting a reasonable value of $\rho_s$. The size and placement of this reinforcement, however, was modified a number of times due to concerns over detailing issues in the reduced-scale structure. The final design consisted of four #6 bars (60 Grade), one located in each corner of the square column. The reinforcement ratio, $\rho_s$, for the column was 0.028, a value that is between the minimum and maximum values of 0.01 and 0.08 prescribed by the building code. Since the model slab-column subassemblies were built to mirror the construction of an actual building, column lap splices were
required. A splice length of 18" (24d_b = 24 x 0.75") was specified which conformed to code standards for a compressive critical design stress. The slope of the inclined portion of the longitudinal bars with the axis of the column was approximately 1 in 7, less than the maximum of 1 in 6 given in the code. In addition, the tie reinforcement was composed of #2 smooth bars with a spacing of 8". The tie spacing was determined by choosing the smallest of 16d_b (16 x 0.75" = 12"), 48d_d (48 x 0.25" = 12"), or the least dimension of the column (8") [14].

### Table 3.2

**Slab Reinforcement Cutoff Points (Model)**

<table>
<thead>
<tr>
<th>Amount</th>
<th>Bottom Reinforcement</th>
<th>Top Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>50%</td>
<td>3&quot; (0.4) = 1 1/4&quot;</td>
<td>2'-5&quot;</td>
</tr>
<tr>
<td>Column Strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Remainder</td>
<td>12&quot;</td>
<td>2'-0&quot;</td>
</tr>
<tr>
<td></td>
<td>COL. Š</td>
<td></td>
</tr>
<tr>
<td></td>
<td>COL. Š</td>
<td></td>
</tr>
<tr>
<td>Middle Strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50%</td>
<td>3&quot; (0.4) = 1 1/4&quot;</td>
<td>100%</td>
</tr>
<tr>
<td>Remainder</td>
<td>14&quot;</td>
<td>2'-0&quot;</td>
</tr>
<tr>
<td></td>
<td>COL. Š</td>
<td></td>
</tr>
<tr>
<td></td>
<td>COL. Š</td>
<td></td>
</tr>
</tbody>
</table>
3.3 Flat-Plate Subassembly

The slab-column subassembly used in the experimental program incorporated the design and detailing requirements of the model structure. As illustrated in Fig. 3.1(a), the subassembly was representative of a portion of the frame located above the first floor and in the interior of the building. Inspection of the deformed shape of a flat-plate frame under lateral loading (Fig. 3.1(b)) aided in selecting boundary conditions for the substructure. It was assumed that points of inflection (i.e. points of zero moment) would typically occur at the midpoints of the beams and columns. These points were used as a basis for choosing the extreme limits of the two slabs and the top and bottom column stubs.

The overall dimensions of the flat-plate subassembly are shown in Fig. (3.2). The 8' column spacing, 4' story-height, 8" by 8" columns and 3 1/2" slab thickness were a direct result of geometric modeling requirements. The columns were extended past the slab at both the top and bottom of the specimen. Each of these column stubs was 1'-4" long and attached to a cleavage assembly that acted as a pinned connection (i.e. no moment was transferred). The center of the supporting steel pin was located between 6" and 7" from the end of the column (1'-10" to 1'-11" from the face of the slab). This placed the pin approximately at the location of the center of the top and bottom columns, had they been continuous.

To fulfill the objective of terminating the members at points of inflection, the slabs should have protruded 4' from the center of the columns in each direction. This approach, however, would have resulted in a specimen too large for the existing reaction frame. The final dimensions of the slab were, therefore, predominately governed by the constraints of
the testing equipment. Within these constraints, it was decided to select slab dimensions
that provided continuity and best simplified the design. As a result, the slabs extended out
from the center of the column 2' in every direction. The total length of the slab was 12'
and consisted of exactly two column strips and a middle strip. In the transverse direction,
the overall width was 4', equal to the width of a column strip. To simplify the test
configuration, the boundary condition at all of the slab edges was free. It has been shown
by Aalami that moment-rotation stiffness exhibited by a floor plate depends largely on
the column size and is not significantly affected by the span of the floor plate or its outer
boundary conditions [45]. Therefore, the aforementioned dimensions and boundary
conditions of the subassembly slab were deemed satisfactory for the proposed testing
program.

The layout of the subassembly slab and column reinforcement is sketched in Figs. 3.3
through 3.5. Important details for these elevation views are provided in Fig. 3.6. Clear
covers of 1/2" and 5/8" were provided in the slabs and columns, respectively. Concrete
with a maximum size aggregate of 3/8" was used in order to insure proper compaction
and reduce the likelihood of honeycombing within the thin, congested areas. In the
direction of loading, the longitudinal slab reinforcement was bent as pictured in Fig. 3.6.
Hooking the reinforcement at these ends ensured proper development of any bars that
were terminated prematurely due to the reduced length of the slab. The transverse
reinforcement, on the other hand, ended without any type of bend. Since there was no
load applied in the out-of-plane direction, this detail was deemed unnecessary and helped
eliminate some fabrication work.
After completing the general design and detailing of the main columns and slabs, special attention was given to the column stubs. The end of each stub served as a connection point between the specimen and the reaction frame. Attempts were made to ensure elastic behavior in the stubs throughout the loading history. An elastic analysis in the structural analysis program RISA-3D [46] approximated the maximum possible load imparted to the retrofitted subassembly. To simplify the computations, the infill panel was represented as a diagonal compression strut. Under this assumption, the compressive strength of the diagonal strut controlled the ultimate failure mechanism of the subassembly. The shears and moments generated from the application of the horizontal load required to produce a failure of the diagonal strut were ultimately used to design the column stubs and connections.

Since the infill panel material was not finalized at this time, the properties of precast autoclaved aerated concrete (PAAC) were assumed in the analysis. The representative material had a compressive strength, $f_{c}'$, of 725 psi, a modulus of elasticity, $E_c$, of $2.54 \times 10^5$ psi and a thickness of 3". Trigo and Leuchars and Scrivener's expression for the ultimate diagonal compressive strength, $R_c$, of masonry infilled frames (Eq. (3.1)) was used to predict the failure load [19].

$$R_c = \frac{2}{3} \alpha t_{inf} f_m \sec \theta$$

Eq. (3.1)

This expression is similar to the equation proposed by Stafford-Smith and Carter [47], but slightly modified to provide conservative agreement with test results of masonry infilled frames. The definition of the variable $\alpha$ is provided in Equation 2.4.
Based on the above analysis, the design forces for the column stubs were found to be, 20 kips in shear, 40 kips in tension and a 440 kip-in moment. To satisfy the shear requirement, the maximum tie spacing of 8" on center for the main columns was reduced to 2" within the column stubs. The resulting stirrup layout is shown in Fig. 3.5. Similarly, the moment capacity of the column stub was less than the estimated maximum possible demand. To address this concern, it was initially proposed to add extra longitudinal stub reinforcement that would be terminated within the main column. Although this solution increased the moment capacity, it generated a number of other concerns ultimately considered to be more detrimental to the structure. The major concern was increasing the congestion in areas that were already tightly spaced. The prospect of voids or honeycombing within the column stub and splice region of the main column ultimately outweighed the desire to increase the moment capacity. Another undesired result was increased moment capacity in the main column due to the embedment of extra stub bars through the slab-column joint. Because of the above concerns, it was decided to forego any changes in the original design. It should be noted that the threaded rods were not included in the calculation of the moment capacity of the section. Furthermore, the concrete strength of 3000 psi used in the computations was later found to be much higher in the actual subassemblies.

Four 2' long by 3/4" diameter A36 threaded-rods were cast into each column stub. The rods protruded 4 1/2" from the end of each column stub and were used to connect the concrete specimen to the steel reaction frame and loading beam. An embedment length of 1'-7 1/2" allowed penetration through the slab-column joint (Fig 3.5). As shown in Fig. 3.6, the threaded rods were located inside the stirrups and adjacent to the main
longitudinal #6 bars. Each column stub was capped with a 1/4" plate, included solely to provide a flat surface for the clevice assemblies to bear upon. The design capacities for a group of four threaded rods was 30 kips in shear and 57.6 kips in tension. These values were greater than the previously determined maximum demand of 20 kips in shear and 40 kips in tension calculated by assuming a diagonal compression strut failure.

3.4 Subassembly Fabrication

3.4.1 Subassembly Formwork

The forms for all four 0.4-scale specimens were constructed simultaneously on the loading dock of Ryon Laboratory. Each form was a freestanding wood structure consisting of a bottom slab, top slab and two columns. Both of the slabs were framed with intermittently spaced 2x4's supported on ten exterior 4x4 posts. 1/2" plywood was used for the bottom of the slab and 3/4" plywood for the side rails. Cross bracing was provided between 4x4 posts that laterally stabilized the wooden assembly. The interior column forms rested on a wooden pedestal. Prior to pouring concrete all of the forms were oiled with a bond breaker to make the stripping process easier. Photos 3.1 & 3.2 show the subassembly formwork.

3.4.2 Subassembly Reinforcement

All of the reinforcement for the slab-column subassemblies was delivered to Rice University in 20' long pieces. These sections were cut to the specified lengths with a horizontal band saw. Any bending of the steel (i.e. hooked ends of slab reinforcement, column ties and inclined longitudinal column bars) required to satisfy the specified
design was also performed at Ryon Laboratory. Individual bars were tied together to construct units that were easy to move and convenient for placement within the forms. Each slab consisted of two identical segments at the ends and two in the middle. The columns were made up of three parts: the bottom stub reinforcement, the main reinforcement and the top connection.

A 1/4" thick plate was attached to the bottom stub reinforcement in order to provide a level surface at the end of the column stub. The plate also served as a template for aligning both the longitudinal #6 rebar and the 3/4" threaded rods. Tapped holes were drilled into the 8" x 8" plate for the threaded rods and plain holes for the rebar. The threaded rods were located on the inside of the longitudinal bars and the column ties. Prior to placement of the column ties, the rebar was properly aligned and welded to the plate. The completed assemblage was then lowered from the top into the formwork. A pedestal with matching holes for the protruding connecting rods supported the entire configuration. The main column cage consisted of four straight #6 bars bound together with smooth #2 ties. The completed unit spanned from the top of the bottom slab to the top of the upper column stub. Finally, the third portion of the column hardware was the top connection. The top connection was identical to the bottom stub reinforcement without the longitudinal rebar and column ties. Pictures of the reinforcement in the forms are shown in Photos 3.3 through 3.6.

3.4.3 Casting of Specimens

It was originally intended to have the concrete batched at a local ready mix plant and delivered by truck to Rice University. In this scenario, the entire pour would have
required two days and mirrored the construction of an actual building. On the first day, the bottom slabs and column stubs for all four specimens would be poured. Once this concrete gained sufficient strength, the main column reinforcement and enclosing formwork could be set into place. The top slab, main columns and top column stubs would all be cast during the delivery of the second load of concrete. Unfortunately, this option presented a number of unforeseen problems.

The formwork configuration provided limited access to the bottom slab due to the closely spaced 4x4 posts and the exterior cross bracing. The chute of a mixer truck, therefore, could not be efficiently maneuvered over all of the bottom slabs in an acceptable amount of time. The top slabs, on the other hand, were unobstructed, but presented a different challenge. All of the forms for the second pour were elevated from six to eight feet above the loading dock floor. At this elevation a normal concrete truck would not be able to discharge concrete without supplementary pumping equipment. Furthermore, the final layout of the formwork on the loading dock of Ryon Laboratory made it nearly impossible to directly reach any portion of the outer two specimens. Even if some of these problems could have been rectified, it was difficult to find a ready mix plant that stocked 3/8" maximum size aggregate. Of the companies that did, none of them wanted to deliver such a small quantity of concrete, especially with knowledge of the difficult placement requirements. Because of these reasons, it was decided to batch the concrete on site.

The required sand and 3/8" maximum size aggregate was stored behind Ryon Laboratory. The concrete was mixed in a portable concrete mixer with a nine cubic foot capacity placed adjacent to the formwork. The mixed concrete was transported by
wheelbarrow to the forms and placed by hand with shovels. An internal poker vibrator was used to provide the proper consolidation. The entire pouring operation took two weeks with a crew of two to four men working at any given time. Fig. 3.7 illustrates the area of concrete covered during each pour. The pouring sequence for all four specimens is shown in Table 3.3.

**Table 3.3**

**Subassembly Pouring Sequence**

<table>
<thead>
<tr>
<th></th>
<th>Bottom Slab</th>
<th>Top Slab &amp; Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Poured</td>
<td>28-Days</td>
</tr>
<tr>
<td>Specimen #1</td>
<td>10/15/98</td>
<td>11/12/98</td>
</tr>
<tr>
<td>Specimen #4</td>
<td>10/19/98</td>
<td>11/16/98</td>
</tr>
</tbody>
</table>

The bottom slab and stub columns for each specimen were poured first (Photos 3.7 & 3.8). Limited by the size of the mixer and the number of workers, only one slab could be finished per day. Each bottom slab pour required three identical batches of concrete in a nine cubic foot portable mixer. As fresh concrete was being placed new concrete was continuously being mixed. The slabs were leveled off with a 2x4 and later floated with a metal hand float (Photo 3.9). The casting of nine 6" x 12" cylinders and two 6" x 6" x 20" beams also accompanied every pour.

After the completion of the bottom slabs, the hardened concrete at the base of the column was roughened and the main column reinforcement and forms were put into place. Because of the elevated height of the top slab, scaffolding was used to provide a
working platform. The concrete was dumped into a wheelbarrow and lifted into place with a twenty-ton overhead crane. The wheelbarrow was suspended directly above the forms and positioned to allow easy access for manual placement of the concrete (Photos 3.10 & 3.11). Each top slab and main column pour required four identical batches of concrete in the portable mixer. Before the top column stubs were cast, the top slab surrounding the column was allowed to slightly harden. This kept the amount of concrete pushing out from under the column stub formwork to a minimum. After fresh concrete filled the top stub form, the top threaded rod connection was inserted into place with the aid of external vibration (Photo 3.12). The finishing of the slabs and the amount of cylinders and beams was identical to that of the bottom slabs.

Soon after casting, all of the concrete slabs were covered by polyethylene sheeting with a thickness of 6-mil (Photo 3.2). The cylinders and beams were covered with the same type of sheeting and stored under the specimens. No additional curing procedures were performed. Most of the formwork was stripped and the polyethylene sheeting was removed during the week of 1/18/99. The majority of the wooden support system (4x4 posts and 2x4’s) and the plywood for the bottom slab of each specimen, however, were left in place until the frames were transported into the testing frame. The main reason for leaving the forms attached until this time was to provide lateral stability for the freestanding flat-plate structures.

3.4.4 Concrete Mix Design

Since the concrete was batched on site, the mix was designed from locally available materials. The required fine and coarse aggregate was delivered from San Jacinto Stone
Company of Houston, Texas and stockpiled behind Ryon Laboratory. Trial mixtures were made in an attempt to get a very high slump concrete with compressive strengths in the range of 3000 psi. Because the concrete was placed manually and the reduced-scale model was fairly congested, fluid concrete with a 6" to 8" slump was targeted. The high slump greatly minimized the amount of work required to achieve proper placement and consolidation.

The maximum coarse aggregate size of 3/8" was selected for two reasons. First, a full-scale structure of this type would have a typical maximum coarse aggregate size of 1". Therefore, in order to represent a dimensionally similar material the 1" coarse aggregate size was scaled by four-tenths. Another reason for scaling the coarse aggregate was to ensure proper encasement of reinforcement and to limit honeycombing. The ACI 318-95 Building Code enforces this condition by limiting the nominal maximum size of coarse aggregate to 1/5 the narrowest dimension between sides of forms, 1/3 the depth of slabs and 3/4 the minimum clear space between bars or steel and formwork [48].

The ACI method of mix design provided the basis for proportioning the constituent materials. Type I portland cement was used with a target water/cement (w/c) ratio of 0.6. At the time of delivery, both the coarse and fine aggregate contained a considerable amount of moisture. Samples of each material were weighed and then left to oven dry overnight. The total moisture content in the aggregate was then calculated as

\[
\frac{W_{STOCK} - W_{OD}}{W_{OD}} \times 100
\]

where \(W_{STOCK}\) = the weight of the material in its field condition and \(W_{OD}\) = the oven dried weight. An attempt was made to correct for the amount of moisture in the aggregate during trial mixes. The final mix proportions used for batching all of the
concrete in the slab-column subassemblies are shown in Table 3.4. The amount of water actually added may have varied slightly from mix to mix. Enough water was added to maintain a uniform mix in terms of consistency and slump.

Table 3.4

**Subassembly Concrete Mix Design**

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/c ratio</td>
<td>0.6</td>
</tr>
<tr>
<td>Coarse Aggregate (3/8&quot;)</td>
<td>1183 lbs.</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1699 lbs.</td>
</tr>
<tr>
<td>Portland Cement (Type I)</td>
<td>640 lbs.</td>
</tr>
<tr>
<td>Additional Mixing Water</td>
<td>316 lbs.</td>
</tr>
<tr>
<td>Yield</td>
<td>1 yd³</td>
</tr>
</tbody>
</table>

3.4.5 Material Properties of Reinforcement

Three sizes of reinforcing bars were used in the construction of the flat-plate subassemblies. #2 smooth and #3 deformed bars were used as slab reinforcement, #6 deformed bars as the longitudinal column reinforcement and #2 smooth bars as the column ties. All of the #3 bars and the majority of the #2 bars (batch #1) were delivered at the same time from Triple-S Steel in Houston, Texas. Additional #2 bars (batch #2) had to be delivered on a separate occasion because of changes to the original design that increased the amount needed. Remaining from previous research, a sufficient supply of #6 rebar was already available at Rice University. Three samples were taken from each batch of steel for material testing. The complete stress-strain behavior for each sample was determined and an average value for the yield stress, \( f_y \), ultimate strength, \( f_u \), modulus of elasticity, \( E_s \), and yield strain, \( \varepsilon_y \), were calculated. If the yield point was not well
defined, as was the case for the #2 bars, the yield strength was determined by taking the value for stress at a strain, $\varepsilon_Y$, of 0.005. A typical stress vs. strain curve for each size bar is shown in Fig. 3.8 and average values for $f_Y, f_u, E_s$ and $\varepsilon_Y$ are listed in Table 3.5.

### Table 3.5

**Properties of Subassembly Reinforcement**

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>No. Tested</th>
<th>$A_s$ (in$^2$)</th>
<th>Mean $f_Y$ (ksi)</th>
<th>Mean $f_u$ (ksi)</th>
<th>Mean $E_s$ (ksi)</th>
<th>Mean $\varepsilon_Y$ (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#2 (Batch #1)</td>
<td>3</td>
<td>0.049</td>
<td>93.2</td>
<td>99.6</td>
<td>27,000</td>
<td>3452</td>
</tr>
<tr>
<td>#2 (Batch #2)</td>
<td>3</td>
<td>0.049</td>
<td>92.7</td>
<td>97.8</td>
<td>25,700</td>
<td>3607</td>
</tr>
<tr>
<td>#3</td>
<td>3</td>
<td>0.110</td>
<td>71.4</td>
<td>86.6</td>
<td>30,100</td>
<td>2372</td>
</tr>
<tr>
<td>#6</td>
<td>3</td>
<td>0.300*</td>
<td>65.2</td>
<td>108.4</td>
<td>30,000</td>
<td>2173</td>
</tr>
</tbody>
</table>

* Reduced diameter to fit in grips of testing apparatus (mean value).

Each specimen had an overall length of 8” and a gage length (between grips) of 4". An extensometer with a gage length of 1" was attached to the specimen and removed after yielding. This transducer recorded the linear-elastic portion of the stress vs. strain curve along with the initial region of yielding. The extensometer was removed at this time so that it would not be damaged under large post yielding strains. Measurements for the #2 (smooth) and #3 bars were conducted on undisturbed samples taken directly from the stockpiled material. The #6 rebar, on the other hand, was too large for the available grips of the tensile testing machine. As a result, the samples were machined down from a diameter of 0.75” to an average value of 0.618”. The deformations on the rebar were completely removed and the final specimens were smooth. All tests were performed on an Instron material testing machine with a capacity of 60 kips.
Tests of the #3 reinforcement yielded \( f_y \) values much higher than anticipated. At first, it was believed that 60 Grade-reinforcement might have been shipped instead of the requested 40 Grade. The ASTM tensile requirements for deformed steel bars, however, verified that the reinforcement was indeed 40 Grade, but with an above average yield strength. Since there is no specification for maximum yield strength, any value above 40 ksi is acceptable. In addition, the ultimate tensile strength of the #3 rebar was 86.6 ksi, greater than the 70 ksi required for 40 Grade steel, but less than the minimum strength of 90 ksi for 60 Grade. Elongation in an 8" section of the suspect material was 11.5%. At the time of testing, a major portion of the reinforcement had already been fabricated based on the original design. Because of this and with the knowledge that the steel was actually 40 Grade, it was decided to use the delivered #3 reinforcement without making any modifications.

### 3.4.6 Material Properties of Concrete

Nine 6" diameter x 12" long cylinders and two 6" x 6" x 20" long beams were cast with every pour. After removal from the forms, the strength specimens were cured alongside the subassemblies under identical conditions. Three of the cylinders from each pour were used to measure compressive strength, \( f'_c \), at 28-days. The remaining cylinders and beams stayed with the subassembly and were later used to determine the compressive strength (ASTM C39-96), splitting tensile strength (ASTM C496-96) and modulus of rupture (ASTM C78-94) of the slab-column models at the time of testing. All of the compression tests were performed in a 220-kip hydraulic MTS material-testing system. The splitting tensile tests and the modulus of rupture tests, on the other hand, were
configured in a 300-kip Tinius Olsen material-testing machine. The results of these tests are tabulated in Table 3.6.

**Table 3.6**

**Mean Properties of Subassembly Concrete**

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Pour</th>
<th>( f_c' ) @ 28 Days (psi)</th>
<th>Age @ Test (days)</th>
<th>( f_c' ) @ Test (psi)</th>
<th>Split Cylinder @ Test</th>
<th>Modulus of Rupture @ Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>BS</td>
<td>5030</td>
<td>346</td>
<td>6210</td>
<td>645</td>
<td>8.18</td>
</tr>
<tr>
<td>S1</td>
<td>TS</td>
<td>4350</td>
<td>336</td>
<td>5200</td>
<td>500</td>
<td>6.93</td>
</tr>
<tr>
<td>S2</td>
<td>BS</td>
<td>5400</td>
<td>475</td>
<td>5500</td>
<td>550</td>
<td>7.42</td>
</tr>
<tr>
<td>S2</td>
<td>TS</td>
<td>4790</td>
<td>465</td>
<td>5260</td>
<td>460</td>
<td>6.34</td>
</tr>
<tr>
<td>S3</td>
<td>BS</td>
<td>4700</td>
<td>524</td>
<td>5660</td>
<td>560</td>
<td>7.44</td>
</tr>
<tr>
<td>S3</td>
<td>TS</td>
<td>5080</td>
<td>516</td>
<td>5990</td>
<td>605</td>
<td>7.82</td>
</tr>
<tr>
<td>S4</td>
<td>BS</td>
<td>5040</td>
<td>563</td>
<td>5480</td>
<td>520</td>
<td>7.02</td>
</tr>
<tr>
<td>S4</td>
<td>TS</td>
<td>5060</td>
<td>553</td>
<td>5950</td>
<td>580</td>
<td>7.52</td>
</tr>
</tbody>
</table>

BS = Bottom Slab; TS = Top Slab & Columns

1 Average of three test specimens

2 Average of two test specimens

The interaction between surrounding frame and infill panel in systems without panel to frame connections is largely influenced by the stress-strain relationship of the constituent materials [35, 47]. For this reason, the modulus of elasticity, \( E_c \), was determined from the plots of stress vs. strain at 28-days (see Fig. 3.9 for a typical stress-strain curve). The elastic modulus was estimated as the chord modulus measured between 40% of the ultimate strength \( (0.4f_c') \) and a stress of 200 psi (the lower stress level was chosen to avoid error caused by a slight concavity in the beginning of the stress-strain curve). The average value for the chord modulus of all four subassemblies was
approximately $2 \times 10^6$ psi. This number was about half of the value calculated with the empirical formula provided in the ACI code [48]. One of the main reasons for this large difference may be attributed to the size and shape of the 3/8” maximum size aggregate.

3.5 Infill Panel

3.5.1 General

An ideal infill panel for flat plate buildings should possess the following characteristics:

- The infill material should be lightweight so that the mass of the structure and subsequent inertial forces are not significantly altered. Furthermore, a lightweight panel is easier for workers to handle and erect.

- Because the infill system is being added to a weak frame, the chosen system should not aggravate any existing problems (i.e. avoid transmitting excessive tensile or shear forces to the columns).

- Infill panels should increase the stiffness of the building and decrease the deformation demand.

- The infill panel must have an efficient mechanism for dissipating the seismic energy imparted to the structure.

- The infill should behave as a ductile fuse, failing prior to failure of the surrounding frame.

- To provide a more efficient construction process the infill should be prefabricated.

- The infill panel system must be cost-effective.
The selection of an appropriate infill panel system required the evaluation of several possible alternatives. Typically found as non-structural partitions, masonry or brick is one of the most common infill materials used throughout the world. Inclusion of masonry panels in structural systems has been shown to increase the stiffness, strength and ductility of bare frames. Any positive impact of masonry infills, however, is usually not considered in design and analysis because of the high degree of uncertainty related to infill behavior. Variability in mechanical properties and workmanship accounts for a low reliability in strength, stiffness and connectivity to the surrounding frame. In addition to the beneficial aspects of masonry infills, there are also a number of negative impacts that cannot be overlooked. Masonry panels can drastically alter the intended structural response, attracting forces to parts of the building that have not been designed to resist them. Frame-infill interaction results in a decrease in fundamental period, and an increase in seismic shear, frequently resulting in shear failure of the columns. Furthermore, injury or loss of life can occur due to out-of-plane fallout of unreinforced infills onto streets or into stairwells.

Another suitable alternative is reinforced concrete, both cast-in-place and precast. Cast-in-place concrete, however, may provide an inefficient retrofit solution because of the difficulties involved in placing large amounts of fresh concrete in an existing building. Precast panels eliminate this concern, but present other challenges such as panel-to-panel and panel-to-frame connections. In addition, normal weight concrete walls will typically add a substantial amount of mass to an existing structure. Instead of behaving as a ductile fuse, these materials will tend to provide a strong infill that may aggravate the deficiencies of the weak surrounding frame. Because of these reasons, a
lightweight prefabricated infill panel that can behave like a ductile fuse is an optimal choice for the flat-plate system. Two types of lightweight concrete were experimentally investigated prior to selecting the most promising material.

