Analysis and behavioral characteristics of hollow-core plank diaphragms in masonry buildings

Aziz A. Sabri
Iowa State University

Follow this and additional works at: https://lib.dr.iastate.edu/rtd

Part of the Civil Engineering Commons, and the Structural Engineering Commons

Recommended Citation
https://lib.dr.iastate.edu/rtd/9576

This Dissertation is brought to you for free and open access by the Iowa State University Capstones, Theses and Dissertations at Iowa State University Digital Repository. It has been accepted for inclusion in Retrospective Theses and Dissertations by an authorized administrator of Iowa State University Digital Repository. For more information, please contact digirep@iastate.edu.
INFORMATION TO USERS

This manuscript has been reproduced from the microfilm master. UMI films the text directly from the original or copy submitted. Thus, some thesis and dissertation copies are in typewriter face, while others may be from any type of computer printer.

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleedthrough, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send UMI a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

Oversize materials (e.g., maps, drawings, charts) are reproduced by sectioning the original, beginning at the upper left-hand corner and continuing from left to right in equal sections with small overlaps. Each original is also photographed in one exposure and is included in reduced form at the back of the book.

Photographs included in the original manuscript have been reproduced xerographically in this copy. Higher quality 6" x 9" black and white photographic prints are available for any photographs or illustrations appearing in this copy for an additional charge. Contact UMI directly to order.
Analysis and behavioral characteristics of hollow-core plank diaphragms in masonry buildings

Sabri, Aziz A., Ph.D.
Iowa State University, 1991
Analysis and behavioral characteristics of hollow-core plank diaphragms in masonry buildings

by

Aziz A. Sabri

A Dissertation submitted to the
Graduate Faculty in Partial Fulfillment of the
Requirements for the Degree of
DOCTOR OF PHILOSOPHY

Department: Civil and Construction Engineering
Major: Structural Engineering

Approved:

Signature was redacted for privacy.

In Charge of Major Work

Signature was redacted for privacy.

For the Major Department

Signature was redacted for privacy.

For the Graduate College

Iowa State University
Ames, Iowa
1991
# TABLE OF CONTENTS

1. INTRODUCTION ........................................... 1
   1.1 General Remarks on Hollow-Core Planks .............. 1
   1.2 Problem Statement .................................. 3
   1.3 Scope of Dissertation .............................. 6

2. LITERATURE REVIEW ........................................ 11
   2.0 Floor Diaphragms .................................... 11
   2.1 Hollow-core Planks ................................. 12
   2.2 Seam Connectors ..................................... 13
   2.3 Analysis of Precast Diaphragms ..................... 22
   2.4 Effect of Vertical Load ............................ 25
   2.5 Finite Element Analysis of Hollow-Core Plank Diaphragms ........................................... 25

3. EXPERIMENTAL INVESTIGATION ............................. 27
   3.0 General ............................................. 27
   3.1 Test Facility ....................................... 27
   3.2 Test Instrumentation ............................... 32
   3.3 Load Program ....................................... 39
   3.4 Test Parameters ..................................... 40
   3.5 Test Results ........................................ 43
   3.5.1 Orientation comparisons .......................... 43
       3.5.1.1 Comparison of Tests #4 and #8 ............ 45
       3.5.1.2 Comparison of Tests #5 and #6 ............ 45
       3.5.1.3 Comparison of Tests #13 and #14 .......... 48
       3.5.1.4 Comparison of Tests #2 and #4 .......... 54
   3.5.2 Boundary condition comparisons .................. 57
       3.5.2.1 Comparison of Tests #6, #7 and #8 ........ 57
       3.5.2.2 Comparison of Tests #4 and #5 ............ 58
       3.5.2.3 Comparison of Tests #2 and #6 ............ 61
       3.5.2.4 Comparison of Tests #12 and #13 .......... 63
   3.5.3 Plank thickness comparisons ...................... 66
       3.5.3.1 Comparison of Tests #4, #9 and #11 ........ 67
   3.5.4 Topping comparisons ................................ 71
       3.5.4.1 Comparison of Tests #6 and #10 ........... 71
   3.5.5 Masonry and steel frames comparisons ........... 83
       3.6 Summary of Experimental Results ............... 85

4. ANALYTICAL PREDICTIVE METHODS ......................... 90
   4.1 Initial Stiffness .................................... 90
       4.1.1 Evaluation of the bending stiffness component ........................................... 91
       4.1.2 Evaluation of the shear stiffness component ........................................... 92
4.1.3 Evaluation of the edge zone stiffness component ........................................ 93
4.1.4 Evaluation of the framing members component ........................................... 103
4.1.5 Evaluation of the initial stiffness ................................................................. 103

4.2 FME Strength Predictions ................................................................. 103
  4.2.1 Finite element analysis ................................................................. 104
  4.2.2 FME prediction for north-south oriented planks ....................................... 104
  4.2.3 FME prediction for east-west oriented planks ......................................... 113

4.3 Limit State Prediction ................................................................. 116
  4.3.1 Limit state prediction for north-south planks ......................................... 119
  4.3.2 Limit state prediction for east-west planks ............................................ 123
  4.3.3 FME and limit state prediction for diagonal tension failure ....................... 126

4.4 Comparison of Experimental and Analytical Results 133
  4.4.1 Initial stiffness .................................................................................. 133
  4.4.2 FME loads ...................................................................................... 136
    4.4.2.1 FME for north-south oriented planks ............................................. 136
    4.4.2.2 FME for east-west oriented planks .............................................. 138
    4.4.2.3 FME loads for topped planks ...................................................... 138
  4.4.3 Limit state loads ................................................................................ 141
    4.4.3.1 Limit state loads for north-south oriented planks ............................. 141
    4.4.3.2 Limit state loads for east-west oriented planks ................................ 143
    4.4.3.3 Limit state loads for topped planks .............................................. 143

5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS ..................................... 147
  5.1 Summary ............................................................................................. 147
  5.2 Conclusions ....................................................................................... 148
    5.2.1 Experimental conclusions .................................................................... 148
    5.2.2 Conclusions From Analysis .................................................................. 151
  5.3 Recommendations for Continued Study ..................................................... 152

6. REFERENCES ....................................................................................... 154

7. ACKNOWLEDGEMENTS ......................................................................... 157
1. INTRODUCTION

1.1 General Remarks on Hollow-Core Planks

Many of today's masonry buildings are constructed with precast, prestressed concrete planks. These concrete planks are used for both floors and walls. In the past several decades, hollow-core slab production has increased sharply and is now the single most used product in the precast, prestressed concrete industry [1]. A typical building floor construction utilizing these planks is shown in Figure 1.

Hollow-core floor systems maintain several advantages over other more traditional building material systems [2]. Precasting offers improved quality control, higher strength concrete, accelerated curing techniques and better opportunities for standardization. These factors, in turn, allow for compressed construction time schedules. Prestressing permits the use of shallower depths, longer spans, more controllable performance in terms of cracking and deflections and less material usage. The use of concrete offers increased fire resistance and durability over other materials such as timber and steel.

As expected, precast, prestressed concrete floor panels also possess several disadvantages. Precasting requires closer tolerances in casting; there is less margin for error. Creep strains are greater in prestressed concrete because of the compression introduced with the prestressing strands.
Figure 1. Typical building floor construction utilizing hollow core planks
Compared to steel, concrete floor systems are heavier and bulkier. In seismic areas, this additional mass can cause an increase in the lateral forces within a structure. Therefore, further study of the lateral forces within a precast structure must be undertaken.

1.2 Problem Statement

Lateral forces, typically produced by earthquakes or winds, are resisted by the use of a space frame system and/or shear walls. In either case, the lateral loads are transmitted from one wall to another through the floor system, as shown in Figure 2. For seismic design, the slab or horizontal diaphragm is one of the essential components in the structure. This type of system is often referred to as a "box" system since each component serves the function of transferring the lateral force.

The distribution of the horizontal forces to the shear wall or space frame system depends on the properties of the diaphragm slab and the resisting system. In the case of a shear wall building, the diaphragm can be considered to be a horizontal beam with the roof or the floor system acting as the web of that beam. With simple, transverse lateral loads, the forces flow out to the shear walls as is shown in the force distribution diagram given in Figure 3. In order to optimize the performance of the floor system, the in-plane stiffness of the diaphragm should exceed that of their
Figure 2. Lateral force distribution
Figure 3. Schematic force distribution diagram
respective vertical subsystems. Diaphragms of this type are categorized as rigid [3,4] (refer to Figure 4 for a conceptual sketch). In this instance, the diaphragms act as a flat plate that transmits lateral loads to the vertical bracing elements in proportion to their relative rigidities. Conversely, with flexible diaphragms, loads are distributed to vertical subsystems as a continuous beam using tributary areas. Both rigid and flexible systems should be able to retain a sufficient amount of in-plane stiffness or strength, well beyond the elastic range, in order to prevent collapse.

1.3 Scope of Dissertation

Research at Iowa State University has been performed to study the behavioral characteristics of hollow-core plank diaphragms subjected to in-plane shear forces. Seventeen full-scale diaphragms have been tested. The objectives of this research project were to determine the basic failure modes, ascertain behavioral characteristics, and investigate analytical methods for predicting strength properties of precast, prestressed hollow-core plank diaphragms.

Diaphragm strengths were characterized by 1) First Major Event (FME) strength, 2) limit state strength, and 3) ultimate strength. The FME strength is the load associated with the initial diaphragm breakdown. The cause for this breakdown may be due to a major crack at the seam between adjacent planks, a diagonal tension crack propagating across the diaphragm, or
Figure 4. Diaphragm stiffness classification
any other event that results in a significant change in stiffness and eventual transformation of the diaphragm into the inelastic range. The limit state strength is defined as the peak stabilized strength, and the ultimate load refers to the peak virgin strength. Displacements associated with these peak strengths may or may not necessarily coincide. Achievement of a specific limit state strength for a particular diaphragm is more likely to be reproducible, since this strength is attained during stabilization cycles. On the other hand, the ultimate strength occurs during the virgin cycle (first time incremental displacement), and represents a load that may not be counted on under similar circumstances.

The effects of various parameters were investigated. These parameters included:

- boundary condition (number of sides connected to the loading frame)
- orientation (placement of the planks with respect to the direction of the applied lateral load)
- slab thickness (plank depth of 6, 8, and 12 inches)
- aspect ratio (geometric configuration of the diaphragm)
- topping (addition of a 2-inch cast-in-place concrete slab)
- seam connectors (variation in the number of seam connectors to verify the implications of attaining an alternate failure mode for the untopped tests).
• framing member rigidity, i.e. the effect of replacing the steel testing frame with masonry walls. Table 1 summarizes the relationship of these parameters to the individual diaphragm tests.

This dissertation will be directed to the evaluation of the different parameters affecting the diaphragm behavior and formulating analytical equations to predict the diaphragm strength.
Table 1. Summary of Parameters for Diaphragm Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Plank depth (in.)</th>
<th>Number of sides connected</th>
<th>Orientation</th>
<th>Topping</th>
<th>Weld ties per seam</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>2</td>
<td>T</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>2</td>
<td>P</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>2</td>
<td>T</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>2</td>
<td>T</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>4</td>
<td>T</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>8</td>
<td>4</td>
<td>P</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>8</td>
<td>3</td>
<td>P</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>2</td>
<td>P</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>8B</td>
<td>8</td>
<td>2</td>
<td>Y</td>
<td>N</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>2</td>
<td>T</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>4</td>
<td>P</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>11</td>
<td>12</td>
<td>2</td>
<td>T</td>
<td>N</td>
<td>3</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>2</td>
<td>P</td>
<td>Y</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>8</td>
<td>4</td>
<td>T</td>
<td>Y</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>8</td>
<td>4</td>
<td>P</td>
<td>Y</td>
<td>0</td>
</tr>
<tr>
<td>15</td>
<td>8</td>
<td>4</td>
<td>T</td>
<td>N</td>
<td>15</td>
</tr>
<tr>
<td>16</td>
<td>8</td>
<td>4</td>
<td>P</td>
<td>Y</td>
<td>0</td>
</tr>
</tbody>
</table>

Notes:

All two-sided tests, with the exception of Test #2, are connected to the loading beam and the restrained support.

The orientation refers to the direction of the applied load, i.e. P means parallel to the applied load (EW), T means transverse to the applied load (NS).

Test #16 utilized masonry walls with a steel loading frame as opposed to all other tests which utilized steel testing frame.
2. LITERATURE REVIEW

2.0 Floor Diaphragms

A well designed diaphragm is essential for the structural integrity of a building during earthquake or wind induced motions. Shear force traditionally is distributed to the various elements of the lateral load resisting system in proportion to their rigidities relative to that of the diaphragm. Thus, knowledge of the behavioral characteristics of a diaphragm is necessary to perform a lateral load (seismic) analysis of a multi-story building.

Diaphragms may be categorized according to their composition into the following common types: cold-formed steel, composite steel deck, timber, reinforced concrete, and precast concrete. Each of these groups are similar in that they provide in-plane shear resistance, but they exhibit unique behavioral characteristics. The seismic performance of each of these systems is different and depends on the characteristics of the diaphragm and the event.

