Tilt-up Concrete Panels: an Investigation of Flexural Stresses and Punching Shear during Lifting

by

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B.S. Mechanical Engineering
University of Missouri – St. Louis, 2009

SUBMITTED TO THE DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF ENGINEERING IN CIVIL AND ENVIRONMENTAL ENGINEERING AT THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY

JUNE 2011

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Submitted to the Department of Civil and Environmental Engineering on May 13, 2011 in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in Civil and Environmental Engineering

ABSTRACT

Tilt-up construction is becoming more popular in the United States due to its ease of construction, reliability, and relatively low construction and maintenance costs. In its most typical form, a concrete panel is cast on the ground. After the concrete sets and has reached a prescribed compressive or flexural strength, a crane lifts the panel off the ground and hoists it into place. The flexural stresses during liftoff are often times greater than those corresponding to service loads. Concentrations of high shear stress and the associated punching shear in the vicinity of the pick points could result in pullout. For these reasons, it is particularly important to design the concrete and steel reinforcement to handle the flexural and shear stresses associated with panel erection.

This thesis investigates the flexural stresses and punching shear of a concrete panel designed for tilt-up. Finite element models confirm static hand calculations, and experimental results indicate that these models appropriately predicted erection stresses. The acceptability of the design provided is confirmed.

Thesis Supervisor: Jerome J. Connor
Title: Professor of Civil and Environmental Engineering
Acknowledgements

There are many thanks to go around for my success on this thesis and in the MEng Program. Above all, I thank God. Through Him, all things are possible.

I am forever indebted to my parents, Sam and Mary Lou for their constant advice, support, and encouragement. From a very early age they instilled in me the importance of education, but also the importance of doing what makes me happy. These things together made for an excellent experience at MIT. It would have been a much tougher year without the monthly care packages of cookies, cake, chocolate candy, cereal, and pop tarts. These are the times I was most popular with my peers.

My sister, Christine who began her graduate school education at the same time I did; for allowing me to provide structural engineering consulting on some of her architecture projects, which helped reinforce my understanding of many concepts.

Trish Detwiler, for her undying faith in me and constant encouragement; for the early morning wakeup calls and the late night Skype & study sessions.

My roommates, Jeroen (Julian) Houbrechts and Matthew Fontana. To Julian, for the inspiration to never give up, even on the many Saturday nights we spent in the MEng room; for our weekly excursions to Boston and the conversations about anything but school; for the assistance on PSets; for the ability to make me laugh even when we are delirious from studying; and for proofreading this thesis and providing valuable feedback. To Matthew, for always greeting me with a smile and a warm “welcome!” and the small but meaningful gifts around the holidays.

Shoshanna LaCarte, for the Saturday morning walks, which filled me with happiness; for humoring me with stories from the anthology of the great K.J. Bathe; for late night meals of frozen dinners and orange juice; for the great memories of the time spent along the train tracks; and for her endless loyalty and affection.

My alma mater, the University of Missouri – St. Louis, for providing an excellent foundation for my success in the MEng program; for the relationships I was able to build, which have had a profound impact on my life and which I will remember forever.

Jonathan Sigman (B.S. in CE, 1995, S.M. in CEE, 1997), my professor and mentor from UM – St. Louis, who was my inspiration for applying to MIT. Through his guidance over the past 5 years, I realized that Structural Engineering is what I was meant to do and that MIT is where I belong for my studies.

Joey Gruber, FOCUS Minister at MIT, for his infinite wisdom, which I called on many times throughout the year. Joey is partly responsible for my strengthened faith and closer-than-ever relationship with God.

Professor Jerome Connor, for his continuous support and personal investment in my success.

Lisa O’Donnell, for her advice and direction of our MEng Project this year.
Simon LaFlamme, for his tireless assistance as our TA and Mama Duck. The many hours spent in his office for PSet help or in recitation learning SAP, Simulink, and the Galerkin Method were offset by the laughter and many fun times not only at MIT, but during our study trip in Paris.

Lauren McLean, for taking care of us and keeping us sane.

Last but not least I owe many thanks to my fellow MEngers. It was an honor and a privilege to work with each one of them. I have learned so much from them during the last 9 months and even though we will be scattered across the country and indeed the globe after we graduate, I hope to remain in touch with them for many years. Particular mentioned goes to those in my MEng Project group: Bridget Navarro, Zachary Boswell, and Robert (Marne) Zahner. Miryam’s pretty cool too.
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1. Introduction

Tilt-up construction has been around for decades. It is fast and simple, reliable, and inexpensive. This is why it continues to be the construction method of choice for many buildings today. It has been used for many types of structures, from office buildings to Olympic sports venues to nuclear housing facilities [ACI 7, 1986]. Its versatility is one of its most attractive attributes. In a society that values sustainable structures, and a culture that tends toward the use of materials that have minimal environment impact, tilt-up construction is one of the most “green” methods in use today. The continued emphasis placed on LEED Certified construction brings all of the benefits of tilt-up to the attention of a design team [Kramer, 2006]. It is one of the most environmentally friendly construction practices these days for a number of reasons: panels are cast on site, so transportation costs remain low; approximately 25% of panels are insulated, which means the buildings they enclose are affected less by heat loss or infiltration; they are designed for disassembly, so they can be easily moved, removed and reused, or replaced and recycled if necessary [Portland Cement Association, 2011]. Tilt-up panels, as their name suggest are tilted up into place with the use of a crane that is connected to anchored inserts embedded within the panel (Figure 1.1) [ACI 7, 1986]. With a team of creative engineers and an informed and experienced construction crew, a design and construction sequence can be devised that makes the most efficient use of materials, space, and crane operating time. This thesis looks at the history of tilt-up concrete construction - when it began and how it is used today. It will also discuss its advantages and some of its limitations. It is a very attractive construction method, but like anything it has its limitations and is not always the ideal choice. The objectives of this thesis are to help the reader understand the versatility of tilt-up construction, introduce a design example, proceed with a finite element model that analyzes flexural stresses during erection, and then discuss punching shear as an important design consideration.

1.1 Problem

There is much more to engineering than just selecting the lightest members or the least expensive materials to create the most “economical” design. Much more comes into consideration when a structural engineer is devising his or her scheme for design and the corresponding structural
system. One has to think about availability of materials, environmental effects, and familiarity with construction methods among other things [Ricker, 2007]. The latter has a significant effect on the time and cost associated with construction. Nowadays more than ever, the increase in the cost of a building is attributed more to the cost of labor than the cost of materials. A century ago, buildings and bridges were adorned with magnificent artistry and beautiful architectural expression because labor was much cheaper than materials. Over the course of the last century those relative costs have inverted, meaning that it is becoming increasingly important to make construction quick and easy. This means, however that structures can be boring, lacking any sort of architectural appeal. But, it does not mean that they must lack any semblance of personality or character. One of the many challenges for a structural engineer then is that of optimizing constructability with aesthetic beauty. There is one system that is able to accomplish this better than any other: tilt-up concrete. This method is fast, simple, reliable, and inexpensive for low rise buildings, and its popularity is increasing.

