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Analytical and Experimental Investigation of
Modular Structural Concrete Insulated Panels

by

Usha Pant

A thesis

submitted in partial fulfillment of the requirements for the degree of

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Committee Approval

To the Graduate Faculty:

The members of the committee appointed to examine the thesis of Usha Pant and find it satisfactory and recommended that it be accepted.

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Analytical and Experimental Investigation of Modular Structural Concrete Insulated Panels (SCIPs)

Thesis Abstract-Idaho State University (2021)

Structural Concrete Insulated Panel (SCIP) is a relatively new construction technology which is an alternative to the traditional wood framing for residential and low-rise commercial buildings. A typical SCIP is composed of an Expanded Polystyrene (EPS) core with a thin layer of concrete on each side. A concrete layer houses a galvanized steel mesh which are connected to each other with shear connectors also known as truss diagonals. Past research has demonstrated that SCIPs offer good structural resistance, thermal and acoustic insulation properties. SCIPs are generally unknown to practicing structural engineers. Most engineers find it difficult to predict the axial, flexural, and shear capacities of SCIPs. This research presents analytical and some experimental investigation of SCIPs that utilize precasting and modular construction. Simplified flexural analysis in-line with the principles of reinforced concrete design is proposed for the experiments previously conducted at Idaho State University on full-scale SCIPs. Analytical results are compared with the experimental data to validate the analysis approach for SCIPs with different spans, materials properties, and thicknesses. Due to lack of experimental data on shear strength of SCIPs, two specimens were tested under direct shear loading in the structural laboratory (SLAB) at Idaho State University. The panels exhibited substantial deformation prior to failure. The failure mechanism under direct shear loading was observed to be shear-compressive for both panels. Upon inspection of the tested panels, significant buckling of diagonal connectors was observed. Experimental results from the shear tests were used to generate simplified analytical approaches for prediction of shear strength of

SCIPs with different material properties and thicknesses. Furthermore, the research provides concepts for construction of structures made of SCIPs using precast concrete technology with wet and dry connections.

Key Words: MetRock SCIP panels; Precasting of slabs; shear design; Flexure design; lateral design; Buckling; Large- scale testing; Analytical modelling; Experimental investigation; One story building.

Chapter 1. Introduction

1.1 Background

Between 2017 and 2060, the population in the United States is expected to grow by about 79 million people, from around 326 million to 404 million. In 2058, the population is expected to surpass 400 million (Vespa et al, 2018). The increase in human population has adverse effects in the field of construction, necessitating highly sustainable and energy efficient building systems. To cope with the increase in the demand of residential houses, structural insulated concrete panels (SCIPs) can be used as an alternative to traditional wood framing system. Despite being vulnerable to water damage, fire, decay, and termites, wood is still the most popular system of construction used for residential and small commercial. SCIPs can solve many of the aforementioned problems associated with wood construction. SCIPs can offer buildings that have adequate structural integrity, energy efficiency, and durability.

A structural sandwich is a type of laminated composite made up of several different materials that are bonded together to take advantage of the qualities of individual component for the overall structural benefit of the assembly. SCIPs employ the panel construction concept, in which most of the structural components are standardized and manufactured in plants located away from the construction site, then brought to the site for assembly (Brzev, 2010).

SCIPs are typically composed of an insulated core and galvanized steel mesh on both sides, held together by diagonal steel shear connectors as shown in Figure 2-1 and 2-2. A layer of concrete, called wythe, which is typically between 1-2 in. is applied to the sides of the panel. The modular SCIP system integrates readily available and off-the-shelf components which include recycled Expanded Polyester Styrofoam (EPS), steel mesh, and steel trusses. The panels can be made in a portable and simple hydraulic jig press. For cast-in-place SCIP construction, concrete can be applied after erection of the panels on-site. This is generally in the form of shotcrete and has been the common construction practice for SCIPs. For precast modular SCIPs, the panels are poured in a precast bed, similar to precast sandwich wall panel construction, and are then transported to the site for assembly. Connections between precast modular SCIPs can be either wet (e.g. grouted) or dry (welded or bolted).

SCIP technology has been employed in construction of residential and commercial low-rise buildings in many countries since its inception in the late 1960s. SCIPs are particularly appealing in places prone to strong winds and seismic activity because of characteristics such as greater thermal/sound insulation, structural stability, higher stiffness, reduced seismic weight, and sustainability. SCIPs can also be used in construction of buildings located in coastal areas where corrosion and structural degradation due to the saline environment and humidity are significant for certain construction materials. Past literature indicates that buildings made of SCIPs have great reserve strength and ductility when exposed to natural hazards such as seismic activity (Mashal, 2011). Most structural engineers in the United States are not familiar with SCIPs and thus are hesitant to use them for construction of residential buildings. In addition, there is a lack of building code requirements and design/detailing guidelines for SCIPs. Each manufacturer of SCIP uses different material properties and fabrication process to produce the panels. The purpose of this research is to introduce the concept for precast modular SCIPs also known as MetRock SCIPs. The research provides simplified analytical calculations for flexural, axial, and shear design of MetRock SCIPs. Experimental work is also conducted to quantify the shear resistance of MetRock SCIPs.

1.2 Research Motivation and Scope

The research motivation is the lack of research on the structural capability of SCIP in the construction field as an alternative to traditional timber or masonry construction techniques. There has been a massive explosion of innovations and technology since the industrial revolution. Engineers have used science and research to defy conventional wisdom and attain levels of accomplishment previously unimaginable. Unfortunately, the construction industry has yet to develop a fresh, inventive, and cost-effective construction approach. Although breakthroughs such as SCIP construction can provide structurally sound sustainable buildings that are suitable for locations prone to seismic activity and wind, design engineers rarely consider them as an alternative to wood or masonry methods due to a lack of understanding of the material and construction technique (Mashal, 2011).

1.3 Objectives

A simplified design process for estimating the panel strength properties must be developed before SCIPs can be widely employed in the building sector and acknowledged by engineers. This study utilizes experimental results from the previous testing of MetRock (MR) slab and wall panels at Idaho State University (Gurung, 2019). Experimental results are used to validate the simplified analytical procedures proposed in this research. New experiments are conducted to investigate shear behavior of MR panels. Some of the key objectives of the investigation are listed below:

1. Introduce concept for precast modular Structural Concrete Insulated Panel (SCIP)
2. Describe MR panels and provide concepts for precasting and connections
3. Discuss material properties for the components of MR panels
4. Provide simplified analytical procedures for estimating flexural and axial capacities of MR panels

5. Conduct a large-scale experimental study on out-of-plane shear behavior of MR Panels
6. Utilize test results from shear testing to provide a procedure for estimating out-of-plane shear capacity of MR panels
7. Define the composite action achieved by the MR panels when subjected to out-of-plane bending
8. Provide sample design examples for MR panels

1.4 Thesis Structure

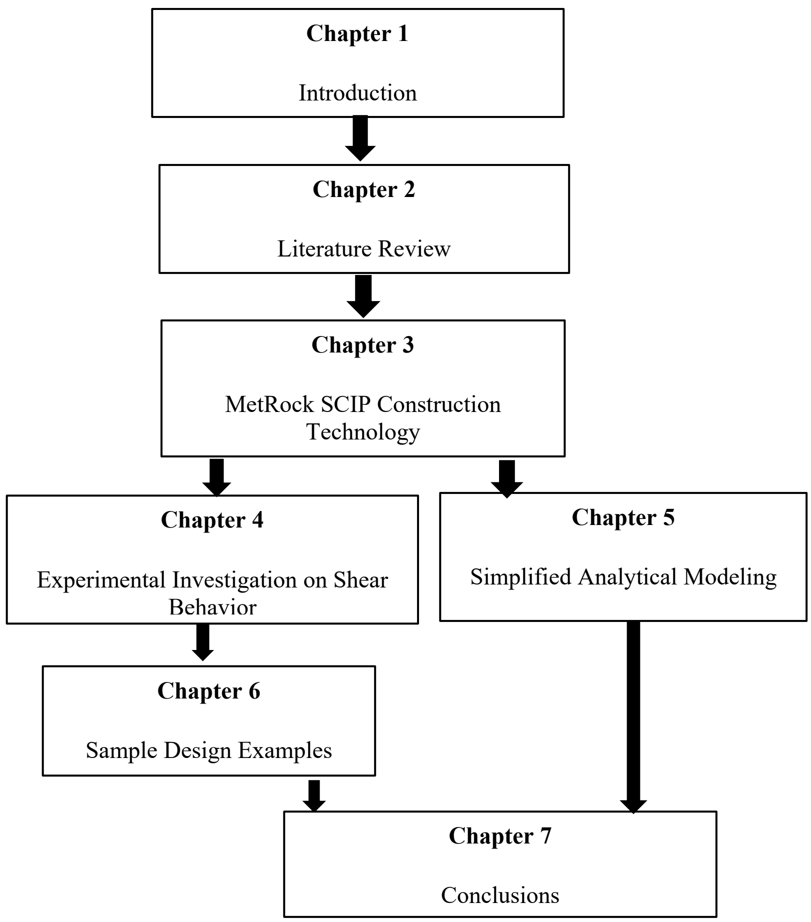


Figure 1-1: Thesis Structure

1.5 Overview

Chapter 1 describes a brief introduction of the research. It includes some background information on SCIP technology, research motivation, and objectives.

Chapter 2 contains past literature reviews on SCIPs.

Chapter 3 incorporates a detailed study of MetRock SCIP construction technology and its advantages over other traditional methods. It also introduces concepts for precast and modular construction of SCIPs. Information about the construction process for the precast MR panels that are developed for shear testing, is also provided.

Chapter 4 discusses the results of experimental investigation on shear behavior of precast MR panels. Results such as the average load-deflection curve and ultimate shear capacity of the MR panels are discussed in this chapter. Testing results are used to provide a simplified and conservative methodology for estimating shear capacity of MR panels with various material properties and concrete thicknesses in accordance with standards such as ACI 318-18.

Chapter 5 describes simplified analytical modelling of MR panels. It includes design charts that can be used for production of a handbook for MR panels. The chapter also includes details of flexural, shear, axial, and lateral design for the MR panels along with some examples.

Chapter 6 includes some examples and information for the design of one-story residential building with MR SCIPs as part of non-load bearing wall system.

Chapter 7 summarizes the experimental results that were presented in Chapter 4 and Chapter 5. Conclusions about shear behavior of SCIPs are presented. Future work involving modeling of SCIPs and producing design guidelines for SCIPs are also provided.

Also included are the table of contents, figures, tables, and appendices. The appendices include spread sheet calculations, experimental data, instrumentation, and material data sheets that would help design engineers.

Chapter 2. Literature Review

2.1 Structural Concrete Insulated Panels

Structural Concrete Insulated Panel (SCIP) is an alternative to standard wood framing construction. SCIPs are a version of Structural Insulated Panels (SIPs) in which reinforced concrete is used instead of plywood to provide the two load-bearing faces, which are held together by a complicated shear transfer system. Buildings ranging in height from one to ten stories make up urban low-rise structures. SCIPs are appropriate for urban low-rise structures since they may be utilized for both load-bearing and non-load-bearing walls, roofs, and floors. Moment-resisting frames and additional concrete columns can be used in conjunction with SCIPs for buildings that are more than two-stories in height. The SCIP technology was first developed and patented in the late 1960s by Victor Weismann in Pasadena, California, under the name “thin shell sandwich panel construction” (Mashal, 2014). SCIPs are three-dimensional concrete panels made up of an Expanded Polystyrene Styrofoam (EPS) core sandwiched between two cold-rolled steel wire meshes connected by a diagonal transversal truss connector. The assemblage is subsequently covered on both sides with a layer of concrete or high-strength cementitious mortar (El Demerdash, 2013). There are various producers of SCIPs that utilize different procedures for production of panels. These are briefly introduced below.

2.1.1 EVG 3D

The 3D Building Method is a relatively new, cost-effective construction system that is based on industrially built 3D panels and has a wide range of applications. The 3D panels are made up of an EPS core varying in thickness from 40 to 100 mm (1.57 to 3.94 in) sandwiched between two plane-parallel welded wire mesh sheets (cover meshes) and inclined diagonal wires running through the EPS core and welded to the cover meshes. The panels are produced in a factory with sophisticated automation process.

As a result, a light-weight three-dimensional truss structure with high inherent stiffness is created. Between the EPS core and the cover mesh, there is a distinct gap of 13 to 19 mm (0.51 to 0.75 in). The usual width of 3D panels is 1.20 m (1.00 m) (47.2 to 39.4 in), but the element's length is customizable (in 10 cm (3.94 in) increments) and depends on the application field. 3D panels are supplied to the job site as positioned pieces that can be readily attached to wall and slab constructions. Splice meshes serve to seal the joints between the 3D panels. This results in a continuous mesh structure (reinforcement) throughout the whole structure.

After that, a 40 to 60 mm (1.57 to 2.36 in) thick cement-mortar layer (concrete layer) is placed to both sides of this dry construction, either manually or mechanically. The EPS cores of the 3D panels serve as a shuttering and plaster base throughout this procedure. The 3D structure of the panels provides structural and functional strength as soon as the concrete has solidified. The result is a composite sandwich system with wire diagonals organized in a truss-like form with adequate shear strength connecting the two reinforced concrete shells on the outside. The section of a 3D wall is shown in Figure 2-1.