During previous seismic events, the performance of precast concrete units without topping has been poor, while the precast concrete units with topping have exhibited variable to good performance [5]. Martin and Korkosz [1] stated that the absence of continuity and redundancy (between the precast slabs) has caused some designers to question the stability (of precast structures) under high lateral loads.
This statement is echoed in most references on this subject [e.g. 1,3,6,7,8,9].

2.1 Hollow-core Planks

Hollow-core planks are most commonly used as structural floor or roof elements, but may also be used as wall panels for load bearing or non-load bearing purposes. Typical spans for hollow-core planks range from 16 to 42 feet with possible depths of 6, 8, 10, and 12 inches. Presently, six types of hollow-core plank products are commercially available, as listed in Reference [10].

- **Dynaspan**: Made in 4- or 8-foot widths by a slip forming process with low-slump concrete. Each slab has 14 cores.

- **Flexicore**: A wet cast product poured in 2-foot widths and 60-foot long spans. Voids are formed with deflatable rubber tubes.

- **Span-Deck**: A wet cast product poured in two sequential operations with the second being a slip cast procedure. The planks are 4 or 8 feet wide with rectangular voids.

- **Spancrete**: Made in 40-inch wide units by tamping an extremely dry mix with three sequential sets of tampers in order to compact the mix around the slip forms.
**Spiroll**

An extruded product made in 4-foot wide units with round voids formed by augers which are part of the casting machine.

**Dy-Core**

An extruded 4-foot wide product made by compressing zero slump concrete into a solid mass by a set of screw-conveyors in the extruder. High frequency vibration combined with compression around a set of dies in the forming chamber of the machine produces the planks with oblate, or octagonal shaped voids.

Due to the close proximity of the manufacturing plant and several other factors, the Span-Deck planks were used exclusively in the diaphragm tests conducted as part of this investigation.

### 2.2 Seam Connectors

Four methods of connections are currently being utilized [11]. These are cast-in-place topping, welded hardware, projecting reinforcement, and shear friction with grouted joints.

Specimens with the cast-in-place topping provide the best lateral force resisting system. The 2-inch minimum topping, shown in Figure 5, has performed well. The topping mandates that all of the individual panels act as a single rigid unit. Section 17.5 of the American Concrete Institute Building Code (ACI 318-89) may be adopted for use in topping design.
Figure 5. Plank system with topping
Welded hardware connectors comprise the second category of hollow-core connections. The Japanese Prestressed Concrete Association has stated that weld joints are suitable for seismic resistance provided that the parts to be welded are suitably doweled in the concrete to create the necessary bond [12]. This connection, shown in Figure 6, is quite common for precast members. The Prestressed Concrete Institute (PCI) Design Handbook [2] defines a method of strength prediction based on the angle, length and type of reinforcing bar. Values are presented in several texts and papers on this subject for different types of connection ties [13,14]. A value of approximately 10 kips in shear is referenced for a generic weld tie similar to those used in the diaphragm tests [2]. Elemental tests are recommended in order to determine the exact strength of any particular unit [6].

The most popular type of connection is the untopped, grouted-reinforced joint. This design employs reinforcement parallel and perpendicular to the joint at the extremes of each plank unit as is shown in Figure 7. The seam, however, is only filled with grout. Experimental observations have shown that the coefficient of friction in the seam after the initial crack approaches a value of 1.0 [11]. A conservative value of 80 psi is given for grout shear strength in several sources [2,6]. Some references list actual experimental values for various types of planks and seams [13,14,15].
Figure 6. Weld tie details
Figure 7. Schematic of shear friction joint
Walker's article [15] "Summary of Basic Information on Precast Concrete Connections", alluded to information concerning shear strength tests of Spancrete slabs with grouted joints. These eight tests, which investigated various slab thicknesses, were performed for Arizona Sand and Rock Company, Phoenix, Arizona (1964). The grouted seams were subjected to a static, monotonic direct shear load applied on the center of the three slabs of the test specimen.

Proprietary tests were conducted by Tanner Prestressed and Architectural Company [16], which investigated the shear strength of the grouted horizontal shear joint in 8-inch Span-Deck planks. As in the previously discussed tests, a force was applied to the center of three sections, so that the load was equally transmitted to the 5-foot long seams. The failure mode for each of the three tests was a longitudinal shear crack propagating along the grout-plank interface.

An experimental investigation of the shear diaphragm capacity was undertaken by Concrete Technology Corporation in February, 1972 [11]. The objectives of this test were to measure and evaluate the ability of the 8-inch Spiroll Corefloor slabs to transfer horizontal shear through the grouted longitudinal joints without shear keys, as well as to determine the coefficient of friction, which served as a direct measure of the effectiveness of shear friction reinforcement in the end beams. The longitudinal joints were
subjected to pure shear as the load was applied to the center slab while the exterior slabs were held in place. The shear strength was not tested to ultimate capacity, since a measure of the shear friction effectiveness was one of the desired objectives. After the joints were artificially cracked, the coefficient of friction was measured and was found to vary between 1.3 and 2.0. These values indicated that the reinforcement had performed satisfactorily and that the 1.0 value was conservative for planks with extruded edges.

A publication of the Concrete Technology Associates by Cosper, et. al., [13] reviewed hollow-core diaphragm test results for the shear strength of the grouted keyway between adjacent 12-inch Dy-Core panels. Longitudinal shear loading was accomplished by applying a load against sixteen 1/2-inch prestressing strands, which were in an "X" arrangement across the seam. Parameters researched included the following: 1) the shear capacity of an uncracked grouted joint, 2) the effectiveness of shear-friction reinforcement in transferring shear across a joint, 3) the ductility of the system after the bond between panels had fractured, and 4) the effect of cyclic loading on the system. The uncracked grouted seam demonstrated a high capacity in resisting lateral shear loads. Shear-friction steel placed in the edge beam supplied adequate clamping forces once the seam had fractured. Ductility demands were satisfied as well, since the shear strength
continued to rise after joint displacement. Finally, the diaphragm exhibited sufficient resistance to cyclic loading by maintaining a stabilized strength after repeated cycles above design requirements.

Another experimental study, by Reinhardt [14], tested the joint between hollow-core planks under shear loading while subjected simultaneously to a normal force. Variable strengths of mortar and lengths of the grouted connection (0.3 to 2.1 meters) were accommodated for the single seam. Joint length was found to have a significant influence upon shear stress at fracture for their particular testing configuration. Failures were characterized by brittle fractures of the bond at the mortar and grout interface. Each of these tests used a slightly different testing frame two of which are shown in Figure 8. With such testing arrangements, however, the actual maximum shear is not simply the load divided by the contact area. A correction factor which accounts for the non-uniformity of the shear stresses must be used. Chow, Conway, and Winter state that the distribution of shear stresses in deep beams (beams whose depths are comparable to their spans) depart radically from that given by the ordinary, simple formulas [17]. Using finite difference, strain-gage measurements, and photoelastic measurements, Roark and Young have tabulated the correction factor for various testing
Figure 8. Sample elemental shear testing frame
arrangements [18]. The values for one such arrangement are shown in Figure 9.

2.3 Analysis of Precast Diaphragms

The current design practice for diaphragms is based on the seam connection capacity. Therefore, with estimates of the strength of the seam connections, the analysis of the diaphragm is possible. In order to simplify the analysis procedures the following assumptions are generally made [19]:

• the panels initially remain in the linear range,
• all the nonlinear deformations occur first in the edge zone connections, and
• the horizontal panel systems (slabs) are usually rigid.

The magnitude of the horizontal unit in-plane shear force, $V_u$, is calculated according to the shear stress formula [13,20]:

$$V_u = \frac{VQ}{I}$$  \hspace{1cm} 2-1

where:

$V_u$ = in-plane shear stress, kips/in.
$V$ = applied shear, kips
$Q$ = first moment of area, in.$^3$
$I$ = first moment of inertia, in.$^4$

or
<table>
<thead>
<tr>
<th>RATIO L/d</th>
<th>RATIO Span/d</th>
<th>MAX Mc/l</th>
<th>MAX Mc/l</th>
<th>MAX V/D</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2.875</td>
<td>0.970</td>
<td>1.655</td>
<td>1.57</td>
</tr>
<tr>
<td>2</td>
<td>1.915</td>
<td>0.960</td>
<td>1.965</td>
<td>1.60</td>
</tr>
<tr>
<td>1</td>
<td>0.958</td>
<td>1.513</td>
<td>6.140</td>
<td>2.39</td>
</tr>
<tr>
<td>1/2</td>
<td>0.479</td>
<td>5.460</td>
<td>15.73</td>
<td>3.78</td>
</tr>
</tbody>
</table>

(From Reference 18)

Figure 9. Influence of s/d ratio on maximum shear and maximum fiber stress
where:

\[ V_u = \frac{1.5M}{sh} \]  \hspace{1cm} 2-2

\text{M} = \text{service load moment, kip-in.}
\text{h} = \text{thickness of diaphragm, in.}
\text{s} = \text{diaphragm span, in.}

The allowable unit shear force is then calculated by the following formula based on a recommended shear stress of 80 psi from References 2 and 6:

\[ V_n = 0.08t \]  \hspace{1cm} 2-3

\text{t} = \text{effective seam thickness, in.}

A strength reduction factor of 0.85 is normally multiplied by the allowable unit shear force. Load factors are then multiplied by the calculated unit shear force values to obtain a controlling equation. For example, using the recommended load factor of 1.3 from Reference 6, the following equation results:

\[ 1.3V_u = 0.85V_n \]  \hspace{1cm} 2-4

The foregoing analysis procedure reflects the current practice which will be replaced later on with the proposed analysis and design recommendations.
2.4 Effect of Vertical Load

Most of the previous in-plane diaphragm tests have been conducted without the presence of vertical load. A comparison of tests with and without vertical load showed that the behavior of the systems was approximately the same [21]. Nakashima, Huang and Le-Wu Lu state that the crack pattern, failure mode and stiffness degradation were similar for tests with and without vertical load. In addition, the ultimate loads were within fifteen percent of each other. A study employing composite deck diaphragms was also performed and similar results were obtained [22]. The behavior of a floor slab under in-plane load is therefore assumed to be two-dimensional problem and hence vertical load effects were ignored in this study.

2.5 Finite Element Analysis of Hollow-Core Plank Diaphragms

Research conducted by the United States Steel Corporations [23] focused on a finite element analysis of staggered-truss framing system with the horizontal diaphragm consisting of precast prestressed hollow-core planks. Several cases involving different parameters were studied: both cored and solid planks, the addition and exclusion of spandrel shear attachments, and whether or not the joints between adjacent floor planks were cracked. A shear force of 1000 kips was applied to each truss, and stresses were determined. (This assumed wind-shear was equivalent to applying a high wind
pressure of 40 psf to a 40 story structure.) The procedure undertaken for the finite element model and results obtained were discussed. The stress diagrams indicated that a shear diaphragm accurately described the majority of the behavior of the plank assembly with respect to the manner in which loads were transferred. However, locally high principal tensile stresses were noted in opposite corners of the floor. These were reduced with the shear attachment of the spandrels to the planks. Also, the substitution of the solid planks at the edges of the floor was not effective in reducing corner stresses. Adversely high stresses resulted when the joints were assumed to be cracked, thereby causing individual plank rotation. Finally, for tall structures, sufficiently high diagonal tension stresses existed, and therefore, must be considered in the design of the horizontal diaphragm.
3. EXPERIMENTAL INVESTIGATION

3.0 General

Seventeen full-scale plank diaphragms were tested at Iowa State University. This chapter will describe the testing arrangements and instrumentation as well as the results. The complete description of these tests can be found in References 24 and 25.

3.1 Test Facility

A cantilever test frame with a restrained edge was used for the testing of the plank diaphragms. The loading beam represented a masonry shear wall subjected to the horizontal (in-plane) drift induced by an earthquake. The restrained edge modeled a stiff adjoining panel or another shear wall. The side beams simulated interior or exterior masonry load bearing walls.

The testing frame for Test #16 utilized masonry walls to replace the steel frame used for the first sixteen tests. The steel testing frame is shown in Figure 10, while the masonry frame is depicted in Figure 11. The system was designed for a working load of approximately 400 kips and a maximum displacement of ± 5 inches [26,27,28,29].

The restrained end of the testing frame was formed with three large concrete reaction blocks anchored to the laboratory floor with 2-inch diameter high-strength bolts post-tensioned to 240 kips. A steel plate was embedded into
Figure 10. Steel testing frame
Figure 11. Masonry testing frame
Figure 11. Continued

Applied Load

Bond Beam

Steel Beam

Hollow-Core Planks

Rollers

Steel Beam

Bond Beam

Loading Beam

Applied Load

42"
Figure 11. Continued
the reaction blocks to facilitate placement of the studs. The remaining sides of the steel testing frame were composed of W24X76 wide flange framing beams. These beams were connected using flexible T-shaped elements.

