Testing procedures and guidelines for flexural stresses during lifting of full-scale, tilt-up panel instrumentation in the field

by

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Abstract

According to the Tilt-Up Concrete Association, over 10,000 buildings, which enclose more than 650 million square feet, are constructed each year using the tilt-up concrete construction method. This construction method is economical when done correctly. Panel lifting is proprietary and independently tested. Typically, the panel lifting inserts are designed for the highest tension forces, which occur when the panel is initially lifted from horizontal due to the initial suction forces to release the panel from the slab, where the failure of the lift insert would be tension and the concrete surrounding the insert would be punching shear. Lift inserts are also designed for the highest shear forces, which occur when the panels are set in their final vertical position prior to the crane rigging being removed, due to the inserts supporting the panels until they are properly braced and released. A scaled test has been conducted to determine the locations of maximum compressive and tensile strains (Abi-Nader, 2009). However, little to no published research exists on the actual behavior of full-size tilt-up panels during lifting. This report outlines the steps and procedures to conduct full-scale testing of panels in order to have a more comprehensive understanding of tilt-up panel flexural behavior during erection.

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Chapter 1 – Introduction

This report is focused on the development of a testing procedure for a full-scale tilt-up panel. The chapter will cover an introduction to this research and why it is important. Additionally, in this chapter the reader will get an overview of tilt-up concrete, to include: advantages versus limitations, rigging and lifting configurations, as well as design and construction practices. Following this chapter, a literature review will introduce the concepts focused on during the development of a testing procedure for full-scale, tilt-up panels. Finally, a detailed account of the testing procedure developed will be discussed in addition to lessons learned in the testing methods.

1.1 Research Overview

Tilt-up is a two-step process of concrete construction. The first step involves the concrete walls, referred to as *panels* in this report, which are cast horizontally on an existing slab. After the concrete for the panels is horizontally placed and cured, the panels are lifted when the compressive strength reaches 75 percent of the design specified compressive strength or a minimum of 2500 pounds per square inch (psi), depending on the lifting insert layout and panel configuration and size. The panels are lifted or “tilted” with a crane into the final position in the building (TCA, 2019b). Typical panel weight varies from 80,000 to 120,000 pounds with the heaviest panel on record weighing 369,000 pounds (Guzzon, 2019). Tilt-up construction is recognized as a form of precast concrete construction in both the American Concrete Institute *Building Code Requirements for Structural Concrete and Commentary* (ACI 318) and the International Building Code (IBC, 2018). The construction market for tilt-up is one of the quickest growing in the United States of America due to cost effectiveness at the beginning of the project as well as fast-track schedule, durability, and architectural versatility (TCA, 2019a).

Lifting hardware consists of the inserts and the attachments to the rigging for the lifting and numerous patents of lifting hardware for tilt-up panels exist. In Figure 1-1, the lifting hardware can be seen laid out in the panel prior to the concrete being poured – the inserts have the yellow caps on them to protect the connection during the pour. This report focuses on the lifting inserts for design of the testing methodology due to the stresses induced in the panel at these locations. The panel lifting inserts installed for construction are checked using calculations for the highest tension forces (which occurs when the panel is being lifted from the horizontal position) and the highest shear forces (when the panels are placed in the vertical position). All

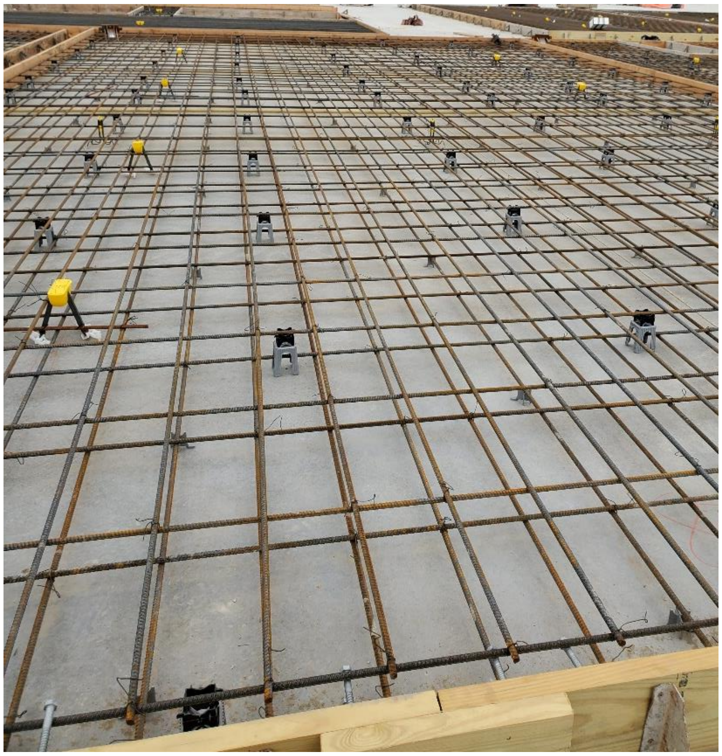


Figure 1-1: Lifting hardware prior to concrete pour

lifting inserts are proprietary and tested to determine general insert ratings via pull and shear tests on smaller sample/test specimens rather than a full-scale tilt-up panel (Tilt-Up, 2011).

Little to no research/test data on how full-size, tilt-up panels perform during the lifting/construction process exists. Some theoretical and scaled, lab-controlled research regarding the behavior of tilt-up concrete panels in the vertical position has been conducted in the past (ACI SEACO, 1982). Numerous patents for lifting devices for tilt-up panels exist. Lifting hardware consists of the inserts and the attachments to the rigging for the lifting.
The panel lifting inserts installed for construction is checked for the highest tension forces (which occurs when the panel is being lifted from the horizontal position) and the highest shear forces (when the panels are placed in the vertical position). This proprietary lifting inserts are typically tested via pull and shear tests on smaller sample/test specimens than a full-scale tilt-up panel.

However, little to no research/test data on how full-size, tilt-up panels perform during the lifting/construction process exists. This report investigates the required testing equipment and procedures to generate the flexural responses of full-size, tilt-up panels during the lifting process, specifically examining the strains in the concrete and reinforcing steel. Since the stresses and strains during lifting are generally the highest a panel will experience, having a better understanding of the subsequent cracking behavior and flexural moments within the panel may lead to an improved lifting design.

By establishing the procedure to gather test results from full-scale, on-site tilt-up panels, the design and construction industry may have a more complete understanding of how this building system truly behaves during lifting. Utilizing field tests to establish benchmark data can help determine what future research is needed for tilt-up concrete panels as a design and construction method. This report focuses on the critical step of defining the testing procedure.

This full-scale, field testing is needed: (i) field conditions (outdoors) are more variable and are truer to what is seen by structures than lab controlled tests, (ii) few laboratories in the US could house a full-scale panel test, and (iii) scaled down testing changes the size of the reinforcing steel and aggregate of the concrete (scale factor) and in turn the material properties of composite action.

1.2 Goals & Objectives

The goal of this report is to establish a written procedure and documentation for on-site testing of full-scale, tilt-up panels creating a baseline of panel behavior during the erection process. The following objectives for this report are:

1. Create a systematic method to quantify stress variations in tilt-up panels during the erection process by utilizing field tests.
2. Understand common issues in testing and create a standardized solution to streamline future research.

1.3 History of Tilt-Up Use and Design

Tilt-up is a form of concrete construction that has been in use since the early 1900's in North America (TCA, 2019b). Tilt-up is known for its speed-of-construction, economy, and having similar advantages of other concrete construction. However, it was initially only used for large warehouses and box type stores. Today, owners and contractors have begun utilizing tilt-up construction in retail centers, schools, and office buildings (TCA, 2019b). Tilt-up is one of the faster growing concrete construction methods in the United States of America with roughly

10,000 new buildings constructed annually (TCA, 2019b). The following section introduces the advantages and disadvantages associated with this construction method.

1.4 Advantages and Limitations of Tilt-Up

The responsibility of the design of tilt-up concrete panels can be unique compared to other reinforced concrete structures. Two design approaches can occur. One where more than one engineering company is involved and another when the Engineer-of-Record (EOR) does the completed building design and the construction design of the panels. In the both methods, the structural EOR for the project is responsible for the design of panels for the building performance, such as designing the panels for wind and seismic loads with the roof, floor, and foundation connections in-place. Additionally, the EOR will design the diaphragm and connections of all of the structure framing into the panels. Where the two methods deviate is who is responsible for the design of the panels during construction, i.e. the determining lifting locations, lifting inserts, rigging, and bracing requirements. Traditionally, the EOR is not responsible of the means and methods of construction and any additional loading or instability of the structure that can occur during the construction process. In the first method, the general contractor will hire another engineer to analyze the potential stresses in the panel to locate the lift points and bracing locations. This engineer will also analyze the panels to determine if additional reinforcement is required during construction, such as strongbacks, which are stiffener elements (vertical) that are added to a panel to provide temporary reinforcing. This designer is hired in addition to the structural EOR due to the proprietary nature of lifting inserts manufacturing. In the second method, the EOR who specializes in tilt-up, design the panels for the completed structure and during construction.

Tilt-up has several main advantages compared to other concrete structures, including poured and precast concrete, the largest being economy during construction. Tilt-up is economical for several reasons: (i) less formwork due to shared edges, (ii) faster erection timeline, and (iii) lower operating/labor costs during construction and post-building occupancy (TCA, 2019a). Tilt-up is also advantageous due to the durability of concrete design, architectural versatility, and overall speed of construction. On the construction side, tilt-up prevents a lower overall site safety risk. This is due to the limited number of crew members that are within a dangerous proximity to the panel during lift, compared to other construction where you could have a large number of workers on project scaffolding for instance.

However, tilt-up is limited by the knowledge of the designers/engineers and contractor. Tilt-up when done properly is extremely safe and reliable, but when the coordination with the contractor and the engineer is lacking and the experience of the construction team is limited, tilt-up can be extremely dangerous. While the overall safety risk is lower, tilt-up does still present a unique crew and site safety problem. The panels are generally large and heavy, and if the rigging breaks or the panel is not properly braced upon erection, panels can cause loss of life to the crew and/or crane operators, as well as large values of monetary damage to the site and surrounding buildings. Additionally, tilt-up panels must reach an adequate strength in shear and flexure prior to lifting which is another limitation of panel design and overall building height is a limitation.

1.5 Rigging and Lifting

Rigging consists of the cables and harnesses used in conjunction with a crane to lift a panel from horizontal position to its final vertical position (Bono, 2011). Several standard rigging configurations exist to minimize the stresses from pick point locations. Lift companies such as MeadowBurke and Dayton Superior have engineers who run analysis through their proprietary software on the panels to evaluate stresses as they are lifted to determine the best pick point locations. The analysis is important because as the panel rotates the stresses changes, which can result in one insert experiencing an increased in tension while another decreases (Dayton Superior, 2018).

1.5.1 Rigging Variations

For smaller panels (i.e. a geometry of 21’-0” wide by 15’-0” tall), the rigging layout is relatively simple and has a lower risk for injury to workers or panel, because it has fewer complications. Larger panels have a more complex rigging such as multiple slings and braces to be maneuvered around, but standards are in place to reduce stresses by taking mechanical advantage of pulleys. After lifting and setting a panel into the final position, the rigging balances

Figure 1-2: Field Rigging 2 Wide x 4 High

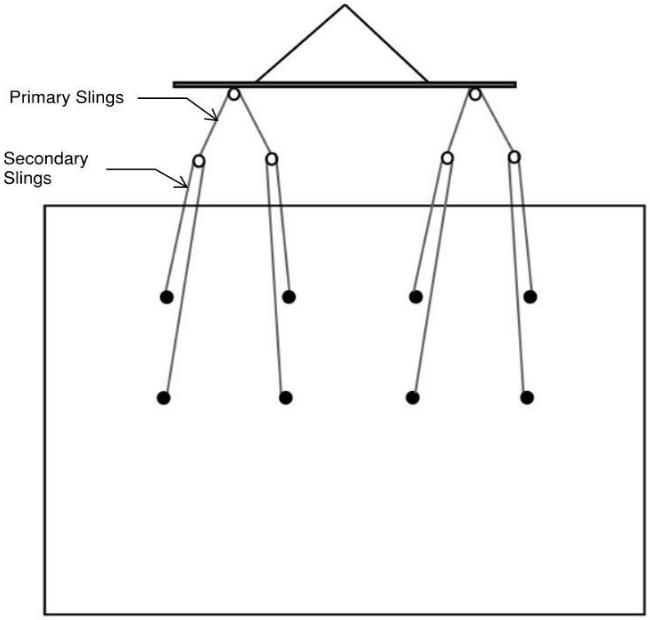


Figure 1-3: 4 Wide x 2 High Rigging

the panel until braces and strongbacks are arranged to support the panel. Once the braces are securely fastened to their foundations (slab, etc.) and the panels are relatively plumb, the crane rigging can be removed. The braces are the only support needed by the panel until the rest of the panels are placed (Bono, 2011). Figures 1-2 and 1-3 show examples of typical rigging configurations and indicate primary versus secondary slings, which are used with pulleys on larger panel layouts.

1.5.2 Lift Insert Locations on Panels

Lift inserts are placed in the panel "…so that the cantilevered portions of the panel sides or top will reduce the bending moments between lift points, thereby reducing the compressive and tensile stresses in the concrete" (Dayton Superior, 2018). Rigging configurations also typically apply to lift insert configurations. Special attention must be given to this component of design as the erection of panels is the most crucial and risky part of tilt-up design due to the weight and size of the panels. Lift inserts are always tied to the reinforcing steel per manufacturer specifications and may be flush with the panel face or have a slight offset.

1.6 Panel Strength at Time of Lift

As discussed previously, tilt-up panels must reach an adequate strength in shear and flexure prior to lifting. Due to the initial suction forces and the flexural stresses that occur during the rotation of the panel from horizontal to vertical, the stresses seen during lifting are generally the largest a panel will experience over its lifetime. The biggest impact of this is the material strength of the concrete and reinforcing steel. Material strength also directly impacts performance of the flexural stresses induced within the panel during lifting. For these reasons, it is important to understand these material properties when testing and taking measured readings.

Concrete is a composite material consisting of aggregates that are chemically bonded together by hydrated Portland cement. The stress-strain curve is nonlinear even though concrete consists of essentially elastic, brittle materials. Typically, concrete strength refers to the uniaxial compressive strength as measured by a compressive test of a standard test cylinder (ASTM C39) – this test is used to monitor the concrete strength for quality control or acceptance purposes. Generally, four by eight-inch cylinders are used. They are placed in a compression machine where a load is continually applied, until the specimen fails.

In addition to compressive strength, knowing the tensile strength and modulus of rupture of the concrete at the time of lift is important. Two types of tests to determine the tensile strength are widely used. The modulus of rupture can be determined by a flexural test (ASTM C78), where a plain concrete beam, generally six-inch squared by 2'-6" long, is loaded in flexure at the third points of a 24-inch span until it fails due to cracking on the tension face. The modulus of rupture, f_r, from a modulus-of-rupture test is calculated from the following equation, assuming a linear distribution of stress and strain:

$$\boldsymbol{f_r = \frac{6M}{bh^2}}$$ **Equation 1-1**

M = moment applied

b = width of the specimen

h = overall depth of the specimen

The second common tensile test is the split cylinder test (ASTM C496), in which a standard 0'-6" by 1'-0" compression test cylinder is placed on its side and loaded in compression along a diameter. The split-cylinder strength, f_{ct}, from a split-cylinder test is computed as:

$$\boldsymbol{f_{ct} = \frac{2P}{\pi l d}}$$ **Equation 1-2**

P = maximum applied load in the test

l = length of the specimen

d = specimen diameter

Different strengths are given from various types of tension tests for the same concrete, but generally, the strength decreases as the volume of concrete that is highly stressed in tension is increased. ASTM C78 modulus of rupture strength on average is 1.5 times that of a split-cylinder strength (ASTM C496). Although the tensile strength of concrete increases with an increase in the compressive strength, the ratio of the tensile strength to the compressive strength decreases as the compressive strength increases. The tensile strength is approximately proportional to the square root of the compressive strength. The American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318-19), Section 19.2.3.1 defines the modulus of rupture of concrete as:

$$\boldsymbol{f_r = 7.5\lambda\sqrt{f'_c}\ [psi]} \qquad \textbf{Equation 1-3 (ACI 318-19 Eqn. 19.2.3.1)}$$

λ = lightweight modification factor (λ = 1.0 for normal weight concrete)

Design of lift inserts does not use a specific equation to "estimate the modulus of rupture" due to material inconsistencies with concrete mixes across the country (Benton & Collins, 2020). Generally, a minimum modulus rupture and compressive strength is specified, and the panel inserts are designed based off of those (f_r=500 psi, f'_c = 2500 psi). Additionally, certain mix specifications can be given, and the insert designers can generate design of the concrete performance based off the specific mix (Benton & Collins, 2020).

The strength and behavior of reinforced concrete members is controlled by the size and shape of the members and by the stress-strain properties of the concrete and the reinforcement. The stress-strain curve defines the modulus of elasticity of the concrete, E_c. The modulus of elasticity is also known as the elastic modulus and is the ratio of applied stress to the corresponding strain and indicates the stiffness of the material. The modulus of elasticity of concrete consists of three components: (1) the secant modulus of elasticity, (2) initial tangent modulus of elasticity, and (3) tangent modulus of elasticity. Typically, the secant modulus is defined by using the point corresponding to $0.4f'_c$, representing service-load stresses as seen in Figure 1-4.

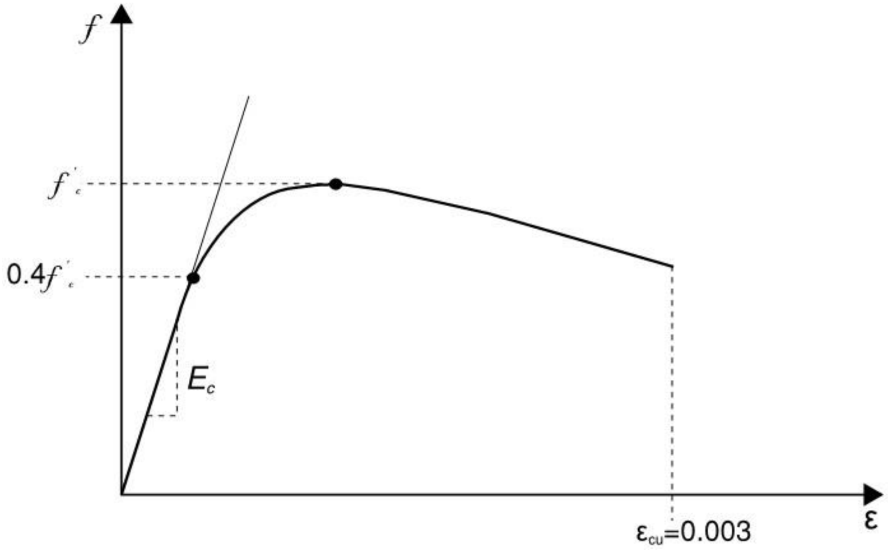


Figure 1-4: Secant Modulus of Elasticity

ACI 318-19 Equation 19.2.2.1.b defines modulus of elasticity of normal weight concrete:

$$\boldsymbol{E_c = 57000\sqrt{f'_c}\ [psi]} \qquad \textbf{Equation 1-4 (ACI 318-19 Eqn. 19.2.2.1.b)}$$

Flexural behavior of reinforced concrete depends on the tensile strength of the concrete and the reinforcing steel used. The stress in the concrete and then the reinforcing steel (once concrete cracks) is calculated from the strains in the material and is generally done using a stress-strain curve. Due to the brittle nature of concrete as a material, reinforced concrete relies on the reinforcing steel for its ductile properties. Flexural cracking in the tension region occurs once the stress in the extreme tension fiber of the concrete reaches the modulus of rupture. From there the tension reinforcing steel will enter the elastic region until the applied load reaches the yield point of the reinforcing steel and then the member will be in the inelastic range, which would indicate a failure stage of the member. If a single layer of reinforcing steel is not adequate for the designed flexural stresses, then other layers of reinforcing steel can be added.

1.7 Current Lifting Practices and Variations Within Industry

Several surveys have been issued to the construction industry over the practices and procedures they follow in tilt-up construction. This section discusses current practices in industry.

1.7.1 General Practices in Tilt-Up Construction

In tilt-up, the construction practices can vary either due to engineer design or local standards as mentioned above. This section will discuss some of those practices and variations. While these do not directly impact the testing methodology it is important to note the differences and that the test method proposed in this report can work for all of these scenarios.

Abi-Nader in 2009 conducted a survey to determine general practices in tilt-up construction. His survey was sent to 196 construction firms with experience in tilt-up construction and he received with responses from 54 of those companies (response rate of 28%). Based on a survey done in his research in 2009, most of the tilt-up construction is done in the Southeast and Western/Midwestern regions of the USA, by mid- to smaller sized companies with 20+ years of experience (Abi-Nader, 2009).

One of the items surveyed was how much time was specified by the engineer of record elapsed between when the concrete was placed for the panels and when the panels were lifted.

After the concrete for the panels are placed and cured, the panels must reach a certain compression and flexural strength before lifting begins. Ninety percent of the respondents were lifting panels before 14 days after the pour and of those fifty percent were between day five and ten.

The survey conducted by Abi-Nader also revealed that a large variation in construction practices occurs with the bond breakers used prior to the concrete pour for the panels (Abi-Nader, 2009). Bond breaker is key in tilt-up because it reduces the effect of the initial suction forces during lifting. In a study done by Moran in 2019, a survey was sent to 500 participants in construction, engineering, architecture and educators with knowledge/experience with tilt-up construction. Based on a total of 81 responses, it was found that seventy-five percent of respondents used a bond-breaker that was chemically active, and twenty-five percent used a non-chemically active bond-breaker. This same survey, found that the biggest issue with effectiveness of bond breakers was weather – high humidity and recent precipitation. Additionally, it was found that the manufacturers’ recommendations are not always followed in bond-breaker application (Moran-Puentes, 2019). Further discussion will occur in Chapter 3 over the impact weather can have, specifically moisture.

1.7.2 Applicable Codes and Standards of Practice

Currently the American Concrete Institute, ACI 318-19 *Building Code Requirements for Structural Concrete* and the 2018 *International Building Code* recognize tilt-up concrete as a construction method and the ACI has issued 551.1R *Guide to Tilt-Up Construction* to serve as a design aid. Additionally, the following ACI documents and standards have been issued for tilt-up construction: ACI 117 *Specifications for Tolerances for Concrete Construction and*

Materials, ACI 301 *Specifications for Structural Concrete for Buildings*, ACI 305 *Specification for Hot Weather Construction*, ACI 306 *Guide to Cold Weather Concreting*, ACI 315 *Standard for Design Details and Detailing Concrete Reinforcement*, and ACI 318 *Building Code Requirements for Structural Concrete*.

