ABSTRACT

RAGAN, DAVID MICHAEL. Behavior of Efficient Two Story Precast Concrete Wall Panels. (Under the direction of Dr. Sami Rizkalla)

Precast concrete panels are utilized by engineers in order to create an efficient structural system. These panels can be used in the design of high rise buildings, residential units, and commercial structures. Market demand requires a time and cost efficient system, but at the same time, this system can not compromise the structural integrity of the system. Precast concrete panels provide several advantages including fabrication and product quality control, a significant reduction in construction and manufacturing time, fire resistance, durability, architectural variability, thermal and acoustical efficiency, and most importantly a savings in cost. There are several types of precast concrete wall panels including solid panels, insulated panels, sculptured panels, hollow core panels, and ribbed panels. The ribbed panels are comprised of a certain number of structural ribs connected by a thin concrete slab or skin. These panels can be reinforced with rebar, welded wire mesh, or a fiber reinforcing material. A research program was conducted at the Constructed Facilities Laboratory at North Carolina State University in order to investigate the behavior of two story precast concrete ribbed wall panels. An experimental investigation was conducted on two full-scale panels in order to find a failure mechanism. A finite element model was then created in order to predict panel behavior and analyze performance. This model was then reconfigured with different design features in order to optimize panel design.
Behavior of Efficient Two Story Precast Concrete Wall Panels

by
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BIOGRAPHY

David Michael Ragan was born September 6th, 1987 in Raleigh, NC. He spent his entire childhood and adolescence in Cary, North Carolina. After graduating from high school he decided to attend North Carolina State University in order to pursue an undergraduate degree in Civil Engineering. After graduating, he decided to continue his education at North Carolina State University in order to receive a Master’s of Science in Structural Engineering and Mechanics. He intends to become a professional engineer with a focus on structural design after completing his Master’s education.
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1 INTRODUCTION

1.1 Background

Precast reinforced concrete wall panels are typically used for creating the building envelope of industrial and commercial structures as well as the structural frame of these buildings. Structural panels are designed to support gravity loads, including the dead and live loads from roof and floor systems, as well as lateral loads, due to wind. Such panels can thus be utilized in the design of buildings, residential units, and commercial structures. Market demand for these types of panels requires that the panels be cost efficient and detailed for simple erection methods in order to reduce construction time. These panels should also be structurally designed to satisfy all code provisions. Precast concrete wall panels have proven records to meet such demands.

Precast reinforced concrete wall panels can be configured to produce strong, durable structures which are cost-effective, sustainable, provide for thermal efficiency, and highly capable of resisting natural hazard environments. They also have excellent quality control, fire resistance, allowances in architectural variability, and a significant reduction in fabrication and construction time. Several types of panels have been developed including solid panels, hollow-core panels, insulated panels, sculptured panels, tee section panels, and ribbed panels. Each panel is designed in order to create the most efficient system for the specified application.

Therefore, continuous research is needed to optimize the structural configuration as
well as the thermal efficiency. Also, research is necessary to investigate the performance and behavior of any new panel system. The research typically focuses on ways to optimize the panel’s structural configuration and design. This optimization strives to increase overall efficiency and savings with respect to the use of material.

This research focuses on a new proposed two-story precast concrete ribbed wall panel. This panel is intended to be used for residential and commercial applications, and utilizes randomly oriented synthetic fibers mixed in the concrete known as fiber reinforced concrete (FRC), to control cracking of the exterior face. The ultimate goal of the panel is to provide a structurally and thermally efficient system with savings in cost of construction and material. Ribs are used to allow for the inclusion of rigid foam insulation to increase the thermal efficiency of the panel.

1.2 Objective

The main objective of this research is to evaluate the structural behavior and failure modes of the proposed two story precast concrete ribbed wall panels, under the effect of a loading equivalent to a 50-year design life. The second objective is to develop an analytical model to predict structural behavior and use the experimental results to calibrate the analytical model. The third objective is to use the calibrated model to optimize the panel configuration and to provide alternative designs.
1.3 **Scope**

The scope of this project consisted of three phases. The first phase consisted of an extensive literature review with regards to precast reinforced concrete panels. The second phase included an experimental program testing two full scale panels. The third phase involved the formulation of an analytical model based on finite elements utilizing the experimental results to calibrate the model. The analytical model was used to optimize the design of the proposed panel.

The literature review, in Chapter 2, consisted of two main parts. The first part intended to highlight the efficiency of precast concrete panel systems. This part of the review also describes benefits and ways to optimize panel design. The second part of the review intended to discuss and describe relevant research and analysis that has been completed with regards to precast concrete panels.

The experimental program, in Chapter 3, describes the experimental program conducted on two full scale two-story precast concrete ribbed wall panels under the effect of gravity and equivalent wind loads. The testing was conducted at the Constructed Facilities Laboratory at North Carolina State University, within the Department of Civil, Construction, and Environmental Engineering. Panels were tested vertically in a frame which allowed for the application of combined gravity and lateral loads. Load and support conditions were applied in order to simulate typical design and field assumptions. Critical deflections and strains were measured during the entire experimental program. The panel was first subjected to a 50-year design life fatigue loading cycle. Lateral loads were then increased to
investigate failure mechanisms. All lateral load sequences were performed with the inclusion of the gravity loads. The behavior of the panel was evaluated during fatigue loading, at service and design loads, at failure loads, and finally at the maximum allowable load within the test setup constraints.

Chapter four describes the modeling techniques used to optimize the panel configuration. A frame analysis is first completed and then compared with a finite element model analysis. The results from this analysis were used to justify a 3-D solid finite element model of the panel. The solid finite element model was calibrated using results from the frame analysis and the measured values from the experiment. The calibrated model was used in conducting a parametric study on the proposed precast panel. The parameters changed in panel configuration included fewer interior ribs, thinner exterior ribs, and the exclusion of the middle support in order to evaluate a full two story height panel.

Chapter five summarizes the research project, and provides conclusions and recommendations based on the research findings.

Specimen details and dimensions, additional experimental results, and supplemental figures and tables are provided in Appendices at the end of the thesis.
2 LITERATURE REVIEW

2.1 Introduction

This chapter is a general overview of precast reinforced concrete wall panels. The sections within this chapter include Background and Development, Advantages of Precast Wall Panels, Design Features, Structural Behavior, and Special Code Provisions. The Background and Development section provides a brief description of precast concrete and its application to wall panels. The Advantages section showcases the benefits of these panels and the Design Features section describes the necessary design considerations. The Structural Behavior section covers relevant academic study on the topic. Finally, the Special Code Provisions section describes special design features found in the applicable codes and how they are used in the design and analysis process. It should be noted that precast reinforced concrete wall panels are briefly summarized in order to coincide with the relevant literature being referenced. The term may be quoted as ‘precast panel’ or ‘concrete panel’.

2.2 Background and Development

Precast concrete is an efficient means for structural design of commercial, residential, and special industry buildings. Research and development has enabled a progressive trend towards the use of precast concrete. Different precast design techniques are being applied in order to use the various products in the most effective way.
Precast reinforced concrete presents many advantages. These structural elements provide an efficient and effective method of construction while also providing a practical and simplified tool for designers. Precast reinforced concrete, as described by PCI, allows for faster construction speed, quality control during fabrication, fire resistance, durability, architectural variability, thermal and acoustical control, and an ability to be constructed in all forms of weather (PCI Design Handbook 6th Edition). Some of these benefits however are a direct function of some certain practicing philosophies. These philosophies, as described by PCI, include the use of repetitive designs, economical layouts, and combining structural features within a precast member (PCI Design Handbook 6th Edition).

Precast concrete can be applied to many structural elements such as wall panels. These panels can be used in a variety of applications such as high rise buildings, parking decks, residential units, and commercial structures. All the prior advantages listed by the PCI Design Handbook can be utilized by these panels in order to make design and construction more efficient.

Precast panels are generally divided into two categories meant to serve two different purposes. The first category is exterior cladding. Cladding forms a barrier between interior space and the weather outside. With respect to structural demands, cladding only transfers lateral loads to the structural frame. The next category is structural load bearing walls. These walls allow for more sophisticated structural designs. Structural walls are defined as walls proportioned to resist combinations of shears, moments, and axial loads (ACI 318-08). These include bearing walls, shear walls, nonbearing walls, tilt-up walls, or a combination of
these. Therefore a bearing wall can not only resist lateral loads, but also provide structural support for the gravity loads created by the roof and floor systems as well as other in-plane loads.

There are many generic and basic styles for designing precast concrete panels. These include solid panels, hollow-core panels, insulated panels (sandwich), ribbed panels, and sculptured (architectural) panels (ACI 533R-93).

### 2.3 Advantages of Precast Wall Panels

There are several advantages specific to using precast reinforced concrete panels in addition to those listed by the PCI Handbook. These advantages can all lead to a more efficient structural design which is the ultimate goal in using a precast panel system. However, designers must take into account multiple considerations in order to realize all the potential advantages that can be created by using these panels.

Freedman (1999) details the advantages of these panels and their relationship with the design procedure. The first advantage of using concrete panels is that the structural frame can be eliminated. This is due to the load bearing capabilities of the panels. The use of the concrete panels will satisfy structural demands for both lateral and gravity loads, thus eliminating the need for an exterior frame and typically allowing for a reduction in the size and scale of the interior frame. The benefits include savings in material and construction cost as well as an increase in floor space.
Another advantage listed by Freedman (1999) is the simplicity of design due to the repetitive nature of concrete panels. Panels are designed for certain load criteria. Therefore an efficient panel system will meet all these demands with the use of the same panel. The benefits include savings in design time, construction time, fabrication time, and overall costs. Also, precast panels can serve as permanent formwork for cast in place concrete. This will eliminate the need for temporary formwork as well as the labor, material, and time demands required to erect it.

The last advantage listed by Freedman (1999) of precast panels is that they can be used as supplements to other structural systems. They can be combined with cast-in-place systems, steel framing systems, timber systems, or other precast systems. This gives a designer more freedom in their approach and allows them to combine the benefits of the different systems.

The inclusion of thermal control features within a panel in order to meet building code requirements must be considered. This is typically done by adding an insulating foam material to the concrete panel. Due to the necessary inclusion of the foam material, different designs of the panel have been developed in order to effectively incorporate it. For example, ribbed panels can fill the spaces between ribs with foam material in order to increase the thermal efficiency.

In order to gain the advantages listed prior and thus provide the benefits generated by using precast panels, certain fundamental design philosophies must be satisfied. Freedman (1999) states that the first philosophy is that the panels must be designed to a maximum size.
This sizing requirement will reduce the number of panels needed and reduce the number of joints and connections. This in turn will create savings in fabrication, transportation, and erection and make a more efficient system.

Freedman (1999) also states that the way loads are applied within the system will determine the efficiency of a precast panel system. The ability for the panels to act as shear walls factors into the ability to eliminate a shear resisting system. Also, the uniform distribution of loads within a panel is important. This relates to the way that stresses are created within the panel section due to applied loads and also to the way that loads are transferred between connections and to foundations.

Finally, the dimensioning, details, and connections will also affect the use of these precast panels. The dimensioning requirements affect panel sizes, spacing, and the allowances for openings. Also, the ability for connections to transfer loads between panels creates an efficient precast wall system.

Koncz (1995) conducted a study on the efficiency of using concrete panel systems in building construction. The author states that the large panel building system was developed in France in the 1950’s and that it has thus been utilized with increasing efficiency in the structural market. This precast panel system can be applied to high rise structures, residential units, and commercial construction. Koncz (1995) specifically explains a system referred to as the all-wall load bearing system which is currently in use in European and Asian countries. This system connects precast panels in order to create one structural unit as opposed to separate elements. The author analyzes the system using finite elements. The finite element
analysis predicts the development of stresses within the system. The author states that through experimental work the model is justified in predicting stresses. Thus the author concludes that large panel systems can be used in order to increase the efficiency of structural systems as the predicted and experimental stresses developed throughout the system are acceptable.

Zielinska and Zielinski (1982) completed research on the overall efficiency of using precast ribbed panel systems. The authors researched the application of the panels to many structural forms including small residential units and large scale structures. The panels were ten feet in height with ribs spaced four feet apart and ten inches thick. Welded wire mesh was used to reinforce a 2.5 inch thick exterior skin. The authors cited several examples utilizing the panels. One example required 2.5 days to erect the precast framing system. Also, the overall cost of the structure was comparable with other similar structures. Another example combined a timber roof with precast panels. The structure showcases the structural adaptability of precast panels. The authors conclude that precast panel systems are effective for decreasing costs while increasing overall efficiency.

2.4 Design Features

There are several considerations that need to be made when formulating an approach to an overall design for precast reinforced concrete wall panels. This section will initially discuss considerations relative to structural design and then discuss considerations that a structural engineer must take with respect to construction and architecture.
The first consideration to be made is the design load cases. Gravity or vertical loads can create purely axial forces when applied to a member concentrically. These loads can also be applied eccentrically, creating additional moments that must be considered. Concrete panels will also be influenced by lateral loads due to wind and seismic forces. Therefore a designer must consider the panel as a member under both axial and lateral loads. Lastly, the designer must consider whether or not to treat precast panels as shear walls.