The load was applied to the steel frame along the front beam through two double-acting hydraulic cylinders. Specially fabricated 240-kip load cells were attached in series to measure the load applied by the hydraulic actuators. Each of the hydraulic cylinders was mounted within two C15X40 channels connected to wide flange sections anchored to the floor with four high strength bolts post-tensioned to 240 kips. A closed-loop MTS control system was used to control the displacement during the test. A direct current differential transducer (DCDT) was mounted on the loading beam to deliver the feedback signal. The loop was completed by a servo-valve which controlled the hydraulic actuators. Loop stability could be maintained within 0.001 inch [26,27,28,29].

3.2 Test Instrumentation

Instrumentation was used to measure the behavior of the diaphragm throughout the tests. The behavior was characterized by the loading beam displacement, applied loads, in-plane and out-of-plane plank displacements, relative slip and split between the planks, relative slip between the diaphragm and framing members, and strains in the loading members.
In-plane and vertical displacements were measured with direct current differential transducers (DCDT) and mechanical dial gages. A DCDT located near the northeast corner of the diaphragm was connected to the steel loading beam and served to provide feedback to the MTS servo-controller. For the masonry loading frame the DCDT was placed at the center of the planks along the front wall. Dial gages and DCDTs were placed at each corner of each of the planks to measure edge displacement relative to both the floor and the framing members. Relative seam slip was measured with a DCDT on each end and at the center of every seam. Figure 12 shows typical placement of the dial gages and DCDTs for the first sixteen tests, while Figure 13 shows the same for Test #16.

Strain gages were attached to the webs of the framing beams to measure the strains along these edge beams. On the first and second tests, uniaxial and rosette strain gages were mounted on the northeast quadrant of the diaphragm, however, accurate readings were not obtained because of the core voids within the planks. For Test #16, strain gages were place on the reinforcing dowels in the walls and the topping.

All of the DCDTs, strain gages and loading cells were monitored by the data acquisition system (DAS). The DAS consisted of a 150-channel Hewlett Packard (HP) Model 3497A data acquisition control unit interfaced with an HP Model 85 microcomputer. The microcomputer was connected to two disk
DCDT OR DIAL GAGE WHICH MEASURES:

- ▲ RELATIVE PLANK SLIP
- ■ ABSOLUTE PLANK DIPLACEMENT
- ● DISPLACEMENT OF PLANK RELATIVE TO TESTING FRAME
- ○ DISPLACEMENT OF LOADING BEAM

Figure 12. Typical diaphragm test instrumentation
DCDT and/or Dial gauge to measure:

- Relative in-plane slippage
- Relative out-of-plane separation
- Relative deflection between top and bottom of wall
- Relative vertical separation
- Wall out-of-plane displacement
- Wall in-plane displacement

Figure 13. Instrumentation for Test #16
drives, a digital plotter and a high speed printer. At each load point, the DAS recorded all readings on magnetic disk and printout. Between readings, the DAS constantly monitored and plotted the in-plane load and the in-plane displacement. In order to create this plot, the DAS recorded load and displacement readings at the rate of one reading per second during the entire time the displacement was being applied. The plot program also had the capability of integrating the area under the hysteretic plot, which represents the energy between load point readings. Figure 14 is an example of a plot produced during Test 5.

The concrete surface of the test specimen was painted with a soluble white latex paint to identify the cracks. The surface was also marked with a rectangular grid, as is shown in Figure 15, to aid in monitoring crack locations. After each load point, a search of the surface was conducted. The interstices were traced with a black marker and the load point was written next to the end of the crack. The location of these cracks was noted on a tape recorder for future reference. A camera mounted thirty feet above the specimen was also used to document surface deformation. In addition, many close-up photographs were taken during each test.

After the first several tests, a record of the condition of the studs throughout the test was deemed necessary. A wire was attached to the top of each stud before grouting. The
Figure 14. Example hysteretic plot from Test #5

Displacement (in) vs. Load (kips)

-125.00
-0.50
0.00
0.25
0.50
125.00
Figure 15. Plank grid schematic
wires for all the studs were then connected to a switching box and this box was attached to an ohmmeter. A ground wire was put on the loading beam to complete the circuit. Thus, when a stud broke, the ohmmeter measured infinite resistance. These data were recorded at the end of each of the stabilization cycles.

3.3 Load Program

The Sequential Phased Displacement, SPD, loading program [30] was used for each of the tests. This program employed standard stabilization cycles beginning at approximately a 0.0125-inch displacement. In addition to these cycles, this technique utilized decaying displacement cycles to better define the hysteretic behavior. These degradation loops assisted in the establishment of the correlation between demand and capacity for inelastic deformations [30].

The procedure for the SPD program involved executing progressively larger increments of displacement for each cycle prior to the first major event (FME). In each of the diaphragm tests, the FME was either a seam or diagonal tension crack. Once the FME occurred, a sequential phased displacement loading procedure was followed. At every new increment of loading, both degradation and stabilization cycles were completed. The degradation intervals were one-fourth the original displacement and were followed by at least three stabilization cycles. More than three
stabilization cycles were required if the strength of the final cycle was less than ninety-five percent that of the previous cycle. Figure 16 is a schematic of a typical loading plot.

The SPD procedure was used because it more accurately represents the earthquake excitation pattern than the usual monotonic or simple reversed cyclic loading patterns. Most seismic events contain many low-energy points between the major spikes as shown in the typical earthquake ground motion record in Figure 17. Saatcuglu, et al., [31] noted that in many instances the maximum deformation can occur early in the excitation response with few inelastic cycles proceeding it. Thus, by using degradation cycles, the lower bound within a given hysteretic curve can be identified. The stabilization cycles are also essential in order to calculate the "stabilized" energy. Additional details on the SPD procedure and rationale can be found in Reference 30.

3.4 Test Parameters

The plank diaphragm characteristics identified were:

- Initial stiffness,
- First Major Event, FME,
- Limit State Strength.

The full-scale tests were designed to test the effect of different parameters on these characteristics. The parameters tested included:
Figure 16. Sample SPD loading program
Figure 17. Typical earthquake ground motion record
• Orientation of the planks with respect to the load,
• Boundary conditions, i.e. number of sides connected to the loading frame,
• Plank thickness (6- and 12-inch versus 8-inch planks)
• Addition of a 2-inch topping,
• Number of seam fasteners.
• Effect of the connecting members, i.e. steel frame versus masonry walls.

Table 2 shows a summary of the tests with the different parameters tested.

3.5 Test Results

The results obtained from the experimental investigation will be presented in the next sections along with the comparison of the different parameters tested.

3.5.1 Orientation comparisons

Orientation of the untopped precast planks has proven to be a significant factor in achieving diaphragm action. The planks can either be oriented parallel or transverse to the shear wall (loading beam). A comparison of several of the diaphragm tests allows for an assessment of the effects of this parameter on the overall behavioral characteristics of a hollow-core floor system.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Thickness of planks (in.)</th>
<th>Compressive strength of</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Seams (psi)</td>
<td>Cores (psi)</td>
</tr>
<tr>
<td>1</td>
<td>8</td>
<td>3800</td>
<td>7800</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>6500</td>
<td>6500</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>5700</td>
<td>5700</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>6116</td>
<td>6652</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>5600</td>
<td>7700</td>
</tr>
<tr>
<td>6</td>
<td>8</td>
<td>5591</td>
<td>6301</td>
</tr>
<tr>
<td>7</td>
<td>8</td>
<td>2879</td>
<td>6007</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>2425</td>
<td>6100</td>
</tr>
<tr>
<td>8B</td>
<td>8</td>
<td>N/A</td>
<td>6100</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>4216</td>
<td>6136</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>4192</td>
<td>4539</td>
</tr>
<tr>
<td>11</td>
<td>12</td>
<td>3487</td>
<td>5835</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>N/A</td>
<td>5500</td>
</tr>
<tr>
<td>13</td>
<td>8</td>
<td>4246</td>
<td>5109</td>
</tr>
<tr>
<td>14</td>
<td>8</td>
<td>4895</td>
<td>6547</td>
</tr>
<tr>
<td>15</td>
<td>8</td>
<td>4000</td>
<td>6500</td>
</tr>
<tr>
<td>16</td>
<td>8</td>
<td>7233</td>
<td>7233</td>
</tr>
</tbody>
</table>

Notes:

* Plank strength not available, this value is assumed
3.5.1.1 Comparison of Tests #4 and #8

Test #4 consisted of four planks oriented transverse to the loading beam. The system was connected to the loading beam and the restrained end. Test #8 was connected in a similar fashion, however, the planks were oriented parallel to the loading beam.

A comparison of the cyclic stiffness of each system throughout the test is a good indicator of the behavioral characteristics. The average cyclic stiffness is defined as the slope of the line between the maximum positive and negative load values of the third hysteresis loop of each displacement increment as is shown in Figure 18 [28]. Test #4 produced dramatically higher FME and ultimate strength values than Test #8, 376% and 154%, respectively. An evaluation of the stiffness plots (Figure 19) confirms this statement. Test #4 had a much higher initial stiffness (1281 kips/in. versus 706 kips/in.) and maintained a higher stiffness through 2 inches of displacement.

3.5.1.2 Comparison of Tests #5 and #6

A comparison of Tests #5 and #6 also serves to isolate the orientation parameter. These tests were identical with the exception of the orientation of the planks. Both were 8 inches thick and were connected on all four sides. The planks in Test #5 were oriented transverse to the loading beam, and
Figure 18. Calculation of average cyclic stiffness, $K_{\text{cyclic}}$ [28]
Figure 19. Cyclic stiffness comparison for Tests #4 and #8
those in Test #6 were oriented parallel to the application of load. A study of the stiffness plots, shown in Figure 20, reveals that Test #5 maintained a higher stiffness throughout the test, although after the 0.75-inch displacement, the values were quite close.

Figure 21, a plot of the virgin and stabilized curves, confirms that the transverse orientation produced a somewhat stronger system. This drawing, however, exposed another observation. After the ultimate load, or approximately the 1-inch displacement, the strength of both the virgin and stabilized load versus displacement envelopes for Test #6 exceed those for Test #5 in the east direction. A 50% to 70% loss in load occurred between the limit state and the 3-inch displacement for the virgin curve of Test #5. Only a 25% to 30% decline in virgin load occurred over a similar interval for Test #6. This phenomenon is probably due to the fact that Test #5 secured a greater amount of diaphragm action early in the test. Consequently, more studs failed at this time (24 broken in Test #5 versus 14 in Test #6) which, caused a decrease in the load during the later stages of the test.

3.5.1.3 Comparison of Tests #13 and #14

Tests #13 and #14 both contained 8-inch floor slabs plus a 2-inch topping on the diaphragm. The floor slabs for Test #13 were oriented with the seams perpendicular to the loading beam and were connected to the testing frame on all perimeter
Figure 20. Cyclic stiffness comparison for Tests #5 and #6
Figure 21. Envelope curves for Tests #5 and #6
edges. Thus, this comparison reflects a change in the orientation parameter. Figure 22 is a comparison plot of the stiffness versus cyclic displacement curves for Tests #13 and #14. The values of stiffness are nearly the same for similar displacements. The only difference between the tests is the initial stiffness values. The initial stiffness for Test #14 was 3289 kips/in., which represents an increase of 21.9% with respect to the initial stiffness value of 2698 kips/in. for Test #13. The higher compressive strength of the topping may have influenced this behavior.

A comparison plot of the hysteretic displacement curves for Tests #13 and #14 is shown on Figure 23. The curves have a similar shape with the loads recorded for Test #14 being only slightly higher. The maximum load for Test #14 was 302 kips, which represents an increase of 2.2% with respect to the ultimate strength of 295.6 kips recorded during Test #13. Failure modes attained at the FME for both were the diagonal tension mode. Comparable FME strengths were also achieved, with 230.4 kips and 260.8 kips for Tests #13 and #14, respectively. This represents a difference of only 13.2% with respect to Test #13.

In general, these numbers reflect that, experimentally, orientation of the planks under the topping has little influence on strength and other behavioral characteristics of the topped diaphragm.
Figure 22. Stiffness comparison of Tests #13 and #14
Figure 23. Envelope curves comparison of Tests #13 and #14
3.5.1.4 Comparison of Tests #2 and #4

Tests #2 and #4 both contained 8-inch floor slabs connected to the testing frame on two sides (the loading beam and restrained end). The floor slabs for Test #2 were oriented with the seams parallel, whereas, Test #4 planks were oriented with the seams perpendicular to the loading beam. A comparison plot of the virgin/stabilized displacement curves for Tests #2 and #4 is shown on Figure 24. The curves have a similar shape with the loads recorded for Test #4 being higher (up to the 2-inch displacement). The maximum load for Test #4 was 90.5 kips, which represents an increase of 55% with respect to the ultimate strength of 58.4 kips recorded during Test #2. Failure modes attained at the FME for Test #2 was tensile-bond while Test #4 failed in shear-bond mode. FME strengths were different, with 58.4 and 88.0 kips for Tests #2 and #4, respectively. This represents a difference of 50.7% with respect to Test #2.

Figure 25 is a comparison plot of the stiffness versus cyclic displacement curves for Tests #2 and #4. The values of stiffness are higher for Test #4 than these recorded for Test #2. The initial stiffness for Test 4 was 1172 kips/in., which represents an increase of 81.6% with respect to the initial stiffness value of 645 kips/in. for Test #2.