3.5.2 Precast Autoclaved Aerated Concrete (PAAC)

Precast autoclaved aerated concrete (PAAC) is a lightweight, prefabricated building material with a uniform cellular structure. PAAC is comprised of sand, lime, gypsum, water and an expanding agent that must be manufactured in a plant under a controlled environment. The volume of the final product expands up to five times the volume of the raw materials, and contains an air content of 70% to 80% depending on strength and density. For this reason, the material is lightweight (as little as one-fifth the weight of standard concrete) and has compressive strengths ranging from 300 to 1000 psi. Several diagonal tension tests were performed on 2' x 2' x 4" panels containing a minimum amount of welded wire reinforcement. These specimens were cut from a 10' long section of standard wall panel material with a hand held reciprocating saw. Load-deflection and the corresponding crack propagation were plotted (Fig. 3.10).

3.5.3 Lightweight Pumice Stone Concrete (LWPSC)

Pumice is a naturally occurring volcanic material with a fairly even texture of small inter-connected cells and a low bulk specific gravity. The high internal porosity is a result of the bloating (i.e. puffing up to a greatly expanded size) that occurs at elevated temperatures. Bloating can be attributed to a rapid build up of gas within the particle, which cannot readily escape. Because of the high internal porosity, the absorption
capacity is much greater than normal weight aggregates, demanding a modified approach to mix design.

Celdacrete, a precast concrete supplier in Guadalajara, Mexico provided both practical and technical assistance for working with lightweight concrete made with pumice. The concrete produced in Guadalajara is composed of pumice, cement, water and an engineered admixture (trade name Celdacrete) that acts as a water-reducer and air-entraining agent. This chemical also behaves as an expansion agent, ultimately decreasing the unit weight and the compressive strength of the final product. As explained later, this property of the admixture was found valuable for a number of reasons.

The pumice found in Guadalajara is taken directly from its natural state and used with little further processing. It is fairly well graded and from experience has provided a consistent and workable mix. This material can be batched, reinforced and handled like typical normal-weight concrete. Currently, it is being used as a complete precast structural system for low-rise construction in Guadalajara, Mexico. A series of preliminary tests were conducted at Rice University to determine basic material properties and evaluate the potential for using this material in a seismic retrofit application.

3.5.4 Preliminary Material Testing of LWPSC

3.5.4.1 Mix Design

A number of cylinders and square 2' x 2' x 4" panels were cast in Guadalajara, Mexico (MIM) and shipped to Rice University. These specimens were made with the same
concrete mix used to produce Celdacrete's typical precast units. The exact quantity of the constituent materials in this mix, unfortunately, was not available. Celdacrete did provide, however, a typical mix design based on a cubic meter of wall panel material (Table 3.7). In addition, the owner of the plant visited Rice University on 2/11/99 and was able to demonstrate proper proportioning and batching procedures.

Table 3.7
Typical Celdacrete Pumice Stone Mix Design

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Pumice</td>
<td>1.1 m³</td>
</tr>
<tr>
<td>Portland Cement (Type I)</td>
<td>250 kg</td>
</tr>
<tr>
<td>Admixture (Celdacrete)</td>
<td>2.5 kg</td>
</tr>
<tr>
<td>Additional Mixing Water</td>
<td>200-220 L</td>
</tr>
<tr>
<td><strong>Yield</strong></td>
<td><strong>1 m³</strong></td>
</tr>
</tbody>
</table>

On this date, three separate test batches were produced in the laboratory, each one consisting of a different aggregate. Two of the mixes contained pumice stone, the first using pumice from Guadalajara (PM) and the second with pumice from Pacific Custom Materials in California (PC). The Mexican pumice was used in its natural state and was moist to the touch. No special treatment was performed after mining it from the quarry. On the other hand, the pumice stone from California was cleaned, graded by particle size and in a very dry state. A third batch, consisting of landscape quality "lava rock" (LR) purchased in Houston, Texas was produced for comparison purposes. Each of the three mixes were intended to yield enough concrete for a slump test, two 6" x 12" cylinders and one 6" x 6" x 20" beam. No special curing measures were provided for any of the
mixes. Instead, all of the specimens were simply left to air dry prior to testing. Information about the PM and PC mix designs are given in Table 3.8.

**Table 3.8**

LWPSC Mix Designs Proportioned at Rice University

<table>
<thead>
<tr>
<th>Batch</th>
<th>Pumice (lbs)</th>
<th>Cement (lbs)</th>
<th>Additive (grams)</th>
<th>Water (lbs)</th>
<th>Slump (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PM</td>
<td>63</td>
<td>33</td>
<td>150</td>
<td>19.4</td>
<td>5.5</td>
</tr>
<tr>
<td>PC</td>
<td>60*</td>
<td>35</td>
<td>150</td>
<td>26.4</td>
<td>3.5</td>
</tr>
</tbody>
</table>

* 19 lbs. of 1/2", 19 lbs. of 3/16" and 22 lbs. of an 80/20 mix.

3.5.4.2 Mechanical Properties

The standard test method for compressive strength of cylindrical concrete specimens, ASTM Designation: C39-96, was used to determine the compressive strength of the cylinders cast in Guadalajara, Mexico and at Rice University in Houston, Texas. All of the tests were performed on a 220-kip MTS material-testing machine. A 6" diameter and a 12" length were the nominal dimensions of the cylindrical test specimens. Because of the expansive properties of the chemical admixture, however, all of the cylinders experienced an overall increase in length of 1/4" to 1/2".

The dry density for each batch of concrete was determined by weighing hardened compression test cylinders before they were tested. The actual length of the expanded cylinders was used in calculating the volume of each specimen. Two samples were weighed and averaged to arrive at an approximate unit weight value. A brief summary
comparing the compressive strength and unit weight of the four different mixes is provided in Table 3.9.

Table 3.9

<table>
<thead>
<tr>
<th>Casting Location</th>
<th>Pumice Source</th>
<th>Concrete Type</th>
<th>Age @ Test (days)</th>
<th>Mean Dry Unit Weight (lbs/ft³)</th>
<th>Mean $f_c'$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mexico</td>
<td>Mexico</td>
<td>MIM</td>
<td>42</td>
<td>75.6</td>
<td>1066</td>
</tr>
<tr>
<td>Rice Univ.</td>
<td>Mexico</td>
<td>PM</td>
<td>33</td>
<td>67.6</td>
<td>1060</td>
</tr>
<tr>
<td>Rice Univ.</td>
<td>California</td>
<td>PC</td>
<td>33</td>
<td>77.7</td>
<td>1330</td>
</tr>
<tr>
<td>Rice Univ.</td>
<td>N/A</td>
<td>LR</td>
<td>33</td>
<td>98.5</td>
<td>1061</td>
</tr>
</tbody>
</table>

The standard test method for the splitting tensile strength of cylindrical concrete specimens, ASTM Designation: C496-96, was used to determine the splitting tensile strength of three cylinders cast in Guadalajara, Mexico (MIM). All of the tests were performed on a 300-kip Tinius Olsen material-testing machine. Three specimens poured on 2/2/99 were tested on 3/18/99 at an age of 44 days. The cylinders had an average diameter, $D$, of 5.93" and an average length, $l$, of 12.25". An average splitting tensile strength, $f_s$, of 264 psi was recorded. The ratio of splitting tensile strength to the square root of the compressive strength was 8.08. All of the pumice was fractured during the test leaving a smooth failure surface.

The standard test method for flexural strength of concrete (using simple beam with third-point loading), ASTM Designation: C78-94 was used to determine the modulus of rupture of beams cast at Ryon Laboratory on 2/11/99. All of the tests were performed on
a 300-kip Tinius Olsen material-testing machine. Three different specimens were tested at an age of 35 days, one each of type PM, PC and LR. The molds for these beams had nominal dimensions of 6" x 6" x 20". Actual cross section measurements for all the specimens, however, differed from the nominal measurements because of the expansive properties of the chemical admixture. The height, \( h' \), was maintained at 6", but the width had an average value of 6.45". A span length, \( L_s \), of 18" was used in the final test configuration. The modulus of rupture, \( f_r \), found for the three specimens were as follows: 
\[ f_r(\text{PM}) = 120 \text{ psi}, f_r(\text{PC}) = 140 \text{ psi and } f_r(\text{LR}) = 315 \text{ psi}. \]

### 3.5.4.3 Diagonal Tension Tests

A series of diagonal tension tests were performed to study the behavior and ultimate strength capacity of LWPSC panels loaded by compressive forces acting along the diagonal. The test configuration was designed to simulate the loading conditions on an infill panel subjected to lateral forces. Under increasing horizontal load a separation between the frame and infill eventually develops if there are no mechanical connections between infill panel and surrounding frame. Fig. 3.11(a) illustrates this interaction between bounding frame and infill panel. The resulting distribution of stresses is complex and dependant on a number of variables [23, 24]. Before studying this interaction, however, the behavior of lightweight concrete panels without bounding frames was initially investigated. The diagonal forces generated within an infilled frame were simulated through the testing configuration illustrated in Fig. 3.11(b) and Photos 3.13 through 3.16.
A total of twelve 2' x 2' x 4" square panels were constructed in Guadalajara, Mexico (MIM). As shown in Fig. 3.12, three different designs were provided. The nominal dimensions of each design were the same, but the amount and layout of the reinforcement was varied. All of the reinforcement was high strength (Grade 6000) deformed bars with a diameter of 5/32". Sulfur capping compound was used on the majority of the specimens in order to provide a smooth bearing surface. An attempt was made to eliminate placement errors in the final testing configuration by pouring the capping compound directly into the steel angle support shoes. Unfortunately, problems did occur and proper panel alignment was difficult, and in some cases, impossible to achieve. Minor eccentricities in the load path probably accounted for some of the premature failures and excessive crushing observed at the loaded corners.

Initially, a 3" bearing length at each leg of the angle was provided at the loaded ends. High compressive stresses at the corners, however, led to crushing of the concrete prior to the development of any diagonal cracking in the panel. The remaining panels were, therefore, reconfigured to have a 5" (approximately 20 percent of the length of the panel side) seated length. The panels were subjected to a monotonic loading controlled by a constant deformation rate of 0.05 in/min. Overall, the behavior of the specimens was similar and is summarized in Table 3.10.

Of the panels that did not fail prematurely due to crushing of the corner, the load vs. deflection relationship was fairly typical (Fig. 3.13). Under increasing deformation, the load increased until the onset of a diagonal crack. Immediately following the development of a diagonal crack the load resistance temporarily declined. This period of reduction was brief, however, and the applied load quickly rebounded, eventually
exceeding the initial cracking force. The maximum load occurred shortly after the initial cracking stage and usually corresponded to a compression failure at one of the bearing corners. After the crushing of a corner, the load carrying capacity of the specimen was diminished and the test was stopped. Cracking in many of the test samples was also observed originating from the bearing angle on the edges of the panel. Eventually, these fractures propagated perpendicular to the originating side, becoming visible on the face of the specimen. Aside from these cracks and the split of the main diagonal, few other cracks were detected. Other visible deterioration included spalling of surface concrete in the areas bordering the bearing angles (Fig. 3.10 and Photos 3.13 through 3.16).

The provided reinforcement did not appear to have any effect on the diagonal cracking load or the ultimate capacity of the panel. To a slight degree, the reinforcement was able to control the widening of the main diagonal crack. Comparison of the final crack width between the type A and type B1 panels supported this assertion. At the completion of the tests, all the panels with reinforcement remained in one piece. The plain concrete design (type A), on the other hand, was prone to break into sections upon removal from the testing equipment.
Table 3.10

Results of Diagonal Tension Tests of LWPSC

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Bearing Length (in)</th>
<th>Age @ Test (days)</th>
<th>Load @ First Diag. Crack (kips)</th>
<th>Ultimate Load (kips)</th>
<th>Ultimate Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>3</td>
<td>61</td>
<td>12.7 (after peak)</td>
<td>22.5</td>
<td>Crushing @ Top Bearing</td>
</tr>
<tr>
<td>A-2</td>
<td>3</td>
<td>61</td>
<td>12.2 (after peak)</td>
<td>22.1</td>
<td>Crushing &amp; Splitting @ Bottom Bearing</td>
</tr>
<tr>
<td>A-3</td>
<td>5</td>
<td>65</td>
<td>27.4</td>
<td>27.4</td>
<td>Crushing @ Top Bearing</td>
</tr>
<tr>
<td>A-SC (Scored)</td>
<td>5</td>
<td>106</td>
<td>N/A</td>
<td>31.1</td>
<td>Crushing @ Top Bearing</td>
</tr>
<tr>
<td>B1-1</td>
<td>3</td>
<td>61</td>
<td>N/A</td>
<td>15.2</td>
<td>Test Stopped Due to Crushing @ Top</td>
</tr>
<tr>
<td>B1-2</td>
<td>5</td>
<td>66</td>
<td>28.7</td>
<td>31.4</td>
<td>Crushing @ Top Bearing</td>
</tr>
<tr>
<td>B1-3</td>
<td>5</td>
<td>67</td>
<td>25.9</td>
<td>32.0</td>
<td>Crushing @ Top Bearing</td>
</tr>
<tr>
<td>B2-1</td>
<td>3</td>
<td>61</td>
<td>N/A</td>
<td>15.2</td>
<td>Test Stopped Due to Crushing @ Top</td>
</tr>
<tr>
<td>B2-2</td>
<td>5</td>
<td>66</td>
<td>7.4 (after peak)</td>
<td>15.9</td>
<td>Early Crushing Due to Uneven Bearing</td>
</tr>
<tr>
<td>B2-3</td>
<td>5</td>
<td>67</td>
<td>18.5</td>
<td>18.5</td>
<td>Crushing @ Top Bearing</td>
</tr>
<tr>
<td>B2-RS (Recessed)</td>
<td>5</td>
<td>158</td>
<td>11.8</td>
<td>13.9</td>
<td>Crushing in Recessed Region</td>
</tr>
</tbody>
</table>

3.5.4.4 Discussion of Preliminary Test Results

The main objective for conducting these material tests was to assess the structural behavior of lightweight concrete made with pumice and the chemical admixture, Celdacrete. Construction of the test specimens also provided an excellent opportunity to receive assistance in proper proportioning of a typical mix. Engineers from Celdacrete
demonstrated the correct batching procedure from which quantitative mix designs were determined.

The mean compressive strengths for both sets of test cylinders made with the pumice stone from Mexico were nearly identical ($f'_{c}$ (MIM) = 1066 psi and $f'_{c}$ (PM) = 1060 psi). This provided a good indication that the mix design created in the laboratory at Rice University (PM) was representative of the one used in Mexico (MIM). On the other hand, the concrete produced with the pumice from California had an average compressive strength ($f'_{c}$ (PC) = 1330 psi) that was approximately 25 percent greater. Examination of the individual mix designs revealed a number of factors that probably attributed to this increased capacity.

First, the final w/c ratio for the PC mix was probably less than that of the PM mix. Because of high internal porosity, very dry pumice, similar to the sample from California, will ultimately absorb a large quantity of the additional mix water. Absorbed water should not be included in the determination of the actual w/c ratio of the concrete. Therefore, although the actual amount of water added to the PC mix (26.4 lbs) was 36 percent greater than that of the PM mix (19.4 lbs), the resulting w/c ratio may have been smaller. Since the quantities of pumice, cement and additive were roughly equivalent, the 2" slump difference also provided evidence that the PC batch had a lower w/c ratio.

Another likely reason for the variance in the compressive strengths was the quality and gradation of the aggregate. The pumice stone from Mexico was taken directly from the quarry, while the Californian pumice was washed and graded prior to delivery. The Mexican pumice was, therefore, more prone to possessing extraneous matter possibly harmful to the concrete. Furthermore, gradation of the aggregate for the PM and MIM
mixes was determined solely based on the in situ condition of the material. An optimal arrangement of individual particles was probably not achieved under this condition. Conversely, the PC pumice was washed and subjected to a sieve analysis. The final mix was composed of three different size particles with the relative proportions (by weight) of: 32 percent of the 1/2", 32 percent of the 3/16" and 36 percent of the 80/20 blend. These relative ratios were selected based on a visual assessment of the combined stone.

As shown in Table 3.9, the mean compressive strength, $f'_c$, of all the mixes made with pumice was directly proportional to the mean dry unit weight. The difference in density between the two samples batched at Rice University was most likely the result of having greater control over the gradation of the aggregate in the PC mix. Compared to normal weight concrete, the unit weight of the LWPSC was approximately 45 to 55 percent less.

The split tensile strength test and the modulus of rupture test were performed to provide an indirect measurement of the tensile strength of lightweight concrete. For normal weight concrete, the splitting tensile strength, $f_s$, is approximately equal to $6.7\sqrt{f'_c}$ (ACI 11.2.1.1) where $f'_c$ and $f_s$ are both in psi [48]. This expression is derived from the evaluation of a large number of tests with wide scatter in the test data. On the average, the tensile strength of concrete made with lightweight aggregate tends to be less than one made with normal weight coarse and fine aggregate. The mean value for the splitting tensile strength of 264 psi was 21 percent greater than the value obtained (219 psi) with the ACI expression. On the average, the modulus of rupture, $f_r$, should be approximately 1.5 x $f_s$. This series of tests, however, produced values much lower than expected. Undetected defects in the specimens, malfunction of the testing equipment or the limited number of tests may have accounted for this discrepancy.
Another method used to evaluate the tensile strength of lightweight pumice stone concrete was the diagonal tension tests of 2' x 2' x 4" panels. A 3" bearing length was originally chosen to correspond to the dimensions prescribed in ASTM C 1391-81, the standard test method for diagonal tension (shear) of masonry assemblages. Prior to the development of any diagonal cracking, crushing occurred at the bearing. Calculations based on the work of Stafford Smith and Carter [24, 47] revealed that this behavior was due to the contact length of the support seat. With the given panel geometry, material compression strength, \( f'_c \), and length of contact, the theoretical cracking load was calculated to be greater than the theoretical crushing strength. For the remainder of the tests the bearing seat was modified. The contact lengths were increased from 3" to 5", which was 20 percent of the span of a side. Fig. 3.13 displays the load-deflection plots and Table 3.11 compares the results of the tests with the increased bearing length. The experimentally determined cracking load and ultimate load are compared with the theoretical values. Initial and cracked stiffness for these specimens is also tabulated.

Reinforcement layouts used in the test specimens of type B1 and B2 did not affect the panels in any discernible manner. The initial intention for including reinforcement in these panels was to observe the influence different layouts had on post-cracking behavior. The final design objective was to provide a reinforcement arrangement that adequately confined the concrete and generated a large number of small, closely spaced cracks, rather than a few large ones. Unexpectedly, the rebar design and panel thickness was slightly altered in order to facilitate production in Mexico. Because of these changes, the provided steel was not detailed as originally anticipated. Smaller diameter (5/32") rebar was used instead of the standard U.S. sizes. The B1 arrangement had a ratio of
reinforcement to gross concrete area, \( \rho_g \), of 0.0008 in both directions, much less than the minimum required for walls in the ACI code. All of the reinforcement that was included terminated at the ends without any hooks so the development of the bars was also questionable. Similarly, the B2 scheme had a number of problems. The overall \( \rho_g \) was the same as that of B1, but instead of one layer of bars at the middle of the panel stirrups were used. Although stirrups can provide an excellent method of confining the concrete, the placement of these ties at the exterior edges of the specimens was not beneficial for the design intent of distributed cracking.

### Table 3.11

**Load and Stiffness Data for LWPSC Diagonal Tension Tests (5" Bearing)**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( P_{cr} ) (kips)</th>
<th>( R_{cr} ) (kips)</th>
<th>( P_u ) (kips)</th>
<th>( R_u ) (kips)</th>
<th>( K_i ) (Kip/in)</th>
<th>( K_{cr} ) (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-3</td>
<td>27.4</td>
<td>25.6</td>
<td>27.4</td>
<td>31.6</td>
<td>198</td>
<td>N/A</td>
</tr>
<tr>
<td>B1-2</td>
<td>28.7</td>
<td>25.5</td>
<td>31.4</td>
<td>31.6</td>
<td>168</td>
<td>71</td>
</tr>
<tr>
<td>B1-3</td>
<td>25.9</td>
<td>25.3</td>
<td>32</td>
<td>31.1</td>
<td>226</td>
<td>120</td>
</tr>
<tr>
<td>B2-3</td>
<td>18.5</td>
<td>25.5</td>
<td>18.5</td>
<td>31.1</td>
<td>340</td>
<td>N/A</td>
</tr>
<tr>
<td>A-SC</td>
<td>N/A</td>
<td>25.2</td>
<td>31.1</td>
<td>31.1</td>
<td>312</td>
<td>N/A</td>
</tr>
<tr>
<td>B2-RS</td>
<td>11.8</td>
<td>12.2</td>
<td>13.9</td>
<td>15.1</td>
<td>167</td>
<td>99</td>
</tr>
</tbody>
</table>

\( P_{cr} \) = Load at which first diagonal crack was visible.

\( R_{cr} \) = Theoretical diagonal cracking strength, assuming \( f'_c = 0.1f'_c \),
(based on Stafford Smith & Carter, 1969 [47]).

\( P_u \) = Ultimate load.

\( R_u \) = Theoretical compressive strength, \( R_u = 1.4f'_c \alpha t \),
(based on Stafford Smith, 1966 [24]).

\( K_i \) = Initial stiffness, Calculated as the slope between \( P_{cr} \) & the point at 5 kips.

\( K_{cr} \) = Cracked stiffness, calculated as the slope between \( P_u \) & \( P_{cr} \).
3.5.5 Infill Panel Material Selection

Both of the examined lightweight concrete materials were evaluated based on the
to the final decision. Ultimately, the pumice stone concrete was found to be a more promising alternative for a
number of reasons.

The material properties and results of preliminary diagonal tension tests for both
PAAC and LWPSC were presented in Section 3.5.3 and 3.5.4. As previously illustrated,
the unit weight for the PAAC can be as much as 50\% less than that of the LWPSC. The
strength and stiffness, on the other hand, of the LWPSC was substantially greater than
that of the PAAC. Behavior under diagonal tension was also compared as shown in Fig. 3.10. The final decision on infill material, however, was probably equally influenced by
practical and economic concerns.

Since the production of PAAC requires specialized equipment and plant facilities, it is
extremely capital intensive. Economic feasibility is achieved only if prefabricated pieces
are highly standardized and mass-produced. Currently available precast units could not be
effectively used in an infill panel system without major structural modifications. This
made it very difficult to get reduced-scale panels that could even be used for the
subassembly tests. Because of bond issues and for ease of placement, the reinforcement
provided in PAAC panels was limited to plain welded-wire fabric placed in one or two
layers. One of the final design objectives, providing continuous hoops parallel to both
sides of a panel, was not possible with this type of reinforcement. Panel to panel and
panel to frame connections were two other important issues that were more difficult to
address with PAAC. Low panel strength (especially in the regions around connections) and inflexibility in the fabrication process make it very difficult to design efficient and economical connections.

On the other hand, LWPSC can be produced by unskilled labor under conditions no different than those for normal weight concrete. The panels can also be reinforced in any manner and designed to accommodate a variety of connections. These were valuable properties that allowed all of the reduced-scale infill panels to be constructed at Rice University. Furthermore, pumice is an extremely abundant natural resource found throughout the world.

3.5.6 Infill Panel Design Philosophy

3.5.6.1 Infill Panel Not Connected to Frame

In order to facilitate handling and simplify installation, the infill wall was comprised of small, prefabricated units. The overall size of the individual units was determined based on the ability to manually place the panels without the assistance of any additional mechanical equipment. One of the main design objectives was to obtain distributed panel cracking through closely spaced longitudinal and transverse reinforcing. Panel reinforcement, therefore, consisted of closely spaced horizontal and vertical closed-loop stirrups, extending into the joint and forming a three dimensional cage. Hoops were provided to allow cracked panels to continue resisting load without deterioration and spalling of the concrete as observed in past research [6]. Continuous joint reinforcement was placed through the protruding panel hoops to enhance the shear transfer mechanism between individual panels. The capacity of the panel-to-panel connections were,
evaluated using the concept of shear friction. Tying the individual panels together in this
fashion also contributed to the resistance against out-of-plane inertial forces.

Initially, an infill wall comprised of 16 identical panels of uniform thickness was
considered. In order to investigate the simplest construction method and most cost
effective solution, no connections to the existing frame were included at this time.
Because of the lack of connectivity at the wall-frame interface, the proposed infill panel
was expected to act as a diagonal brace capable of resisting only compressive forces,
similar to that of a masonry infill. Analytical models developed for masonry infills were,
therefore, used as a guide in attempting to predict the behavior of the retrofitted
subassembly. The objective was to design a panel that dissipated energy through
dispersed diagonal cracking, and failed prior to damaging the surrounding frame. To
achieve this goal, the bounding frame was required to resist the maximum distributed
forces imparted to the columns and slabs due to frame-infill interaction. A 3" panel
thickness was selected for the preliminary calculations.

The maximum allowable diagonal strut force was determined by assessing the shear
capacity of the subassembly's columns and slabs. For the columns, the contribution of
concrete to shear strength, $V_c$, was based on a predictive equation proposed in Section
6.5.2.3 of FEMA 273 [9, 49] and shown in Equation 3.2.

$$V_c = 3.5 \lambda \left( k + \frac{N_u}{2000 A_g} \right) \sqrt{f_e' b_w d} \quad \text{Eq. (3.2)}$$

In the above equation, $k = 0$ in regions of moderate and high ductility demand and 1.0 in
regions of low ductility demand, $\lambda = 0.75$ for lightweight concrete and 1.0 for normal
weight concrete and the term, \( \frac{N_u}{2000A_e} \), represents the contribution of axial compressive stresses. Since no superimposed axial loads were applied to the subassembly, \( N_u \) was set equal to zero and the column shear strength capacity represented a lower bound. The final expression for \( V_c \) assumed \( k = 1.0 \) and was, therefore, reduced to \( V_c = 3.5\sqrt{f'c'b_u'd} \). Values for \( f'c' \) were determined from compression tests performed at the time of the bare frame test. Similarly, the shear capacities, both one-way and two-way, of the slab were calculated. It was ultimately found that the shear strength of the column controlled the design.