Volume changes created from creep, shrinkage, and temperature variations must also be considered. A designer should size joints and space them according to the determined volume changes. Allowable tolerances related to both design demands and also fabrication and erection standards should be considered.

Connections are another significant design consideration. Typical panel failure occurs at a connection as opposed to a structural failure. Connections allow for movement, transfer of forces, and stability. Connections include bolting, welding, grouting, or a combination of those listed. These connections attach the panel to the foundation, floor slabs, roofs, and to other panels.

Several design considerations related to construction and architectural aspects of the concrete panels must also be investigated. These aspects of design are addressed in order to design the most overall efficient concrete panel. Design of a precast panel can be divided into many categories including transportation, erection, fabrication, and architectural design.

Fang et al. (1994) lists and describes these categories. The transportation of a
reinforced concrete panel must be considered in order to estimate shipping costs, delivery truck size and capacity, and shipping distance. Also, the means and methods of construction are necessary in order to determine equipment required for construction and the speed at which the panels can be erected as well as site access. Another consideration to be made is the fabrication process with respect to the ease of replication of a panel, overall amount of material used, and the complexity of the panel fabrication. Finally, architectural considerations must be made. This relates to ventilation, thermal properties, overall dimensioning and details, and aesthetic value.

2.5 Structural Behavior

The primary requirement of precast structural panels is to resist both lateral and gravity loads. Research initially sought to create and optimize design standards. More recent research has sought to investigate the behavior of precast panels with varying configurations and designs.

Oberlender (1977) conducted a research program in order to investigate the bearing capacity of precast reinforced concrete panels. This experimental program tested 54 different panels. Each panel had a thickness of 6 inches. The slenderness ratio, concrete material properties, and reinforcement were varied. Axial compressive loads were applied in two different ways in the gravity direction. The first was concentrically placed over the cross section while the second was eccentric to the neutral axis. Each load application was uniformly distributed along the panel ends. The axial and lateral deflections were measured.
along with the concrete strains until failure of each panel. The experimental results were compared to those predicted by Chapter 10 and Chapter 14 of ACI standards. The comparison was drawn between the ultimate experimental load and the predicted ultimate load. Values were analyzed primarily with respect to panel slenderness. Panels that were loaded concentrically generally failed from concrete crushing while panels that were loaded eccentrically generally failed in buckling. The ultimate test load was significantly higher than code predicted values for panels with low slenderness ratios while the value was only slightly higher for panels with higher slenderness ratios. The authors formulated an adjusted design equation to more accurately predict the failure load as opposed to the equation given by ACI. The equation is given as:

\[ Pu = 0.6\Phi f_c'b h [1-(l/30h)^2] \]  

Equation 2.1

Where:

\( \Phi \) = understrength factor of 0.7

\( f_c' \) = concrete strength

\( b \) = width of panel

\( h \) = effective thickness

\( l \) = unsupported height

The authors conclude that the code predicts ultimate loads that are considerably less than those from experimental tests particularly for panels with lower slenderness ratios.
author also recommends the adjusted design value given.

Zielinski et al. (1983) conducted an experimental program that investigated the bearing capacity of ribbed wall panels. Panels were 89 inches in height with a width of 44 inches while the panel skin was only 1.5 inches thick. The panels were reinforced with welded wire mesh. The panels were loaded concentrically with axial compressive forces. Deflections were measured and cracking patterns were identified. The goal of the research was to determine the ultimate axial load and then compare them to the predicted code values. Vertical cracks initially developed on the face of the ribs. Failure occurred when the base of the top rib separated from the skin of the panel through concrete crushing. The experimental results were compared to the ACI design code predictions. The experimental ultimate loads were significantly higher than those predicted by the code. The authors state that this underprediction is due to the increase in steel reinforcement in the experimental panels as opposed to code assumptions. Therefore the authors propose a modification factor to adjust for the ratio of reinforcement area to gross area of concrete. This equation is given as:

\[
Pu = 0.55\Phi f'c A_g [1-(l_c/40h)^2][1+(m-1)\rho_m]
\]

Equation 2.2

Where:

\(\Phi\) = understrength factor

\(f'c\) = concrete strength

\(A_g\) = gross area of concrete
\( h = \text{effective thickness} \)

\( l_c = \text{unsupported height} \)

\( m = \text{ratio of steel to concrete strength} \)

\( \rho_m = \text{ratio of steel to concrete area} \)

The authors conclude that panel failure occurs due to concrete crushing and not due to buckling. Also, the authors conclude that the code predicts ultimate loads that are considerably less than those from experimental tests particularly for panels with higher reinforcement ratios. The author recommends an adjusted design value.

The research has thus led to the most recent equation given in ACI 318-08 for empirical design given as:

\[
P_u = 0.55\Phi f_{c'} A_g [1-(l_c/32h)^2]
\]

Equation 2.3

Where:

\( \Phi = \text{understrength factor} \)

\( f_{c'} = \text{concrete strength} \)

\( A_g = \text{gross area of concrete} \)

\( h = \text{effective thickness} \)

\( l_c = \text{unsupported height} \)
Lee and Pessiki (2008) conducted a research program on the flexural behavior of precast reinforced concrete panels. This research was done on sandwich panels. The project focused on composite action of the panel but also analyzes the flexural behavior of the panel. Panels were loaded laterally in order to simulate wind loads. No axial load was applied to the panel. The specimens were scaled down to 6 feet 8 inches wide, 35 feet in length, and 8 inches thick. A uniform pressure load was applied to the entire face of the panel and the specimens were restrained at the base and top locations in the lateral directions. Strain gages and LVDT’s were used in order to measure concrete strains and panel deflection. Some cracks were observed before testing. The authors contribute these cracks to a number of sources including prestressing, shrinkage, and panel handling. The panel was loaded to a maximum load of 15 kips and recorded a deflection of 6.4 inches. Cracking was observed due to flexure and stiffness was lost as cracking propagated. Failure did not occur because the tests were terminated before the panels reached ultimate load. The panel’s load-deflection behavior is linear until the formation of large flexural cracks. The panels act in a ductile manner as evidenced by the large deflections. ACI design code predictions were compared to experimental results but experimental tests were not loaded to this ultimate predicted value. The panels experienced no flexural cracking at service or design loads. The authors also conducted a finite element analysis in order to predict stresses and thus the locations of cracking within the panel. The model was compared to the experiment and the authors concluded that the model was an effective analysis tool in predicting cracking.

Saia (1985) conducted research in order to justify the use of finite element modeling in order to predict the behavior of precast concrete panels and create an analysis procedure.
The author uses a flat solid panel 30 feet wide by 8 feet high and only 5 inches thick. Appropriate material properties were applied to the panel while steel reinforcement was neglected. Self weight was applied to the structure as well as a lateral wind load of 10 psf. The 3D solid model was divided into 240 eight noded rectangular elements. The stresses, strains, and deflections associated with the loading were then analyzed. The author concludes from experience and experimental results that the predicted values follow expected behaviors. Thus the analysis is successful. The author suggests that a design engineer can use the model to analyze the panel and decide on locations and amounts of reinforcement necessary. The author also states that the model is easily modified to examine different parameters for design such as load cases and support locations by changing the input for the model. The author then gives a general analysis procedure. This procedure is a three step process. The first step is data input, the second is a solution step, while the last is the post solution step. The data input step includes setting the geometry of the panel, specifying loads, and defining the elements. The solution step involves the time and cost associated with solving the problem. Finally, the post solution step involves interpreting the data by creating graphs, tables, and plots of the solutions for analysis by the designer. Lastly, the author concludes that finite element modeling is practical for predicting precast concrete panel behavior and that these models can be used in parametric studies.

Maneetes et al. (2009) developed a finite element model of a precast reinforced concrete panel. The authors state that finite element models for concrete paneling are typically treated as uncracked and infinitely stiff. One reason for this is to focus on the connection locations on the panel, since connections are usually the weakest part of a
reinforced concrete member. Also, the modeling can predict serviceability requirements, deflections and cracking patterns, safety requirements, and failure of the model. Maneetes et al. (2009) utilized finite element analysis in order to predict the response and behavior of a precast concrete panel. The panel was 24 feet long, 7 feet tall, and 8 inches thick. The model used 6,000 solid brick elements. The results were then compared to an already completed experimental test. The authors concluded that the model and experimental results compare favorably and that the modeling will predict expected panel behavior. A parametric study was then completed to analyze connections and reinforcing layout. Another conclusion was that the model showed stiffer characteristics than the test specimen, but this is not significant in overall application of the model. Lastly, the authors concluded that finite element analysis can reduce the number of experimental test programs needed in order to analyze panels. The labor, time, and overall cost savings are significant and can be done in commercially available software packages.

Hobelmann and Schachter (2009) conducted another research program in order to investigate the structural response of a precast concrete panel frame system from lateral loading. This was done by utilizing frame analysis techniques. A structural frame was created using 2D frame elements. The panels were 30 feet wide, 14 feet high, and 8 inches thick. Gravity loads were combined with lateral wind design loads and applied to the model. The authors used ACI design code predictions to monitor cracking. Cracking was not allowed and therefore controlled the design. The authors concluded that the framing model was capable of predicting crack location and thus applicable for analyzing precast panels.
2.6 Special Code Provisions

The Reinforced precast concrete wall panels are to be designed by the code provisions and design methods given in Chapters 8, 9, 10, 11, 12, 14, and 16 of the Building Code Requirements for Structural Concrete, ACI 318-08. Special considerations and details are laid out in the Guide for Precast Concrete Wall Panels ACI 533R. This section of the literature review will focus on these two design codes.

Wall panels must consider all design loads due to construction, handling, erection, transportation, impact, gravity, and live load cases. Stress concentrations due to connections, creep and shrinkage deformations, and thermal movements and bowing must also be considered. Finally, member design must conform to the requirements for flexural, bearing, shear, service, and minimum reinforcement design criteria. Due to panel minimum sizing and dimensions, most of these design criteria will be satisfied. Therefore, this section will not cover all details of panel design but will only look at the special considerations and features that contribute significantly to panel design.

The first significant feature of precast panel analysis is determining the effective panel thickness. This equation for solid, hollow-core, and ribbed panels is given as:

\[
h_{\text{eff}} = 3 \sqrt{\frac{12fg}{b}}
\]

Equation 2.4

Where:
\[ h_{\text{eff}} = \text{effective panel thickness} \]

\[ I_g = \text{gross moment of inertia (neglect reinforcement)} \]

\[ b = \text{width of cross section} \]

This effective thickness is used when calculating other required design values in order to analyze a precast panel. This value is used as opposed to the total thickness of the panel when calculating values such as design shear strength, design bearing strength, and other panel dimensions and limitations. Another more conservative approach to calculating the effective thickness is to use three times the radius of gyration of the panel in the direction in which stability is considered (Speyer 1976).

Another significant feature of precast panel analysis is the evaluation of the slenderness effect. This applies to bearing structural wall panels. The maximum ratio is given as:

\[ k \, l_u / r < 200 \quad \text{Equation 2.5} \]

Where:

\[ k = \text{effective length factor} \]

\[ l_u = \text{unsupported length} \]

\[ r = \text{radius of gyration} \]

It should also be noted that if the slenderness ratio is greater than 150, then
slenderness effects must be taken into account.

The maximum ratio between the height of the panel and the thickness is given as:

\[ \frac{l_u}{h_{\text{eff}}} < 25 \]  

Equation 2.6

Where:

\[ l_u = \text{unsupported length} \]

\[ h_{\text{eff}} = \text{effective panel thickness} \]

The distance between supports should not exceed 32 times the effective width of the compression flange or the effective panel thickness. The effective flange width is not to exceed the center to center distance between ribs, 8 times the flange thickness on either side of the rib, or one fourth of the clear span.

The minimum ratio of reinforcement area to gross concrete area should not be less than 0.001. Also, transverse reinforcement is not generally required for precast panels with a width less than or equal to 8 feet. However, transverse rebar may be required for casting and handling purposes. Therefore careful consideration should be taken for casting and handling methods in order to minimize flexural stresses. This is more effective than transverse reinforcing steel (Speyer 1976). Also, areas of abrupt change in cross section should be reinforced (Speyer 1976). The code also gives maximum rebar spacing for precast panels. For exterior panels the maximum spacing is 18 inches while interior panels are allowed a maximum spacing of 30 inches. However, the code provides that a detailed structural
analysis may be conducted in order to eliminate this requirement. Therefore, if analysis shows satisfactory strength behavior and serviceability with an increased spacing, this will be allowed. Panels that are thicker than 6 inches must have two layers of rebar. Finally, interior surface cover must be at least equal to the nominal bar diameter but not less than 5/8 inch while exterior surface cover must be at least equal to the nominal bar diameter but not less than ¾ inch.

The code also outlines requirements for serviceability. These requirements deal with maximum deflections and cracking of concrete. For load-bearing precast panels, after the application of the live load, the maximum deflection may not exceed l/360 or ¾ inches, whichever is smaller. For non-coated reinforced panels, exterior panel face cracks that are less than 0.005 inches will be allowed while interior panel face cracks that are less than 0.01 inches will be allowed. For coated rebar, the architect will determine the general allowance for cracks. This allowance relates to the aesthetic value of the panel. Therefore for exposed panel surfaces, architects will typically not allow cracking.
3 EXPERIMENTAL INVESTIGATION

3.1 Introduction

In order to investigate the structural behavior and determine the failure mechanism of a two story precast concrete ribbed wall panel, an experimental program was undertaken at the Constructed Facilities Laboratory at North Carolina State University, within the Department of Civil, Construction, and Environmental Engineering. The experiment included the testing of two full scale panels. This testing provided a reliable tool to calibrate future model analysis to optimize the panel configuration.