The Tilt-Up Concrete Association (TCA) has a Guideline Specification for Tilt-Up Concrete Construction. Additionally, Dayton Superior has a handbook that they have developed to help engineers and contractors in the application of tilt-up construction (Dayton Superior, 2018). Many of the ASTM standards that cover concrete design are also applicable for tilt-up concrete construction.

Chapter 2 - Literature Review: Tilt-Up Concrete

The articles discussed in this chapter were selected to give a more in-depth overview and understanding to the reader of tilt-up construction. The discussion and methods for design and construction presented in these articles and papers builds upon the information introduced in Chapter 1. Additionally, these papers provide a baseline for understanding and developing a full-scale test procedure for tilt-up concrete panels.

2.1 Introduction to Literature

While tilt-up concrete construction has occurred for decades, no independent research over full scale testing of lifting the panels is available to the public. A small scaled test conducted in a laboratory setting over the flexural properties of a small panel during lifting creates a baseline for future full-scale testing, which is discussed in depth in Section 2.4. Additionally, the sections in this chapter also offer a comprehensive understanding of construction and design decisions correlated to the construction site for tilt-up. Lastly, an introduction to the analysis procedures used in tilt-up design is offered.

2.2 “Location of Inserts – Stresses in Panels” by Peter D. Courtois

Courtois’ article “Location of Inserts – Stresses in Panels” from *Concrete International* presents big picture concepts for an introduction to tilt-up concrete. Courtois describes tilt-up panels as sections of a wall that are horizontally cast concrete, and once cured, are lifted and rotated by a crane to a vertical position and set in their final position (Courtois, 1980). The lifting inserts are attached to the flexural and temperature reinforcement in the panel. This reinforcement is required for panel function in its final vertical position but additional reinforcement is generally provided in lift locations, as shown in Figure 2-1.. Locations of inserts are designed to “…reduce the bending moments between pickup points, thereby reducing the compressive and tensile stresses in the concrete” (Courtois, 1980). Factors affecting insert location are placed in the panel based on the height, width, location of openings (for doors and windows) and weight

Figure 2-1: Inserts Tied to Rebar

of the panel. Lifting inserts may experience small impact loads during the erection process when the panels stick to the slab and are released. Today, lifting inserts have a 2:1 safety factor included in the capacity (Labor, 2019). All inserts should be securely installed per manufacturer specifications.

Due to the physical size of some panels, the concrete may experience some tensile stresses, especially if the panel only has one layer of reinforcement. Courtois indicates that engineers can work around this by calculating an acceptable tensile stress from the concrete, because at the time this article was written, panels were thicker and designed as uncracked sections. This tension stress is derived from the modulus of rupture of the concrete and a safety factor of two is generally applied. Concrete compressive strength is very important, and the panel cannot be lifted until a minimum compressive strength is achieved. In the 1980's, the normal range for tilt-up concrete tensile strength is 375-400 psi and compressive strength was 2,500 psi. Today, these values are typically the minimums allowed for lifting, which occurs at 75% strength as indicated previously. Courtois broke down the analysis of the panels into the following nine-step iterative procedure:

1. Draw loading diagram.
2. Calculate/determine the y-axis component of the panel's center of gravity from the bottom of the panel [$\bar{y}$].
3. Calculate panel weight.
4. Select lift point configuration and location to check.
5. Calculate the centerline lift (line of action from force due to crane).
6. Calculate force due to crane at initial lift.
7. Draw shear diagram.

8. Draw moment diagram.
9. Compute tensile stresses.

If the initial design is not adequate and stresses are too high, the process is repeated, and new insert locations and angles of lift need to be determined. In the current state of the industry, this design procedure would be done by the lift designer.

Early coordination with the contractor regarding panel design decisions is vital for the success of any tilt-up project. Since tilt-up panels are cast horizontally, a separate casting slab or the building base slab (first floor, slab-on-grade) needs to be poured and have enough curing time prior to any of the panels being cast. This slab, if it is the base slab for the building must be designed to take the load of the panels during casting. The height of the panels and the crane selection are also dependent on one another. This is due to the crane arm reach, which impacts the location of the crane for safe erection. Additionally, the crane is selected for a capacity of at least twice the largest panel on the job.

Courtois also generated a series of questions and answers that covered site logistics and information regarding the construction timeline. The main topics of these in constructability order are panel forming, curing and bond-breakers, panel erection and panel bracing. For panel forming in 1980, concrete trucks needed a clear path of access around the entire perimeter of the slab (building pad) if the panels are being poured on the base building slab. Today, pump trucks are used, and panels are cast on 80 percent of the slab; therefore, this is not required for concrete placement, but the crane will still typically need access around the entire perimeter of the building pad. The panels should be poured as near their final position as possible and for projects where stacked panels are used (one panel is poured and cured, then an additional panel is poured and cured on top of it), the lower panel should be adequately coated with bond-breaker and formed. Bond-breakers used in construction should be selected based on the ease of application, compatibility with the curing agents within the concrete and what the architectural finishes on the base slab and panel will be.

For the panel erection, it is crucial that the panels have reached the required compressive strength before lifting. As introduced previously, the compressive strength is important for the panel behavior as well as safety during lifting. Courtois indicates that the following three crucial items during the lifting of the panels needs to be addressed:

1. The crane operator and crane line runners should be careful in the initial lift of the panel (during the break of the suction from the slab to the panel).
2. Moving the panel into final place, especially if the panel is “walked” (when the crane and panel must move beyond just the rotation of the crane arm).
3. The placement of all corner panels.

These components of the panel lift are the most susceptible to a major issue should failure of inserts or crane rigging happen. The number of braces used on the panel after placement by the crane before the panel is self-standing and structurally part of the wall system will vary with the size of the panel.

While there are multiple facets to tilt-up construction, the focus on the design of the lifting inserts is the most important component of design. Lifting inserts will impact the panel performance during lift as well as the safety of the lift for the crane operator and erection team. This is the reason why the test methodology proposed in this report looks specifically at insert location for the placement of strain gauges, which will be introduced and discussed further in Chapter 3.

2.3 “Analysis, Design, and Construction of Tilt-Up Wall Panel” by Chim Lim

Chim Lim’s thesis “Analysis, Design, and Construction of Tilt-Up Wall Panel” presents the status of tilt-up concrete panels within the architecture/engineering/construction industry in 1987. Similar to Courtois’ *Location of inserts – stresses in panels*, an overview of tilt-up construction is presented. In addition, Lim presents an in-depth evaluation of the design and construction components affected when tilt-up construction is implemented.

To begin any tilt-up concrete project, the site and subgrade must be adequately prepared for construction; however, extra consideration needs to be given to soil and subgrade capacity to support the crane needed for the tilt-up panels (or to determine if the crane will be limited by the site and thereby limiting the panels). As mentioned before, the floor slab for the structure is typically serving a double purpose because it is the casting surface for the panels as well. Due to the stresses that this will create on the slab, the flexural stresses will generally determine the slab thickness. Additionally, the slab needs to have a smooth finish to help prevent a mechanical bond between the slab and the panel once it is poured (Lim, 1987). If the panel layout requires a panel to be poured over a portion of the slab that has an opening, the opening can be filled with compacted sand and covered with a thin concrete layer to create a consistent casting surface under the panel (the thin concrete coat will be knocked out after panel erection).

Lim indicates for panel casting, tilt-up has an advantage by having a lot of repetition and shared sides of panels, which allows for economy and ease in construction with less formwork; therefore, it is important for the designer/engineer to reduce large variations amongst panels. Openings in wall panels is unavoidable and panels with openings do need some additional design considerations. Specifically, panels with openings should not have too narrow an edge along the side of the opening – generally 2’-0” is the minimum width allowed between the edge of the opening and the end of the panel (Lim, 1987). Also, the designer/engineer should avoid having an opening shared between panels if possible. If an unstable panel shape, or a panel with part of an opening along the edge produces a cantilever condition, then strong backs or extra bracing may be required.

Additionally, Liam indicates prior to pouring concrete, the “panels” are laid out on the casting slab in chalk and then formwork is secured to the casting slab. Since some panels have shared edges, some panels can share formwork which contributes to the economic advantages of tilt-up concrete. It is vital though that the formwork is aligned and squared prior to the pour.

Bond-breakers and their application are one of the more crucial components of tilt-up design and are applied after formwork but before the pour. The bond-breaker “…prevents or reduces the mechanical bond between the casting surface and wall panel…” (Lim, 1987). The three main types of bond-breakers are:

1) Synthetic resin solutions
2) Waxes with metallic soaps
3) Solutions of silicone and esters

All bond-breakers are applied to the casting surface concrete prior to reinforcement placement/install and the panel being poured and must be applied per manufacturer specifications. Certain bond-breakers can have an impact on the finished surface of the concrete.

Similar to Courtois, Lim explains that the largest flexural stresses experienced by a panel typically are during erection. Lim further explains that attention to reinforcing steel placement can help eliminate cracking during erection as well as aid to design serviceability. Extra reinforcement is specifically important around openings and their corners (Lim, 1987). Lift inserts are securely tied to the reinforcing steel and sometimes will have a setback from the finish surface per manufacturer recommendation as seen by the yellow cap in Figure 2-1 on page 5. Lim cautions the reader that the inserts should never be welded to the panel reinforcing steel. The type of lifting inserts are determined by their position within the panel and can either be face or edge inserts. Face inserts occur within the center "face" of the panel and edge inserts are used to position bolts at the edge of a panel in the case a panel is not lifted from its horizontal face. Figure 2-2 shows the difference in face and edge lifting inserts. An important note for edge pick-up inserts is that the tension loop for crane and rigging connection must be a minimum of 1'-0" in length.

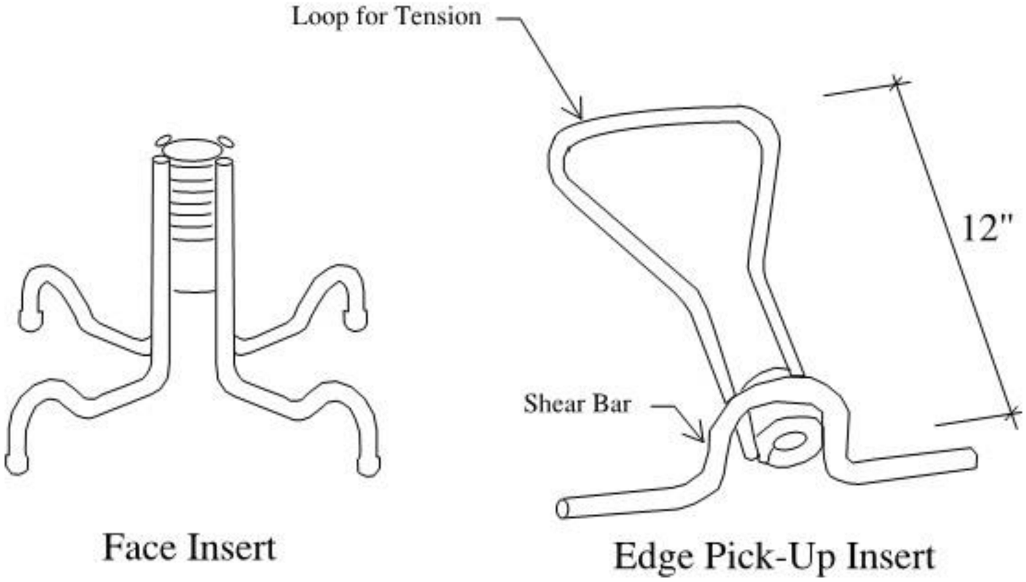


Figure 2-2: Pickup Insert Classification

Inserts are designed for four different failure modes: (i) concrete pullout from bonding failure, (ii) shear cone failure of concrete with insert, (iii) insert breaks, and (iv) insert unwinds.

Rigging can greatly impact the economy of tilt-up because of its impact on the crane time and therefore variations in rigging should be minimized where possible. Rigging configurations are generally described using a ‘high’ system. The “…second number in the designation indicates the number of inserts in the ‘high’ (top to bottom) direction” (Lim, 1987). All rigging configurations have a specific cable, lifting hardware, cable spreader bar, and cable minimum length (given by the manufacturer). Figure 2-3, shows a common four-high by two-wide lift configuration.

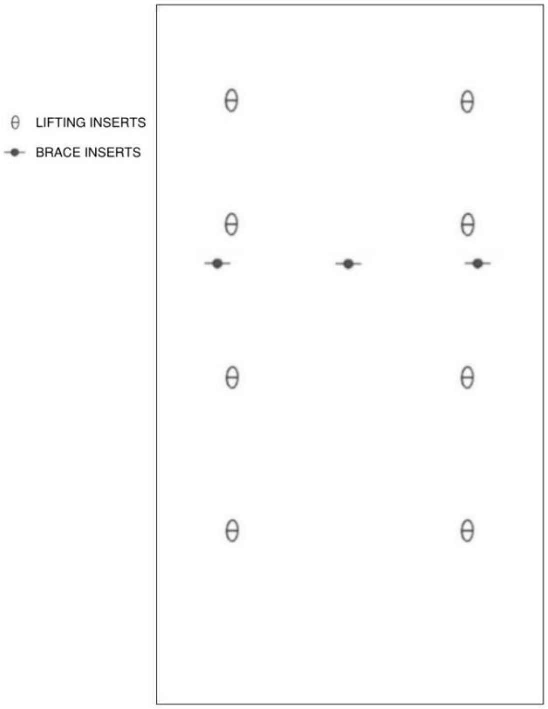


Figure 2-3: 4x2 Lift Configuration

For the erection of the panels, the site should be prepared in advance for the crane, with extra attention for the path around the site where the crane must travel - attention to debris and hazards such as overhead powerlines is important. All panel braces should be attached prior to lift and adjusted to the approximate height, this will help with the speed of panel positioning post lift (Lim, 1987). The cable used should be on the upper end of the size requirements. The larger diameter will help eliminate the potential of cable elongation due to shear. Additionally, the larger cable will help reduce impact loading from cable bouncing which can occur in thinner cable. Typically, once a panel is placed and braced into its final place the rigging is removed via a quick release system which helps prevent workers from having to climb ladders and speeds up the process. In a quick release system, “…disengage the lifting hardware… apply a quick downward force on the release rope and the lifting unit should be ejected from the panel" (Lim, 1987) and while it does not always release right away, this method is faster and generally safer than removing the rigging by getting on ladders in-between the braces.

Tilt-up panels have typical structural connections in addition to panel-to-panel connections, but panel-to-panel connections are becoming less common. All connections must be designed for adequate strength per the governing code, ACI 318. As with most connections, any connections that are visible (and especially panel-to-panel) must be watertight, and adequately coated for corrosion and fire protection. Due to the nature of tilt-up, all connections need to meet ductility and rotation requirements/capacity to prevent brittle failure of the connecting member material. The four connection types for roof and floor systems connected to the panels are: (i) cast-in-place connection, (ii) welded metal connection to an embed, (iii) embedded inserts, and (iv) drilled-in inserts which is similar to embedded inserts that are installed post erection. At floor or roof connections, a pocket can also be recessed into the panel to allow for an angle seat to be secured. This is common with roof systems with joists. Panel-to-panel connections will vary amongst designers/engineers; however, in long and narrow buildings, enough panel-to-panel connections are required to allow the wall as a system to resist overturning forces. Additionally, panel connections to the foundation should be aligned to "…keep them in relative position to each other…" (Lim, 1987).

Construction of tilt-up concrete panels, impacts all components of the panel design. Communication with the construction team will produce the most economical design of inserts, panel size, and lift/rigging configuration. Therefore, the communication with contractors during the development of full-scale testing was also crucial for similar reasons to design.

2.4 “Erection Stresses in Reinforced Concrete Tilt-Up Wall Panels” by Guy F. Abi-Nader

Guy Abi-Nader evaluates the design procedure for tilt-up concrete panels in conjunction with a small-scale tilt-up panel test in his 2009 thesis titled “Erection Stresses in Reinforced Concrete Tilt-Up Wall Panels”. In addition to traditional design checks, Abi-Nader also evaluated the use of the maturity method for concrete compressive and flexural checks which are outlined in ASTM C 1074 Standard Practice for Estimating Concrete Strength by the Maturity Method. The maturity method allows concrete strength to be estimated in early stages of curing. In 2009, it is applied to projects involving roadways and pre-cast members (Abi-Nader, 2009). However, this method could be useful in tilt-up construction because of the strength estimations in early curing, which assists with the timeline of a panel lift.

When using the maturity method, the main basic assumption is that two samples of the same age or maturity will have the same strength even if they were exposed to different curing conditions (Abi-Nader, 2009). This can be expressed in terms of temperature and time or equivalent age at specific temperatures. In his thesis, Abi-Nader focuses on temperature and time.

The testing involved in the small-scale (ten feet wide by nine feet tall) panel test will be described in further detail, but included: concrete tests during the panel pour, strength tests on cured concrete samples, maturity method tests, and the panel lift itself (lifted per construction standards). During the panel pour, Abi-Nader conducted the following tests: (i) slump test per ASTM C 143 Standard Test Method for Slump of Hydraulic-Cement Concrete, (ii) air content test by pressure method per ASTM C 231 Standard Test Method for Air Content of Freshly

Mixed Concrete by Pressure Method, (iii) unit weight test per ASTM C 138 Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete, and (iv) temperature test per ASTM C 1064 Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete.

Tests on cured concrete include: (i) compression testing, (ii) third point flexure testing, and (iii) splitting tensile testing. For the compression tests, specimens were broken at 1, 3, 7, 14, and 28 days, all in accordance with ASTM C 39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens. For each test, two specimens were broken and the average strength was determined and used for analysis. The modulus of elasticity of the concrete on the day of the lift was calculated in accordance with ASTM C 469 Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression from the concrete compression cylinder tests. This test was important to do the day of the lift so that the compression strength of the concrete during the lift was known as accurately as possible to improve the panel analysis during the lift ultimately increasing understanding about the stresses induced in the panel. The flexure test specimens were broken at 1, 3, 7, 14, and 28 days as well, again with two test specimens broken at each interval and the average used for analysis. The third point flexure tests were conducted over center point loading due to the increase in accuracy of the flexural capacity and were performed in accordance with ASTM C78 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading). Figures 2-4 and 2-5 shows the difference in the two flexural test types.

Splitting tensile tests were also conducted at 1, 3, 7, 14, and 28 days, again with two specimens broken at each time interval and the average used in analysis. The cylinders were each 6-inches in diameter by 12-inches tall, and the tests were conducted in accordance with

ASTM C 496 Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens. This test was done in conjunction with the maturity testing component of the research due to the correlation that occurs between these properties (Abi-Nader, 2009).

The maturity method was tested using the Nurse-Saul (temperature and time relationship) and compared to mechanical tests for two different panels and was found to be very accurate. The maturity method of estimating concrete strength in the field was used in conjunction with the above-mentioned tests to compare its accuracy and potential use with tilt-up concrete. The use of the temperature and time relationship to determine the maturity index, as used in this research, is also called the Nurse-Saul method. The Nurse-Saul equation is:

$$\boldsymbol{M = \sum[T(a) - T(o)]\Delta t} \qquad \textbf{Equation 2-1}$$

“where M = maturity (time-temperature factor) at age t

T(a) = average concrete temperature during time interval Δt

T(o) = datum temperature

Δt = time interval” (Abi-Nader, 2009).

Two main steps were used to conduct a maturity test: (i) creating a maturity calibration curve and (ii) measuring the maturity of the concrete. The maturity calibration curve was derived from the maturity index (which in this case is a time-temperature factor), concrete strength from mechanical tests, and establishing the corresponding relationships. The maturity of the concrete and its corresponding strength was then pulled from the curves. To measure the maturity of the concrete poured for the panel, IntelliRock ™ loggers from ENGIUS were used to log the temperature and maturity every hour (Abi-Nader, 2009). The datum temperature was the temperature below the point of no active hydration of cement in concrete for strength. Datum temperature was then determined via cube test cubes per ASTM C 1074 (Abi-Nader, 2009).

The small-scale panel used for analysis had a geometry of 10’-0” wide by 8’-0’’ tall, and 3-1/2” thick. The reinforcement was one layer of #4 reinforcing steel and was located in the tension region of the panel during lifting, no temperature and shrinkage steel was used. For the panel analysis of the small-scale panel test, the panel was evaluated in the X-X (horizontal) and Y-Y (vertical) directions for panel distributed weight and panel reactions. Once these values and reactions were determined a 1.5 safety factor was applied for the analysis of the panel at the initial lift (0°) addressing the suction stresses imposed during the release of the casting slab from panel. These suction forces were approximately a 20% increase of the dead load for panels cast on a steel bed and 40% increase of the dead load for panels cast on a concrete casting slab and were specific to Abi-Nader’s scaled test (Abi-Nader, 2009).

The small-scale panel for this research had a single row lift configuration and utilized RL-24 plate anchor lift inserts from Meadow Burke. The panel had two maturity loggers embedded inside the panel and no strain gauges were placed on the rebar. The strain gauges used were hard wire connected to a strain indicator and recorder, and the strain gauges determined static strains. The P3 strain reader and recorder had four input channels and used a quarter bridge circuiting. The gauge factor settings were set for a range of 0.500 – 0.900. A total of eight surface mounted strain gauges were placed on the panel, with six oriented along the Y-axis and two oriented with the X-axis and were placed at the expected locations of maximum moments.

For the analysis of the panel, the panel was considered a simply supported beam. Since this panel was small-scale, the reinforcement was located at the neutral axis and the stress on the ‘top’ surface was assumed to be equal but opposite to the stress on the ‘bottom’ surface of the panel. The study found that at 0° the stresses from hand calculations with a safety factor of 1.5

applied due to the suction forces at initial lift, and the computer analysis were very close to the actual stresses recorded in the lab test. The rest of the results were then tabulated by angle of inclination of the panel and compared between hand calculations, software and lab results. The final results showed that the majority of the software calculated stresses were similar to the experimental data collected during the lift.

2.5 "Tilt-Up Concrete Panels: An Investigation of Flexural Stresses and Punching Shear During Lifting" by Matthew P. Bono

Matthew Bono conducts a comparison of finite element modeling to evaluate accuracy to the resulting 'lab' results in his 2009 thesis "Tilt-Up Concrete Panels: An Investigation of Flexural Stresses and Punching Shear During Lifting". This thesis was evaluated in conjunction with the work done by Abi-Nader in 2009 (discussed in the above section), since the design example and 'lab' results used by Bono are based on the final panel design and test results from Abi-Nader's 2009 work at the University of Florida.

Bono begins his work by verifying the loads and governing conditions from Abi-Nader's design example specifically looking at the panel properties and behavior during the initial lift at 0°, since this is when the panel will have the highest punching shear. The panel for this study had an opening in it; therefore, the analysis had to evaluate the panel in both the X-X and Y-Y axes. After verifying the initial hand calculation design, Bono compared the static calculation design in SAP2000 and ADINA, both which are finite element modeling programs (Bono, 2011).