The location of the load imparted to the column from the infill panel was estimated based on the guidelines for masonry infills of Section 7.5.2.2 in FEMA 273 [9]. This approach assumed an eccentric diagonal brace as shown in Fig. 3.14. Although the interaction forces were actually distributed over the entire contact length, a concentrated force applied at a distance, \( l_{eff} \), measured from the top or bottom of the infill panel, was used to simplify the analysis. At these points, the horizontal component of the diagonal force was applied to the column. The procedure to determine the position of the vertical component of the diagonal strut on the slab was nearly identical. Expressions for the variables in Fig. 3.14 are given below.

\[
a = 0.175(\lambda \ h_{col})^{-0.4} r_{inf} \quad \text{Eq. (3.3)}
\]

\[
l_{eff} = \frac{a}{\cos \theta_c} \quad \text{where:} \quad \tan \theta_c = \frac{h_{inf} - \frac{a}{\cos \theta_c}}{L_{inf}} \quad \text{Eq. (3.4)}
\]
\[ l_{\text{eff}} = \frac{a}{\sin \theta_b} \quad \text{where:} \quad \tan \theta_b = \frac{h_{\text{inf}}}{L_{\text{inf}} - \frac{a}{\sin \theta_b}} \quad \text{Eq. (3.5)} \]

The ultimate diagonal crushing strength of the infill panel was estimated from the expression in Equation 3.1. It was found that with a 3" thick infill panel shear failure of the columns would occur prior to the development of the maximum diagonal strut force. As previously mentioned, this behavior was unacceptable. A 2" thick panel was subsequently checked with the same procedure and produced similar results. At this point, further reduction in the overall thickness of individual panels was not considered because of the high panel slenderness and the decrease in relative in-plane stiffness. In addition, a panel thickness less than two inches would be difficult to fabricate with the proposed design details (i.e. rebar hoops in both directions and grooved panel edges). Therefore, instead of decreasing the entire width of the infill, the thickness was reduced only in the central portion of the wall.

The recessed central panel was intended to behave as the ductile fuse of the system. The reduced thickness was designed to control the amount of force that could be generated through the diagonal strut mechanism in either direction. Due to large tensile stresses, the formation of closely spaced diagonal cracks in this region would provide seismic energy dissipation. The anticipated panel behavior was simulated at Rice University with a diagonal tension test on a 2' x 2' x 4" LWPSC panel with a 1' x 1' x 1" recess both front and back (B2-RS). An unreinforced 2' x 2' x 4" solid panel of type B2 (Fig. 3.12) was altered to produce the final recessed panel. The 1' x 1' center region was scored in perpendicular directions with a hand held grinder and the remaining concrete
was chipped away. The final cracking pattern and failure mechanism is shown in Fig. 3.10 and Photo 3.16. It is apparent that the ultimate crushing failure mode has been relocated from the loaded corner to the recessed region. The experimental and theoretical (based on Stafford Smith’s formulation [24]) compressive strengths (Table 3.11) are in good agreement for both cases. Similarly, the prediction of the onset of diagonal cracking correlated well with test results. In general, the stiffness of the panel did not appreciably decrease with the addition of the 1" recesses. Eliminating B2-3, which failed at a load much lower than expected, and A-SC, which had 1/4" wide x 1" deep scores at 6" on center on both sides, the initial stiffness of B2-RS was only about 15% less than the average of specimens A-3, B1-2 and B1-3. Since the majority of the damage will occur in the recessed panel, upgrade after an earthquake may simply consist of the removal and replacement of this section.

3.5.6.2 Infill Panel Connected to Frame

From the study of masonry infills, it was determined that the most efficient and reliable infill wall system required some type of connectivity with the existing frame. A number of benefits associated with adding mechanical anchorage were anticipated. First, the load path and resistance mechanism would be altered, reducing the large concentrated shear forces thrust into the column through infill wall-frame interaction. Second, since the load path was better understood, a connected wall had a more robust and predictable behavior. Finally, connections also secured the panel against out-of-plane inertial forces.

As shown in Fig. 2.3, a number of concepts for connecting infill walls to existing frames have been previously designed and tested. A major concern in this study was to
provide a connection that was both practical to use and simple to construct. Ultimately, rebar dowels, similar to the shear lugs used by Frosch et al. [8], were selected. To enhance the shear transfer capability between frame and wall, dowels were placed at the slab-wall interface. Each dowel was located at a panel-to-panel joint and designed to have enough capacity to allow the infill wall to fail first. It was determined, that any benefits associated with anchorage to the columns were not substantial enough to offset the difficulty in actually making these connections. For this reason, the wall was attached to the frame only through the top and bottom slabs.

Because of the relatively weak strength of existing flat-plate frames, infill strength was an extremely important issue. The main design goal was to create a panel that enhanced the seismic performance of the building without producing excessive damage or failure within the exterior frame. No additional retrofit measures, such as increasing the shear capacity of the columns, were considered since the infill wall was projected to behave as the ductile fuse of the system. Perforations within the infill panels were used to control, and ultimately fine-tune, the behavior of the wall.

Two different styles of perforations, circular holes and rectangular openings, were proposed and tested. The block-outs were uniformly located throughout the area of the wall and designed to produce distributed cracking. Of the two arrangements, the behavior of the wall containing rectangular openings was easier to predict. The wall was modeled similar to a shear wall system with uniform openings. Because of the aspect ratio of both the openings and the individual panels, the perforated wall was most analogous to a wall/frame system with strong girders and weak piers. An upper bound on the infill wall
shear strength was, therefore, determined by considering the shear capacity of all the separate piers at a given elevation.

3.5.7 Infill Panel Configuration

3.5.7.1 General

Overall dimensions of the reduced-scale infill units were predominantly governed by the projected weight of a full-scale panel. The primary design objective was to provide individual precast modules that were light enough to be constructed manually, without the need for any mechanical equipment. The final reduced-scale standard panel had a length of 1'-8 1/4", a height of 9" and a width of 3" (Fig. 3.15 and Photo 3.17).

Grooves were provided around the perimeter of each panel to improve the shear transfer capacity. Proportions of the groove satisfied the requirements of Sec. 6.18.1 in the PCI Design Handbook [50], which recommends minimum dimensions of 1 1/2" deep and 3" wide. Because these limits were provided for full-scale construction, the subassembly scale-factor was used to determine the minimum recess size, 0.6" deep and 1.2" wide, for the reduced-scale panels. An attempt was made to make the groove as large as possible within the constraints imposed by the small panel thickness. The initial design was 2" wide and 1" deep with a side slope of 63°. Formwork was constructed for this case and a prototype panel was poured. Difficulty in stripping the panel and spalling at the lip of the groove, however, prompted a revision to the layout. The final cross section consisted of a 3/4" deep by 1 3/4" wide groove with a side slope of 63° (Fig. 3.15).
Panel reinforcement in both directions consisted of continuous steel hoops. Sufficient steel was provided to fulfill the minimum reinforcement requirements of both the UBC and ACI building codes for wall construction in seismic regions. Section 21.6.2.1 of ACI 318-95 maintained that the reinforcement ratio, $\rho_v$, for structural walls should not be less than 0.0025 along the longitudinal and transverse axes [48]. The ratio of vertical reinforcement area to gross concrete area for an individual standard panel was 0.0025 and that of horizontal reinforcement area to gross concrete area was 0.0028. In addition, spacing of both horizontal and vertical rebar was less than the code specified maximum of three times the wall thickness or 18". Two slightly different reinforcement layouts (i.e. Type A & Type A' in Fig. 3.15) were used. Erection of the wall consisted of placing these panels in an alternating pattern. The purpose of providing two separate configurations was to ensure that the protruding hoop steel of adjacent panels was not located in the same area. Reinforcement layout and section details for the standard LWPSC panels are illustrated in Fig. 3.15.

Panel-to-panel joints contained two 5/32" diameter deformed bars running through the overlapping hoops of adjacent panels. All joints were fully grouted as shown in Fig. 3.16. The dowel connections, on the other hand, consisted of #6 rebar run through the entire thickness of the slab and embedded 10" into the vertical panel-to-panel joint (Fig. 3.17). Three connections were provided at both the top and bottom of the wall.

The following sections contain detailed descriptions of each retrofitted specimen that was tested. Sketches of all the tested specimens are provided in Figs. 3.18, 3.20 & 3.21. In addition, a summary of the major variables is also given in Table 3.12.
Table 3.12

Summary of Retrofitted Subassemblies

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Connections with Frame</th>
<th>Perforations</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Type</td>
<td>Location</td>
</tr>
<tr>
<td>S1-BF</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>S1-RN</td>
<td>None</td>
<td>None</td>
<td>N/A</td>
</tr>
<tr>
<td>S2-HC</td>
<td>Between Slabs &amp; Infill Wall</td>
<td>Circular (2 1/2&quot; Ø Holes)</td>
<td>Middle Portion of Wall</td>
</tr>
<tr>
<td>S2-HCA</td>
<td>Between Slabs &amp; Infill Wall</td>
<td>Circular (2 1/2&quot; Ø Holes)</td>
<td>Entire Wall</td>
</tr>
<tr>
<td>S3-OC</td>
<td>Between Slabs &amp; Infill Wall</td>
<td>Rectangular (5&quot; x 4&quot;)</td>
<td>Entire Wall</td>
</tr>
<tr>
<td>S4-OIX</td>
<td>Between Slabs &amp; Infill Wall</td>
<td>Rectangular (5&quot; x 4&quot;)</td>
<td>Entire Wall</td>
</tr>
</tbody>
</table>

3.5.7.2 Specimen S1-RN

Specimen S1-RN was constructed from specimen S1-BF after the completion of the test program for the bare frame. The panel-to-frame joints were tightly grouted, but there was no mechanical attachment to the existing frame. Because of the lack of connectivity, the behavior of the infill panel was anticipated to mirror that of a masonry infill wall. To control the maximum concentrated force that could be transferred to the columns, the infill wall was recessed over its central region as shown in Fig. 3.18. The area of reduced-thickness was designed to behave as the ductile fuse of the system.

The recessed portion of the infill wall was composed of a single panel that covered the same area as four standard panels and measured 3'-5 1/2" wide by 1'-7" tall. In order to
provide identical edge conditions as those of the standard panel, the thickness at the boundary was 3". This thickness extended for 3" all around the panel and allowed the use of identical joint detailing (i.e. a continuous groove around the perimeter and hoop reinforcement extending into the grout joint). As a result, the recessed central region had dimensions of 2'-11 1/2" x 1'-1" and a uniform thickness of 1". A single layer of reinforcement, consisting of 5/32" diameter rebar located at 3" on center in each direction, was used in this section. See Fig. 3.19 and Photo 3.18 for the geometry and reinforcement layout of this panel. Aside from the reduced-thickness panel, the remaining wall was constructed from panels with the standard configuration.

3.5.7.3 Specimens S2-HC & S2-HCA

Specimens S2-HC & S2-HCA both investigated the inclusion of perforations within the infill wall on the behavior of the retrofitted subassembly. The perforations for this series of tests consisted of circular holes located within the interior of the standard panel. Sixteen standard panels without openings were initially installed into the slab-column frame subassembly. Unlike specimen S1-RN, dowel connections between the infill panels and the slabs were included at both the top and the bottom of the wall. No mechanical attachment was provided at the column-wall interface.

Since standard panels were cast without perforations, the circular holes were created after the panels were grouted into the bounding frame. A carbide-tipped rotary saw was used to drill 2 1/2" diameter holes between the existing reinforcement. Specimen S2-HC consisted of a series of holes within the middle two stacks of standard panels. Each panel contained two symmetrically positioned openings as shown in Fig. 3.20. A total of 2%
(78.5 in\(^2\)) of the surface area of the infill wall was removed. Because of the relatively small overall percentage of drilled holes, specimen S2-HC was very similar to a solid wall. After testing specimen S2-HC, the wall was further weakened by the inclusion of additional holes. The resulting configuration consisted of three holes per panel uniformly distributed throughout the entire wall. Unlike S2-HC, S2-HCA contained 235.6 in\(^2\) of openings, or 6% of the infill panel's total area. The elevation layout of specimen S2-HCA is illustrated in Fig. 3.20.

3.5.7.4 Specimen S3-OC

Similar to specimens S2-HC and S2-HCA, specimen S3-OC also examined the effect of perforations on the performance of infilled slab-column frames. Instead of circular holes however, rectangular openings were included throughout the infill wall. The rectangular openings occupied approximately 640 in\(^2\), or 16% of the total wall area. Placing wooden block-outs within the existing formwork for the standard panels created the voids (Photo 3.19). Each panel contained two 5" wide by 4" deep openings symmetrically positioned as shown in Fig. 3.21. The overall dimensions of the resulting infill panel are given in Fig. 3.22. In order to accommodate the rectangular perforations, the reinforcement layout of the standard panels had to be slightly adjusted. The hoop dimensions and amount of reinforcement, however, remained unaltered. Every other parameter, including dowel connections to the slab, was identical to specimen S2-HCA.
3.5.7.5 Specimen S4-OCX

Specimen S4-OCX was the same as specimen S3-OC except for the inclusion of additional panel reinforcement. Between the voids within each standard panel diagonal reinforcement was placed. Two 5/32" diameter bars, one at each face of the panel, were used in both directions and located inside of the main vertical stirrups (Fig. 3.23 & Photo 3.20). The diagonal bars extended past the corners of the openings and were bent into the concrete directly adjacent to the top and bottom edge of the block-out.

3.6 Infill Panel Fabrication

3.6.1 Infill Panel Formwork

Eight individual wooden forms, four of type A and four of type A' (Fig. 3.15), were constructed for the standard panels. In order to produce the required number of panels, the forms were reusable. Each unit was built horizontally and consisted of four side rails attached to a 3/4" plywood base. A typical side rail was composed of a wooden block cut to the dimensions of the edge groove and attached to the center of a section of 2x4. The 2x4 was trimmed to a width of 3" and cut to the appropriate length. After securing the groove block to the 2x4, a radial arm saw was used to provide notches at the locations where the 5/32" diameter hoops protruded from the formwork. These notches also acted as a template for lying out and supporting the reinforcement. A photograph of the completed formwork for a standard panel is shown in Photo 3.21.
3.6.2 Infill Panel Reinforcement

All of the infill panels were reinforced with 5/32" diameter deformed bars. Aside from the recessed central portion of specimen S1-RN, the reinforcement entirely consisted of continuous hoops situated along both the horizontal and vertical axes of each panel. A three dimensional cage was thus created, confining the concrete and limiting deterioration under cyclic loading. Spacing limitations were based on the ACI 318-95 [48] seismic design criteria for shear walls. Closely spaced reinforcement provided an efficient mechanism for controlling the size and spacing of the cracking within the infill wall.

The 5/32" diameter reinforcement was shipped from Mexico to Rice University in 20' pieces. Bolt cutters were used to cut the wire into sections of the required length and the bending was done by hand at Ryon Laboratory. In order to properly develop the rebar, the free legs of all stirrups overlapped a distance of 7". The vertical stirrups were placed inside of the horizontal ones and a 1/2" clear cover was maintained between the top and bottom surfaces of each panel. In addition, the ends of each hoop were extended 1 1/2" past the edge of the panel to provide continuity between the reinforcement of adjacent panels. The typical reinforcement layout of a standard panel is shown in Photo 3.21.

3.6.3 LWPCSC Pouring

The LWPCSC was mixed at Ryon Laboratory in an open-top revolving blade mixer with a capacity of 3.5 ft³. Identical batching procedures were established for every load of the LWPCSC. First, the total weight of pumice stone and cement was loaded into the drum. The dry ingredients were allowed to mix for a minute before half of the water was added. Next, the admixture, Celdacrete (in powder form), was slowly spread into the rotating
drum. After inclusion of all the Celdacrete, the remainder of the water was added and mixing continued for another few minutes. A slump of 3" to 4" was consistently achieved.

Because only enough forms were constructed to cast half the panels at one time, each infill wall required two pours. Each pour consisted of two separate batches of concrete, yielding approximately four panels and four 6" diameter x 12" long cylinders. The panels were filled by hand and adequate consolidation was achieved by placing the panels on a 20" x 20" vibrating table. The vibration was performed twice, first when the form was roughly half full and second when it was full. A hand trowel was then used to remove any excess concrete and provide a smooth, finished surface. The formwork was removed after 48 hours, so that the process could be repeated for the remaining panels. All of the precast panels and test cylinders were cured under polyethylene sheeting with a thickness of 6-mil.

3.6.4 LWPSC Mix Design

The pumice stone, mined in Greece, was provided by Tarmac America of Chesapeake, Virginia. Four tons of material was shipped from a storage facility in Brownsville, Texas to Houston, Texas and stockpiled behind Ryon Laboratory. The aggregate met the requirements of ASTM C331 with a dry loose unit weight of 45.5 pcf and a dry rodded unit weight of 55.0 pcf. Because of the reduced-scale of the infill panels, the maximum particle size was limited to 3/8". A sieve analysis performed according to ASTM C136 is presented in Table 3.13.
### Table 3.13

**Sieve Analysis for Pumice Stone**

<table>
<thead>
<tr>
<th>Sieve Size (ASTM C136)</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot;</td>
<td>100</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>99.7</td>
</tr>
<tr>
<td>#4</td>
<td>82.5</td>
</tr>
<tr>
<td>#8</td>
<td>51.9</td>
</tr>
<tr>
<td>#50</td>
<td>22.6</td>
</tr>
<tr>
<td>#100</td>
<td>13.8</td>
</tr>
</tbody>
</table>

A company in Guadalajara, Mexico provided the admixture, Celdacrete, used to increase the workability of the LWPSC. In addition, Celdacrete entrained air and was also said to increase the adhesion properties of the concrete. Prior to pouring the infill walls, a series of trial mixes were designed and tested in order to better understand the quantitative effects of this additive. Four similar batches of LWPSC were proportioned, identical except for differing amounts of Celdacrete (Table 3.14). Relationships between the amount of Celdacrete and the wet and dry properties of LWPSC were established. In Fig. 3.24 through 3.26 the effect of Celdacrete on slump, wet unit weight, dry unit weight, yield, cement content, compressive strength and splitting tensile strength is presented.

Final mix design proportions for the infill panels were based on the findings of this investigation, the properties of the pumice stone and on a preliminary study of LWPSC. The ratio of pumice stone to type I portland cement was 3 to 1 by weight and the w/c ratio was between 0.52 and 0.55. The exact amount of water added was varied slightly between different batches to produce the desired consistency. Finally, 1.5% (by weight of...
cement) of Celdacrete was added to each load. The above mix design was maintained for the infill panels of all specimens.

**Table 3.14**

<table>
<thead>
<tr>
<th>Mix Design for LWPSC with Celdacrete</th>
<th>Mix Designs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P-0C</td>
</tr>
<tr>
<td>Pumice Stone (0&quot;-3/8&quot;) (lbs)</td>
<td>84</td>
</tr>
<tr>
<td>Cement (Type I) (lbs)</td>
<td>28</td>
</tr>
<tr>
<td>Water (lbs)</td>
<td>16</td>
</tr>
<tr>
<td>Celdacrete (lbs)</td>
<td>0</td>
</tr>
<tr>
<td>Celdacrete (% Weight of Cement)</td>
<td>0</td>
</tr>
</tbody>
</table>

3.6.5 **Material Properties of Reinforcement**

All of the infill panel and joint reinforcement consisted of 5/32" diameter deformed bars purchased from a supplier in Mexico and shipped to Rice University. The use of small-diameter deformed bars made it possible to provide bi-directional hoops in the reduced-scale panels. Given the four-tenth-scale of the subassembly, the selected reinforcement was representative of standard #3 rebar. Three samples were taken from the shipment and tested to determine the material properties.

Each specimen was tested under the same conditions as the reinforcement for the subassembly. Shown in Fig. 3.27, the typical stress-strain relationship was similar to that of the #2 smooth bars. The nominal bar area used in determining the engineering stress
was 0.0192 in\(^2\). The mean value for the yield stress, \(f_y\), calculated at an offset strain of 0.2\%, was 94.1 ksi., and the mean ultimate stress, \(f_u\), was 101.4 ksi. An average modulus of elasticity of 31,519 ksi was recorded.

3.6.6 Material Properties of LWPSC

Eight 6" diameter x 12" long cylinders were cast along with every infill wall. After removal of the forms, the strength specimens were cured adjacent to the wall panels in an identical environment. Four of the cylinders from each specimen were used to measure compressive strength, \(f'_c\), at the time of the quasi-static test. The remaining four cylinders were used to determine the splitting tensile strength, \(f_s\), also at the time of the test. Test procedures conformed to ASTM C39-96 and ASTM C496-96 for the compressive strength and splitting tensile strength, respectively. All of the compression tests were conducted in a 220-kip hydraulic MTS material-testing system. The splitting tensile tests, on the other hand, were performed on a 300-kip Tinius Olsen material-testing machine.

In addition to the strength data, the modulus of Elasticity, \(E_c\), of the LWPSC was determined from the stress-strain relationship. The elastic modulus was estimated as the chord modulus measured between 40\% of the ultimate strength (0.4\(f'_c\)) and a stress of 200 psi. 200 psi was selected because it was a good approximation of the end of an initial concavity in the stress-strain curve (Fig. 3.28). This early behavior was due to the seating of the specimen. Prior to performing any tests, unit weight values for the LWPSC wall panels were established by weighing the strength test cylinders. The average density was computed based on the expanded volume of the cylinders. Values for all of the above mentioned material properties are given in Table 3.15.
Table 3.15

Mean Properties of LWPSC for Infill Walls

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Pour</th>
<th>Age @ Test (days)</th>
<th>( f_{c'} ) @ Test (psi)</th>
<th>( f_s' ) (psi)</th>
<th>( \frac{f_s'}{\sqrt{f_c'}} )</th>
<th>( E_c' ) @ Test (x 10^5 psi)</th>
<th>Density' @ Test (lbs/ft^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RN</td>
<td>BHW</td>
<td>54</td>
<td>1460</td>
<td>300</td>
<td>7.85</td>
<td>5.74</td>
<td>77.6</td>
</tr>
<tr>
<td></td>
<td>THW</td>
<td>49</td>
<td>1170</td>
<td>250</td>
<td>7.31</td>
<td>4.49</td>
<td>75.8</td>
</tr>
<tr>
<td>S2-HC &amp; S2-HCA</td>
<td>BHW</td>
<td>62</td>
<td>1390</td>
<td>270</td>
<td>7.24</td>
<td>4.57</td>
<td>78.0</td>
</tr>
<tr>
<td></td>
<td>THW</td>
<td>58</td>
<td>1360</td>
<td>225</td>
<td>6.10</td>
<td>4.92</td>
<td>78.4</td>
</tr>
<tr>
<td>S3-OC</td>
<td>BHW</td>
<td>23</td>
<td>1190</td>
<td>200</td>
<td>5.80</td>
<td>6.37</td>
<td>76.8</td>
</tr>
<tr>
<td></td>
<td>THW</td>
<td>20</td>
<td>1120</td>
<td>215</td>
<td>6.42</td>
<td>4.93</td>
<td>77.4</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>BHW</td>
<td>25</td>
<td>1280</td>
<td>200</td>
<td>5.59</td>
<td>6.19</td>
<td>82.8</td>
</tr>
<tr>
<td></td>
<td>THW</td>
<td>22</td>
<td>1150</td>
<td>195</td>
<td>5.75</td>
<td>4.94</td>
<td>81.9</td>
</tr>
</tbody>
</table>

BHW = Bottom half of wall (Pour #1); THW = Top half of wall (Pour #2)

1 Average of three test specimens

3.7 Retrofit Fabrication

The final procedure developed for constructing the infill walls was primarily based on practical concerns. The main goal was to create a system that could be manually built by a small number of unskilled laborers without the need for any additional mechanical equipment. Outside of some minor assistance, a single individual erected the infill panels for all of the reduced-scale subassemblies in this study.

Before any of the infill walls were constructed, holes were drilled through the slabs to support the dowel connections between the infill wall and the surrounding frame. A 2" diameter opening was provided at the location of every vertical panel-to-panel joint on both the top and bottom slabs. Since the drilling required cutting portions of reinforcing
steel, a diamond-tipped hole-saw was used. Loose material was removed from the surface of the holes with a wire brush. No other preparation to the bounding frame was performed prior to grouting.

Once the dowel connections were drilled, four wooden panels consisting of 1/2" thick plywood were placed within the slab-column frame, and aligned with the back face of the infill wall. Each segment spanned vertically from the top of the bottom slab to the underside of the top slab. Placed side-to-side the individual pieces filled the entire area that the infill wall would occupy. Once in position, the planks were clamped to each other and to the main column sections. In addition, 2 x 4's were used to brace the plywood in the direction perpendicular to the plane of the infill panels. The wooden wall was designed to not only align and support the LWPSC panels, but also to act as formwork for one side of the grout strip. Photo 3.22 shows the assembly of the retrofit strategy up to this point.

After the insertion of the support wall, the individual precast panels were positioned. Each panel was supported by two 1/2" diameter bolts that were threaded into inserts located along the flat portion of its bottom edge (Photo 3.23). The inserts consisted of nuts secured to the side of the forms and cast into the panels. In addition to support, the bolts were used to maintain the proper width of the horizontal joint.

All of the panel to panel and panel to frame joints were completely grouted for each of the retrofitted subassemblies. It was originally intended to use the same LWPSC material in the grout strips as was used to fabricate the panels. Concerns about workability, however, eliminated this option as a suitable alternative. Because the grout would be placed by hand, without the aid of a pump or additional vibration, into very narrow formwork, a fluid consistency was required. The LWPSC could not achieve this
consistency and, therefore, a commercially available non-shrink / non-stain grout complying with ASTM C 1107 was used. To obtain a fluid consistency, 1.2 gallons of water were mixed with each 50-pound sack of dry material.