3.2 Overview

In this experimental program (2) identical two-story precast concrete ribbed wall panels were tested. Both panels measured 8’ by 20’ by 7.25’’ thick. Panels were tested vertically in a frame which allowed for the application of combined gravity and lateral loads. Gravity loads were applied to the top panel surface, and also to corbels at the panel mid-height. These gravity loads were applied in order to simulate the loads typically imposed on a panel from the roof and floor structures. With the gravity loads in place, reverse-cyclic lateral loads were applied at each panel quarter-height to simulate the effects of wind.

Each panel was supported as a two-span continuous structure with simple supports. The base of the panel was restrained with a pin support, while the mid-height connection and
top connection were roller supports. The pin support allowed rotation, but no translation. The roller supports allowed rotation and vertical translation, but no lateral translation. These connections were selected to simulate typical design assumptions.

Both panels were tested in two stages. In the first stage, each panel was subjected to approximately 5700 reverse-cyclic lateral load cycles, selected to simulate loads expected over the panel’s 50-year design lifetime. In the second stage, panels were loaded in incrementally-increasing lateral load cycles to evaluate the panel behavior at service load, design load, and failure load. Factored gravity loads were maintained during all lateral loading stages. Deflections, strains, and applied loads were monitored continuously throughout all tests.

3.3 Test Specimens

Two identical two-story precast concrete ribbed wall panels were tested in this program. Each specimen measured 8 feet wide by 20 feet tall by 7.25” thick. The first panel tested will be referenced as RWP1 while the second panel tested will be referenced as RWP2 in this thesis. Specimens were conventionally reinforced concrete ($f'_c = 5000$ psi) with integrated thermal insulation. The concrete on the exterior panel surface contained randomly oriented chopped synthetic fibers. This fiber reinforced concrete created the thin skin that connected the panel ribs and was only 1.25 inches thick. Exterior ribs were 6 inches thick while interior ribs were only 2.5 inches thick. After testing the panels, the concrete compressive strength was measured by compression tests on drilled core samples. The
measured compressive strength had large variance but had an overall average of 8100 psi. The general panel dimensions are shown in Figure 3.1. Details of the panel configuration and of the internal reinforcement for both specimens are presented in Appendix A. A typical test specimen is shown in Figure 3.2.

Figure 3.1 – General Dimensions of a Two-Story Precast Concrete Ribbed Wall Panel
Figure 3.2 – Typical Two-Story Precast Concrete Ribbed Wall Panel

3.4 Test Setup

Each panel was tested in a vertical position in a specially designed steel reaction frame. The loading frame provided lateral support at three levels (bottom, mid-height, and top). Idealized boundary conditions were provided, as shown in Figure 3.3.
Figure 3.3 – Schematic View of Constraint Locations

Gravity loads were placed on each panel at two levels, as shown in Figure 3.4. The first gravity loads were applied to the panel near the mid-height. These loads were applied eccentrically through two mounted corbels projecting from the interior panel face. The vertical forces were applied by hydraulic jacks and transfer beams secured to the reaction frame. The second set of gravity loads was applied concentrically to the top surface of the panel using additional hydraulic jacks secured to the reaction frame. The loads applied by all jacks were monitored throughout the tests with loadcells. Hydraulic accumulators were used to continuously adjust the jacks, maintaining constant axial loads as the panel deformed due
to the axial and lateral loads.

Figure 3.4 – Schematic View of Load Locations (Dimensions shown are in inches)

Lateral loads were applied to each panel in two locations, as shown in Figure 3.4. Lateral loads were applied at the mid-span locations to simulate wind pressure and suction. Two 22-kip hydraulic actuators, mounted to a strong reaction wall, were connected to each panel at its quarter-height locations. Square steel tubing (HSS) was attached to the panel to spread the lateral loads across the panel width. Thick neoprene pads were placed between the concrete surfaces and the HSS, and threaded rods were run through oversized holes in
each panel to avoid excessively restraining the panel at the points of load application. The two actuators were configured to produce identical loads at all times.

3.5 Details of Test Setup

Steel columns and beams were used to construct the test frame. The bottom support was provided by placing the base of the panel into a steel channel mounted onto a roller, creating a pin connection. The pin connection was secured to the floor of the laboratory space. At the mid-height, a steel beam spanned the reaction frame and allowed for a roller connection at the mid-height of the panel. Finally, at the top of the panel, a light steel angle frame was secured to the panel and tied to the reaction frame to create another roller connection.

At the quarter-heights, HSS steel sections sandwiched the width of the panel and were attached to the hydraulic actuators. Mid-height gravity loads were applied through two additional HSS steel sections which hung from the support frame and from the panel corbels. Two hydraulic jacks were used to apply load to these HSS sections, and the corbel reaction was monitored. Top level gravity loads were applied by four hydraulic jacks reacting off the test frame and bearing on pads resting directly on the top surface of the panel. Isometric sketches and a description of the construction sequence are given in Figure 3.5. The completed test frame is shown in Figure 3.6.
Figure 3.5 – Conceptual Test Setup

1) Test frame is erected with stressed columns to the floor. Channel section is pinned to floor.

2) Load system is erected. Actuators attached to strong-wall and jacks attached to cross-beams at mid-height and full height.

3) Test specimen is placed into channel and attached at mid-height connection.

4) Test specimen is tied into top connection and attached to actuators at quarter-heights.
3.5.1 **Support Conditions**

Each precast panel was supported laterally in three locations. The support conditions replicate idealized design assumptions, but were selected so as to not differ substantially from field conditions. Vertical support was accomplished by placing the panel in a steel channel and sealing the base in with grout. The base connections for the panel were also welded to the channel. The channel was pinned to the floor, providing lateral restraint, but allowing for rotation. The lower support is shown in Figure 3.7.
A lateral connection was provided at the mid-height of the panel. This connection was created by attaching an HSS steel tube to the panel T-bolt connections at each end. The ends of the HSS tube were welded to a pin which rested in a slot anchored to the test frame, creating the desired roller connection. These details are shown in Figure 3.8.

Figure 3.8 – Overview (left) and Close-View (right) of Mid-Height Connections
A lateral connection was also used at the top of each panel. A light frame of steel angles was fabricated to slip over the top of the panel. The frame was connected at each end to a slotted steel plate attached to the test frame, creating the desired roller connection. Details of the top connection are shown in Figure 3.9.

Figure 3.9 – Overview (left) and Close-View (right) of Top Connections

3.5.2 Loading Sequence

Panels RWP1 and RWP2 were loaded in a similar fashion. Loads were applied to the panel in the same manner in which the panel would be loaded in field conditions. The factored gravity loads were first applied at the mid-height and then the top locations, prior to applying any lateral loads. The mid-height gravity loads were applied simultaneously to both mid-height corbels. A 5-kip point load was applied to each corbel at a location 3.5 inches from the inner face of the wall. With the mid-height loads in place, the top-level gravity loads, four point loads, 3-kips each, were applied concentrically along the top edge of the panel at 2 feet on center. Hydraulic accumulators were used with both gravity load systems.
to maintain constant axial load levels by adjusting for small vertical deformations.

With the factored gravity loads in place, reverse-cyclic lateral loads were applied at the \( \frac{1}{4} \) height and \( \frac{3}{4} \) heights. Over 5700 initial lateral load cycles were applied to each panel to simulate a 50-year design lifetime, prior to taking a panel to the design lateral load.

Lateral load levels were determined from a selected design wind speed of 150 mph, corresponding to a service level equivalent uniform pressure of 47.5 psf. At the factored load level, a load factor of 1.6 was used, giving both panels a full factored design pressure of 76 psf. For these tests, the lateral wind loads were applied using a matched pair of 22-kip capacity hydraulic actuators located at the quarter-height locations. At service load, which will be as described as \( W \) throughout the rest of this thesis, each actuator needed to apply 3800 pounds; at the factored load, 1.6\( W \), each actuator needed to apply 6080 pounds.

Both panels were tested with the same loading sequence. First, the two axial loads of 5 kips were applied and maintained at the mid-height location. Then, four axial loads of 3 kips were applied and maintained on the top surface of the panel. A panel was then subjected to nearly 5700 reverse-cyclic lateral load cycles prior to being subjected to additional incrementally-increasing lateral loads.

The applied loading sequence for each panel consisted of several loading levels and approximately 5700 total cycles. The fatigue loading cycles that both panels were subject to are summarized in Table 3-1. One load cycle is considered as taking the panel from zero loading to the specified lateral load level in both positive and negative directions and back to
zero. The loading cycles were selected to simulate a 50-year design lifetime using a Weibull distribution, as described in the following section.

Table 3-1 – Applied Loading Sequence

<table>
<thead>
<tr>
<th>Load Step</th>
<th>Mid-Height Axial Load</th>
<th>Full-Height Axial Load</th>
<th>Lateral Load</th>
<th># of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load Level</td>
<td>Load at Each of Two Corbels (lbs.)</td>
<td>Load at Each of 4 Locations (lbs.)</td>
<td>Load Level</td>
</tr>
<tr>
<td>1</td>
<td>Factored</td>
<td>5000</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>40% 1.6W</td>
</tr>
<tr>
<td>4</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>45% 1.6W</td>
</tr>
<tr>
<td>5</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>50% 1.6W</td>
</tr>
<tr>
<td>6</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>60% 1.6W</td>
</tr>
<tr>
<td>7</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>80% 1.6W</td>
</tr>
<tr>
<td>8</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>100% 1.6W</td>
</tr>
<tr>
<td>9</td>
<td>Factored</td>
<td>5000</td>
<td>Factored 3000</td>
<td>2.0W, 2.4W, etc.</td>
</tr>
</tbody>
</table>

It should be noted that Panel RWP1 was loaded until failure of the mid-height lateral connection at 2.4W. After this initial failure, the mid-height connection was strengthened, and panel RWP1 was re-tested to a lateral load of 4.0W. The test was terminated at this level. Panel RWP2 was tested with the strengthened connection initially in place. After all loading cycles were completed, panel RWP2 was loaded laterally to 4.0W, and the test was terminated. Panel RWP2 was then unloaded, the axial loads were removed, and the panel was reloaded to 4.0W with no axial loads applied.

3.5.2.1 Fatigue Cycles

In order to simulate the effects of wind loading over a 50-year design life, the panel
had to be subjected to fatigue loading cycles. The use of a Weibull distribution function allows for the selection of both load increments and the number of cycles required for fatigue loading. This selection process is based upon other published work (Hau 2000, Xu 1995, Manwell 2002).

A Weibull distribution function models the probability of a given wind speed’s occurrence over a given period of time for a certain location. The probability distribution equation is given as:

\[
P(U) = 1 - \exp\left[-\left(\frac{U}{c}\right)^k\right]
\]

Equation 3.1

Where:

\(P(U)\) = Weibull probability distribution function

\(U\) = wind speed under consideration

\(k\) = shape factor

\(c\) = scale factor

The shape factor and scale factor are functions of the mean wind speed and the standard deviation of wind speeds at the respective location. These values are location specific and based upon actual wind data from that location. In many cases, the location specific data is not available, therefore a general assumption is made to use a shape factor equal to 2 (Hau 2000, Xu 1995). This special form of the Weibull Distribution is referred to
as the Rayleigh Distribution. In this variation of the Weibull Distribution, the ratio of the standard deviation to the mean is kept constant. The Rayleigh Distribution therefore is determined with respect to the average mean wind speed. The distribution function is given as:

\[
P(V) = 1 - \exp\left[-\frac{\pi}{4} \left(\frac{V}{\bar{V}}\right)^2\right]
\]

Equation 3.2

Where:

\[P(V) = \text{Rayleigh probability distribution function}\]

\[V = \text{wind speed under consideration}\]

\[\bar{V} = \text{average mean wind speed for specific location}\]

The Rayleigh distribution versus wind speed is shown in Figure 3.10 for a mean wind speed of 15.0 mph.
The number of cycles can then be calculated based upon the Rayleigh Distribution Function. P(V) is the probability that a specific wind gust will occur at a certain point in time. Therefore, the equation [1-P(V)] is the probability of a wind gust of a given strength or more occurring at any given moment. Therefore the expected exposure time to given wind speeds is given as:

\[ [1-P(V)] \times \text{Lifetime} = \text{Probable exposure time at or above a given wind speed} \]

Where the lifetime is in seconds, therefore exposure time is in seconds.

Using the following given assumptions, a 50-year design life, a 150 mph design wind speed, and a mean annual wind speed of 15.0 mph, the number of cycles expected to occur at
a given wind speed can be estimated by multiplying the exposure time with the expected vibration frequency of the panel. The assumption can be made that one cycle (load applied in both directions to the panel) will function as each second of exposure (1 Hz), therefore the total number of cycles will be equal to the total number of seconds of exposure for a given design lifetime. The number of cycles calculated using the Rayleigh distribution for different wind speeds are shown in Table 3-2.