**Figure 1.1:** concrete tilt-up panel lifted with a crane; figure courtesy of Google images
1.2 Objective

It is important to explore the options that are available when considering the construction of a building. The objective of this thesis is to investigate one of those options. Tilt-up concrete panels are increasing in popularity because of their inherent benefits when it comes to construction of low-rise (less than 5 stories) buildings. In this manner, more and more engineers will have to become familiar with their design methodology, designing not only for service loads, but also for construction loads. There are high localized stresses at the points from which the panel is lifted. Pullout forces are great and additional reinforcement is required. In their immediate vicinity and in between these points, there are also high flexural stresses during erection. This thesis will consider a design example and then investigate these flexural stresses and pullout forces. Basic static hand calculations will be compared to those found by finite element software including SAP2000 and ADINA.
2. Tilt-up Concrete

2.1 History and Uses of Tilt-up Concrete

Tilt-up construction involves the use of precast concrete slabs that are lifted into place with a crane. These panels are cast on the ground on the building floor or a temporary casting slab usually made of concrete (sometimes steel). Tilt-up construction provides a quick, reliable, and economical means for building a low-rise (less than 5 stories) structure. The idea of tilt-up has been around for over a century. In 1893 at Camp Logan, Illinois, Robert Hunter Aiken, recognized by many as the father of tilt-up construction, used a specially designed tilting table to cast and erect walls for homes. Then, in 1911 he and his Aiken Reinforced Concrete Company used the tilt-up method to construct a 111-ft x 644-ft building for the Los Angeles Railway Company; this was the largest building of its era constructed by tilt-up methods. The method fell out of favor by the mid 19-teens because the concrete technology of the day did not allow tilt-up to be very useful. It was not until the advent of ready mix concrete and mobile cranes, a development coming directly out of World War II, that tilt-up construction began to flourish [Dayton Superior, 2009]. With the advances in concrete and the introduction of specially designed inserts and lifting systems, its popularity has grown and is a very attractive construction method for low-rise buildings.

2.2 Advantages and Limitations

The main advantages of tilt-up concrete panels are their ease of erection, reliability, low cost, and versatility when it comes to architectural expression. Since panels are cast on the ground, the amount and complexity of formwork is significantly reduced. The main limitation of tilt-up construction is the height. Typically, tilt-up buildings are limited to about 3-4 stories. There are cases where it has reached as high as 5 or 6 stories, but these instances are rare. While tilt-up is a great option for low rise buildings, offices, and warehouses for example, it is not possible for skyscrapers or tall buildings. This is due to the lack of free space horizontally on the ground. A two story building might be about 25 feet tall; it is perfectly reasonable for a construction site to have 25 feet of free ground space. However, a ten story building might be 120 feet tall. It is unrealistic to expect that such large spaces are free and will remain unobstructed during
construction. In addition, the rigging would become quite complex for a ten story panel, and the erection would no longer be efficient. An apparent solution is segmented construction, but this would be difficult and cumbersome for a tall building.

2.3 Rigging and Lifting

Rigging is the means by which a panel is attached to cables or harnesses to be lifted by a crane. For larger panels the rigging scheme can become quite complex, but there are accepted standards for placement of pick points to reduce stresses, and the use of pulleys for mechanical advantage. For smaller panels the rigging scheme is fairly simple and the risk of injury is lower. The number and location of pick points during lifting is important to ensure that the stresses in the concrete do not exceed the strength it was allowed to develop\(^1\). The rigging also serves to balance the panel. Figure 5.1 shows a few schematics of rigging setups. Wall braces that use telescope pipes (shown in Figure 2.1), or other temporary reinforcement such as strongbacks may also be used. Panels experience their largest flexural stresses during lifting; service loads are usually much lower. Cracks are much more likely to occur during this stage than during its service life. It is important then to determine the stresses the panel experiences during erection, and to consider the strength of the panel before lifting takes place [ACI 7, 1986].

![Wall braces supporting tilt-up panels before they are anchored to the ground or adjacent panels; figure courtesy of Google images](image)

\(^1\) Panels are not allowed to cure for 28 days to reach their full compressive strength. They are usually lifted 10 - 14 days after pour.
2.4 Strength and Stresses of Panels

Concrete walls can always be cast in their vertical position, but expensive formwork is usually required. One of the main advantages of tilt-up concrete panels is that such compulsory formwork can be avoided when the panel is cast on the ground. The tricky part, however, is determining when the panel is ready for erection. It must cure long enough to attain adequate strength; the most important strengths are shear and flexural. Since the panel is picked up by a crane at discrete points, it experiences high concentrations of shear stress at these points, and high flexural stress in between them. Shear stress can lead to pullout, a failure by punching shear. The engineer must ensure that the concrete has sufficient shear strength to resist impending pullout forces. If the strength of the concrete itself is not enough, steel reinforcement is required to resist the remainder of the shear. The rebar would resist the shear by dowel action [Wang et al., 2001]. Flexure creates tensile stress; concrete does not perform well under tension, so it is crucial to locate the inserts such that moments are minimized. In general contractors perform compression and/or flexural bending tests on specimens on site to determine the strength of the panel before lifting. Technologies do exist to determine strength in a nondestructive manner. For example, the “maturity method” uses electronic devices embedded in the concrete that use a combination of temperature and age to determine strength according to empirical formulas [Abi-Nader, 2009]. Each engineer will have his own prescription for the required strength before lifting, but one source suggests a minimum flexural strength of 450 psi [Audi, 2011]. The engineer can also prescribe a minimum compressive strength before lifting (e.g. 2.5 ksi), but flexural strength is more critical. This thesis investigates flexural stresses and punching shear during erection of a tilt-up panel.
3. Design Example

The design example that follows was inspired by the Ph.D. thesis work of Guy Georges Abi-Nader. The concrete panel that will be analyzed is 10 ft wide by 9 ft tall, with two pick points 7 ft from the bottom ("one wide by two high" rigging system), and one large void that is 4 ft by 4 ft, located 1 ft from the bottom and 2 ft from the right. See the illustration below in Figure 3.1 for a schematic of the geometry. The concrete is assumed to have a compressive strength of 4000 psi, a mass density of 150 pcf, and is 3.5 in thick. The following design example will repeat Abi-Nader’s work using basic static hand calculations and verify the results with the structural analysis and finite element software programs SAP2000 and ADINA. In order to make direct comparison to other results, including both simulation and experimental results, this analysis will use the same geometry and material properties.

Abi-Nader’s work, out of the University of Florida in 2009, looked at the flexural stresses of tilt-up panels during erection. The focus of his research was the use of a “maturity method” that measures the instantaneous strength of concrete. It does this with an embedded electronic device that translates time and temperature of curing into a maturity index. Depending on the type of concrete mix, this value is used in an empirical formula that can be related to compressive and flexural strengths. In this manner it would not be necessary for contractors to perform compression or bending tests on cylinders or beams before erection. In order to test the adequacy of the maturity method he constructed a concrete panel and used a crane to lift it into place. To measure flexural stresses he used strain gages oriented along both the X-X and Y-Y directions; he placed these where initial static hand calculations indicated maximum bending stresses would occur. He then compared the experimental results not only with his hand calculations, but also with stress values given by four other design firms who used in-house software specifically designed for tilt-up panels. Abi-Nader’s hand calculations indicated that the maximum stresses would be incurred when the panel was initially lifted off the ground; i.e. when the angle relative to the horizontal was zero degrees. For this reason this section will investigate the flexural stresses only during initial liftoff.
This panel (including steel rebar) was design by Steinbicker & Associates, Inc. They prescribed:

- 3.5 in thickness
- \( f_c' = 4000 \) psi
- compressive lift strength = 2500 psi
- #4 reinforcing bars; layout is provided in Figure 4.3
- coordinate system as shown

The analysis procedure will be consistent with Abi-Nader’s method. The rigging system will be “one wide by two high” such that it is acceptable to model the panel as a simply supported beam in both directions.
Figure 3.2: simply supported beam with loading, shear, and moment diagrams

Table 3-23 of the AISC Steel Construction Manual shows that the maximum moment for a simply supported beam occurs at the midspan and is equal to:

$$M = \frac{wL^2}{8}$$  \hspace{1cm} \text{Eq 3-1}

where:

$M =$ moment (lb-ft)

$w =$ distributed line load (plf)

$L =$ length of beam (ft)

The flexural stress, $S_b$, can be found from the moment at any given point according to:
\[ S_b = \frac{M}{S} \quad \text{Eq 3-2} \]

where:

- \( S_b \) = flexural stress (psi)
- \( M \) = moment (lb-in)
- \( S \) = section modulus (in\(^3\))

This assumes an uncracked, unreinforced section [Bartels, 2010]. The section modulus, \( S \) is a geometric property of the cross section and is given by:

\[ S = \frac{b d^2}{6} \quad \text{Eq 3-3} \]

where:

- \( b \) = width of section (in)
- \( d \) = depth of section; thickness of panel (in)

Since this panel has obstructions, it is not completely continuous in all direction, so it must be broken up into sections, with equivalent distributed loading following from that. Figure 3.3 below shows how the panel’s parts are divided up for analysis in the Y-Y direction.