The material properties of the grout were based on tests of 2" cubes. Stress-strain curves were generated and used to determine the compressive strength, $f_g'$, and secant modulus of elasticity, $E_g$, for each specimen. Because of an initial concavity in the stress-strain curve, the secant modulus was defined as the slope of the line connecting the data point at a stress of $0.4f_g'$ with that at a stress of 200 psi. Fig. 3.29 shows a typical stress-strain relationship along with a graphical representation of the secant modulus. In addition, all of the findings are summarized in Table 3.16.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age (days)</th>
<th>Number Tested</th>
<th>$f_g'$ (psi)</th>
<th>$E_g$ (x 10^5 psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-RN</td>
<td>19</td>
<td>4</td>
<td>6138</td>
<td>6.38</td>
</tr>
<tr>
<td>S2-HC &amp; S2-HCA</td>
<td>24</td>
<td>4</td>
<td>7225</td>
<td>9.02</td>
</tr>
<tr>
<td>S3-OC</td>
<td>12</td>
<td>4</td>
<td>6246</td>
<td>4.87</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>11</td>
<td>3</td>
<td>5603</td>
<td>4.75</td>
</tr>
</tbody>
</table>

Table 3.16

Mean Compressive Strength of Grout for Infill Wall

The infill wall was built in stages to facilitate the placement of grout, especially within the horizontal joints. Each row of panels was placed, properly aligned and completely grouted before the next row was erected (Photos 3.24 & 3.25). The grout was mixed adjacent to the specimen in a portable paddle type mixer with a capacity of 1 cu ft. Since only one row of panels was set at a time, access to the grout strips was provided at the top
of the vertical joints. The grout was manually poured into these openings, and because of its fluidity, did not need any additional vibration. For each horizontal tier of infill panels, the entire process took approximately one day. At the horizontal joint at the top of the wall the grout was poured through the upper slab into the holes created for the dowel connections. As a whole, the method developed to grout the infill panels was proven to be adequate for the reduced-scale the specimens. A more efficient process, however, could be achieved for full-scale retrofit with the assistance of a portable grout pump.

All of the grout strips were reinforced with two 5/32" diameter deformed bars identical to those used in the infill panels. The joint reinforcement was positioned inside of the overlapping hoops that protruded from the panel edges (Photo 3.26). The dowels consisted of #6 bars taken from the same stock as the longitudinal column reinforcement in the slab-column subassemblies. Each dowel was embedded into the frame for the entire width of the slab and into the infill wall for 10" (Photo 3.27). Furthermore, at the dowel connection the rebars in the vertical joint also extended into the slab.

Finishing the fabrication of the wall, the retrofit procedure was complete for all of the specimens except S2-HC and S2-HCA. These specimens contained a number of 2 1/2" diameter holes that were drilled into the wall after it was constructed. Because of the abrasive properties of the pumice stone, a carbide-tipped rotary saw was used to drill the holes. The LWPSC was extremely easy to cut through with the use of only a standard cordless drill.
3.8 Test Configuration

The specimens were tested in a self-stressing steel reaction frame as shown in Fig. 3.30 and Photo 3.28. Top and bottom column stub connections were designed and fabricated to behave in a pinned manner. Prior to setting the specimen in the final position, a t-shaped assembly of steel plates was attached to each column stub (Fig 3.30). This assembly consisted of a 3" thick vertical ear plate bolted to a 2" thick horizontal plate containing four 7/8" diameter holes. Through the center of the 3" thick plate 2" diameter holes were machined for the top connections and 2 1/2" diameter holes for the bottom connections. The entire configuration was set in place with the embedded 3/4" diameter threaded rods protruding through the 7/8" diameter holes. Nuts threaded at the end of the 3/4" rods anchored the connection hardware.

At the bottom, the ear plate slid between two 3" thick plates bolted to the top of a W18x119 steel crossbeam. Similar to the vertical plate attached to the column, each 3" thick plate also contained a 2 1/2" diameter hole. After proper alignment, a 2 1/2" diameter steel pin was tapped through the three openings and the ends secured with retaining rings. The top column stub connection, on the other hand, was not anchored directly to the reaction frame, but rather to the loading beam. Two 2" diameter holes were drilled in the beam to match the size of the opening in the top ear plate. A 2" diameter steel rod was used to fasten the upper assemblage. Pictures of the top and bottom connections are shown in Photos 3.29 and 3.30.

In addition to transferring the lateral force, the loading beam was also used to transport the structure from the loading dock into the reaction frame (Photo 3.31). The actual beam was fabricated from two 20' long MC 6 x 18 (A36) sections placed back to back with 4"
spacers. The spacers were 2" diameter pipe cut to the proper length and spaced at approximately 1'-7" on center. 1/2" diameter bolts were used inside the pipe to compress the sections together. The total width of the final built up section was 11". Because of the precision fit required by the pin connection, the 2" diameter holes were bored on a milling machine to a tolerance of one-thousandth of an inch. 4" x 6" x 1" plates with 2" diameter holes were welded to the interior web of the MC sections in order to strengthen the region of bearing. After welding, the final openings were filed by hand until the pins achieved the proper fit and were able to rotate smoothly. A 2" thick stiffened angle was bolted to the top flanges of the MC sections near the middle of the beam. Attached to this angle was a 110-kip hydraulic actuator that provided the horizontal loading (Photo 3.32).

The tests were conducted without any superimposed gravity load applied to the specimens. Because axial loads tend to increase column shear strength, these tests represented a lower bound (i.e. worst case scenario) on the column shear capacity. Slab capacity, on the other hand, decreases with the application of gravity loading. The relationship between slab gravity load and lateral drift (Fig. 2.2), however, has been previously investigated by a number of researchers and is thoroughly understood. These findings were, therefore, used in interpreting the results of this study.

3.9 Loading History

All of the tests were performed under displacement control and consisted of fully reversed cyclic loading at increasing levels of deformation. Due to the configuration of the testing equipment, the system’s control and feedback signals were both located at the elevation of the actuator. Peak displacements for each level of deformation were,
therefore, determined in terms of the overall drift (the actuator displacement divided by the total height of the specimen) of the subassembly. It was determined that values for the overall drift and inter-story drift would be fairly consistent for the bare frame (Eqs. (3.6) & (3.7)).

\[
\text{Overall Drift} = \Delta_{od} = \frac{\delta_{\text{top}}}{83.5''} \quad \text{Eq. (3.6)}
\]

\[
\text{Interstory Drift} = \Delta_{id} = \frac{\delta_{\text{rel.}}}{h_{\text{story}}} = \frac{\delta_{ts} - \delta_{bs}}{48''} \quad \text{Eq. (3.7)}
\]

where:
- \(\delta_{\text{top}}\) = displacement of the actuator (in)
- \(\delta_{ts}\) = displacement of the top slab (in)
- \(\delta_{bs}\) = displacement of the bottom slab (in)

The loading protocol conformed to the provisions of FEMA 273 [9] and consisted of increasing the overall drift in increments of 0.25% up to 1.0% (Fig. 3.31). Three full cycles were performed at every drift level to investigate the degradation in stiffness, strength and energy dissipation under repeated motions. After completing the series of cycles at 1.0% drift, one cycle each at 0.75%, 0.50% and 0.25%, respectively, were executed. This sequence of cycles was included to quantify the residual stiffness after damage had occurred. The remaining portion of the load history was designed to develop the ultimate failure mechanism of the subassembly.

The bare frame specimen was subjected to an additional single cycle at 1.5% overall drift. This drift level represented a reasonable upper bound for punching shear failure of older flat-plate buildings under typical gravity load conditions. Since the bare frame
specimen was eventually retrofitted and re-tested, the test was terminated at this point prior to failure of the subassembly. All of the infilled subassemblies, on the other hand, were subjected to one additional cycle at 2.0% and 2.5% overall drift. These large displacement levels were included to capture the apparent loss of lateral and/or gravity load resisting capacity.

Comprised of a series of ramp and hold commands, the MTS TestStar testing system was used to program the loading history. The actuator movement was held every 0.105 inches so that all of the strain gage and LVDT data could be acquired. Data collection lasted approximately two minutes. The loading rate between hold commands was 0.01 in/sec. Displacements to the west (actuator pushing) were negative and those to the east (actuator pulling) were positive.

3.10 Instrumentation

3.10.1 Subassembly Strain Gages

A total of 24 strain gages were placed on the reinforcing steel at various locations throughout each test specimen. Standard 1/8" (CEA-06-125UN-120), foil backed, electrical resistance strain gages were used. All of the gages were 120.0 Ω ± 0.3% with a gage factor of 2.065 ± 0.5%. The area of installation on all deformed rebar was de-scaled and smoothed around the circumference with a grinder wheel. Surface preparation, bonding of gages, soldering of lead wires and application of protective coatings was performed using standard Micro-Measurements procedures. The location and numbering of the strain gages is illustrated in Fig. 3.32 (the location of the slab gages in plan is
provided in Figs. 3.3 and 3.4). As shown, gages were applied on the slab reinforcement, longitudinal column reinforcement and column stirrups.

It should be noted that the data obtained with this type of strain gage is typically not accurate enough to provide any reliable quantitative information. The main intention of providing the strain gages, therefore, was to capture general behavioral trends, such as the onset of yielding, in a qualitative manner. At each slab-column connection, 2" from the face of the column, there were two gages located on the top slab reinforcement and two gages on the bottom. The instrumented rebar in the top layer included the center bar, which was continuous through the column, and the next adjacent bar, located just outside the column. Both bottom bars were in the center of the slab and protruded into the column 2 3/4". All of the gages on the upper reinforcement were placed on the top portion (closest to the top of the slab) of the steel and those on the lower where situated on the bottom (closest to the bottom of the form). The top gages monitored the state of stress in the steel as a result of negative moment in the slab. Since the bottom gages were located on slab reinforcing bars that were not continuous through the column, their primary purpose was to provide an indication of the point at which pullout due to insufficient development length occurred.

Because the subassembly was symmetric and subjected to a cyclic loading, only one of the columns was instrumented. A total of four gages were placed on the longitudinal steel in the main column. Two of these gages were located 2" from the bottom of the top slab. They were positioned on the outside of the bars in the direction of loading. The other two were placed in the splice region, 2" from the top of the bottom slab. Unlike the top two gages, these were situated on the inside (relative to the direction of loading) of
the column reinforcement that was terminated above the slab. Congestion in the splice region was the primary concern in choosing to position the gages in this manner. The main reason for including this group of gages was to provide an indication of the tension forces imparted to the specimen’s columns.

The final four gages were located on the column ties. Along the main column span, the top two and bottom two stirrups were instrumented. All of these gages were placed on the outside of the tie, relative to the core of the column and on a leg parallel to the direction of loading. The original intent was to have some measurement of the shear stresses resisted by the columns.

All of the strain gages were instrumented in a quarter bridge configuration and excited with 3.333 volts. Voltage data was acquired through two National Instruments SCXI-1122 modules and converted to strain by a LabVIEW virtual instrument. Two 2.5 kΩ ± 0.02% resistors were provided by the SCXI-1122 hardware. An additional 120 Ω ± 0.25% resistor was connected to each strain gage channel to complete the Wheatstone bridge. Strain measurements were taken after every one-tenth of an inch of deflection in the hydraulic actuator. The motion of the actuator was held during the recording of each strain reading. Positive strain readings reflected tensile movements.

3.10.2 Linear Variable Displacement Transducers (LVDT’s)

Three LVDT’s (Linear Variable Displacement Transducers) were used to measure the lateral deformation of the subassembly. An LVDT attached to the actuator (TD) monitored the top displacement and was the control signal for the loading history. The other two transducers were located on the east face of the specimen at the elevation of
both the top (TS) and bottom (BS) slabs. Each of these instruments was positioned at the midpoint of the specimen in the transverse direction and 1 1/2" below the top surface of the slab. An aluminum angle clamped to the slab was the attachment point for the transformer’s movable core. The main shaft of the LVDT was supported on an aluminum channel mounted at the center of a 3" x 3" x 5/16" steel angle. The angle iron was bolted to the columns of the reaction frame and provided a fixed reference point.

The maximum span of the LVDT in the actuator was ± 5 inches and at the slabs it was ± 2.5 inches. Control (displacement) and feedback (load) data from the actuator was recorded through the MTS TestStar system every second. Displacement data at the slab levels was acquired with a National Instruments SCXI-1122 module. Each channel was excited with 10 volts by an external power supply. Measurements were made after every one-tenth of an inch of deflection in the hydraulic actuator. The motion of the actuator was held during the recording of each displacement reading. The primary reason for situating the LVDT’s at each floor level was to monitor the interstory drift throughout the loading history. The interstory drift, \( \Delta_{ld} \), was calculated as the difference in the displacements of the top and bottom slabs divided by the story height (48 inches).

Wire-type (UniMeasure LX-PA series) displacement transducers with a 4-inch range were placed diagonally across the infill surface (D1 & D2) to monitor infill shear deformations. Each unit was attached to an aluminum plate with two brass screws. Prior to mounting the LVDT, however, the aluminum plate was glued to the surface of the infill with a high-performance contact adhesive. In the opposite corners a small eyelet was attached to the infill wall in a similar fashion. Fishing line connected the end of the LVDT to the eyelet giving an overall diagonal length of 6'-4 1/4" for specimens S1-RN,
S2-HC and S2-HCA. Because of the size and position of the rectangular openings in specimens S3-OC and S4-OCX, the location of the LVDT’s was slightly adjusted. The resulting configuration had a diagonal length of 5'-11 3/4". Data acquisition was identical to that of the LVDT’s at the slab levels. The infill shear stain was determined using Eq. (3.8).

\[
\gamma = \frac{\Delta E + \Delta S}{2h_d \cos \theta}
\]  

Eq. (3.8)

where:  
\( \gamma \) = shear strain (in/in)  
\( \Delta E \) = diagonal extension (in)  
\( \Delta S \) = diagonal shortening (in)  
\( h_d \) = vertical distance from LVDT to eyelet (in)  
\( \theta \) = angle infill panel diagonal makes with horizontal

The LVDT layout is illustrated in Fig. 3.33.
CHAPTER 4
DISCUSSION OF TEST RESULTS

4.1 Introduction

The following chapter presents all of the experimental test results along with discussions on the findings. The first section provides a detailed description of the observed behavior and cracking patterns for all of the specimens. Each retrofitted subassembly is evaluated based on its ability to enhance the bare frame performance without inducing brittle failures within the slabs or columns. Next, all of the measured test results including; cyclic load-displacement relationships, shear strain behavior for the infill walls and strain gage histories are summarized and discussed. Calculated from the load-displacement plots, energy dissipation and peak-to-peak stiffness values are given for each cycle of loading. Finally, the results of repeated cycles at the same drift level are tabulated and compared.

4.2 Observed Behavior & Cracking Patterns

4.2.1 General

All of the cyclic loading was performed in stages. At the maximum deflection for each cycle the specimen was examined and all of the cracking highlighted with felt tip markers. Different colors were used to distinguish between the damage occurring at different levels and to visually monitor crack propagation. After each stage of loading,
photographs were taken to document the condition of the subassembly. Cumulative
damage under repeated loading at the same deformation level was investigated during the
overall drift levels of 0.25% through 1.0%. The entire experimental protocol for each
subassembly required approximately four days of testing. All cycles for a given drift level
were completed within the same day. At the end of each session, the specimen was
always positioned at the point of zero displacement. Readings from the instrumentation
(strain gages, LVDT's and load cell) were very consistent between the end of testing on
one day and the start of testing the following day. Because of this, creep of the specimen
was not considered a significant factor. Dates for the beginning and ending of all the tests
are given in Table 4.1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Start Date</th>
<th>End Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-BF</td>
<td>9/17/99</td>
<td>9/22/99</td>
</tr>
<tr>
<td>S1-RN</td>
<td>1/17/2000</td>
<td>1/20/2000</td>
</tr>
<tr>
<td>S2-HCA</td>
<td>2/10/2000</td>
<td>2/14/2000</td>
</tr>
</tbody>
</table>

4.2.2 Specimen S1-BF

The bare frame specimen was tested to a maximum overall drift of 1.5% at which time
the test was stopped and the subassembly retrofitted. 1.5% drift was chosen as the
termination point for the bare frame for two reasons. First, this drift level was considered
a reasonable upper bound for flat-plate buildings prior to experiencing punching shear
failure. Secondly, since the specimen would ultimately be retrofitted and retested, an attempt was made to limit any excessive damage.

The front elevation view of specimen S1-BF before the beginning of the subassembly test is shown in Photo 4.1. Visible cracking first occurred in the specimen at an overall drift of 0.25%. Each slab-column joint experienced a similar crack situated at the interior face of the column and spanning the entire width of the slab. These cracks were the result of flexure and are illustrated in Fig. 4.1. No other visible damage to the subassembly was apparent at this time. As the loading increased, additional flexural cracks developed spreading out toward the center of the slabs. The final cracking pattern at 1.5% overall drift is shown in Fig. 4.2. In addition, the incremental development of cracking at the bottom/west slab-column connection is charted in Photos 4.2 through 4.4. Damage to the slabs was limited to these regions, and no distress to the columns was evident.

Since no gravity load was applied to the slab, punching shear failure was not anticipated in the bare frame specimen. Drift levels as large as 4.0% were expected prior to experiencing punching failure (Fig. 2.2). Under typical design loading, however, the possibility of punching would have been a concern at the maximum drift levels reached during the testing.

Although the specimen was not tested to failure, the non-ductile reinforcing details did have some effect on the behavior of the bare frame subassembly. Because the bottom bars were not continuous through the column, the positive moment capacity of the slab at the connection was limited to the cracking strength of the concrete. In addition, the bottom reinforcement would have been unable to suspend the slab if punching failure was to occur. Since the shear strength of the columns was much greater than the demand
imposed by the lateral loading, large tie spacing had no effect on the behavior of specimen S1-BF. Column compression splices were also not a factor since the tensile forces in the columns was very low.

4.2.3 Specimen S1-RN

After testing specimen S1-BF it was retrofitted and retested as S1-RN. This assembly contained a recessed center panel and no connections to the surrounding frame. The anticipated behavior was expected to be similar to that of a masonry infill panel without any connections to the frame.

During the very first cycle of loading (0.25% overall drift) separation between the infill wall and the subassembly columns was observed. In each direction, the separation was visually verified following a loud popping noise that coincided with a top deflection of approximately ± 0.10 in. and a corresponding lateral load of about 3.2 kips. No other damage was witnessed at this stage. As the displacement was increased to 0.50% overall drift, separation occurred between the infill wall and the slabs. A loud noise also accompanied this detachment that occurred at a top displacement of roughly −0.42 in. and a horizontal load of −11.1 kips. At this point the wall was completely separated from the surrounding frame. Gaps at the unloaded corners were detectable and began to open up under increased loading. The development of the diagonal compression strut mechanism was noticeable in both directions of loading. Damage to the subassembly at this point did not extend beyond the frame-panel interface (Photo 4.5).

Under increasing deformations the infill panel initially began to crack in the recessed portion of the wall along the diagonal. Accompanied by an audible release of energy, the
first diagonal tension crack occurred during the cycle to 0.75% overall drift and was detected at a load of −12.5 kips and a top displacement of −0.52 in. The formation of the diagonal cracks was situated in a parallel direction to the loaded diagonal. The first diagonal crack in the opposite direction was noticed at a top displacement of 0.62 in. and a load of 14.2 kips. Crack propagation and new crack development was evident as the specimen was cycled under repeated loading. All of the damage to the infill was mainly confined to the recessed central portion of the infill (Photo 4.6). This was expected since the theoretical principle tensile stress was a maximum at the center of the panel and reduced as it moved outward. At the conclusion of the 0.75% overall drift level, the slab-column frame exhibited no signs of distress.

Similarly, the cycle to 1.0% overall drift produced more diagonal cracks in the central region, but no other apparent damage. At an overall drift of 1.5% the diagonal cracking began to propagate outside the recess, toward the corners of the panel. The main columns experienced flexural cracking that was concentrated in the upper half of the span. In addition, the lower portion of the west column exhibited a large X-shaped crack oriented at approximately 45° to the longitudinal axis (Photo 4.7). These cracks developed due to high concentrated shear forces being transmitted to the columns from the infill wall, a risk that demands proper use of infill panels. The large stirrup spacing in the columns did not adequately confine the concrete core and allowed the cracks to propagate unhindered. As a result, during the first negative cycle to 2.0% overall drift the column continued to deteriorate in this region (Photo 4.8). Excessive shear cracking led to further damage and eventual spalling of portions of the exterior cover. The ultimate failure, however, occurred as the specimen was cycled in the opposite (positive) direction. A tensile failure
of the lap splice at the base of the west column stopped the test as illustrated in Photo 4.9. This is another important concern in the use of infills, especially those not intentionally provided to resist lateral forces. A 3/4" gap existed between the slab and column/wall after the pull out of the longitudinal steel occurred (Photo 4.10). The reason the lap splice failed in tension can be attributed to the severe shear damage weakening the bond between reinforcing steel and concrete. The maximum load before the tension failure was 25.1 kips. Sketches detailing the progression of the cracking within the slabs, columns and infill wall are shown in Figs. 4.3 and 4.4.

4.2.4 Specimen S2-HC

Specimen S2-HC was only loaded to a maximum top displacement of ±0.42", corresponding to an overall drift of 0.50%. The test was stopped at this time so that the wall could be further weakened and retested. Photo 4.11 shows the condition of the subassembly prior to the application of any load. Aside from a number of shrinkage cracks in the grout, inspection at the conclusion of the cycles to 0.25% overall drift revealed no apparent damage to either the surrounding frame or the infill wall. During the subsequent three cycles (0.50% overall drift) minor separation between the columns and the wall at the loaded corners was observed. Separation between the slab and wall, however, was prevented because of the dowel connections at the top and bottom of the infill.

Cracking within the infill panel was insignificant, remaining practically unchanged from the initial three cycles of loading. The bounding frame, on the other hand, did experience some visible distress after loading to 0.50% overall drift (Photo 4.12). A few
minor flexural cracks developed in the main columns and in the top column stubs. In addition, both the top and bottom slabs cracked due to flexure at the slab-column joint. Extending the length of the slab along the interior face of the column, these cracks were similar to those that first formed in the bare frame specimen (S1-BF). Final cracking patterns for the plan view of the slabs and for the front elevation are presented in Figs. 4.5 and 4.6, respectively.

4.2.5 Specimen S2-HCA

After the conclusion of the testing for specimen S2-HC, the infill wall was weakened and the subassembly was retested as specimen S2-HCA. Additional 2 1/2" diameter holes were drilled throughout the wall in a uniform pattern. As a result, each standard panel contained three identical openings (Photo 4.13).

The first three cycles of loading at 0.25% overall drift did not produce any visible damage to the subassembly. Unlike specimen S2-HC, cracking developed in the infill wall during the cycles to 0.50% overall drift. All of the cracks originated from the circular openings and spread outward at approximately 45° angles. As the subassembly was loaded in the negative direction cracks formed parallel to the diagonal spanning from the lower west corner of the wall to the upper east corner. As the direction of loading was reversed the cracking pattern followed the opposite diagonal. Unlike specimen S1-RN, the damage was uniformly distributed throughout the entire area of the infill panel. The greatest amounts of distress were initially observed within the two middle stacks of standard panels. Of these eight panels, the four directly adjacent to the top and bottom slabs had the largest concentration of cracking. This indicated the transfer of shear forces
between the slab and infill wall through the dowel connections. Similarly, the outer stacks of standard panels experienced the same type of damage but to a lesser degree. The condition of the surrounding frame remained identical to that at the beginning of the testing history.

With the application of increasing displacement the cracking in the infill wall continued to propagate. Damage spread outward from the most highly stressed regions and eventually covered every individual infill panel (Photos 4.14 through 4.16). The final cracking pattern was extensive, but it did not lead to a failure of the infill wall (Photos 4.17 and 4.18). Instead, the ultimate failure mechanism of the specimen was governed by the shear capacity of the west column. After reaching 2.0% overall drift and a maximum load of −33.7 kips, shear cracks were detected on both of the main columns. At this point the top column stubs were significantly damaged due to the transmission of high shear forces and bending moments (Photo 4.19). Excessive cracking reduced the lateral stiffness of the top stubs, but they were still capable of resisting increased horizontal loads. During the final loading cycle (2.5% overall drift) the lower portion of the west column exhibited further shear damage similar to that of specimen S1-RN (Photo 4.20). It was decided to terminate the test at this time because failure of the column was imminent. The maximum load reached in this cycle was −33.3 kips, lower than the previous peak. Details of the final slab cracking are given in Fig. 4.7. Furthermore, cumulative visual damage as viewed from the front elevation is provided in Fig. 4.8 for overall drift levels up to 1.0%.
4.2.6 Specimen S3-OC

Similar to specimen S2-HCA, the infill wall for specimen S3-OC also consisted of perforated panels. Instead of circular holes however, rectangular openings 5" long and 4" high were uniformly distributed throughout the panel. Each standard LWPSC panel contained two such block-outs symmetrically located about both the vertical and horizontal axis (Photo 4.21). Aside from the different type of openings, all other test parameters between Specimen S2-HCA and S3-OC were identical.

During the very first cycle of loading separation between the columns and the infill wall occurred. As in specimen S2-HCA, this de-bonding at the boundary occurred at a top deflection of about 0.10 in. and a corresponding horizontal load of 3.5 kips. Contact between the infill panel and the slabs, however, was maintained because of the presence of the dowel connections. Damage at this stage was limited to some minor hairline cracking within the infill wall. All of the cracks emanated from the corners of the rectangular openings and propagated diagonally. The cracks were confined mainly to the middle two tiers of panels and were typically less than an inch in length.

Distress to the surrounding frame was first visible during the displacement to the overall drift level of 0.50%. Flexural cracks across the length of each slab at the interior face of the columns were observed after the second cycle, and corresponded to a top displacement of approximately 0.42" and a lateral load of 11.2 kips. In addition to the slab's damage, the infill panel also began to develop more substantial cracking at this stage. Diagonal cracks formed between the rectangular openings as a result of horizontal shear forces. Under reversed loading, an X-shaped pattern developed at these vertical piers, initiating from the centrally located panels and moving outward. The overall
behavior of the infill wall was similar to that of a coupled shear wall with relatively stiff spandrels and flexible piers. During the following three cycles of deformation (0.75% overall drift) the localized diagonal cracking continued to spread throughout the entire wall. Photo 4.22 illustrates the cracking pattern up to this point. Except for the segments adjacent to the columns, the entire row of vertical piers at the second level from the top was completely cracked. The maximum load corresponding to this point was 15.8 kips in the negative direction and 16.2 kips in the positive. Because no horizontal reinforcement was provided in these short fixed-ended piers, the shear capacity was limited to the shear strength of the LWSPC. Aside from a few flexural cracks in the east column, no other evidence of damage was recorded.

At an overall drift of 1.0% (Photo 4.23) cracking in the aforementioned row of piers spread to the joints between the wall and the columns. Because no horizontal reinforcement was present in the piers, once the entire row was cracked the top portion of the infill wall could move relative to the bottom. This was verified by inspection of the specimen as it was further loaded. Under deformation in the negative direction the top portion of the infill wall was bearing against the west column and a gap existed at the east column. The bottom section of the wall, on the other hand, was in contact with the east column and a separation developed at the boundary with the west column. The maximum load at this point remained fairly consistent with that of the previous cycles. Under repeat cycles, the X-shaped cracks began to open up to widths of 1/8" to 3/8". Concrete adjacent to the openings and bounded by the X-cracking began to deteriorate. Spalling of the cover in these regions was, therefore, initiated. The presence of vertical steel reinforcing hoops at the edge of each opening provided some confinement for the LWSPC as it
began to break up. The state of the surrounding frame remained fairly intact from the previous cycles of loading. Several additional flexural cracks extended within the slab-column joint area. Some flexural cracking was observed in the west column, but overall the condition of the main portion of the columns appeared sound.