**Table 3-2 – Rayleigh Probability of Exposure at Certain Wind Speeds**

<table>
<thead>
<tr>
<th>% of 150 mph Design Wind Speed (%)</th>
<th>Load (per actuator) (lbs)</th>
<th>Given Wind Speed (mph)</th>
<th>1-F(V) (% Probability)</th>
<th>Probable # of Cycles (#)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>608</td>
<td>15</td>
<td>45.59381%</td>
<td>718923240</td>
</tr>
<tr>
<td>20</td>
<td>1216</td>
<td>30</td>
<td>4.32139%</td>
<td>68139706</td>
</tr>
<tr>
<td>30</td>
<td>1824</td>
<td>45</td>
<td>0.08514%</td>
<td>1342548</td>
</tr>
<tr>
<td>40</td>
<td>2432</td>
<td>60</td>
<td>0.00035%</td>
<td>5499</td>
</tr>
<tr>
<td>45</td>
<td>2736</td>
<td>67.5</td>
<td>0.00001%</td>
<td>195</td>
</tr>
<tr>
<td>50</td>
<td>3040</td>
<td>75</td>
<td>0.00000%</td>
<td>5</td>
</tr>
<tr>
<td>55</td>
<td>3648</td>
<td>90</td>
<td>0.00000%</td>
<td>0.0</td>
</tr>
<tr>
<td>60</td>
<td>4256</td>
<td>105</td>
<td>0.00000%</td>
<td>0.0</td>
</tr>
<tr>
<td>70</td>
<td>4864</td>
<td>120</td>
<td>0.00000%</td>
<td>0.0</td>
</tr>
<tr>
<td>90</td>
<td>5472</td>
<td>135</td>
<td>0.00000%</td>
<td>0.0</td>
</tr>
<tr>
<td>100</td>
<td>6080</td>
<td>150</td>
<td>0.00000%</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Design wind speed is not utilized in the determination of the Rayleigh distribution.
Therefore the probability of a given wind speed occurring will not change with a variation in the design wind speed. The distribution is only dependent on the mean wind speed for a given location. The design wind speed selected in this experimental program is intended to represent the gulf coast of the United States (15.0 mph average).

3.5.3 Instrumentation

Deflections, strains, and applied gravity and lateral loads were monitored throughout each test with three types of instrumentation. All instruments were wired to an electronic data acquisition system which recorded data periodically at a rate of 5 Hz during the automated fatigue cycles, and continuously at a rate of 1 Hz during the later-stage cycles. Details of the instrumentation are provided in the following sections.

3.5.3.1 Load Cells

Load cells were used in four locations to monitor and record the loading being placed on the panels. The first load cell was integrated into the hydraulic actuators and was used to record the applied lateral loads. The two actuators were connected together to ensure that they applied identical lateral loads at all times. The second and third load cells were attached to the two hydraulic jacks used to provide axial loads on each corbel at the panel mid-height. These jacks were also connected together to ensure matching loads. The final load cell was attached to a hydraulic jack, and was used to measure the loads applied to the top surface of
each panel. A load cell attached to the hydraulic jack used for the top surface loads is shown in Figure 3.11.

Figure 3.11 – Load Cell

3.5.3.2 String Potentiometers

String potentiometers ("string pots") were used to measure lateral displacements of each panel at several locations. Panels were instrumented with 7 sting pots at the locations shown in Figure 3.12, and are numbered accordingly. Outward deflections, those moving away from the exterior panel surface, were recorded as positive. Inward deflections, those moving away from the interior panel surface, were recorded as negative. String pots were utilized at locations being described as the critical locations. These include the three restraint locations to monitor boundary conditions and also at the quarter-height locations to monitor
the predicted maximum deflections. Deflections were measured at mid-width and quarter-width locations at the quarter-heights. This was done in order to see if there was any bowing of the panel. Typical string pots installed on a panel are shown in Figure 3.13.

![Diagram of string pot locations](image)

**Figure 3.12 – String Pot Locations**
3.5.3.3 *Pi Gages*

Re-usable strain gages, or PI-gages, were used to record tensile (positive) and compressive (negative) concrete strains on both the interior and exterior faces of the panels. Orthogonal pairs of gages were placed in six locations (PG 1, PG 3, PG 5, PG 6, PG7, PG 8), and single gages were placed in two locations (PG 2 and PG 4), as shown in Figure 3.14. Strains were measured on the panel ribs at the quarter-height and mid-height on both the interior and exterior panel faces. Also, the foam insulation was removed in a small area on the inner panel face to enable recording of the strain on the inside surface of the interior face.
For orthogonal pairs of PI-gages, the vertical gage is indicated by a V while the horizontal gage is indicated by an H. Typical PI-gages installed on a panel are shown in Figure 3.15.

**Figure 3.14 – Pi Gage Locations**
3.6 Test Results

Results from the load tests of panel RWP1 and panel RWP2 are presented in the following sections. Data are plotted with respect to the applied lateral force (lbs). The applied force was considered positive when the exterior side of the wall was in tension, and negative when the exterior side of the wall was in compression. Displacements causing the panel to move away from the exterior surface were considered positive. Positive strains were always considered as tension.

3.6.1 Summary of Test Results

Specimens RWP1 and RWP2 both resisted applied loads well in excess of their factored design loads prior to failure or test termination. Both specimens also sustained 5700
fatigue cycles with no perceptible damage, equivalent to a 50-year design lifetime, as described above. A summary of maximum sustained loads and observed failure modes for the tests of panels RWP1 and RWP2 are provided in Table 3-3. The failure modes and cracking patterns observed for the two specimens are described in detail in the following sections.

Table 3-3 – Summary of Ultimate Loads for Panels RWP1 and RWP2

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>RWP1: Test 1</td>
<td>+9,120 (2.4W)</td>
<td>-9,120 (2.4W)</td>
<td>Mid-Height Connection</td>
</tr>
<tr>
<td>RWP1 retest:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>strengthened connection</td>
<td>+15,200 (4.0W)</td>
<td>-15,200 (4.0W)</td>
<td>Limit of Test Setup</td>
</tr>
<tr>
<td>RWP2: Test 1</td>
<td>+15,200 (4.0W)</td>
<td>-15,200 (4.0W)</td>
<td>Limit of Test Setup</td>
</tr>
<tr>
<td>RWP2: no axial load</td>
<td>+15,200 (4.0W)</td>
<td>-15,200 (4.0W)</td>
<td>Limit of Test Setup</td>
</tr>
</tbody>
</table>

3.6.2 Panel RWP1: Test 1 Results

Test results including load-deflection, load-strain, cracking behavior, and mode of failure will be discussed in the following section for the first test of specimen RWP1.

3.6.2.1 Deflection: Service Load Level

The measured lateral deflections at the upper quarter-height location (SP-2) of
specimen RWP1 are shown in Figure 3.16 for 5700 loading cycles and up to the service and design load levels. The net lateral deflection of the panel was found by taking the average of the measured deflections at the restraint locations (SP-1 and SP-3), and then subtracting that average support deflection from the deflection measured at the upper quarter-height location (SP-2). Support deflections were similarly subtracted from other panel deflections. All figures show net panel deformations with any support deformations removed. Test data indicate a linear relationship between lateral load and deflection up to service and design load levels.

Figure 3.16 – Deflection (SP-2) vs. Lateral Load, 50-year Design Life and Service Load
The measured lateral deflections from the lower quarter-height location (SP-4) of specimen RWP1 are shown in Figure 3.17 for the 5700 loading cycles and up to the service and design load levels. Test data indicate a linear relationship between lateral load and deflection up to service and design load levels.

![Lateral Deflection (SP-4) vs. Load (Failure)](image)

**Figure 3.17 – Deflection (SP-4) vs. Lateral Load, 50-year Design Life and Service Load**

### 3.6.2.2 Strain: Service Load Level

The strain data measured during the initial test of RWP1 are presented in this section. Strain data are plotted in Figure 3.18 and Figure 3.19 from the 50-year life cycle fatigue test.
conducted on panel RWPI up to the service and design load levels. Figure 3.18 displays the vertical strain located at location PI-2V. As shown in Figure 3.14, the location of this gage is on the interior panel face, 8 inches above the mid-height support. The gage at this location measured the strain in the longitudinal direction of the interior surface of the concrete rib.

Figure 3.19 displays the vertical strain located at location PI-8V. As shown in Figure 3.14, location PI-8V is on the exterior face of the panel, 8 inches below the quarter-height test connection. This Pi-gage was used in order to measure the strain in the longitudinal direction opposite the concrete ribs on the exterior surface. These two locations are highlighted in this section as points of interest because they are the locations where the maximum stresses were developed on the panel. Both locations showed linear strain-load relationships. The magnitudes of measured strain values were small and were in tension or compression depending upon the loading direction. The small magnitudes of measured strain indicate that there was no concrete cracking or crushing.
Figure 3.18 – Strain (PI-2V) vs. Lateral Load, 50-year Design Life and Service Load
3.6.2.3 Cracking Pattern: Service Load Level

Specimen RWP1 did not show any evidence of cracking at either the service or factored design load levels. Specimen RWP1 after testing to the service load is shown in Figure 3.20. There are no structural cracks.
3.6.2.4 Failure Mode

Initial failure of panel RWP1 was observed at +2.4W. The panel sustained a load of -2.4W, with the mid-height connection in compression. After unloading from -2.4W and reloading to +2.4W, failure occurred due to the pull out of the T-bolts used to anchor the mid-height connection to the concrete. Specimen RWP1 and its failure mode at +2.4W are
shown in Figure 3.21.

![Figure 3.21 – Overview (left) and Close-view (right) of Panel RWP1 Failure Mode](image)

### 3.6.2.5 Deflection: Failure Load Level

The measured lateral deflections at the upper quarter-height location (SP-2) of specimen RWP1 are shown in Figure 3.22 for 5700 loading cycles and up to the initial connection failure. The net lateral deflection of the panel was found by taking the average of the measured deflections at the restraint locations (SP-1 and SP-3), and then subtracting that average support deflection from the deflection measured at the upper quarter-height location (SP-2). Support deflections were similarly subtracted from other panel deflections. All figures show net panel deformations with any support deformations removed.

Test data indicate a linear relationship between lateral load and deflection to a load level of approximately 9 kips (2.4W), well beyond the factored load level of 1.6W. Beyond the 9 kip lateral load level, a sudden drop in load is observed, indicating the onset of the
pullout failure at the mid-height connection. The sudden drop in load is followed by a sudden increase, and then a second sudden decrease as the T-bolts were pulled from their sockets.

![Graph showing Lateral Deflection (SP-2) vs. Load (Failure)](image)

**Figure 3.22 – Deflection (SP-2) vs. Lateral Load, 50-year Design Life and Failure**

The measured lateral deflections from the lower quarter-height location (SP-4) of specimen RWP1 are shown in Figure 3.23 for the 5700 loading cycles and the initial connection failure. Test data also indicate a linear relationship between lateral load and deflection to a load level of approximately 9 kips (2.4W). Beyond the 9 kip lateral load level, a sudden increase in deflection is observed, indicating the onset of the pullout failure at
the mid-height connection.

![Lateral Deflection (SP-4) vs. Load (Failure)](image)

**Figure 3.23 – Deflection (SP-4) vs. Lateral Load, 50-year Design Life and Failure**

3.6.2.6 *Strain: Failure Load Level*

The strain data measured during the initial test of panel RWP1 are presented in this section. Strain data are plotted in Figure 3.24 and Figure 3.25 from the 50-year life cycle fatigue test conducted on RWP1 up to the initial connection failure. Figure 3.24 displays the vertical strain located at location PI-2V. As shown in Figure 3.14, the location of this gage is on the interior panel face, 8 inches above the mid-height support. The gage at this location
measured the strain in the longitudinal direction of the interior surface of the concrete rib.

Figure 3.25 displays the vertical strain located at location PI-8V. As shown in Figure 3.14, location PI-8V is on the exterior face of the panel, 8 inches below the quarter-height test connection. This Pi-gage was used in order to measure the strain in the longitudinal direction opposite the concrete ribs on the exterior surface. These two locations are highlighted in this section as points of interest because they are the locations where the maximum stresses were developed on the panel. Both locations showed linear strain-load relationships. The magnitudes of measured strain values were small and were in tension or compression depending upon the loading direction. The small magnitudes of measured strain indicate that there was no concrete cracking or crushing.

The initial connection failure of panel RWP1 is shown in the same figure at a level of approximately 9 kips (2.4W). The failure is particularly evident in Figure 3.25, where the release of the middle connection sent the outer panel surface rapidly into tension.
Figure 3.24 – Strain (PI-2V) vs. Lateral Load, 50-year Design Life and Failure
3.6.3 Panel RWP1: Retest (Strengthened Connection)

After the initial connection failure, panel RWP1 remained nearly undamaged. Minimal cracking had developed, and the maximum measured strains were small. Therefore, it was decided to strengthen the mid-height connection to allow for further testing. The connection was re-attached to the panel, as shown in Figure 3.26, by drilling two through-holes and bolting the HSS to the panel with threaded rods. The panel was then re-tested under combined gravity and lateral loads. With the strengthened connection, panel RWP1
sustained a maximum lateral load of 4.0W in both the positive and negative directions before the test was terminated due to the capacity limit of the test setup. At the conclusion of the retest, specimen RWP1 had not failed, although heavier cracking on both panel surfaces had developed, and the panel had exhibited larger strains and deformations. Specimen RWP1 after testing to +/- 4.0W is shown in Figure 3.27. Note that all visible cracking were digitally highlighted in subsequent sections.