A large number of compressions, bending, and shear tests were conducted by recognized research institutes in Austria and abroad in order to demonstrate the structural efficiency of 3D structures. All these experiments revealed that the fundamental theories of reinforced concrete building systems, as well as the resultant calculation methodologies, are applicable to 3D structures without limitation (EVG,2005).

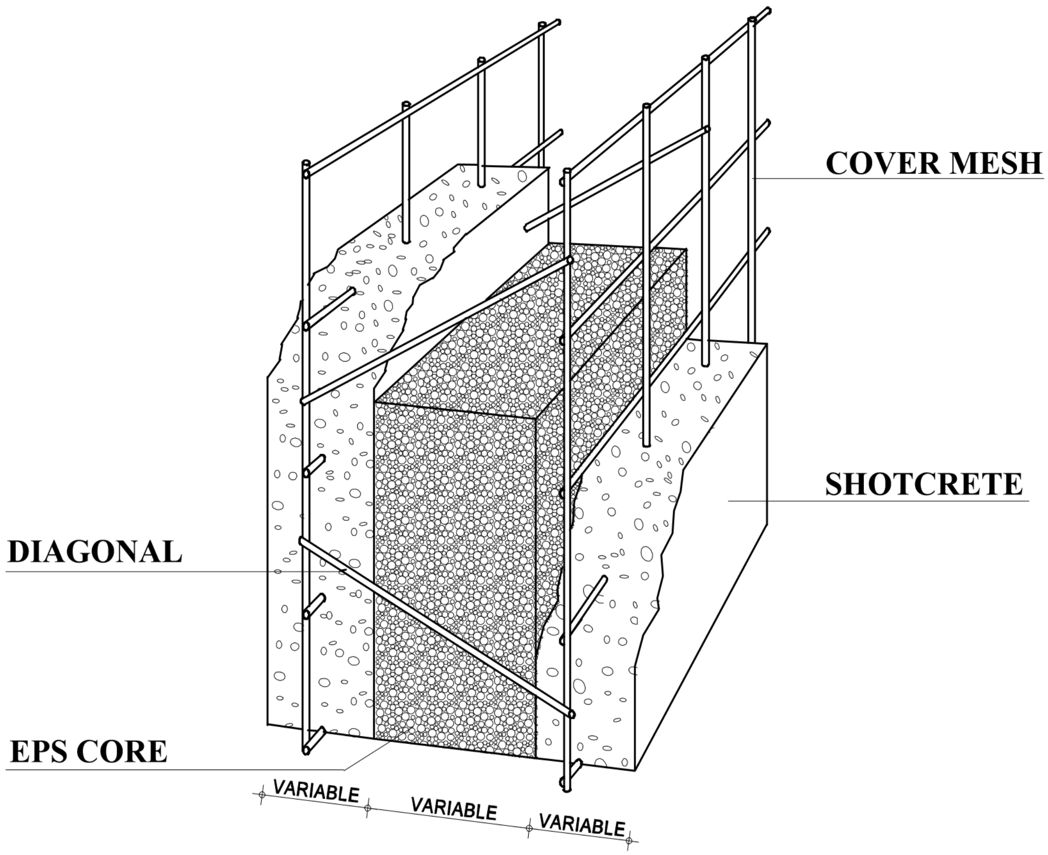


Figure 2-1: Section of a 3D wall (EVG,2005)

2.1.2 MetRock SCIP

MetRock (MR) Panels, also known as the MetRock Structural Concrete Insulated Panel, is a modular form of SCIP. The components for MR panels are typically transported as individual, off-the-shelf elements. The components are assembled into panels using a portable hydraulic jig press and a pneumatic hog ring fastener, as opposed to conventional SCIP, which are generally fabricated in production plants and transported as solid panels to the construction site. An MR panel is made up of a recycled Expanded Polystyrene Styrofoam (EPS) block core bordered on both sides by a 14-gauge galvanized wire mesh and linked by 3/16-inch galvanized diagonal steel wire trusses spaced at six inches. Insulation core blocks are six inches broad, four inches thick, and ten feet long. Following the installation of the panels, the assembly is covered with a one-inch coating of concrete on both sides to create loadbearing faces. Figure 2-2 provides the details for a typical MetRock SCIP core (MetRock SCIP, 2019). More details of MetRock SCIP panels are discussed in Chapter 5.

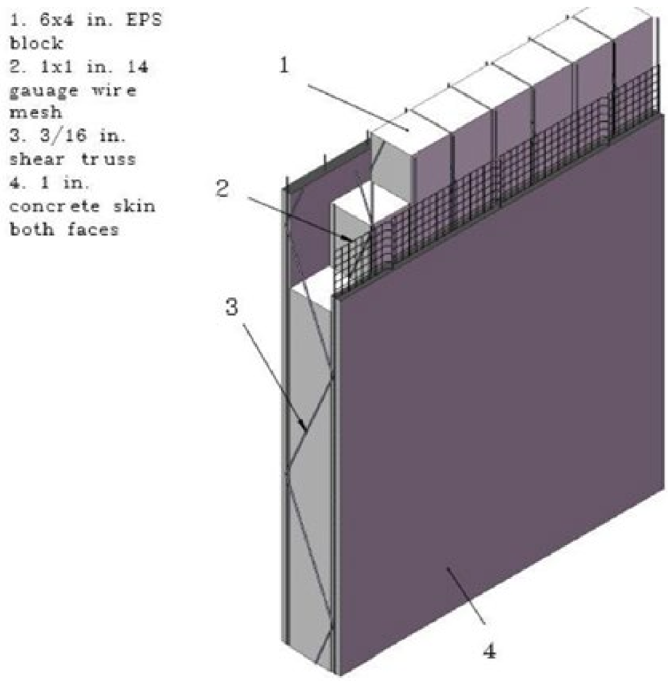


Figure 2-2: Details for a typical MetRock SCIP (MetRock SCIP, 2019)

2.1.3 Emmedue Panel

The Emmedue panel is another version of commercially available SCIP. The panel is industrially produced, and it is then assembled and completed on site with two layers of concrete. The self-supporting panels can be made in various shapes.

Emmedue offers a range of building elements: load bearing walls, floors, roofing, stairs, partitions and curtain walls. Therefore, buildings can be entirely constructed with the same building system, optimizing different supply and timing phases as well as workforce availability.

Laboratory tests carried out on full-scale prototype houses have shown that the Emmedue structures can withstand seismic loads (Spa, 2021).

2.2 Advantages of precast structural concrete insulated panels

The use of SCIPs in building has many benefits. Some of them are listed below:

2.2.1 Thermal Insulation

One of the most important benefits of SCIPs is the thermal resistance of the structures built. Over the last several years, the demand for better thermal performance and structural economy in building construction has pushed the development of concrete panels. Furthermore, lighter and thinner concrete wythes capable of transmitting shear stresses between panel layers and minimizing localized heat loss have been made possible by new high-performance concrete (Amran, 2020). The insulated concrete form (ICF) is a developing method that may be able to meet the needs of current building industry. ICF is a concrete-and-expanded polystyrene (EPS) combination that improves the building's insulation and mechanical characteristics. The inclusion of EPS in ICF aids in the transformation of sandwiched concrete from brittle to ductile failure, as measured in the terms of plastic deformation. To investigate the thermal performance of ICF panels using R-value, a simple experimental technique is presented. The suggested design is successful for determining the thermal resistance of wall panels, with an R-value of 5.22 W/m^2-K for the ICF panel, which is 7.9 times greater than the plain concrete panel. ICFs have a higher R-value, which means they give more insulation to the building by maintaining a constant temperature for longer periods of time. ICF's thermal insulation characteristic lowers electricity consumption and improves the building's energy efficiency. As a result, the ICF system aids in the construction of sustainable buildings by providing excellent thermal insulation while also improving structural strength (Solomon & Hemalatha, 2020).

2.2.2 Time efficiency

SCIPs increase the efficiency of construction at the site and provide accelerated construction if they are prefabricated (e.g. elimination of on-site Shotcreting). A full building envelope, comprising the external membrane, moisture barrier, insulation, and interior finish, may be achieved with a concrete insulated wall panel system. The time necessary to construct the building envelope may be considerably reduced by utilizing precast insulated wall panels since all of these systems can be built in one rapid procedure instead of creating separate wall parts involving several trades. In addition, precast insulated wall panels are manufactured off-site. Soon after the first precast panels arrive on-site, wall manufacturing is usually virtually complete. The window system may also be installed in precast panels at the manufacturing site, reducing the amount of time it takes to close in a structure. Panels can then be supplied to the project site as needed to expedite the construction process (Says et al., 2012).

2.2.3 Energy Efficient

Traditional buildings utilize more than 40% of all power produced, according to a conservative estimate. This sector's demand is rising at a pace of almost 14% per year, the greatest among all other industries. Energy efficient buildings that can be constructed quickly, are needed in order to solve the current challenges in the construction industry (Bhatti, 2016). Concrete has a higher thermal mass when compared with other less-massive materials. Thermal mass is defined as a property that enables materials to absorb, store and later release significant amounts of heat. Concrete’s inherent ability to absorb and store heat and cold can delay and reduce peak HVAC (Heating, Ventilation, and Air Conditioning) loads. Due to the slow release of heat and cold, the thermal mass of concrete can also shift the demand to off-peak time periods when utility rates are lower, thereby decreasing energy costs further (Says et al., 2012).

2.2.4 Environment friendly

Precast SCIPs are considered green construction and are environment friendly. For instance, MR panels utilize recycled EPS for insulation. Also, the concrete mix contains cementitious materials such as fly ash that replaces a portion of the cement. According to a recent life cycle assessment (LCA) of exterior cladding products commissioned by the Natural Stone Council and conducted by the University of Tennessee's Center for Clean Products in 2009, "precast concrete and granite exhibit the greatest advantages, although it's unclear which is the most environmentally preferable overall." The focus of their research was on a two-story load-bearing structure. Precast concrete can also help to reduce the environmental effect of a construction project on the construction site. The negative impact that precast concrete insulated wall panels have on a site is relatively low because they are made off-site and crane-lifted off the delivery truck and placed straight onto the structure (Says et al., 2012).

2.3 Previous Experimental and Numerical Investigations

There have been some experimental and numerical investigations in the past for structural analysis and behavior of SCIPs under flexural and axial loads. However, limited research is done on shear behavior of SCIPs. Despite advancements in computational methodologies and greater computer capacity, existing analytical approaches and computational models based solely on mechanical principles are insufficient and inaccurate for design (ACI 347.2 R-13, 2013). To better understand and anticipate the structural behavior of SCIPs, large-scale shear testing is required. Some of the previous investigations on SCIPs are summarized below.

2.3.1 Static and dynamic analytical and experimental analysis of 3D reinforced concrete panels

This study investigates a three-dimensional panel system that was proposed as a new way of building in Jordan, employing relatively high-strength modular panels for walls and ceilings. The panel is made up of two steel meshes on both sides of an expanded polystyrene core, which are linked with a truss wire to form a three-dimensional structure. The ceiling panel's top face was poured with standard concrete mix, while the bottom face and both faces of the wall panels were shotcrete (dry process). A thorough experimental testing program for ceiling and wall panels exposed to static and dynamic loadings was undertaken to examine the structural performance of this system. For beam and shear wall elements, as well as wall elements subjected to transverse and axial stresses, load-deflection curves were produced. The performance of the suggested structural system was tested and compared to a conventional three-dimensional reinforced concrete frame system for buildings with the same floor area and number of floors using static and dynamic analysis. A ceiling panel's compressive strength capacity is measured for gravity loads, whereas its flexural capacity is determined for wind and seismic loading. The strength and serviceability criteria for structures erected utilizing the three-dimensional panel system was found to be easily met. The research concluded that due to its high stiffness to mass ratio, the 3D panel system outperforms traditional frame systems in terms of dynamic performance (Numayr & Haddad, 2009).

2.3.2 Behavior of the MR Sandwich Panel in Flexure

The housing sector is an important part of the American economy, accounting for roughly 4% of total economic activity. The purpose of this research was to investigate the SCIP system's flexural behavior and to explain the manufacturing and construction elements of the SCIP system. A step-by-step design technique was created to anticipate the load bearing capability of the panels and give the engineer with a dependable tool for designing the panels. For varied size panels, an experimental program was carried out. Ten full-scale specimens were evaluated in flexural for the experimental program. The specimens utilized in this experiment were all 24-inches wide and had different span-to-depth ratios. All of the tests were carried out in accordance with ASTM E72-05. The panels were quasi-statically loaded in 1000-pound increments until failure was reached. The edge trusses showed in-plane buckling due to the lack of side and end confinement in the panels examined for this investigation, but the specimens continued to resist higher stress due to the redundancy in the shear connections. Although all of the specimens failed horizontally close to the support points at ultimate load capacity, which is not the desired mode of failure (very brittle), the results revealed that the panels were moderately ductile with a wide range of non-linear behavior. Overall, the specimens did not incorporate the actual edge detailing that would be expected in real-life applications, thus premature failure of the panels occurred (Fouad et al, 2008) . A sample load-deflection graph for a six-inch thick specimen is illustrated in Figure 2-3. Some analytical models were presented to estimate the capacity of the panels under flexure.