The maximum load for the subassembly occurred during the 1.5% overall drift level (Photos 4.24 & 4.25) and was recorded as 17.7 kips in the negative direction and 17.6 kips in the positive direction. Further horizontal deflection to 2.0% and 2.5% overall drift maintained similar, but decreasing, loads. From 1.5% to 2.5% overall drift the maximum load decreased 14.6% for the negative cycle (17.7 kips to 15.1 kips) and 11.3% for the positive (17.6 kips to 15.6 kips). Damage to the excessively cracked row of piers progressively worsened until a large percentage of the concrete had crumbled and spalled off. After single cycles to 2.0% and 2.5% overall drift, loss of concrete cover exposed much of the reinforcing steel in this region. X-shaped shear cracking eventually propagated to all the vertical piers within the infill wall as shown (Photos 4.26 through 4.28). As the loading increased, deterioration progressed in a comparable manner to that discussed earlier. Unlike specimens S1-RN and S2-HCA, the main columns remained structurally sound. Providing better control over the infill wall capacity eliminated brittle column shear failure. Of the tested configurations, infill panels with rectangular perforations were, therefore, the most reliable and promising system.

Two shear cracks on the west column were noticed after the cycle to 1.5% overall drift. These cracks, however, did not develop further during subsequent cycles. Some additional flexural cracking was also seen on both columns. Furthermore, the top and bottom column stubs did not exhibit any considerable signs of distress. Both slabs
continued to experience flexural cracking as the deformations increased. The breaks extended across the width of the slab and began migrating away from the slab-column joint and closer to the center as shown in Fig. 4.9. Cracking to the front of the infill wall and surrounding frame is tracked for overall drift levels up to 1.0% in Fig. 4.10.

4.2.7 Specimen S4-OCX

Specimen S4-OCX was identical to specimen S3-OC (Photo 4.29) except for the addition of diagonal reinforcement located between the rectangular openings in each panel. Reinforcement was included to control relative movement between the top and bottom portions of the infill wall as seen in specimen S3-OC. The intent was to provide a force transfer mechanism after shear cracking of the individual vertical piers.

During the first stage of loading, 0.25% overall drift, very little cracking occurred within the infill wall. Horizontal cracking in the vertical grout strips and a few small cracks emanating from the corners of the openings were noted. Separation between the column and infill wall was observed, but tight contact remained. The first cracks in the flat-plate frame occurred in the top slab at both slab-column connections. Each connection region was fractured through the width of the slab at the interior face of the column. Continuation of loading to 0.50% overall drift increased the amount and size of cracks originating from the opening corners. In addition, a few diagonal cracks in the vertical piers between openings also developed. Most of the damage was located in the middle half of the wall (Photo 4.30). Similar to the top slab at 0.25% drift, the bottom slab experienced flexural cracking at this level of displacement. No additional distress to the surrounding frame was noted.
Cracking within the infill wall became more significant during the cycles to 0.75% overall drift (Photo 4.31). Unlike specimen S3-OC, however, the cracking was not predominately concentrated in the vertical piers between openings. Instead, the cracking was more uniformly distributed, starting from the central region of the wall and moving outward. The additional diagonal reinforcement between openings strengthened the piers and provided a clamping force once cracking did occur. For these reasons, damage was not concentrated solely within the piers. The first signs of distress to the columns were observed at this level. Flexural cracks developed on both columns in the upper half of the members. Behavior during the 1.0% overall drift phase was comparable to that of 0.75% with even a greater distribution of cracking (Photo 4.32). At this point, contact between the infill and columns was still maintained and no shear failure of the infill was witnessed.

In the course of the cycle to 1.5% overall drift, the same row of piers that failed in specimen S3-OC had diagonal cracking in every pier except those adjacent to the columns (Photos 4.33 & 4.34). This state of damage coincided with a top displacement of ±0.94" and a lateral load of approximately 20 kips. With further deformation to the subassembly the diagonal cracking spread to other regions of the panel. Because of the diagonal reinforcement in the piers, shear cracking throughout this horizontal level did not immediately result in a large relative displacement between the upper and lower portions of the wall. Cracking also continued to form outside of the piers mostly inclined parallel to the diagonal of the wall. Unlike specimen S3-OC, fall out and excessive crushing of the concrete did not occur at this time. The overall maximum loads of −23.0 kips and 22.6 kips were reached during this cycle and coincided with top displacements
of -1.15" and 1.25" respectively. No significant additional damage was noticed in the main span of the columns. Both slabs, however, experienced further cracking within the slabs. Cracks in the bottom slab were similar to those formed throughout the previous cycles. In addition to this type of cracking, the top slab also cracked in the interior of the span near the location of the outside two connections to the wall.

Damage within the infill wall continued with the cycles to 2.0% and 2.5% overall drift as shown in Photos 4.35 & 4.36. The gap between the columns and infill panels became more pronounced, but not as large as that in specimen S3-OC. As the specimen was loaded in the negative direction, the separation developed in the upper east corner and the lower west corner. This behavior was directly opposite when the loading direction was reversed. Existing cracks in the infill wall widened and new cracks continued to spread. Some minor spalling of the cover concrete within horizontal piers and above some openings was observed. Some localized crushing of concrete was also witnessed at the corners of the openings. The overall distress to the infill wall was not as severe as that of specimen S3-OC, and can be attributed to the addition of diagonal reinforcement. The reinforcement tied the wall together after shear cracking and accounted for the development of a larger number of uniformly dispersed cracks. Cracks in the bottom slab propagated in the same manner as those of the top slab during the 1.5% overall drift level (Fig. 4.11). The columns did not experience any shear cracking and remained structurally sound. Complete failure of the subassembly was, therefore, never achieved. Although the test was terminated, it did reach a maximum interstory drift level greater than 1.5%, a practical limit for punching shear failure of flat-plate structures. A detailed drawing of the crack propagation for the first four stages of overall drift is presented in Fig. 4.12.
4.3 Measured Experimental Test Results

4.3.1 Cyclic Load – Displacement Plots

4.3.1.1 General

Cyclic load-deflection plots were generated for each specimen using data gathered from three LVDTs and the load cell connected to the hydraulic actuator. The top deflection, $\delta_{\text{top}}$, vs. lateral load compared the overall displacement measured at the actuator with the applied horizontal load. From these graphs the overall specimen behavior, including that of the column stubs, was illustrated. In addition, LVDTs located at each floor level monitored the relative floor displacement. Using this data, plots of relative floor displacement, $\delta_{\text{rel}}$, vs. lateral load and interstory drift, $\Delta_{\text{id}}$, vs. lateral load were produced. Since large overall deformations could result from localized damage to the column stubs, relative slab displacement was considered the most insightful method of comparing the contribution of the infill panel to the subassembly behavior.

The lateral load-top displacement plots for all of the specimens are shown in Figs. 4.13 through 4.18, and the lateral load-interstory drift plots in Figs. 4.20 through 4.25. For easier comparison of all the specimens, Figs. 4.19 and 4.26 provide the load-displacement relationships in envelope form. Maximum load points occurring in the first cycle of each deflection level were connected to generate the above envelopes. In addition, Table 4.2 provides a comparison of the absolute peak load reached in each specimen. Deflection and corresponding loads to the west were represented as negative (push) and those to the east as positive (pull). For deformation levels of 0.25%, 0.50%, 0.75% and 1.0% overall drift the subassemblies were subjected to three fully reversed
cycles. Data describing the load, stiffness and energy dissipation degradation under repeated loading was, therefore, also obtained from the load-displacement plots. A full discussion of these findings is given in Section 4.6, Repeated Drift Cycles and Residual Stiffness.

Table 4.2

Comparison of Maximum Loads

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Negative Direction (West)</th>
<th>Positive Direction (East)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Load, Load (Specimen)</td>
<td>Peak Load, Load (Specimen)</td>
</tr>
<tr>
<td></td>
<td>(kips) Load (S1-BF)</td>
<td>(kips) Load (S1-BF)</td>
</tr>
<tr>
<td>S1-BF</td>
<td>-3.5 1.0</td>
<td>3.6 1.0</td>
</tr>
<tr>
<td>S1-RN</td>
<td>-25.5 7.2</td>
<td>25.1 6.9</td>
</tr>
<tr>
<td>S2-HCA</td>
<td>-33.7 9.5</td>
<td>32.8 9.0</td>
</tr>
<tr>
<td>S3-OC</td>
<td>-17.7 5.0</td>
<td>17.6 4.8</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>-23.0 6.5</td>
<td>22.6 6.2</td>
</tr>
</tbody>
</table>

As a general trend the load-deflection loops for all of the specimens, except S2-HC and S2-HCA, were symmetric in both loading directions. Specimen S2-HC and S2-HCA exhibited some reduction in strength and stiffness when loaded in the positive (east) direction. As the loading increased, however, this difference became negligible. The observed behavior was the result of an unanticipated actuator retraction prior to the start of the actual test, and not for any structural inconsistencies in the subassembly.

In addition, damage sustained in the first half of any cycle (negative direction) typically did not affect the behavior of the subassembly during the second half cycle (positive direction). All of the retrofitted subassemblies increased the stiffness and ultimate strength over that of the bare slab-column frame. Specific performance,
however, was significantly varied as illustrated by the lateral load-interstory drift envelopes in Fig. 4.26. The following discussion provides a detailed description of the individual load-deflection behavior of each specimen.

### 4.3.1.2 Specimen S1-BF

The bare frame subassembly, specimen S1-BF, behaved in a manner consistent with results found for past tests conducted on slab-column connections [3, 11]. Because of the geometry of the test specimen, values for the overall drift and the interstory drift for specimen S1-BF were nearly identical. As a result, both of the lateral load-displacement plots had similar shape and characteristics (Figs. 4.13 & 4.20). The envelopes of the hysteresis loops in each direction were approximately bilinear (Figs. 4.19 & 4.26) and provided the lower bound for all the specimens tested. Deviation from the initial linear-elastic portion of the curve occurred at a load of 2.1 kips and an interstory drift of 0.19% (2.0 kips and 0.21% in the opposite direction). The decrease in stiffness can be attributed to flexural cracking in the slab observed during the first cycle to 0.50% overall drift. From this point, the envelope had a gradually increasing slope up to the peak load of -3.5 kips and the maximum interstory drift of -1.26% (3.5 kips and 1.28% in the opposite direction). For reasons discussed earlier, the test was terminated at this point even though the load resisting capacity was still increasing.

The severely pinched hysteresis loops indicated a substantial loss of stiffness as well as low energy dissipation capacity. Under repeated loading at the same drift level the loops exhibited considerable degradation of maximum load and energy dissipation. The percentage decrease was most pronounced between the first and second cycles. Although
the third cycle had the greatest amount of degradation, the behavior of the second and third cycles was very similar. Section 4.6 provides comprehensive data on the degradation under repeated cycles for the first four stages of loading.

4.3.1.3 Specimen S1-RN

Unlike specimen S1-BF, specimen S1-RN behaved in a nearly elastic manner. The maximum load-displacement envelope (for both overall drift and interstory drift) was approximately linear up to a peak load of ±24.3 kips (Figs. 4.19 & 4.26). This load corresponded to a top deflection of ±1.24" and interstory drifts of -0.46% and 0.52%. After this point, there was little increase in load during the cycle to 2.0% overall drift. The apparent yielding of the specimen was due to severe shear cracking in the bottom portion of the west column. Another minor change in stiffness was observed in the lateral load vs. interstory drift plot during the first cycle to -0.50% overall drift (Fig. 4.21). Since there were no connections between the existing frame and the infill panels, this softening of the system was attributable to de-bonding at the frame-wall interface. After the complete separation of the infill from the frame, the LWPSC wall behaved similar to a masonry wall. Ultimately, a diagonal compression strut mechanism formed and the stiffness of the retrofitted subassembly rebounded.

Pinching of the hysteresis loops at later stages in the loading history was attributed to the shear deformation of the wall as the direction of the load was reversed. Low stiffness resulted as the points of contact between the infill wall and the surrounding frame were readjusted. Once the loaded corners were fully reengaged and the compression strut developed in the opposite direction, the stiffness of the subassembly increased to a value
consistent with the previous cycle. The termination of the final hysteresis loop in the first quadrant of the load-deflection plots represented tension failure in the west column. No additional lateral load could be applied due to the brittle nature of the rupture.

4.3.1.4 Specimen S2-HC

Unlike specimen S1-RN, specimen S2-HC had dowel connections between the infill wall and the frame. The strength and stiffness of the subassembly was believed to closely approximate that of a slab-column frame retrofitted with a solid infill wall. Although sixteen 2 1/2" diameter holes were provided in the center portion of the infill, the ratio of openings to total wall area was deemed low. The load-displacement curves for specimen S2-HC were all very linear with the loading and unloading portions of each curve practically coincident. After observing the behavior of the specimen through an overall drift level of 0.50%, the wall was determined to be too strong for the surrounding frame. At this point, it was decided to further weaken the infill panels and retest the subassembly. Yielding or failure of the wall, therefore, was never achieved. The maximum loads reached were -15.7 kips and 11.4 kips at an interstory drift of ±0.09%. The difference in values for the positive and negative directions was the result of an unexpected application of load prior to the beginning of the actual test. Actuator movement in the positive (east) direction may have initially damaged the subassembly, although no visible signs of distress were noticed. Very little degradation of the stiffness or maximum load occurred during repeated cycles to the same deformation level.
4.3.1.5 Specimen S2-HCA

After the inclusion of additional holes, specimen S2-HC was retested as specimen S2-HCA and subjected to the entire loading history. Similar to specimen S1-RN, the force-displacement relationship and the envelope for the maximum load-interstory drift of specimen S2-HCA was linear for the duration of the test. Hysteresis curves for the top displacement and the corresponding envelope, on the other hand, exhibited yielding after cycles to 1.0% overall drift. Both envelopes for S2-HCA formed the upper bounds of the curves for all the specimens (Figs. 4.19 & 4.26). The difference in the plots of the top displacement and the relative floor displacement was attributed to severe damage in the column stubs. Shear and flexural cracking, especially within the top stubs, contributed to large localized deformations. Since the horizontal load capacity of each column stub was 20 kips, the subassembly was capable of resisting greater forces even after the apparent yielding. Increased loads, however, were accompanied by a decrease in the slope of the segments connecting consecutive points on the top displacement envelope. With this knowledge, the most accurate description of the behavior of the main portion of the frame and infill wall was the data relating lateral load to the relative floor displacement. The plots of the relative floor displacement eliminated the contribution of the stubs, and provided a more appropriate standard on which to compare the performance of different specimens. Specimen S2-HCA reached a peak load of −33.7 kips (32.8 kips in the opposite direction).
4.3.1.6 Specimen S3-OC

Different than the aforementioned specimens, the load-displacement envelopes for specimen S3-OC were approximately bi-linear with a fairly well defined yield point (Figs. 4.19 & 4.26). Specimen S3-OC exhibited nearly linear behavior for the first three levels of loading (through 0.75% overall drift). Each consecutive segment of the load-displacement envelopes, however, did possess a small reduction in slope. The maximum loads reached during the initial linear segment were -15.8 kips and 16.2 kips at interstory drifts of -0.29% and 0.26% respectively. During the following cycle, to 1.0% overall drift, the specimen yielded before reaching the peak loads attained in the previous stage. As the specimen was pushed to 1.5% overall drift, the maximum loads of the entire displacement history, -17.7 kips and 17.6 kips, were obtained. The maximum loads for the final two cycles, 2.0% and 2.5% overall drift, were slightly less than the absolute peak. The largest interstory drift reached prior to stopping the test was -1.73% in the negative direction and 1.81% in the positive direction.

4.3.1.7 Specimen S4-OCX

Similar to specimen S3-OC, the maximum load-displacement envelopes for specimen S4-OCX were also bi-linear (Figs. 4.19 & 4.26). Although the general shape of the curves was similar, the additional panel reinforcement in specimen S4-OCX did have an effect on the subassembly’s stiffness, apparent yield point and peak load. The subassembly maintained the initial portion of the bi-linear curve until an overall drift of 1.0%. Yielding of the specimen occurred at a load of -20.6 kips and interstory drift of -0.36% in the negative direction and 20.4 kips and 0.35% in the positive direction. The absolute peak
loads of -23.0 kips and 22.6 kips occurred in the cycle to 1.5% drift. Similar to specimen S3-OC, the second segment of the bi-linear relationship had a downward slope after the maximum load. The largest interstory drift reached prior to stopping the test was 1.66% in the negative direction and 1.65% in the positive direction.

4.3.2 Shear Strain Plots for Infill Wall

Wire-type LVDTs located in the bottom corners of the infill panels and spanning across the wall diagonals were used to determine the shear strain in the wall. Strain readings for both wire-type LVDTs were taken at the same time as all the other transducers. Using the acquired data, the infill wall shear strain was calculated by Eq. (3.8). Complete histories of the shear strain vs. overall lateral load for each retrofitted specimen were plotted and are presented in Figs. 4.27 through 4.33. These plots determined the isolated performance of the infill wall. The shapes of these curves were similar to the lateral load-interstory drift plots, because the infill panels dominated the behavior of the subassembly.

In order to better compare the shear strain results, the shear modulus of elasticity, \( G \), was estimated for each infill wall. The nominal infill shear stress was obtained by dividing the applied lateral force by the gross area (3" x 88" = 264") of the wall. Reduction in area due to recessed panels or perforations were not included in the stress calculations. The shear modulus was determined by dividing the shear stress by the measured shear strain. Table 4.3 provides the elastic shear modulus for each specimen.

Specimens S2-HC and S3-OC represented the upper and lower bounds, respectively, for all of the subassemblies. As illustrated in Table 4.3, increasing the amount of circular
openings decreased the elastic shear stiffness by about 43% (see values for specimens S2-HC & S2-HCA). Although not connected to the frame, specimen S1-RN had a shear modulus of elasticity greater than specimen S2-HCA. Once the wall separated from the frame, however, the diagonal strut mechanism developed and the wall stiffness was reduced. In addition, the inclusion of diagonal reinforcing in panels with rectangular perforations increased the shear modulus. The value for specimen S4-OCX was approximately 26% greater than that for specimen S3-OC.

Table 4.3

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear Modulus of Elasticity (ksi)</th>
</tr>
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<tbody>
<tr>
<td>S1-RN</td>
<td>202.5</td>
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<tr>
<td>S2-HC</td>
<td>268.4</td>
</tr>
<tr>
<td>S2-HCA</td>
<td>153.8</td>
</tr>
<tr>
<td>S3-OC</td>
<td>84.8</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>107.2</td>
</tr>
</tbody>
</table>

4.3.3 Strain Gage Histories

4.3.3.1 General

Strain gages were attached to the reinforcing steel of the slab-column frame in twenty-four locations. Curves of strain vs. overall lateral load for each strain gage were generated and are provided in Figs. 4.34 through 4.39. Each plot represented the entire testing history for the given subassembly. The experimentally determined yield strain was also shown for any gages approaching or exceeding this value. A schematic diagram next to
each plot shows the location of the gage within the slab-column subassembly. Furthermore, a more detailed description of the strain gage placement is presented in Figs. 3.3, 3.4 and 3.32. Before each test internal shunt calibration was used to verify proper strain gage bridge configuration. In addition, initial voltage measurements were taken for all channels and used to balance the bridges.

As illustrated in Figs. 4.34 through Fig. 4.39, not all of the channels gave reliable results. Although great care was taken in applying the strain gages, a number of reasons can account for inaccurate readings. Imperfections in the protective coating, damage during the pouring of concrete and loss of adhesion between the strain gage and rebar were all possible causes that, unfortunately, were extremely difficult to entirely eliminate. It should be noted, however, that even with intact gages any data of this type is only of qualitative value. Regardless of perceived performance, all of the strain gage results were included in the aforementioned plots. A more detailed description of the strain gage behavior follows.

4.3.3.2 Discussion

Gages #1 through #16 were applied on the slab reinforcement at the slab-column connections. Each connection had four gages, two placed on the top steel and two on the bottom. All of the bottom gages were on #3 rebar that ran into the column, but terminated 1 1/4" from the column centerline. Similarly, all of the gages at the top of the slab were also situated on #3 rebar. Gages #1, #5, #9 and #13 were located on the bar at the centerline of the column, and gages #2, #6, #10 and #14 on the bar offset 1 1/2" from the
edge of the column. This series of gages was included to provide a qualitative assessment of the slab behavior at the connections.

The general shape of the lateral load-strain plots for the slab reinforcement was similar for all of the specimens. The individual location of each gage determined whether the tensile strains were associated with positive or negative horizontal displacements. Typically, compressive strains in the slab steel were very small. Some of the steel reached its yield strain as can be seen in Figs. 4.34 through 4.39. The strain histories for all of the specimens were fairly consistent. This was expected since the observed slab cracking was similar for all of the tested subassemblies.

In addition, the east column reinforcement was also instrumented. Four gages, #17 – #20, were placed on the longitudinal steel (#6 bars) and four, #21 – #24, on the column ties (#2 smooth bars). Of all the gages, the ones located on the #6 rebar gave the most reliable results. None of these gages appeared to have malfunctioned during the testing. The reason these gages performed better than the others was related to the relative size of the rebar. Since the same strain gage was used for all diameters of steel, the circumference of the #6 bar provided a larger and flatter surface for application. Tensile strains in the column reinforcement were produced due to a combination of both axial load and moment. A majority of the longitudinal steel in specimens S1-RN and S2-HCA experienced strains in excess of the yield strain. Conversely, the strains in specimens S3-OC and S4-OCX were typically below yield. The greater capacity of the infill wall in specimens S1-RN and S2-HCA generated larger moments and axial forces in the column to account for this difference. Based on the strain history data, the deflected shape of each slab-column frame was similar.
Unlike gages #17 – #20, the gages on the smooth #2 ties did not produce very dependable results for a number of reasons. First, as previously mentioned, the small diameter of the #2 bars was much more difficult to work with as compared with the #6 bars. Many of these gages may not have bonded properly due to the high transverse curvature of the steel. Secondly, since the shear forces in the columns were small, the strains in the ties were also small. Tracking such slight deviations in strain is not accurate with this type of measurement device. The largest strains in the hoop steel occurred in specimens S1-RN and S2-HCA. On the other hand, the tie strains of specimens S3-OC and S4-OCX were negligible. These results were consistent with the visible shear cracking and observed subassembly behavior.

4.4 Energy Dissipation

4.4.1 General

When evaluating the inelastic response of a structure under reversed loading, energy dissipation capacity is an important parameter to consider. The energy expelled during a complete cycle is defined as the area within the corresponding load-displacement curve. To maximize the amount of energy released, it is most desirable to have load-displacement loops that are as open and full as possible.

Energy dissipation capacity was found for each specimen from the cyclic plots of both lateral load vs. top displacement and lateral load vs. relative displacement. The amount of energy dissipated was determined for each half cycle of loading by calculating the area within the hysteretic loop. A linear variation was used to interpolate between recorded data points. Results for each half cycle of loading are presented in bar graph format in
Figs. 4.40 through 4.51. In addition to the data for each half cycle, these figures also provide the cumulative energy dissipation per specimen. For easier comparison of all specimens, Figs. 4.52 and 4.53 plot all of the cumulative energy dissipation curves on the same graph.

4.4.2 Discussion

The bare frame specimen had very low energy dissipation as evidenced by the severely pinched hysteresis loops. Energy dissipation occurred at the slab-column connections, predominately due to cracking within the slab. Specimen S1-BF represented the lower bound on the plots of cumulative energy dissipation per cycle (calculated from the lateral load-top displacement curves) for all specimens. On the charts composed from the plots of lateral load vs. relative floor displacement, however, specimen S2-HCA was below the bare frame. Relative strength between infill wall and slab-column frame accounted for this difference.

Although specimen S2-HCA experienced a substantial amount of cracking within the infill panels, the excessive strength of the wall forced an eventual column failure. Uniformly distributed cracking did dissipate some energy, but the overall behavior of the infill was fairly linear. The strength of the infill wall allowed the subassembly to resist the largest horizontal loads of all the specimens. As a result, the column stubs experienced a considerable amount of damage that eventually dissipated a majority of the system's energy. For this reason, specimen S2-HCA formed both the lower bound for the cumulative energy dissipated based on the relative floor displacement and the upper bound for the same plots generated from the data for top displacement. Similarly,
specimen S2-HC remained nearly elastic during the duration of its loading. Very little energy was dissipated, as would be expected since the test was terminated prior to the subassembly being significantly damaged.

Of all the retrofitted specimens tested, specimen S1-RN had the lowest cumulative energy dissipation based on the plots of top displacement. Diagonal cracking, especially within the recessed center panel, provided one mechanism for energy dissipation. Because the infill panels were not connected to the surrounding frame, however, high concentrated shear forces damaged the main columns prior to failure of the wall. During cycles to 1.5% and above, shear and flexural cracking in the columns greatly contributed to the amount of dissipated energy.

The infill panels used to retrofit subassemblies S3 and S4 dissipated the most energy of all the specimens as illustrated in Fig. 4.53. Because the shape of the load-displacement plots for specimen S3-OC and S4-OCX were very similar the energy dissipation characteristics were also comparable. Uniform cracking throughout the infill wall dissipated the majority of the energy. Most of the cracking in specimen S3-OC was concentrated between rectangular openings forming X-shaped diagonal shear cracks. Damage spread across the width of the wall at 1.0% overall drift resulting in shear failure of the wall and large fuller hysteresis loops. Some pinching of the hysteresis loops at greater drift levels occurred due to shear slippage. On the other hand, diagonal reinforcement placed between openings in specimen S4-OCX better controlled the localized damage across the width of the wall.
4.5 Stiffness

4.5.1 General

The actual stiffness of a specimen for any given cycle was difficult to determine because it continuously changed as the subassembly was damaged and the loading progressed. For this reason, the peak-to-peak (secant) stiffness, as proposed by other authors, was determined to be a practical substitution of the true stiffness [51]. The peak-to-peak stiffness for each cycle was found by computing the slope of the line connecting the maximum point in the positive direction with that in the negative. Secant stiffness values for specimens S2-HC and S2-HCA, on the other hand, were determined only from the negative cycles. The positive cycles were not evaluated because an unanticipated movement of the actuator before the testing was believed to have softened the system in the positive direction. The resulting difference in secant stiffness was largest in the earlier cycles, but disappeared at larger drift levels. From this point, all references to stiffness will be understood to represent the peak-to-peak stiffness unless otherwise noted.