Figure 3.26 – Specimen RWP1 with Strengthened Mid-Height Connection
Figure 3.27 – Specimen RWP1 at Conclusion of Test (After Sustaining +/- 4.0W)

3.6.4 Panel RWP1: Retest Results

Test results including load-deflection, load-strain, cracking behavior, and mode of failure will be discussed in the following section.
3.6.4.1 Deflection: Maximum Load Level

The measured lateral deflections from the upper quarter-height location (SP-2) of specimen RWP1 are presented in Figure 3.28 for all load cycles applied during the retest of RWP1 (with the strengthened mid-height lateral connection).

With the strengthened connection in place, the panel was cycled to 4.0W in each direction with no indications of panel failure. The data indicate a linear relationship between lateral load and deflection up to a lateral load level of 12 kips. There is a distinct change in lateral stiffness near the 12 kip lateral load level in both the positive and negative directions with the onset of large cracking.
The measured lateral deflection from the lower quarter-height location (SP-4) of specimen RWP1 is presented in Figure 3.29, for all cycles conducted during the retest with the strengthened connection in place. The data from this location also indicate an initially linear relationship between lateral load and deflection, but a change in stiffness is observed at the 10 kip lateral load level. This decreased stiffness is a result of cracking of the panel at the measured locations. Also, there is an additional moment in the lower portion of the panel, due to the eccentric location of the gravity load at the mid-height of the panel. This moment increases or decreases according to the deflected shape of the panel due to P-Delta effects.
This may also explain the measured non-linear behavior associated with the load-deflection data seen in Figure 3.29. This change in stiffness was observed at a lower lateral load (10 kips) at this location (SP-4) as compared to the change in stiffness observed at 12 kips for the upper portion of the panel (SP-2).

![Lateral Deflection (SP-4) vs. Load (Maximum)](image)

**Figure 3.29 – Deflection (SP-4) vs. Lateral Load, Maximum Load Cycles**

3.6.4.2 *Strain: Maximum Load Level*

The strain data shown in Figure 3.30 and Figure 3.31 are for the re-test of RWP1 after strengthening the lateral connection. Figure 3.30 displays the strain measured from gage PI-
2V. As shown in Figure 3.14, PI-2V is on the interior face of the panel 8 inches above the mid-height constraint. Figure 3.31 presents the strains measured at location PI-8V. As shown in Figure 3.14, PI-8V is located 8 inches below the quarter-height actuator connection. Initially, both locations PI-2V and PI-8V showed similar linear relationships between applied load and strain. However, once the panel was subjected to higher level cycles and cracks began to develop, the measured strains significantly increased. The large crack that developed through PI-8V can be clearly seen in Figure 3.31. At both locations, the compressive strain did not approach the ultimate compressive strain of the concrete. Also, the measured tensile strain of the concrete indicates the presence of cracks and reflects the effective contribution of the steel in providing resistance to the applied loads. The effect of the gravity loads is clearly evident because the measured strains are in compression with zero applied lateral load. In addition, Figure 3.31 indicates two significantly different slopes to the strain-load relationship, dependant on whether the outer panel surface was in tension or compression. This is due to concrete cracking as well as P-Delta effects created by the eccentrically applied gravity loads. These gravity loads will increase or decrease the moments dependent upon which direction the lateral loads are applied.
Figure 3.30 – Strain (PI-2V) vs. Lateral Load, Maximum Load Cycles
3.6.4.3 **Cracking Pattern: Maximum Load Level**

Cracks were observed and marked to the extent possible during testing. After all tests were complete, panel RWP1 was removed from the test frame and all cracks were marked. The final cracking pattern of RWP1 after the retest is shown in Figure 3.32 and Figure 3.33.
Figure 3.32 – Interior Panel Cracking: Specimen RWP1 after all Tests (Surface cracks located on edges of panel across full height, cracks digitally highlighted)
3.6.5 Panel RWP2: Test 1

Specimen RWP2 also showed no visible signs of distress at either the service or factored loads. Recall that panel RWP2 was originally fitted with the strengthened connection to allow for testing to loads above 2.4W. As with panel RWP1, panel RWP2 was
loaded laterally to +/- 4.0W and the test was terminated due to capacity limitations of the test setup. Similar to RWP1, RWP2 did not show any conclusive signs of failure at the +/- 4.0W load levels. However, cracking was clearly evident on both panel surfaces. Panel RWP2 after loading to selected levels are shown in Figure 3.34 and Figure 3.35.

Figure 3.34 – Specimen RWP2 after Sustaining +/- Service Load: No Cracks
Figure 3.35 – Specimen RWP2 at Conclusion of Test (After Sustaining +/- 4.0W)
3.6.6 Panel RWP2: Test 1 Results

Test results including load-deflection, cracking behavior, and mode of failure will be discussed in the following section.

3.6.6.1 Deflection: Maximum Load Level

The measured lateral deflections at the upper quarter-height location (SP-2) of panel RWP2 is shown in Figure 3.36 for all lateral loading cycles completed on the panel in the presence of the applied gravity loads, including the 5700 initial cycles. The figure indicates a linear relationship between lateral load and deflection in the positive load direction up to the termination of the test at approximately 15 kips (+/- 4.0W). In the direction of negative applied load, the load-deflection response was linear through approximately 10 kips. This deviation from linearity at this load level is due to cracking and the P-Delta effects created by the eccentric gravity loads. The panel sustained -4.0W with stable lateral deflections.
The measured lateral deflection from the lower quarter-height location (SP-4) of specimen RWP2 is shown in Figure 3.37 for all loading cycles completed on the panel with the presence of gravity loads. The figure also indicates a linear relationship between lateral load and deflection in the positive direction up to approximately 10 kips. A sudden increase in deflection occurred at the 10 kip level due to the initiation of cracks at this level accompanied lower stiffness behavior. The eccentric gravity loads also contributed to the changes in stiffness.
3.6.6.2 Strain: Maximum Load Level

The strain data measured during the initial test of RWP2 are presented in this section. Strain data is plotted in Figure 3.38 and Figure 3.39 from the 50-year life cycle fatigue test conducted on RWP2 up to the maximum level cycles. Figure 3.38 displays the measured strain at location PI-2V. As shown in Figure 3.14, this location is on the interior panel face, 8 inches above the mid-height constraint. The gage at this location is used to measure strain in the longitudinal direction on the interior surface of the concrete rib.
Figure 3.39 displays the measured strain located at location PI-8V. As shown in Figure 3.14, location PI-8V is on the exterior face of the panel, 8 inches below the quarter-height test connection. This gage was used in order to measure the strain in the longitudinal direction opposite the concrete ribs on the exterior surface. These two locations are highlighted in this section as points of interest since they are the locations where the maximum stresses were developed on the panel. Both locations showed linear strain-load relationships. However, the strain profiles reflect that the concrete surface is cracked beyond a load level of 10 kips as evidenced by the large tensile strain. The measured compressive strain is significantly less than the ultimate compressive strain of concrete. This represents an effective and typical behavior of a reinforced concrete member.
Figure 3.38 – Strain (PI-2V) vs. Lateral Load, 50-year Design Life and Maximum Load
Figure 3.39 – Strain (PI-8V) vs. Lateral Load, 50-year Design Life and Maximum Load

3.6.6.3 Cracking Pattern: Maximum Load Level

Cracks observed during and after testing are shown in Figure 3.40 and Figure 3.41 for panel RWP2.
Figure 3.40 – Interior Panel Cracking: Specimen RWP2 after all Tests (Surface cracks located on edges of panel across full height, cracks digitally highlighted)

Figure 3.41 – Exterior Panel Cracking: Specimen RWP2 after all Tests (Cracking around the lower lateral load point, cracks digitally highlighted)
3.6.7 Panel RWP2: Retest (No Gravity Loads)

At the conclusion of the maximum lateral load cycles test conducted on RWP2, a second test was completed on the panel. This test was similar to the initial test on RWP2 but eliminated the effects of the gravity loads. The panel was subjected to the higher level lateral load cycles as before while the axial loads applied at the mid-height and top of the panel in this instance were not. Again, deflections and concrete strains were monitored in order to investigate the behavior of the panel without the effects of the gravity loads.

3.6.8 Panel RWP2: Retest Results

Test results including load-deflection, load-strain, cracking behavior, and mode of failure will be discussed in the following section.

3.6.8.1 Deflection: Maximum Load Level

The measured lateral deflections from the upper quarter-height location (SP-2) of specimen RWP2 are shown in Figure 3.42 for all lateral load cycles completed with the gravity loads removed. The figure indicates a linear relationship between lateral load and deflection until the termination of the test at approximately 15 kips (4.0W). The elimination of the gravity loads is visible in the plot. The maximum deflection in both positive and negative directions is the same while the lateral deflection with no lateral load applied was equal to zero. There was no indication of panel failure.
The measured lateral deflections from the lower quarter-height location (SP-4) of specimen RWP2 are shown in Figure 3.43 for all lateral load cycles completed with all gravity loads removed. The figure indicates a linear relationship between lateral load and deflection until termination of the test at approximately 15 kips (4.0W). The elimination of the gravity loads is visible in the plot. The maximum deflection in both positive and negative directions is the same while the lateral deflection with no lateral load applied was equal to zero. There was no indication of panel failure.
Figure 3.43 – Deflection (SP-4) vs. Lateral Load, Maximum Load (No Gravity Load)

3.6.8.2 Strain: Maximum Load Level

The strain data plotted in Figure 3.44 and Figure 3.45 are from the test conducted on RWP2 when no gravity load was applied. Figure 3.44 displays the strain measured from gage PI-2V. As shown in Figure 3.14, PI-2V is on the interior face of the panel 8 inches above the mid-height constraint. Figure 3.45 presents the strains measured at location PI-8V, 8 inches below the quarter-height actuator connection.

At first, both locations showed similar linear strain-load relationships. However,
once the panel started to undergo the higher level cycles, strain significantly increased. This is due to the pre-existing cracks from the first RWP2 testing. Both Figure 3.44 and Figure 3.45 indicate two significantly different slopes to the strain-load relationship, depending on whether the outer panel surface is in tension or compression. However, at both locations, the compressive strain did not approach the crushing strain.

![Image: Strain (PI-2V) vs. Load (Maximum)]

**Figure 3.44 – Strain (PI-2V) vs. Lateral Load, Maximum Load (No Gravity Load)**
3.6.9 Effects of Gravity Loads

Both panels RWP1 and RWP2 were initially loaded with gravity loads at the mid-height. These gravity loads were simulated using axial forces applied to the panel corbels simultaneously. These axial forces were eccentrically applied to the interior face of the panel and consequently created concentrated moments at the mid-height of the panel. Gravity loads were also applied concentrically to the top face of the panel. However, the center of gravity of the panel with respect to its thickness will not be the same as the location of these
top loads. Therefore, these axial loads are in reality applied eccentrically as well. Prior to any lateral loads being applied, deflections and strains were monitored in order to analyze the effects of these eccentric axial loads. Figure 3.46 shows the critical displacements at the top quarter-height location (SP-2) and at the bottom quarter-height location (SP-4). The effects of the eccentric load are visible as the concentrated moments create an initial negative displacement at the top quarter-height location and an initial positive displacement at the bottom quarter-height location. This initial displacement due to the gravity loads is visible in all load-deflection data plots seen in prior sections of the thesis. Each plot shows an initial shift in displacement, whether negative (SP-2) or positive (SP-4), when the applied lateral load is zero.
Figure 3.46 – Lateral Deflection vs. Gravity Load, No Lateral Load

The concentrated moments also create initial concrete strains on the panel. Table 3-4 displays the strain at location PI-2V and location PI-8V. The effects of the eccentric load are visible as the moment puts the panel surface in tension at both locations creating a positive strain. This initial tensile strain due to the gravity loads is visible in all strain-deflection data plots seen prior. Each plot shows an initial shift in strain in the positive tensile direction.
when the lateral load is zero at these locations.

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<thead>
<tr>
<th>Load</th>
<th>Gage 2</th>
<th>Gage 8</th>
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</thead>
<tbody>
<tr>
<td>All Gravity Loads</td>
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<td>0.0000561</td>
</tr>
</tbody>
</table>

### Table 3-4 – Strain vs. Gravity Load, No Lateral Load

3.7 **Experimental Summary and Conclusions**

Two precast concrete ribbed wall panels were tested in this experimental program. Both specimens measured 20’ tall by 8’ wide by 7.25” thick. Each specimen consisted of a concrete ribbed panel filled with integrated thermal insulation and topped with a concrete layer containing randomly-oriented synthetic fibers. The panels were tested to simulate a 50-year design life prior to applying higher level loads to evaluate the behavior and failure mechanism.