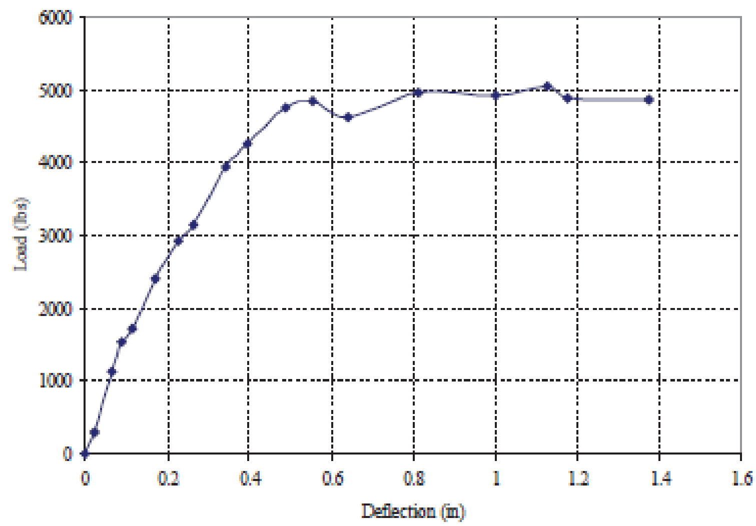


Figure 2-3: Average load vs deflection for the MetRock SCIP slabs (Fouad et al, 2008)

2.4 Summary

A literature review was conducted to highlight past research on SCIPs and precast insulated wall panels. Some of the key findings are listed below:

1. The major difference between a standard precast wall and a precast sandwich wall panel is the thermal insulation layer, which is included not only to decrease the panel's weight but also to increase its thermal resistance. SCIPs can be precast like precast sandwich wall panel, however, without the prestressing steel.
2. In comparison to other conventional structural sections, the sandwich panel has a high flexural-to-stiffness and strength-to-weight ratio. This is valid for SCIPs as well.
3. The precast concrete sandwich panel system can be constructed to achieve up to 100 percent composite action, depending on the ability of embedded connectors to transfer the shear generated by longitudinal flexures. This would not be possible for SCIPs due to limitation on the strength and geometry of the diagonals (steel trusses).
4. Under bending action, the load-deflection curves from the past research showed that SCIPs carry load as partially composite panels. In the linear elastic zone, the stresses and strength of each panel can be computed by linear elastic structural analysis. In the non-elastic region, the section behaves as a partially composite section, and the analysis should be performed based on strain distribution.

5. The design and arrangement of the shear transfer mechanism have a considerable impact on SCIP panel stiffness and deflections. The type of EPS also has an impact on SCIP panel rigidity but is generally neglected in calculations.
6. Precast SCIPs can be employed as load bearing walls and floors in housing systems.
7. There are various producers of SCIPs around the world. Most of the SCIP producers use sophistical machinery in a factor to produce panels.
8. MetRock (MR) panels are made from off-the-shelf components and can be assembled into panels up to 18 ft. in length and 4 ft. wide using a portal hydraulic jig press on-site.

Chapter 3. MetRock SCIP Construction Technology

3.1 Introduction

The MetRock SCIP (MR panel) construction technology is described in this chapter. This chapter outlines the specifics of how MR panels are made and how it may be used in the construction of a residential structure. An alternate precast technique for producing MR panels is discussed. The chapter aims to assist engineers with simplified calculations for the panel design (flexure design, shear design and axial design) based on common and widely adopted standards (e.g., ASTM, ACI). As mentioned in the previous chapters, MR panels are a modular form of SCIP. MR panels are generally transported as individual elements that can be assembled into panels using a portable hydraulic jig press and a pneumatic hog ring fastener, as opposed to conventional SCIP that are generally fabricated in production plants and transported as solid panels to the construction site (Gurung, 2019).

3.1.1 Product Description

An MR panel is made up of an Expanded Polystyrene Styrofoam (EPS) block core bordered on both sides by a 14-gauge galvanized wire mesh and linked by 3/16-inch galvanized diagonal steel wire trusses located every six inches. The insulating core blocks are six inches wide and four inches thick, with a length of up to 10 feet. After the panels are in place, a one-inch coating of concrete is applied to both sides of the assembly to create load-bearing faces (MetRock SCIP, 2019). MR panel bars are sandwiched between EPS blocks, hog ring ties are used to secure them to the mesh. MR panels were put through a series of full-scale tests to establish their structural capacity and feasibility for residential buildings. The tests were completed at Idaho State University's structural laboratory. Test results showed good performance of slab and wall specimens made of MR panels under out-of-the -plane bending, and in-plane seismic testing, respectively (Gurung, 2019).

3.1.2 Component

A welded–wire space frame is combined with a polystyrene insulated core in a standard MR panel. A galvanized diagonal steel truss structure holds the two layers of mesh together. The cores of MR panels are typically two to four feet wide, three to eight inches thick, and up to 18 feet long. They are made with a pneumatic hog ring tie and a portable hydraulic jig press. All of the components needed to make an MR panel are widely accessible and are off-the-shelf materials. A recycled EPS core, cold rolled galvanized steel mesh, and diagonal truss connection are the main components. In a sandwich construction, the EPS core serves two roles. For starters, it separates the two load-carrying skins. Second, the core offers the necessary insulation to the panels, lowering the amount of energy required for heating and cooling. The EPS core is utilized because of its low density, good resistance to temperature and moisture changes, durability, and chemical breakdown resistance over time. SCIP's tensile reinforcement is usually provided by the steel mesh. Cold-rolled galvanized wire mesh in a squares grid arrangement makes up the majority of mesh reinforcements. A spot weld is used to join the longitudinal and transverse wires. The wires of a typical MR panel’s mesh range in size from 14 gauge (0.08-inch diameter) to 11 gauge (0.12-inch diameter). Through the insulation layer, the diagonal shear truss system joins the two load-bearing concrete wythes. Concrete web, steel components, or plastic ties can all be used as connections. The components of MR panel are shown in Figure 2-2. Table 3-1 shows the material characteristics of SCIP elements in general.

Table 3-1: Material Properties

Element	Dimension	ASTM Standard	Yield Strength (psi)	Ultimate Strength (psi)
14-gauge (1.63 mm) galvanized wire mesh	1 in. × 1 in. (25.4 mm × 25.4 mm)	ASTM A1064 (2018)	72,910	82,232
shear connectors	3/16 in. (4.76 mm) galvanized steel wire truss	ASTM A951 (2011) ASTM A1-64 (2018) ASTM A641 (2019)	60,000	70,703

3.1.3 Fabrication

The insulating core of MR panels is Type I EPS. EPS is made by a variety of vendors throughout the world and meets ASTM C78 standards. The core has an average density of 1.0 pound per cubic foot and a modulus of elasticity of 180 psi, according to the product data sheet (Gurung, 2019). MetRock SCIPs are made from EPS blocks that are six inches wide and ten feet long, with a thickness ranging from three to eleven inches. Cores longer than 10 feet can be made by combining two EPS blocks. The block supplier should specify that the EPS blocks used for the MR Panels be "straightened". A one-by-one-inch, cold rolled, 14-gauge, galvanized wire mesh flanks the EPS core on both sides. The ASTM A82 standard can be used for the wire mesh. Furthermore, the wire mesh of MR panels is equipped with a patent screed system (Patent No.: US 8,122,622 B2) that facilitates the application of concrete. The wire mesh includes two specifically constructed screed ribs 12 inches off center on each side. The screed ribs ensure a consistent thickness of concrete is applied to the panel. The MetRock SCIP screed system is shown in Figure 3-1. The screed ribs must not line up horizontally with the truss wires when pushed into the wire mesh. The wire mesh will not be able to be attached to the truss where the screed rib is placed if the screed ribs line up with a truss wire. The screed ribs should be placed about 20 inches apart. To guarantee that the mesh achieves appropriate concrete embedment and cover, a half-inch gap between the EPS and the mesh is maintained. A 3/16-inch galvanized steel wire truss, usually referred to as "K-bars," holds the two layers of mesh together. The 120 Super Heavy Duty (SHD) Lox-all truss type wall reinforcement criteria were met by the truss connections utilized to construct the MR panels. The 120 trusses connectors are generally used for horizontal mortar joints in masonry walls, but they also work well as shear connections for the MR panels. The longitudinal shear stress is transferred between the two load-bearing sides via the diagonal bars. The shear trusses are sandwiched between two EPS blocks and are spaced every six inches. A pneumatic hog ring tie is then used to secure them to the mesh.

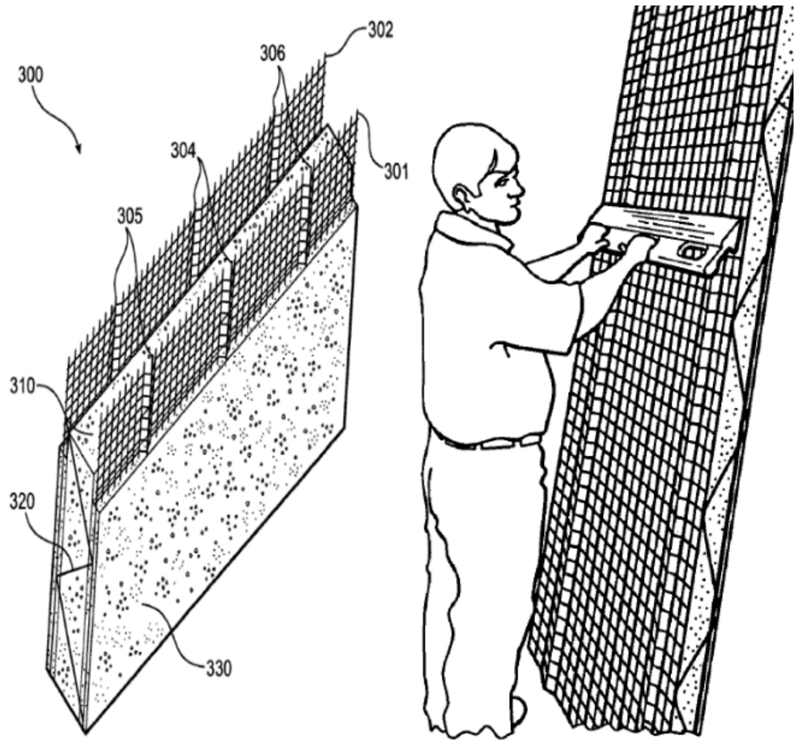


Figure 3-1: MetRock SCIP Screed System

To create a finished MetRock SCIP, a portable hydraulic jig press is utilized to join the EPS, steel mesh, and truss. A typical portable jig press may create panels up to 2-4 feet wide, 10-18 feet long, and 6-13 inches thick.

3.2 Construction Technology

This section introduces monolithic MetRock SCIP construction technology.

3.2.1 Monolithic Construction

The panel cores are transported to the building site on flatbed trucks before the work begins. Wider panels are occasionally made to suit door and window openings, although standard panels with a width of two to four feet are typically utilized for convenience of shipping and handling. The panels may be kept for weeks on a level surface after they arrive on the job site. Wind may be able to harm the panels if they become airborne, thus the panels should be sufficiently anchored. They should be kept out of the sun if stored for an extended length of time.

3.2.2 Foundation System

The installation of a strip footing is the first step in the MetRock SCIPs building process. Every 12 in., a starting bar (typically #3) is installed in the strip footings. If necessary, the starting bars can be spaced closer together. The starting bars are designed using the same techniques and formulae as the wall-to-footing connections for concrete masonry units. The starter bars are alternated between the inner and outer walls. Alternatively, holes can be bored in the footing parallel to the walls and the bars placed and grouted in place. Panel cores are inserted between the mesh and the EPS core once the footing is constructed, allowing the starting bars to slip in between them.

3.2.3 Erection of Walls

The building of walls must always begin at the corner since this ensures the rigidity of the structure.

3.2.3.1 Wall-to-Footing Connection

Standard quick-tie wires are used to bind the bars to the mesh at the wall-to-footing connection. These wires keep the panels from uplifting before the base coat is applied. To provide adequate connection strength, a minimum installation length of 18 inches is suggested, or a length corresponding to the development length from the building code. Pneumatic ties are used to fasten the adjacent panels together.

3.2.3.2 Openings

Openings for doors and windows can be cut before or after the panels are installed. These modifications to the panels could be made using regular hand-saws. The apertures must be designed in such a way that the structural integrity of the building is not endangered. To avoid cracking once the concrete skin is placed, an 8 in. long × 6 in. wide rectangle piece of cover mesh is generally installed on any door or window corner. The illustration is shown in Figure 3-2.

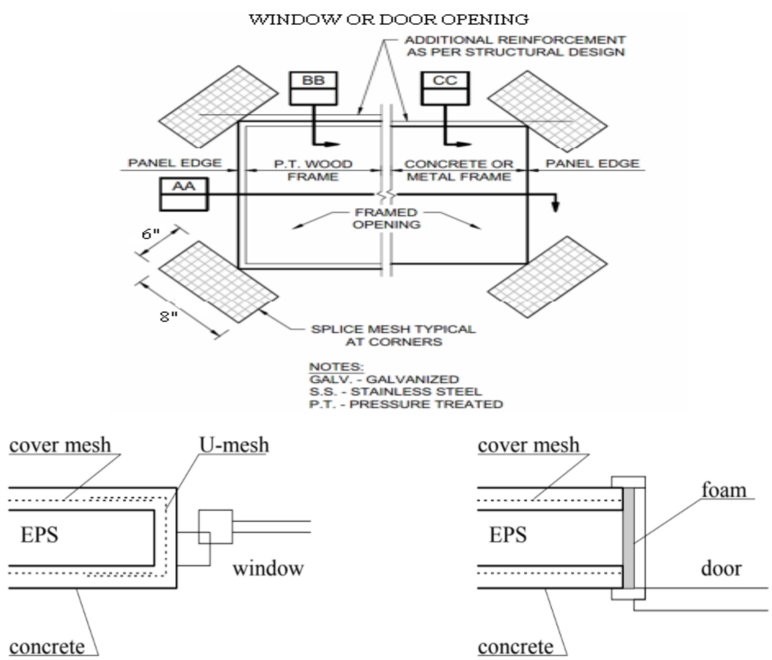


Figure 3-2: Opening Details

A MetRock SCIP surface is similar to that of a regular concrete wall. The windows and doors can be installed in the same way as they would be in a brick or concrete structure. If the aperture is not built of concrete, a wood frame the same width as the wall must be utilized and secured with compression-resistant foam.