Bono's study focused on the initial lift of the panel - the panel was modeled as a 2-D planar element. To simplify the finite element model analysis in the SAP 2000 program, the panel was modeled with no internal reinforcing and was pinned about the base of the panel to allow the design to be modeled in a stable condition. In order to establish more accurate results, "…finer meshes around points of high stress, which include the pick points and areas adjacent to the void" were modeled (Bono, 2011). For the remainder of the panel, a square reinforcing mesh of 1'-0" was used. Additional analysis were run using a finer mesh patterns throughout the rest of the panel because poor meshing in a finite element model can result in unusual variations within the analysis calculations and results. For this reason, using square mesh elements will produce consistent results.

For the analysis in ADINA, the areas with abnormal stress results, such as pick points, need to be modeled. Similar to the SAP 2000 model, the geometry of the panel and boundary conditions must be established in the model. Since the panel is being checked in static condition for the initial lift, the dead load due to self-weight of the panel is applied as a distributed pressure load to the surfaces in ADINA (Bono, 2011). As with the SAP 2000 model, the accuracy and simplicity of the meshing is important to achieve reasonable results that are not too computationally heavy. In ADINA, a 3-D and 2-D element were analyzed. The 2-D shell was found to produce more reasonable and comparable results to measured test data (Bono, 2011). ADINA 3-D produced analysis design results that were similar to ADINA 2-D but the 2-D analysis was slightly closer to the measured experimental results, with the 3-D solid analysis having an average difference of 51.5% and the 2-D shell analysis having an average difference of 42.8%. SAPP 200 fine mesh analysis had an average difference of 44.2% from the experimental results but was similar to ADINA 2-D. Overall, the static hand calculations were the closest to the experimental results with an average difference of 26.6% (Bono, 2011). It was found that the SAP 2000 and the ADINA 2-D models produced closer results to the static hand calculations and the experimental results from the small-scale panel lift conducted by Abi-Nader for the majority of the panel. These results showing a relatively significant difference between what a panel actually experiences during lifting and what is consider the acceptable design procedure used today clearly indicate a need for additional research in this area. This report is the first step in understanding complete panel performance during lifting. The research done by Abi-Nader and followed up by Bono, acts as the first step in creating a testing method and procedure for full-scale panels on site and was vital in the progression of the research presented in this report.

Chapter 3 - Testing

The basis for the test procedure and set-up outlined in this chapter uses the knowledge and research of tilt-up construction that has been outlined in this report thus far. The main variation in this work compared to the research previously mentioned is the need for a full-scale test on a job site, which would require a wireless strain gauge recorder and transmitter system. For initial research, the panel size(s) selected should be close to the average panel size used in industry today.

This chapter will go over decisions that affect the testing methodology for each specific project and how to make those decisions based on jobsite factors. Next, the material tests required will be discussed and broken into two categories: (i) testing during the concrete pour, and (ii) testing within 24 hours of panel erection. Finally, an overview of strain gauge placement and testing design is discussed and related to its impact on further finite element analysis.

3.1 Testing Methodology

The focus of this section is the selection and use of strain gauges and the on-site testing procedures. This includes the install and set-up for the strain gauges, along with recommendations for trouble-shooting some potential errors. The test panels will later be part of a building therefore are not tested to failure. The goal is to determine the actual stresses imposed on a full-size panel in the field during a lift and compare these to the design values.

Strain gauges are the recommended form of data collection, because as discussed previously strain and stresses in reinforced concrete design are used to understand the tensile strength of the element being tested, which for the purpose of this report is the tested panel(s). In order to get a more complete understanding of the panel tested, both surface mounted and reinforcing steel mounted strain gauges should be used. This allows the research team to get the flexural responses on both faces of the panel as it rotates from horizontal to vertical.

Understanding the basic set-up and the communication between the strain gauges, wireless nodes, wireless gateways, and the base station is important to properly select all the testing equipment. The strain gauges collect the strains experienced in the panel at various locations which are then hard-wired to the wireless transmitting nodes on the panel. These wireless nodes then transmit the data collected via radio frequencies to the gateways which transmits the data to the base-station and laptop used by the research team.

It is important to note that wireless data collection methods were selected because the scale of full-size panel testing does not allow for a wired data collection. Wired connections are not feasible on a full-size tilt-up panel tests done in the field unlike lab testing. This is due to the safety concerns of the entire construction team during the erection process, along with the inability to have long enough lead wires from the strain gauges to accurately collect data without resistance drop in the wire. Additionally, wireless data collection minimizes risk of the wired connection being disconnected due to movement of the panel during lift. The wireless network also minimizes the on-site safety risk of the research team.

3.1.1 Determining Strain Gauge Locations

Prior to instrumentation and testing of the panels, strain gauge locations need to be determined. The goal is to locate these in positions of highest compression strain and highest tensile strain. These locations will allow the research team to then check the panel using statics and mechanics of materials to calculate the flexural moments induced in the panel during the lift. For the selection of reinforcing steel strain gauges, it is important to note that some simple material math will need to be done to convert the stress and strain from the location of the strain gauge on the reinforcing steel, to the extreme tension fiber in concrete on the bottom face of the panel (where it is not possible to place a surface strain gauge).

3.1.1.1 Strain Gauges

The strain gauges are sensors with resistances that vary as the applied force changes. These electrical resistances occur due to forces, pressures, etc. which can be measured. In the application of tilt-up construction the external forces applied during lifting are resulted in stress and strains.

For the mock research of this report, RISA 3D was used to determine the magnitude and location of the maximum positive and negative moments would occur at the initial lift (0°), 30°, 45°, and 60°. To simplify the analysis and reduce the potential for error, the panel was checked as a two-dimensional beam element. The beam element was checked in the "vertical" or height of the panel with rotation about the end that the panel would rest on the foundation, and in the "horizontal" which was the width of the panel. The "horizontal" moment check was done at each row of lifting inserts. For the purposes of the mock research analysis, the panel was analyzed as a 2-D model in RISA 3D; therefore, the z-axis movement was locked for each of the nodes, refer to Appendix C for an example of RISA analysis. Different sign conventions are used in analysis software for compression and tension. The research team should be aware of this. Nodes occurred at each end of the panel and at the pick point locations for the mock four-high two-wide panel lift. To account for the 2D model, the self-weight applied to the beam member included half of the panel width.

As mentioned already, it is important to check the panel(s) for strain gauge placement in the x- (width) and y- (height) directions. For x- and y-axes analysis, it might be found that in between the lift inserts both a positive and negative moment (one in each direction) occurs. If this situation occurs, then it is worth noting that two strain gauges, one surface and one reinforcing steel, should be used to capture both of those moments for the x- and y-axes.

In order to compare results between software and with the independent reviewer (who is used to verify placement of the strain gauges), a detailed and organized storage of raw data from the analysis is important. For an example of how to document the expected panel deflections and moments refer to Appendix C.

Another software that can be used in addition to or in substitution of RISA is Tilt-Werks. Tilt-Werks is a program offered by Dayton Superior to act as a design aid for tilt-up panels. Dayton Superior is one of the major designers and manufacturers for panel lift inserts. Tilt-Werks allows designers and engineers to design multiple panels at once and can generate detailed reports. However, Tilt-Werks generates panel design based on a full wall; therefore, more information about the building is needed beyond that of an individual panel. The software has additional limitations, such as, panel story height is limited to two, continuous bearing is assumed, and ACI 318-08 is the concrete code used, which may not be the design code followed by the engineer of record or the lift designer (Tilt-Up Design Systems, 2011).

The moment and shear locations determined in panel analysis (also checked by an independent reviewer) should be compared to the reinforcing steel shop drawings and panel drawings. The shop drawings should be requested from the general contractor/sub-contractor point of contact. Comparison between the actual panel design and the panel analysis allows the research team to make general predictions, such as, loads expected to be captured during the lift.

Prior to the finalization of strain gauge placement on the panels, the locations need to be confirmed with the contractor. Due to constructability concerns and crew safety, certain locations of strain gauges might need to be removed from the panel. Examples of these locations would be under or around the brace locations on the lower half of the panel.

3.2.1 Strain Gauge and Wireless System Selection

There are many different options available in the marketplace for strain gauges and wireless sensor nodes. Four factors should be considered when determining the most appropriate equipment for a test:

1. Jobsite Size – *How close in proximity can the research team setup to the panel during the lift?*
2. Panels – *How many panels is the research team going to run analysis on in a day?*
3. Panel Size – *How many reinforcing steel gauges versus concrete surface gauges does the research team need per panel and total?*
4. Sample Size – *How many data points is the research team wanting to gather?*

The strain gauge and wireless sensor node manufacture will need to understand the answer to these four decision factors to help you select the proper equipment. This report focuses on using Micro Measurement strain gauges and LORD Strain wireless sensor nodes. LORD Strain wireless sensor nodes were selected because they have experience with wireless strain gauge data collection and have a NEMA rated sensor housing which is important for protecting the equipment from the weather. The four factors indirectly effect how the strain gauges transfer the data collected from the panel(s) back to the research computer.

Discussion of the four factors will need to occur with the general contractor or with the tilt-up sub-contractor. This discussion is to confirm the researcher’s answers to jobsite size, number of panels to be tested in a day, panel size, and if strain gauges can be placed on the reinforcing steel before contacting the sales representatives of the recording devices.

3.2.1.1 Jobsite Size

The size of the jobsite plays a role in the selection of the wireless sensor node gateway. The wireless sensor nodes transmit the data collected from the strain gauges and the gateway allows the data to then be transferred to the base-station. The base-station is setup on the researcher’s laptop which will need to be located at a point on the site which will need to be close to the wireless node if a wireless gateway amplifier is not also used.

Based on an interview with Michael Golden in October 2018, a LORD Strain Sales Representative, the nodes, gateway and the base-station transmit data via radio frequency with 16 channel options, which can allow multiple panels to be instrumented at one time. LORD Strain offers four different gateways each with varying strength and configuration options. Communication with the general contractor is required since the general contractor will determine how close the researchers will be allowed during lifting to set up the base-station and the gateway to the panels. The sales representative will direct the researcher to select the model best suited for this specific distance (Benton & Golden, 2018). One of the gateway models, WSDA-200-USB, is the direct base station plug in for the laptop/computer direct via USB, as seen in Figure 3-1. This is beneficial in smaller test applications because it eliminates the need for additional equipment and expense. However, this model has a limited range of roughly 2000 feet and direct line of sight must be maintained without

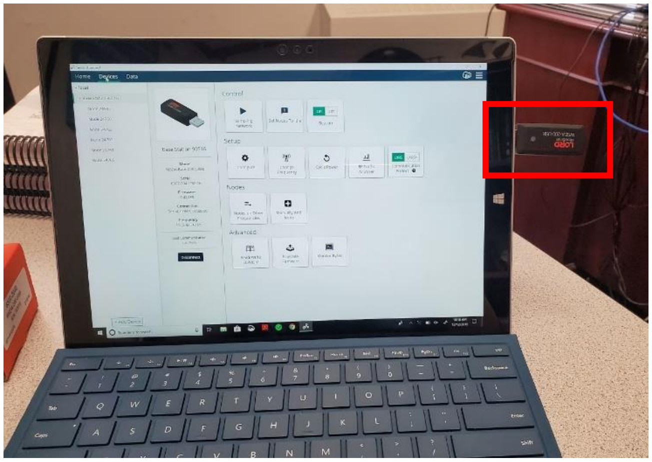


Figure 3-1: Base Station Direct USB Connection

major interference from any steel and other concrete that may be on-site. When direct line of sight to the test panel is not possible, one of the other gateways can be used in addition to the USB base-station plug in. However, for best results, consult with your sales representative to make sure the gateway selected works best for the researcher's specific application (LORD Strain).

In Figure 3-2, the main gateway options from LORD Strain are shown below. Gateways are the piece of the wireless system that collects and transfers the data from sensors to the researcher's computer. All the gateways are small, less than 0'-6" by 0'-6" and can be easily set-up on site without impeding work on site.

USB gateway for configuration and data collection.

Network-ready gateway that uploads data to SensorCloud. Includes J1939 CAN, Ethernet, and USB interfaces.

USB data gateway with eight analog outputs replicating any connected sensor.

RS232 data gateway for easy, reliable sensor data acquisition.

Figure 3-2: LORD Strain Gateway Options (Source: LORD Strain)

The main factor to remember when selecting a LORD Strain gateway (or equivalent) is that the distance the data needs to be transmitted from the wireless nodes to the base-station is extremely important and should be consider for all panels being tested.

3.2.1.2 Panels

The number of panels that the general contractor can allow the research team instrument will affect the selection of the wireless nodes and the gateway selected. Since the nodes collect data from the strain gauges, the quantity of panels instrumented for testing and lifted in the same day is needed. The wireless gateways and nodes have a limitation in the number of channels that they can operate at once, as well as the sampling rate that the nodes transfer data. Hence the importance of the number of panels tested in a day, as more than one gateway be required.

Panels on the jobsite are exposed to the weather; therefore, the information presented is based on using the SG-Link-200 wireless nodes that have a NEMA rated enclosure. Currently, these are the only weatherproof rated wireless nodes manufactured by LORD Strain. Each SG-Link node can be configured with up to three strain gauges. The strain gauges that are being used to instrument the panel are full, half, or quarter-bridge configurations which requires the node to be properly calibrated for the strain gauge shunt (LORD Strain, 2019). The strain gauge shunt configuration will be discussed in the next Section 3.1.1.3, when discussing panel size and strain gauge selection.

To properly make the final selection for the wireless gateway, the research team needs to know the number of nodes that will be instrumented and lifted at the same time. Although very unlikely, the construction team may lift more than one panel at a time, if this were to occur, or if three panels being lifted back-to-back, the sampling size and rate (amount of times the nodes take readings from the strain gauges per second) would result in a larger data acquisition which could require multiple gateways. This will result in either additional gateways to help process the data transfer from the nodes to the base station, or a base station will be needed for each of the panels lifted that day. Discussion with your sales representative will help the research team determine the best option.

3.2.1.3 Panel Size

The size of the panel(s) also affects the number of reinforcing steel strain gauges and surface strain gauges. For initial research, the panel size(s) selected should be close to the average panel size used in industry today, which is a solid 20’ wide by 40’ high (single-story).

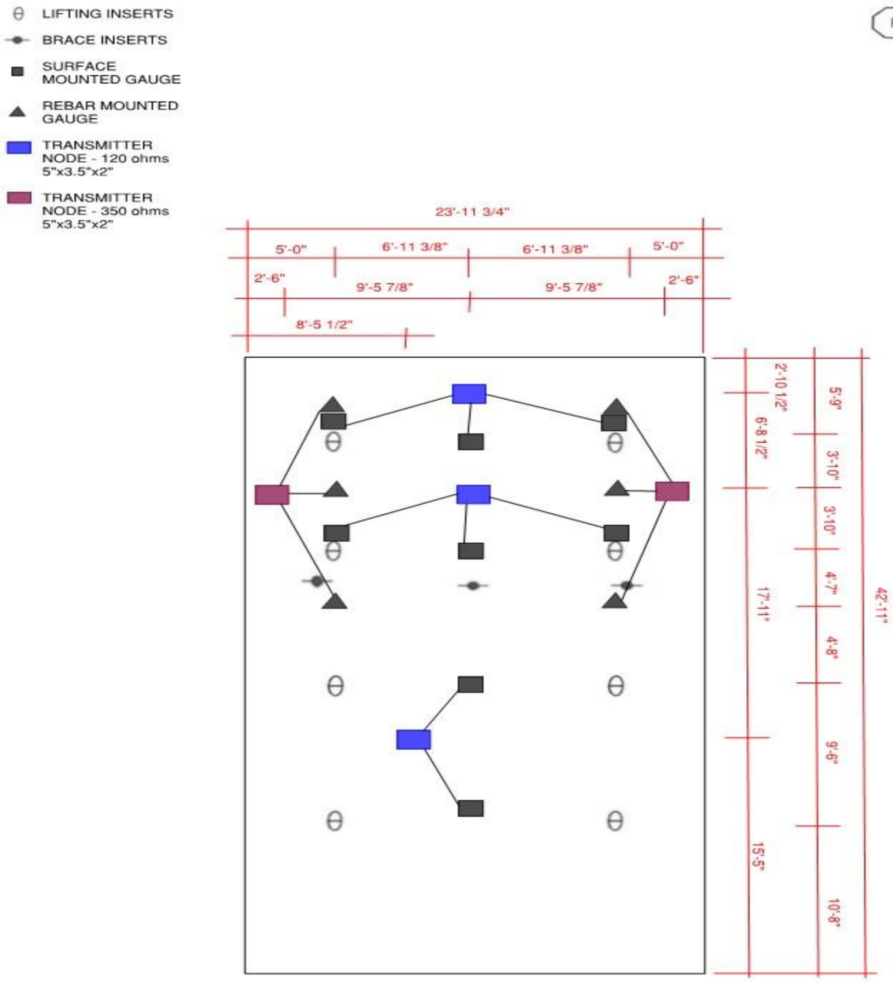


Figure 3-3: Example Test Panel Layout

In order to properly layout the nodes on the panel, it is important to know how many of each type of strain gauge the research team will be instrumenting and the resistance of the strain gauges. An example of possible panel layout with the strain gauges and wireless nodes is shown in Figure 3-3.

The manufacturer can provide the strain gauge sensor resistance and type once the research team determines the strain gauges required. This information needs to be conveyed to the node and gateway manufacturer. In tilt-up erection, the strain that is being evaluated is axial and bending, and these are typically measured with quarter- or half-bridge strain gauge configuration (National Instruments - White Papers, 2019). In order to properly collect data from strain gauges, especially quarter-bridge, a dummy resistor is required to complete the circuit and the wireless nodes need to be calibrated to account for this resistor if needed. The calibration of the strain gauges is required. If using a quarter-bridge configuration, the quarter-bridge wiring may be configured with two or three wires. A typical layout for these wirings, where the wires in the light blue box are indicating the wires to be connected to the wireless node from the strain gauges, are shown in Figures 3-4 and 3-5.

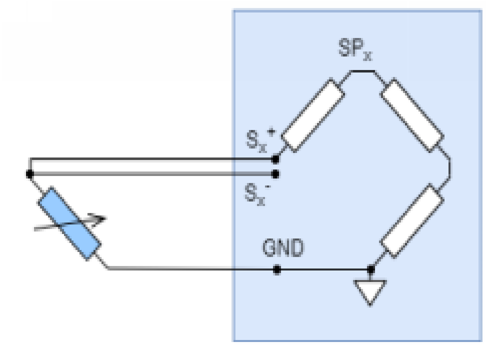


Figure 3-4: Three Wire Quarter-Bridge (Source: LORD Strain, 2019)

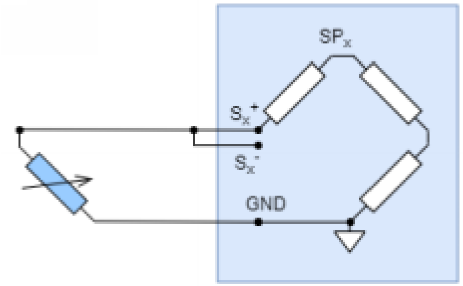


Figure 3-5: Two Wire Quarter-Bridge (Source: LORD Strain, 2019)

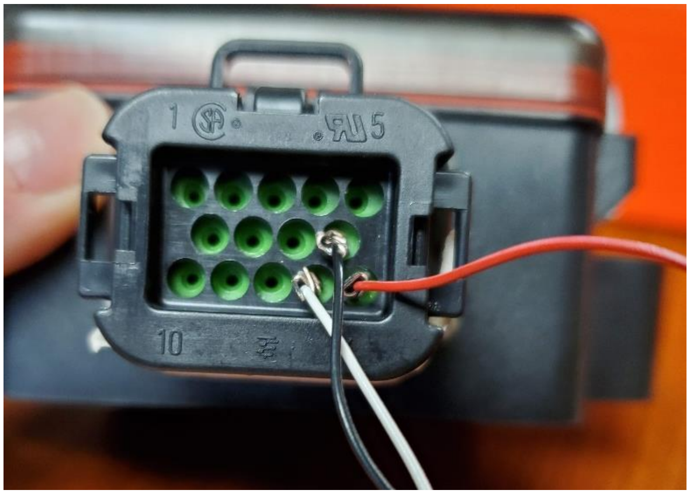


Figure 3-6: SG-Link 200 – Single Channel Wiring Example

Each of the SG-Link-200 nodes have three-channels that can be calibrated for any strain gauge. However, for optimal design of the test-set up, it is better to keep the same strain gauge resistance on a single node, instead of having a different strain resistance on each of the node’s three-channels. This will help eliminate potential issues when the researcher(s) install the nodes and gauges in the field. For an example of a three wire quarter bridge connection for a single channel in the SG-Link Nodes, refer to Figure 3-6. Further examples of this and other lessons learned are discussed in Chapter 5.

The panel size and lift configuration has a direct impact on the location of maximum moment stresses in the panel during lifting. To help explain this further, a mock panel design example was created for the purpose of this research. The example created is for a four high-two wide panel lift/rigging configuration as shown in Figure 3-7. Based on design calculations the maximum negative moment was found to occur in the middle of the two wide lift inserts in the horizontal direction and in the middle of the four high lift inserts in the vertical direction. For this situation, the maximum strain is located at the underside of the panel. Since a surface strain gauge cannot be placed on the underside of the panel prior to lifting, reinforcing steel strain gauges are required. In the locations of maximum positive moment, surface mounted strain gauges were used. Refer to Appendix C for the full tabulated results from an example RISA 3D check.

Figure 3-7: 4 High x 2 Wide Lift Rigging

The designer of the lift inserts and rigging will generally use a proprietary software to understand what the resulting positive and negative moments are. This can be recreated by the research team using analysis software, with the pick points acting as the supports for a simplified 2-D analysis. A generic analysis was done in RISA 3D, at 0°, 45°, and 90° to analyze the change in moments as the panel is rotated for the mock panel design example; discussed in this report. While RISA was used for this mock research, any analytical method would be appropriate in strain location determination.

3.2.1.4 Sample Size

The sample size is dependent on all the above-mentioned factors, including the frequency of readings per second. In the mock panel considered for this report, one panel would be lifted at a time and would be transmitting data from approximately 12 to 14 strain gauge sensors and six nodes. When speaking with our LORD Strain representative, a sample rate of ~128 Hz/gauge was determined to be a good fit; however, this number would change if more strain gauges and wireless nodes were connected to the same gateway (Benton & Golden, 2018).

Important configuration notes are discussed further in Chapter 4.

3.2.2 On-Site Testing Procedure

All on-site visits for installation and testing need to be confirmed with the researcher's general contractor contact prior to the visit. This allows confirmation of timing with the site personnel for any changes in time or date for concrete pours, lift day, etc. Also, prior to any site visit, the researcher/team needs to check with the general contractor as to the proper personal protective equipment (PPE) that they are required to wear on-site.

3.2.2.1 Rebar Strain Gauge Install

Step one of on-site testing is the install of rebar strain gauges. This step needs to be completed once the panels have been formed and the reinforcement installed prior to the placement of the concrete.