These comparisons reflect that for the untopped slabs, diaphragms with the planks oriented perpendicular to the load
Figure 24. Envelope curves for Tests #2 and #4
Figure 25. Cyclic stiffness comparison of Tests #2 and #4
directions attained a significantly higher strength than those with the planks oriented parallel to the load direction. This behavior is not evident in the topped slabs, thus, the addition of the topping nullifies the orientation parameter for topped diaphragms.

3.5.2 Boundary condition comparisons

By varying the number of sides of a diaphragm connected to the testing frame, the effects of the boundary condition parameter may be studied. As with any study of this nature, only the variable under consideration may be altered. Thus, Tests #4 and #5, which were identical with the exception of the number of sides which were connected, may be studied. Tests #6 versus #7, #6 versus #8, or #2 versus #6 may likewise be reviewed to determine the effects of this particular parameter.

3.5.2.1 Comparison of Tests #6, #7 and #8

A comparison of the results of Tests #6, #7 and #8 demonstrates the effects of connecting two, three and four sides, respectively, of a diaphragm with similar orientation and thickness. Test #6 was connected on four sides, Test #7 on three sides (loading beam, restrained end and cantilever end), and Test #8 was connected on two sides (restrained end and loading beam). The FME strengths of Tests #6 and #7 were 72% and 65%, respectively, greater than Test #6. Similarly,
the limit state strength of Tests #6 and #7 were 122% and 108% greater than that for Test #8, respectively. The virgin and stabilized strength curves followed a similar pattern; that is, the four-sided connection resulted in significantly greater diaphragm action and consequently the highest strength. The stiffness diagrams for each of these tests are given in Figure 26. The initial stiffness of Tests #6 and #7 was 1474 kips/in. and 1584 kips/in., respectively; while Test #8 yielded an initial stiffness of only 1101 kips/in. In addition, a careful inspection of this sketch reveals that the systems connected on three and four sides retained a higher stiffness throughout the test. This information serves to verify the conclusion that the greater the number of sides connected, the more diaphragm action achieved.

3.5.2.2 Comparison of Tests #4 and #5

Tests #4 and #5 were similar in every aspect except that Test #4 was connected at the restrained end and along the loading beam, and Test #5 was connected on all four sides. Although the FME loads were very similar, the limit state load for Test #5 was 21.4% greater than that for Test #4. In addition, the stiffness comparison, shown in Figure 27, revealed that Test #5 had a higher initial stiffness (2005 kips/in. compared with 1281 kips/in. for Test #4). The stiffness remained higher until the 0.5-inch displacement cycle.
Figure 26. Comparison of stiffness for Tests #6, #7 and #8

DISPLACEMENT, in.

STIFFNESS, Kips/In.

- TEST #6
- TEST #7
- TEST #8
Figure 27. Cyclic stiffness comparison of Tests #4 and #5
The virgin and stabilized strength curves (Figure 28) further show that the four-sided test exhibited greater diaphragm action. Although both test configurations were symmetric and should have resulted in a symmetric envelope curve, clearly, on the average, Test #5 yielded a greater capacity.

3.5.2.3 Comparison of Tests #2 and #6

Another boundary condition comparison can be found in Tests #2 and #6. Note that any comparison utilizing Test #2 must be analyzed in the context in which it was tested. The results of Test #2 may not be utilized directly in the latter stages of the test because of the failure of the connections (refer to Reference 24).

These tests exemplify another aspect of the boundary condition parameter; that is, the diaphragm must be adequately attached to the shear wall. Test #2 and Test #6 both consisted of four planks oriented parallel to the loading beam. Test #6 was connected to the loading beam (shear wall) and to the restrained end. Test #2, on the other hand, was mainly connected to the side beams (bearing walls). Only a minimal attachment was made to the loading beam. Diaphragm Test #6 exhibited a higher maximum load (35% higher). In addition, the initial stiffness of Test #6 was 54% higher than that for the second test. In short, Test #2 showed that when
Figure 28. Envelope curve comparison for Tests #4 and #5
a plank floor system is primarily connected on the sides perpendicular to the applied shear load (masonry walls) and inadequately connected to the loading beam (shear walls), a significant deficiency in diaphragm action occurs.

3.5.2.4 Comparison of Tests #12 and #13

The distinguishing parameter between Tests #12 and #13 was the boundary conditions difference. Test #13 was fastened to all perimeter edges of the testing frame while Test #12 was connected to only the loading beam and restrained edges. The other difference was the orientation which was proven not to affect the topped diaphragm strength (see Section 3.5.1).

Figure 29 is a comparison of the stiffness of diaphragm in Tests #12 and #13. The stiffness of Test #12 was considerably less than the stiffness of Test #13 throughout most of test. The initial stiffness were 1596 kips/in. and 2698 kips/in. for Tests #12 and #13, respectively. This represents a 69.0% increase in stiffness with respect to Test #12. Test #13, with all four sides connected, should have yielded a somewhat larger initial stiffness.

Figure 30 is a comparison plot of the virgin/stabilized envelope curves for Tests #12 and #13. These curves show that a larger load was recorded for Test #13 than for Test #12, especially during the 0.10- to 0.5-inch range. The larger strength capacity for Test #13 was attributed to the boundary condition parameter. The maximum load for Test #13 was 195.6
Figure 29. Cyclic stiffness comparison of Tests #12 and #13
Figure 30. Envelope curves for Tests #12 and #13
kips compared to an ultimate strength of 135.8 kips for Test #12. This represents a 118% increase in ultimate load for Test #13 with respect to Test #12. Some of this increase may have been caused by the concrete in the cores, seams, and topping of Test #13 having slightly higher strength than those used in Test #12. This difference is primarily attributed to the boundary condition difference.

Tests #12 and #13 both failed in the diagonal tension failure modes, while attaining considerably different capacities. The FME loads for Tests #12 and #13 were 127.5 kips and 230.4 kips, respectively. No seams cracked in either test, indicating the absence of the seam shear-bond failure mode. The major difference in the behavior of Tests #12 and #13 was the higher strength capacity associated with Test #13.

The comparison of Tests #12 and #13 corresponds with the results of Tests #4 and #5. In both comparisons, the connection to all four sides of the testing frame increased the strength of the diaphragm, although the extent to which the boundary condition parameter influenced the behavior was dependent upon whether or not the systems were topped.

2.5.3 Plank thickness comparisons

Another parameter of particular interest was the thickness of the plank diaphragm. Tests #4, #9, and #11 were all connected in a similar fashion, but they were all different thicknesses. In addition, Tests #6 and #10 were
identical with the exception of the thickness of the planks. A review of these tests allows for the study of the effects of plank thickness on the behavioral characteristics of the system.

3.5.3.1 Comparison of Tests #4, #9 and #11

Data from the stiffness plot comparison for Tests #4, and #9 (shown in Figure 31) indicates that from 0.0125- to 0.05-inch displacements, stiffnesses were greater for Test #9, contrary to expectations. A higher compressive strength of the edge zone grout may have triggered this occurrence. However, from 0.05- to 0.75-inch displacements, the diaphragms yielded slightly greater stiffness for Test #4. Differences were negligible after 0.75-inch values. Also shown in this figure is a comparison between these tests and Test #11. Test #11 yielded a considerable increase in initial stiffness over both Test #4 and #9 and continued this trend through displacements up to 0.3 inch. Greater stiffness was expected for Test #11, because of the larger plank depth.

In comparing envelope curves for the virgin/stabilized loads versus displacements for Tests #4 and #9 (Figure 32), similarities in contour are found. While a maximum value of 78.4 kips was attained in Test #9, 90.5 kips was the peak for Test #4. Given the fact that both tests utilized the same configuration, and that maximum strength was associated with impending failure of the first set of seam weld ties, peak
Figure 31. Cyclic stiffness for Tests #4, #9 and #11
Figure 32. Envelope curves for Tests #4, #9, and #11
strengths should have been comparable. Actually, this increase could be attributable to the depth parameter. With the greater 8-inch depth, greater frictional resistance in the seams developed as the displacements were induced. Also, the FME loads for Test #4 and #9 differed by a similar margin of 11.4% with respect to Test #4. The FME loads were 88.0 kips and 78.0 kips for Test #4 and #9, respectively. Since the seam shear-bond failure mode controlled for both tests, a proportional increase in the FME strengths should have reflected the grout penetration depths. Typical depths were approximately 6.5 and 5 inches for the 8- and 6-inch planks, respectively. (This would account for the 23.1% difference.) As expected, the thicker 8-inch diaphragm in Test #4 revealed a larger load capacity than the 6-inch diaphragm Test #9.

Test #11 can also be compared with Test #4 and #9. Since the FME strength of Test #11 resulted from the diagonal tension failure mode, a direct comparison of these numbers cannot realistically be of value. The 12-inch diaphragm system in Test #11 did yield a higher peak strength by 40.2% and 53.7% than Test #4 and #9, respectively. The diagonal tension failure mode combined with the shear-bond failure of the seams severely damaged the diaphragm system, such that at the 1.5-inch displacement, less than 50% of the peak capacity
was achieved during the virgin cycle. At the 2-inch displacement cycle this value fell to just 10% of the peak capacity.

3.5.3.2 Comparison of Tests #6 and #10

The stiffness for Test #10 was generally less than the stiffness recorded in Test #6 as is shown in Figure 33. For displacements up to 0.05 inch, the stiffness values of Test #10 were essentially the same as those for Test #6. Between the 0.075- and 0.75-inch displacements, the stiffness of Test #10 was approximately 30% less than that recorded for Test #6. The stiffness of Test #6 was about twice the stiffness of Test #10 for the 1- and 3-inch displacements. This is attributed to the increased plank depth of Test #6 over Test #10 (8 versus 6 inches).

A comparison of the virgin/stabilized curves, as shown in Figure 34, indicates that the diaphragm in Test #6 recorded a larger load for similar displacements than Test #10. The FME in Test #6 was at a displacement of 0.025 inches and a load of 31.9 kips; however, the FME in Test #10 occurred at a displacement of 0.035 inches and a load of 41.2 kips. The peak strengths for Tests #6 and #10 were 78.6 and 53.0 kips, respectively. This represents a 33.0% decrease in load for Test #10 with respect to Test #6. Both the north and south seams broke very early in the testing program of Test #6. Only the south seam fractured at the FME for Test #10.
Figure 33. Cyclic stiffness for Tests #6 and #10
Figure 34. Envelopes curves for Tests #6 and #10
3.5.4 Topping comparisons

The addition of the 2-inch topping changes the diaphragm characteristics drastically. The following sections will discuss the effect of the topping on the plank diaphragm characteristics.

3.5.4.1 Comparison of Tests #4 and #12

Data from the stiffness versus cyclic displacement plots for Tests #4 and #12 (see Figure 35) indicate that stiffness were greater for Test #12 for all displacements. The initial stiffness for Test #12 was 1596 kips/in. which represents an increase of 24.6% with respect to the Test #4 initial stiffness of 1281 kips/in. An increase in Test #12 stiffness was expected due to the additional 2-inch topping on the Test #12 diaphragm.

Figure 36 is a comparison plot of Tests #4 and #12 virgin/stabilized envelope curves. The general shape of the graph demonstrates that larger loads were recorded for Test #12 than for Test #4, through the 1-inch displacement increment. The ultimate strength for Test #12 of 135.8 kips represents a 50% increase with respect to the maximum load of 90.5 kips for Test #4. Limit state strengths were 119.7 kips and 70.0 kips for Tests #12 and #4, respectively. The increase in load capacity of Test #12 can be attributed to the 2-inch nominal thickness topping, which provided for a 62%
Figure 35. Cyclic stiffness of Tests #4 and #12
Figure 36. Envelope curves for Tests #4 and #12
increase in average slab depth. The FME for Test #12 was recorded at a 0.3-inch displacement and a load of 127.5 kips (diagonal tension failure). The seams did not crack during Test #12 (except during the late stages of testing) indicating the absence of a shear-bond or tensile-bond failure. The FME load for Test #4 was 88.0 kips and resulted from a shear-bond failure at the seam and grout interface.

3.5.4.2 Comparison of Tests #5 and #13

Tests #5 and #13 were oriented with the diaphragm seams transverse to the loading beam with all four sides of the planks fastened to the testing frame. Test #5 had a plank thickness of 8 inches, while Test #13 had a plank thickness of 8 inches plus a 2-inch topping.

The failure mode for Test #5 was seam shear-bond failure at an FME load of 84.0 kips. All seams failed during this test. The failure mode for Test #13 was diagonal tension with no seam failures recorded. The FME load of Test #13 was 230.4 kips, which represents a 174% increase with respect to Test #5. The addition of the topping with Test #13 causes a significant and beneficial behavior alteration.

Figure 37 is a comparison plot of the cyclic stiffness for Tests #5 and #13. Test #13 generally had higher stiffness values than Test #5 due to the addition of the topping. The initial stiffness for Test #13 was 2698 kips/in., which
Figure 37. Stiffness comparison of Tests #5 and #13
represents an increase of 34.9% with respect to Test #5 stiffness of 2005 kips/in.