- \( W_1 \) = weight of section 1
- \( w_1 \) = distributed load along Y-Y direction of section 1
- \( W_2 \) = weight of section 2
- \( w_2 \) = distributed load along Y-Y direction of section 2
- \( W_3 \) = weight of section 3
- \( w_3 \) = distributed load along Y-Y direction of section 3

These weights and loads are solely the self weight of the concrete panel. It does not include any suction or dynamic loads.

\[ W_1 = (10 \text{ ft} \times 1 \text{ ft})(150 \text{ pcf})\left(\frac{3.5 \text{ in}}{12 \text{ in/ft}}\right) = 437.5 \text{ lb} \]
\[ w_1 = \frac{437.5 \text{ lb}}{1 \text{ ft wide}} = 437.5 \text{ plf} \]

\[ W_2 = (4 \text{ ft} + 2 \text{ ft})(4 \text{ ft})(150 \text{ pcf}) \left( \frac{3.5 \text{ in}}{12 \text{ in/ft}} \right) = 1050 \text{ lb} \]

\[ w_2 = \frac{1050 \text{ lb}}{4 \text{ ft wide}} = 262.5 \text{ plf} \]

\[ W_3 = (10 \text{ ft} \times 4 \text{ ft})(150 \text{ pcf}) \left( \frac{3.5 \text{ in}}{12 \text{ in/ft}} \right) = 1750 \text{ lb} \]

\[ w_3 = \frac{1750 \text{ lb}}{4 \text{ ft wide}} = 437.5 \text{ plf} \]

**Figure 3.3:** panel section divisions for determining stresses in the Y-Y direction; figure courtesy of Abi-Nader
These linear line loads can be taken and modeled in a structural analysis program to achieve shear and moment diagrams. It is simple enough to do these calculations by hand, but for sake of clarity of results they were performed in RISA 2D.

Figure 3.4 shows that the maximum moment is 1403.6 lb-ft. This coincides with a location of zero shear. The left support represents the floor (reaction = 1006.2 lb) and the right support represents the pick points for the crane (reaction = 1356.3 + 875 = 2231.3 lb; since there are two points, each one takes half the load, 1115.6 lb each). The positive moment (1403.6 lb-ft) in the middle indicates that the top surface of the panel is in compression; the negative moment (-875 lb-ft) occurs at the right support and indicates that the top surface is in tension. Figure 3.5 shows the moment diagram superimposed over the panel.

![Figure 3.4: loading (plf), shear (lb), and moment (lb-ft) diagrams for the equivalent beam system (Y-Y direction) in RISA 2D](image)

Now that the moments are available for this panel, we can develop preliminary values for flexural stress at points of interest. We are interested in the stresses at 3.17 ft from the bottom and 7 ft from the bottom since these are the locations of highest and lowest moment, respectively. At 3.17 ft from the bottom the effective width of that section is only 6 ft because of
the void at that height. The width at 7 ft from the bottom is taken as 10 ft. Using Eq 3-3 for the section 3.17 ft from the bottom,

\[ S = (10 \text{ ft} - 4 \text{ ft}) \left(12 \frac{\text{in}}{\text{ft}}\right) \left(\frac{3.5 \text{ in}^2}{6}\right) = 147 \text{ in}^3 \]

The flexural stress can then follow according to Eq 3-2,

\[ S_b = \frac{(1403.6 \text{ lb} \cdot \text{ft}) \left(12 \frac{\text{in}}{\text{ft}}\right)}{147 \text{ in}^3} = 114.58 \text{ psi} \]

For the section that is 7 ft from the bottom,

\[ S = (10 \text{ ft}) \left(12 \frac{\text{in}}{\text{ft}}\right) \left(\frac{3.5 \text{ in}^2}{6}\right) = 245 \text{ in}^3 \]

The flexural stress then follows as,

\[ S_b = \frac{(-875 \text{ lb} \cdot \text{ft}) \left(12 \frac{\text{in}}{\text{ft}}\right)}{245 \text{ in}^3} = -42.86 \text{ psi} \]

It important to note that the panel experiences a maximum flexural stress of 114.58 psi in compression and 42.86 psi in tension on the top surface of the panel in the Y-Y direction.

The panel also needs to be analyzed in the X-X direction. The same procedure as above will be followed, this time with each pick point serving as a support. In calculating the distributed line load, only the area above 3.17 ft is considered since this was the location of zero shear from the previous analysis. Figure 3.6 displays how the sections are divided.

The same notation will be used here as above such that

- \( W_1 \) = weight of section 1
- \( w_1 \) = distributed load along X-X direction of section 1
- \( W_2 \) = weight of section 2
- \( w_2 \) = distributed load along X-X direction of section 2
- \( W_3 \) = weight of section 3
- \( w_3 \) = distributed load along X-X direction of section 3
These weights and loads are solely the self weight of the concrete panel. Additional loads associated with suction and dynamic effects are not accounted for here.

![Diagram](image)

**Figure 3.5**: moment diagram in Y-Y direction; figure courtesy of Abi-Nader

\[
W_1 = (9 \, ft - 3.17 \, ft)(4 \, ft)(150 \, pcf) \left( \frac{3.5 \, in}{12 \, ft} \right) = 1020.25 \, lb
\]

\[
w_1 = \frac{1020.25 \, lb}{4 \, ft \, \text{wide}} = 255.2 \, \text{plf}
\]

\[
W_2 = (4 \, ft)(4 \, ft)(150 \, pcf) \left( \frac{3.5 \, in}{12 \, ft} \right) = 700 \, lb
\]
\[ w_2 = \frac{700 \text{ lb}}{4 \text{ ft wide}} = 175 \text{ plf} \]

\[ W_3 = (9 \text{ ft} - 3.17 \text{ ft})(2 \text{ ft})(150 \text{pcf}) \left( \frac{3.5 \text{ in}}{12 \text{ in/ft}} \right) = 510.4 \text{ lb} \]

\[ w_3 = \frac{510.4 \text{ lb}}{2 \text{ ft wide}} = 255.2 \text{ plf} \]

**Figure 3.6:** panel section divisions for determining stresses in the X-X direction; figure courtesy of Abi-Nader

From the analysis shown for the shear in the Y-Y direction, it is necessary to impose reactions of 1115.6 lb at each support to avoid inconsistent reactions and rotation of the panel. These reactions represent the tension in the lifting cables. With the presence of these imposed reactions there is also a moment shift required since the existing load distribution will not result in zero
moment at the right end. Figure 3.7 indicates that a moment shift of 192.4 lb-ft is required at the right end. 192.4 lb-ft will be added to the magnitude of the moment at the right insert. The moments in the center and at the left insert will remain as shown. Figure 3.8 shows the final moment diagram including the adjusted moment at the right insert.