Proper installation might vary among manufacturers; therefore, confirmation of the following procedure should occur with the manufacturer once gauges are selected. The reinforcing steel strain gauge installation is broken into two parts (1) surface prep and (2) gauge install (*CEA strain gage installation with M-bond 200 adhesive (training video)*, 2019). The steps for proper surface preparation of the reinforcing steel are:

1. Create/find a smooth surface to allow the entire gauge to lay flat on the rebar. *Note: Do not grind down any surface smooth without getting written consent from the contractor and engineer of record.*
2. Clean the area which the gauge will be attached using acetone and an industrial cloth. Two to three wipes in the same direction, using different parts of the cloth until the cloth comes clean of any dirt/grime/dust.
3. Repeat the above step with distilled water.
4. Allow cleaned area to completely dry before moving on to gauge install.

The steps for gauge installation are:

1. Double check the strain gauge is working with a voltmeter.
2. Ensure the two-lead wires are not touching, using a pair of tweezers helps.
3. Seal the separated lead wires with electrical tape.
4. Position the gauge so that the assembly is straight, longitudinally, with the rebar.
5. Place electrical tape along the wires leading up to the strain gauge for two to three inches. Wrap some of the tape completely around the rebar to secure the wires in place.
6. Use scotch tape to anchor the gauge in the correct location along the rebar.

7. Lift the clear tape at a 45° angle, lifting the gauge completely off the rebar but leaving some of the tape still attached at the same end of the gauge as the soldered connections.
8. Apply required adhesive per the manufacturer's instructions.
9. Slowly reapply the gauge with the adhesive, ensuring the gauge lays longitudinal along the rebar. Gradually push from the anchored end to the opposite end to remove an air bubbles under the gauge.
10. Hold the gauge in place with a firm pressure for the required amount of time as specified by the manufacturer. (Approximately a minute and a half.)
11. Remove the clear tape slowly, again at a 45° angle, constantly checking to make sure the gauge does not come up with the tape.
12. Check strain gauge again with voltmeter. If a reading no longer occurs, remove gauge and reinstall a new gauge, and following steps 1-11.
13. Anchor the strain gauge wires along the rebar with zip-ties to ensure the gauge does not get ripped off during the concrete pour, while still allowing for maximum bonding between rebar and concrete.
14. Using a small amount of the TM sealant tape seal off the strain gauge.
15. Secure the wire along the rebar up and out of the panel. Secure the wire outside of formwork for minimal impact during the pour, while still protecting the wire during finishing. See Figures 3-8 and 3-9 for an example of how to secure the wire for the pour.

See Appendix B for examples of all site checklists and procedures. Figures 3-7 and 3-8 show an example of what you may expect to see in the field during installation and taping the strain gauges.

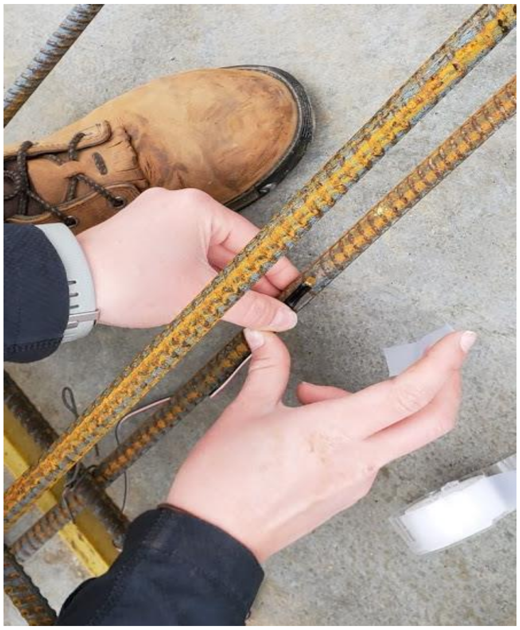


Figure 3-8: Strain Gauge Aligned on Rebar

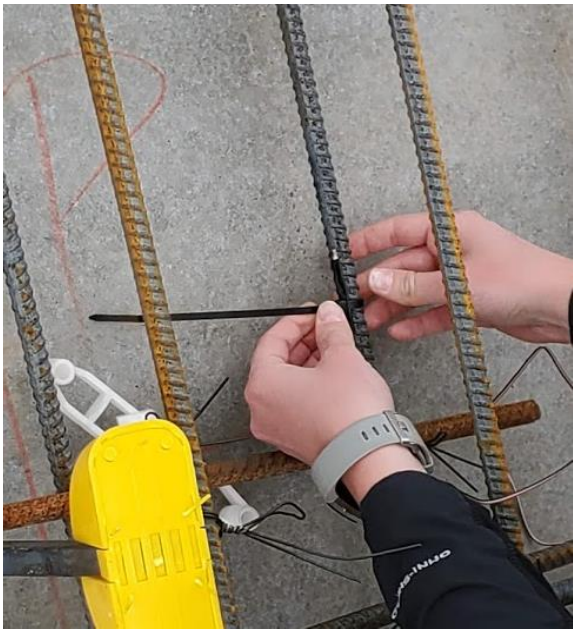


Figure 3-9: Strain Gauge After Being Adhered

3.2.2.2 Concrete Pour and General Tests

While the general contractor will have an independent group collect test samples during the concrete pours for flexural and cylinder tests, it is best if the research team conducts their own test specimens from the concrete trucks to ensure verify strength readings at the time of the lifts. While the research team can request that data through the general contractor, it is recommended that the research team collect a few cylinders and beams as well. Having compression strength and flexural (tension strength/modulus of rupture) readings for the concrete within a 24-hour window of the lift is ideal.

The researcher will have to work with their local lab, local independent, or a university lab if the project is more than a four-hour drive from the job site. This can impact if the research team will be able to break the cylinders and beams the day before, of, or after the lift. The cylinders will be used to determine the compressive strength of the concrete following the procedures defined by ASTM C39. Beam tests will determine the concrete flexural strength (tension strength/modulus of rupture), via three-point loading per ASTM C78. See Section 3.3 for more a detailed description of each of the test procedures.

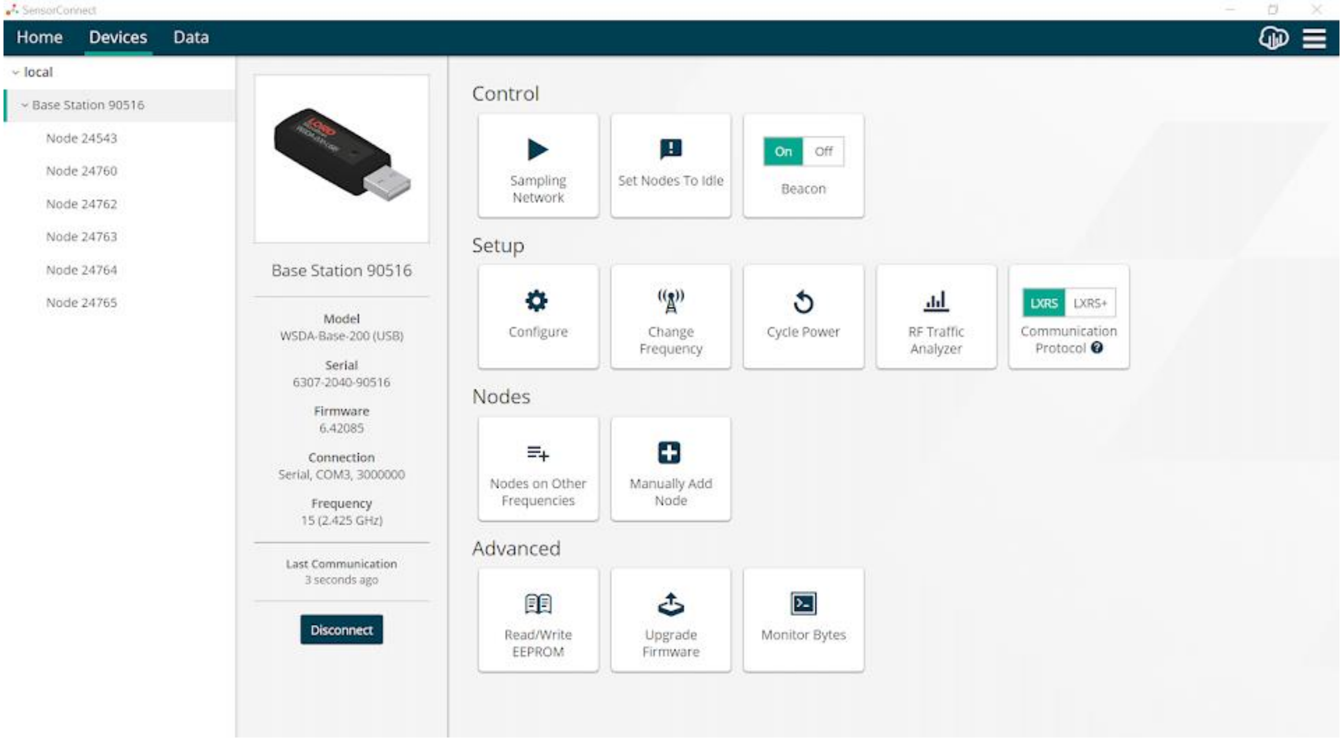
During the concrete pour, the researcher should make note of any inconsistences or issues with the instrumented panels. Additionally, the researcher should stay until the concrete finish crews are done with the panels to make note of any inconsistencies or issues with the panels or rebar strain gauge wires. The strain gauge wires for the reinforcing steel strain gauges can be protected from the concrete pour by poking it through foam. This reduces the chances of the wires getting cut during the finishing process. An example is shown in Figures 3-10 and 3-11.

Figure 3-10: Wires Protected in Foam

Figure 3-11: Rebar Wiring Protection

3.2.2.3 Wireless Node Setup – Pre-Site Visit

Prior to visiting the site for wireless node installation and preferably before any site visit for rebar strain gauge installation the wireless nodes need to be configured to insure proper channels are set-up for each strain gauges that will be connected to a node. This is done from the base station, using LORD Strain's Sensor Connect software. Once configured, the channels will maintain their configuration after they are turned off. When the Sensor Connect is opened, a blank screen will appear and when the base station is connected you can select the base station by reference number. As seen in Figures 3-12, the home page base station gives several options for actions, the wireless node homepages are very similar. "Sampling Network" turns the network on and starts a running collection of data. In order for this to run the Beacon needs to be turned on and the nodes set to idle (all which is done from the top row of action items). The



Base Station 90516

Model
WSDA-Base-200 (USB)

Serial
6307-2040-90516

Firmware
6.42085

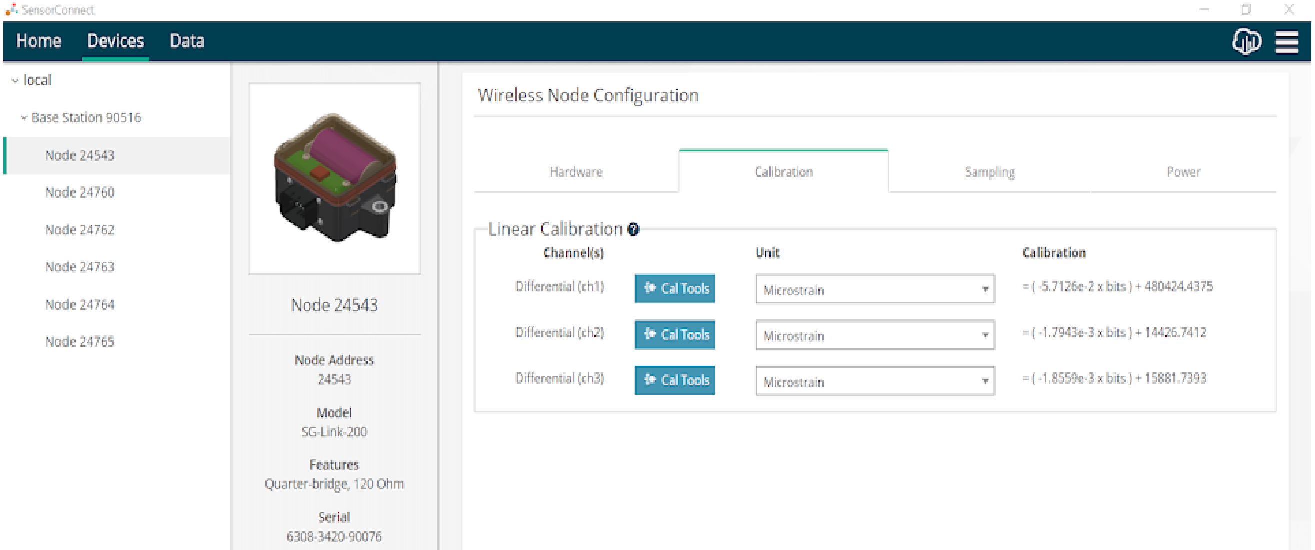
Connection
Serial, COM3, 3000000

Frequency
15 (2.425 GHz)

Last Communication
3 seconds ago

Figure 3-12: Base Station Homepage in Sensor Connect second row of commands, should be used during the calibration and configuration process for the strain gauges and wireless nodes prior to the lift date.

From the wireless node homepage, you can configure the node based on the strain gauges and calibration as mentioned previously. Four different tabs need to be modified: Hardware, Calibration, Sampling, and Power. For the most part, the last three tabs will need modification based on each individual node. In the calibration tab, shown in Figure 3-13, each channel can be calibrated individually based on the strain gauges connected on that channel, with a maximum of three gauges. This will vary with the strain gauge type, resistance and shunt. Shunt calibration is an indirect method of "verifying or setting the output of a strain gage instrument relative to a predetermined mechanical input" (Vishay, 2013). According to a Technical Note issued by Vishay Precision Group (part of Micro-Measurements), either can be used for 'instrument scaling' or 'instrument verification'. For the purpose of this research, generally instrument scaling is needed. Instrument scaling is achieved by adjusting the gage-factor control of the instrument. This procedure can also be applied in the adjustment of a single gauge versus



Wireless Node Configuration

Node 24543

Node Address
24543

Model
SG-Link-200

Features
Quarter-bridge, 120 Ohm

Serial
6308-3420-90076

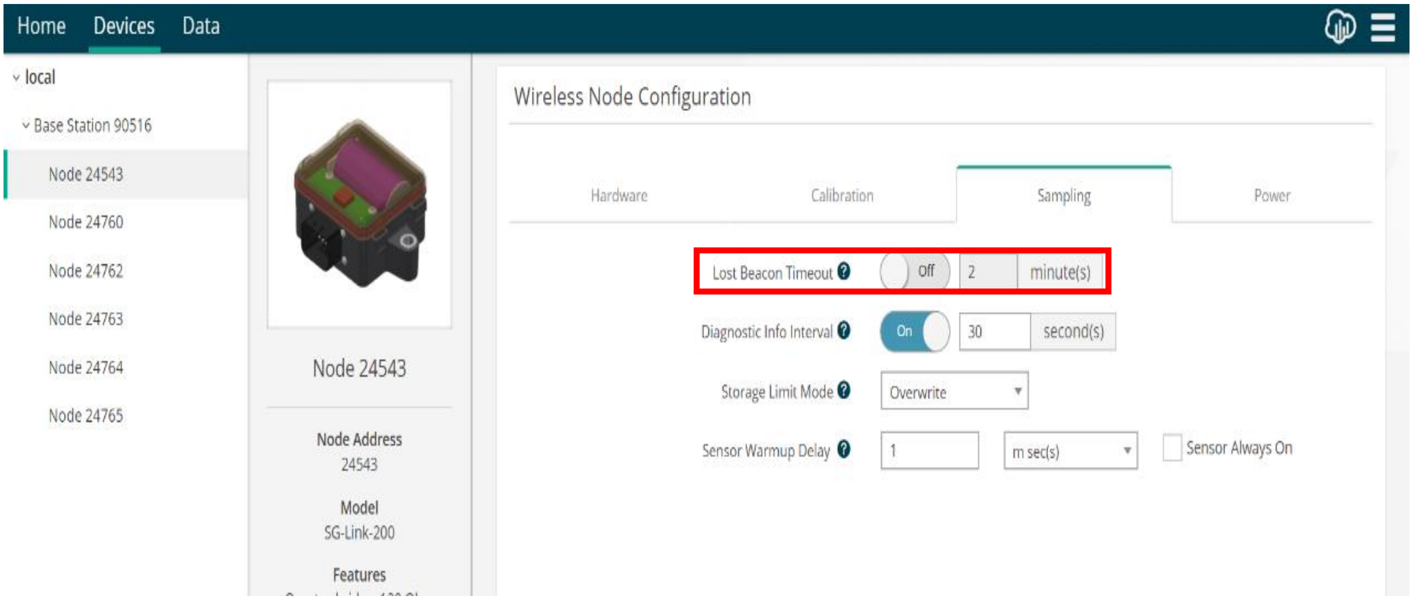
Linear Calibration

Channel(s)		Unit	Calibration
Differential (ch1)	Cal Tools	Microstrain	= (-5.7126e-2 x bits) + 480424.4375
Differential (ch2)	Cal Tools	Microstrain	= (-1.7943e-3 x bits) + 14426.7412
Differential (ch3)	Cal Tools	Microstrain	= (-1.8559e-3 x bits) + 15881.7393

Figure 3-13: Wireless Node Calibration Tab

multiple gauges at a time if a half-bridge strain gauge configuration is used by the research team. For compressive strains, shunting generally results in a corresponding decrease in arm resistance, which references the branch of the circuit that is being shunted (Vishay, 2013). Additionally, in quarter-bridge configuration the strain gauge and instrumentation can be scaled up or down to compensate for lead wire resistance to improve the accuracy of the readings. At high strain measurements, an additional consideration for non-linearity needs to be considered. The gauge and instrumentation need to be calibrated using the “simulated strain” which is the applied values/strain to the input terminals to calibrate the equipment (Vishay, 2013).

From the Sampling tab data storage and Lost Beacon Timout can be controlled. After conversation with the LORD Strain representatives, it was learned that this the lost beacon timeout control needs to be turned off. The storage mode for the nodes can also be controlled from this tab and can be seen along with the Lost Beacon Timeout in Figure 3-14. By turning this control off during the lift, the node will not automatically turn itself off if it loses network



Node Address
24543

Model
SG-Link-200

Features

Lost Beacon Timeout: Off, 2 minute(s)

Diagnostic Info Interval: On, 30 second(s)

Storage Limit Mode: Overwrite

Sensor Warmup Delay: 1 m sec(s)

☐ Sensor Always On

Figure 3-14: Wireless Node Sampling Tab/Lost Beacon Timeout Control communication/connection with the base station, which helps ensure that the data is collected throughout the entire panel lift.

Once all the nodes and channels are configured, set up a dashboard with each of the channels. Several widgets allow the researcher/team to set-up visual graphs to track the data during the lift. These should be set-up in the dashboard prior to lift day.

3.2.2.4 Concrete Surface Gauge Install & Wireless Node Install

Installation of the surface strain gauges and wireless nodes can be completed in the same trip to the site, or can be broken into two trips. Proper install might vary among manufacturers; therefore, confirmation of this procedure with the manufacturer once gauges are selected is required. Concrete surface strain gauge install is separated into two parts: (1) surface preparation and (2) gauge install (*CEA strain gage installation with M-bond 200 adhesive (training video).* 2019). For proper surface preparation a smooth surface in the area where data collection is required. If the panel was finished to industry standards, the concrete panel surface does not need to be grinded smooth. The steps for strain gauge installation are:

1. Double check the strain gauge is working with a voltmeter.
2. Ensure the two-lead wire are not touching, using a pair of tweezers helps.
3. Seal the separated lead wires with electrical tape.
4. Align the gauge in the correct location. Place electrical tape along the wires leading up to the strain gauge for two to three inches. Place a strip of tape perpendicular to the wires to anchor the wires in place.
5. Use clear tape to anchor the gauge in the correct location and direction along the panel.

6. Lift the clear tape at a 45° angle, lifting the gauge completely off the rebar but leaving some of the tape still attached at the same end of the gauge as the soldered connections.
7. Apply required adhesive per the manufacturer’s instructions.
8. Slowly reapply the gauge with the manufacturer’s adhesive, ensuring the gauge maintains the correct position on the concrete. Gradually push from the anchored end to the opposite end to remove an air bubbles under the gauge.
9. Hold the gauge in place with a firm pressure for the required amount of time from the manufacturer. (Approximately a minute and a half.)
10. Remove the scotch tape slowly, again at a 45° angle, constantly checking to make sure the gauge does not come up with the tape.
11. Check strain gauge again with voltmeter. If a reading no longer exists, remove gauge and reinstall a new gauge, following steps 1-11.
12. Anchor the strain gauge wires along the rebar with zip-ties to ensure the gauge does not get ripped off during the concrete pour, while still allowing for maximum bonding between rebar and concrete.
13. Using a small amount of the TM sealant tape seal off the strain gauge.

See Appendix B for examples of all site checklists and procedures. Figures 3-15, 3-16 and 3-17 show an example of the field during installation for surface mounted strain gauges. Wireless nodes are anchored to the panel surface with Fastenal screws.

Figure 3-15: Installed Surface Strain Gauge

Figure 3-16: Installed Surface Strain Gauge

Figure 3-17: Surface Gauges & Wireless Node Attached to Panel

3.2.2.5 Panel Lift Day

On the day of the lift confirm with the general contractor what time the researcher/team should arrive on-site to arrive with enough time prior to the lift to turn on the nodes and prepare to sample data. It is important for the researcher to be in clear communication with their on-site contact in order to ensure the testing goes smoothly.