The virgin/stabilized displacement curves for Tests #5 and #13 are shown in Figure 38. The general shape of the curves show that a larger load was recorded for Test #13 displacements, especially during the range from 0.1 to 0.75 inch. After displacements of 1 inch, the recorded loads are similar for both tests. The maximum load recorded for Test #13 was 295.6 kips compared to an ultimate load of 109 kips attained during Test #5, indicating the potential strength benefits of adding a topping to the diaphragm system.

3.5.4.3 Comparison of Tests #6 and #14

Data from the stiffness versus cyclic displacement plots for Tests #6 and #14, see Figure 39, indicate that stiffness were greater for Test #14 for all displacements. The initial stiffness for Test #14 was 3289 kips/in., which represents an increase of 123% with respect to the Test #6 initial stiffness of 1474 kips/in. The increase in stiffness that the topping contributed was quite significant.

The FME load for Test #14 of 260.8 kips was reached as the diaphragm failed in diagonal tension mode. In contrast, Test #6 was a seam tensile-bond failure and yielded a FME load of 31.9 kips. Figure 40 is a comparison plot of the virgin/stabilized envelope curves for Tests #6 and #14. Test #14 generally recorded a much larger load for displacements
Figure 38. Envelope curve comparisons of Tests #5 and #13
Figure 39. Cyclic stiffness for Tests #6 and #14
Figure 40. Envelope curve comparisons of Tests #6 and #14
than the loads for Test #6. The maximum load for Test #14 was 302.0 kips, which represents a 284% increase with respect to the maximum strength for Test #6 of 78.6 kips. This increase in capacity for Test #14 was expected due to the additional 2-inch topping.

3.5.5 Masonry and steel frames comparisons

Test #16 can be compared to Tests #13 and #14. All of these tests consisted of 8-inch planks and 2-inch topping. Test #16 and Test #13 planks were oriented in the north-south, while Test #14 planks were oriented in the east-west direction. The orientation of the untopped planks were found to have negligible effect on the topped diaphragm characteristics. Therefore, both tests (#13 and #14) can be used for comparison purposes. The frame used for Tests #13 and #14 consisted of steel sections (refer to Figure 10), while the frame used for Test #16 consisted of masonry walls (see Figure 11).

Figure 41 shows the cyclic stiffness comparison of the three tests. The cyclic stiffness for Test #16 was smaller than that for Tests #13 and #14 up to 0.75-inch displacement. This can be attributed to the lower stiffness of the masonry wall frame as compared to the steel frame. After the 0.75-inch displacement the stiffness of all three tests was essentially the same.
Figure 41. Stiffness comparison of Tests #13, #14, and #16
Figure 42 shows the envelope curves comparison of the three tests. The peak load was similar for both Tests #13 and #14, but slightly lower for Test #16. The lower value of the peak load for Test #16 is attributed to the failure of the connection between the side and the back walls [25]. The initial stiffness of all three tests was within 15% of each other.

3.6 Summary of Experimental Results

A summary of test results for the diaphragms which are included in this study is given in Table 3. Included in this table are the load and displacement at the limit state, the load at first major event and the initial stiffness for each test.

Several comparisons were made in order to study the boundary condition parameter. Boundary conditions are extremely important in achieving maximum diaphragm action. A definite correlation between the number of sides connected and the amount of diaphragm action achieved was determined. Connecting four sides results in the greatest strength and stiffness and is thus the most desirable.

Comparisons involving the orientation of the planks within the diaphragm revealed that the most diaphragm action was obtained by placing the planks transverse to the applied shear load. Higher strengths and stiffnesses were obtained in
Figure 42. Envelope curve comparison for Tests #13, #14 & #16
Table 3. **Diaphragm Test Results**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Stiffness (Kips/in)</th>
<th>FME Load (Kips)</th>
<th>Ultimate Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Displacement (in.)</td>
</tr>
<tr>
<td>1</td>
<td>1375</td>
<td>70</td>
<td>---</td>
</tr>
<tr>
<td>2</td>
<td>675</td>
<td>68</td>
<td>---</td>
</tr>
<tr>
<td>3</td>
<td>504</td>
<td>22</td>
<td>---</td>
</tr>
<tr>
<td>4</td>
<td>1281</td>
<td>88</td>
<td>0.496</td>
</tr>
<tr>
<td>5</td>
<td>2005</td>
<td>84</td>
<td>0.490</td>
</tr>
<tr>
<td>6</td>
<td>1376</td>
<td>32</td>
<td>0.500</td>
</tr>
<tr>
<td>7</td>
<td>1647</td>
<td>20</td>
<td>0.133</td>
</tr>
<tr>
<td>8</td>
<td>716</td>
<td>19</td>
<td>0.498</td>
</tr>
<tr>
<td>8B</td>
<td>1003</td>
<td>43</td>
<td>0.099</td>
</tr>
<tr>
<td>9</td>
<td>1486</td>
<td>78</td>
<td>0.151</td>
</tr>
<tr>
<td>10</td>
<td>1367</td>
<td>41</td>
<td>0.303</td>
</tr>
<tr>
<td>11</td>
<td>2143</td>
<td>118</td>
<td>0.303</td>
</tr>
<tr>
<td>12</td>
<td>1569</td>
<td>127</td>
<td>0.292</td>
</tr>
<tr>
<td>13</td>
<td>2698</td>
<td>230</td>
<td>0.433</td>
</tr>
<tr>
<td>14</td>
<td>3298</td>
<td>261</td>
<td>0.440</td>
</tr>
<tr>
<td>15</td>
<td>2518</td>
<td>98</td>
<td>0.514</td>
</tr>
<tr>
<td>16</td>
<td>3064</td>
<td>271</td>
<td>0.310</td>
</tr>
</tbody>
</table>

**Notes:**

- Test #1 experienced hydraulic surge.
- Test #3 had only two planks and experienced rigid body motion.
- Test #4 is a duplicate of Test #1.
- Test #16 utilized masonry walls.
those systems which were oriented transverse to the loading beam. One possible theory associated with this phenomenon is that the diaphragms with planks oriented parallel to the applied shear load initially crack primarily in tension across the seam joints. This event is most clearly demonstrated in Test #8. The FME and other seam cracks in this test were documented as being caused by a tensile splitting of the seam between the two southernmost planks. Diaphragms #6 and #7 also revealed a similar situation although not to the extent shown in Test #8. The boundary conditions of the two latter tests prohibited the dramatic tensile cracking which Test #8 demonstrated. With the seams of the planks oriented transverse to the applied shear load, the system cracked in shear-bond between the planks. Although tension was present at these locations, it was not the controlling mode of failure. Calculations will be presented in Chapter 4 which will validate this theory.

A study of the thickness parameter revealed several conclusions. For untopped planks, the greater the diaphragm depth, the greater the diaphragm strength and stiffness. The 12-inch depth demonstrated the failure mode of diagonal tension prior to also failing in shear-bond. This diagonal tension mode hastened deterioration of the diaphragm energy absorption capacity.
A review of the addition of the 2-inch topping, revealed that topping creates a much stiffer and stronger system. The general effects of the other parameter, boundary conditions, is still apparent and similar to those observed in the untopped tests. However, the addition of the topping appears to nullify the orientation parameter.

The effect of the type of the supporting frame was considered. The failure of the connection between the side and back wall, for the diaphragm with masonry wall frame, limited the capacity of the diaphragm (beyond the FME). However, the FME and Peak loads were not greatly affected. This comparison suggest that the results of all the tests can be applied to the masonry buildings without modification except for the stiffness of the supporting members.
4. ANALYTICAL PREDICTIVE METHODS

The object of this chapter is to analytically determine the behavioral characteristics of hollow-core, prestressed plank diaphragms. These are defined as: 1) Initial stiffness, 2) First Major Event load, and 3) Limit state load. The theoretical development is based on the edge zone concept, [25,27,28] and the geometry and equilibrium of the planks. Finite element analysis was performed to validate the assumed stress distribution between adjacent planks as will be discussed later. Comparisons of the analytical and experimental results are presented to verify the analysis techniques.

4.1 Initial Stiffness

The initial stiffness calculations are based on the edge zone concept [25,27,28] developed at Iowa State University and modified herein to accommodate the plank diaphragms. The initial stiffness calculation is based on the assumption that plank displacement is a resultant of four displacement components:

a- Bending component $\Delta_a$

b- Shear component $\Delta_s$

c- Edge zone deflection component $\Delta_e$

d- Plank Deflection relative to the framing beam $\Delta_p$
\[ \Delta_{tot} = \Delta_b + \Delta_s + \Delta_z + \Delta_t \quad 4-1 \]

Since the displacements are related to the shear force by the stiffness factor, \( K \), then

\[ \Delta_b = \frac{V}{K_b} \quad 4-2 \]

\[ \Delta_s = \frac{V}{K_s} \quad 4-3 \]

\[ \Delta_z = \frac{V}{K_z} \quad 4-4 \]

\[ \Delta_t = \frac{V}{K_t} \quad 4-5 \]

and

\[ \Delta_{tot} = \frac{V}{K_{tot}} \quad 4-6 \]

it follows that:

\[ \frac{V}{K_{tot}} = \frac{V}{K_b} + \frac{V}{K_s} + \frac{V}{K_z} + \frac{V}{K_t} \quad 4-7 \]

or

\[ \frac{1}{K_{tot}} = \frac{1}{K_b} + \frac{1}{K_s} + \frac{1}{K_z} + \frac{1}{K_t} \quad 4-8 \]

### 4.1.1 Evaluation of the bending stiffness component

The diaphragms are assumed to act as a composite cantilever beam, the cross section of which is composed of the planks as the web and the loading beams as the flanges.
For a cantilever beam:

\[ \Delta_b = \frac{V a^3}{3 (E_c I_c + E_b I_b)} \times \frac{V}{K_b} \]  

where

- \( E_c \) = Modulus of elasticity of concrete planks, Ksi
- \( I_c \) = Moment of inertia of concrete planks, in^4
- \( E_b \) = Modulus of elasticity of the framing beam, Ksi
- \( I_b \) = Moment of inertia of framing beam, in^4

When calculating the moment of inertia of the planks, the area of the prestressing bars are transformed and added to the concrete cross section only if the prestressing is perpendicular to the loading direction. The effect of the topping is considered by transforming the thickness of the topping to an equivalent area of planks.

4.1.2 Evaluation of the shear stiffness component

The planks are again considered as a cantilever beam.

Therefore,

\[ \Delta_s = \frac{V a}{b G_c t_c} \times \frac{V}{K_s} \]  

where

- \( V \) = Applied shear force, kips
- \( a \) = Length of the planks perpendicular to the load direction, in
- \( b \) = Length of the planks parallel to the load direction, in
- \( G_c \) = The shear modulus of elasticity of the planks, Ksi
93

\( t_e \) = The equivalent thickness of the planks, in this derivation assumes that for the composite beam, only the web (planks) resist the shear.

4.1.3 Evaluation of the edge zone stiffness component

The edge zone concept states that the force is transformed from the loading beams to the diaphragm within a relatively very short distance. Thus, the edge zone is defined as the distance, from the edge, at which 95\% of the load is transferred to the diaphragm. A brief derivation is included here for reference.

As shown in Figure 43, a finite segment of the loading beam is considered. Thus summing forces in the horizontal direction

\[
K \Delta (x) \, dx = A_b \, d\sigma
\]  \hspace{1cm} 4-11

\[
K \Delta (x) = A_b \frac{d\sigma}{dx}
\]  \hspace{1cm} 4-12

\[
K \Delta (x) = A_b \sigma'
\]  \hspace{1cm} 4-13

The axial deformation in the beam must equal the deformation in the planks,

\[
\Delta'(x) = \frac{\sigma}{E_b}
\]  \hspace{1cm} 4-14

\[
\Delta''(x) = \frac{\sigma'}{E_b}
\]  \hspace{1cm} 4-15
Figure 43. Horizontal forces on typical edge beam segment
\[ \Delta''(x) = \frac{K \Delta(x)}{A_b E_b} \quad 4-16 \]

\[ \Delta''(x) - g^2 \Delta(x) = 0 \quad \text{where} \quad g = \sqrt{\frac{K}{E_b A_b}} \quad 4-17 \]

Which is the governing differential equation. Solving for each edge beam separately, and substituting the boundary conditions, the edge force distribution is found. For the loading beam (front beam), the boundary conditions are:

\[ \sigma(0) = 0 \quad \text{and} \quad \Delta(0) = \Delta_{tto} \]

the solution becomes

\[ \Delta(x) = \Delta_{tto} \cosh(g_t x) \quad 4-18 \]

and since

\[ q_{tto} = K_t \Delta_{tto} \quad 4-19 \]

\[ q_{tb} = K_t \Delta_{tb} \]

\[ q_{tt1} = K_t \Delta_{tt1} \quad 4-21 \]

where,

\[ q_{tto} = \text{edge shear stress at center of loading beam} \]

\[ \Delta_{tto} = \text{slab to edge beam relative displacement at the center of the loading beam (parallel to applied shear)} \]

\[ K_t = \text{equivalent edge spring stiffness parallel to the applied shear force} \]

\[ q_{tb} = \text{slab to restrained end relative displacement} \]

\[ q_{tt1} = \text{edge shear stress at end of loading beam} \]
\( \Delta_{\text{rel}} = \) slab to edge beam relative displacement at end of loading beam (parallel to applied shear)

therefore,

\[ q_t(x) = q_{t0} \cosh(g_t x) \quad 4-22 \]

and since \( q_t = q_{t0} \) at \( x = b/2 \), then

\[ q_{t0} = q_{t0} \cosh(g_t \frac{b}{2}) \quad 4-23 \]

\[ q_{t0} = q_{t0} \cosh(g_t x) \quad 4-24 \]

where

\[ g_t = \sqrt{\frac{k_t}{E_p A_b}} \quad 4-25 \]

following similar procedure, it follows

\[ q_p(x) = q_{p1}\text{sech}(g_p a) \cosh(g_p(x - a)) \quad 4-26 \]

\[ q_p = q_{p1}\text{sech}(g_p a) \quad 4-27 \]

where

\[ g_p = \sqrt{\frac{K_p}{E_p A_b}} \quad 4-28 \]