Observations from the testing of RWP1:

1. Minimal cracking was observed along the edges of both the interior and exterior surfaces of RWP1 up to the factored design load. The factored design load was applied after the 50-year design life fatigue cycles were completed. Factored and gravity loads were in place during all lateral load cycles.

2. Panel RWP1 initially failed at approximately 9,120 lbs (2.4W) on the positive
portion of the loading cycle (pushing on the inner face).

3. The initial failure mode observed for panel RWP1 was pullout of the mid-height connection. The two 5/8” T-bolts pulled out of their associated embedded slots.

4. The mid-height connection was strengthened and panel RWP1 was re-tested with the gravity loads applied.

5. The retest of panel RWP1 was terminated at +/- 15,200 lbs (+/- 4.0W).

6. At the conclusion of the retest, panel RWP1 had developed many large surface cracks on the exterior side of the panel. However, the panel was able to sustain the applied loads at these high levels.

Observations from the testing of RWP2:

1. Minimal cracking was observed along the edges of both the interior and exterior surfaces of RWP2 up to the factored design load. The factored design load was applied after the 50-year design life fatigue cycles were completed. Factored gravity loads were in place during all lateral load cycles.

2. Panel RWP2 was tested with the strengthened connection in order to test the
panel’s response to high level loading while avoiding mid-height connection failure.

3. The test of panel RWP2 was terminated at +/- 15,200 lbs (+/- 4.0W).

4. At the conclusion of testing, the panel had developed many large surface cracks on the exterior face, but did not show any indication of failure.

5. Panel RWP2 was re-tested with the gravity load removed to +/- 4.0W.

Conclusions:

1. The precast concrete ribbed panels behaved in a satisfactory manner at service and design factored loads. Cracking and deformations were minimal at service and factored design levels. The measured load-deflection behaviors remained linear through the factored load level in all cases.

2. The panel design appears to be limited by the strength of the lateral connections.

3. The panel behaved in a very stiff manner, and all measured deflections and strains remained relatively small at the factored design level.
4 ANALYTICAL INVESTIGATION

4.1 Introduction

This chapter presents an analytical model developed to simulate the behavior of the full scale testing completed and presented in Chapter 3. The model was used to study selected parameters believed to affect the panel behavior. The study attempted to optimize the panel configuration and predict panel behavior. Using the analytical model, the expenses of undertaking many full scale tests to study these types of panels can be eliminated.

4.2 Overview

The goal of this study was to develop an analytical model to predict panel behavior which would allow for the analysis of several parameters to optimize designs. The first model used was based on simple frame analysis. The maximum deflections, which will be referred to as the critical deflections, were the output results of interest. However, the frame analysis did not have the capability of accurately predicting localized effects, particularly with respect to stress and strain, at critical locations. Therefore it was necessary to develop a finite element model to accurately predict the state of stresses and strains over the entire panel. The results from the frame analysis were mainly used to judge the overall accuracy of the finite element model by comparing the critical deflections. This model was then configured to replicate the experimental procedure. The boundary and load conditions were assumed to be the same. The finite element model was then calibrated using the deflection
and strain values from the experimental program. Once the model was calibrated, a parametric study was conducted on alternative panel configurations. The selected parameters considered in this study were reduction in number of interior ribs, decrease in width of exterior ribs, and exclusion of the middle support. The critical deflections and strains from the results of the reconfigured models were then assessed to judge the practicality of each panel configuration.

4.3 Phase I: Frame Analysis

The first part of the modeling procedure was a frame analysis. This analysis was completed using the frame analysis software program SAP2000. A standard 2D beam frame element was established to simulate the tested panel. The restraint conditions used were a pin support and a roller support. The pin support was located at one end of the beam while a roller support was located at the other end. The middle roller support was located 125 inches from the pin support and 115 inches from the roller support which is an accurate representation of the experimental setup. An elastic modulus value, $E$, of the concrete material was used for the entire panel. This elastic modulus, $E$, was determined using the ACI 318-08 equation in terms of measured ultimate compressive strength at 28 days, $f'_c$, and concrete unit weight, $w_c$, as follows:

$$E = 33(w_c^{1.5})\sqrt{f'_c}$$  \hspace{1cm} \text{Equation 4.1}

The moment of inertia ($I$) used in the analysis was variable along the length of the panel.
Separate values were estimated for the moment of inertia of the solid concrete sections of the panel and for the ribbed sections of the panel. These different I values were then applied to the corresponding sections within the panel in the simple frame analysis. This was done in order to account for the change in panel stiffness of the indeterminate system used to test the panels.

Unit lateral loads (1 kip) were applied to the panel at two locations. These locations were at the lower quarter-height of the panel and the upper quarter-height of the panel. The loads were positive in value. Only these lateral loads were applied. There were no gravity loads. The frame analysis was then performed. The critical deflection results were then analyzed. These locations will be referred to as SP-2 for the top quarter-height location and SP-4 for the bottom quarter-height location. These reference names are used in order to correspond with locations defined in Chapter 3 of this thesis. The unit lateral loads were then applied in the opposite direction given a negative value. Once again, no gravity loads were applied, only the lateral loads. The frame analysis was then performed and the same critical results were analyzed. The critical deflections are given in Table 4-1 for the loading in both directions. The displacement diagrams of the beam are shown in Figure 4.1.

**Table 4-1 – Frame Analysis: Critical Deflections**

<table>
<thead>
<tr>
<th>Frame Analysis</th>
<th>Load</th>
<th>Positive (1 kip)</th>
<th>Negative (-1 kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.0021 in.</td>
<td>-0.0021 in.</td>
<td></td>
</tr>
<tr>
<td>SP-4</td>
<td>0.0031 in.</td>
<td>-0.0031 in.</td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.1 – Frame Analysis: Deflections

The induced deflections from the analysis were equal, as expected, due to the omission of the gravity loads. While the frame analysis allows for an accurate prediction of these deflections, it does not accurately provide a way to predict local effects such as stress and strain. This is due to the combination of ribbed and solid panel sections and the complexity of the experimental set-up. Therefore a finite element model was required in order to better analyze the behavior of precast concrete panels. The frame analysis was used
only to judge the modeling performed using finite element techniques by comparing the
deflections at the selected critical locations.

4.4 Phase II: Finite Element Model

A finite element model was developed to simulate the behavior of the tested panels. The model was established using the finite element analysis software program Abaqus/CAE. A 3D solid model was chosen in order to replicate the panel as closely as possible. This solid was dimensioned according to the panel specifications to model the section properties of the panel. The same elastic modulus, E, used in the frame analysis, was also used for the finite element model. The load and support conditions were selected to be identical to the frame analysis. A pin restraint was placed across the entire width of the bottom face of the panel in the center to allow for rotation only. Roller restraints were placed at the mid-height and top locations to allow for rotation and vertical translation. The mid-height support was placed across the entire width of the interior face of the panel 125 inches from the base of the panel or 115 inches from the top of the panel. The top support was placed across the entire width of the top face of the panel in the center. Also, lateral loads were applied to the panel at the lower quarter-height and upper quarter-height. This was done slightly different than the frame analysis due to the three dimensional nature of the model. Instead of using one individual unit point load at each location, 24 point loads of magnitude 0.04167 kips were spaced equally across the exterior panel face at the quarter-height locations. The sum of these loads was unity (1 kip). No gravity loads were applied for this initial analysis. The
solid model is shown in Figure 4.2. All support conditions and the lateral loads are also shown in Figure 4.2.

![Solid model](image)

**Figure 4.2 – Finite Element Analysis: Solid Model**

Using a solid element, the elements were selected using basic finite element principals. Several iterations were performed using tetrahedral, wedge, and hexahedral elements. Each iteration attempted to refine the mesh using higher order elements. The final element selected was a rectangular solid element (brick element). The 3D solid mesh is shown in Figure 4.3. This element has eight nodes located at each corner of the brick. This
allows for 24 possible degrees of freedom, three per node in the x, y, and z direction. The order of the shape functions are trilinear due to the three linear terms (xyz). Each element size was approximately two inches. The shape functions and size of the element were limited by the software’s computing capacity. The finite element analysis was performed to determine the critical deflections when loaded equivalent to unity in both the positive and negative directions in order to compare these results to the frame analysis that were initially performed. These results are given in Table 4-2.

![Figure 4.3 – Finite Element Analysis: Meshed Solid Model](image)
Table 4-2 – Finite Element Analysis: Critical Deflections

<table>
<thead>
<tr>
<th>Load</th>
<th>Positive (1 kip)</th>
<th>Negative (-1 kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.00207 in.</td>
<td>-0.00207 in.</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.00314 in.</td>
<td>-0.00314 in.</td>
</tr>
</tbody>
</table>

The results from the finite element modeling were then compared with the results from the frame analysis. This was done in order to judge the accuracy of the finite element model. The comparison is summarized in Table 3. The table compares the deflections at the upper quarter-height (SP-2) and lower quarter-height (SP-4). The finite element analysis yielded deflections similar to those of the frame analysis. The percent difference was less than 2 percent for both critical locations. Therefore, the accuracy of the finite element model is confirmed and the frame analysis justifies the use of the model in analyzing the panel.

Table 4-3 – Comparison of Results: Frame Analysis and FEA

<table>
<thead>
<tr>
<th>Analytical Panel Deflection</th>
<th>Frame Analysis</th>
<th>FEA</th>
<th>% Diff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2 (in.)</td>
<td>0.0021</td>
<td>0.00207</td>
<td>1.4</td>
</tr>
<tr>
<td>SP-4 (in.)</td>
<td>0.0032</td>
<td>0.00314</td>
<td>1.9</td>
</tr>
</tbody>
</table>

4.5 Phase III: Calibration of the Finite Element Model

The finite element model was then reconfigured to simulate the tested panels and imposed boundary conditions. The material and section properties were kept the same. The support conditions described in the previous section all remained the same as well, but the
load conditions changed. The first change was the application of the lateral loads. The experimental lateral loads described in Chapter 3 were applied by rigid HSS members attached to the panel with neoprene pads. This test procedure spaced the lateral loads uniformly across the ribs with a width of 4 inches, the size of the HSS section. Therefore, the lateral loads were applied to the finite element model using pressure loads on 4 inch widths across the ribs. The total pressure load over the total load surface area was equal to the forces applied to the panel by the 22-kip actuators. This lateral load was applied in both directions to simulate the positive and negative load cycles. The second change to the model was the addition of the gravity loads. These were placed at the top of the panel and at the mid-height in order to simulate the test conditions. The top gravity loads were applied using pressure loads over a 4x4 inch area. This pressure with respect to the given bearing area was equal to the force applied by the top actuators during testing, or 3 kips per location. Finally, the mid-height gravity loads were applied using a concentrated force at each corbel location as well as a concentrated moment. The axial force was equal to the gravity load force applied during the test, or 5 kips per location. The concentrated moment was applied in order to simulate the effects of the eccentrically loaded axial force. This moment was equal to 17.5 kip-inches per location. The finite element model is shown in Figure 4.4. The reconfigured lateral loads as well as the gravity loads can be seen in addition to the boundary conditions.
The model was initially run and the results were compared with the measured values from the testing. The critical deflections and strains predicted by the model were all smaller in magnitude than measured from the testing. A comparison of results indicates that the model behaves much stiffer than the panels observed during testing. The concrete element’s stiffness was based on values determined using the ACI 318-08 equation. This equation typically overpredicts the elastic modulus value for high strength precast concrete. Also, the effects of small aggregates in the flange could also cause a decrease in the elastic modulus.
Therefore, it was necessary to use the experimental results to calibrate the model. It was decided to use 70 percent of the values predicted by the ACI equation to account for the small aggregate in the slabs.

The deflections for the panel loaded at the design level (1.6W) are shown in Figure 4.5 for both positive and negative loading. The critical deflections at the upper and lower quarter-heights (SP-2 and SP-4) from the finite element model are summarized in Table 4-4 as well as the critical deflections from the measured values. A graphical representation of the deflections from both the experiment and the model is shown in Figure 4.6.

![Figure 4.5 – Deflections: FEA](image-url)
Table 4-4 – Critical Deflections: FEA and Experimental Results

<table>
<thead>
<tr>
<th>Panel Deflection (in.)</th>
<th>Experimental</th>
<th>Finite Element</th>
<th>%Diff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.03006</td>
<td>0.02402</td>
<td>22.3</td>
</tr>
<tr>
<td>SP-2</td>
<td>-0.06072</td>
<td>-0.03449</td>
<td>55.1</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.05163</td>
<td>0.04886</td>
<td>5.5</td>
</tr>
<tr>
<td>Sp-4</td>
<td>-0.02271</td>
<td>-0.01824</td>
<td>21.8</td>
</tr>
</tbody>
</table>

Figure 4.6 – Comparison of Critical Deflections: FEA and Experimental Results
The comparison indicated that the deflection pattern is similar for both the predicted and measured values at most locations. The predicted values may be slightly smaller than the measured values. The percent differences are relatively small at the bottom quarter-height location (SP-4) as well as for the top quarter-height location (SP-2) when loaded in the positive direction. However, the percent difference at the top quarter-height location (SP-2) when loaded in the negative direction is higher. The effects of the gravity loads can also be seen in the results. The lateral load is the same whether loaded in the positive or negative direction. However, the addition of the eccentric gravity loads, and thus concentrated moments, create a larger deflection at the bottom quarter-height location (SP-2) when loaded in the positive direction and at the top quarter-height location (SP-4) when loaded in the negative direction.