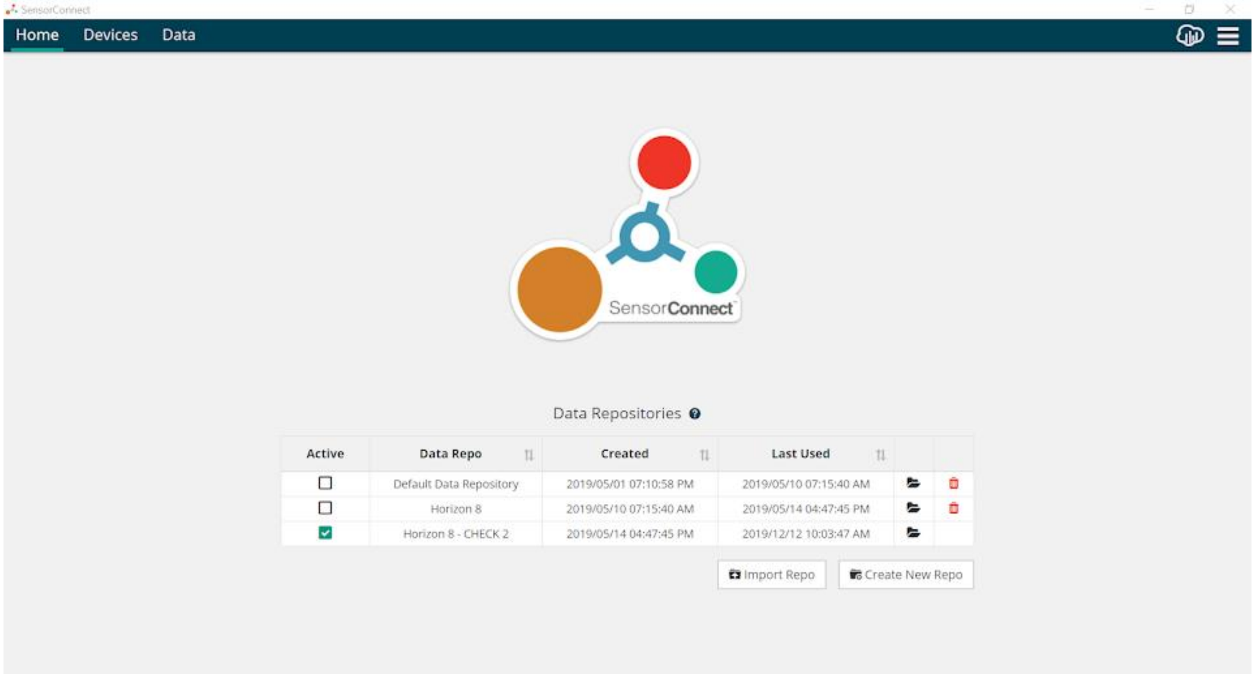
Prior to arrival the general contractor should be informed of how close the researcher and/or base station need to be in reference to the panel for data collection. Once the base-station is in place, the researcher can monitor the nodes during the lift. Double check the wireless nodes are configured to have Lost Beacon Timeout turned off. This will ensure the nodes continue to collect data even if something strange happens and issues with the gateway and base-station occur (Benton & Deering, 2019). Figure 3-18 indicates a base station and laptop set-up.

Figure 3-18: Base Station & Laptop Set-Up on Site

3.2.3 Post-Lift

After the panel lifts are complete, work with the general contractor to remove the wireless nodes from the panel(s). The sensors and wires can be cut, but the nodes need to be retrieved for data collection. The two main tasks post-lift are; (1) data collection/analysis and (2) general checks. General checks were discussed further in Section 3.2.

Data collected can be downloaded off the base-station using the Sensor Connect software. Depending on the widgets used during testing, the data can be downloaded via a .csv file or in excel in the table or graph format seen on the dashboard. The nodes download information by channel, which is why it is key to label the nodes during install and to have record of which sensor is on each channel of the wireless nodes. The data collected can be downloaded from the Sensor Connect Data Repositories page within Sensor Connect as shown in Figure 3-19.



Data Repositories

Active	Data Repo	Created	Last Used		
☐	Default Data Repository	2019/05/01 07:10:58 PM	2019/05/10 07:15:40 AM		
☐	Horizon 8	2019/05/10 07:15:40 AM	2019/05/14 04:47:45 PM		
☑	Horizon 8 - CHECK 2	2019/05/14 04:47:45 PM	2019/12/12 10:03:47 AM		

Figure 3-19: Data Repositories Homepage

The organization of data will vary based on personal preferences of each researcher/team; however, the time the lift started and ended for each panel should be recorded while on site so that the data in the time stamp is what is closely evaluated. The recorded time will allow the researcher to pull the data from the collection later that only pertains to the lift of the panel. Once the strain for each sensor is captured during the lift, mechanics of materials can be applied to calculate the moments seen on both sides of the panel during erection. Additionally, the compressive and tensile stresses can be calculated on each face of the panel from the strains. These stresses will be used to determine the actual moments in the panel during lifting. A comparison with the calculated moments, stresses, and strains will give the researcher/team a better comprehensive understanding of the actual panel behavior during lifting.

3.3 Concrete Tests

Concrete tests for the purposes of this report categorized: (1) Testing During Concrete Pour and (2) Testing Within 24 Hours of Panel Erection. These tests are described in detail in this section. Since concrete is impacted by water content and humidity, the weather on site should be tracked. Weather can effect of concrete strength as it cures, a log should be kept of the weather from the day of the pour until the day of the lift. If multiple panels are being lifted the weather log should continue until the day of the final lift. For each day, this log should have the high and low temperatures, precipitation chance and amount, and the relative humidity at a minimum, which can be collected from a weather app or database. Additional parameters to track are the cloud coverage of the days (i.e. partly cloudy, sunny, isolated thunderstorms, etc.).

Abi-Nader in 2009 conducted a survey to determine general practices in tilt-up construction (which was discussed in more detail earlier). Different flexural tests were being conducted. Sixty-one percent conducted center-point loading beam tests (ASTM C293), while seventeen percent perform third-point loading beam tests (ASTM C78). Additionally, only thirty-three percent stored test specimens on site near the panels, and sixty-seven percent stored test specimens in a lab curing room prior to testing (Abi-Nader, 2009).

3.3.1 Testing and Sampling During Concrete Pour

The main test done during the concrete pour is a concrete slump test. An air content (or air entrainment) test can also be done at the discretion of the researcher/team. If the researcher/team is unable to conduct either of these tests, they should work with the general contractor to determine if they will be able to get this data from the independent testing group hired by the contractor to determine concrete strength prior to lifting and to make sure the concrete meets the specified material properties.

3.3.1.1 Concrete Slump Test

Concrete slump tests measure the consistency of fresh concrete before it sets and is performed to check the workability of fresh concrete. This test also helps ensure uniformity during the pour between multiple trucks. The slump is the distance the concrete settles after the cone is removed. This test is conducted per ASTM C143, which covers the methodology for determining the consistency and ductility of the concrete.

3.3.2 Testing Within 24 Hours of Panel Erection

The following three tests are procured on site in compliance with ASTM C31 for making test specimens in the field. The concrete test specimens will all be broken within 24 hours prior to or post panel erection. Basic test procedure for each of the three tests – concrete beam, concrete cylinder, and reinforcing steel – are described in more detail in this section. It is crucial that both the cylinders and beams are left on-site for at least 8 hours for initial curing before they are transported back to the lab and curing room. The initial on-site curing, and transportation to the lab/curing room are outlined in ASTM C31. To meet the requirements for ASTM C31, the lab used by the researcher/team needs to be within a four-hour drive of the jobsite.

3.3.2.1 Concrete Beam Test

Abi-Nader in 2009 conducted a survey to determine general practices in tilt-up construction (which was discussed in more detail earlier). Different flexural tests were being conducted. Sixty-one percent conducted center-point loading beam tests (ASTM C293), while seventeen percent perform third-point loading beam tests (ASTM C78). Additionally, only thirty-three percent stored test specimens on site near the panels, and sixty-seven percent stored test specimens in a lab curing room prior to testing (Abi-Nader, 2009).

A concrete beam test is used to determine the tensile strength as previously discussed of the concrete. For the purposes of this research a center-point loading test procedure was followed in compliance with ASTM C293. In a survey conducted by Abi-Nader, (as introduced in the Chapter 2) the most common flexural test among contractors (61 percent of respondents) is the center-point loading even though this test generally over-estimates flexural strength by approximately 20 percent (Abi-Nader, 2009). The over-estimation in flexural strength is due to shear forces present at the loading point as well as the bending. This is not preferred over the third point loading. The three-point bending flexural test provides more accurate material properties for the tensile capacity of the concrete and the modulus of rupture.

The test specimens are collected during the concrete pour. A minimum of two beam samples for each concrete truck used on the panel being instrumented is recommended. The forms for the beams should be coated in a solution to help break the concrete free from the form after it has cured (typically WD-40 is used). The equipment available to test the beams can govern the beam size. Communicating with the experiment set-up staff at the lab is crucial to determine if this applies. For the purpose of this mock research, a 0’-3”x0’-4”x0’-16-3/4” beam was used.

Testing equipment varies between laboratories. The mock research is based on the Civil Engineering Labs at Kansas State University. For the beam test, a Shimadzu, AG-IC 50kN, machine was used to break the beams. The cracks that begin to form at the bottom of the beam during the test are very subtle, and the failure of the beam is sudden. This failure type corresponds well with plain concrete material properties. Figure 3-20 shows an example of a beam test set-up at the start of the test.

Figure 3-20: Shimadzu Beam Test Set-Up

For each test specimen, the beam span should be consistent when loaded from the Shimadzu machine. Additionally, the rate of load should be consistently increased at the head of the test machine, which should be programed with this rate prior to testing. All tests should use the same loading rate for each test.

If the lab that the research team has access to is able to use third-point loading from ASTM C78, a more conservative value for the flexural strength can be obtained because shear at the loading point is not a component of the stresses evaluated during the test (Abi-Nader, 2009).

3.3.2.2 Concrete Cylinder Test

Compressive strength is tested for concrete based on ASTM C39. This test applies a constant compressive axial load to the cylinders to determine the average concrete compressive strength. The same concerns for transporting the test specimens apply to the cylinders as for the beam test.

Similar to the concrete beams, the test specimens are collected during the concrete pour. A minimum of three-cylinder samples for each concrete truck used on the panel(s) being instrumented is recommended. The forms for the cylinders should be coated in a solution to help break the concrete free from the form after it has cured (typically WD-40 is used). For the purpose of this mock research 4x8" cylinders were used. The test involves placing a cylinder in a compression machine shown in Figures 3-21 and 3-22. The cylinders should be loaded at a constant rate and the maximum load when the specimen breaks should be recorded.

Figure 3-21: Forney Compression Cylinder Test System

Figure 3-22: Unbroken Cylinder Test

The cylinders, for the mock research testing, were broken using the Forney cylinder compression machine. The manner a cylinder might fracture are shown in Figure 3-23, from ASTM C39. The main fracture mode for concrete cylinders is a cone, but a cylinder can also break in a cone/split, cone/shear, shear, or columnar pattern.

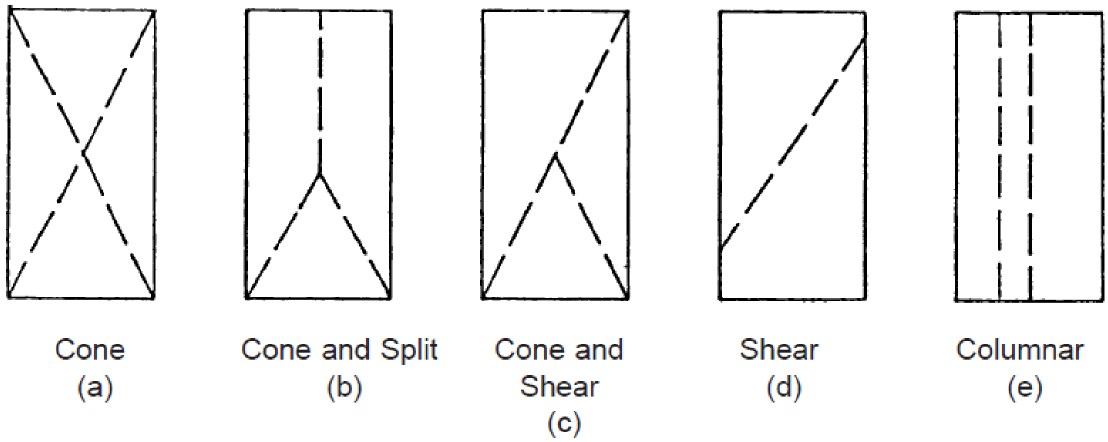


Figure 3-23: Sketches of Types of Fractures (Source: ASTM C39/C ©)

The most common failures of the cylinder are cone and cone and split. Columnar splitting generally indicates an issue with the test procedure or set-up. One cause of this could be due to lubricant or space between the cylinder and the cylinder cap, which reduces the lateral confining and results in the columnar break.

3.3.2.4 Reinforcing Steel Tensile Strength Test

To test the yield strength of the rebar used, the test sample of rebar is placed in a hydraulic wedge grip and slowly placed under constant increasing tension forces until the sample fails (ASTM A1035). The machine used in the mock research/testing was the MTS 647 Hydraulic Wedge Grip. For this particular machine a minimum test sample length was about 14" which can be communicated with the general contractor to allow the researcher/team to get the appropriate sample size and number for testing. The researcher/team should try to have two samples per panel instrumented to test for reinforcing steel tensile strength at minimum.

3.4 Results Comparison

Once the strain data is collected from the wireless nodes, using basic mechanics of materials the bending strain can be used to determine the maximum compressive and tensile moments in panel at those locations. Comparative analysis can be conducted for the expected design moments, determined using the analysis outlined in the previous section, and the calculated moments from the panel lift on-site.

3.4.1 Detailed Design Analysis

Another option for a detailed design check, is finite element analysis. As discussed in Chapter 2, a variety of finite element analysis software can be used with the discretion of the research team. However, a software that is specific for finite element modeling and analysis will produce more accurate results if modeled properly than using a 3D software such as RISA 3D.

The analysis in the above section was adequate for initial design checks and confirmation with the lift-insert designers for placement of strain gauges. Finite element analysis should result in a more accurate comparison to the actual performance of the panel during lifting. This is due to the increased inputs and grid projections in 3D layout of the panel. This becomes especially important at the lift insert points in the panel, due to the high concentration of stresses in the panel.

Finite element modeling requires the following six inputs for proper analysis: (1) nodal point locations, (2) elements connecting nodes, (3) mass parameters, (4) boundary conditions, (5) loads and/or forcing functions, and (6) analysis method. As outlined in Matthew Bono' thesis (discussed in Chapter 2), the boundary conditions will need to be applied to nodes. While in actual application the panel will have movement in all three-dimensional axes, again for computational purposes, certain z-axis rotations and translations should be locked (Bono, 2011). The elements connecting the nodes are also known as the mesh or meshing and the loads are applied here. For analysis of the panel during lift, only self -weight of the panel should be applied to the mesh as the loading. Depending on the research teams design philosophy, a forcing function could be applied at the lift points to find the panel performance or they could just be treated as supports. It is important to have a fine enough mesh to get accuracy in the analysis, without being so fine that it wastes time running computations that are very similar in value. Additionally, depending on the software selected the stiffness matrix of the element may need to be defined and can be calculated with typical structural analysis procedures.

When compiling the results from the finite element analysis, examining the elements at the lift points is important. Additional data should be recorded at the locations where the strain gauges are “located” within the panel. If the panels have any openings, high concentrations of strain could occur at the corners and warrant additional analysis.

Chapter 4 - Conclusion

This chapter will discuss lessons learned while developing the testing procedure. Specifically, this chapter focuses on the configuration and setup of the wireless test network for the nodes and gateway. Finally, this chapter covers future research recommendations.

4.1 Lessons Learned

This paper is a guide/procedure for future full-scale panel testing. This section discusses the lesson's learned for future full-scale testing.

Communication with the strain gauge manufactures and the wireless node manufactures, and the research team is crucial for the proper selection of the wireless nodes. The first step to selecting the wireless nodes should first be the selection of the reinforcing steel and surface mounted concrete strain gauges. The selection of the strain gauges is when the communication with the strain gauge manufacturer will be helpful. Making clear the governing parameters of the job-site set-up, such as panel size and the number of panels and strain gauges per panel, will help with selection of the wireless nodes once the strain gauges have been selected.

Since most manufacturers will not be familiar with the process of tilt-up, sending your manufacturer's representative a video of a tilt-up panel being erected can be a useful resource to describe the testing that the research team is trying to achieve. Since the wireless nodes will be left on the panel for at least a day prior to the lift, they should be rated for weather exposure.

Additionally, it should be noted that the mock research used as an example in this report have multiple locations for strain gauge placements. In future research, it may be deemed necessary to place more than one strain gauge in a general area where a maximum moment is expected to occur. The reason for this decision would be to increase the redundancy of the data.

This would help ensure that if a strain gauge failed for any reason then data would still be collected for that location/general area to be used for the determination of actual moments imposed on the panel.

4.1.1 Wireless Node Configuration

The strain gauges will have a direct impact on parts of the configuration of the wireless nodes and need to be considered. Strain gauge shunt calibration will vary with gauges and instrumentation as well as application. The strain gauge manufacturer's representative can give the research team specific help in this process for the gauges that are being used.

The wireless nodes used in this mock research were designed that the shunt calibration for the strain gauges and the nodes should be relatively easy using the calibration tab in the Sensor Connect Software.

Additionally, the wireless nodes must be configured to have the Lost Beacon Timeout turned off. This will ensure that the strain gauge data will still be collected by the wireless nodes during the lift even if a loss of communication between the wireless nodes and the wireless gateway or the wireless gateway and the base station occurs. While it seems unlikely that this should happen especially if the gateway and nodes were selected based on the safety radius of the lift, issues on lift day that could require the research team to move further away from the panel during the lift may occur.

4.1.2 Wireless Gateway Selection

The wireless gateway selection is selected based on the project parameters and the number of wireless nodes transmitting at one time. Therefore, the wireless gateways need to be selected after the strain gauges and wireless nodes have been selected.

Additional consideration for the wireless gateway selection is the distance the signal from the wireless nodes need to be transmitted. If the general contractor agrees that the research team can be within a close enough proximity to the panel lift(s), then the gateway and base station could be the same piece of equipment. However, if a longer transmission is required, or will need to be transmitted around concrete walls that are already constructed then a separate wireless gateway and base station should be evaluated. Again, clear communication with the wireless node and gateway manufacturer can help with the proper wireless gateway selection.

4.1.3 Concrete Field Test Specimens

Depending on the number of panels that are being instrumented will affect how many concrete trucks are used to pour the panel(s) being tested by the research team. Even a single panel could have more than one truck of concrete. At least two, preferably three, compressive cylinders and flexural beams from each concrete truck should be collected. Test specimens will need to be cured in a concrete lab that is within a four-hour drive from the job site. Therefore, an agreement with a testing lab is required prior to the concrete pour.

All samples need to remain on-site for at least 24 hours before being moved back to the curing room in the lab used by the research team. Communication with the general contractor will ensure that the test specimens can be safely left on-site until the following day. For transportation back to the lab after the 24 hours, the test specimens can be placed in styrofoam or some other form of padding to keep them from being rattled for the drive.

4.1.4 Concrete Testing

For the compressive cylinder and flexural beam tests outlined in Chapter 4, communication with the test coordinator for the lab being used by the research team is crucial.

Most labs that have done concrete testing should have all of the materials to set up the proper tests, but may need time to pull them out of storage or from another test.

Additionally, it is important to confirm with the testing coordinator that the flexural test and compressive test are set up per the ASTM's the day before the specimens need to be broken. At this time, it is also helpful to determine if the testing coordinator will be present when the researcher/team is testing specimens. If the testing coordinator cannot be present, coordinate that the lab space will be unlocked at the time of testing and ensure that the researcher/team has a complete understanding of how the equipment in that lab works and that all safety requirements are understood.

4.2 Best Practices for Testing

The guidelines created for testing procedure in Chapter 4 were established based on industry standards for testing through ASTM specifications in addition to established knowledge and practices in the labs and professors at Kansas State University. Any future research should follow the most current ASTM specifications and standards.

4.3 Research Recommendations

Design calculations and scaled lab testing show us that tilt-up design is accurate in application. However, these studies are limited by the assumptions that are made based on scaled models to simplify the design and analysis procedure. Further analysis and comparison of design and actual stresses during erection need to be conducted on full size panels out in the field. Specifically, field testing will produce more accurate data for flexural stresses because of unexpected loading a panel can result in due to an uncontrolled environment.

Additionally, past research on tilt-up panels is becoming less relevant as panel design slims and panels are designed as slender members and cracking during lift is expected. Using the test methodology proposed in this report, panels can continually be tested as design has evolved to today's slender elements, and into the future as tilt-up concrete design continues to evolve. In line with this potential to evaluate panels as design changes, the test methodology proposed in this paper could be modified in future research along with the gauges within/on the test panels and could continue to be used to evaluate structure performance during natural events over the span of the building's life.

The testing procedure outlined in this paper can be modified to include a digital image correlation field to determine the surface strains. This method utilizes a camera focused on the surface to capture the full strain field of the concrete. If this method is utilized with this testing procedure, it is recommended that a trial run be done in a laboratory setting to help identify and mitigate constructability and lifting issues with multiple cameras fixed to the panel surface. The reason this was not included in the body of this report, is because the strains on the bottom surface of the panel would still require strain gauges to capture the data during lift.

Once enough testing of full-scale tilt-up panels has been conducted to produce a baseline for solid panels, this test methodology can be applied to evaluate other aspects of tilt-up construction. More detailed testing of the lift inserts specifically within the panels will help determine the variations and concentrations in flexural stresses that occur at these locations during lift. Additionally, performance of panels with larger or multiple openings should be the following research to evaluate the resulting stresses around these openings and at the corners to be compared with solid panels, and current design assumptions for panels with such openings.

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Appendix A - Example Packet to Propose Research & Receive Permissions from GC

HORIZON 8

Riverside, MO

LITHKO – Meadow Burke
Thesis Research - Benton
Spring 2019

RESEARCH GOALS

Some theoretical and lab controlled research regarding the behavior of tilt-up concrete panels in the vertical position has been conducted. (ACI SEACO, 1982) Numerous patents for lifting devices for tilt-up panels exist. The panel lifting hardware are checked for the highest tension when the panel is horizontal, and the highest shear when the panels are in the vertical position. This proprietary lifting hardware is typically tested by pull tests and shear tests on smaller specimens compared to an actual tilt-up panel.

However, little to no research on how full-size, tilt-up panels perform during the lifting/construction process exists. For the HORIZON 8 project, this research is proposing placing resistance strain gauges on the reinforcing steel and surface of the concrete panels in the locations where the highest moments are expected during the pick process to determine if the panel cracks.

The proposed strain gauge locations have be determined by the panel lifting insert pattern and basic static analysis, to ensure the largest moments are captured by the gauges. The proposed locations have all been check with Meadowburke to further ensure no moments are missed.

The main objectives of this research study are:

i. To quantify the actual stress variation in tilt-up panels during the lifting process.
ii. To assess the accuracy of current methodologies.
iii. To understand degree of cracking that occurs during the lifting and the subsequent behavior of the reinforcing steel.
iv. To examine and possibly develop relevant combined tension and shear criteria for lifting inserts which may reduce the embedded conservatism, factor of safety.
v. To improve construction practices by possible allowing reorientation of panels after initial lift.