Integrating \( q_t(x) \), \( q_p(x) \) and dividing by the respective total length, we get

\[ q_{t, \text{avg}} = \frac{2q_{t0}}{bg_t} \sinh(g_t \frac{b}{2}) \quad 4-29 \]
For simplification define the following variables

\[ r_1 = \frac{\Delta p_1}{\Delta_{pav}} = \frac{q_{p1}}{q_{pav}} = g_p \coth(g_p a) \]

\[ r_2 = \frac{\Delta p_2}{\Delta_{pav}} = \frac{q_{p2}}{q_{pav}} = g_p \csch(g_p a) \]

\[ r_3 = \frac{\Delta tf_1}{\Delta_{tfav}} = \frac{q_{tf1}}{q_{tfav}} = \frac{b}{2} g_c \coth(g_c \frac{b}{2}) \]

Figure 44 shows the resulting edge zone stress distribution. The forces in the edge beams and the restrained end are shown in Figure 45. The total edge zone deflection can be determined from static and geometry. Summing the forces on the front edge beam results in:

\[ V = q_{tfav}(b + r_3 a''') - q_{tb} a'' \]

Similarly, summing forces on the restrained edge gives

\[ V = q_{tb} b + \frac{a''}{3a} (q_{tb} (3a - 3a'') - q_{tfav} r_3 a''') \]

Combining these two equations, yields

\[ q_{tfav}(b + r_3 a'''(2 - \frac{a''}{3a})) = q_{tb}(b + a'') \]

Letting
Figure 44. Edge zone stresses in the elastic range [26]
Figure 45. Initial forces in the framing members [26]
\[ r_t = \frac{\Delta_{tb}}{\Delta_{tfa}} = \frac{q_{tb}}{q_{tfa}} = \frac{b + r_3 a''(2 - \frac{a''}{3a})}{b + a''} \quad 4-37 \]

\[ l''_t = \frac{a''}{3a} (r_3 (3a - a'') - a''r_4) \quad 4-38 \]

and solving for \( q_{tb} \), Equation 4-34 becomes

\[ V = q_{tfa} (b + l''_t) \quad 4-39 \]

Summing moments at the south end of the restrained edge yields

\[ V = q_{pav} (r_1 \frac{b^2}{6a} + b + \frac{r_2 b''}{6a} (3b - 2b'')) \quad 4-40 \]

and letting

\[ l''_p = \frac{r_1 b^2 + 3r_2 b b'' - 2r_2 b''^2}{6a} \quad 4-41 \]

Equation 4-40 becomes

\[ V = q_{pav} (b + l''_p) \quad 4-42 \]

Figure 46 shows the geometrical relation between \( \Delta_{tfa}, \Delta_{tb}, \Delta_{p1} \), and \( \Delta_t \). The total edge zone displacement is separated into transverse and parallel edge zone displacements. Addition of the two contributions results in

\[ \Delta_z = \Delta_{tfa} + \Delta_{tb} + \frac{2a \Delta_{p1}}{b} \quad 4-43 \]

By substituting Equation 4-43 into Equations 4-31, 4-33, 4-35, 4-39 and 4-42, the following expression is obtained
Figure 46. Edge zone displacements [28]
The total edge zone stiffness is the reciprocal of the right hand side of Equation 4-44, or

\[ K_e = \frac{1}{\frac{r_3 + r_4}{K_c(b+l'_c)} + \frac{2a r_1}{K_p(b+l''_p)}} \]  \hspace{1cm} 4-45

The equivalent edge zone spring stiffness, \( K_e \) or \( K_p \), is determined using the empirical stud load/displacement relation from Reference 28 as follows

\[ K_{eq} = 145.3 \frac{Q_{su}}{S_s} \]  \hspace{1cm} 4-46

where

- \( K_{eq} \) = equivalent stiffness, \( K_e \) or \( K_p \)
- \( Q_{su} \) = stud connector capacity in the load direction
- \( S_s \) = stud spacing

The stud load/displacement relation requires the stud connector capacity, \( Q_{su} \), the following relation predicts this capacity [32]:

\[ Q_{su} = 6.66 \times 10^3 A_s f_c^{0.3} E_c^{0.44} \leq 0.9 A_s f_c \]  \hspace{1cm} 4-47

where

- \( A_s \) = area of stud
- \( E_c \) = modulus of elasticity of the concrete
\[ f' = \text{compressive strength of concrete} \]
\[ f_y = \text{yield strength of the studs} \]

### 4.1.4 Evaluation of the framing members component

The final component of the diaphragm initial stiffness was the axial flexibility of the framing members. This component served as a correction of the movement of the framing members and its connections and has been determined experimentally to be 10,000 kips/in [25,27,28].

### 4.1.5 Evaluation of the initial stiffness

The total stiffness was calculated by substituting the values of the individual components into Equation 4-8. This equation is simplified to

\[ K_{tot} = \frac{1}{\frac{1}{K_b} + \frac{1}{K_s} + \frac{1}{K_z} + \frac{1}{K_r}} \] 4-48

### 4.2 FME Strength Predictions

The FME load capacity of the diaphragm was limited by one of three failure modes: shear-bond, tensile-bond and diagonal tension failure. The shear-bond failure mode dominated for the untopped north-south oriented planks. The tensile bond failure mode dominated the east-west untopped tests. Finally the topped planks failed by diagonal tension mode. The untopped planks should be checked for their respective failure
mode (based on orientation) and in addition the diagonal failure mode.

4.2.1 Finite element analysis

The stress distribution between adjacent planks is required in order to evaluate the FME and limit state loads. A general purpose finite element analysis program (ANSYS), was used to check the validity of the assumed normal and shear stress distributions. The diaphragm was divided into 400 two-dimensional, isoparametric, solid stress elements. Spring elements were used to model the seam interaction between the planks. Two models were used, one for each direction. The finite element models are shown in Figures 47 and 48. The element properties were based on Test #5 for The north-south oriented planks and Test #6 for the east-west planks. The resulting stress distributions between the planks are shown in Figures 49 through 52.

4.2.2 FME prediction for north-south oriented planks

The stress distribution between two adjacent planks is assumed to be as shown in Figure 53. The derivation of the FME load follows below:

from Equation 4-42 $q_{pa}$ can be written as

$$q_{pa} = \frac{V}{b + l_p}$$ 4-49
Figure 47. Finite element model of north-south oriented planks
Figure 48. Finite element model of east-west oriented planks
Figure 49. Normal stress distribution for north-south model
Figure 50. Shear stress distribution for north-south model
Figure 51. Normal stress distribution for east-west model
Figure 52. Shear stress distribution for east-west model
Figure 53. FME force distribution on exterior plank for north-south oriented planks
Referring to Figure 53 and summing forces in the north-south direction, we get

\[ V_{seam} = q_{pav} a + \frac{1}{2} q_{p2} b'' + \frac{q_{p1}}{2} \left( 1 + \frac{96}{b} \right) \left( \frac{b}{2} - 48 \right) \]  \[ \text{(4-50)} \]

or

\[ V_{seam} = q_{pav} a + \frac{1}{2} q_{p2} b'' + \frac{q_{p1}}{b} \left( \frac{b^2}{4} - 48^2 \right) \]  \[ \text{(4-51)} \]

Also

\[ V_{seam} = T'_{av} d_p l_s \]

\[ \text{(4-52)} \]

equating Equations 4-51 and 4-52, we get

\[ T'_{av} d_p l_s = q_{pav} a + \frac{1}{2} q_{p2} b'' + q_{p1} \left( \frac{b^2}{4} - 48^2 \right) \]  \[ \text{(4-53)} \]

substituting the values of \( q_{pav} \) and \( q_{p1} \) in terms of \( V \), we get

\[ T'_{av} d_p l_s = \frac{V}{b + l''_p} + \frac{V b'' a g_p}{2 (b + l''_p) \tan (g_p a)} \cdot \frac{\text{sech} (g_p a)}{\tanh (g_p a)} + \frac{V a g_p (b^2 - 48^2)}{b (b + l''_p) \tanh (g_p a)} \]  \[ \text{(4-54)} \]

for simplification, let

\[ r_5 = 1 + \frac{b'' g_p \text{sech} (g_p a)}{2 \tanh (g_p a)} + \frac{g_p \left( \frac{b^2}{4} - 48^2 \right)}{b \tanh (g_p a)} \]  \[ \text{(4-55)} \]

Equation 4-54 becomes

\[ T'_{av} d_p l_s = \frac{V a}{b + l''_p} r_5 \]  \[ \text{(4-56)} \]

and solving for \( V \), which is the predicted FME load
where

\[ V_{FME}^{p} = \frac{T'_{s} d_{p} l_{s} (b + l'_{p})}{a r_{s}} \]

4-57

\[ V_{FME}^{p} = \text{Predicted FME load for north-south oriented planks} \]
\[ T'_{s} = \text{Seam shear strength from elemental tests} \]
\[ d_{p} = \text{Grout depth} \]
\[ l_{s} = \text{Length of the seam} \]

4.2.3 FME prediction for east-west oriented planks

The forces on the south plank are shown in Figure 54. Utilizing the equations developed earlier for the distribution of \( q_{t} \) and \( q_{p} \), we find

\[ q_{p}^{*}(x) = q_{p} z \text{sech}(g_{p} a) \text{cosh}(g_{p} (x-a)) \]

4-58

where \( x \) is measured from the restrained end, or

\[ q_{p}^{*}(x) = q_{p} z \text{sech}(g_{p} a) \text{cosh}(g_{p} (z-a)) \]

4-59

In the case of this study \( z \) is equal to 42" (distance from the first stud to the end of the south plank), therefore, the total shear force \( Q_{p} \) is defined as

\[ Q_{p} = \int_{0}^{z} q_{p}^{*}(x) \, dx \]

4-60

and substituting \( q_{p}^{*} \) from Equation 4-27, then

\[ Q_{p} = \frac{q_{p}^{2}}{g_{p}} [(\sinh(-g_{p} a) - \sinh(g_{p} (z-a))] \]

4-61
Figure 54. PME force distribution on south plank for east-west oriented planks
referring to Figure 54, and summing the moments at west end of seam we get

\[ 2 \left( q_{cb} \frac{a''}{2} \frac{b}{2} + q_{cb} b z + q_{pl} b \frac{b}{4} \frac{b}{4} - q_{pl} b \frac{b}{4} \frac{b}{6} \right) \]

\[-Q_{p} b + N_{c} \left( b - \frac{L_{c}}{3} \right) - N_{t} \frac{L_{c}}{3} = 0 \]

since \( N_{c} = N_{t} \), because of symmetry, therefore, solving for \( N_{c} \)

\[ N_{c} = \frac{-3V}{3b - L_{c} - L_{c}} \]

where

\[ r_{6} = \frac{r_{4}}{b + L_{c}^{''}} \left( z a'' - a'' a^{''} + z b \right) \]

\[-a \sech(g_{p} a) \left( \sinh(-g_{p} a) - \sinh(g_{p}(z-a)) \right) \]

solving for \( V \), we get

\[ V = \frac{N_{c}(3b-L_{c}-L_{c})}{-3r_{6}} \]

for tensile-bond failure to occur, the stress in the seams must exceed the tensile strength as determined by elemental tests, or

\[ \sigma_{tav} = \frac{N_{c}}{d_{p}L_{t}} \]

substituting in Equation 4-65 the predicted FME load is obtained

\[ V_{FME}^{p} = \sigma_{tav} d_{p} \frac{3b-L_{c}-L_{c}}{-3r_{6}} \]
The shear stress in the seam should also be checked so as not to exceed the capacity as determined by the elemental tests. Therefore, summing forces parallel to the seam, we get

\[ V_{\text{seam}} = b q_{tb} + 2 \left( q_{tb} \frac{a''}{2} \right) \]  

4-68

substituting for \( V_{\text{seam}} \) from Equation 4-52, into Equation 4-68, we get

\[ q_{tb} = \frac{T_{av} d_p l_s}{b + a''} \]  

4-69

from Equation 4-37

\[ q_{tb} = I_L q_{t,\text{av}} \]  

4-70

and from Equation 4-39

\[ q_{t,\text{av}} = \frac{V}{b + l_t} \]  

4-71

solving for \( V_{\text{FME}}^{p} \) the predicted FME load can be expressed as:

\[ V_{\text{FME}}^{p} = \frac{T_{av} d_p l_s (b + l_t)}{b + a''} \]  

4-72

Both Equations 4-67 and 4-72 should be checked and the lowest of the two values is used.