Figure 3.7: loading (plf), shear (lb), and moment (lb-ft) diagrams for the equivalent beam system (X-X direction); components courtesy of Abi-Nader
Now that the moments are available for this panel, we can develop preliminary values for flexural stress at points of interest. We are interested in the stresses at the right most insert (not the left insert because the flexural stress at the right insert is greater than that at the left) and the location 4.54 ft from the left (as indicated on the moment diagram where the greatest moment occurs). Using Eq 3-3 for the right insert and the location 4.54 ft from the left,

\[ S = (4 \text{ ft})(12 \text{ in/ft}) \left( \frac{3.5 \text{ in}^2}{6} \right) = 98 \text{ in}^3 \]

The flexural stress for the right insert is found according to Eq 3-2,

\[ S_b = \frac{(-625.7 \text{ lb ft})(12 \text{ in/ft})}{98 \text{ in}^3} = -76.62 \text{ psi} \]

The flexural stress at 4.54 ft from the left,
\[ S_b = \frac{(484.96 \text{ lb ft})(12 \text{ in})}{98 \text{ in}^3} = 59.38 \text{ psi} \]

It is important to note that the panel experiences a maximum flexural stress of 59.38 psi in compression and 76.62 psi in tension on the top surface of the panel in the X-X direction.

### 3.1 Panel Design

The design of the panel reinforcement is not discussed here since it is beyond the scope of this thesis. However, a schematic is shown with rebar details in Figure 3.9. This panel was designed for Abi-Nader by Steinbicker & Associates, Inc. It is not shown here, but there is additional rebar provided around the inserts with RL-24 plate anchors to prevent pullout during erection. The projected area of this insert (\(\approx 16 \text{ in}^2\)) is accounted for in the ADINA simulation to follow.

![Figure 3.9: schematic design of rebar; figure courtesy of Abi-Nader](image_url)
3.2 Finite Element Models

Now that the static calculations have been completed, we have an idea of the kinds of stresses the panel will see during erection. As noted above, only the stresses incurred at zero degrees of inclination are considered because the flexural stresses are greatest here; they will only decrease during erection because the projected length of the panel decreases. The hand calculations above idealized the panel into simply supported beams. It is common practice in engineering to convert a realistic structure into something more basic by idealizing it [Schwabauer, 2010]. For most parts of the panel this should yield sufficiently accurate results. That said, it is important to take a closer look to see more precisely the stresses the panel experiences during liftoff. Areas of interest are in the neighborhood of the inserts, the region between the two inserts, and the region between the inserts and the line around which the panel rotates during erection. Stresses determined by three design firms that show the expected results according to software specifically tailored to tilt-up panel design are provided. Results according to SAP2000 and ADINA are also provided. All of the results, including the static calculations above are compared with experimental results.

3.2.1 SAP2000

SAP2000 is a structural analysis software program that can also be used for finite element analyses. It is used here to model the tilt-up panel with the dimensions shown in Figure 3.1. The panel is modeled as a 2-D planar element. There is no direct load applied to the element, but the geometric properties are given such that it experiences a distributed pressure of 43.75 psf due to self weight. The material properties are input such that it behaves like unreinforced concrete. In the model, the low side of the panel is constrained such that it allos rotation about the x-axis, but restricts rotation about all other axes as well as translations in all directions. In order for the system to be considered stable by the program – and for the most appropriate representation of the physical situation – the pick points are modeled as pin supports. This restricts motion in and out of the plane (z-direction). All other translations and rotations are allowed. After the properties are input, analysis is run with the self weight of the concrete applied as the load. The following figures show the mesh (starting with coarse and becoming finer with each model) along with moments in the X-X and Y-Y directions. It is important to create finer meshes
around points of high stress, which include the pick points and the areas adjacent to the void. Each mesh becomes more computationally expensive, but converges to the analytical result. Once the moments converge, the mesh is fine enough and the results accurately represent the actual stresses. After the figures are presented, Table 3.1 summarizes the bending moments at the locations of interest as shown in Figure 3.10. Strain gages 1-5 show stresses in the Y-Y direction and 6-8 in the X-X direction (note their orientation). Gages 1-3 are located close to 3.17 ft from the bottom (recall the location of zero shear and max moment in the Y-Y direction; see Figure 3.5). Gages 4 and 6, and 5 and 8 are located near the left and right inserts, respectively. These are locations of interest because they are locations of high moment in both directions. That said it is difficult to give an exact account of the levels of these stresses. The locations of the strain gages in reality are also limited in precision. Values shown in Table 3.1 for gages 4, 5, 6, and 8 are values of nearby points. The points taken into account are the same each time. Gage 7 is located where the moment is maximum in the X-X direction; see Figure 3.8. Then, Table 3.3 shows the flexural stresses; they are not directly proportional because section moduli vary according to direction and location. Figure 3.11a shows a coarse mesh of the panel. Each square is 1 ft x 1 ft with the exception of the mesh immediately surrounding the pick points. The mesh becomes skew near the top. The blue lines are bumped to the left so they can run through the pick points and include the pin supports. This is necessary because the program will otherwise assume that the pin supports are not a part of the element. Figure 3.11b shows the reactions at the inserts. It is difficult to tell from the screen shot but the left insert provides a reaction force of 1097 lb and the right a reaction force of 1135 lb. We are less concerned with the reactions at the base since they are not areas of high stress; however, they are included here for completeness. Figures 3.11c and 3.11d show the moment diagrams for the X-X and Y-Y directions, respectively. Values of interest are presented in Table 3.1. Three meshes with reactions and moment diagrams from SAP are presented in total. The format is the same, so explanations of each will not be provided unless additional comment is necessary. Meshes of varying fineness are presented for a study of convergence.
Figure 3.10: strain gage locations; figure courtesy of Abi-Nader
Figure 3.11a: coarse mesh of panel in SAP

Figure 3.11b: reactions at inserts and along the base of coarse mesh: left insert = 1097 lb, right insert = 1135 lb
Figure 3.11c: moments in X-X direction of coarse mesh

Figure 3.11d: moments in Y-Y direction of coarse mesh
Figures 3.12a – 3.12d correspond to a semi-fine mesh. This mesh includes approximately four times as many elements as the previous mesh. In order to include the pick points in the mesh, all the rectangles in this mesh are not the same size (notice the skew elements near the top right of the void). It is not strictly required that elements be the same size, however stress jumps occur with poor meshing, so using square elements is best\(^2\). However, they are close enough that accurate results can be achieved.

\[\text{Figure 3.12a: semi-fine mesh of panel in SAP} \]

\(^2\) Using triangular elements would allow one to refine the mesh in the neighborhood of stress concentrations; however, SAP does not do well with triangular elements. Stress jumps occur that are not indicative of the actual behavior. For this reason, only rectangular elements are used in all SAP models.
Figure 3.12b: reactions at inserts of semi-fine mesh: left insert = 1097 lb, right insert = 1134 lb

Figure 3.12c: moments in X-X direction of semi-fine mesh
**Figure 3.12d**: moments in Y-Y direction of semi-fine mesh

Figures 3.13a – 3.13d correspond to a fine mesh. The entire panel is meshed with square elements of equal size. Comparing this mesh and the previous, convergence is achieved.
Figure 3.13a: fine mesh of panel in SAP

Figure 3.13b: reactions at inserts of fine mesh: left insert = 1097 lb, right insert = 1134 lb
Figure 3.13c: moments in X-X direction of fine mesh

Figure 3.13d: moments in Y-Y direction of fine mesh
Figures 3.14a and 3.14b show the moment diagrams in the X-X and Y-Y directions, respectively for the same mesh as Figure 3.13a. The difference is that the figures below include the effects of suction and dynamics. According to Abi-Nader, dynamic effects can contribute an additional 20% of the weight, and suction up to an additional 50%. Combining these additional inertia effects with the self weight of the panel, the effective load on the panel is now 1.8 times the self weight. The corresponding moments and associated flexural stresses are summarized in Table 3.5. Calculations for flexural stresses are omitted here for brevity, but they can be found in the same way as shown earlier in this section.

\[
3 \text{ In general the dynamic and suction terms are additive, which would yield an additional 70% load. However, in order to be more conservative, this thesis assumes that the two are not mutually exclusive, such that one affects the other, which introduces an additional 80% load in the model.}
\]
3.2.2 ADINA

ADINA is another finite element program. While SAP2000 also functions as a structural analysis program, ADINA was designed to specifically as a finite element program. For this reason, ADINA is used to verify the results from SAP. In SAP we are not too concerned with stress singularities (e.g. at the pick points). In fact, it models them as having finite stresses, which is physically impossible), but ADINA requires that an area for these locations be prescribed. This model is thereby inherently more accurate because it more adequately incorporates the geometric variability, and is used to verify the results from SAP. ADINA is a little more well-suited for finite element analyses because the user is allowed to prescribe many more inputs including the interpolation function type, element type (4- or 9-node e.g.), and the element group among other things. The following pages walk through each step of the formulation of the ADINA model from the layout of the geometry to the final stress results. It is important to note that the coordinate system inherent to ADINA is different than that of SAP. In SAP’s default 2-D projection, left-right corresponds to the x-axis, and up-down corresponds to the y-axis. In ADINA, left-right corresponds to the y-axis, and up-down corresponds to the z-axis. Note the coordinate labels in many of the figures to follow. To avoid confusion, the tables at the

Figure 3.14b: moments in Y-Y direction of fine mesh including dynamic and suction effects
conclusion of this chapter give flexural stresses with respect to strain gage locations, which unambiguously indicate the direction of bending.