3.2.3.3 Reinforcing Details

At the splices and corners of two panels, overlapping mesh reinforces the splices and seams. For rough apertures for doors and windows, extruded PVC material can be utilized instead of treated timber. If the rough entrance is made of wood, it is vital not to allow it to get wet until the concrete skins have been placed, because wet wood can distort. To keep the corners and edges of the walls contained, special L and U-shaped mesh is utilized. Utility conduits can also fit between the foam and the wire mesh; if more room is needed, sections of the foam can be chopped away or burnt. Manifold Pex water systems are recommended so that there are not water connections located inside the concrete skins. All water connections (from the water line to the fixture and the manifold) must be outside the concrete skins. Splicing the panels would require reinforcing rebars in addition to splice mesh.

3.2.3.4 Bracing

The needed bracings are given to the wall and floor panels after all of the panels have been installed. The walls are braced by diagonal bracing.

3.2.3.5 Concrete Application

The wet method, where all of the water has been added to the mix design prior to loading the material into a concrete pump, is commonly used to apply concrete. To apply the concrete skins to the panel, a 16 CFM air compressor capable of producing up to 100 psi may be utilized to spray them at a low velocity. This is quite similar to how one coat of stucco is applied. The concrete thickness for MetRock construction typically ranges from 1 to 1.25 inches. The ACI 506R "Guide to Shotcrete" should be followed for all shotcrete mix designs and applications. In addition, shotcrete can be replaced by hand application. The water-to-cement ratio for a typical dry or gunite mix should be in the range of 0.15-0.20, and the 28-day compressive strength should be in the region of 3000-4000 psi. To improve specific characteristics, admixture can be added in the shotcrete or wet spray up mix design. Air entertainers, for example, are frequently employed in the wet spray up process to minimize freeze thaw damage, enhance workability, and reduce rebound. For the wet spray up procedure, the water to cement ratio should be around.45-.50. Spraying the walls with a dry or gunite method is good but spraying overhead with a wet spray up procedure is preferred.

3.2.3.6 Erection of Slabs

3.2.3.6.1 Placement to avoid the need to spray overhead

The roof slab cores can be added after all of the wall panels have been set and fastened. The added safety will be provided by the L-shaped hair pins placed on both sides of the panel, which are quite robust.

3.2.3.6.2 Shoring

To shore the slab, variable-height props (Figure 3-3) are preferred. These props must be supported by girders running perpendicular to the slab panels. The flexural resistance of the panels must be considered while determining the distance between the support rows. The panels are strengthened with the following components, which are already installed on the floor, to make work easier.

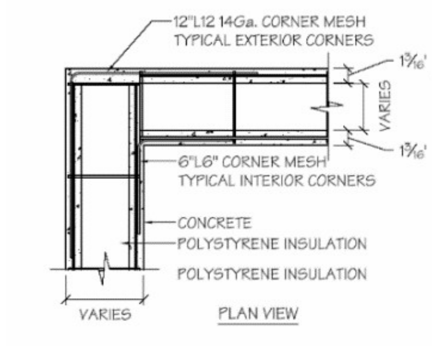
- Additional reinforcement (bars) at the bottom.
- Splice mesh at the bottom (on one side).
- U-shaped stirrups at the support.

The panels are then physically raised and fastened to the slab with tie wire. The panels also have splice mesh on the top side to prevent fractures on the top side of the slabs above non-load bearing walls.

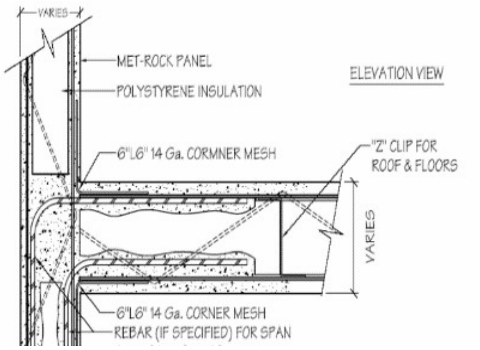
Figure 3-3: 3D Slab Ready for Concrete Application

3.2.3.6.3 Slab-to-Wall Connection

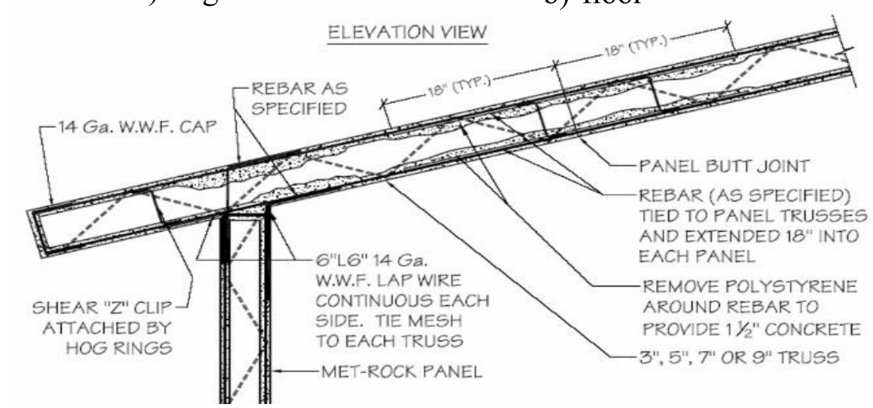
The design engineer should provide specific strengthening features for the connections between the wall and the slab. An example connection detail utilized in SCIP construction is shown in Figure 3-4. In addition to splice mesh, additional strengthening in the form of hairpins is always suggested. The fundamental concepts of reinforced concrete may be used to create the hairpins. A slab-on-grade home has no basement, no basement walls, and is built on a single slab of concrete. They are not appropriate for all construction locations. As a result, adequate site preparation is required at all times.



a) Edge



b) floor



c) Wall to roof

Figure 3-4: Typical Connection Details for SCIP Construction

3.2.3.6.4 Concrete Application

The wet spray up method is commonly used to apply concrete to the bottom sides of the slab. Typically, pumped concrete is used to cover the top side of the slab. In all circumstances, the weight of the concrete on each side of the panel should not exceed 15-19 pounds of concrete per square foot of panel. After the concrete skins have been placed, the panel's overall weight should be around 30 pounds per square foot. Figure 3-5 shows the typical construction sequence using SCIPs.

Figure 3-5: Typical Construction Sequence for a Residential House using SCIP (Gurung, 2019)

3.2.4 Load Bearing vs. Non-Load Bearing Elements

All of walls in a typical MetRock SCIP building are load-bearing walls. This distributes the gravity loads over all of the walls, substantially reducing the amount of gravity load on a single portion. MR panels can be used as partition or non-load bearing panels.

3.2.5 Application of Concrete

The concrete skins may be applied in the field using either the dry shotcrete gunite technique or the wet spray up process. The “Guide to Shotcrete,” produced by the ACI 506 Committee and included in the 506R publication, is suggested as a guide for the dry process (Gurung, 2019). The dry method requires the use of ASTM C33 concrete sand with a moisture content of less than 5% by weight, a minimum of 20% cement by weight, and no more than 5% by weight of fly ash as a cement replacement in the mix design. The wet spray up method is also known as a one-coat stucco process in the trades. For the wet process, concrete modifications such as water reducers and air entrainers are advised. The ASTM C33 standards for concrete sand should be followed in the mix design. The cement content, or the cement plus fly ash content, must be at least 20% by weight.

As mentioned before, a screed pushed into the wire mesh is used in MR panels. This screed is used as a visual depth screed to show the nozzleman that 1in. of material has been put to either side of the panel. The operator may let the cutting blade ride along the leading edge of the screed rib and cut the concrete material straight and flat extremely fast using a "cutting rod" or blade that extends from the leading edge of one screed to the next screed. Allowing the material to begin the setting process after it has been applied is critical before the operator begins cutting the material flat and straight. This is true for both the dry and wet processes. For both the dry and wet processes, the 1 in. material per side of the panel may be applied in a single pass.

3.2.5.1 Wall panels

Exterior wall panels receive the first coat of concrete, followed by inner walls.

3.2.5.2 Floor slabs

On the bottom side of the slab, the first layer of concrete is placed. At the very least, this concrete layer extends to the slab's borders. The concrete is then poured on top of the slab.

3.3 Alternative Precast Modular Construction

An alternative precast technique can be utilized to generate full-scale slab and wall panels instead of the traditional shotcrete or cast-in-place concrete procedure. For precast modular MR Panels, the construction technology for producing laboratory specimens at Idaho State University and concepts for real-life connections are discussed in the subsequent sections. These concepts can be used for construction of actual precast SCIPs made of MR panels.

3.3.1 Foundation System

A precast bed is created for the specimen construction. The walls of the precast bed were built modular using 0.75 in. (19 mm) thick plywood and 1.5 in. (38.1 mm) by 3.5 in. (88.9 mm) Douglas-fir timber so the span of the beds could be changed. A plastic liner is used to extend the life of the bed and prevent concrete leaks.

3.3.2 Precasting of Walls

3.3.2.1 Concrete Mixture

The ACI absolute volumetric technique is used to develop and create a Self-Consolidating Concrete (SCC) mix. Type I Portland cement with Navajo fly ash which is one of the better pozzolanic fly ashes in the US., is utilized for the cementitious ingredient. The fly ash influences the concrete's ultimate strength. The aggregate is made up of crushed sand and extremely fine pea gravel. Two lifts are used to apply the concrete layers for the specimen. The lowest layer of the concrete bed is poured first. To produce a consistent one-inch layer, a hand trowel is utilized to distribute the concrete. To guarantee a consistent layer of concrete before mounting the panel, a one-inch depth indication is utilized.

3.3.2.2 Construction Sequence

The MR panel is installed into the precast bed after the bottom layer of concrete has been poured. The core is physically raised and pushed into place, with enough pressure and lateral movement used to produce a homogeneous one-inch bottom layer. Following the placement of the panel within the precast bed, the top layer of concrete is poured in the same manner as the bottom layer. After enough SCC has been poured on top of the panel, hand trowels are used to distribute the concrete uniformly. The concrete is covered with a plastic lining. A moist burlap is used to cure the specimen for three days within the bed. Before removing the panel from the bed, the strength of the concrete is tested, and the panel is then brought to the curing rack, where it is lifted up on its side using a long spreader steel beam and construction grade straps. The specimens were cured for 28 days using damp burlap that was wrapped in plastic wrap.

3.3.2.3 Wall-to-Footing Connection

A socket footing is designed and constructed to provide the fixed connection required for a cantilevered wall specimen. The details for socket footing are given in Figure 3-6.

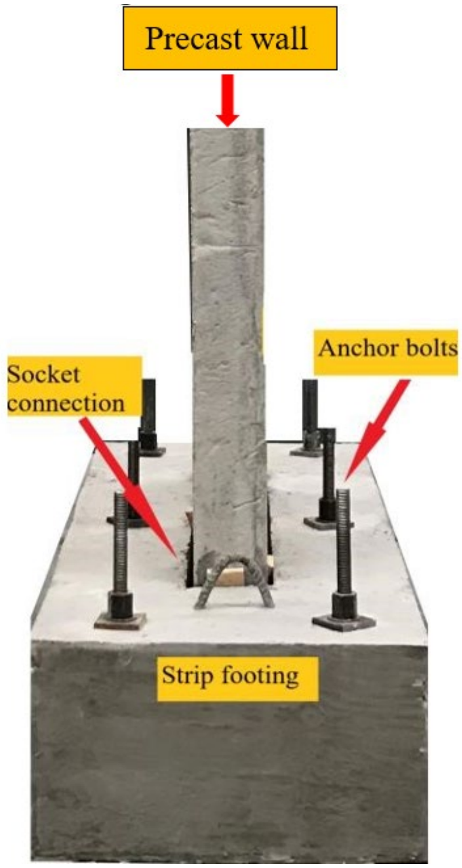
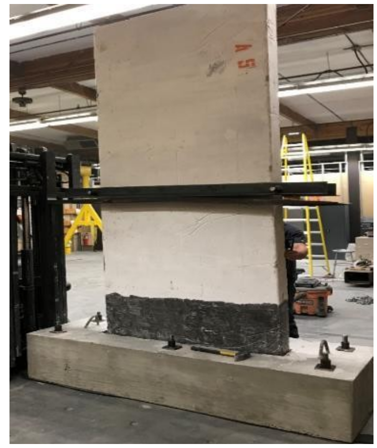


Figure 3-6: Details for Socket Footing

From the precast bed to the structural lab, the socket footings are carried. Then, using a tilt-up mechanism, they are fastened to the sturdy floor and raised vertically. The panels are put into the socket footing after they have been constructed, clamped, and raised. Dayton 1107 high-strength, non-shrink grout is then used to grout the panels in place. The assembling procedure is shown in Figure 3-7.

a) Clamping of panel



b) Placement of the panel

c) Mixing of the grout

d) Application of the grout

Figure 3-7: Assembly Process (Gurung, 2019)

3.3.3 Precasting of Slabs

The sequence for the precasting of slab is outlined in the following subsections.