The model was deemed to be acceptable and calibrated with respect to the experimental deflection results. There are several reasons for this. First, the magnitudes of the deflections for the experiment are less than one tenth of an inch. Therefore, the predicted values from the model fall within the accuracy of the measuring instruments. The second reason is due to the experimental setup. The supports were built in order to simulate certain restraining conditions. The model on the other hand will theoretically idealize these restraints. Therefore some inaccuracies will exist between the experimental and predicted results, which results in larger experimental deflections. This can particularly be seen in the deflection results from the experimental panel at the top-quarter height location (SP-4) when loaded in the negative direction. The top of the panel has a shorter clear height (115 inches) than the bottom clear height of the panel (125 inches). Therefore, engineering theory
predicts that the deflection at the top quarter-height can not be larger than the deflection at the bottom quarter-height. This expected result is not seen in the experimental results, unlike in the predicted results. This can likely be attributed to some variation in the simulation of the boundary conditions.

The strains for the panel loaded at the design level (1.6W) are shown in Figure 4.7 for both positive and negative loading. The critical strains from the finite element model are summarized in Table 4-5 as well as the critical strains from the experimental test results. These gage locations are at the bottom quarter-height on both the interior face of the ribs (Gage 4) and opposite on the exterior face (Gage 8). The other gage locations are at the upper intersection of the solid panel section and the ribs on the interior face (Gage 2) and opposite on the exterior face (Gage 6).
Table 4-5 – Critical Strains: FEA and Experimental Results

<table>
<thead>
<tr>
<th>Strain Data</th>
<th>Load (kips)</th>
<th>Gage 2</th>
<th>Gage 6</th>
<th>Gage 4</th>
<th>Gage 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>6.08</td>
<td>0.1370</td>
<td>-0.0875</td>
<td>-0.1852</td>
<td>0.0933</td>
</tr>
<tr>
<td></td>
<td>-6.08</td>
<td>-0.1128</td>
<td>0.0217</td>
<td>0.0589</td>
<td>-0.0985</td>
</tr>
<tr>
<td>FEA</td>
<td>6.08</td>
<td>0.1713</td>
<td>-0.0943</td>
<td>-0.1944</td>
<td>0.0679</td>
</tr>
<tr>
<td></td>
<td>-6.08</td>
<td>-0.1211</td>
<td>0.0358</td>
<td>0.0651</td>
<td>-0.0904</td>
</tr>
<tr>
<td>% Diff.</td>
<td>6.08</td>
<td>-22.2</td>
<td>-7.5</td>
<td>-4.9</td>
<td>31.5</td>
</tr>
<tr>
<td></td>
<td>-6.08</td>
<td>-7.1</td>
<td>-49.0</td>
<td>-10.0</td>
<td>8.6</td>
</tr>
</tbody>
</table>
The predicted strain values are close to the measured results at most of the locations. The model in most cases predicts larger concrete strains than the measured values except on the exterior face of the panel at the bottom quarter-height. Also, the model seems to predict compressive strain values closer to the measured values than it does for tensile strain values.

The model was deemed to be acceptable and calibrated with respect to the experimental strain results. There are several reasons for this. First, the predicted values from the model fall within the accuracy of the measuring instruments. Another reason is that the model does not take into account the effects of the fiber reinforced concrete. The chopped fibers will reduce the measured strain values on the exterior surface of the panel. The last reason is relative to the instruments. The Pi-gages used measure average concrete strain over a certain gage length. The model predicts concrete strain in a more localized manner. Therefore it would be expected that the measured average strain would be less than the predicted local strain. This trend is seen in the majority of Table 4-5.

Once the model was calibrated using the experimental deflections and strains, it was deemed that the finite element model was an acceptable tool for analyzing and predicting the behavior of the tested precast concrete panel.

4.6 Phase IV: Parametric Study

The ultimate goal of using precast concrete panels as structural elements is to create the most efficient structural system possible and to speed the time of construction. The
experimental program indicates that the panel failed at a load over twice the service load (2.4W) due to a localized connection failure. The panel itself remained structurally adequate and serviceable. Therefore it may be desirable to optimize the panel configuration to create more efficient panels. An alternative design to the current panel would then create savings in material, size and weight, and fabrication cost and time, as well as a more simplistic design, and an easier construction process. These new panel configurations could have fewer ribs, thinner exterior ribs, or be used without a mid-height connection allowing for a clear two story height which is convenient for commercial applications.

The analytical model that was developed was intended to be used as a tool to investigate the potential behaviors of the reconfigured panels. The controlling design features are those related to serviceability and not strength requirements. Since, these panels are intended to be used as the exterior envelope of buildings they will be visible. Thus, it is not desirable to have large cracking on the exterior surface. Therefore, the formation of cracks on the exterior surface should be controlled or eliminated. ACI 318-08 will be used in order to predict the formation of cracks. The rupture or cracking stress was calculated and then converted to strain using the reduced value of E. The code approximates cracking strain about 0.00019. Also, maximum deflections must remain under the serviceable limit states. For precast concrete panels this limit is defined by ACI 533. The code states that the deflection may not exceed 1/360 or ¾ of an inch. In this case, 1/360 is equal to 0.333 inches. The following sections will discuss the parametric study, present the results from the study, and provide recommendations based on the parametric studies.
4.6.1 Effect of Applied Uniform Pressure Load vs. Concentrated Load

The first analysis performed for the parametric study used the original model that simulated the tested panel. The only difference in this study is the simulation of the lateral load. Instead of using two uniform loads at the quarter-heights on the ribs, similar to the testing protocol, a uniform pressure load was applied to the entire exterior surface of the panel. This was done in order to investigate the behavior of the panel under a more realistic wind load condition. A positive pressure was applied to the panel as well as a negative pressure, or suction. The gravity loads were applied at all times during the analysis at both the mid-height and on top of the panel. The pressure load applied was equal to the design load (1.6W). The maximum deflections and concrete strains were then analyzed to assess the performance and behavior of the panel. A summary of the critical deflections at the upper quarter-height (SP-2) and lower quarter-height (SP-4) are shown in Table 4-6. Results of the analysis indicated that the deflections were smaller and decrease in magnitude relative to the results generated from the model using experimental conditions. This is expected because a uniform loading applied over the entire surface, as opposed to a more concentrated load at the quarter-heights, will generate smaller deflections. All deflections remain under the serviceable limit for this panel configuration. A comparison between the critical deflections at the top quarter-height and bottom quarter-height generated by the original model and the pressure loaded model is shown in Figure 4.8.
Table 4-6 – Critical Deflections: Uniform Pressure Load

<table>
<thead>
<tr>
<th>Load</th>
<th>+1.6W</th>
<th>-1.6W</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.01331</td>
<td>-0.02115</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.03482</td>
<td>-0.00531</td>
</tr>
</tbody>
</table>

Figure 4.8 – Comparison of Critical Deflections: Uniform Loading
A summary of the critical strains are given in Table 4-7 and the strain contour is shown in Figure 4.9. When the panel is loaded in the positive direction (suction), the maximum tensile strains are located at the upper intersection of the ribs with the solid middle section while the maximum compressive strains are located at the lower-quarter height on the ribs. When the panel is loaded in the negative direction (pressure), the maximum tensile strains are located at the upper-quarter height on the ribs while the maximum compressive strains are located at the lower intersection of the ribs with the solid middle section. The maximum tension strain on the exterior surface of the panel must also be considered in order to predict the exposure of cracking. The strain values for the pressure loading model are lower than those from the original quarter-height loading model as well.

<table>
<thead>
<tr>
<th>Strain Data</th>
<th>Load (kips)</th>
<th>Maximum Tension (Ribs)</th>
<th>Maximum Compression (Ribs)</th>
<th>Maximum Tension (Ext. Face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>6.08 (1.6W)</td>
<td>0.0002068</td>
<td>-0.0001518</td>
<td>0.00003565</td>
</tr>
<tr>
<td></td>
<td>-6.08 (-1.6W)</td>
<td>0.00007186</td>
<td>-0.0001823</td>
<td>0.00003862</td>
</tr>
</tbody>
</table>
A few conclusions can be drawn from the model loaded with the uniform pressure load. The first is that an analysis using the quarter-height load techniques will result in larger deflections and strains. Therefore the panel that was tested experimentally will perform with less desirable results as opposed to a panel that is tested with full surface uniform pressure loads. In otherwords, the testing represents worst case scenarios. Also, the model predicts that the panel will behave adequately at the service and design load levels. The deflections are small and much less than the limit states. Also, the strains are small and no cracking is
predicted to occur on the exterior face of the panel. The maximum tensile strains do approach the cracking strain and may exceed this value on the interior ribs. However, since the interior face is not exposed it will be allowed.

4.6.2 Use of Only Two Interior Ribs

The second analysis run in the parametric study investigated the influence of the number of interior ribs. The middle rib was eliminated and the two remaining ribs were offset from the exterior ribs at an equal distance. The reconfigured panel is shown in Figure 4.10.
A uniform pressure load was applied to the exterior surface of the panel in both directions. This pressure was equal to the design load (1.6W). The gravity loads were applied at the top and mid-height locations at all times. The maximum deflections and concrete strains were then analyzed. A summary of the critical deflections at the top quarter-height (SP-2) and bottom quarter-height (SP-4) are shown in Table 4-8. The deflections increase from those seen in the original experimental program, but the overall deflections are still small.
Table 4-8 – Critical Deflections: Two Interior Ribs

<table>
<thead>
<tr>
<th>Load</th>
<th>+1.6W</th>
<th>-1.6W</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.1062</td>
<td>-0.1194</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.1342</td>
<td>-0.09786</td>
</tr>
</tbody>
</table>

A summary of the critical strains are given in Table 4-9. When the panel is loaded in the positive direction, the maximum tensile strains are located at the upper intersection of the ribs with the solid middle section while the maximum compressive strains are located at the lower-quarter height on the ribs. When the panel is loaded in the negative direction, the maximum tensile strains are located at the upper-quarter height on the ribs while the maximum compressive strains are located at the lower intersection of the ribs with the solid middle section. The maximum tension strain on the exterior surface of the panel must also be considered in order to predict exposed cracking. The strains increase from those seen in the original experimental program, but the overall strains are still very small.

Table 4-9 – Critical Strains: Two Interior Ribs

<table>
<thead>
<tr>
<th>Strain Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load (kips)</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>2 Ribs</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

The same limit states, deflection and exterior surface panel cracking, will control this panel. However, as the spacing of the ribs increases, the localized stiffness of the exterior
skin will start to affect the behavior of the panel. Therefore, special considerations should be made. First, a new deflection limit state relative to the width of the panel (96 inches) should be considered. This limit will be equal to 0.267 inches. Next, a reduced value for cracking strain should be used within the skin of the panel. This limit will be 0.000075.

A few conclusions can be drawn from the model that only contained two interior ribs. The first is that this panel will result in larger deflections and strains. However, the deflections are small in magnitude, all within allowable limitations, and follow the same trends as seen in prior analysis. The strains are small as well and no cracking is predicted to occur on the exterior face of the panel. However, the maximum tensile strains do predict the onset of cracking on the interior face of the panel on the ribs. Since the interior panel face is not exposed this will be allowed. Lastly, the model predicts that the panel will behave adequately at the service and design load levels. Therefore, the conclusion can be drawn that this panel configuration will optimize the benefits of precast concrete and provide a more efficient system.

4.6.3 Use of Only One Interior Rib

The next analysis run in the parametric study continued the investigation on the influence of the interior ribs. The center interior rib was left in place in this analysis, but the two remaining interior ribs were eliminated. The reconfigured panel is shown in Figure 4.11.
A uniform pressure load was applied to the exterior surface of the panel in both directions. This pressure was equal to the design load (1.6W). The gravity loads were applied at the top and mid-height locations at all times. The maximum deflections and concrete strains were then analyzed. A summary of the critical deflections at the top quarter-height (SP-2) and bottom quarter-height (SP-4) are shown in Table 4-10. The deflections increase from those seen in the original experimental program, but in this case, the deflections have now exceeded those allowed. These maximum deflections are all located within the center of the exterior skins as shown in Figure 4.12. Therefore the effects of the
localized stiffness created within the skin control the resulting deflections.

Table 4-10 – Critical Deflections: One Interior Rib

<table>
<thead>
<tr>
<th>Load</th>
<th>+1.6W</th>
<th>-1.6W</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.3868</td>
<td>-0.4029</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.4166</td>
<td>-0.3704</td>
</tr>
</tbody>
</table>

Figure 4.12 – Deflection: One Interior Rib
A summary of the critical strains are shown in Table 4-11. When the panel is loaded in the positive direction, the maximum tensile strains are located at the upper intersection of the ribs with the solid middle section while the maximum compressive strains are located at the lower-quarter height on the ribs. When the panel is loaded in the negative direction, the maximum tensile strains are located at the upper-quarter height on the ribs while the maximum compressive strains are located at the lower intersection of the ribs with the solid middle section. The maximum tension strain on the exterior surface of the panel must also be considered in order to examine possible cracking. The strains increase from those seen in the original experimental program, but in this case, the strains have now exceeded those allowed. The panel strains are shown in Figure 4.13.