Two assessments of stiffness were determined from the acquired data. Based on plots of lateral load vs. $\delta_{iop}$ and lateral load vs. $\delta_{rel}$, an overall stiffness, $K_{iop}$, and an interstory stiffness, $K_{rel}$, respectively, were calculated. Initial stiffness values for all of the specimens, calculated as the peak-to-peak stiffness of the first cycle to 0.25% overall drift, are presented in Table 4.4. As illustrated, all of the specimens exhibited a significant increase in stiffness over that of the bare frame. The actual amount of increase for interstory stiffness varied from 890% to 1640%. Bar graphs are provided in Figs. 4.54 through 4.59 that track both the overall and interstory stiffness of each specimen.
throughout the loading history. In addition, Figs. 4.60 and 4.61 monitor the stiffness degradation under increasing drift.

Table 4.4

Comparison of Initial Peak-to-Peak Stiffness

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Lateral Load vs. $\delta_{top}$</th>
<th>Lateral Load vs. $\delta_{rel}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial Stiffness, $K_{top}$ (kips/in)</td>
<td>$K_{top}$ (Specimen) $\frac{K_{top}}{K_{top}}$ (S1-BF)</td>
</tr>
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<td>S2-HCA</td>
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<td>3.0</td>
</tr>
<tr>
<td>S3-OC</td>
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<td>2.9</td>
</tr>
<tr>
<td>S4-OCTX</td>
<td>35.2</td>
<td>3.0</td>
</tr>
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</table>

4.5.2 Discussion

Regardless of the type of infill panel tested, initial overall stiffness values were all very similar. The smallest initial stiffness, 31.6 kips/in (specimen S1-RN), was only 16.4% less than the largest initial stiffness, 37.8 kips/in (specimen S2-HC). In addition, the degradation of overall stiffness per cycle (Fig. 4.60) was also fairly consistent. Small differences attributable to the relative strengths of the infill panels were observed between the retrofitted specimens. Overall stiffness, however, was predominately influenced by the configuration of the test subassembly, and not the strength of the infill wall. Because the load was applied to the top of the subassembly, damage to the column
stubs had a substantial effect on the overall stiffness of the specimen. The resulting values were, therefore, fairly independent of the characteristics of the infill panels.

Interstory stiffness, on the other hand, provided a more concise picture of the true improvement in subassembly behavior with the addition of each infill wall. As shown in Table 4.4, all of the specimens achieved substantial increases in the interstory initial stiffness. Specimen S2-HC, which closely approximated the behavior of a solid panel, had an interstory stiffness of 355.2 kips/in. This value was 16.4 times that of the bare frame and represented the maximum for all of the specimens. Specimen S3-OC had the lowest initial interstory stiffness, 192.9 kips/in, of all the retrofitted subassemblies. Although only 54% of specimen S2-HC, the initial interstory stiffness of specimen S3-OC was still 8.9 times that of the bare frame.

Fig. 4.61 shows the interstory stiffness degradation throughout the loading history of each specimen. Specimen S2-HC formed the upper bound and specimen S1-BF the lower bound for all of the curves. As anticipated, the bare frame subassembly was extremely flexible. Because no other lateral load resisting elements were provided, the moment resistance of the slab-column connections controlled the lateral stiffness. The largest decrease in stiffness for specimen S1-BF occurred between the cycles of 0.25% and 0.50% overall drifts. Cracking within the slab at the slab-column joint accounted for the relatively large initial reduction. The initial cracking extended the entire width of the slab at the interior face of the column (Fig. 4.1) and was due to flexure. Under increased deformation, the connection region experienced additional damage that lead to further decline in the lateral stiffness. Diagonal torsion cracks originated from the sides of the columns and propagated to the edge of the slabs. In addition, more flexural cracks formed
and spread toward the center of the span. All of these cracks affected the behavior of the connection throughout the loading history. The most dominant cracks, however, were those located immediately adjacent to the column.

Unlike specimen S1-BF, cracking in the slab did not have a noticeable effect on the interstory stiffness of specimens S2-HC or S2-HCA. Throughout the two cycles of loading specimen S2-HC was subjected to, the interstory stiffness remained constant. During the first three stages of loading for specimen S2-HCA the interstory stiffness actually increased. After this point, however, the interstory stiffness decreased at a fairly constant slope. The loss in stiffness was the result of both cracking within the infill panel and damage to the surrounding frame; particularly shear distress in the columns.

Although the initial interstory stiffness and rate of degradation between specimen S1-RN and specimens S3-OC & S4-OCX were comparable, this was purely coincidental. Since the infill wall of specimen S1-RN was not attached to the surrounding frame, it was predicted to behave as a diagonal compression strut. The initial interstory stiffness, however, was representative of the system prior to separation at the frame-wall interface. A 37% decrease in stiffness between cycles to 0.25% and 0.50% overall drift occurred due to complete separation between the infill and the flat-plate frame. At this point, the stiffness was controlled by braced frame action. Stiffness degradation continued after the strut mechanism developed, but at a much slower rate.

As illustrated in Fig 4.61, the shape of the interstory stiffness degradation curves for specimens S3-OC and S4-OCX were nearly identical. Stiffness values for specimen S4-OCX were greater than specimen S3-OC at every level of interstory drift. Additional diagonal reinforcement in the piers of the infill panels accounted for this difference. The
interstory stiffness of both specimens degraded at the same rate up until yielding of the infill wall. After yielding, the rate of degradation was much smaller. Loss of stiffness was mainly the result of damage to the individual piers within the infill panels.

4.6 Repeated Drift Cycles and Residual Stiffness

All of the specimens were subjected to repeat cycles during overall drift levels up to and including 1.0%. At each stage three fully reversed cycles were completed in order to investigate the degradation of peak load, stiffness and energy dissipation. The results for all specimens are given in Table 4.5 through Table 4.7. After the last cycle to 1.0% overall drift, three reduced drift cycles (one each at 0.75%, 0.50% and 0.25%) were performed. These cycles were included to measure the residual stiffness, strength and energy dissipation capacity once damage had occurred. Comparison of the residual lateral load-displacement plots with the initial behavior is presented in Figs. 4.62 through 4.66.

As a general trend, the peak loads for repeated cycles to the same drift level were not significantly reduced (Figs. 4.67 through 4.69). The largest relative difference existed between the first and second cycles. Degradation between the second and third cycles was typically negligible. During the cycles to 0.25% and 0.50% overall drift, specimen S1-BF had the greatest amount of degradation of all the specimens. In the second cycle of loading to 0.25% overall drift the peak load degradation was 18.8%. Similarly, the second cycle to −0.50% overall drift had a degradation of 14.8%. These high percentages can be contributed to the onset of flexural cracking within the slab at the slab-column joint. Conversely, because of the high in-plane strength of the infill wall, slab cracking had a negligible effect on the maximum loads of the retrofitted specimens. All of the peak
load degradation values were less than 10% except during the cycles to 1.0% overall drift for specimen S3-OC. Reductions of 18.7% in the negative direction and 11.9% in the positive direction were observed in the third cycle. It was found that this loss of strength in specimen S3-OC corresponded with the apparent yielding of the infill wall.

Unlike the maximum loads, energy dissipation degradation under repeated cycles was more severe. Typically, the initial cycle to a given drift level was fuller and more open than the subsequent cycles. New damage to the specimen predominately occurred during the first cycles, hence allowing the greatest release of energy at this time. The reloading portion of cycles two and three closely followed the path of the unloading segment of cycle one. Comparable to peak load degradation, the percentage of energy dissipation degradation was usually largest between cycles one and two. The hysteresis loops for cycles two and three generally had a consistent shape, and therefore expelled similar amounts of energy.

Peak-to-peak stiffness degradation varied depending on the configuration of the infill wall. Compared with the bare frame, specimens S1-RN, S2-HC and S2-HCA experienced little stiffness loss under repeated cycles. Stiffness was maintained because the infill walls of these specimens did not undergo any substantial damage prior to failure. As shown in Figs. 4.21 through 4.23, the load-interstory drift relationships were approximately linear. On the other hand, specimens S3-OC and S4-OCX had greater stiffness degradation, mainly due to damage sustained within the perforated infill wall. The maximum amount of degradation in specimen S3-OC was 26.0%, occurring during the third cycle to 1.0% overall drift. This reduction in stiffness was attributable to yielding of the infill wall during the first cycle at 1.0% overall drift. Because of
additional reinforcement in the panels of specimen S4-OCX, yielding did not occur at this stage and the maximum degradation was 12.7%, again during the third cycle to 1.0% overall drift.
Table 4.5

Maximum Load Degradation Under Repeated Cycles

<table>
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<tr>
<th>Specimen</th>
<th>$\Delta_{cd}$ (%)</th>
<th>Cycle (Overall)</th>
<th>Cycle (in $\Delta_{cd}$)</th>
<th>Maximum Load (1st Cycle) (kips)</th>
<th>Load Degradation (%)</th>
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<td></td>
<td></td>
<td>(-)</td>
<td>(+)</td>
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### Table 4.6

Energy Dissipation Degradation Under Repeated Cycles  
(Calculated From Lateral Load vs Relative Floor Displacement Plots)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Δ₀d (%)</th>
<th>Cycle (Overall)</th>
<th>Cycle (in Δ₀d)</th>
<th>Energy Dissipated (1st Cycle) (kip-in)</th>
<th>Loss of Energy Dissipation (%)</th>
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<td>(-) (+)</td>
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### Table 4.7

Peak-to-Peak Stiffness Degradation Under Repeated Cycles
(Calculated From Lateral Load vs Relative Floor Displacement Plots)

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<tr>
<th>Specimen</th>
<th>$\Delta_{od}$ (%)</th>
<th>Cycle (Overall)</th>
<th>Cycle (in $\Delta_{od}$)</th>
<th>Stiffness ($1^{st}$ Cycle) (kip/in)</th>
<th>Stiffness Degradation (%)</th>
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4.7 Comparison of Specimens S3-OC and S4-OCX

The only difference between specimens S3-OC and S4-OCX was the addition of X-shaped steel within the vertical piers of specimen S4-OCX. As illustrated by comparing the curves in Fig. 4.26, the presence of the extra reinforcement affected the behavior of the retrofitted subassembly in a number of ways. First, the maximum capacity of specimen S4-OCX was about 29% greater than that of specimen S3-OC. The contribution of the steel to the ultimate load is estimated in Section 5.1.6.2. Although the capacity of the infill wall was substantially increased, the initial stiffness was only about 9% greater. This would be expected since the gross section geometry and material properties were not altered between the two tests. The addition of such a small quantity of steel would not significantly increase the overall subassembly stiffness.

Another distinction in specimen behavior was the yield point of the infill wall. The diagonal reinforcement increased the interstory drift level at which yielding occurred. Specimen S4-OCX's infill wall yielded at an average interstory drift of 0.36% compared to 0.28% for specimen S3-OC. Furthermore, the X-shaped steel provided resistance to sliding once substantial shear cracking occurred. The end result was to delay the development of a knee-braced frame mechanism (See Section 5.1.6.2 for further discussion). At every level of drift, specimen S4-OCX experienced less damage than specimen S3-OC. It was ultimately found that the addition of a small amount of steel substantially benefited the behavior and performance of the infill wall containing rectangular perforations.
CHAPTER 5
ANALYSIS

5.1 Prediction of Stiffness and Strength

5.1.1 General

Predictions for the stiffness and strength of the infilled subassemblies were calculated and compared to the actual experimental values. Existing analytical methods for infilled frames were considered when deemed appropriate. The overall goal was to propose a number of simple, yet accurate, models suitable for design use.

5.1.2 Stiffness of Bare Frame

Exemplified by a flat, bilinear lateral load-interstory drift relationship (Fig. 4.26), the bare slab-column frame subassembly had very low lateral stiffness. The transition from the elastic stiffness to the post yield stiffness occurred after flexural slab cracking at the slab-column joint. The experimental initial stiffness was estimated as the peak-to-peak stiffness of the first cycle to 0.25% drift. As shown in Table 4.4, a value of 21.6 kips/in was found.

The structural analysis software RISA-3D [46] was used to predict the initial, uncracked stiffness of the bare frame subassembly. Actual component geometry, boundary conditions and concrete properties were input into the structural model. Identical to the test set-up, a static load of 1 kip was applied at the top of the top column
stubs. Next, the program determined the joint deflections using linear elastic analysis. 
The initial stiffness was finally determined by dividing the applied load of 1 kip by the 
computed interstory drift. The predicted value of 23 kips/in was in very good agreement 
with the experimental results.

5.1.3 Stiffness of Infilled Frame without Connections to Slab

The lateral stiffness of infilled frames without connections has been estimated by 
replacing the infill wall with an equivalent diagonal compression strut. For stiffness 
calculations, the infill wall is replaced with an equivalent uniform strut having the same 
thickness as the panel and a length equal to that of the panel diagonal. The effective 
width, \( w \), of the strut has been determined by a number of researchers and is significantly 
dependant on the relative stiffness of the frame to infill and the stress-strain relationship 
of the component materials.

The lateral stiffness of specimen S1-RN was calculated using the method proposed by 
Stafford Smith [23] for single-story rigid frames. Strain energies resulting from tension in 
the east column, compression of the equivalent strut and bending of the frame were 
combined as shown in Eq. (5.1) to predict the lateral stiffness. All of the components of 
Eq. (5.1) are given in Eqs. (5.2a) through (5.2c) with the corresponding variables defined. 
The effective width of the infill, \( w \), was estimated as \( 0.16 r_{in} \) (approximately 16") , using 
Stafford Smith's relationship for the effective width of diagonally loaded panels for 
various length/height ratios [23]. This relationship was verified by comparing theoretical 
and experimental strain distributions along the loaded diagonal of mortar panels with 
different length-to-height ratios. Nearly identical results were obtained at the center of the
panel. Due to higher concentrated stresses away from the center of the panel, however, experimental strains tended to be larger at the loaded corners.

\[
K = \frac{A + B + C}{C(A + B)} \tag{5.1}
\]

\[
A = \frac{h_{col} \tan^2 \theta}{A_g E_c} \tag{5.2a}
\]

\[
B = \frac{r_{inf}}{w t_{inf} E_{inf} \cos^2 \theta} \tag{5.2b}
\]

\[
C = \frac{h_{col}^3 (3 I_b h_{col} + 2 I_{col} L)}{12 E_c I_{col} (6 I_b h_{col} + I_{col} L)} \tag{5.2c}
\]

where: \( h_{col} \) = height of column from centerline to centerline of beams

\( \theta \) = angle of diagonal to horizontal

\( A_g \) = gross x-section area of columns

\( E_c \) = elastic modulus of frame

\( E_{inf} \) = elastic modulus of infill

\( r_{inf} \) = diagonal length of infill

\( w \) = effective width of infill

\( t_{inf} \) = thickness of infill

\( I_b \) = moment of inertia of beam

\( I_{col} \) = moment of inertia of column

\( L \) = length of beam from centerline to centerline of columns

Using the above procedure, the lateral stiffness of a 3" solid panel was calculated to be 262.3 kips/in. Specimen S1-RN, however, did not have a uniform thickness of 3" throughout the entire infill wall. Instead, the middle four standard panels (about one-
quarter of the total infill wall area) were reduced to a thickness of 1". To account for the
central recess, the overall lateral stiffness was recalculated using a reduced thickness for
the infill. Assuming approximately one-quarter of the panel was 1" thick and three-
quarters was 3", an average thickness of 2.5" was estimated. Using a thickness of 2.5"
the predicted lateral stiffness was 230.4 kips/in. This value is in fairly good agreement with
the experimentally found initial peak-to-peak stiffness of 210.4 kips/in.

5.1.4 Stiffness of Infilled Frame with Connections to Slab

Except for specimen S1-RN, all of the retrofitted subassemblies were constructed with
dowel connections between the infill wall and the bounding slabs. Since the behavior of
the perforated panels was greatly dependant on the size and configuration of the
openings, the first objective was to estimate the initial stiffness of a solid panel. Test
results for specimen S2-HC were considered a good approximation of the actual
experimental stiffness. Two methods were used to estimate the stiffness, the diagonal
compression strut analogy presented in the previous section and the shear beam model.

Material properties of specimen S2-HC were used for the computation of stiffness in a
solid panel. The stiffness calculation was nearly identical to that of specimen S1-RN.
Aside from a smaller value for the elastic modulus of the infill, $E_{inf}$, all of the other
variables were the same. As a result, the predicted subassembly stiffness of specimen S2-
HC, 248.5 kips/in, was approximately 5 % lower than that of specimen S1-RN.

The shear beam model, on the other hand, estimated a much greater stiffness. Based
on elementary strength of materials, this model combined the deflection due to shear with
that due to flexure to determine the overall stiffness of the structure. Because of its
relatively large in-plane stiffness compared with that of the frame, only the infill wall properties were considered in the formulation. All of the equations required for the calculation are given in Eqs. (2.1) through (2.3). The elastic stiffness was found to be 1022 kips/in, about four times greater than the stiffness determined by the diagonal compression strut analogy. Neither of these estimates, unfortunately, accurately approximated the experimental value of 355.2 kips/in. From these findings, it was determined that the addition of interface connections between the slabs and the wall increased subassembly stiffness. The exact quantitative increase, however, was difficult to predict.

Estimating the stiffness of specimens S3-OC and S4-OCX using the aforementioned methods was more difficult. Because of the size, location and number of rectangular openings, the infill behavior was considerably different than that of a solid wall. Simple procedures, such as the shear beam model, were not capable of handling the complexities introduced by the addition of the rectangular perforations. Attempts to model the wall as a moment frame were not successful, producing results much higher than the experimental ones. Similarly, assuming all the deflection occurred in the fixed-ended piers shown in Fig. 5.1 was also extremely inaccurate. More powerful and computationally intensive approaches, such as the finite element method, are required, but were outside the scope of this work.

5.1.5 Strength of Infilled Frame without Connections to Slab

A number of researchers have proposed methods for determining the strength of infilled frames without any connections between the frame and wall [35, 47]. The
capacity of specimen S1-RN was estimated using expressions reported by Priestley and Calvi [52] for masonry infills. Both compression strength and the load required to initiate diagonal tension cracking were calculated and compared with the experimental findings. Since specimen S1-RN was not of uniform thickness, results were tabulated for both a solid 1" and 3" panel.

The horizontal force required to produce crushing of the infill is given by:

\[ H_c = R_c \cos \theta \quad \text{Eq. (5.3)} \]

where \( R_c \) is defined in Eq. (3.1). Furthermore, based on the relationship for tensile stress in a disk loaded along a diameter, the horizontal force needed to induce diagonal tension cracking can be estimated as:

\[ H_{dt} = \frac{\pi}{2} t_{inf} r_{inf} f_t \cos \theta \quad \text{Eq. (5.4)} \]

The results for these calculations are presented in Table 5.1. In addition, Table 5.1 also includes the maximum horizontal load the infill panel can resist prior to experiencing a shear failure in the surrounding columns. This load, \( H_{sf} \), was calculated using the procedure outlined in section 3.5.6.

**Table 5.1**

<table>
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<th>Panel Thickness (in)</th>
<th>( H_{sf} ) (kips)</th>
<th>( H_c ) (kips)</th>
<th>( H_{dt} ) (kips)</th>
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From inspection of the above data, the infill panel was not properly proportioned to fail prior to the slab-column frame. Even if the entire panel was 1" thick, the crushing strength, $H_c = 31.8$ kips, was 40% greater than the horizontal load required to initiate column shear failure, $H_{sf} = 18.9$ kips. Diagonal tension cracking, however, was expected since the thickness of the recessed central region controlled this behavior, and $H_{dt}$ for a 1" panel was less than $H_{sf}$. These estimations predicted the actual behavior of specimen S1-RN with a good degree of accuracy.

All overall applied horizontal loads were assumed resisted by the compression strut mechanism of the infill wall. Any contribution due to flexure of the slab-column frame was negligible, especially since the bare frame specimen was previously damaged during testing. As illustrated in Fig. 4.62, the residual strength of the bare flat-plate frame at the maximum interstory drift experienced by specimen S1-RN was only about 1 kip. The overall lateral loads, therefore, were deemed a good approximation of the horizontal force generated within the infill wall.

The experimental overall lateral load at the first observed diagonal crack was 12.5 kips ($\Delta_{id} = 0.24\%$) in the negative direction and 14.2 kips ($\Delta_{id} = 0.28\%$) in the positive direction. These values are in good agreement with the diagonal cracking strength of 12.1 kips determined for a 1" solid panel. Inaccuracy in the estimation of the true tensile strength of the LWPSC may account for differences in these values. For the cracking strength calculations, the tensile strength of the LWPSC was taken as $0.75*(0.1*f'c)$. Since splitting tension values, $f_s$, tend to be slightly higher than direct tension values, they were not used in the calculations. The experimental splitting tensile strength of specimen S1-RN was approximately 21% of the compressive strength, $f'c'$. As stated in ACI 318-95
[48], the typical tensile strength is about 10 to 15 percent of the compressive strength. 10 percent was, therefore, used for the cracking strength estimates. In addition, the factor of 0.75 was included, as per ACI 318-95 [48], to account for the lower tensile capacity of lightweight concrete as compared to normal weight concrete.

The load associated with observed column shear cracking was 24.3 kips ($\Delta_{id} = -0.46\%$ and $+0.52\%$) in both directions of loading. After shear cracking, the specimen was not capable of resisting much additional force and failed during the next stage of increased drift. The maximum load reached was 25.5 kips ($\Delta_{id} = 0.52\%$) in the negative direction and 25.1 kips ($\Delta_{id} = 0.57\%$) in the positive direction. Differences between the actual failure load of specimen S1-RN and the estimated load, $H_{sf}$, could have arisen for a number of reasons. First, the shear capacity of the subassembly column may have been greater than that predicted with the simplified design equation given in Eq. (3.2). Second, because of the number of variables involved, the contact length predicted by Eq. (3.4) may not have accurately predicted the actual conditions. Ultimately, because of the column shear failure the infill wall did not crush during the test.

5.1.6 Strength of Infilled Frame with Connections to Slab

5.1.6.1 Infill Wall with Circular Perforations

Specimen S2-HC and S2-HCA both contained circular holes within the infill panels, but only specimen S2-HCA was tested to failure. The main objective in adding circular perforations was to weaken the infill panels and insure that the wall failed prior to the exterior frame. Unfortunately, specimen S2-HCA ultimately failed due to shear cracking in the columns. The maximum capacity of the infill wall was, therefore, never reached.
5.1.6.2 Infill Wall with Rectangular Perforations

The maximum capacity of the retrofitted subassembly, $F_n$, was calculated by adding the strength of the infill wall, $F_{iw}$, to the strength of the bare slab-column frame, $F_{bf}$, as shown in Eq. (5.5a). The infill wall was modeled as a shear wall containing uniform openings. Similar to a system with flexible piers and strong spandrels, the peak resistance of the infill wall was determined by summing the capacities of the individual fixed-end piers at a given elevation (Eq. (5.5b)). Figure 5.1 illustrates the location, boundary conditions and dimensions of the piers considered in finding the peak load.

$$F_t = F_{iw} + F_{bf} \quad \text{Eq. (5.5a)}$$

$$F_{iw} = F_1 + F_2 + \ldots + F_9 \quad \text{Eq. (5.5b)}$$

where: $F_{bf} = \text{force in bare frame}$

$F_9 = \text{force in given pier}$

Because of the low aspect ratio of the LWPSC, shear, and not flexure, controlled the behavior of the piers. Shear capacity of each pier was determined using the ACI code provisions for walls. The nominal shear strength, $V_n$, was composed of two portions, the nominal shear strength provided by concrete, $V_c$, and the nominal shear strength provided by reinforcement, $V_s$. Concrete shear strength was calculated using Eq. (11-31) from ACI 318-95 [48], shown in Eq. (5.6):

$$V_c = 3.3 \sqrt{f_c'}hd + \frac{N_u d}{4l_w} \quad \text{Eq. (5.6)}$$
Eq. (5.6) corresponds to a principle stress of \(4\sqrt{f_c'}\) at the centroid of the pier cross section. The second term in the equation reflects the contribution of axial load, which in this case was zero. Concrete shear strength, therefore, was only a function of \(\sqrt{f_c'}\), \(h\) (wall thickness) and \(d\) (distance from extreme compression fiber to the centroid of the tension steel). Because the concrete was lightweight, the term \(\sqrt{f_c'}\) was replaced with \(f_s/6.7\), as per section 11.2.1.1 of the ACI code. Experimentally determined values of \(f_s\) were used in the calculations. No special provisions were made to account for the difference in strength between the LWPSC and the non-shrink grout used in the panel-to-panel and panel-to-frame joints. Table 5.2 lists the pertinent information and resultant concrete shear strength, \(V_c\), for the piers of specimens S3-OC & S4-OCX.

### Table 5.2

Properties of LWPSC Piers

<table>
<thead>
<tr>
<th>Pier (#)</th>
<th>(l_w) (in)</th>
<th>(d) (in)</th>
<th>(V_c) (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>S3-OC</td>
</tr>
<tr>
<td>1</td>
<td>4.625</td>
<td>3.875</td>
<td>1231</td>
</tr>
<tr>
<td>2</td>
<td>5.0</td>
<td>4.0</td>
<td>1271</td>
</tr>
<tr>
<td>3</td>
<td>6.25</td>
<td>5.5</td>
<td>1747</td>
</tr>
<tr>
<td>4</td>
<td>5.0</td>
<td>4.5</td>
<td>1430</td>
</tr>
<tr>
<td>5</td>
<td>6.25</td>
<td>5.5</td>
<td>1747</td>
</tr>
<tr>
<td>6</td>
<td>5.0</td>
<td>4.0</td>
<td>1271</td>
</tr>
<tr>
<td>7</td>
<td>6.25</td>
<td>5.5</td>
<td>1747</td>
</tr>
<tr>
<td>8</td>
<td>5.0</td>
<td>4.5</td>
<td>1430</td>
</tr>
<tr>
<td>9</td>
<td>4.625</td>
<td>3.875</td>
<td>1231</td>
</tr>
</tbody>
</table>

Total = 13,105 11,886

Since the piers in specimen S3-OC had no shear reinforcement, the nominal shear capacity, \(V_n\), was equal to the nominal capacity of the concrete, \(V_c\). Specimen S4-OCX,
on the other hand, had X-shaped reinforcement within piers #2, #4, #6 and #8. The extra steel was added to restrain the growth of inclined cracking, increase wall ductility and limit shear slippage after diagonal cracks developed. An increase in overall specimen strength was experimentally observed in specimen S4-OCX and attributed to the extra reinforcement. For this reason, the shear strength of the steel, \( V_s \), was included in the calculation of the nominal shear capacity of the aforementioned piers.