3.3.3.1 Concrete Mixture

For slab panels, the concrete composition and construction sequence could be the same as for wall panels. However, the connections would be different. Some concepts for the connections are presented below.

3.3.3.2 Slab-to-Wall Connection

After all the wall panels have been fitted and fastened, the floor and roof slab cores are installed. The design engineer should specify the connections between the wall and the slab.

3.3.4 Concepts for Construction of Modular Units

Modular construction is a novel method of building that involves constructing a structure away from the construction site, curing it in a controlled environment, transporting it to the construction site, and installing it. The materials utilized, as well as the design regulations and standards, are identical to those used in traditional construction. Precast MR panels can be used for construction modular units such as residential and correctional facilities. The precasting of the whole unit or individual panels can be done in a prefabricated yard. This is discussed below.

3.3.4.1 Prefabrication Using Shotcrete

The water-to-cement ratio for a typical dry mix should be in the range of 0.3 to 0.5, and the 28-day compressive strength should be in the range of 3000-8000 psi. To improve specific characteristics, admixtures can be included in the shotcrete mix design. For example, air entertainers are frequently employed in shotcrete to minimize freeze-thaw damage, enhance workability, and reduce rebound. For a modular application of MR panel, the first step is to cast the bottom side of the floor panel and bottom side of the roof panel. The next step is to put up the four walls and the floor. However, because the roof already has concrete on the bottom wythe, spraying just inside of the four walls is advised before joining the top floor to avoid relying solely on the strength of the EPS core of the walls. The outside wythes of the four walls can then be sprayed, providing sufficient strength and the choice to cast or spray the top wythe of the top floor. This avoids the need to spin the module 90 degrees in order to spray at a 90-degree angle to the target, which is critical. It also removes the need to spray above, which is extremely difficult in modular construction. SCIPs modular homes can be constructed in a precast yard and then transported to the site as whole or partial unit.

3.3.4.2 Transport

After concrete is applied and cured, the precast unit can be transported to the site. Some concepts for the details of the connection for modular construction are illustrated in Figures 3-8 to 3-12.

3.3.4.3 On-site Assembly

Appropriate connection details for the wall-to-foundation and wall-to-slab are identified. Using these connection details, all the walls and slabs are assembled. Using the corner mesh (L-shaped mesh), one side of the "L" can be cast into the bottom side of the floor and roof, leaving the other side connected to anchor the floor and roof into the walls. Only the top side (where cover mesh is used) is used to link the floor and roof tops in this case. The underside of the floor and roof will have a seam.

3.3.4.4 Connection Details

3.3.4.4.1 Connection of Wall to Footing with Headed Studs

Since the SCIPs panels can come up to 18 ft long pieces, 1 ft off the EPS core can be cut from the panels and replaced with 1ft of solid concrete instead. This 1ft section shall be utilized to have angle iron welded to a #3 rebar (0.375" diameter (Ø)). It should be ensured that #3 rebars have enough embedment length. In the foundation, a base plate with four headed studs shall be pre-installed.

The connection of wall-to-footing using headed studs shall comply with PCI Design Handbook, Precast and Prestressed Concrete (7^{th} Edition)-Chapter-6, 6.4.1 and 6.5.

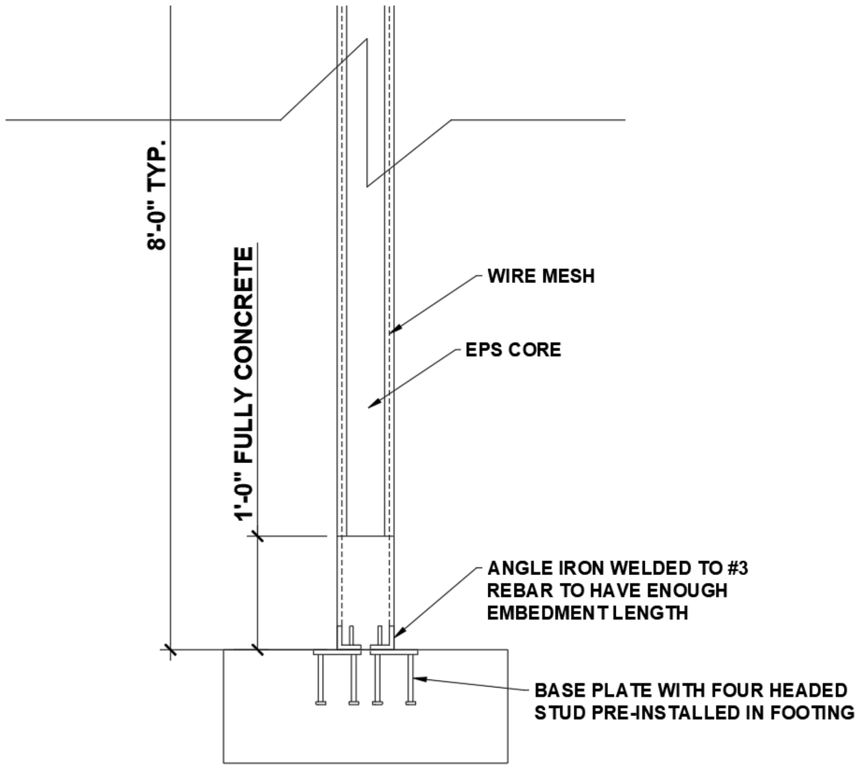


Figure 3-8: Connection of Wall-to-Footing with Headed Studs

3.3.4.4.2 Connection of Wall to Foundation with U-Channel

A U-Channel with a thickness of (0.6 – 0.8) mm shall be used. U-shaped splice mesh shall be used under the U-channel and the mesh shall have enough development length. The concrete screw can be used to clamp U-channel to the footing.

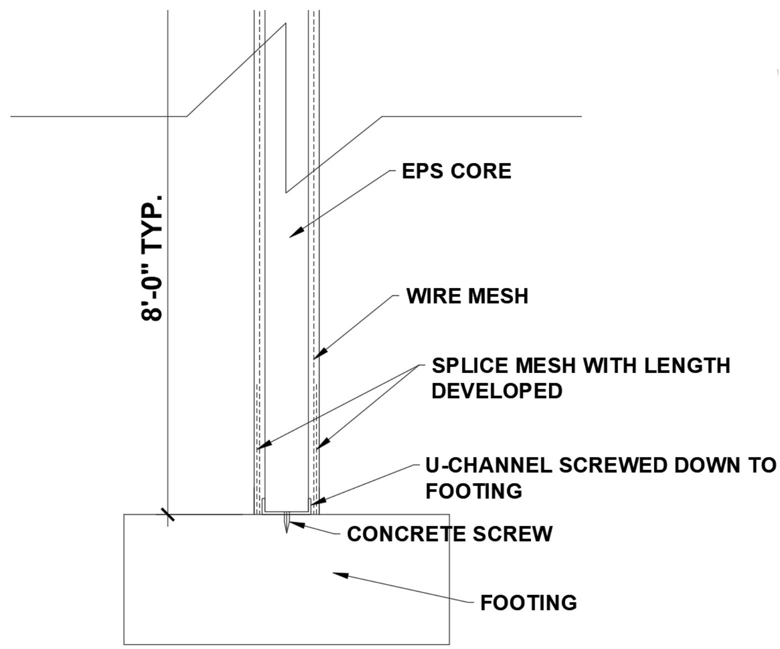


Figure 3-9: Connection of Wall-to-Footing with U-Channel

3.3.4.4.3 Connection of Wall to Foundation with Starter Rebar/Dowel

The dowel shall have spacing of at least the thickness of the EPS core, i.e., about 4 in. The dowel/starter bar shall have enough development length both below and above the footing. Splice mesh also shall be used above footing on the wall to the length of the dowel embedment to provide more resistance against cracking.

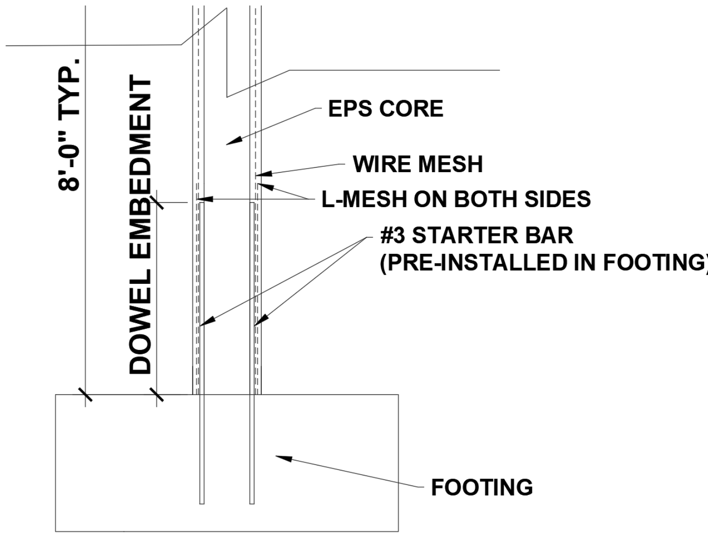


Figure 3-10: Connection of Wall-to-Footing with Starter Rebar/Dowel

3.3.4.4.4 Connection of Wall to Foundation with Socket Footing

Footing shall have opening of 7 in. wide and 15 in. deep for a typical 8 ft long panel. Once the panel is placed inside the opening, they shall be grouted using high-strength, non-shrink grout.

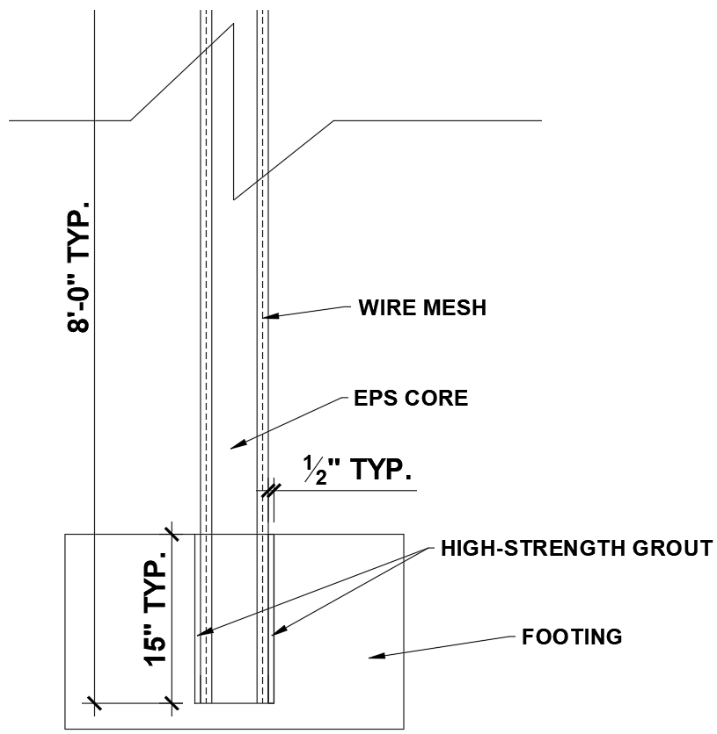


Figure 3-11: Connection of Wall-to-Foundation with Socket Footing

3.3.4.4.5 Connection of Wall to Slab

#3 rebars (0.375" Ø) shall be used 1-ft center-to-center to have connection between slab and the wall. The rebar used shall have enough development length both inside the slab and the wall panels. L-mesh shall also be used alongside the rebar for more confinement and resistance against cracking.