<table>
<thead>
<tr>
<th>Load (kips)</th>
<th>Maximum Tension (Ribs)</th>
<th>Maximum Compression (Ribs)</th>
<th>Maximum Tension (Ext. Face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Rib</td>
<td>6.08 (1.6W)</td>
<td>0.0003273</td>
<td>-0.0002316</td>
</tr>
<tr>
<td></td>
<td>-6.08 (-1.6W)</td>
<td>0.0001265</td>
<td>-0.0002637</td>
</tr>
</tbody>
</table>

Table 4-11 – Critical Strains: One Interior Rib
The same limit states, deflection and exterior surface panel cracking, will control this panel. However, as the spacing of the ribs increases, the localized stiffness of the exterior skin will start to affect the behavior of the panel. Therefore, special considerations should be made. First, a new deflection limit state relative to the width of the panel (96 inches) should be considered. This limit will be equal to 0.267 inches. Next, a reduced value for cracking strain should be used within the skin of the panel. This limit will be 0.000075.

A few conclusions can be drawn from the model that included only one interior rib.
The first is that this panel will result in significantly larger deflections and strains. These maximum deflections exceed the allowable deflections for this precast panel configuration. The strains increase but still remain relatively small without initiation of cracking of the exterior face of the panel. However, the maximum tensile strains do predict the onset of cracking on the interior face of the panel on the ribs. Lastly, the model does not predict that the panel will behave adequately at the service and design load stages. The predicted deflections exceed the serviceability requirements for the panel. Therefore this panel configuration would not be recommended.

4.6.4 Reducing the Width of the Exterior Ribs

The following analysis performed in the parametric study investigated the influence of the width of the exterior ribs. The exterior ribs were reduced in width from the specified 6 inches to match the width of the interior ribs and were thus equal to 2.5 inches. The reconfigured panel is shown in Figure 4.14.
A uniform pressure load was applied to the exterior surface of the panel in both directions. This pressure was equal to the design load (1.6W). The gravity loads were applied at the top and mid-height locations at all times. The maximum deflections and concrete strains were then analyzed. A summary of the critical deflections at the top quarter-height (SP-2) and bottom quarter-height (SP-4) are shown in Table 4-12. The deflections increase from those seen in the original experimental program, but the overall deflections are still very small.
Table 4-12 – Critical Deflections: Thin Exterior Ribs

<table>
<thead>
<tr>
<th>Load</th>
<th>+1.6W</th>
<th>-1.6W</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.01304</td>
<td>-0.02496</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.04401</td>
<td>-0.01237</td>
</tr>
</tbody>
</table>

A summary of the critical strains are shown in Table 4-13. When the panel is loaded in the positive direction, the maximum tensile strains are located at the upper intersection of the ribs with the solid middle section while the maximum compressive strains are located at the lower-quarter height on the ribs. When the panel is loaded in the negative direction, the maximum tensile strains are located at the upper-quarter height on the ribs while the maximum compressive strains are located at the lower intersection of the ribs with the solid middle section. The maximum tension strain on the exterior surface of the panel must also be considered in order to examine possible cracking. The strains increase from those seen in the original experimental program, but the overall strains are still very small.

Table 4-13 – Critical Strains: Thin Exterior Ribs

<table>
<thead>
<tr>
<th>Strain Data</th>
<th>Load (kips)</th>
<th>Maximum Tension (Ribs)</th>
<th>Maximum Compression (Ribs)</th>
<th>Maximum Tension (Ext. Face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin Ribs</td>
<td>6.08 (1.6W)</td>
<td>0.00002391</td>
<td>-0.0001904</td>
<td>0.000004321</td>
</tr>
<tr>
<td></td>
<td>-6.08 (-1.6W)</td>
<td>0.00008211</td>
<td>-0.0002044</td>
<td>0.00005995</td>
</tr>
</tbody>
</table>

A few conclusions can be drawn from the model that reduced the width of the two exterior ribs. The first is that this panel will result in larger deflections and strains. The
deflections are small though, follow the same trends as seen in prior analysis, and are much less than the allowable limitations. The strains are small as well without initiation of cracking of the exterior face of the panel. However, the maximum tensile strains do potentially predict the onset of some initial cracking on the interior face of the panel on the ribs. Lastly, the model predicts that the panel will behave adequately at the service and design load stages. Therefore, the conclusion can be drawn that this panel configuration will optimize the benefits of precast concrete and provide a more efficient system. However, the limiting dimensions for any panel connections on these exterior ribs would also need to be considered.

4.6.5 Two Story Clear Height Panels

The next analysis run in the parametric study investigated the influence of the middle support of the panel. Instead of treating the panel as a continuous frame element with a roller condition at the mid-height, the panel was treated as one span equal to 20 feet in height. This was done by eliminating the middle restraint condition. In addition to eliminating the middle support, the gravity loads were no longer applied at the mid-height location. However, the rest of the panel features remained the same as the original experimental panel including the dimensions, load conditions, and restraints. The maximum deflections and concrete strains were then analyzed. A summary of the critical deflections at the top quarter-height (SP-2), bottom quarter-height (SP-4), and at the mid-height of the panel are shown in Table 4-14. The maximum allowable deflection given by the code will change due to the increase in clear
height. Instead of using a limit of 0.333 inches, a limit of 0.667 inches will be used. The panel deflections are shown in Figure 4.15.

### Table 4-14 – Critical Deflections: No Middle Support

<table>
<thead>
<tr>
<th>Load</th>
<th>+1.6W</th>
<th>-1.6W</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-2</td>
<td>0.4105</td>
<td>-0.3642</td>
</tr>
<tr>
<td>Mid-height</td>
<td>0.5474</td>
<td>-0.4854</td>
</tr>
<tr>
<td>SP-4</td>
<td>0.4105</td>
<td>-0.3642</td>
</tr>
</tbody>
</table>

**Figure 4.15 – Deflection: No Middle Support**
A summary of the critical strains are shown in Table 4-15. When the panel is loaded in the positive direction, the maximum tensile strains are located on the exterior face of the panel opposite both the upper and lower intersection of the ribs with the solid middle section. The maximum compressive strains are located at both the upper and lower intersection of the ribs with the solid middle section. When the panel is loaded in the negative direction, the maximum tensile strains are located at both the upper and lower intersection of the ribs with the solid middle section. The maximum compressive strains are located on the exterior face of the panel opposite both the upper and lower intersection of the ribs with the solid middle section. The maximum tension strain on the exterior surface of the panel must also be considered in order to examine possible cracking. The strains significantly increase from those seen in the original experimental program. The panel strains are shown in Figure 4.16.

**Table 4-15 – Critical Strains: No Middle Support**

<table>
<thead>
<tr>
<th>Strain Data</th>
<th>Load (kips)</th>
<th>Maximum Tension (Ribs)</th>
<th>Maximum Compression (Ribs)</th>
<th>Maximum Tension (Ext. Face)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Middle Support</td>
<td>6.08 (1.6W)</td>
<td>0.0002457</td>
<td>-0.0005690</td>
<td>0.0002457</td>
</tr>
<tr>
<td>-6.08 (-1.6W)</td>
<td>0.0004746</td>
<td>-0.0002436</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>
A few conclusions can be drawn from the model that eliminated the middle support. The first is that this panel will result in significantly larger deflections and strains. The deflections are much larger than those seen in prior results. However, due to the new deflection limit state, the maximum predicted deflections still remain within the limit. The strains are also much larger. In fact, the model predicts that cracking will occur before the design limit state on the exterior face of the panel. It would be recommended to do an investigation of the cracking on the panel. This investigation would determine whether or
not the cracking would be allowed by the architect. Therefore it is unclear whether or not this panel configuration would be acceptable.

4.7 Summary and Conclusions

An analytical model was created in order to predict the performance of precast concrete ribbed wall panels. Initially a frame analysis was conducted, but due to the complexity of the panel section it was necessary to create a finite element model to analyze local behaviors. Therefore the frame analysis was used to verify the results generated by the finite element model. The deflections and strains from the experimental program were then used to calibrate the model. Once the model was calibrated it could be modified in order to investigate the behavior of different panel configurations. By doing this, the finite element model looked to optimize panel design creating a more efficient system.

The finite element model developed to analyze this panel was deemed to be acceptable. This is justified by the results from the frame analysis and due to the calibration from the results of the experimental program. There are some discrepancies between the model and experimental results but these can be attributed to several reasons. The first is the accuracy of the instruments used in the experiment. The next is the fact that the fiber reinforced concrete layer was not created in the model. Lastly, the instruments and finite element model generate results that are a mix of localized and average results which will inevitably create discrepancies.
The parametric study yielded results in order to optimize the panel configuration. The first study focused on a more realistic wind loading condition using a uniform pressure load over the entire exterior surface of the panel. This model was intended to predict panel behavior in the field. The panel performed in an acceptable manner with respect to critical deflections and strain values. These results reinforce the quality of design of the original panel. The reduction in interior ribs gave different results. When the panel was configured with two interior ribs, the strains and deflections increased, but were still within the allowable limits. When the panel was configured with one interior rib, the strains and deflections increased as well, but in this scenario the deflections were too large. These deflections are located within the center of the exterior skin of the panels. Thus the localized stiffness values will have a significant control over the design. The conclusion can be made that reducing the interior ribs from three to two will be acceptable. The reduction in exterior rib width from 6 inches to 2.5 inches also gave deflection and strain results that were within their allowable limits. Therefore the conclusion can also be made that reducing the exterior rib width from 6 inches to 2.5 inches will be acceptable. Finally, the elimination of the middle support gave mixed results. The deflections were all within the allowable limits but the strains were not. The conclusion to be made is that this panel is not acceptable due to the initiation of cracks on the exterior surface. However, the recommendation would be to experimentally investigate these cracks and determine whether or not they are acceptable or not.

In conclusion, the panel that was tested experimentally was deemed to be acceptable with respect to strength and serviceability. This was then confirmed by the use of an
analytical model. Finally, the model predicts that the panel can be optimized to create a more efficient system by reducing the interior ribs from 3 to 2 or by reducing the exterior rib width from 6 inches to 2.5 inches. It may also be possible to use the panel in a two story clear span application depending upon cracking limitations set by the architect.
5 OBSERVATIONS, CONCLUSIONS, RECOMMENDATIONS

5.1 Summary of Research

As the demand for increased efficiency in structural wall systems has increased, new reinforced concrete panels have been developed. Two story precast concrete ribbed wall panels are one applicable design. This panel provides savings in material and design costs with fast and easy construction while still providing strong and durable structural systems. Research was conducted at North Carolina State University in order to evaluate the behavior of precast panels and investigate more efficient configurations. The main objectives of this research were to experimentally evaluate panel behavior and then develop an analytical model to predict structural performance. The experimental phase tested two full scale panels under laboratory conditions. The analytical phase formulated a model that was used to optimize overall panel design. General observations, conclusions, and recommendations for future research are presented in the subsequent sections of this chapter.

5.2 Experimental Observations and Conclusions

Testing of two full scale panels yielded significant results with regard to failure mode and panel behavior. Observations and conclusions from the testing of the two panels are as follows:

1. Both experimental panels behaved adequately under service and design load
conditions. Minimal cracking was observed along the edges of both panels and all deflections were small. Gravity loads were in place during all lateral load cycles.

2. Panel failure occurs at approximately 2.4 times the service load. The failure is due to pullout of the mid-height connection.

3. The mid-height connection was strengthened and testing continued up to a load of approximately 4.0 times the service load. Both panels developed many large surface cracks but did not show any indication of failure.

4. Panel RWP2 was tested without the gravity loads up to a load level of approximately 4.0 times the service load.

5. The panels behaved in a satisfactory manner at service and design load levels and were limited by the strength of the lateral connection. Both panels overall behaved in a stiff manner with small measured deflections and strains.

5.3 Analytical Observations and Conclusions

Model analysis of the original and reconfigured concrete panels yielded significant results with regard to panel behavior. Observations and conclusions from the analysis of the
panels are as follows:

1. A finite element model was developed to predict panel behavior and was verified using frame analysis techniques.

2. The finite element model was calibrated using measured strain and deflection results from the experiment.

3. The finite element model was deemed to be an effective tool for predicting panel behavior and confirmed the results measured in the experiment.

4. The model was reconfigured in order to optimize panel design and to create a more efficient structural system. These new configurations included a reduction in the number of interior ribs, a reduction in the width of the exterior ribs, and also the utilization of the panel in a clear two story height.

5. The model predicts that a panel with only two interior ribs as well as a panel with a reduced exterior rib width will behave adequately under service and design load levels.

6. The model predicts that a panel with only one interior rib will not perform adequately under service and design load levels.
7. The model predicts the formation of exterior surface cracks on a panel configured for a clear two story height.

5.4 Recommendations

Several recommendations can be made after assessing the observations and conclusions of this research project in order to create a more efficient precast panel. Future research may include the following:

1. Experimental verification of proposed reconfigured panel designs including a panel with only two interior ribs and a panel with reduced exterior rib width.

2. Further examination of the two story clear height panel configuration with emphasis on the development of exterior surface cracking under service and design load levels.
6 REFERENCES


APPENDIX
Figure A.0.1 – Production Drawing