The shear capacity contributed by steel, \( V_s \), was based on Eq. (11-33) of ACI 318-95 and is given in Eq. (5.7).

\[
V_s = \frac{A_v f_y d}{s_2} \tag{5.7}
\]

where:  
  \( A_v \) = area of horizontal shear reinforcement  
  \( f_y \) = yield strength of reinforcement  
  \( s_2 \) = spacing of reinforcement within \( d \)

Since the steel was inclined at 51°, only the horizontal component of the rebar was included in the strength computations. Two bars crossed the potential cracking plane in each of the reinforced piers. The total contribution of all the steel was 9.0 kips.

The bare frame contribution was estimated from the experimentally determined maximum load-interstory drift envelope of specimen S1-BF. \( F_{bf} \) was the load corresponding to the interstory drift at failure of the retrofitted specimen. Table 5.3 compares the theoretical and experimental results for specimens S3-OC & S4-OCX. The experimental peak loads given for specimen S3-OC occurred at yielding of the wall. After shear failure of all the individual piers at the second level from the top, the load
resisting mechanism changed. Since the piers could no longer transmit shear forces, there was relative movement between the top and bottom portions of the wall. As a result, a knee braced frame mechanism developed as illustrated in Fig. 5.2. The absolute maximum loads (-17.7 kips and +17.6 kips) were achieved during this time and were approximately 10% greater than the yield point. The peak experimental load for specimen S4-OCX occurred prior to any pronounced slippage between the upper and lower halves of the wall.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Direction</th>
<th>$F_{iw}$ (kips)</th>
<th>$F_{bf}$ (kips)</th>
<th>$F_t$ (kips)</th>
<th>Experimental (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-OC</td>
<td>-</td>
<td>13.11</td>
<td>2.31</td>
<td>15.42</td>
<td>15.76</td>
</tr>
<tr>
<td>S3-OC</td>
<td>+</td>
<td>13.11</td>
<td>2.21</td>
<td>15.32</td>
<td>16.23</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>-</td>
<td>20.89</td>
<td>2.66</td>
<td>23.55</td>
<td>23.02</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>+</td>
<td>20.89</td>
<td>2.72</td>
<td>23.61</td>
<td>22.57</td>
</tr>
</tbody>
</table>

5.2 Performance-Based Retrofit Analysis

5.2.1 Subassembly Backbone Curves

Lateral force-deformation pushover (backbone) curves were developed from the experimental lateral load-interstory drift plots. The backbone curves represented the composite frame-wall subassembly behavior and were generated based on the procedure specified in FEMA 273 [9]. A multi-linear representation was constructed by connecting the points corresponding to the intersection of the first cycle curve for the $(i)^{th}$
deformation step with the second cycle curve for the (i-1)th deformation step, for all i steps. The resulting backbone curves are presented in Fig. 5.3 through Fig. 5.7.

In order to facilitate performance-based retrofit, the experimentally generated backbone curves were later transformed into idealized lateral force-deformation curves (See Section 5.2.2). All the idealized curves were plotted in the positive quadrant (positive force versus positive deformation) similar to most design aids. Since the specimens and the loading were symmetrical and the observed behavior was nearly identical in both loading directions, a composite curve was generated from those in the positive and negative quadrants. Each segment of the composite curve was given the average stiffness of the corresponding segments. Similarly, the ending deformation value for each composite section was taken as the average of those from the positive and negative cycles. Idealized load-displacement relationships were ultimately determined using the composite curve.

5.2.2 Limiting Deformation Values

In addition to the backbone curves, Fig. 5.3 through Fig. 5.7 also provide information on the limiting deformation values for the critical behavior states encountered during the tests. The extent of the elastic and plastic range is highlighted for both the positive and negative directions. Significant stages such as the onset of frame/wall damage and the loss of vertical and/or lateral load resistance are also noted. All of the limiting deformations were determined in terms of interstory drift and are tabulated in Table 5.4.
Table 5.4

Limiting Deformation Values from Experimental Data
(In Terms of Interstory Drift, $\Delta_{id, \%}$)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load</th>
<th>End of Elastic Range</th>
<th>End of Plastic Range$^1$</th>
<th>Onset of Apparent Damage</th>
<th>Loss of Vertical Resistance</th>
<th>Loss of Lateral Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Slab</td>
<td>Wall</td>
<td></td>
</tr>
<tr>
<td>S1-BF</td>
<td>-</td>
<td>0.11</td>
<td>0.82</td>
<td>0.09</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>+</td>
<td>0.20</td>
<td>0.84</td>
<td>0.10</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>S1-RN</td>
<td>-</td>
<td>0.45</td>
<td>None</td>
<td>N/A$^3$</td>
<td>-0.24</td>
<td>N/A$^2$</td>
</tr>
<tr>
<td></td>
<td>+</td>
<td>0.50</td>
<td>None</td>
<td>N/A$^3$</td>
<td>0.28</td>
<td>0.5</td>
</tr>
<tr>
<td>S2-HCA</td>
<td>-</td>
<td>0.33</td>
<td>None</td>
<td>0.09</td>
<td>0.16</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>+</td>
<td>0.27</td>
<td>None</td>
<td>0.09</td>
<td>0.15</td>
<td>—</td>
</tr>
<tr>
<td>S3-OC</td>
<td>-</td>
<td>0.27</td>
<td>1.29</td>
<td>0.17</td>
<td>0.17</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>+</td>
<td>0.25</td>
<td>1.33</td>
<td>0.15</td>
<td>0.15</td>
<td>—</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>-</td>
<td>0.34</td>
<td>1.16</td>
<td>0.07</td>
<td>0.15</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>+</td>
<td>0.34</td>
<td>1.18</td>
<td>0.08</td>
<td>0.15</td>
<td>—</td>
</tr>
</tbody>
</table>

$^1$ Based on last point on backbone curve
$^2$ Test stopped prior to failure
$^3$ Slab damage existing from test of specimen S1-BF

Based on the backbone curves in Figs. 5.3 through 5.7, the behavior of each specimen was matched with the appropriate curve type given in Fig. 5.8. The limiting deformation values from Table 5.4 were then used to determine the generalized deformation parameters defined in Fig. 5.8. The result was the development of simplified backbone curves for each subassembly. Data defining the entire lateral load-interstory drift relationships is provided in Table 5.5. All of the specimens, therefore, conformed to one of the three curve types given in Fig. 5.8, and were completely characterized by the values in Table 5.5.

Since the test of the bare frame specimen was stopped prior to failure, the actual values for deformation parameters $\Delta_2$ and $\Delta_3$ may be larger than those given. Similarly,
specimens S3-OC and S4-OCX also did not experience complete failure. Instead, these subassemblies were tested to an overall drift of 2.5% ($\Delta_{id} > 1.5\%$), which was believed to surpass any practical upper bound limits on interstory drift for flat-plate buildings subjected to earthquakes. Both specimens retrofitted with infills containing rectangular openings never did reach a point of substantial reduction in strength capacity. Although the slope of the final segments on the backbone curves in Fig. 5.6 and 5.7 were negative, the reduction in capacity was minimal. The absolute maximum load for both specimens occurred after the yield point, but the difference was not appreciable. For these reasons, the idealized backbone curves were best approximated by the relationship shown in Fig. 5.9. The deformation parameter $\Delta_{plastic}$ (Fig. 5.9) was taken as the value for deformation parameter $\Delta_2$ provided in Table 5.5. The slope of the post-yield portion of the curve was considered to be zero. Since the behavior past the maximum drift level achieved in the test was not certain, any residual strength after reaching the peak load, represented by parameter $Q_{residual}$ in Fig 5.9, should be assumed zero.

### Table 5.5

**Parameters for Component Behavior Curves**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Curve Type</th>
<th>Deformation Values ($\Delta_{id}$)</th>
<th>Slope From 1 to 2 (% of Elastic Stiffness)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\Delta_1$ (%)</td>
<td>$\Delta_2$ (%)</td>
</tr>
<tr>
<td>S1-BF</td>
<td>1 or 2</td>
<td>0.16</td>
<td>0.83</td>
</tr>
<tr>
<td>S1-RN</td>
<td>3</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>S2-HCA</td>
<td>3</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td>S3-OC</td>
<td>1 or 2</td>
<td>0.26</td>
<td>1.31</td>
</tr>
<tr>
<td>S4-OCX</td>
<td>1 or 2</td>
<td>0.34</td>
<td>1.17</td>
</tr>
</tbody>
</table>
Idealized backbone curves for specimens S1-RN and S2-HCA, on the other hand, were generated using the "Type 3" curve in Fig. 5.8. The maximum deformation of the subassembly, parameter $\Delta_1$ in Fig. 5.8 and Table 5.5, corresponded to both the end of the elastic region and the ultimate failure of the specimen. Because of the brittle nature of the failure, the retrofit configurations as tested for specimens S1-RN and S2-HCA cannot be expected to possess any ductility.

### 5.2.3 Acceptance Criteria

In performance-based retrofit, earthquake performance levels are coupled with earthquake design levels to create a rehabilitation objective (See Fig. 2.5). Earthquake design levels represent the demand to the system and are a function of the location of the building with respect to the causative faults, the regional and site-specific geology and the specified ground motion hazard levels. Performance levels describe the post-earthquake condition of a building and can be specifically related to the amount of loss due to seismic damage. Three structural performance levels are designated in FEMA 273: immediate occupancy, life safety and collapse prevention. Immediate occupancy is defined as a building that experiences only minor damage and is safe to occupy [9]. Unlike the immediate occupancy level, the life safety level allows for substantial damage, but insures that the structure is stable and has significant reserve capacity. Collapse prevention is the lowest performance level. Major damage and loss is acceptable, but the building remains standing.

Acceptance criteria are permissible values of properties such as drift and inelastic deformation used to decide the acceptability of an element's predicted behavior at a given
performance level. Based on previous research and design provisions, interstory drift was deemed the most appropriate measure for evaluating the acceptance criteria for infilled frames. Before determining acceptance criteria, however, the subassemblies were classified as either being force-controlled or deformation-controlled.

The definitions of force-controlled and deformation-controlled components were based on the three general component behavior curves shown in Fig. 5.8. Assemblies were classified as force-controlled unless either of the following two conditions applied: (1) the backbone curve was of "Type 1" or "Type 2" and the deformation parameter $\Delta_2$ was at least twice the deformation parameter $\Delta_1$, (2) the backbone curve conformed to "Type 1" and the deformation parameter $\Delta_2$ was less than twice the deformation parameter $\Delta_1$, but the deformation parameter $\Delta_3$ was at least twice the deformation parameter $\Delta_1$ (See Fig. 5.8 for definition of all parameters). According to these criteria, specimens S1-RN and S2-HCA were force-controlled and specimens S1-BF, S3-OC and S4-OCX were deformation-controlled. As illustrated in the subassembly backbone plots for specimens S1-RN (Fig. 5.4) and S2-HCA (Fig. 5.5), the properties and shapes of these curves were very similar to those of the "Type 3" curve in Fig. 5.8. Linear elastic behavior up to the point of maximum load was followed by an abrupt and complete loss of lateral load resistance. The maximum capacity of each subassembly was limited by the shear strength of the bounding columns and not by the capacity of the infill wall. Because of the premature failure of the surrounding frame, these infill panels were never able to yield and deform into the inelastic range. The strength capacity for all performance levels was taken as the maximum strength reached during the test.
Specimens S1-BF, S3-OC and S4-OCX, on the other hand, were deformation-controlled and, therefore, capable of maintaining large inelastic deformations (See Figs. 5.3, 5.6 and 5.7). In this case, the acceptance criteria are presented in terms of deformation. For primary lateral load resisting elements, FEMA 273 provides a guideline on determining the appropriate values from the results of laboratory tests [9]. All of the deformations correspond to points on the curves in Fig. 5.8 and are presented as follows:

- **Immediate Occupancy:** the deformation at which significant, permanent, visible damage occurred in the experiments
- **Life Safety:** 0.75 times the deformation at point 2 on the curve
- **Collapse Prevention:** 0.75 times the deformation at point 3 on the type 1 curve, but not greater than point 2

For both specimens S3-OC and S4-OCX immediate occupancy would correspond to the interstory drift associated with the yield of the infill wall. At this point there was significant damage to the infill panels, but the surrounding frame sustained only flexural slab cracking. Although the infill panels would probably need replacement after reaching this drift level, the building would be fully operational. The life safety and collapse prevention criteria could be determined using the above recommendations. Point $\Delta_2$ in Table 5.5 should be taken as the maximum allowable interstory drift.

As illustrated in the previous discussion, the most promising infill wall configuration for retrofitting flat-plate buildings was the LWPSC panels containing rectangular perforations. This system had a ductility capacity (deformation parameter $\Delta_2$ in Table 5.5 divided by deformation parameter $\Delta_1$ in Table 5.5) of 5 for specimen S3-OC and 3.5 for specimen S4-OCX. Because the test was stopped prior to experiencing ultimate failure,
the above-calculated values for ductility capacity may have even been larger. Unlike specimens S3-OC and S4-OCX, specimens S1-RN and S2-HCA did not possess any ductility. Loss of lateral load carrying capacity due to shear failure of the bounding columns stopped the tests. For seismic applications, brittle failure mechanisms such as these are extremely undesirable. Although the infill walls used for specimens S1-RN and S2-HCA did improve certain aspects of the frame behavior, they should not be used without proper adjustments to alter the failure mode.

5.3 Numerical Simulation

5.3.1 Background

IDARC 2D Version 4.0, a nonlinear dynamic analysis program developed at the State University of New York at Buffalo [53], was used to simulate the experimental results. System identification was performed to establish the most appropriate hysteretic parameters for the retrofitted subassemblies. This procedure has been attempted by numerous researchers, and basically consists of adjusting a set of non-dimensional control parameters until the predicted and experimental hysteretic loops provide a good match [54]. To assist in this process, the exact displacement history the subassemblies were subjected to was reproduced using a quasi-static analysis module provided in the software. The hysteretic representations provided in IDARC 2D for beams and columns were based on the three-parameter Park model [53]. Infill walls, on the other hand were modeled using a smooth hysteretic model.

The multi-linear (three-parameter Park) hysteretic model provided in IDARC 2D is capable of capturing the stiffness degradation, strength decay, and pinching behavior
typically associated with inelastic cyclic loading of reinforced concrete structures. Stiffness degradation is adjusted through the parameter $\alpha$, where a value of $\alpha$ equal to 200 represents no degradation. The effect of $\alpha$ on the hysteretic properties is illustrated in Fig. 5.10. All load-reversal branches of the load-deflection plots target a pivot point located on the elastic branch at a distance of $\alpha*P_y$ on the opposite side of the x-axis. In this expression, $P_y$ represents the moment or shear corresponding to yielding of the component. Smaller values of $\alpha$ will, therefore, yield greater amounts of stiffness degradation.

Strength deterioration is based on two control parameters, $\beta_1$ and $\beta_2$. $\beta_1$ is the ductility-based strength decay parameter ($\beta_1 = 0.001$ indicates no degradation) and $\beta_2$ is the hysteretic energy-based strength decay parameter ($\beta_2 = 0.001$ indicates no degradation). As shown in Fig. 5.10, strength degradation under repeated cycles to the same deformation level can be easily simulated. Any increase in the default value of the strength decay parameters corresponds to an increase in the amount of strength degradation.

For a number of reasons, including crack closure and reinforcement slip, load-displacement curves often exhibit a narrowing or pinching under inelastic load reversal. This phenomenon is common in reinforced concrete structures and results in a decrease in the area enclosed within the hysteretic loops. Since this area indicates the amount of energy that is dissipated, accurately representing pinching is vital for any inelastic analysis. IDARC 2D captures pinching behavior through the parameter $\gamma$. Decreasing $\gamma$ from its default value of 1.0 (no slip) will increase the severity of the pinching. By
manipulation of these four parameters, $\alpha$, $\beta_1$, $\beta_2$ and $\gamma$, a large range of hysteretic behaviors can be simulated.

The response of infill panels, on the other hand, is modeled in IDARC 2D using a smooth hysteretic model. Similar to the multi-linear model described above, the smooth model also includes the effects of stiffness degradation, strength deterioration and pinching. Instead of four parameters, however, infill panel behavior requires the definition of thirteen variables. The formulation of the currently available hysteretic model is based on the Wen-Bouc model.

The constants, $A$, $\beta_{\text{inf}}$, $\gamma_{\text{inf}}$ and $n$ are all parameters defined in Wen-Bouc's model. $A$, $\beta_{\text{inf}}$ and $\gamma_{\text{inf}}$ define the shape of the generated hysteresis loops, and $n$ controls the rate of transition from the elastic to the yield state. In addition, post-yielding stiffness is controlled by the variable $\alpha_{\text{inf}}$. It should be noted, that the subscript "inf" was added to $\beta$, $\gamma$ and $\alpha$ to distinguish them from the same variables used in the multi-linear model. Stiffness and strength degradation are modeled through the parameters $s_k$ and $s_{p1}$ & $s_{p2}$, respectively. Three variables, $A_s$, $Z_s$ and $Z_{bs}$ control the pinching behavior and $\mu_u$ reflects the ductility capacity of the panel.

5.3.2 Comparison of Results

IDARC 2D simulations were performed for specimens S1-BF, S3-OC and S4-OCX. Specimens S1-RN and S2-HCA were not analyzed because their extents of deformation were limited to the elastic range. In order to establish the hysteretic parameters for the slabs and columns of the bounding frame, the bare frame specimen was evaluated first. Past research on system identification for gravity load designed flat-plate buildings was
used as an initial estimate of the hysteretic parameters. Luo et al. performed a series of quasi-static tests on both isolated interior and exterior slab-column connections and slab-column subassemblies consisting of one interior and two exterior connections [55]. The observed moment-curvature relationships for each slab were used to identify the hysteretic parameters using version 3.0 of IDARC 2D. Comparing the calculated load-drift response with the measured results verified the suitability of the estimated parameters. As a result, average values of the hysteretic parameters were identified as $\alpha = 2.0$, $\beta_1 = 0.02$, $\beta_2 = 0.0$ and $\gamma = 0.12$ for slabs at interior connections and $\alpha = 2.0$, $\beta_1 = 0.02$, $\beta_2 = 0.02$ and $\gamma = 0.12$ for slabs at exterior connections.

Specimen S1-BF was modeled using the actual strengths of the concrete and the steel at the time of the test. Section properties were input for each component and IDARC 2D generated the appropriate tri-linear strength envelopes. The moment-curvature relationship for the columns was symmetrical. Slab behavior, on the other hand, differed in the positive and negative direction. Since the bottom slab reinforcement was not continuous through the columns, the positive moment transfer capacity was limited to the cracking strength of the slab. The lateral load-interstory drift behavior was very sensitive to the slab's stiffness degradation parameter, $\alpha$, but not as much to the other three parameters. As shown in Fig. 5.11, the best representation of the experimental results was achieved with $\alpha = 6.0$, $\beta_1 = 0.02$, $\beta_2 = 0.02$ and $\gamma = 0.12$. Subjecting the analytical model to the same displacement history as the test specimen generated the simulated response. Because the columns remained in the elastic range throughout the loading history, the overall subassembly behavior was independent of the hysteretic parameters assigned to
the columns. The hysteretic parameters identified for the slabs and columns of the bare frame were used in the subsequent evaluation of the infilled frames.

The experimental and simulated responses for specimens S3-OC and S4-OCX are given in Fig. 5.12 and Fig. 5.13 along with the values for all infill panel control parameters. As can be seen, the two curves for both specimens are in fairly good agreement. Default values provided in IDARC 2D were used for all variables except $A_s$, $\alpha_{inf}$ and $\mu_u$. The strength of the infill wall, $F_{iw}$, calculated in section 5.1.6.2 was taken as the yield strength of the infill wall, $V_y$. Initial infill panel stiffness, $K_0$, was estimated as the peak-to-peak stiffness of the maximum drift level reached prior to yield. A value of 120 kips/in was found to give reasonably good results for both specimens. One aspect of the experimental results difficult to reproduce was the reduction in area of the hysteresis loops under repeated loading to the same drift level.

5.3.3 Prediction of Isolated Infill Panel Behavior

One of the main objectives of the above IDARC 2D simulations was to identify the infill panel parameters necessary to reproduce the overall subassembly behavior. In many analysis programs the contribution of the infill panel must be considered separately, and not as a frame-wall assembly. For this reason, an attempt was made to determine the lateral load-interstory drift relationship of the isolated wall. As previously noted, the initial elastic stiffness, yield load and hysteretic control parameters were identified for specimens S3-OC and S4-OCX. The simulated lateral load-interstory drift plots of the infill walls for the entire loading history are presented in Fig. 5.14. From this data, idealized backbone curves conforming to the diagram in Fig. 5.9 were developed. Initial
stiffness values and yield strengths noted in Figs. 5.12 and 5.13 were used to generate the elastic portion of the idealized curve. The extent of the plastic range, $\Delta_{\text{plastic}}$, in Fig. 5.9 can be taken as value $\Delta_2$ from Table 5.5. Since the test was terminated at this point, it was decided to assume that the parameter $\Delta_{\text{ultimate}}$ was equal to $\Delta_2$.

5.4 Design Recommendations for LWPSC Infill Panels

5.4.1 Dowel Connections to Frame

Dowel connections were used between the slabs and the infill wall for all the retrofitted subassemblies except specimen S1-RN. The addition of interface connections altered the load transfer mechanism, substantially affecting the behavior of the infill wall. Although specimens S1-RN and S2-HCA both experienced a column shear failure, the maximum attained load was very different. Specimen S2-HCA had a maximum load approximately 31% greater than that of specimen S1-RN. Without any mechanical attachment, the infill wall acted as a diagonal compression strut providing a braced frame action. Once connections were provided, however, shear forces were transferred at the slab-wall interface. Cracks initiating near the dowel locations verified the existence of high shear forces. The proposed infill wall configuration required one dowel to be placed within each panel-to-panel joint. Every dowel was an identical piece of #6 rebar (60 Grade) cut from the same stock as the column reinforcement.

The shear capacity of the three interface dowels must be sufficient to develop the maximum capacity of the infill panel. It is vital that the dowel connections are strong enough to remain elastic and force the inelastic behavior to occur within the infill wall. To insure that the connection capacity is sufficient, it is recommended that an over-
strength factor be applied to the predicted wall capacity. Sufficient embedment depth of the dowel into the wall joint and into the slab must also be determined. Enough embedment must be provided to eliminate a bearing failure of the concrete adjacent to the connection. Concrete bearing strength can be calculated using Section 10.17 of ACI 318-95 [48].

5.4.2 Panel-to-Panel Connections

Visual inspection and crack monitoring of the infill panels throughout the tests indicated that the individual precast units behaved as a monolithic wall. The joint capacity was assessed using the concept of shear friction as described in Section 11.7 of ACI 318-95 [48]. Both the hoop steel protruding from the panel edges and the joint reinforcement added to the ultimate shear capacity. In addition, the grooved edges around the panel perimeter increased the surface area in contact between the LWPSC and the grout. The groove also allowed the hoop steel to extend past the outside edge of the adjacent panels, altering the potential cracking plane.

In order to insure proper inelastic infill panel behavior, the shear capacity of the panel-to-panel joints must be large enough to develop the full shear capacity of the wall. Because the LWPSC and the joint grout are cast at different times, shear transfer across the interface plane controls the joint capacity. A crack is assumed to occur along the grooved edge of the panel. All the hoop steel plus any additional joint reinforcement that crosses the anticipated crack location must resist relative displacement between the two concretes. Because of rough and irregular faces, separation of the interface surfaces may accompany relative slip. The reinforcement that crosses the cracking plan, therefore,
provides a clamping force, which at ultimate is equal to the total area of steel multiplied by the steel's yield stress. Friction between crack faces, shear capacity of protrusions on the crack surface and dowel action of the reinforcement all contribute to the resistance of the applied shear. For design purposes, Eq. (5.8) (Eq. (11-25) in ACI 318-95 [48]) should be used to predict the shear capacity of the panel-to-panel joints.

\[ V_n = A_f f_y \mu \]  

Eq. (5.8)

The coefficient of friction, \( \mu \), should be taken as 0.6\( \lambda \), where \( \lambda = 0.75 \) for all lightweight concrete.

5.4.3 Standard Panel Configuration

The standard panel configuration was developed to satisfy a number of important design considerations. Results from the experimental tests verified that these provisions were satisfactory within the scope of the investigated parameters. Design of the individual precast units should adhere to the following recommendations.

One of the original design objectives was to create a panel that was light enough to be manually handled and placed. To achieve this goal, the final dimensions of the standard panel were 1'-8 1/4" long by 9" high, and a total of sixteen panels were needed to fill the entire bay. The addition of perforations further reduced the weight, improving the efficiency with which the system was constructed. For bay sizes or aspect ratios different than that of the prototype structure, the proposed panel configuration, however, may need to be adjusted to fulfill the weight requirements. Most typical flat-plate systems will not
have bays with much larger column-to-column spacing than the one investigated, but the aspect ratio may easily vary.

Reinforcement within the panel should consist of hoops placed in both the horizontal and vertical directions. The presence of hoops will better confine the concrete after damage has occurred. Section 21.6.2.1 of ACI 318-95 [48] should be used to determine the quantity and spacing of the provided reinforcement.

5.4.4 Retrofit Recommendations for Existing Flat-Plate Buildings

Based on the results of this research, LWPSC infill panels can be effectively used to retrofit existing flat plate buildings. The most promising configuration consists of an infill wall with uniformly distributed rectangular perforations. Under conditions similar to those simulated in the testing, the following recommendations can be made.

- Of the choices investigated, prefabricated LWPSC infill panels with rectangular perforations are the best option.
- Dowel connections between the slab and the wall should be used to modify the load resistance mechanism. Without connections large concentrated shear forces are transmitted to the columns due to frame-wall interaction. Connections between the wall and the columns are not necessary. Furthermore, the dowel connections should be designed to remain elastic. All the inelastic activity should be forced to occur within the infill panels.
- All the LWPSC panels should be reinforced with hoops provided in both the horizontal and vertical directions. The amount and spacing of the reinforcement must conform to the provisions provided in Section 5.4.3.
• Panel-to-panel joints should be designed using shear friction principles (Section 5.4.2). Both protruding hoop steel and joint reinforcement can be included in the calculations. The joints must be designed with a capacity greater than that of the wall. This will eliminate the possibility of joint failure and ensure that the infill wall behaves in the expected manner.