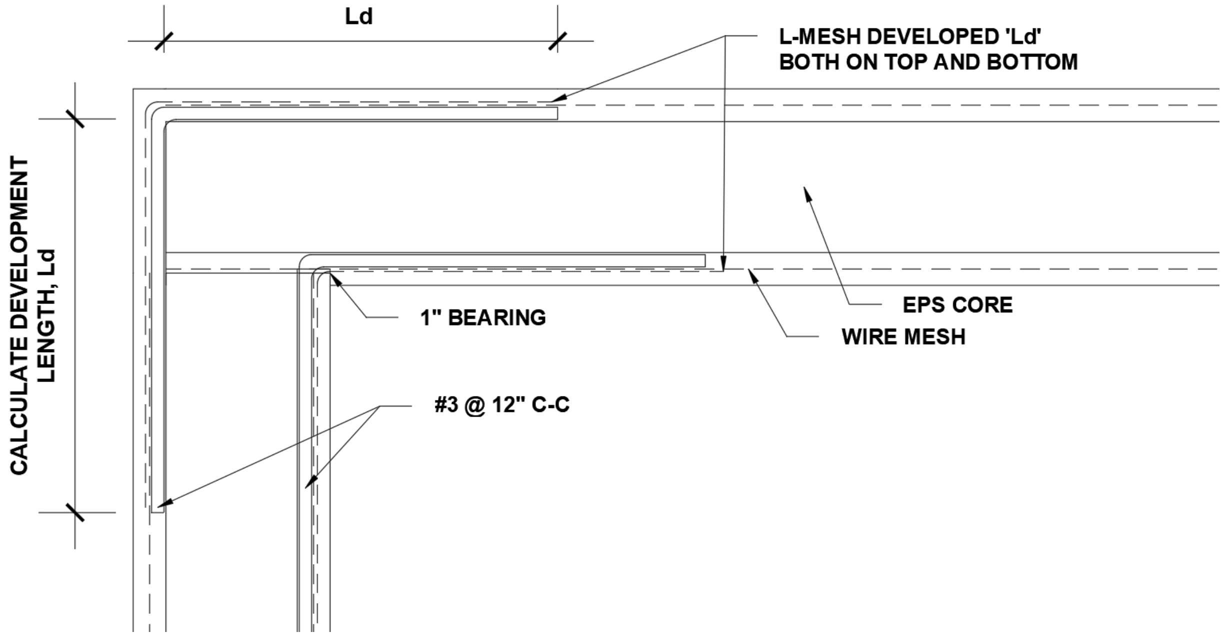


Figure 3-12: Connection of Wall-to-Slab

3.3.4.5 Limitations

1. **Transportation issue:** The construction site may be located some distance from the precast concrete factory. In such scenarios, trailers can be used to transport the precast SCIP elements to the job site. In many situations, the lower cost of precast concrete is offset by the higher cost of transportation.
2. **Modification limitation:** It is difficult to change the construction of a precast SCIP structure. For example, dismantling a structural wall for alteration will have an influence on the structure's overall stability.
3. **Handling Difficulties:** When handling precast concrete, extreme attention and care must be exercised. Precast SCIP panels can be heavy and massive, making them difficult to handle without causing damage. Precast members are often handled with portable or tower cranes.

3.4 Simplified Analysis of MR Slabs

In previous research at Idaho State University (Gurung, 2019), several slab specimens made of precast MR panels were tested to failure in order to establish their ultimate load capacity, yield moment capacity, failure mechanisms, and maximum deflection. The specimens were tested under four-point bend test which corresponds to a transverse out-of-plane stress of the MR panels. The ASTM E72-05 “Standard Testing Method of Conducting Tests of Panels for Building Construction” was used to conduct the flexural tests (2005, ASTM E72). Experimental results are used to provide simplified analytical procedures for flexural design of MR panels. Shear and axial design considerations are also discussed. It is important to note that slabs made of SCIP should be analyzed as one-way slabs.

3.4.1 Flexure Design

The flexural strength can be determined according to ACI 318 (ACI, 2019) requirements. Some guidelines are discussed below.

The slabs are evaluated as a simply supported component that resists an out-of-plane bending moment caused by its own weight and the normal loads applied to it. Slabs are often subjected to flexural loading in the form of concentrated or distributed loads. The bottom wythe is considered to be in tension and the upper wythe to be in compression for this study. The upper wythe is likewise believed to have withstood all compressive pressures, while the lower wythe’s reinforcing is assumed to resist all tensile loads.

3.4.1.1 Splicing Details

3.4.1.1.1 SCIPs Without Additional Splice Reinforcing

The short-span (10 ft) MR panels do not have to be spliced due to their shorter length. However, the medium and long-span cores will have to be spliced using flat mesh on each side of the panel as well as staggering the EPS blocks as shown in Figure 3-13.

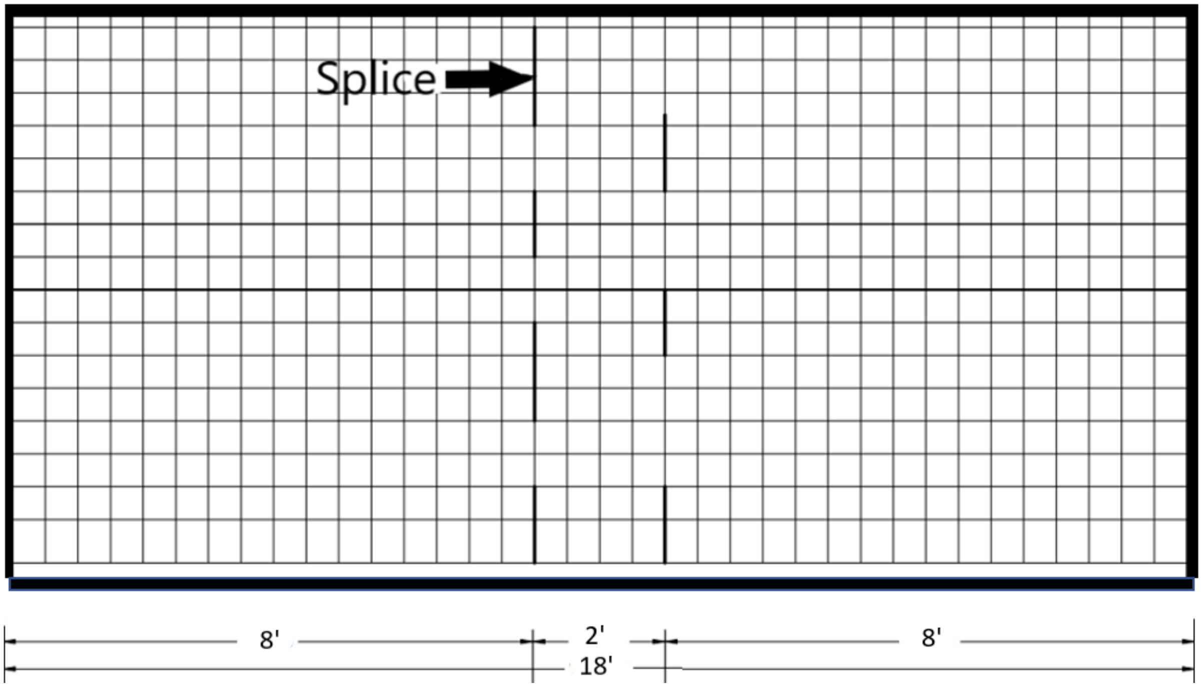


Figure 3-13: Plan View of a Long-Span MR Panel

3.4.1.1.2 SCIPs With Additional Splice Reinforcing

SCIP specimens can have extra reinforcement in the splice zone in the form of grade 60 (413 MPa) # 3 (9.5 mm) rebars. The additional quantity of reinforcement necessary to create a sufficient splice is calculated using the short span specimen's ultimate moment capacity. To calculate the extra reinforcement needed, panels are assumed to have a completely composite section. To avoid pullout, the rebars are extended for the entire development length as specified by ACI 318 (2019).

3.4.2 Shear Design

The shear forces are estimated using conventional structural analysis for shear design of slab panels. Experiments were conducted to verify the suitability of the shear design technique mentioned below. Shear stresses are resisted in an MR slab panel by a combination of two concrete wythes and steel shear connecters (truss diagonals). The contribution of the top and bottom wire meshes and EPS core can be ignored. Concrete shear contribution can be computed similarly to one-way reinforced concrete slabs. Shear contribution from diagonals can be done in accordance with the technique outlined below. For this purpose, the diagonal's buckling load is of high. The buckling point of a diagonal can be taken as the highest shear contribution as well as shear failure load for the slab panel. This may provide conservative results which is suitable in this scenario until further testing data becomes available.

For calculation of concrete shear contribution (V_C), ACI 318 (ACI, 2019) 22.5.5.1 provides:

$$V_c = 2\lambda\sqrt{f'_c}\, b_w\, d \tag{3-1}$$

Where,

b_w = web width (in.)

d = depth (in.)

f'_c = Specified compressive strength (psi)

λ = Modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength.

The total shear force contributed from concrete:

$$Vc = Vc1 + Vc2 \qquad (3\text{-}2)$$

Where,

V_{c1} = Top layer shear capacity

V_{c2} = Bottom layer shear capacity

If both layers are of similar thickness, then total shear force contribution from concrete is:

$$Vc = 2 \text{ x } Vci \qquad (3\text{-}3)$$

Where,

V_{ci} = Typical shear capacity of a one layer

The mesh and longitudinal truss bars make up the panel reinforcing system. Cold rolled diagonal shear connectors are commonly used to join the two sides of SCIPs. In most cases, the connections are built within the EPS core. The bars in the MetRock SCIP are sandwiched between the EPS blocks and secured to the mesh by a hog ring, as illustrated in the diagram below.

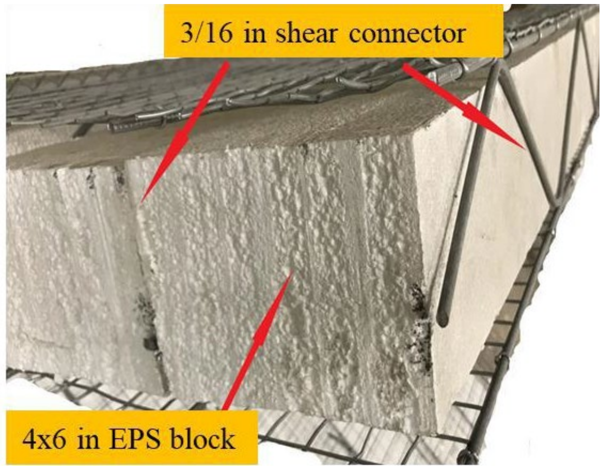


Figure 3-14: Diagonal Connector for MetRock SCIPs (Gurung, 2019)

As mentioned previously, the diagonal trusses are sandwiched in between the EPS blocks and are attached to the mesh with a hollow ring tie every six inches. After that, a thin coating of self-consolidating concrete is applied on both sides to complete the assembly. Similarly, the two layers of concrete and the diagonal steel connections primarily withstand all shear loads.

The general shear design procedure for one-way concrete slab can be used for shear design of MetRock SCIPs. The shear capacities can be calculated according to ACI 318 (ACI, 2019).

The shear strength at a section of a member (V_i) is the sum of concrete strength "V_c" and reinforcement strength "V_s".

$$Vi = Vc + Vs \tag{3-4}$$

Where,

V_c is the contribution of the concrete to shear strength.

V_s is the contribution of shear reinforcement

The trusses (diagonals) in MR panels withstand shear forces in terms of compression and tension forces. While the tension force in the brace may not be a concern, the bracing (diagonals) length for thicker slabs would be greater, making them prone to buckling. That is why the Euler buckling load (Pc) for the diagonal/brace under compression is very important. Once the buckling load is determined, it can be converted to an equivalent V_S for the slab, depending on the number of the diagonals and trusses available in the panel.

3.5 Axial Design

For a typical MR SCIP construction, all the walls in the building can be load-bearing walls. This causes the gravity load to be distributed among all the walls, which significantly reduces the amount of gravity load applied on an individual section. Axial loads are primarily resisted by the concrete wythes. The axial contribution of trusses, side wire meshes, and EPS core are negligible. The ultimate compression capacity of each wall is simply the concrete strength multiplied by the section area of the concrete.

$$Pf = f'c \times Ag \tag{3-5}$$

Where,

P_f = axial failure load

f'_c = Concrete strength

A_g = Section area of concrete

If the wall is too tall, it may buckle first before failing in compression. In this scenario, a more detailed buckling analysis should be performed.

Note: It is recommended to keep the axial load on a MR panel limited to 3-5% of $f'_c \times A_g$.

3.6 Design for Lateral Loads

3.6.1 Lateral Design of MR Wall Panels

The most effective way to resist lateral loads in a SCIP structure is to use the wall panels as shear walls. To resist lateral loads, perimeter walls, including some internal walls, can be designed to act as shear walls. In most cases, seismic loads are larger than wind loads for MR panel construction. Material properties, thickness of wythes, location of side meshes, and composite action between the two wythes determine the in-plane lateral shear resistance of MetRock SCIPs. In general, sandwich wall panels should be designed and constructed as either cantilever or fixed-end structural components, depending on the connection details and number of floors. The stiffness of each wall panel determines how much of the overall lateral force it attracts. Because of the volume change associated with temperature fluctuations, connecting many wall panels to make a lengthy perimeter wall might result in an unwanted buildup of forces which can cause cracking. Instead, it is recommended to limit the number of the connected wall panels or provide expansion joints if the length of the perimeter wall exceeds 100 ft or more.

For seismic analysis of SCIPs, Mashal and Filiatrault (2012) used the FEMA P695 Methodology to obtain seismic performance factors for SCIPs. The FEMA P695 Methodology is intended for new structural systems and offers a logical way of assessing seismic performance factors (SPFs) such as the response modification coefficient (R-factor), system overstrength factor (0), and deflection amplification factor (C_d). FEMA recommends that a response modification coefficient (R-factor) of 3.5, a system overstrength factor (0) of 3.0, and a deflection amplification factor (Cd) of 3.5, can be used for calculations of seismic loads on structures made of SCIPs. Further seismic analysis and testing is required to develop guidelines and quantify seismic performance factors for MR panels.

Chapter 4. Experimental Investigation on Shear Behavior

4.1 Introduction

To identify shear failure mechanism and capacities for MR slab panels, it is essential to conduct large-scale testing. For this purpose, two specimens made of precast MR panels were considered for experimental testing. The specimens were constructed using a precasting approach, as discussed before.

4.2 Construction of the Specimens

Two MR panels were prepared and evaluated according to ASTM E72 standards. The specimens featured edge confinements to prevent buckling of the end diagonals. Figure 4-1 shows cross-section details of the MR slab specimens utilized in this experiment. The edges of the panels were confined with a half-inch of concrete and a U-mesh. The edge confinement was employed to prevent premature failure of the panels as shown in Figure 4-2.