3.2.2.1 3-D Solid Simulation

The first step in creating the ADINA model is to input the data points, from which surfaces can be derived. The surfaces define the geometry of the panel (Figure 3.15a). Boundary conditions are included in this figure. Boundary condition $B$ corresponds to the pick points. It allows translation in all directions except $x$, which is transverse to the load; rotation in all directions is allowed. Boundary condition $C$ corresponds to the line around which the panel rotates during lifting. Translation and rotation in all directions are restricted with the exception of rotation around the y-axis. After the surface is prescribed, the volume in the 3-D model is formed. In this case, the surfaces are extruded a distance equal to the thickness of the panel (Figure 3.15b). The next step is to define the loading on the panel. Since self weight is the only load the panel experiences, an evenly distributed pressure load is prescribed here. The self weight of a 3.5 inch thick panel is 0.30382 psi as shown in Figure 3.15c. This load pattern is then applied to all the surfaces, which is illustrated in Figure 3.15d. Figure 3.15e shows how the pressure is applied. The behavior and response of the panel is dependent on its material properties. Figure 3.15f defines the Young’s Modulus (psi) as well as Poisson’s ratio (“density” is also shown in this window, but does not have an effect on the model). Since the Poisson’s ratio is less than 0.5 – which would indicate incompressible media – we use the default displacement-based interpolation functions (as opposed to the u-p formulation) [Bathe, 2006]. The accuracy and convergence of the results is dependent on the mesh density. A very fine mesh of cubes of approximately 1 in on each side is desired in this simulation. However, this is too computationally expensive, so a more coarse mesh is used, shown in Figure 3.15g. Figure 3.15h shows this density projected on the top surface. After the mesh density is defined, the panel itself needs to be meshed. The first step in doing this is defining the element group. Figure 3.15i shows that the panel is composed of 3-D solid 27-node elements. Figure 3.15j shows an isometric view of the final 3-D mesh of the panel. Figure 3.15k through Figure 3.15o show the stresses in the panel (flexural stresses are given directly instead of deriving them from moment diagrams). These figures do not take into account any dynamic or suction effects. As mentioned above, dynamic and suction effects acting in congress superimpose 80% more load. The model
for this simulation is the same, adjusting only the applied pressure, so the figures explaining the set-up are omitted for brevity. Figure 3.15p through Figure 3.15r show the stresses including these inertia effects. For clarity, zoomed-in sections with narrower stress bands are provided for all the 3-D results. These plots provide less precision than the SAP results, so stresses are rounded to the nearest multiple of 5. Results are shown in Table 3.6.

Figure 3.15a: surfaces that define the geometry of the panel; B and C define boundary conditions
**Figure 3.15b:** defining the volume of the panel

**Figure 3.15c:** defining pressure load
Figure 3.15d: applying pressure load to all surfaces

Figure 3.15e: pressure load applied to surfaces
Figure 3.15f: defining material properties of the concrete panel

Figure 3.15g: defining mesh density
Figure 3.15h: mesh density

Figure 3.15i: defining element group as 3-D solid calling the material defined in Figure 3.15f
Figure 3.15j: final 3-D mesh
Figure 3.15k: flexural stresses in Z-Z direction of 3-D solid in ADINA

Figure 3.15l: zoom in on Figure 3.15k to show stresses at gage locations 1, 2, and 3
Figure 3.15m: zoom in on Figure 3.15k to show stresses at gage locations 4 and 5

Figure 3.15n: flexural stresses in Y-Y direction of 3-D solid in ADINA
Figure 3.15o: zoom in on Figure 3.15n to show stresses at gage locations 6, 7, and 8

Figure 3.15p: zoom in on Z-Z stresses at gage locations 1, 2, and 3 including dynamic and suction effects
Figure 3.15q: zoom in on Z-Z stresses at gage locations 4 and 5 including dynamic and suction effects

Figure 3.15r: zoom in on Y-Y stresses at gage locations 6, 7, and 8 including dynamic and suction effects
3.2.2.2 2-D Shell Simulation

The mesh in the 3-D simulation provided above was coarse because of the limits of computing power. It was more coarse than desired; it is clear that a finer mesh would produce better results. There are noticeable stress jumps and in some cases the values of flexural stresses deviate a lot from other results. A 2-D shell element analysis is subsequently conducted to corroborate results. The set-up for the simulation is the same as for the 3-D case with the exception of defining the element group. Except for this step, shown in Figure 3.16a, the set-up figures are omitted for brevity. The figures corresponding to the SAP analysis began with the moment diagrams, and flexural stresses were derived directly from them. The figures from ADINA below show the stresses directly; stress values are reported in Tables 3.4 and 3.6. Close-ups are provided for clarity.

Figure 3.16a: defining 2-D shell element group in ADINA
Figure 3.16b: flexural stresses in Z-Z direction of 2-D shell

Figure 3.16c: zoom in on Figure 3.16b to show stresses at gage locations 1, 2, and 3
Figure 3.16d: zoom in on Figure 3.16b to show stresses at gage locations 4 and 5

Figure 3.16e: flexural stresses in Y-Y direction of 2-D shell
Figure 3.16f: zoom in on Figure 3.16e to show stresses at gage locations 6, 7, and 8

Figure 3.16g: zoom in on Z-Z stresses at gage locations 1, 2, and 3 including dynamic and suction effects
**Figure 3.16h:** zoom in on Z-Z stresses at gage locations 4 and 5 including dynamic and suction effects

**Figure 3.16i:** zoom in on Y-Y stresses at gage locations 6, 7, and 8 including dynamic and suction effects
3.3 Results and Discussion

The figures above give a graphical representation of flexural stresses during liftoff. Below is a series of tables that summarizes these results and provides points of comparison with other results, including static hand calculations (including self weight only, and additional inertia effects), SAP results, ADINA results, and results from three other programs used by design firms according to Abi-Nader’s research\(^4\).

The following tables make reference to the strain gages placed on the panel during liftoff. Refer to Figure 3.10 for the locations of these strain gages. Positive values of moment and flexural stress indicate that the top surface of the panel is in compression; negative values indicate that the top surface of the panel is in tension. Table 3.1 indicates that the coarse mesh does not show the presence of any tension zones in the top of the panel. This is clearly erroneous; tensile stresses must exist in order to accommodate the negative moment regions in the vicinity of the inserts. It indicates that the coarse mesh here is a “bad” mesh. That said, the results are included for a study of convergence.