EXPERIMENTAL PROGRAM

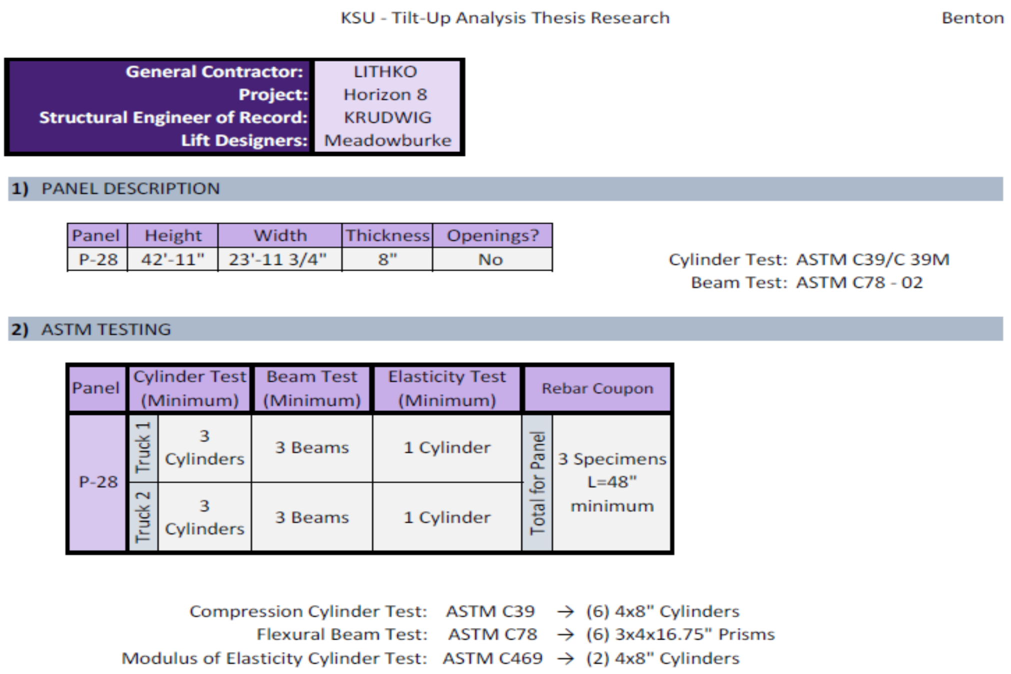
1. Strain Gauges

Since a panel is lifted from zero to 90 degrees in tilt-up construction, the strain gauges are capable of determining dynamic strains will be used. The strain gauges will have a wired connection to transmitter nodes at the panel. These transmitter nodes will transmit the data gathered from the gauges via radio frequency to a central supervisor, which will be plugged into a laptop of one of the researchers.

Surface mounted strain gauges will be implemented in multiple locations along the panel to allow for data collection without affecting the architectural features of the panel or structural integrity. The reinforcing steel strain gauges will provide data on the behavior of the concrete towards the panel rebar. The gauges will provide direct data on the behavior of the steel throughout the lifting process and the subsequent forces the panel is experiencing.

2. Testing

The 3 potential panels for instrumentation are all solid panels. The rebar instrumentation will be attached to the reinforcing steel prior to placement of concrete. After the panels compressive strength of concrete has been reached and the forms are stripped, the panels will be cleaned of debris prior to the placement of the surface mounted instrumentation. During the lifting procedure, constant readings will be taken as the panel is pried from the casting slab in the horizontal position to the vertical position and placed into the final position for building construction.



KSU - Tilt-Up Analysis Thesis Research Benton

General Contractor:	LITHKO
Project:	Horizon 8
Structural Engineer of Record:	KRUDWIG
Lift Designers:	Meadowburke

1) PANEL DESCRIPTION

Panel	Height	Width	Thickness	Openings?
P-28	42'-11"	23'-11 3/4"	8"	No

Cylinder Test: ASTM C39/C 39M
Beam Test: ASTM C78 - 02

2) ASTM TESTING

Panel		Cylinder Test (Minimum)	Beam Test (Minimum)	Elasticity Test (Minimum)		Rebar Coupon
P-28	Truck 1	3 Cylinders	3 Beams	1 Cylinder	Total for Panel	3 Specimens L=48" minimum
	Truck 2	3 Cylinders	3 Beams	1 Cylinder		

Compression Cylinder Test: ASTM C39 → (6) 4x8" Cylinders
Flexural Beam Test: ASTM C78 → (6) 3x4x16.75" Prisms
Modulus of Elasticity Cylinder Test: ASTM C469 → (2) 4x8" Cylinders

General Contractor:	LITHKO
Project:	Horizon 8
Structural Engineer of Record:	KRUDWIG
Lift Designers:	Meadowburke

3) PANEL INSTRUMENTATION

Panel	Height	Width	Thickness	Openings?
P-28	42'-11"	23'-11 3/4"	8"	No

Pick Configuration: 4 High x 2 Wide

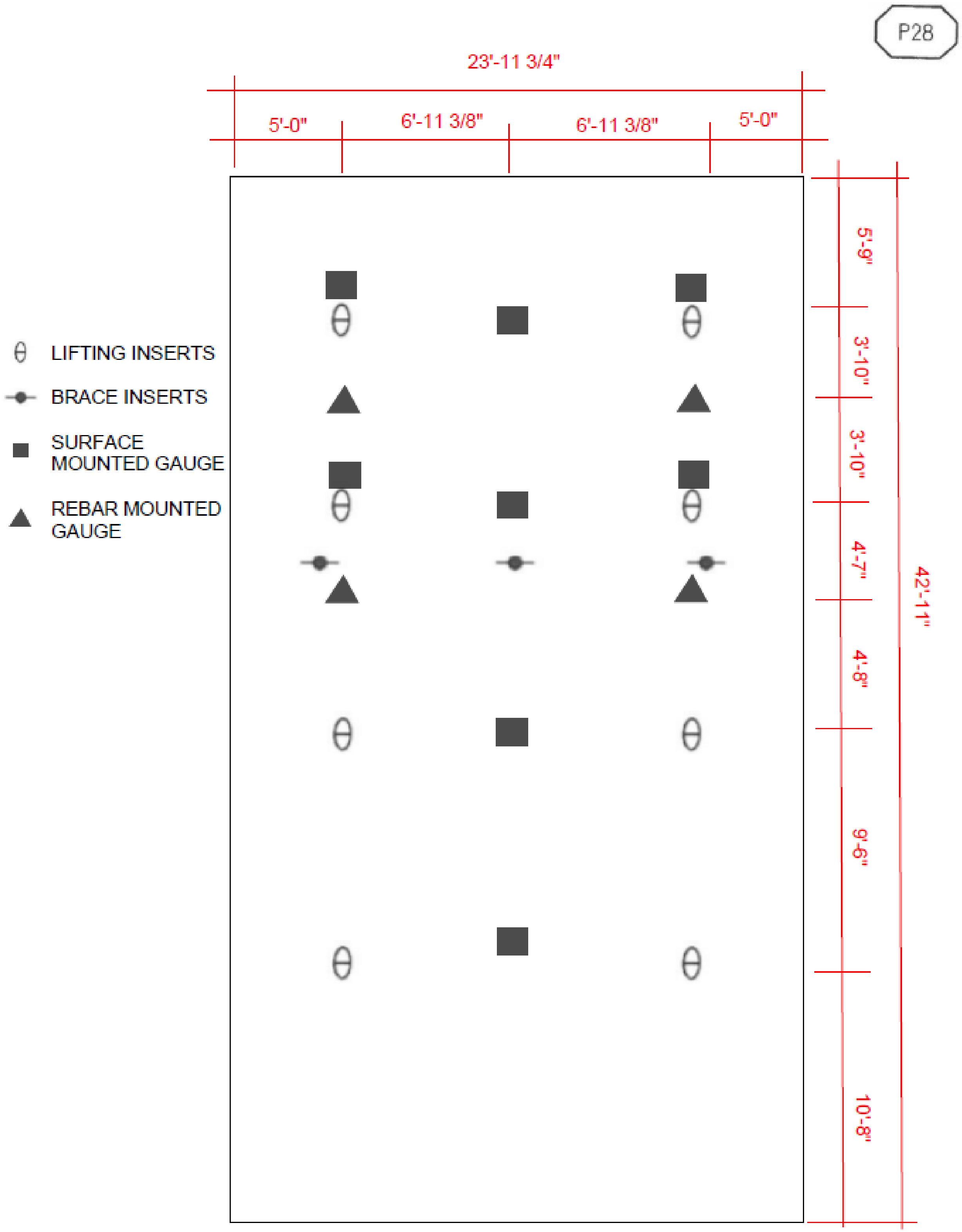
→ Surface Mounted Strain Gauge:	CEA-00-250UW-120/P2
→ Rebar Mounted Strain Gauge:	C2A-06-062LW-350/P

Connector cables from each strain gauge (both surface and rebar mounted) to the SG-Link 200 will be a maximum of 1/4" in diameter.

As shown in the above image, the rebar mounted strain gauge wires will need to be brought to the surface of the panel to connect to the SG-Link 200 OEM transmitter.

*The gauge will not be taped down as shown in this image and will fit between the rebar deformations. The wire can be zip tied to the rebar until they are brought to the surface.

- → SG-LINK 200 OEM
- → WSDA 200 USB


P28
23'-11 3/4"
5'-0"
6'-11 3/8"
6'-11 3/8"
5'-0"
5'-9"
3'-10"
3'-10"
4'-7"
4'-8"
9'-6"
10'-8"
42'-11"
LIFTING INSERTS
BRACE INSERTS
SURFACE MOUNTED GAUGE
REBAR MOUNTED GAUGE

General Contractor:	LITHKO
Project:	Horizon 8
Structural Engineer of Record:	KRUDWIG
Lift Designers:	Meadowburke

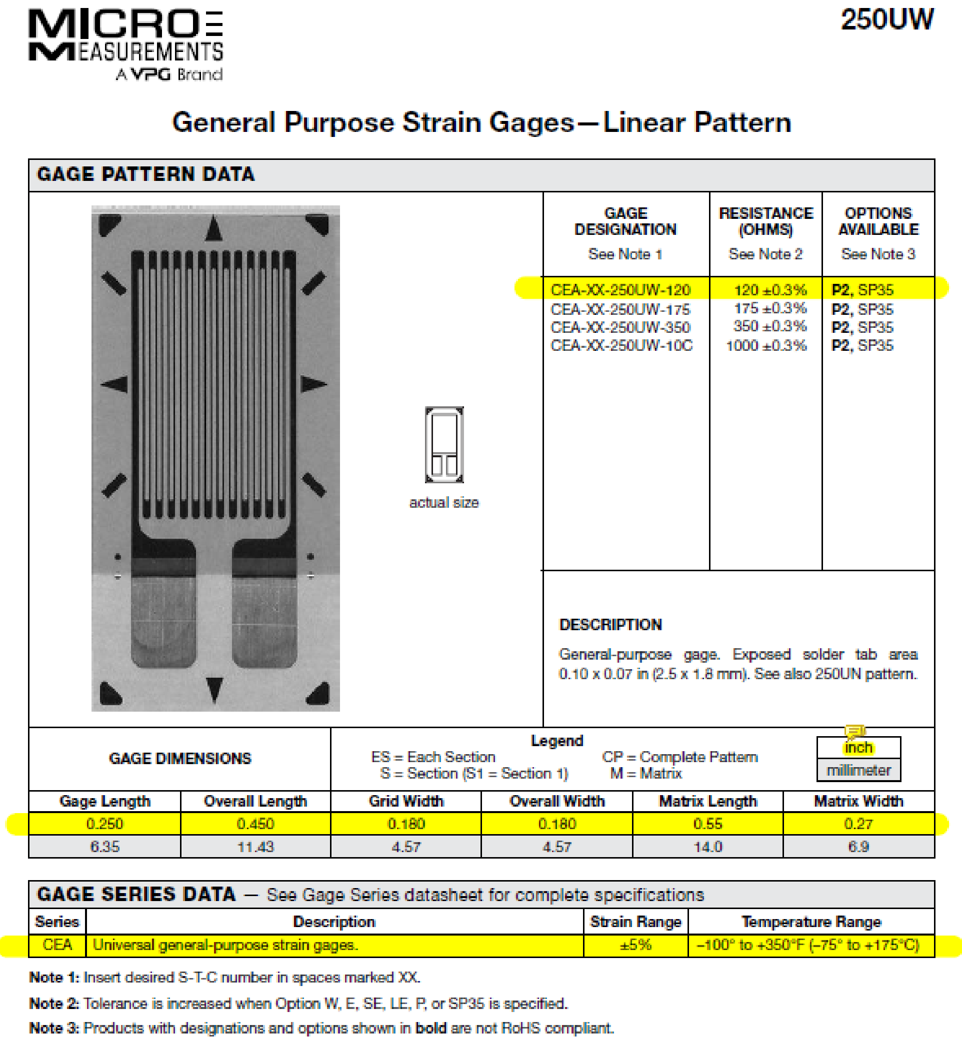
4) SENSOR INFORMATION

MicroMeasurements:

- For weathering protection the strain gauges can be coated in the protective coating M-COAT JA.
- Any surface mounted gauges that are lower than 15'0" on the panel can be removed after erection using acetone.

LORD Sensing:

- The LORD Sensing SG-Link and WSDA 200 USB transmitter and reciever connect using radio waves. Therefore, no internet connection should be required by the researchers on site.
- Each panel will have a minimum of four (4) SG-Link 200 OEM gateway/transmitters connected to the gauges. The SG-Link 200 OEM is 2" x 1.5" x 0.5" and can be attached to the panel with screws or an adhesive. The research team can work with the LITHKO to determine the most viable connection to the panel.



MICRO MEASUREMENTS A VPG Brand

250UW

General Purpose Strain Gages—Linear Pattern

GAGE PATTERN DATA

GAGE DESIGNATION See Note 1	RESISTANCE (OHMS) See Note 2	OPTIONS AVAILABLE See Note 3
CEA-XX-250UW-120	120 ±0.3%	**P2**, SP35
CEA-XX-250UW-175	175 ±0.3%	**P2**, SP35
CEA-XX-250UW-350	350 ±0.3%	**P2**, SP35
CEA-XX-250UW-10C	1000 ±0.3%	**P2**, SP35

DESCRIPTION

General-purpose gage. Exposed solder tab area 0.10 x 0.07 in (2.5 x 1.8 mm). See also 250UN pattern.

GAGE DIMENSIONS

Legend
ES = Each Section
S = Section (S1 = Section 1)
CP = Complete Pattern
M = Matrix

inch
millimeter

Gage Length	Overall Length	Grid Width	Overall Width	Matrix Length	Matrix Width
0.250	0.450	0.180	0.180	0.55	0.27
6.35	11.43	4.57	4.57	14.0	6.9

GAGE SERIES DATA — See Gage Series datasheet for complete specifications

Series	Description	Strain Range	Temperature Range
CEA	Universal general-purpose strain gages.	±5%	–100° to +350°F (–75° to +175°C)

Note 1: Insert desired S-T-C number in spaces marked XX.

Note 2: Tolerance is increased when Option W, E, SE, LE, P, or SP35 is specified.

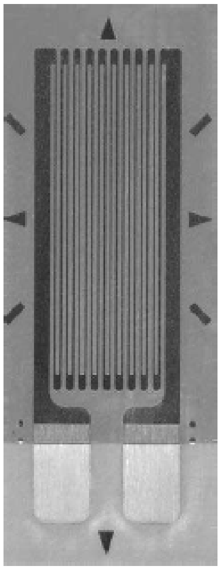
Note 3: Products with designations and options shown in **bold** are not RoHS compliant.

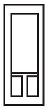
375UW

MICRO MEASUREMENTS
A VPG Brand

General Purpose Strain Gages—Linear Pattern

GAGE PATTERN DATA





actual size

GAGE DESIGNATION See Note 1	RESISTANCE (OHMS) See Note 2	OPTIONS AVAILABLE See Note 3
CEA-XX-375UW-120	120 ±0.3%	**P2**, SP35
CEA-XX-375UW-350	350 ±0.3%	**P2**, SP35

DESCRIPTION

General-purpose gage. Exposed solder tab area 0.10 x 0.07 in (2.5 x 1.8 mm).

GAGE DIMENSIONS

Legend

ES = Each Section
S = Section (S1 = Section 1)
CP = Complete Pattern
M = Matrix

inch
millimeter

Gage Length	Overall Length	Grid Width	Overall Width	Matrix Length	Matrix Width
0.375	0.575	0.180	0.180	0.67	0.27
9.53	14.61	4.57	4.57	17.0	6.9

GAGE SERIES DATA — See Gage Series datasheet for complete specifications

Series	Description	Strain Range	Temperature Range
CEA	Universal general-purpose strain gages.	±5%	–100° to +350°F (–75° to +175°C)

Note 1: Insert desired S-T-C number in spaces marked XX.

Note 2: Tolerance is increased when Option W, E, SE, LE, P, or SP35 is specified.

Note 3: Products with designations and options shown in **bold** are not RoHS compliant.

LORD DATASHEET

WSDA®-200-USB

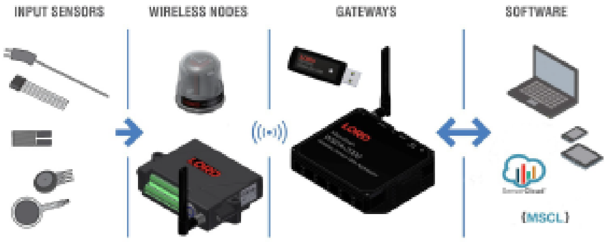
Wireless USB Gateway

WSDA-200-USB - *USB gateway for configuration and data collection*

LORD Sensing Wireless Sensor Networks enable simultaneous, high-speed sensing and data aggregation from scalable sensor networks. Our wireless sensing systems are ideal for test and measurement, remote monitoring, system performance analysis, and embedded applications.

The gateways are the heart of the LORD Sensing wireless sensing system. They coordinate and maintain wireless transmissions across a network of distributed wireless sensor nodes. The LORD Sensing LXRS and LXRS+ wireless communication protocols between compatible nodes and gateways enable high-speed, synchronized sampling and lossless data throughput at rates up to 16 ksps

Users can easily program nodes for continuous, periodic burst, or event-triggered sampling with the SensorConnect software. The optional web-based SensorCloud interface optimizes data aggregation, analysis, presentation, and alerts for sensor data from remote networks.



Product Highlights

- Data acquisition gateway collects synchronized data from scalable networks of wireless sensors
- Provides seamless communication between the wireless sensor nodes and host computer
- Quick deployment with host computer interface
- Compatible with LORD Sensing LXRS and LXRS+ sensor nodes

Features and Benefits

High Performance

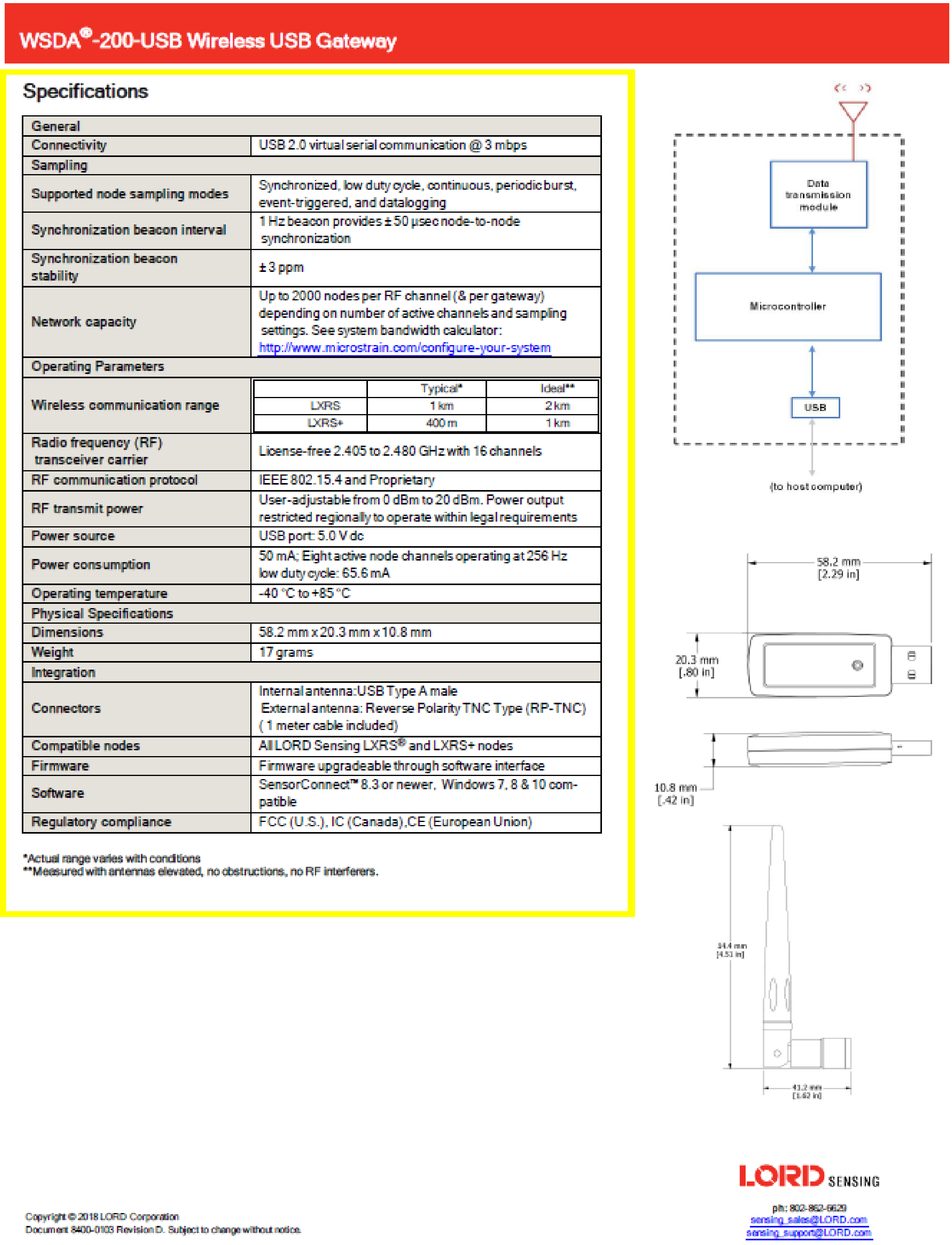
- Lossless data throughput and sampling of ±50 µS in LXRS+ and LXRS-enabled modes
- Wireless range up to 2 km (400 m typical)
- External antenna option for embedded applications or enhanced range

Ease of Use

- Easy out-of-the-box installation with data collection in minutes
- Scalable networks for easy expansion
- Remote configuration, acquisition, and display of sensor data with SensorConnect™
- Data visualization through web-based SensorCloud portal for quick data navigation and analysis
- Easy custom integration with open-source, comprehensive communications and command library (API)
- Hundreds of sensors managed from a single gateway

Applications

- Structural health monitoring
- Equipment performance monitoring, verification, evaluation, and diagnostics
- Test and measurement
- System control
- Environmental monitoring



WSDA®-200-USB Wireless USB Gateway

Specifications

General	
Connectivity	USB 2.0 virtual serial communication @ 3 mbps
Sampling	
Supported node sampling modes	Synchronized, low duty cycle, continuous, periodic burst, event-triggered, and datalogging
Synchronization beacon interval	1 Hz beacon provides ± 50 µsec node-to-node synchronization
Synchronization beacon stability	± 3 ppm
Network capacity	Up to 2000 nodes per RF channel (& per gateway) depending on number of active channels and sampling settings. See system bandwidth calculator: http://www.microstrain.com/configure-your-system
Operating Parameters	
Wireless communication range	 LXRS: Typical* 1 km, Ideal** 2 km LXRS+: Typical* 400 m, Ideal** 1 km
Radio frequency (RF) transceiver carrier	License-free 2.405 to 2.480 GHz with 16 channels
RF communication protocol	IEEE 802.15.4 and Proprietary
RF transmit power	User-adjustable from 0 dBm to 20 dBm. Power output restricted regionally to operate within legal requirements
Power source	USB port: 5.0 V dc
Power consumption	50 mA; Eight active node channels operating at 256 Hz low duty cycle: 65.6 mA
Operating temperature	-40 °C to +85 °C
Physical Specifications	
Dimensions	58.2 mm x 20.3 mm x 10.8 mm
Weight	17 grams
Integration	
Connectors	Internal antenna: USB Type A male External antenna: Reverse Polarity TNC Type (RP-TNC) (1 meter cable included)
Compatible nodes	All LORD Sensing LXRS® and LXRS+ nodes
Firmware	Firmware upgradeable through software interface
Software	SensorConnect™ 8.3 or newer, Windows 7, 8 & 10 compatible
Regulatory compliance	FCC (U.S.), IC (Canada), CE (European Union)

Wireless communication range:

	Typical*	Ideal**
LXRS	1 km	2 km
LXRS+	400 m	1 km

*Actual range varies with conditions
**Measured with antennas elevated, no obstructions, no RF interferers.