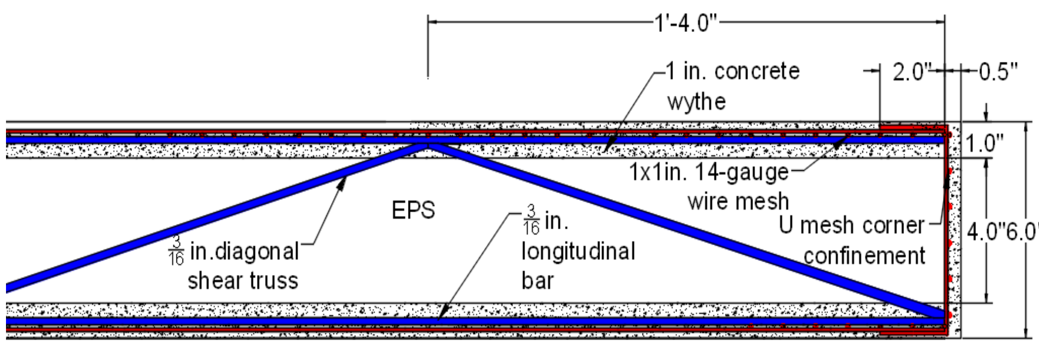


Figure 4-1: Cross-section of a MR SCIP slab specimen (Gurung, 2019)

Figure 4-2: Enclosure at the Edge of the Wall

In construction of the specimens for the shear testing, a 1 in thick concrete was applied on each side of the panel. The concrete layers were applied in two lifts. The bottom layer was the first to be poured onto the precast bed. The concrete was distributed with a hand trowel to produce a consistent one-inch layer. Before placing the panel, a one-inch depth indication was also utilized to ensure a consistent layer of concrete. The MR panel was put into the precast bed after the bottom layer was poured. The core was physically raised and placed into position. Adequate pressure and lateral movement were required to produce a consistent one-inch bottom layer. The top layer of concrete was poured in the same manner as the bottom layer once the panel had been correctly positioned inside the precast bed. Hand trowels were used to uniformly spread the concrete once sufficient SCC was poured on top of the panel. A moist burlap was used to cure the specimen within the bed for seven days. Before removing the panel from the bed, the concrete strength for two specimens was tested. For 28 days, the specimens were wrapped in wet burlap and covered with plastic wrap. The construction process is shown in Figure 4-3.

a) Pouring the Bottom Layer

b) Pouring Top Layer of Concrete

c) Finishing the Top Layer

d) Final Smooth Layer

Figure 4-3: Construction of MR panels for shear testing

4.3 Material Properties

Some of the material properties of MR panels are given in subsections below.

4.3.1 Steel Reinforcement

Reinforcing in a typical MR panel consists of two main components: 14-gauge, cold rolled, galvanized steel wire mesh and 3/16-inch longitudinal steel bars. A four-inch-thick EPS insulation core separated the two reinforcing layers, which were held together using shear trusses. A one-inch square grid design was used on the 14-gauge wire mesh. To ensure that the mesh acquired appropriate embedment and cover, a half-inch gap was maintained between the mesh and the EPS core. Figure 4-4 depicts reinforcing for the MR panel.

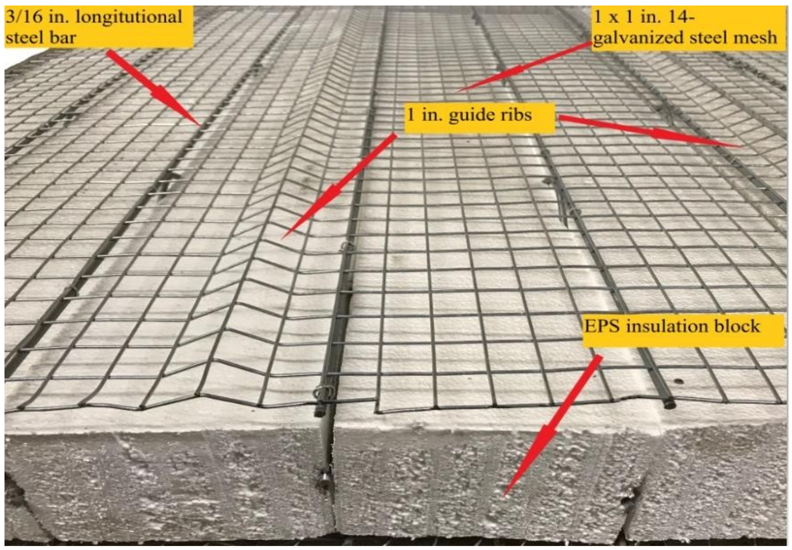


Figure 4-4: Details of Tensile Reinforcement for MR SCIP (Gurung, 2019)

4.3.2 Expanded Polystyrene Styrofoam (EPS) Insulation Core

The MetRock panel is insulated with Cellofoam Type I EPS insulation blocks. The panels for this experiment were made of four-inch-thick, six-inch-wide, and ten-foot-long EPS blocks. Then, the panels were cut into two panels for the experiment. The average density of the EPS core is 0.95 pounds per cubic foot, with a thermal conductivity K factor of 0.24 and a thermal resistance R-value of 4.12 pounds per inch (ASTM C 177). The EPS core has a modulus of elasticity between 180 and 220 psi and shear modulus between 280 to 320 psi. The EPS core has a four percent absorption rate and is fungal and bacterium resistant (Gurung, 2019).

4.3.3 Concrete

The Self-Consolidating Concrete (SCC) mix design for the panels is summarized in Table 4-1 below. Using these measurements, 15 batches were made for the application of concrete in two panels.

Table 4-1: Mix design used for precast MR panels

Material	Volume (ft^3)	Weight (lb)
Air		
Cement	0.165	32.4
Fly Ash	0.056	8.14
Fine aggregates (FA)	0.452	76.6
Coarse aggregates (CA)	0.248	36
Water	0.259	15.8
Total	1.18	168.94

During construction of the specimens, six concrete cylinders were cast for compression tests. The cylinders are shown in Figure 4-5. Three cylinders were tested at 7 days of curing and the other three cylinders were tested at 28 days.

Figure 4-5: Six cylinders for compression test

The results of the test at 7 and 28 days are presented in Table 4-2.

Table 4-2: Summary of concrete compressive strength for the specimens

Compressive Strength						
Sample	Diameter (in)	Height (in)	Area (in²)	Max load (lbs)	Compressive strength (psi)	Days
I	3.99	8.09	12.52	32593	2602.77	7
II	4.01	8.08	12.62	32949	2611.53	7
III	3.99	8.09	12.53	32395	2585.70	7
Average					2600	
IV	4.01	8.10	12.64	34046	2694.46	28
V	4.03	8.09	12.76	35865	2810.28	28
VI	4.01	8.08	12.62	36528	2895.23	28
Average					2800	

4.4 Testing Arrangement

The testing setup for the two MR slab specimens is described in this section. The ASTM E72-05 “Standard Testing Method of Conducting Tests of Panels for Building Construction” was used to conduct test for this experiment (ASTM E72, 2005). All tests were conducted horizontally where the slabs were simply supported in between two rollers and were loaded using a concentrated load at 12.5 in and 13 in distance from the edge of the panel. The tests were conducted at different distances for each panel because they were of different lengths. This was done to exert maximum shear without considerable bending in the slab specimens. Figure 4-6 provides the schematics for the test setup. The actual test setup is shown in Figure 4-7.

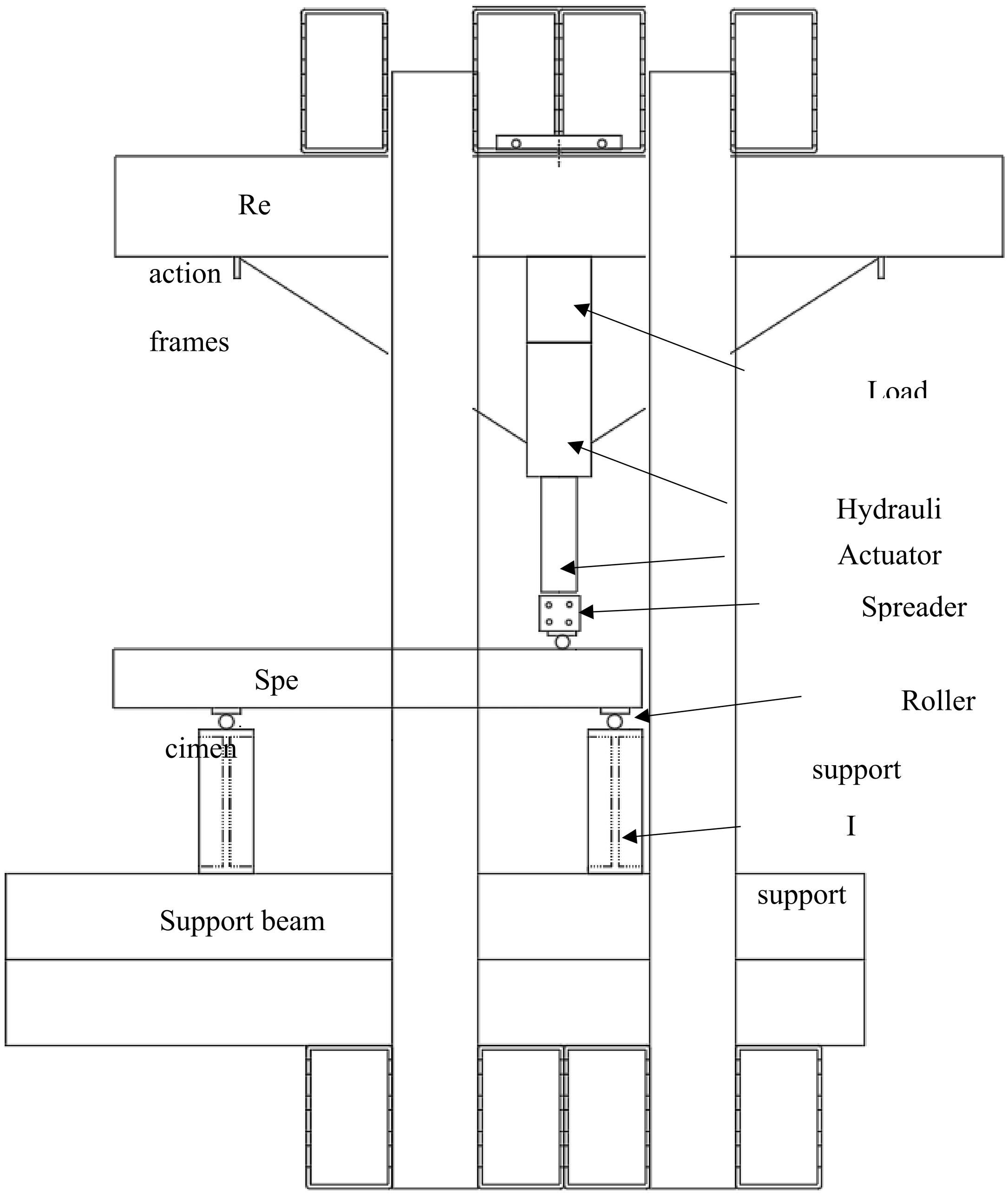


Figure 4-6: Test Setup Details for panel 1

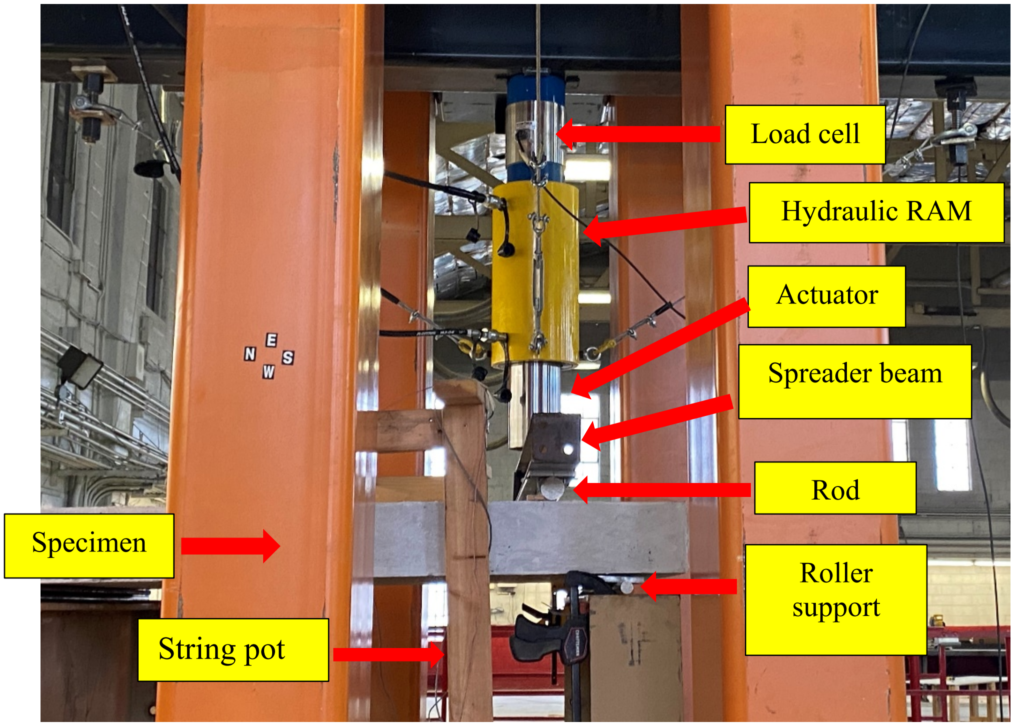


Figure 4-7: Test Setup for Shear Testing (East Side)

In Figure 4-7, above the panel specimen, there is an element called a spreader beam which exerted loads on the rod under. The rod stretched across the width of the specimen. A hydraulic ram exerted forces on the spreader beam which was then transferred on top of the panel. There was a load cell placed along the load path to measure forces during testing.