Table 3.1: normalized moments (lb-ft/ft) for various mesh densities per SAP2000

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Coarse Mesh</th>
<th>Semi-Fine Mesh</th>
<th>Fine Mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>280</td>
<td>230</td>
<td>230</td>
</tr>
<tr>
<td>2</td>
<td>230</td>
<td>230</td>
<td>230</td>
</tr>
<tr>
<td>3</td>
<td>235</td>
<td>235</td>
<td>235</td>
</tr>
<tr>
<td>4</td>
<td>90</td>
<td>-105</td>
<td>-110</td>
</tr>
<tr>
<td>5</td>
<td>85</td>
<td>-105</td>
<td>-110</td>
</tr>
<tr>
<td>6</td>
<td>100</td>
<td>-120</td>
<td>-110</td>
</tr>
<tr>
<td>7</td>
<td>85</td>
<td>90</td>
<td>90</td>
</tr>
<tr>
<td>8</td>
<td>120</td>
<td>-160</td>
<td>-155</td>
</tr>
</tbody>
</table>

Table 3.2 is derived directly from Table 3.1. The values in the above table are taken directly from SAP and are given in units of [lb-ft/ft]. In order to make a direct comparison to the

\(^4\) Abi-Nader provided stresses from four design firms, but one set of results was so far removed from all the others, they were not deemed useful for this comparison.
moments derived by hand calculations, these moments need to be multiplied by their tributary width. The values corresponding to strain gages 1, 2, and 3 are multiplied by 6 (feet of effective width for these vertically oriented gages); values corresponding to strain gages 4 and 5 are multiplied by 10; values corresponding to strain gages 6, 7, and 8 are multiplied by 4.

Table 3.2: extrapolated moments (lb-ft) for various mesh densities per SAP2000

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Coarse Mesh</th>
<th>Semi-Fine Mesh</th>
<th>Fine Mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1680</td>
<td>1380</td>
<td>1380</td>
</tr>
<tr>
<td>2</td>
<td>1380</td>
<td>1380</td>
<td>1380</td>
</tr>
<tr>
<td>3</td>
<td>1410</td>
<td>1410</td>
<td>1410</td>
</tr>
<tr>
<td>4</td>
<td>900</td>
<td>-1050</td>
<td>-1100</td>
</tr>
<tr>
<td>5</td>
<td>850</td>
<td>-1050</td>
<td>-1100</td>
</tr>
<tr>
<td>6</td>
<td>400</td>
<td>-480</td>
<td>-440</td>
</tr>
<tr>
<td>7</td>
<td>340</td>
<td>360</td>
<td>360</td>
</tr>
<tr>
<td>8</td>
<td>480</td>
<td>-640</td>
<td>-620</td>
</tr>
</tbody>
</table>

Table 3.3 takes each value from Table 3.2 and converts it to flexural stress by following Eq. 3-2 and using its respective section modulus. This table shows that the coarse mesh is too coarse. Since there are no negative values the model indicates the there is no tension in the direct vicinity of the pick points. Math, intuition, and the moment diagram dictate that the top surface must be in tension near the inserts. The results from the coarse mesh could have been omitted, but for the sake of a convergence study they are included here. That said, it is noted that the results from the semi-fine and fine meshes indicate that convergence is reached. A finer mesh is not required to achieve accurate results.

Table 3.3: flexural stresses (psi) for various mesh densities per SAP2000

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Coarse Mesh</th>
<th>Semi-Fine Mesh</th>
<th>Fine Mesh</th>
<th>Section modulus (in^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>137.14</td>
<td>112.65</td>
<td>112.65</td>
<td>147</td>
</tr>
<tr>
<td>2</td>
<td>112.65</td>
<td>112.65</td>
<td>112.65</td>
<td>147</td>
</tr>
<tr>
<td>3</td>
<td>115.10</td>
<td>115.10</td>
<td>115.10</td>
<td>147</td>
</tr>
<tr>
<td>4</td>
<td>44.08</td>
<td>-51.43</td>
<td>-53.88</td>
<td>245</td>
</tr>
<tr>
<td>5</td>
<td>41.63</td>
<td>-51.43</td>
<td>-53.88</td>
<td>245</td>
</tr>
<tr>
<td>6</td>
<td>48.98</td>
<td>-58.78</td>
<td>-53.88</td>
<td>98</td>
</tr>
<tr>
<td>7</td>
<td>41.63</td>
<td>44.08</td>
<td>44.08</td>
<td>98</td>
</tr>
<tr>
<td>8</td>
<td>58.78</td>
<td>-78.37</td>
<td>-75.92</td>
<td>98</td>
</tr>
</tbody>
</table>
Table 3.4 compares the flexural stresses derived from hand calculations and those from SAP, ADINA, and 3 other software programs. These values do not include dynamic or suction effects.

**Table 3.4:** flexural stresses (psi) of all models and given data, not including dynamic or suction effects

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Static Calculations</th>
<th>SAP2000 fine mesh</th>
<th>ADINA 2-D Shell</th>
<th>ADINA 3-D Solid</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>114.58</td>
<td>112.65</td>
<td>85</td>
<td>85</td>
</tr>
<tr>
<td>2</td>
<td>114.58</td>
<td>112.65</td>
<td>85</td>
<td>85</td>
</tr>
<tr>
<td>3</td>
<td>114.58</td>
<td>115.10</td>
<td>85</td>
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</tr>
<tr>
<td>4</td>
<td>-42.86</td>
<td>-53.88</td>
<td>-60</td>
<td>-80</td>
</tr>
<tr>
<td>5</td>
<td>-42.86</td>
<td>-53.88</td>
<td>-60</td>
<td>-80</td>
</tr>
<tr>
<td>6</td>
<td>-49.58</td>
<td>-53.88</td>
<td>-50</td>
<td>-50</td>
</tr>
<tr>
<td>7</td>
<td>59.38</td>
<td>44.08</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>8</td>
<td>-76.62</td>
<td>-75.92</td>
<td>-70</td>
<td>-80</td>
</tr>
</tbody>
</table>

Software used by engineering firms [Abi-Nader, 2009]

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Software 1</th>
<th>Software 2</th>
<th>Software 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>115</td>
<td>112</td>
<td>111</td>
</tr>
<tr>
<td>2</td>
<td>115</td>
<td>112</td>
<td>111</td>
</tr>
<tr>
<td>3</td>
<td>115</td>
<td>112</td>
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<tr>
<td>5</td>
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<td>-45</td>
</tr>
<tr>
<td>6</td>
<td>-49</td>
<td>---</td>
<td>-49</td>
</tr>
<tr>
<td>7</td>
<td>53</td>
<td>---</td>
<td>46</td>
</tr>
<tr>
<td>8</td>
<td>-76</td>
<td>---</td>
<td>-71</td>
</tr>
</tbody>
</table>

Table 3.5 shows the moments of the fine mesh from SAP that include dynamic and suction effects. These moments are converted from lb-ft/ft to lb-ft and then to flexural stress (psi) in the same manner shown in Tables 3.2 and 3.3. Table 3.6 compares the experimental results with static calculations, SAP, and ADINA results, all of which include dynamic and suction effects. Percent difference is with respect to the experimental results. In almost all cases the projected stresses were higher than the experimental results indicated (as much as 155% higher). This can be attributed to both to the fineness of the mesh and to the safety factors incorporated in the

---

5 This conversion is made in order to make direct comparison to static hand calculations.
calculations. Safety factors are inherently conservative\(^6\) and it is clear in this case that the inertia effects during liftoff were below the magnitudes anticipated in the worst case.