LORD SENSING

ph: 802-862-6629
sensing_sales@LORD.com
sensing_support@LORD.com

Document 8400-0103 Revision D. Subject to change without notice.

LORD Sensing DATASHEET

SG-Link®-200-OEM

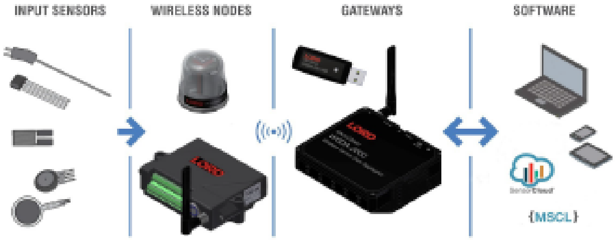
Wireless Analog Input Node

SG-Link®-200-OEM - small, low-cost two-channel analog sensor node ready for OEM integration

LORD Sensing Wireless Sensor Networks enable simultaneous, high-speed sensing and data aggregation from scalable sensor networks. Our wireless sensing systems are ideal for test and measurement, remote monitoring, system performance analysis, and embedded applications.

The SG-Link®-200-OEM allows users to remotely collect data from a range of sensor types, including strain gauges, pressure transducers, and accelerometers. The node supports high resolution, low noise data collection from 1 differential and 1 single-ended input channels at sample rates up to 1 kHz. A digital input allows compatibility with a hall effect sensor for reporting RPM and total pulses, making the sensor ideal for many torque sensing applications.

Users can easily program nodes for continuous, periodic burst, or event-triggered sampling with the SensorConnect software. The optional web-based SensorCloud interface optimizes data aggregation, analysis, presentation, and alerts for sensor data from remote networks.



Product Highlights

- 1 differential and 1 single-ended input channel
- Differential channel compatible with 120, 350, and 1k Ohm Wheatstone bridge sensing circuits
- On-board temperature sensor
- Digital input channel for RPM and pulse counting
- Supply power from 3.3 to 30 V
- Continuous, periodic burst, and event-triggered sampling
- Output raw data and/or derived channels such as mean, RMS and peak-peak
- LXRS protocol allows lossless data collection, scalable networks and node synchronization of ±50 µs
- Remote strain calibration using on-board shunt resistor

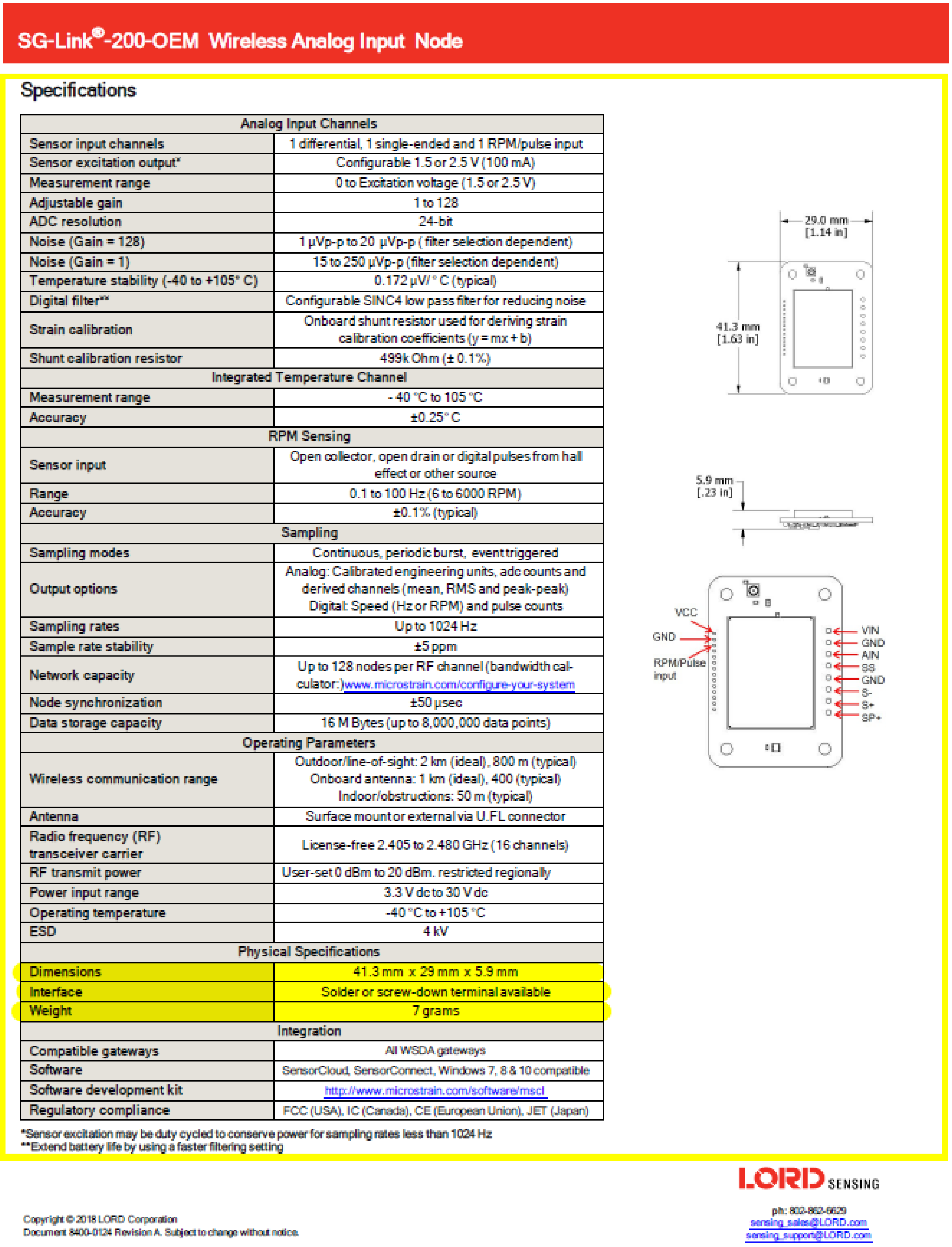
Features and Benefits

High Performance

- Up to 1024 Hz sampling
- Low noise 1.5 or 2.5 V sensor excitation
- Noise as low as 1 µV p-p
- High resolution 24-bit data
- Datalog up to 8 million data points
- Low power operation, well-suited for battery powered applications.
- Wireless range up to 1 km (400 m typical)
- -40 to +105° C operating temperature range

Applications

- Strain, load, force, pressure, acceleration, vibration, displacement, or torque sensing.
- Condition-based monitoring
- Structural load and stress monitoring
- Test and measurement
- RPM and Pulse counting



SG-Link®-200-OEM Wireless Analog Input Node

Specifications

Analog Input Channels	
Sensor input channels	1 differential, 1 single-ended and 1 RPM/pulse input
Sensor excitation output*	Configurable 1.5 or 2.5 V (100 mA)
Measurement range	0 to Excitation voltage (1.5 or 2.5 V)
Adjustable gain	1 to 128
ADC resolution	24-bit
Noise (Gain = 128)	1 µVp-p to 20 µVp-p (filter selection dependent)
Noise (Gain = 1)	15 to 250 µVp-p (filter selection dependent)
Temperature stability (-40 to +105° C)	0.172 µV/ ° C (typical)
Digital filter**	Configurable SINC4 low pass filter for reducing noise
Strain calibration	Onboard shunt resistor used for deriving strain calibration coefficients (y = mx + b)
Shunt calibration resistor	499k Ohm (± 0.1%)
Integrated Temperature Channel	
Measurement range	- 40 °C to 105 °C
Accuracy	±0.25° C
RPM Sensing	
Sensor input	Open collector, open drain or digital pulses from hall effect or other source
Range	0.1 to 100 Hz (6 to 6000 RPM)
Accuracy	±0.1% (typical)
Sampling	
Sampling modes	Continuous, periodic burst, event triggered
Output options	Analog: Calibrated engineering units, adc counts and derived channels (mean, RMS and peak-peak) Digital: Speed (Hz or RPM) and pulse counts
Sampling rates	Up to 1024 Hz
Sample rate stability	±5 ppm
Network capacity	Up to 128 nodes per RF channel (bandwidth calculator:)www.microstrain.com/configure-your-system
Node synchronization	±50 µsec
Data storage capacity	16 M Bytes (up to 8,000,000 data points)
Operating Parameters	
Wireless communication range	Outdoor/line-of-sight: 2 km (ideal), 800 m (typical) Onboard antenna: 1 km (ideal), 400 (typical) Indoor/obstructions: 50 m (typical)
Antenna	Surface mount or external via U.FL connector
Radio frequency (RF) transceiver carrier	License-free 2.405 to 2.480 GHz (16 channels)
RF transmit power	User-set 0 dBm to 20 dBm. restricted regionally
Power input range	3.3 V dc to 30 V dc
Operating temperature	-40 °C to +105 °C
ESD	4 kV
Physical Specifications	
Dimensions	41.3 mm x 29 mm x 5.9 mm
Interface	Solder or screw-down terminal available
Weight	7 grams
Integration	
Compatible gateways	All WSDA gateways
Software	SensorCloud, SensorConnect, Windows 7, 8 & 10 compatible
Software development kit	http://www.microstrain.com/software/mscl
Regulatory compliance	FCC (USA), IC (Canada), CE (European Union), JET (Japan)

*Sensor excitation may be duty cycled to conserve power for sampling rates less than 1024 Hz
**Extend battery life by using a faster filtering setting

LORD SENSING

Document 8400-0124 Revision A. Subject to change without notice.

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Appendix B - Field Install Procedures

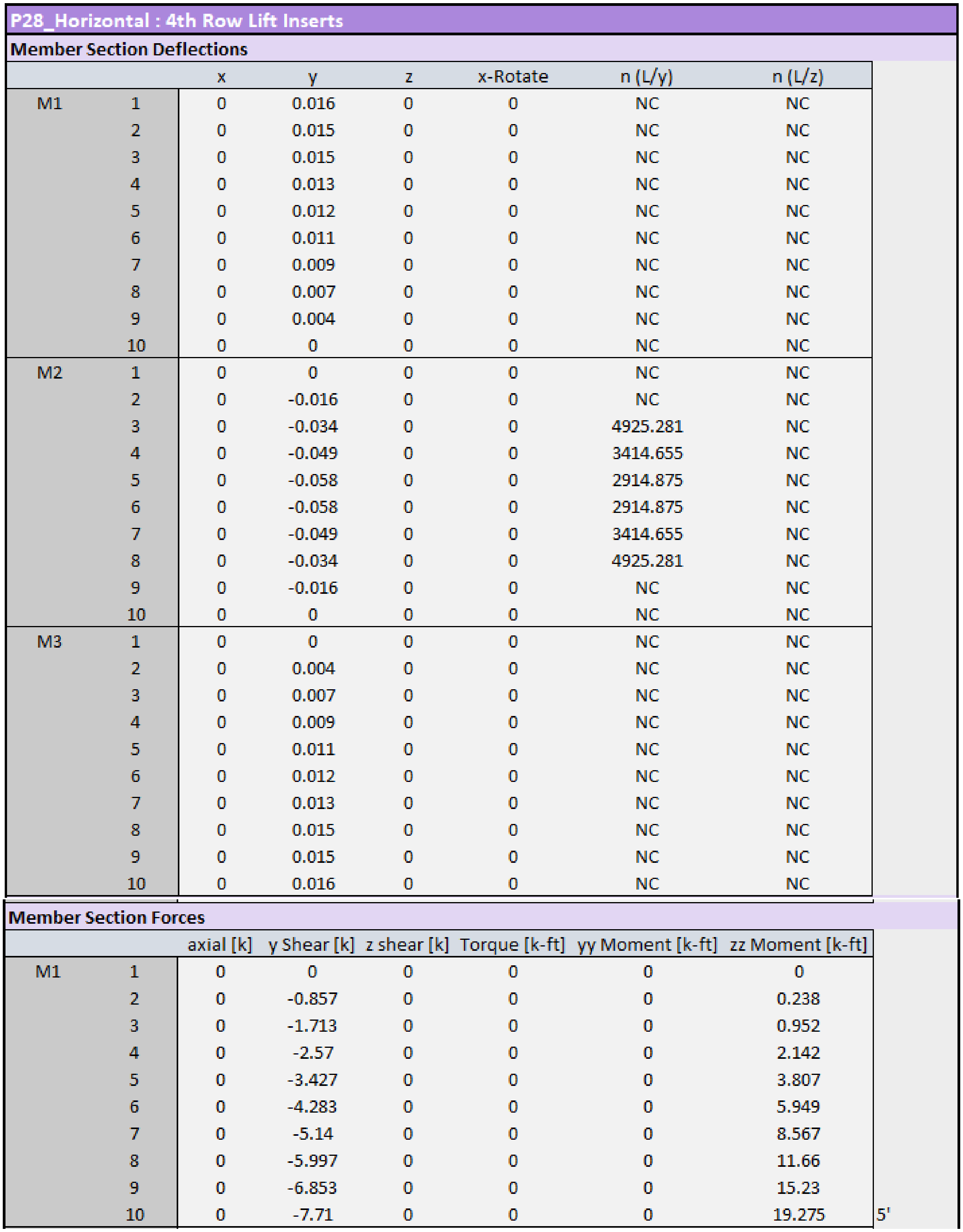
Reinforcing Steel Strain Gauge Install Procedure

✓	STEPS	
Surface Prep		
	1)	Find a smooth surface - grind down if needed
	2)	With an industrial cloth/tissue clean the area with acetone. 2-3 wipes in the same direction until the cloth is clean of dirt/grime/dust.
	3)	Repeat step (2) with distilled water
	4)	Allow install section of rebar to dry completely
Gauge Install		
	1)	Check strain gauge with voltmeter
	2)	Ensure the 2 lead wires are not touching with tweezers
	3)	Seal the separated lead wires in electrical tape
	4)	Position the gauge so that the assembly is straight longitudinally with the rebar
	5)	Place electrical tape anchor along the covered wires
	6)	Use scotch tape to anchor the gauge in the correct place along the rebar
	7)	Lift the scotch tape at a 45° angle, with some still attached at the end of the soldered connections
	8)	Apply required adhesive per the manufactures instruction.
	9)	Slowly reapply the gauge with the adhesive, ensuring it is longitudinal with the rebar. Gradually push from the "anchored" end to the opposite end to remove any air bubbles.
	10)	Hold the gauge in place with a firm pressure for the required amount of time from manufacturer
	11)	Remove the tape slowly at a 45° angle, checking that the gauge does not lift up with the tape
	12)	Anchor the strain guage with electrical tape/zip ties
	13)	Check strain gauge with voltmeter
	14)	Using an indirect heat source warm up a small amount of the TM protective wax
	15)	Using a small brush apply 1 small layer of wax along the strain gauge
	16)	Allow gauge to completely dry
	17)	Use a small amount of the TM tape to seal of the strain
	18)	Check strain gauge with voltmeter
	19)	Using zip ties, secure the wire along the rebar out of the panel
	20)	Secure the wire outside of the formwork for the pour

Concrete Surface Mounted Strain Gauge Install Procedure

✓	STEP	
Surface Prep		
	1)	Find a smooth surface - grind down if needed
Gauge Install		
	1)	Ensure surface is smooth.
	2)	Check strain gauge with voltmeter
	3)	Align gauge in the needed location
	4)	Place electrical tape anchor along the covered wires
	5)	Use scotch tape to anchor the gauge in the correct area and direction on the panel.
	6)	Lift the scotch tape at a 45° angle, with some still attached at the end of the soldered connections
	7)	Apply required adhesive per the manufacturer instruction.
	8)	Slowly reapply the gauge with the adhesive, ensuring it maintains the correct position on the concrete. Gradually push from the "anchored" end to the opposite end to remove any air bubbles.
	9)	Hold the gauge in place with a firm pressure for the required amount of time from manufacturer
	10)	Remove the tape slowly at a 45° angle, checking that the gauge does not lift up with the tape
	11)	Check strain gauge with voltmeter

Appendix C - RISA Data Example – Horizontal Check



P28_Horizontal : 4th Row Lift Inserts

Member Section Deflections

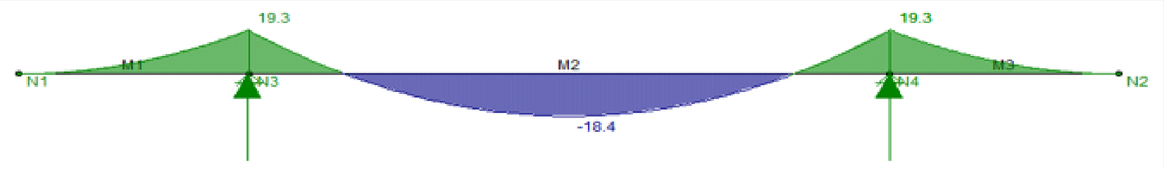
		x	y	z	x-Rotate	n (L/y)	n (L/z)
M1	1	0	0.016	0	0	NC	NC
	2	0	0.015	0	0	NC	NC
	3	0	0.015	0	0	NC	NC
	4	0	0.013	0	0	NC	NC
	5	0	0.012	0	0	NC	NC
	6	0	0.011	0	0	NC	NC
	7	0	0.009	0	0	NC	NC
	8	0	0.007	0	0	NC	NC
	9	0	0.004	0	0	NC	NC
	10	0	0	0	0	NC	NC
M2	1	0	0	0	0	NC	NC
	2	0	-0.016	0	0	NC	NC
	3	0	-0.034	0	0	4925.281	NC
	4	0	-0.049	0	0	3414.655	NC
	5	0	-0.058	0	0	2914.875	NC
	6	0	-0.058	0	0	2914.875	NC
	7	0	-0.049	0	0	3414.655	NC
	8	0	-0.034	0	0	4925.281	NC
	9	0	-0.016	0	0	NC	NC
	10	0	0	0	0	NC	NC
M3	1	0	0	0	0	NC	NC
	2	0	0.004	0	0	NC	NC
	3	0	0.007	0	0	NC	NC
	4	0	0.009	0	0	NC	NC
	5	0	0.011	0	0	NC	NC
	6	0	0.012	0	0	NC	NC
	7	0	0.013	0	0	NC	NC
	8	0	0.015	0	0	NC	NC
	9	0	0.015	0	0	NC	NC
	10	0	0.016	0	0	NC	NC

Member Section Forces

		axial [k]	y Shear [k]	z shear [k]	Torque [k-ft]	yy Moment [k-ft]	zz Moment [k-ft]
M1	1	0	0	0	0	0	0
	2	0	-0.857	0	0	0	0.238
	3	0	-1.713	0	0	0	0.952
	4	0	-2.57	0	0	0	2.142
	5	0	-3.427	0	0	0	3.807
	6	0	-4.283	0	0	0	5.949
	7	0	-5.14	0	0	0	8.567
	8	0	-5.997	0	0	0	11.66
	9	0	-6.853	0	0	0	15.23
	10	0	-7.71	0	0	0	19.275

5'

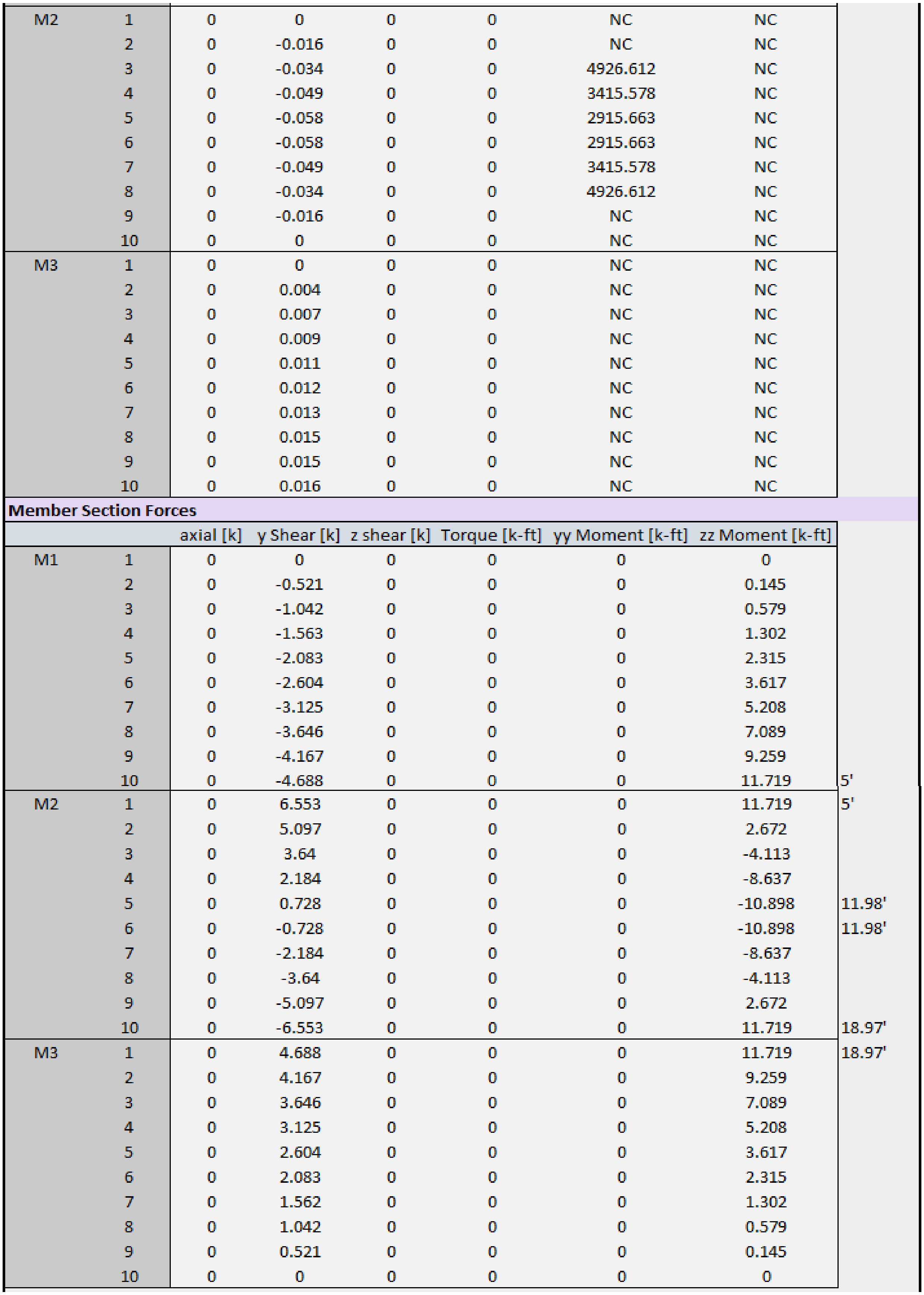
M2	1	0	10.778	0	0	0	19.275	5'
	2	0	8.383	0	0	0	4.395	
	3	0	5.988	0	0	0	-6.766	
	4	0	3.593	0	0	0	-14.206	
	5	0	1.198	0	0	0	-17.926	11.98'
	6	0	-1.198	0	0	0	-17.926	11.98'
	7	0	-3.593	0	0	0	-14.206	
	8	0	-5.988	0	0	0	-6.766	
	9	0	-8.383	0	0	0	4.395	
	10	0	-10.778	0	0	0	19.275	18.97'
M3	1	0	7.71	0	0	0	19.275	18.97'
	2	0	6.853	0	0	0	15.23	
	3	0	5.997	0	0	0	11.66	
	4	0	5.14	0	0	0	8.567	
	5	0	4.283	0	0	0	5.949	
	6	0	3.427	0	0	0	3.807	
	7	0	2.57	0	0	0	2.142	
	8	0	1.713	0	0	0	0.952	
	9	0	0.857	0	0	0	0.238	
	10	0	0	0	0	0	0	