• The infill wall must be designed to fail prior to the existing frame. To ensure that the shear capacity of the columns is greater than the capacity of the wall, the maximum strength of the wall should be determined using the procedure presented in Section 5.1.6.2.

• The use of diagonal reinforcement between the openings (as done in specimen S4-OCX) is recommended. It was found that even a small amount of reinforcement could greatly enhance the behavior of the infilled frame. When using additional diagonal bars, the capacity of the wall must be increased accordingly.

5.4.5 Recommendations for Future Research

In order to provide general design provisions for retrofitting flat-plate buildings with LWPSC infill panels containing rectangular openings, future research is required. The current investigation focused solely on the behavior of isolated subassemblies. Further studies, therefore, must be conducted on the behavior of entire buildings retrofitted in this manner. The effect of overall panel placement on a building's performance needs to be examined. Different configurations should be assessed in order to provide an optimal global panel layout. In addition, buildings of varying heights need to be investigated. Large overturning moments generated by infilling an entire bay can lead to a tension
failure of the bounding columns. Because these forces increase as the number of stories increase, building height limitations may be required. Tools such as non-linear static pushover analysis and non-linear dynamic analysis should be used to quantify the improvement contributed by the LWPSC infill panels. To verify this predicted behavior, reduced-scale shaking table tests are also recommended.
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1 Summary

The overall objective of the presented research was to develop a retrofit system for flat-plate buildings using lightweight, prefabricated infill panels. A number of different materials were investigated including precast autoclaved aerated concrete (PAAC) and lightweight pumice stone concrete (LWPSC). Preliminary tests were conducted to compare the suitability of the different materials. To simulate the forces acting on an unconnected infill panel under horizontal loading, diagonal tension tests were conducted. 2' by 2' square panels were placed in a 110-kip testing machine and loaded through steel supports along the diagonal. Data on the cracking load, maximum load and failure mechanism was gathered and assessed. In addition to strength and stiffness properties, each material was also evaluated based on practical issues. Ultimately, LWPSC was selected as the most promising choice.

To investigate the behavior of flat-plate frames infilled with LWPSC infill panels, four
four-tenth scale slab-column frames were constructed. The reduced-scale subassemblies
were designed for gravity loading based on ACI 318-63 [14]. All of the frames were
identical and represented older non-ductile detailing practices. A total of six quasi-static
tests were performed under displacement control. The load history consisted of fully
reversed cyclic loading at increasing drift levels. Repeated cycles to the same drift level
were included to investigate the degradation of peak load, stiffness and energy
dissipation. Specimen behavior was monitored through a series of strain gages, LVDT's
and by visual inspection.

Since the slab-column frame specimens were identical, each test examined a different
aspect of the infill panels. In order to establish the behavior of a flat-plate subassembly
without an infill wall, a bare frame specimen, S1-BF, was initially tested. The results
from specimen S1-BF were beneficial for two reasons. First, the improvements due to the
addition of the retrofit could be measured and compared in a quantitative manner.
Second, the behavior of the bare frame was later used in an attempt to isolate the
behavior of the infill wall from that of the subassembly. After testing specimen S1-BF,
the frame was retrofitted and retested.

All of the individual, prefabricated units were based on the dimensions of a standard
panel. The desire to create a system that could be manually handled and placed was one
of the main criteria in determining the size of the standard unit. With the chosen
dimensions, sixteen standard LWPSC units were required to fill an entire bay. Hoop
reinforcement that protruded from the edges of the panels was provided in each direction.
In addition, the perimeter of the panel was grooved to aid in the transfer of shear. Joint
reinforcement placed through the overlapping hoop steel was provided in all joints. A
commercially bought non-shrink grout was used for all the panel-to-panel and panel-to-
frame joints. Except for one of the specimens, all the subassemblies had connections
between the wall and the slabs. All connections consisted of placing #6 rebar dowels in
the panel-to-panel joints and into holes drilled through the slabs.
Specimen S1-RN was the only subassembly that did not have any connection to the surrounding frame. Because of this configuration, the behavior reflected that of a masonry infilled frame. In an attempt to control the maximum force transmitted through the infill, the central portion of the wall was recessed. Unlike specimen S1-RN, the remaining specimens all had connections to the bounding frame and investigated the addition of perforations within the infill panels. First, the inclusion of 2 1/2" diameter holes added within the framework of the standard panel was examined. Specimen S2-HC had two holes placed in each panel within the middle two stacks. Because of the small area of concrete actually removed, specimen S2-HC provided a good approximation to the behavior of a solid panel. After testing to an overall drift of 0.50%, additional holes were drilled throughout the entire wall area and the specimen was retested as specimen S2-HCA.

Instead of circular openings, the remaining two specimens were constructed with rectangular perforations. Aside from this difference, all other variables were held constant. The infill wall for both specimens S3-OC and S4-OCX contained two 5" long by 4" high openings per precast unit. The only difference between these two subassemblies was the addition of diagonal reinforcement in the area between openings of specimen S4-OCX.
6.2 Conclusions

6.2.1 LWPSC Infill Panel System

Lightweight pumice stone concrete infill panels can be successfully used to retrofit flat-plate frames if designed and detailed properly. The proposed system for construction of the infill wall is simple enough to be used by unskilled laborers without the need for additional mechanical equipment. As demonstrated in the reduced-scale tests, the precast units behaved as a monolithic wall. Simple dowel type connections located only between the slabs and the wall provided an efficient, yet cost effective, method of attachment. Furthermore, the wide spread availability of pumice stone throughout the world makes it a very desirable building material.

Although the results are beneficial, the small number of tests performed is insufficient for the development of general design provisions. Typical relationships between different geometric dimensions, material properties, reinforcement patterns, aspect ratio etc... and the load-displacement behavior are difficult to ascertain from the available data. It is possible, however, to conclude that LWPSC infill panels are a very promising retrofit solution for flat-plate buildings especially when rectangular perforations are added to the wall. Under conditions similar to those of the quasi-static tests, the proposed system was shown to be very effective. Changes to the properties or configuration of the frame or infill wall, such as a different aspect ratio for the frame opening, requires further investigation.
6.2.1.1  Recessed Panel with No Connection

Adding an unattached infill wall to a slab-column frame requires careful consideration of the concentrated shear forces transferred between the infill and the columns. As proposed, the system used in specimen S1-RN did not provide a satisfactory retrofit solution. Shear failure of the columns at a maximum interstory drift of only 0.57% compromised the gravity and lateral load resisting capacity of the frame. One of the reasons for this failure was a difference in the predicted and actual strengths of the LWPSC. Greater strengths than were anticipated in the preliminary design increased the capacity of the wall, and ultimately the amount of force transmitted to the frame. The addition of the recessed region did, however, initiate diagonal cracking of the wall at a lower level than that of a solid panel.

6.2.1.2  Infill Panels with Circular Perforations

The inclusion of circular perforations throughout the infill panels had some beneficial effects on the subassembly behavior, but did not weaken the wall enough to eliminate a shear failure of the column. Stiffness, strength and energy dissipation were all increased over that of the bare frame specimen. As in specimen S1-RN, the subassembly capacity was controlled by the shear strength of the columns. Due to the presence of the dowel connections, however, the maximum load was 31% larger than that achieved by specimen S1-RN. The infill wall did not yield, but cracking within the panel was wide spread and well distributed. For these reasons, the addition of circular perforations appears promising. Further research is required, though, to determine and refine the optimum size and location of openings to achieve the desired behavior.
6.2.1.3 Infill Panels with Rectangular Perforations

The most promising retrofit strategy that was developed consisted of LWPSC infill panels with uniformly distributed rectangular perforations. Unlike the previously discussed panel configurations, the subassemblies that were retrofitted with these panels experienced only minor damage within the columns. Initial stiffness, ultimate strength capacity and energy dissipation were all substantially increased over that of the bare frame. A simple analytical method for predicting the maximum capacity of infill walls with rectangular openings was developed. The estimated upper bound loads were in very good agreement with the experimentally determined values for both specimens S3-OC and S4-OCX. Additional diagonal reinforcement, as provided in specimen S4-OCX, greatly improved the subassembly behavior with only a minimal amount of extra material and labor. It must be noted that the results and recommendations provided from this research are relevant for a specific set of parameters. Additional work is needed to quantify the effects of changing the aspect ratio of the frame or altering the relative dimensions of the openings.
REFERENCES


[41] SEAOC, Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of California, Sacramento, California, 1996.


NOTATION

\[ A' = \text{shear area of specimen} \]
\[ A_{cv} = \text{net area of concrete section in direction of shear force} \]
\[ A_g = \text{gross area of column cross section} \]
\[ A_s = \text{area of steel reinforcement} \]
\[ A_s' = \text{area of a continuous bottom bar in a slab-column connection} \]
\[ A_{sv} = \text{projection on } A_{cv} \text{ of area of distributed shear reinforcement crossing } A_{cv} \]
\[ A_v = \text{area of horizontal shear reinforcement} \]
\[ A_{rf} = \text{area of reinforcement crossing potential crack} \]
\[ a = \text{width of diagonal strut for determination of elastic in-plane stiffness} \]
\[ b_o = \text{perimeter of critical section for slab-column connection} \]
\[ b_w = \text{width of column} \]
\[ c = \text{effective support size} \]
\[ D = \text{diameter of concrete cylinders for strength tests} \]
\[ d = \text{distance from extreme compression fiber to centroid of tension steel} \]
\[ d_b = \text{diameter of reinforcing bar} \]
\[ d_{tie} = \text{diameter of tie or stirrup reinforcement} \]
\[ E = \text{elastic modulus} \]
\[ E_c = \text{elastic modulus of concrete} \]
\[ E_g = \text{elastic modulus of grout} \]
\[ E_{inf} = \text{elastic modulus of infill panel} \]
\[ E_s = \text{elastic modulus of steel} \]
\[ F = 1.15 - (c/L) \text{ but } \geq 1 \text{ (from ACI 318-63)} \]
\[ F_{bf} = \text{strength of bare slab-column frame specimen} \]
\[ F_{iw} = \text{strength of infill wall with rectangular perforations} \]
\[ F_t = \text{maximum capacity of retrofitted subassembly} \]
\[ f_c' = \text{compressive strength of concrete} \]
\[ f_g' = \text{compressive strength of grout} \]
\[ f_m' = \text{compressive strength of masonry} \]
\begin{align*}
  f_r &= \text{modulus of rupture of concrete} \\
  f_s &= \text{splitting tensile strength of concrete} \\
  f_t &= \text{direct tensile strength of concrete} \\
  f_u &= \text{ultimate strength of reinforcement} \\
  f_y &= \text{yield strength of reinforcement} \\
  f_{ys} &= \text{yield strength of reinforcement in shear} \\
  G &= \text{shear modulus of elasticity} \\
  H &= \text{height of infilled frame specimen} \\
  H_c &= \text{horizontal force required to produce a compressive failure within infill} \\
  H_{dt} &= \text{horizontal force required to induce diagonal tension cracking within infill} \\
  H_{sf} &= \text{maximum horizontal force infill can resist prior to shear failure of column} \\
  h &= \text{thickness of wall} \\
  h' &= \text{height of concrete specimen for modulus of rupture test} \\
  h_{col} &= \text{column height between centerlines of beams} \\
  h_d &= \text{vertical distance from LVDT to eyelet on diagonal of infill} \\
  h_{inf} &= \text{height of the infill panel} \\
  h_{story} &= \text{story height} \\
  I &= \text{moment of inertia of section} \\
  I_b &= \text{moment of inertia of beam} \\
  I_{col} &= \text{moment of inertia of column} \\
  K &= \text{stiffness of infilled frame} \\
  K_{cr} &= \text{cracked stiffness in diagonal tension tests} \\
  K_i &= \text{initial stiffness in diagonal tension tests} \\
  K_{rel} &= \text{peak-to-peak interstory stiffness of test specimens} \\
  K_{top} &= \text{peak-to-peak overall stiffness of test specimens} \\
  L &= \text{span length from center to center of supports} \\
  L_{inf} &= \text{length of infill} \\
  L_s &= \text{span length for modulus of rupture test} \\
  l &= \text{length of concrete cylinders for strength tests} \\
  l_{beam} &= \text{location of concentrated force on beam from masonry infill panel} \\
  l_{cage} &= \text{location of concentrated force on column from masonry infill panel}
\end{align*}
\( l_w \) = length of wall in direction of loading  
\( M_o \) = static moment (Eq. (21-6) from ACI 318-63)  
\( N_u \) = axial force  
\( n \) = total number of reinforcing bars  
\( P \) = applied horizontal force  
\( P_{cr} \) = load at which first diagonal crack was visible in diagonal tension tests  
\( P_u \) = ultimate load reached in diagonal tension tests  
\( R_c \) = diagonal compressive strength of masonry infill  
\( R_{cr} \) = theoretical diagonal cracking strength of infill panel  
\( R_u \) = theoretical compressive strength for diagonal tension tests  
\( r_{inf} \) = length of diagonal of infill panel  
\( s_2 \) = spacing of shear reinforcement within distance d  
\( t \) = thickness of slab  
\( t_{inf} \) = thickness of infill panel  
\( V_c \) = contribution of concrete to shear strength  
\( V_g \) = applied shear on slab-column connection due to gravity loads  
\( V_n \) = nominal shear capacity of reinforced concrete section  
\( V_o \) = theoretical punching shear strength in the absence of an unbalanced moment (Eq. (11-38) from ACI 318-89)  
\( V_s \) = contribution of steel in nominal shear capacity  
\( W \) = total dead and live load on panel  
\( W_{od} \) = oven dry weight of aggregate  
\( W_{stock} \) = weight of aggregate in its field condition  
\( w \) = effective width of diagonal strut  
\( w/c \) = water / cement ratio  
\( w'_{ec} \) = effective width of diagonal strut for ultimate compressive strength  
\( w'_{ek} \) = effective width of diagonal strut for initial stiffness  
\( w'_{et} \) = effective width of diagonal strut for cracking strength  
\( \alpha \) = length of contact between infill panel and surrounding frame  
\( \Delta_{id} \) = interstory drift of subassembly  
\( \Delta_{od} \) = overall drift of subassembly
\[ \Delta E = \text{extension of infill wall diagonal} \]

\[ \Delta S = \text{shortening of infill wall diagonal} \]

\[ \delta_{bs} = \text{horizontal displacement of the bottom slab} \]

\[ \delta_{flexure} = \text{lateral deflection of infilled frame due to flexure (shear beam model)} \]

\[ \delta_{rel} = \delta_{ts} - \delta_{bs} \text{ (relative displacement between top and bottom slabs)} \]

\[ \delta_{shear} = \text{lateral deflection of infilled frame due to shear (shear beam model)} \]

\[ \delta_{top} = \text{horizontal displacement of the actuator at top of specimen} \]

\[ \delta_{total} = \text{total lateral deflection of infilled frame (shear beam model)} \]

\[ \delta_{ts} = \text{horizontal displacement of the top slab} \]

\[ \varepsilon_s = \text{strain in the steel reinforcement} \]

\[ \varepsilon_y = \text{yield strain of the steel reinforcement} \]

\[ \gamma = \text{shear strain of infill wall} \]

\[ \lambda = \text{dimensionless parameter used to define equivalent width of diagonal strut} \]

\[ \mu = \text{coefficient of friction} \]

\[ \theta = \text{angle that the infill panel diagonal makes with horizontal} \]

\[ \theta_b = \text{angle between eccentric strut used to determine beam forces and horizontal} \]

\[ \theta_c = \text{angle between eccentric strut used to determine column forces and horizontal} \]

\[ \rho_s = \text{ratio of area of steel to gross area of concrete} \]

\[ \rho_v = A_{sv}/A_{cv} \text{ (from ACI 318-95)} \]
FIGURES
Fig. 2.1  Non-Ductile Slab-Column Connection
Fig. 2.2  Relationship Between Gravity Shear Ratio & Lateral Drift [15]
Welded Connection (Kahn [6])

Bolted Connection (Higashi et al. [32])

Bolted Connection (Yuzugullu [34])

Exterior Mesh (Bertero & Brokken [7])

Shear Lugs (Frosch et al. [8])

Fig. 2.3  Methods for Connecting Infills to Existing Frames
Fig. 2.4  Infilled Frame Behavior Subjected to Horizontal Forces
## Fig. 2.5  SEAOC's Seismic Performance Matrix

<table>
<thead>
<tr>
<th>Earthquake Design Level</th>
<th>Fully Operational</th>
<th>Operational</th>
<th>Life Safety</th>
<th>Near Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequent (43 year)</td>
<td>▼</td>
<td>●</td>
<td>● Unacceptable Performance (for New Construction)</td>
<td></td>
</tr>
<tr>
<td>Occasional (72 year)</td>
<td></td>
<td>●</td>
<td>●</td>
<td></td>
</tr>
<tr>
<td>Rare (475 year)</td>
<td></td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Very Rare (970 year)</td>
<td></td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
</tbody>
</table>

- ● Basic Objective
- ■ Essential / Hazardous Objective
- ★ Safety Critical Objective
20" x 20" Columns

9" Slab

(a) Undeformed Flat-Plate Frame

- Assumed Point of Inflection

(b) Deformed Flat-Plate Frame

Fig. 3.1  Slab-Column Subassembly in Prototype Structure
(a) Elevation

(b) Plan

Fig. 3.2 Four-Tenth Scale RC Flat-Plate Subassembly
**BOTTOM REINFORCEMENT**
(#3 (solid), 40 Gr. & #2 (dashed) Bars)
(#2 Bars are Smooth)

---

**BOTTOM REINFORCEMENT**
(All Rebar #2 Smooth Bars)

*Fig. 3.3 Subassembly Slab Reinforcement (Bottom)*
TOP REINFORCEMENT
(All Bars #3, 40 Gr.)

TOP REINFORCEMENT
(#3 (solid), 40 Gr. & #2 (dashed) Bars)
(#2 Bars are Smooth)

Fig. 3.4 Subassembly Slab Reinforcement (Top)
Fig. 3.5 Subassembly Column Reinforcement
NOTE: REINFORCEMENT PARALLEL TO THE DIRECTION OF LOADING SHALL HAVE THE LARGER EFFECTIVE DEPTH.

**SECTION C-C**
Fig. 3.7: Pouring Sequence for Subassembly
Fig. 3.8  Stress – Strain Relationship for Subassembly Reinforcement

Fig. 3.9  Stress – Strain Relationship for Subassembly Concrete
Fig. 3.10 Diagonal Tension Tests - Final Cracking Pattern

(a) Typical PAAC Panel

(b) Typical LWPSC Panel

(c) Recessed LWPSC Panel
Fig. 3.11  Behavior of Square Infill Panel

Type A  Type B1  Type B2

Notes: All reinforcement is 5/32" diameter (Grade 6000). All panels have a nominal thickness of 4".

Fig. 3.12  Pumice Stone Panel Design for Diagonal Tension Tests
Fig. 3.13 Load - Deflection Curves for 2"x2"x4" LWSPC Panels (5" Bearing)
Localized Forces Applied to Columns

Localized Forces Applied to Beams

Fig. 3.14  Eccentric Compression Strut Analogy
Reinforcement Layout - Type A

Section A-A

Section B-B

Fig. 3.15 Standard LWSC Infill Panel Details
**Fig. 3.16** Panel-to-Panel Connection

**Fig. 3.17** Panel-to-Slab Connection
Specimen S1-BF

All joints completely grouted

Specimen S1-RN

Note: Letters designate infill panel design

Fig. 3.18 Infill Panel Configuration of Specimens S1-BF & S1-RN
**Specimen S2-HC**

**Specimen S2-HCA**

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**Fig. 3.20  Infill Panel Configuration of Specimens S2-HC & S2-HCA**
Fig. 3.21  Infill Panel Configuration of Specimens S3-OC & S4-OCX
Note: Cross-section details same as those for panel type A & A'

**Reinforcement Layout – Type B**

Note: Cross-section details same as those for panel type A & A'

**Reinforcement Layout – Type B'**

*Fig. 3.22  LWPSC Infill Panel Details for Specimen S3-OC*
Reinforcement Layout – Type C

Note: Cross-section details same as those for panel type A & A'

Reinforcement Layout – Type C'

Fig. 3.23  LWPSC Infill Panel Details for Specimen S4-OCX
Fig. 3.24(a)  Relationship Between Celdacrete Additive and Slump

Fig. 3.24(b)  Relationship Between Celdacrete Additive and Unit Weight
Fig. 3.25(a)  Relationship Between Celdacrete Additive and Compressive Strength

Fig. 3.25(b)  Relationship Between Celdacrete Additive and Split Tensile Strength
Fig. 3.26(a)  Relationship Between Celdacrete Additive and Yield

Fig. 3.26(b)  Relationship Between Celdacrete Additive and Cement Content
Fig. 3.27 Stress – Strain Relationship for Infill Reinforcement

Fig. 3.28 Stress – Strain Relationship for Infill LWPSC
Typical curve from 2" grout cube

\[ \sigma_1 = 0.4f' \]

\[ E_g = \text{Slope of line between } \sigma_1 \text{ & } \sigma_2 \]

\( \bullet \sigma_2 = 200 \text{ psi} \)

* Initial Concavity due to seating of specimen

Fig. 3.29  Stress – Strain Relationship for Infill Grout
Fig. 3.30  Testing Configuration and Details
Note: Values in () represent overall drift of specimen.

(a) Bare Frame Specimen

(b) Retrofitted Specimen

Fig. 3.31 Cyclic Loading History
* Note: Gages 19 & 20 placed on inside of first level column reinforcement of splice.

**SECTION A-A**

**Fig. 3.32 Strain Gage Layout**
Fig. 3.33  LVDT Layout & Details
Fig. 4.1  First Visible Slab Cracking for Specimen S1-BF ($\delta_{\text{top}} = 0.21$ in.)
Fig. 4.2 Final Slab Cracking Patterns for Specimen S1-BF
Top Slab (Top Cracking)

Bottom Slab (Top Cracking)

--- Final Cracking Pattern For S1-BF
---- 2.0% Overall Drift

Fig. 4.3 Final Slab Cracking Pattern for Specimen S1-RN
Fig. 4.4  Final Cracking Pattern of Front Elevation for Specimen S1-RN
Fig. 4.5  Final Slab Cracking Patterns for Specimen S2-HC
Fig. 4.6  Final Cracking Pattern of Front Elevation for Specimen S2-HC
Top Slab (Top Cracking)

Bottom Slab (Top Cracking)

- 0.50% Overall Drift
- 1.5% Overall Drift
- 2.0% Overall Drift
- 2.5% Overall Drift

Fig. 4.7 Final Slab Cracking Patterns for Specimen S2-HCA
Top Slab (Top Cracking)

Bottom Slab (Top Cracking)

--- 0.50% Overall Drift
----- 1.0% Overall Drift
------- 1.5% Overall Drift
-------- 2.0% Overall Drift
---------- 2.5% Overall Drift

Fig. 4.9  Final Slab Cracking Pattern for Specimen S3-OC
Fig. 4.10 Initial Cracking Pattern of Front Elevation for Specimen S1-OC

Cracks Opening
(1/8" to 3/8")
& Spalling Over
Entire Row
(1.0% Drift)

West (-)  East (+)

0.25% Overall Drift
0.50% Overall Drift
0.75% Overall Drift
1.0% Overall Drift
Top Slab (Top Cracking)

Bottom Slab (Top Cracking)

- - - - 0.25% Overall Drift
- - - - 0.50% Overall Drift
- - - - 0.75% Overall Drift
- - - - 1.5% Overall Drift
- - - - 2.5% Overall Drift

Fig. 4.11 Final Slab Cracking Pattern for Specimen S4-OCX
Fig. 4.12  Initial Cracking Pattern of Front Elevation for Specimen S4-OCX
Specimen S1-BF

Fig. 4.13  Lateral Load – Top Displacement Relationship for Specimen S1-BF
Fig. 4.14 Lateral Load – Top Displacement Relationship for Specimen S1-RN
Fig. 4.15  Lateral Load – Top Displacement Relationship for Specimen S2-HC
Fig. 4.16 Lateral Load - Top Displacement Relationship for Specimen S2-HCA
Specimen S4-OCX

Fig. 4.18 Lateral Load – Top Displacement Relationship for Specimen S4-OCX
Fig. 4.19  Maximum Load – Top Displacement Envelopes for All Specimens
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- Separation between infill wall & frame (+0.03%)
- First diagonal cracking within infill wall (-0.24% & +0.28%)
- Column shear failure

Specimen SI-RN

Elastic Range

Interstory Drift, $\Delta_{ub}$ (%)
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Experimental Response

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Specimen S3-OC

\[ A = 1.0 \quad K_o = 120 \text{ kips/in} \]
\[ \beta_{int} = 0.1 \quad V_y = 13.11 \text{ kips} \]
\[ \gamma_{int} = 0.9 \]
\[ n = 2.0 \]
\[ \alpha_{int} = 0.03 \]

Lateral Load (kips)

Interstory Drift, \( \Delta_{id} \), (%)

Simulated Response with IDARC

Specimen S3-OC

\[ A_e = 0.5 \quad s_k = 0.1 \]
\[ Z_e = 0.1 \quad s_{pit} = 0.8 \]
\[ Z_{se} = 0.0 \quad s_{pit} = 1.0 \]
\[ \mu_u = 20.0 \]

Lateral Load (kips)

Interstory Drift, \( \Delta_{id} \), (%)

Experimental Response

Fig. 5.12  Predicted and Experimental Load – Displacement Relationships for Specimen S3-OC
Specimen S4-OCX

- $A = 1.0$
- $K_e = 120 \text{kips/in}$
- $\beta_{af} = 0.1$
- $V_y = 20 \text{kips}$
- $\gamma_{af} = 0.9$
- $n = 2.0$
- $\alpha_{af} = -0.02$

Interstory Drift, $\Delta_{id}$ (%)

- $A_s = 0.5$
- $s_k = 0.1$
- $Z_s = 0.1$
- $s_{pl} = 0.8$
- $Z_{bs} = 0.0$
- $s_{p2} = 1.0$
- $\mu_a = 20.0$

(a) Simulated Response with IDARC

Specimen S4-OCX

Interstory Drift, $\Delta_{id}$ (%)

(b) Experimental Response

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