**Table 3.5:** moment and stress results including dynamic and suction effects per SAP2000

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Moment (lb-ft)</th>
<th>Moment (lb-ft)</th>
<th>Flexural Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>415</td>
<td>2490</td>
<td>203.27</td>
</tr>
<tr>
<td>2</td>
<td>420</td>
<td>2520</td>
<td>205.71</td>
</tr>
<tr>
<td>3</td>
<td>425</td>
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<td>-1500</td>
<td>-73.47</td>
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<tr>
<td>5</td>
<td>-150</td>
<td>-1500</td>
<td>-73.47</td>
</tr>
<tr>
<td>6</td>
<td>-190</td>
<td>-760</td>
<td>-93.06</td>
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<td>80.82</td>
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<tr>
<td>8</td>
<td>-260</td>
<td>-1040</td>
<td>-127.35</td>
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</table>

**Table 3.6:** comparison of all flexural stresses (psi) that include dynamic and suction effects

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Experimental Results [Abi-Nader, 2009]</th>
<th>Static Calculations</th>
<th>SAP2000 fine mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stress</td>
<td>% difference</td>
<td>Stress</td>
</tr>
<tr>
<td>1</td>
<td>154</td>
<td>172</td>
<td>11.7</td>
</tr>
<tr>
<td>2</td>
<td>157</td>
<td>172</td>
<td>9.6</td>
</tr>
<tr>
<td>3</td>
<td>165</td>
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</tr>
<tr>
<td>6</td>
<td>-47</td>
<td>-78</td>
<td>66.0</td>
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<td>7</td>
<td>61</td>
<td>85</td>
<td>39.3</td>
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<td>8</td>
<td>-92</td>
<td>-115</td>
<td>25.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Experimental Results [Abi-Nader, 2009]</th>
<th>ADINA 2-D Shell</th>
<th>ADINA 3-D Solid</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stress</td>
<td>% difference</td>
<td>Stress</td>
</tr>
<tr>
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<td>185</td>
<td>20.1</td>
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<td>157</td>
<td>185</td>
<td>17.8</td>
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<tr>
<td>8</td>
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<td>-120</td>
<td>30.4</td>
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</tbody>
</table>

\(^6\) Assuming that these inertia effects are not mutually exclusive makes the model even more conservative.
4. Punching Shear

As noted earlier there are locations of very high stress at the insert locations. These are modeled as supports where a great portion of the shear is carried. The SAP model does not account for this, but the ADINA model requires that a finite area over which these reactions occur be prescribed. ADINA does not allow for stress singularities. It is not possible to have an infinitesimally small bearing area; it would lead to an infinitely high stress. For this reason, it is important to realize that idealizations cannot be made about how that shear stress is resolved. It is then important to determine the mechanism by which the shear is transferred. In this design, as in all tilt-up panel designs, there is an insert attached to a hook that pulls the panel off the ground. The high concentration of shear wants to pull the insert out of the concrete; this failure mode is referred to as punching shear. In order to reinforce the insert, steel rebar is laid down immediately adjacent and on all four sides to provide shear strength. This is shown in more detail below. The insert used in the design is an RL-24 insert, provided by Dayton Superior. For this study, the projected area of this insert is approximated to be 16 in².

![Figure 4.1: orthographic projections of RL-24 insert](image)

![Figure 4.2: schematic of the quick release attachment to the insert for lifting; figure courtesy of Dayton Superior](image)
In the case of a reinforced slab, both the concrete and the steel are said to provide shear strength. ACI 318-08 can be used to determine the allowable shear at a support based on the combined strengths of concrete and steel rebar according to:

$$V_n = V_c + V_s$$  
Eq ACI 11-2

where:

- $V_c =$ shear strength of the concrete (lb)
- $V_s =$ shear strength of steel reinforcement (lb)

$$\phi V_n \geq V_u$$  
Eq ACI 11-1

where:

- $\phi = 0.75$, the safety factor for shear

$V_u$ is the factored shear force. In our case $V_u$ does not need to be factored (e.g. 1.2 by LRFD for dead load) [ASCE 7-05] because we are very certain of the exact load magnitude.

Section 11.4.6.1 of the ACI 318-08 specifies a minimum amount of steel shear reinforcement if the applied shear force exceeds a certain percentage of the concrete shear strength. That is, if $V_u > 0.5\phi V_c$ a minimum amount of rebar is required. There are exceptions to these rules, one of which provides an exemption for “footings and solid slabs.” That said, research has shown that one-way slabs made of high-strength concrete (>7000 psi) or small coarse aggregate size may not be able to develop the full shear strength, and may fail at loads less than $V_c$. Fortunately, our panel is made of 4000 psi normal-strength concrete and behaves as a two-way slab; this means that there is no minimum steel requirement. However, the geometry of our panel requires steel shear reinforcements. The layout of the rebar is shown in Figure 4.3.

A red rectangle superimposed over Figure 4.3 indicates the location of the right insert. The dimensions of this section of rebar are 1.25 ft x 1.25 ft, which is illustrated more clearly in Figures 4.4 and 4.5. There is a plastic pour stop at the insert location. During the pour, it prevents the concrete from occupying that space. After the concrete sets, the plastic is removed. This provides a void that matches the exact dimensions of the pour stop, such that a crane bail with a lifting body can easily access and hook to the insert. The projected area of this subsequent void is approximated as 4 in x 4 in. Figure 4.5 shows the geometry of the rebar in the
neighborhood of the insert, which will be used to calculate shear capacities for both the concrete and the rebar reinforcement.

**Figure 4.3:** layout of rebar for panel; the area within by the red rectangle is shown in Figure 4.4; figure courtesy of Abi-Nader

**Figure 4.4:** zoom in on Figure 4.3, which shows the yellow plastic pour stop and the rebar, which serves as shear reinforcement to prevent pullout. If the plastic were not there, the insert would resemble Figure 4.1; figure courtesy of Abi-Nader
**Figure 4.5:** geometry of rebar around insert; the square in the center will be nearly devoid of concrete after casting.

This geometry can be used to define a new bearing area of the concrete to calculate shear stress. Section 11.11.3 of the concrete code provides provisions for “shearhead” reinforcement, which is typically realized as a steel channel or I-beam embedded within a concrete slab. The steel members make the shape of a ‘+’ in plan and the effective shear area can be found by drawing straight lines from the outermost extension of each member to that of the adjacent one. This makes a diamond shape, illustrated in Figure 4.6. It can be shown that the shear area is

\[ A_b = 148.5 \text{ in}^2 \]
The concrete panel can be modeled as a two-way nonprestressed slab with the pick points modeled as "interior columns" with rectangular dimensions. The SAP models provided above show that the reactions at the left and right support are 1097 lb and 1134 lb, respectively. When dynamic and suction effects are considered, which contribute an additional 80% load, these reactions become 1975 lb and 2041 lb, respectively. We will look at the right support and round that shear force up to 2050 lb. Let us look at the concrete code to determine how much strength is required in addition to the inherent strength of the concrete. The shear strength of concrete is represented as $V_c$ and is a function of the concrete’s compressive strength and the geometric properties of the section. In this case $V_c$ is interpreted as the reaction force the concrete section can take without punching shear failure. According to section 11.11.2.1, "for nonprestressed slabs and footings, $V_c$ shall be the smallest of:

$$V_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}b_0d$$

Eq ACI 11-31
\[ V_c = \left( \frac{a_s d}{b_o} + 2 \right) \lambda \sqrt{f_{c'} b_o d} \]  
\[ V_c = 4 \lambda \sqrt{f_{c'} b_o d} \]

where \( \beta \) is the ratio of the long side to the short side of the column, concentrated load, or reaction area. In our case we assumed that the shear bears over an area that is symmetrical such that \( \beta = 1 \). \( \lambda \) is a modification factor that accounts for the differences in mechanical properties. For normalweight concrete this shall be taken as 1. \( f_{c'} \) is the 28-day compressive strength of concrete. \( b_o \) is the perimeter distance of the shear area. \( d \) is the distance from the extreme compression fiber to the centroid of the steel reinforcement. In our case this is equal to half the depth of the panel because the neutral axis of the rebar was placed at the vertical midpoint, and spanned laterally. \( a_s \) is a constant based on the placement of the support. It shall be taken as “40 for interior columns, 30 for edge columns, and 20 for corner columns.” The inserts can be most appropriately modeled as interior columns. The values of these coefficients are:

\( \beta = 1 \)
\( \lambda = 1 \)
\( f_{c'} = 2500 \text{ psi} \)
\( b_o = 47.1 \text{ in} \)
\( d = 1.75 \text{ in} \)
\( a_s = 40 \)