4.3 Limit State Prediction

The edge zone stress distribution on the planks at the limit state is shown in Figure 55. The resulting forces on the framing members is illustrated in Figure 56. These
Figure 55. Limit state edge zone stress distribution [28]
Figure 56. Framing members forces at limit state [28]
figures will be used to determine the limit state loads for the plank diaphragms as shown in the following sections.

4.3.1 Limit state prediction for north-south planks

The stress distribution of Figure 57 was assumed for the limit state condition. However, due to the symmetry of this system of forces, normal forces acting at the seam could not be determined directly. Therefore, the FME force distribution was used for normal force computations. This normal force was assumed to vary linearly across the seam with compression at the south end and tension at the north end as the loading beam moved to the west, Figure 58. The length of the seam compression zone, $l_c$, was assumed to be $a/2$. The length of the tension zone, $l_t$, was assumed to be $a - l_c$. The force $Q_{tt}$ is obtained by integration

$$Q_{tt} = \int_{0}^{b/2} q_{tt}(x) \, dx$$  \hspace{1cm} 4.73$$

or

$$Q_{tt} = \frac{q_{tt}}{g_t} \left[ -\sinh(48g_t) + \sinh\left(\frac{g_tb}{2}\right) \right]$$  \hspace{1cm} 4.74$$

Referring to Figure 58 and summing forces transverse to the seam yielded

$$N_c = N_c - Q_{tt} + q_{cb} \left( \frac{b}{2} - 48 \right) + \frac{a''}{2} (q_{cb} - q_{tt})$$  \hspace{1cm} 4.75$$

Summing moments about the front loading beam at the seam gave
Figure 57. Limit state stress distribution for north-south oriented planks
Figure 58. FME force distribution used to calculate normal forces at limit state
The normal forces, \( N_c \) and \( N_e \), were determined by solving Equations (4-75) and (4-76) by substitution. The normal tensile force, \( N_e \), should not exceed the combined capacity of the exterior and center weld ties along the seam.

The shear capacity of the seam at the limit state, \( V_{\text{seam}} \), had three components: the capacity of the three weld ties in shear \( (F_{v(wt)}) \), the shear friction contribution due to the normal compressive forces \( (F_{r(c)}) \), and the weld tie frictional contribution due to self-inducing normal forces \( (F_{t(t)}) \). In equation form,

\[
V_{\text{seam}}^{l_g} = F_{v(wt)} + F_{f(c)} + F_{t(t)}
\]

Based on information from the elemental shear tests, the average weld tie shear capacity was 5.5 kips [24]. A value for the coefficient of friction, \( \mu \), acting in the seams was taken as 0.90. A tensile strength for the weld tie of 16.3 kips was calculated based on the horizontal and vertical bar contribution in tension [24]. Thus, the equation describing the seam capacity at the limit state was simplified to the following

\[
V_{\text{seam}}^{l_g} = 5.5n + 0.9(N_e + N_c)
\]

where \( n \) is the number of weld ties.
From the limit state framing members forces, Figure 56, the predicted limit state force, $V_{ls}$, was related to $q_p$ and $q_p'$. The stresses $q_p$ and $q_p'$ were assumed to be equal. Summing moments at the abutment (restrained edge) gave

$$V_{ls}^p = q_p b + \frac{q_p' b^2}{4a} + \frac{b'' q_p'}{a} (b-b'')$$ \hspace{1cm} 4-79

and letting $l_p' = \frac{(b^2 + 4bb'' - 4b''^2)/4a}$, yielded

$$V_{ls}^p = q_p (b + l_p')$$ \hspace{1cm} 4-80

From the limit state force distribution, Figure 55:

$$V_{seam}^{ls} = q_p (a + \frac{b}{2} - 42)$$ \hspace{1cm} 4-81

By substituting Equations (4-78) and (4-80) into Equation (4-81), $V_{ls}^p$ was determined to be

$$V_{ls}^p = \frac{5.5n + 0.9(N_e + N_l)}{(a + \frac{b}{2} - 42)} (b + l_p')$$ \hspace{1cm} 4-82

4.3.2 Limit state prediction for east-west planks

The normal forces determined from the PME distribution were utilized for the limit state condition, since the limit state distribution had not allowed for their computation. The limit state stress distribution on the exterior plank (south) is shown in Figure 59. Summing forces in the east-west direction
Figure 59. Limit state force distribution on south plank for east-west oriented planks
\[ V_{\text{seam}} = q_c b + 2q_c' a' \]  

Note that \( q_c \) and \( q_c' \) were assumed to be equal.

Referring to Figure 56 and summing forces on the loading (front) beam in the E-W direction, yielded

\[ V_{ls}^P = q_c b + \frac{2a'q_c'(a-a')}{a} \]  

Letting \( l_c' = 2a' - 2a'^2/a \), then

\[ V_{ls}^P = q_c'(b + l_c') \]  

Solving for \( V_{\text{seam}}^l \) in terms of \( V_{ls}^P \) gave

\[ V_{\text{seam}}^l = V_{ls}^P \frac{b + 2a'}{b + l_c'} \]  

Utilizing Equation (4-78) together with Equation (4-86) allowed for the simultaneous solution of \( V_{ls}^P \) as

\[ V_{ls}^P = \frac{(5.5n + 0.9(N_c+N_c))(b + l_c')}{(b + 2a')} \]  

Where \( n \) is the number of weld ties.

For the case where tension along the seam controlled, Equations (4-80) and (4-85) again applied. Summing moments about the west edge beam at the south seam (see Figure 59)

\[ N_c = \frac{1}{b - \frac{l_c}{3}} \left[ q_c (84 - 2a' + 42b) + q_p \left( \frac{b^2}{4} - 42b \right) - \frac{N_c l_c}{3} \right] \]  

and substituting from these equations gave
The predicted limit state shear strength, $V_{ls}$, was the smaller of that given by Equations (4-87) and (4-89).

4.3.3 FME and limit state prediction for diagonal tension failure

The diagonal tension failure represented an upper limit for a concrete diaphragm. This failure occurred for only one of the untopped and all the topped hollow-core diaphragm tests. Diagonal tension failure calculations were based on Equation (11-32) from the American Concrete Institute 318-89 code,

$$V_c = 3.3(f'_c)^{0.5}bd + \frac{N_{cp}d}{4l_v}$$

where

$V_c$ = diagonal shear capacity of the concrete.

$f'_c$ = plank concrete compressive strength, psi.

$b$ = diaphragm width, in.

$d$ = effective plank depth, in.

$N_{cp}$ = normal compressive force (prestressing), lb.

$l_v = 0.8b$

The determination of the effective plank depth, $d$, was very critical in this equation. The shear force flow was assumed
to follow that described in Figure 60. The shear force applied at the loading beam was transferred into the diaphragm through the edge zone. The following areas were assumed non-effective in resisting the in-plane force: the tension zone of the top wythe (if one existed), and the majority of the core web zone, excluding parabolic regions into each of the lower and upper webs.

In order to compute the extent of the non-effective tensile zone of the top wythe, fiber stresses in the top and bottom were determined, based on a linear stress distribution:

\[
f_t = -\frac{P_i}{A} - \frac{P_i e y_t}{I} - \frac{M_o y_t}{I}
\]

\[
f_b = -\frac{P_i}{A} + \frac{P_i e y_t}{I} + \frac{M_o y_b}{I}
\]

where

- \( f_t \) = top fiber stress, psi.
- \( f_b \) = bottom fiber stress, psi.
- \( P_i \) = compressive prestressing force (after relaxation losses), lb.
- \( A \) = cross sectional area of plank, in\(^2\).
- \( e \) = eccentricity of the strands with respect to the plank neutral axis, in.
- \( y_t \) = distance from neutral axis to the top fiber, in
- \( y_b \) = distance from neutral axis to bottom fiber, in.
- \( M_o \) = dead load moment, lbs. in.
Figure 60. Proposed shear force transfer system
When the top fiber was subjected to tension, a modification due to the effect of in-plane shear was considered. The shear stress was computed using

\[ v = \frac{VQ}{I_d t} \]  

where

- \( v \): shear stress at specified location, psi
- \( V \): shear on plank applied at loading beam, lbs.
- \( Q \): first moment of area of the diaphragm, in'.
- \( I_d \): moment of inertia of diaphragm, in'.
- \( t \): average cross-sectional area divided by plank width, in.

Mohr's circle was utilized as shown in Figure 61 to determine the modified tensile stress:

\[ f'_t = \frac{f_t}{2} + \left( \frac{f_t}{2} \right)^2 + v^2 \]  

where

- \( f'_t \): modified tensile stress, psi.

The effective zone of the top wythe subjected to compression, \( d'_{eff} \), was

\[ d'_{eff} = -1.25 \left( \frac{f_b}{f'_t - f_b} \right) \]  

The shear forces were assumed to transfer partially into parabolic regions of the webs between the cores. The following relationship describes this second degree curve:
Figure 61. Principal tensile stresses using Mohr's circle
\[ y_f = a_g X_c^2 \]

where

- \( y_f \) = vertical shear flow limit
- \( a_g \) = web shear flow gradient
- \( X_c \) = core-to-core spacing.

The effective depth which acted to resist the shearing force was computed as follows:

\[ d = d_{eff}^b + d_{eff}^t + d_{eff}^p \]

where

- \( d_{eff}^b \) = the effective zone of the bottom wythe subjected to compression, in.
- \( d_{eff}^p \) = the effective zone of the parabolic region actively transferring in-plane shear forces, in.

Figure 62 demonstrate graphically the effective depths for three cases:

a) 6-inch planks with 4 strands.

b) 12-inch planks with 6 strands.

c) 8-inch planks with 4 strands.

The diagonal shear strength calculated in Equation (4-90), representing the predicted FME strength, had an internal factor of safety. This factor of safety was approximately 1.15 for concrete with a compressive strength of 8300 psi.

The numerical strength results for the diagonal tension mode,
Figure 62. Effective area for 6-, 8-, and 12-inch planks
to be presented in Section 4.4.3.3, reflect the extraction of this factor of safety.

4.4 Comparison of Experimental and Analytical Results

The purpose of the analytical work was to develop predictive equations for the initial stiffness, FME strength and limit state strength for hollow-core concrete diaphragms. The following sections discuss the application of the equations described in the previous sections and compare the results with those from the experimental investigation.

4.4.1 Initial stiffness

The predicted initial stiffness was calculated according to Equation (4-48) and the results are summarized in Table 4. The bending stiffness component was calculated with Equation (4-9). In order to determine the modulus of elasticity for use in this equation, the strength of the concrete was required. The plank system consisted of three different concrete mixes: the plank concrete, the grout in the seams and the grout in the cores. The plank strength was used in the computations for bending and shear stiffnesses. For topped planks the topping was transformed into equivalent plank strength. The shear stiffness component was predicted according to Equation (4-10) and the edge zone component was calculated according to Equation (4-45). The stud spacing variable was assumed to reflect the outer two studs and equal
Table 4. Initial Stiffness Results

<table>
<thead>
<tr>
<th>Test</th>
<th>$K_o$</th>
<th>$K_a$</th>
<th>$K_r$</th>
<th>$K_e$</th>
<th>$K_{tot}$</th>
<th>$K_{oct}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9668</td>
<td>8293</td>
<td>1797</td>
<td>10000</td>
<td>1136</td>
<td>1375</td>
</tr>
<tr>
<td>2</td>
<td>8112</td>
<td>7637</td>
<td>1775</td>
<td>10000</td>
<td>1090</td>
<td>675</td>
</tr>
<tr>
<td>3</td>
<td>1724</td>
<td>4147</td>
<td>1013</td>
<td>10000</td>
<td>524</td>
<td>250</td>
</tr>
<tr>
<td>4</td>
<td>9807</td>
<td>8501</td>
<td>1861</td>
<td>10000</td>
<td>1167</td>
<td>1281</td>
</tr>
<tr>
<td>5</td>
<td>7846</td>
<td>7184</td>
<td>8090</td>
<td>10000</td>
<td>2040</td>
<td>2005</td>
</tr>
<tr>
<td>6</td>
<td>8377</td>
<td>8088</td>
<td>7268</td>
<td>10000</td>
<td>2081</td>
<td>1376</td>
</tr>
<tr>
<td>7</td>
<td>9114</td>
<td>8358</td>
<td>6075</td>
<td>10000</td>
<td>2024</td>
<td>1647</td>
</tr>
<tr>
<td>8</td>
<td>9891</td>
<td>8627</td>
<td>1760</td>
<td>10000</td>
<td>1670</td>
<td>716</td>
</tr>
<tr>
<td>9</td>
<td>9325</td>
<td>7780</td>
<td>1793</td>
<td>10000</td>
<td>1119</td>
<td>1486</td>
</tr>
<tr>
<td>10</td>
<td>7752</td>
<td>7025</td>
<td>6500</td>
<td>10000</td>
<td>1904</td>
<td>1367</td>
</tr>
<tr>
<td>11</td>
<td>11497</td>
<td>11029</td>
<td>1799</td>
<td>10000</td>
<td>1200</td>
<td>2144</td>
</tr>
<tr>
<td>12</td>
<td>12396</td>
<td>12375</td>
<td>1586</td>
<td>10000</td>
<td>1125</td>
<td>1596</td>
</tr>
<tr>
<td>13</td>
<td>10556</td>
<td>11798</td>
<td>6671</td>
<td>10000</td>
<td>2329</td>
<td>2698</td>
</tr>
<tr>
<td>14</td>
<td>10414</td>
<td>11555</td>
<td>6081</td>
<td>10000</td>
<td>2237</td>
<td>3288</td>
</tr>
<tr>
<td>15</td>
<td>8377</td>
<td>8088</td>
<td>6672</td>
<td>10000</td>
<td>2029</td>
<td>2518</td>
</tr>
<tr>
<td>8B</td>
<td>12363</td>
<td>12325</td>
<td>1771</td>
<td>10000</td>
<td>1210</td>
<td>1003</td>
</tr>
<tr>
<td>16</td>
<td>21471</td>
<td>13283</td>
<td>10979</td>
<td>10000</td>
<td>3195</td>
<td>3064</td>
</tr>
</tbody>
</table>
spacing between the remaining studs. Thus for Test #4, during which the diaphragm was not connected along the side beams, the spacing factor for the side beams was the full span, or 192". For an unsymmetrically connected specimen, Test #7, the average stud spacing for both sides was used. The compressive concrete strength used in the edge zone calculations was the Span-Deck plank strength or the core grout strength (grout around the studs) depending on the appropriate concrete being considered in the equation. The final component of the stiffness equation was the axial flexibility of the edge beam abutment connections. An experimentally derived value of 10,000 kips/in. was used as stated in References 25, 27 and 28.