P28_Horizontal : 3rd Row of Lift Inserts

Member Section Deflections

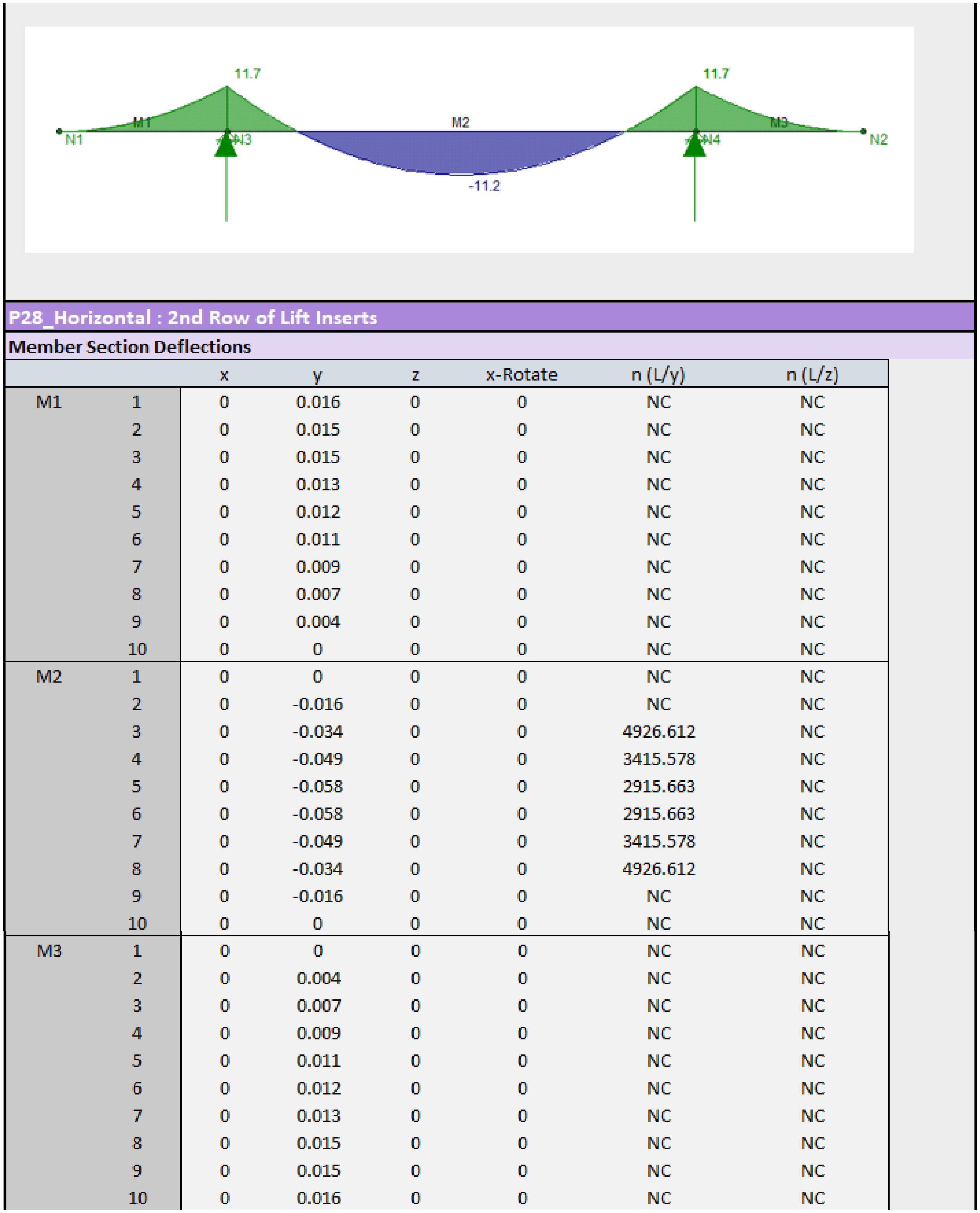
		x	y	z	x-Rotate	n (L/y)	n (L/z)
M1	1	0	0.016	0	0	NC	NC
	2	0	0.015	0	0	NC	NC
	3	0	0.015	0	0	NC	NC
	4	0	0.013	0	0	NC	NC
	5	0	0.012	0	0	NC	NC
	6	0	0.011	0	0	NC	NC
	7	0	0.009	0	0	NC	NC
	8	0	0.007	0	0	NC	NC
	9	0	0.004	0	0	NC	NC
	10	0	0	0	0	NC	NC



M2	1	0	0	0	0	NC	NC
	2	0	-0.016	0	0	NC	NC
	3	0	-0.034	0	0	4926.612	NC
	4	0	-0.049	0	0	3415.578	NC
	5	0	-0.058	0	0	2915.663	NC
	6	0	-0.058	0	0	2915.663	NC
	7	0	-0.049	0	0	3415.578	NC
	8	0	-0.034	0	0	4926.612	NC
	9	0	-0.016	0	0	NC	NC
	10	0	0	0	0	NC	NC
M3	1	0	0	0	0	NC	NC
	2	0	0.004	0	0	NC	NC
	3	0	0.007	0	0	NC	NC
	4	0	0.009	0	0	NC	NC
	5	0	0.011	0	0	NC	NC
	6	0	0.012	0	0	NC	NC
	7	0	0.013	0	0	NC	NC
	8	0	0.015	0	0	NC	NC
	9	0	0.015	0	0	NC	NC
	10	0	0.016	0	0	NC	NC

Member Section Forces

		axial [k]	y Shear [k]	z shear [k]	Torque [k-ft]	yy Moment [k-ft]	zz Moment [k-ft]	
M1	1	0	0	0	0	0	0	
	2	0	-0.521	0	0	0	0.145	
	3	0	-1.042	0	0	0	0.579	
	4	0	-1.563	0	0	0	1.302	
	5	0	-2.083	0	0	0	2.315	
	6	0	-2.604	0	0	0	3.617	
	7	0	-3.125	0	0	0	5.208	
	8	0	-3.646	0	0	0	7.089	
	9	0	-4.167	0	0	0	9.259	
	10	0	-4.688	0	0	0	11.719	5'
M2	1	0	6.553	0	0	0	11.719	5'
	2	0	5.097	0	0	0	2.672	
	3	0	3.64	0	0	0	-4.113	
	4	0	2.184	0	0	0	-8.637	
	5	0	0.728	0	0	0	-10.898	11.98'
	6	0	-0.728	0	0	0	-10.898	11.98'
	7	0	-2.184	0	0	0	-8.637	
	8	0	-3.64	0	0	0	-4.113	
	9	0	-5.097	0	0	0	2.672	
	10	0	-6.553	0	0	0	11.719	18.97'
M3	1	0	4.688	0	0	0	11.719	18.97'
	2	0	4.167	0	0	0	9.259	
	3	0	3.646	0	0	0	7.089	
	4	0	3.125	0	0	0	5.208	
	5	0	2.604	0	0	0	3.617	
	6	0	2.083	0	0	0	2.315	
	7	0	1.562	0	0	0	1.302	
	8	0	1.042	0	0	0	0.579	
	9	0	0.521	0	0	0	0.145	
	10	0	0	0	0	0	0	



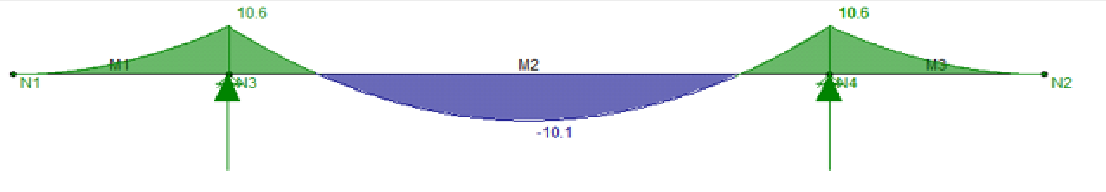
P28_Horizontal : 2nd Row of Lift Inserts

Member Section Deflections

		x	y	z	x-Rotate	n (L/y)	n (L/z)
M1	1	0	0.016	0	0	NC	NC
	2	0	0.015	0	0	NC	NC
	3	0	0.015	0	0	NC	NC
	4	0	0.013	0	0	NC	NC
	5	0	0.012	0	0	NC	NC
	6	0	0.011	0	0	NC	NC
	7	0	0.009	0	0	NC	NC
	8	0	0.007	0	0	NC	NC
	9	0	0.004	0	0	NC	NC
	10	0	0	0	0	NC	NC
M2	1	0	0	0	0	NC	NC
	2	0	-0.016	0	0	NC	NC
	3	0	-0.034	0	0	4926.612	NC
	4	0	-0.049	0	0	3415.578	NC
	5	0	-0.058	0	0	2915.663	NC
	6	0	-0.058	0	0	2915.663	NC
	7	0	-0.049	0	0	3415.578	NC
	8	0	-0.034	0	0	4926.612	NC
	9	0	-0.016	0	0	NC	NC
	10	0	0	0	0	NC	NC
M3	1	0	0	0	0	NC	NC
	2	0	0.004	0	0	NC	NC
	3	0	0.007	0	0	NC	NC
	4	0	0.009	0	0	NC	NC
	5	0	0.011	0	0	NC	NC
	6	0	0.012	0	0	NC	NC
	7	0	0.013	0	0	NC	NC
	8	0	0.015	0	0	NC	NC
	9	0	0.015	0	0	NC	NC
	10	0	0.016	0	0	NC	NC

Member Section Forces

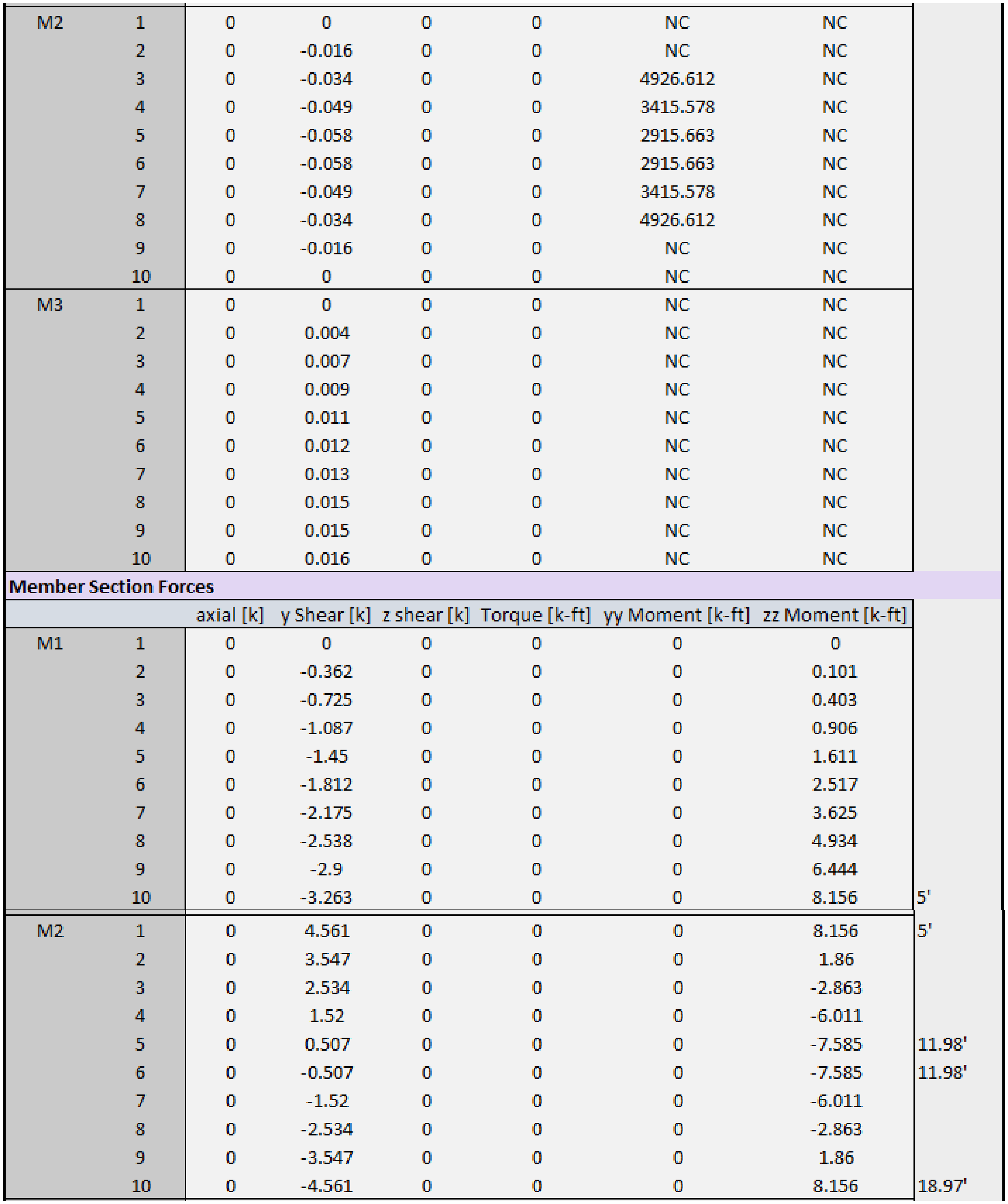
		axial [k]	y Shear [k]	z shear [k]	Torque [k-ft]	yy Moment [k-ft]	zz Moment [k-ft]	
M1	1	0	0	0	0	0	0	
	2	0	-0.472	0	0	0	0.131	
	3	0	-0.944	0	0	0	0.525	
	4	0	-1.417	0	0	0	1.181	
	5	0	-1.889	0	0	0	2.099	
	6	0	-2.361	0	0	0	3.279	
	7	0	-2.833	0	0	0	4.722	
	8	0	-3.306	0	0	0	6.427	
	9	0	-3.778	0	0	0	8.395	
	10	0	-4.25	0	0	0	10.625	5'
M2	1	0	5.941	0	0	0	10.625	5'
	2	0	4.621	0	0	0	2.423	
	3	0	3.301	0	0	0	-3.729	
	4	0	1.98	0	0	0	-7.831	
	5	0	0.66	0	0	0	-9.881	11.98'
	6	0	-0.66	0	0	0	-9.881	11.98'
	7	0	-1.98	0	0	0	-7.831	
	8	0	-3.301	0	0	0	-3.729	
	9	0	-4.621	0	0	0	2.423	
	10	0	-5.941	0	0	0	10.625	18.97'
M3	1	0	4.25	0	0	0	10.625	18.97'
	2	0	3.778	0	0	0	8.395	
	3	0	3.306	0	0	0	6.427	
	4	0	2.833	0	0	0	4.722	
	5	0	2.361	0	0	0	3.279	
	6	0	1.889	0	0	0	2.099	
	7	0	1.417	0	0	0	1.181	
	8	0	0.944	0	0	0	0.525	
	9	0	0.472	0	0	0	0.131	
	10	0	0	0	0	0	0	



P28_Horizontal : 1st Row of Lift Inserts

Member Section Deflections

		x	y	z	x-Rotate	n (L/y)	n (L/z)
M1	1	0	0.016	0	0	NC	NC
	2	0	0.015	0	0	NC	NC
	3	0	0.015	0	0	NC	NC
	4	0	0.013	0	0	NC	NC
	5	0	0.012	0	0	NC	NC
	6	0	0.011	0	0	NC	NC
	7	0	0.009	0	0	NC	NC
	8	0	0.007	0	0	NC	NC
	9	0	0.004	0	0	NC	NC
	10	0	0	0	0	NC	NC

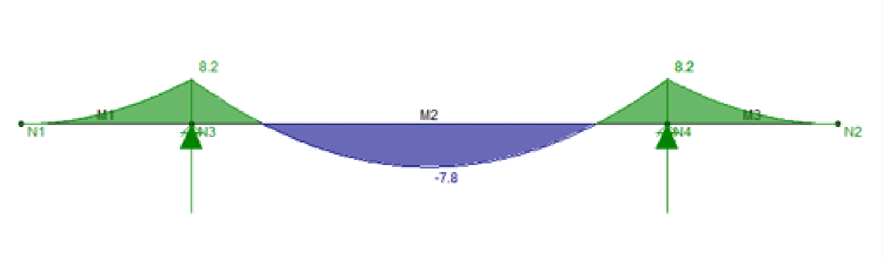


M2	1	0	0	0	0	NC	NC	
	2	0	-0.016	0	0	NC	NC	
	3	0	-0.034	0	0	4926.612	NC	
	4	0	-0.049	0	0	3415.578	NC	
	5	0	-0.058	0	0	2915.663	NC	
	6	0	-0.058	0	0	2915.663	NC	
	7	0	-0.049	0	0	3415.578	NC	
	8	0	-0.034	0	0	4926.612	NC	
	9	0	-0.016	0	0	NC	NC	
	10	0	0	0	0	NC	NC	
M3	1	0	0	0	0	NC	NC	
	2	0	0.004	0	0	NC	NC	
	3	0	0.007	0	0	NC	NC	
	4	0	0.009	0	0	NC	NC	
	5	0	0.011	0	0	NC	NC	
	6	0	0.012	0	0	NC	NC	
	7	0	0.013	0	0	NC	NC	
	8	0	0.015	0	0	NC	NC	
	9	0	0.015	0	0	NC	NC	
	10	0	0.016	0	0	NC	NC	

Member Section Forces

		axial [k]	y Shear [k]	z shear [k]	Torque [k-ft]	yy Moment [k-ft]	zz Moment [k-ft]	
M1	1	0	0	0	0	0	0	
	2	0	-0.362	0	0	0	0.101	
	3	0	-0.725	0	0	0	0.403	
	4	0	-1.087	0	0	0	0.906	
	5	0	-1.45	0	0	0	1.611	
	6	0	-1.812	0	0	0	2.517	
	7	0	-2.175	0	0	0	3.625	
	8	0	-2.538	0	0	0	4.934	
	9	0	-2.9	0	0	0	6.444	
	10	0	-3.263	0	0	0	8.156	5'
M2	1	0	4.561	0	0	0	8.156	5'
	2	0	3.547	0	0	0	1.86	
	3	0	2.534	0	0	0	-2.863	
	4	0	1.52	0	0	0	-6.011	
	5	0	0.507	0	0	0	-7.585	11.98'
	6	0	-0.507	0	0	0	-7.585	11.98'
	7	0	-1.52	0	0	0	-6.011	
	8	0	-2.534	0	0	0	-2.863	
	9	0	-3.547	0	0	0	1.86	
	10	0	-4.561	0	0	0	8.156	18.97'

M3	1	0	3.262	0	0	0	8.156	18.97'
	2	0	2.9	0	0	0	6.444	
	3	0	2.538	0	0	0	4.934	
	4	0	2.175	0	0	0	3.625	
	5	0	1.813	0	0	0	2.517	
	6	0	1.45	0	0	0	1.611	
	7	0	1.087	0	0	0	0.906	
	8	0	0.725	0	0	0	0.403	
	9	0	0.362	0	0	0	0.101	
	10	0	0	0	0	0	0	



Appendix D - Image Permissions

Figure 3-23: Sketches of Types of Fractures (Source: ASTM C39/C ©)

Dear Kati,

This is in response to your email of 24 January to Chris Davis.

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Kind regards,

Kathe

Kathe Hooper
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Figure 1-1: Lifting Hardware prior to concrete pour

Figure 1-2: Field Rigging 2 Wide x 4 High

Figure 2-1: Inserts Tied to Rebar

Figure 3-7: 4 High x 2 Wide Lift Rigging

Figure 3-8: Strain Gauge Aligned on Rebar

Figure 3-9: Strain Gauge After Being Adhered

Figure 3-10: Wires Protected in Foam

Figure 3-11: Rebar Wiring Protection

Figure 3-15: Surface Gauges & Wireless Node Attached to Panel

Figure 3-16: Installed Surface Strain Gauge

Figure 3-17: Installed Surface Strain Gauge

Figure 3-18: Base Station & Laptop Set-Up on Site

Katherine,

Thanks for reaching out for permission and hope the project has been a success for you so far. I do not have any issues with you using the pictures or description in your report and look forward to receiving a copy once complete.

Thanks,

Ted Strahm
Lithko Contracting, LLC
Central Region
M: (816) 813-0511

EOE / Vet / Disabled

Figure 3-1: Base Station Direct USB Connection

Figure 3-2: LORD Strain Gateway Options

Figure 3-4: Three Wire Quarter-Bridge

Figure 3-5: Two Wire Quarter-Bridge

Figure 3-12: Base Station Homepage in Sensor Connect

Figure 3-13: Wireless Node Calibration Tab

Figure 3-14: Wireless Node Sampling Tab/Lost Beacon Timeout Control

Figure 3-19: Data Repositories Homepage

Thank you. It’s fine to use the images. I look forward to getting the report when it is done.

Thanks,

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