Eq ACI 11-32 provides the smallest value of \( V_c \), so the shear strength of the panel at the inserts shall be taken as

\[ V_c = 14,365 \text{ lb} \]

Since \( V_u < 0.5 \varphi V_c \) (i.e. 2050 lb < 5380 lb), 11.4.6.1 indicates that there is no need for steel shear reinforcement because the concrete has sufficient strength. However, this assumes the concrete is continuous in depth and breadth. We have a 16 in\(^2\) void with almost no depth of concrete. It

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\(^7\) Since panels are lifted before the full 28-day compressive strength is reached, let us assume that \( f_{c'} = 2500 \text{ psi} \) for this calculation.
is intuitive then that the insert would easily pull out at less than 2050 lb. Therefore steel reinforcement is most definitely required. The design by Steinbicker & Associates incorporated additional rebar around the insert as shown in Figure 4.4. The shear strength contribution of the concrete is small compared to that of the steel in this case due to the geometry and dimensions provided. Let us then assume that the steel rebar accounts for 100% of the shear strength. The orientation of the rebar dictates that it will resist the shear by dowel action [Wang, 2001]. The shear stress in the rebar can be found according to:

$$\tau = \frac{V}{kA_s}$$

where:

- $\tau$ = shear stress (psi); 60,000 psi for Grade 60 steel rebar
- $V$ = shear force (lb)
- $k = \frac{5}{6}$; coefficient that accounts for the state of non-uniform stress in the bar
- $A_s$ = area of steel (in$^2$)

A safety factor of $\phi = 0.75$ should also be included. From this equation we can solve for a required area of steel and then check the adequacy of the design. Before that, the actual value of $V$ must be calculated; it is not necessarily equal to the reaction force (2050 lb). Figure 4.4 is repeated here to take another look at the rebar configuration at the insert.
The reaction force on this insert is 2050 lb. That load is transferred into the rebar directly above it in double shear by dowel action, which is then transferred to the bars above that in single shear, and then to the concrete itself. We have already shown that the concrete has sufficient strength to take that entire load. Now we need to size the area of the rebar. The insert is sufficiently wide such that that load is divided and taken at two discrete points on each rebar; each bar experiences double shear. This means that the shear the rebar must resist is actually only one-quarter of the reaction force. So, in Eq 6-1, 

$$V = 215.5 \text{ lb.}$$

Applying the safety factor $\varphi = 0.75$ and solving for the area of rebar required,

$$A_s = 0.014 \text{ in}^2$$

This can easily be accommodated by #3 bars ($A_s = 0.11 \text{ in}^2$), but Steinbicker & Associates chose to use #4 bars ($A_s = 0.2 \text{ in}^2$) [Washington University, 2010]. It is possible that #4 bars were required for flexural strength. It is common in practice to use the same size bar if demands are nearly the same or the bar that would otherwise suffice is one above or one below the rest. In this case #3 and/or #4 bars would be used, but it is very easy to confuse the two because they are similar in size. It makes it easier and less confusing for construction to use just one bar size in a case like this.

Even though there are high concentrations of stress at the inserts, it has been shown that the steel rebar can transfer all of it safely to the concrete. It is important to note that the concrete would
have been strong enough to support the punching shear without rebar had there been no void due to the insert. A 4 in x 4 in shear area of solid concrete with the panel dimensions given would have provided enough strength to safely resist 5380 lb. The sole reason for the implementation of rebar is the fact that the insert is nearly devoid of concrete. $V_c$ is the shear strength of unreinforced concrete, but its magnitude actually depends on the reinforcement; not on the rebar strength itself but on its geometry and orientation. Had the rebar been spaced closer together or had the square shown in Figure 4.4 had smaller dimensions, the shear area would have been smaller, decreasing $V_c$. This is ironic and important to note.

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8 This is more than twice the maximum reaction force divided by the safety factor of 0.75. According to 11.4.6.1 no steel shear reinforcement would then be required.
5. Conclusions

5.1 History, advantages, and limitations

Tilt-up construction has been around for over a century. The first use of this quick, reliable, and cost-effective construction method came in 1893. Robert Hunter Aiken is now referred to by many as the father of tilt-up construction. While Aiken’s tilt-up applications were limited to wood structures mostly for residences, the versatility of tilt-up expanded at the advent of ready mix concrete and the mobile crane, both of which were developed out of World War II. The attractiveness of tilt-up construction rests in the fact that it is considerably cheaper than alternate methods. During a time when labor is becoming more expensive relative to material costs, it is becoming even more attractive. While tilt-up is ideal for low-rise buildings it is limited in its use for tall buildings. This is due in large part to the availability of free space on a construction site but also to the fact that panels experience higher stresses as their height increases. Tilt-up is simply not feasible for tall structures. That said, tilt-up still has many advantages and will continue to increase in popularity in the United States.

5.2 Design Example

The design example provided in this thesis was a review of the research done by Abi-Nader. In part, it investigated the flexural stresses in a concrete tilt-up panel during liftoff using both static hand calculations and finite element software. The computer programs showed that static hand calculations provide adequate initial estimates. The locations of highest flexural stresses were the areas in the immediate vicinity of the pick points and at approximate midpoint between supports. Of the three different types of computer simulations used in the analysis, it is recommended to model the panel as a 2-D shell. It is much less computationally expensive than a 3-D solid model and yields sufficiently accurate results with a fine mesh.

This thesis reiterates the importance of designing a tilt-up concrete panel for more than just service loads. Indeed, a panel often experiences the highest magnitude loads during erection, when it is lifted from the ground to its final position. An engineer should pay particular attention to these stresses when designing the panel and the reinforcement within the panel. More than
that, it is crucial that the engineer conduct an analysis to determine the ideal placement of the inserts for the crane. The design example provided in this investigation was a very simple case. Most often a panel is large enough such that a crane must pick it up from 4 or 6 points or more. In that case, moment diagrams are difficult to derive by hand, so specially designed computer software is required. In spite of this, tilt-up construction is an incredibly fast, reliable, and inexpensive method, ideal for low-rise structures. It will remain steadfast in construction and will continue to grow in popularity in the United States.

5.3 Punching Shear

One of the main design considerations in a concrete tilt-up panel is punching shear. This is the failure mechanism by which an embedded support will pull out of the concrete due to large shear stress. In this case the insert provided a void in the concrete, which significantly reduced the available shear strength. The design provided rebar in the immediate vicinity of the void. In the calculations provided in this thesis the rebar were shown to have enough capacity to take all of the shear stress. The rebar provided an order of magnitude more shear area than required, but the engineers likely chose a #4 bar here because it was used elsewhere in the design. The geometry and orientation of the reinforcement allowed a large enough shear area for the concrete to be able to safely transfer all of the shear as well.

5.4 Suggestions for Future Work

The previous study revisited the Ph.D. thesis work of Abi-Nader. The dimensions of his panel were relatively small because of spatial limitations of available facilities. A typical tilt-up panel has dimensions larger than these, requiring a more complicated lifting scheme. It may be required to provide more pick points to decrease the flexural stresses and pullout forces, especially if a larger aspect ratio (largest of length and width, divided by thickness) is considered. Figure 5.1 shows three examples of more complicated lifting schemes. The dimensions around the perimeter of these panels show the best locations for lifting inserts. There are also notes above each panel that give suggestions for minimum cable length relative to panel dimensions and insert location. A continuation of this thesis would consider a panel with larger dimensions such that a more complicated lifting scheme is required.
**Figure 5.1:** various crane lifting schemes; figure courtesy of Dayton Superior
6. References


2. ACI Committee 318 (2008). *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. American Concrete Institute.


6. ACI 7 (1986). “Tilt-up Construction”, American Concrete Institutes, P.O. Box 19150, Redford Station, Detroit, Michigan 42819