Table 4 lists the intermediate stiffnesses as well as the total predicted stiffness for each of the diaphragm tests. The actual experimental values were computed using data from the initial increment of the loading beam displacement. The summation of the loads attained from both the east and west displacements were divided by the total absolute movement. These values, $K_{\text{act}}$, are listed in the final column.

The experimental stiffness for Test #2 may have been inaccurate due to the lack of adequate diaphragm connections. The actual initial stiffness for Tests #6 and #8 may have also been altered due to the initial false starts in the testing
procedure. Values for Test #11 differed somewhat due to the sensitivity of the seam grout compressive strengths. In general, the predicted stiffness values were quite acceptable.

4.4.2 FME loads

The edge zone force distribution discussed earlier was used to determine the predicted strength values. A lotus spreadsheet was developed to perform the calculations according to the equations derived earlier. The results of these calculations are presented in the next sections. In the diagonal tension failure mode calculations, the web shear flow gradient, \( a_g \), was selected to be 0.2 based on a visual interpretation of the flow area (see Equation 4-96, and Figures 60 and 62).

4.4.2.1 FME for north-south oriented planks

The predicted and experimental FME loads for the north-south oriented planks are presented in Figure 63. The analytical predictions are very close to the experimental results except for Test #3. Test #3 consisted of two planks only and experienced rigid body motion during the testing. Therefore, Test #3 is not considered to have adequate diaphragm action and the experimental results are not representative of the predicted capacity of the diaphragm. Figure 63 confirm the adequacy of the analytical procedure for all the other tests.
Figure 63. Predicted and experimental FME loads for north-south oriented planks
4.4.2.2 PME for east-west oriented planks

As illustrated in Figure 64, the FME loads predicted by the analytical equations are very close to the experimental results for the east-west oriented planks, with the exception of Tests #2 and #10. Test #2 was connected to the side beams only. This configuration does not provide good diaphragm action and the experimental results are not representative of the actual diaphragm capacity. Therefore, configuration similar to Test #2 are not recommended. Test #10 achieved a higher FME load than predicted. The analytical equations predict an FME load in the same range as that for Test #6, which is similar with the exception of the plank thickness. The analytical equations predicts the FME loads for the diaphragms with all sides connected more closely than those with two sides connected.

4.4.2.3 FME loads for topped planks

The topped planks failed in diagonal tension mode. The analytical equations compare the appropriate failure mode (based on orientation) to that established by the diagonal tension mode and the lower of the two values are used. Figure 65 illustrates the results of the predictive equations against the experimental results. The diagonal tension equation agrees closely with diaphragm test results when all sides are connected regardless of orientation. For diaphragm tests connected on two sides only the analytical equations
Figure 64. Predicted and experimental FME loads for east-west oriented planks.
Figure 65. Predicted and experimental FME loads for topped planks
overestimate the diaphragm capacity (Test #12). For Test #8B, the planks from Test #8 were reused. This seems to affect the experimental results greatly as can be seen from the lower FME and limit state loads for Test 8B. Also for tests connected on two sides only (loading beam and restrained end), the failure of the seam results in the planks acting individually rather than as a diaphragm. Therefore, the comparison of these results to the analytical equations is invalid for Test 8B, but it is still valid for all other tests.

4.4.3 Limit state loads

The predicted limit state loads are compared to the experimental results in the next sections. The limit state loads are the maximum stabilized load achieved by the diaphragm past the FME. The stabilized values are used since they represent the diaphragm capacity in an earthquake and since they are more likely to be reproduced in similar tests.

4.4.3.1 Limit state loads for north-south oriented planks

The analytical versus experimental results are presented in Figure 66. The analytical equations approximates the limit state loads very closely for these diaphragms with the exception of Test #15. Test #15 utilized fifteen weld ties per seam and the limit state load was that of diagonal tension failure. The sensitivity of the analytical equations to the weld tie capacity (15 weld tie for this test) could explain
Figure 66. Predicted and experimental limit state loads for north-south planks
the difference between the predicted and experimental limit state load. Further study of the weld ties is recommended to establish the capacity of the weld ties. This capacity affects the analytical prediction considerably when a large number of weld ties is used.

4.4.3.2 Limit state loads for east-west oriented planks

Figure 67 shows the results of the predictive equations versus the experimental results for planks oriented in the east-west direction. The analytical expressions approximate the experimental results except for Tests #8. The reason for the disagreement for Test #8 were discussed earlier in section 4.4.2.2.

4.4.3.3 Limit state loads for topped planks

Figure 68 depicts the comparison of the experimental and analytical limit state loads for topped planks. The results approximate the experimental results except for Tests #12 and #8B. Test #8B planks were used earlier in Test #8 which could explain the large difference in Test #8B results. This test had a very low values for the FME and limit state loads despite the diagonal failure mode. The experimental results for Test #12 indicate that for diaphragms with the planks connected on two sides only, the limit state load does not exceed the FME load. This can be explained by the lack of diaphragm action once the individual planks are separated.
Figure 67. Predicted and experimental limit state loads for east-west planks
Figure 68. Predicted and experimental limit state loads for topped planks
The only component holding the diaphragm together is the topping. The analytical equations do not cover this situation and further study for diaphragms of this orientation is recommended.
5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

This investigation into the behavioral characteristics of hollow-core planks subjected to in-plane loading was part of the overall Masonry Building Research Program being conducted by the Technical Coordinating Committee on Masonry Research (TCCMAR). The project was divided into four phases: loading of full-scale diaphragms into their limit state, testing elemental tension and shear specimens to determine seam characteristics, compilation of data, and development of an analytical model with accompanying initial stiffness and strength calculations. The purpose of this study was to ascertain the behavioral characteristics of the concrete plank diaphragms subjected to horizontal (in-plane) shear loading and to develop an analytical model to predict the initial stiffness, the First Major Event (FME) load, and the limit state load.

Seventeen full-scale diaphragm tests and sixty-six elemental tension and shear tests have been completed as part of this investigation [24,25]. The parameters that were tested included plank orientation, number of sides connected to the loading frame, addition of a 2-inch topping, plank thickness, number of seam fasteners, and effect of the testing frame stiffness. The behavioral characteristics identified
were: the initial stiffness, First Major Event (FME) load, limit state load, and failure mode.

Analytical equations describing the initial stiffness for the plank diaphragms were developed based on the edge zone concept. Finite element analyses were performed to verify an assumed stress distribution between individual planks. From the initial and ultimate force distributions and the assumed stress distribution between the planks, a static analysis yielded the predictive equations for the FME and limit state strengths. A comparison of the analytical and experimental results was presented, and conclusions and recommendations are derived and presented in the next sections.

5.2 Conclusions

The following conclusions were based on the investigation summarized above:

5.2.1 Experimental conclusions

1) Three failure modes were identified for the untopped diaphragms: seam shear-bond, seam tension-bond, and diagonal tension failure.

2) For untopped diaphragm tests oriented with seams transverse to the applied shear load, the shear-bond failure mode dominated.
3) For untopped diaphragm tests oriented with seams parallel to the applied shear load, the tensile-bond failure mode controlled.

4) For topped diaphragm tests, the diagonal tension failure mode governed.

5) The diagonal tension failure mode exhibited "low" strength capacities at high displacements due to the extensive cracks through the plank.

6) A study of the stiffness, FME and limit state strengths confirm a definite correlation between the number of sides connected and the amount of diaphragm action achieved. Diaphragms with three and four sides connected achieve higher diaphragm capacity.

7) For diaphragms with planks oriented parallel to the applied shear load and with only two sides connected, the failure of the seams reduce the limit state load drastically. This is due to the planks acting individually and not as a diaphragm once the seams fail.

8) The greatest amount of diaphragm action is achieved by orienting the planks transverse to the applied shear load.
9) Generally, the greater the diaphragm depth, the greater the strength and stiffness for the given orientation.

10) Increasing the plank depth increased the peak load. However, the ductility was adversely affected.

11) Weld ties provided a means of extending the total displacement capability of the diaphragm system by restructuring seam slippage and separation.

12) The increase of the number of seam fasteners, increases the diaphragm strength for untopped diaphragms. This parameter also leads to a change in the failure mode as observed in Test #15 where the failure mode changed from shear-bond to diagonal tension.

13) Diaphragms with planks connected to the side beams only (similar to Test #2) exhibit low strength and should be avoided.

14) The diaphragm with masonry wall exhibited similar strength and failure mode as those utilizing steel frame.

15) The connection between the diaphragm and the masonry walls for Test #16 withstood the applied load, thus forcing the failure to occur in the diaphragm assembly.
16) The failure of the connection between the side and back walls for Test #16 reduced the capability of the diaphragm to achieve peak load higher than the FME load.

5.2.2 Conclusions From Analysis

1) The edge zone concept was found to be valid and was utilized as the basis for calculating the initial and limit state force distribution systems.

2) From the elastic distribution, the initial stiffness were determined. Comparisons with the experimental results were favorable.

3) For the shear-bond and tensile-bond failure modes, FME and limit state loads were computed based on states of the initial and limit state force distribution systems, respectively.

4) The predictive strength for the diagonal tension mode was determined to be a function of the effective plank depth that resisted the in-plane shear forces.

5) The analytical equations predict the diaphragm behavior closely.

6) The analytical equations for diagonal tension failure mode does not take into account the number of sides connected.
7) The weld tie capacity needs further testing and verification. The analytical equations become sensitive to this value when large number of weld ties are used.

5.3 Recommendations for Continued Study

1) Perform additional diaphragm tests on other types of hollow-core slabs to verify that the results obtained are representative for the entire precast industry.

2) Strengthen plank joints between seams by either modifying the plank edge profile or developing a better weld tie.

3) Perform further tests with masonry frame to verify the behavioral characteristics of diaphragms connected to masonry walls.

4) The stiffness of the masonry testing frame need to be further investigated and more properly evaluated.

5) The capacity of the weld ties need to be more accurately evaluated by further testing.

6) The connection between the side and back walls for masonry frames needs further study and evaluation.

7) The predictive equations of the topped plans need to be modified to take into account the number of sides connected.
8) Prepare a set of design recommendations and a design procedure based on the three predictive failure modes for hollow-core plank diaphragms.
6. REFERENCES


7. ACKNOWLEDGEMENTS

This study was conducted as part of a research project sponsored by the National Science Foundation through the Engineering Research Institute at Iowa State University. The research was coordinated by the Technical Coordinating Committee for Masonry Research. The funding and guidance provided from these organizations is greatly appreciated.

The author would like to acknowledge the following organizations for their help and contributions. Prestressed Concrete Operation, a division of Wheeler consolidated, provided the planks for the testing. Nelson Stud Welding Division of TRW donated the shear studs and the stud welding gun. Central Pre-mix Concrete Company furnished the weld ties. The National Concrete Masonry Association provided a scholarship to the author during part of the time spent on this research.

The help and guidance of Dr. Max L. Porter, the author’s major professor, is greatly appreciated. Thanks are also due to Drs. Lowell F. Greimann, Terry J. Wipf, Frederick M. Graham, and Thomas R. Rogge, who were members of the author’s committee, for their assistance and cooperation.

Last but certainly not least, a great deal of thanks goes to my wife, JoNella M. Sabri, for her support, encouragement, and patience during the long hours of the research and preparation of this dissertation.