After curing, the specimens were moved into the structural lab on carts and were lifted on to the supports using a spreader beam and a forklift. After that each specimen was set in place for testing. Two string potentiometers (pots) were attached one on each side of the panel where the loading rod is located. Two more string pots were attached at the center (2 ft from the support) on each side of the panel as shown in Figure 4-8.

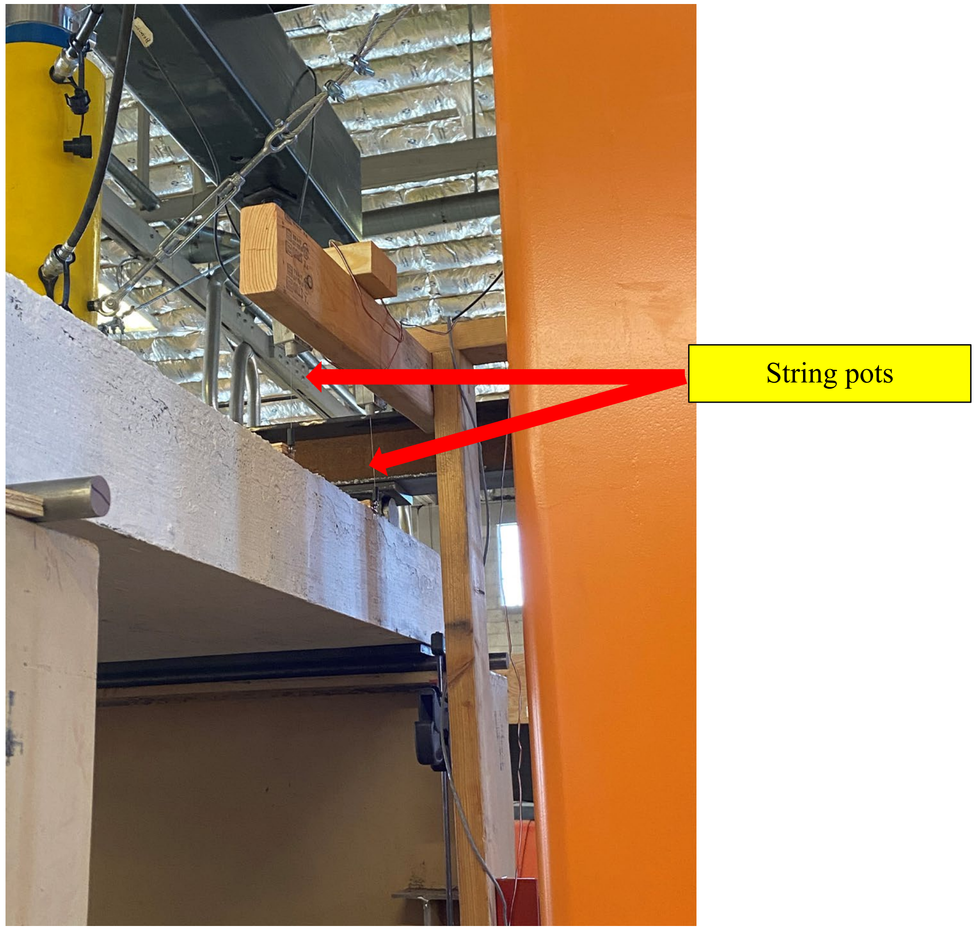


Figure 4-8: String Pots Attached on the Specimen

The slab specimens were all orientated horizontally and seated on a strong steel beam support using a one-inch-diameter roller. To prevent bearing failure, a half-inch steel plate was placed between the specimen and the rollers. The details for the end support conditions are provided in Figure 4-9.

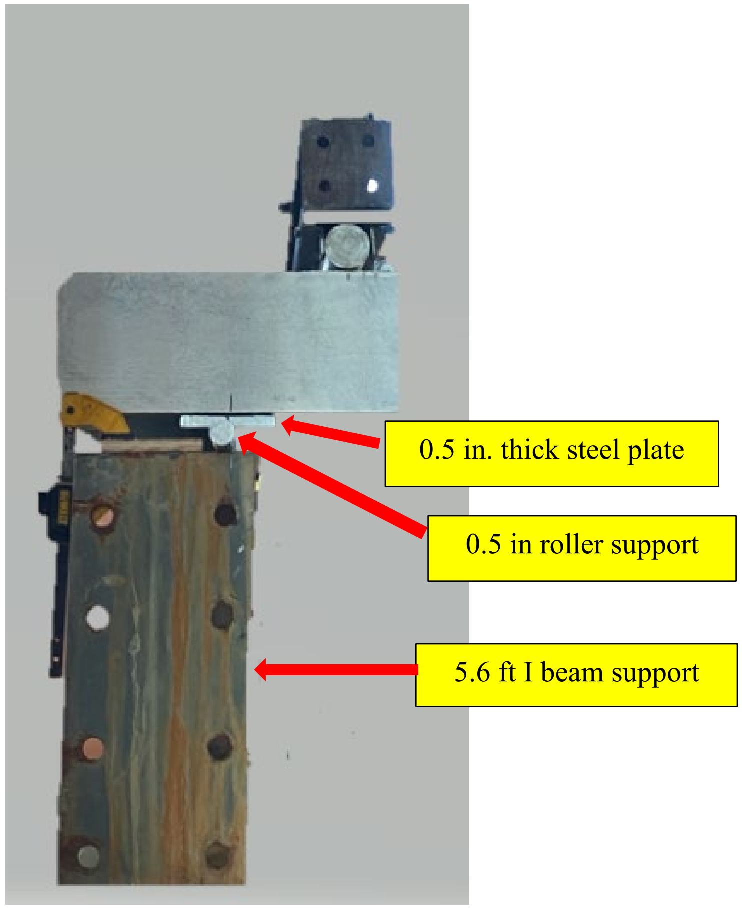


Figure 4-9: End Support Details for Shear Test

The loads were applied using the hydraulic ram that was pressing against the self-contained reaction frame. The 4-ft long rod that stretched across the width of the panel exerted a line of concentrated loads on the panel. The hydraulic ram and the reaction frame are shown in Figure 4-10.

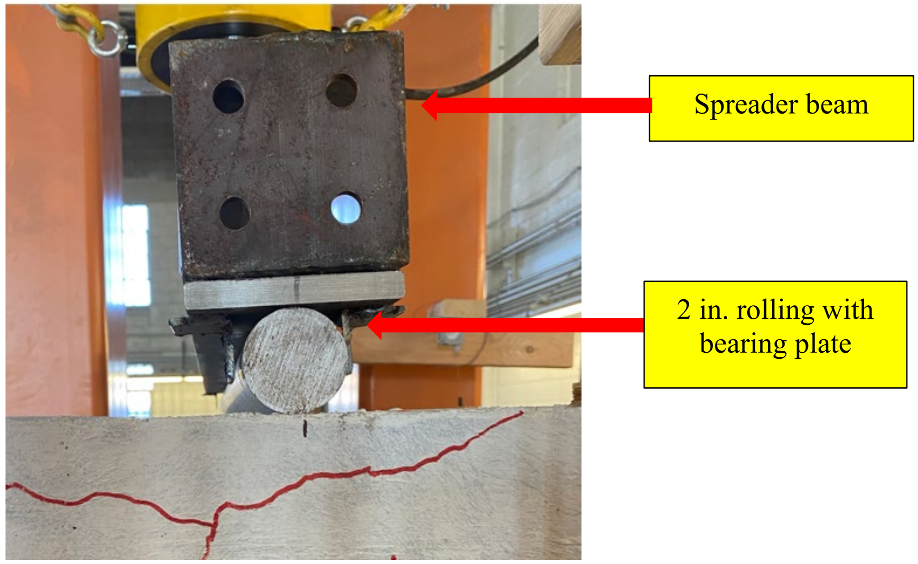


Figure 4-10: Point Loading Detail for Shear Test of Slab Specimen

4.4.1 Data Acquisition System

All the instruments such as load cell and potentiometers were connected to a Campbell Scientific data acquisition system. The loading rate was set to be the slowest possible (i.e. one millimeter per second) with an acquisition rate of at least 1Hz.

4.5 Experimental Results

Each specimen was subjected to a crack propagation study to visually examine the spread of damage and the overall performance of the specimen throughout testing. This was accomplished by keeping track of the location, type, and size of cracks during the test. The data collected for each test was supplemented by observations at different stages of testing.

To evaluate the ultimate load capacity, failure modes, and maximum deflection, each specimen was tested to failure. Observations from testing and experimental results for each specimen are discussed as follows:

4.5.1 Panel 1

The dimensions of Panel 1 were 60.5 in x 49 in. The load was applied at 13 in from the edge of panel. The two string pots were attached at 18.5 in from the edge of the panel and at the center of panel. The panel was made on 10/05/2020 and tested on 12/22/2020. The 28-day concrete strength of the specimen was 2800 psi. Figure 4-11 shows the load versus midpoint deflection plot for Panel 1. The ultimate load capacity of the panel was approximately 20000 lb. Between the load cycles of 8 kips and 9 kips, the first crack appeared during the testing. The panel was able to withstand a maximum load of 20.1 kips with an average mid-span deflection of 0.4 in.

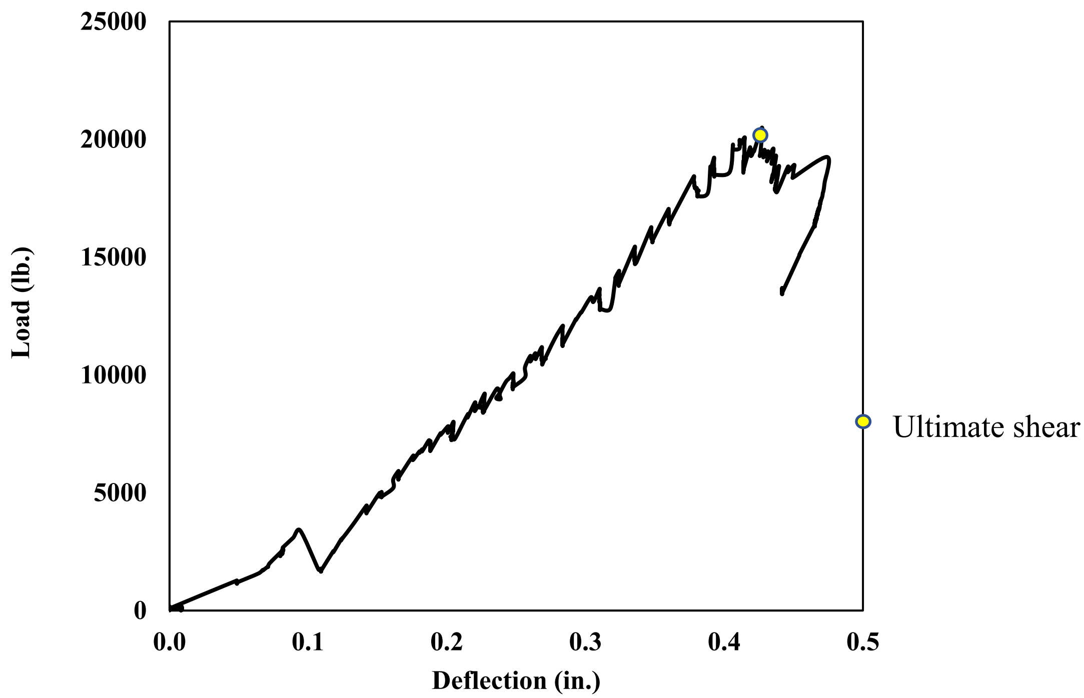


Figure 4-11: Load-Deflection Plot for Panel 1

For Panel 1, the dip in the deflections at 4000 psi is just a point at which observation was made and the data acquisition read a point, The reduction is likely associated with the data acquisition system. No fracture was observed at this point during testing.

The mode of failure for Panel 1 was observed to be shear-compression failure. The first crack (0.1 mm) was formed on the east side. After that, 0.2 mm width crack appeared on the south side. Then, the length of the crack increased, and another crack appeared at the end of the panel. There was a crack on the west side as well. Some cracks were formed on the bottom wythe of the specimen close to the mid-span. As the load increased, more hairline cracks were formed on the bottom side. The cracks went straight across the width of the slab on the bottom wythe and continued to the edge walls. Figure 4-12 shows some of the cracks observed during testing. Several new hairline cracks were formed before the specimen started to yield. In post yielding of the specimen, very few new cracks were formed. Instead, the existing cracks increased in width. The panel stopped picking up more force after about 20 kips. However, the panel did not fail completely. The snapping sound of the wires and truss chords could be heard towards the end of the experiment. Post failure analysis of the specimen showed noticeable buckling of the diagonal bars as shown in Figure 4-13.

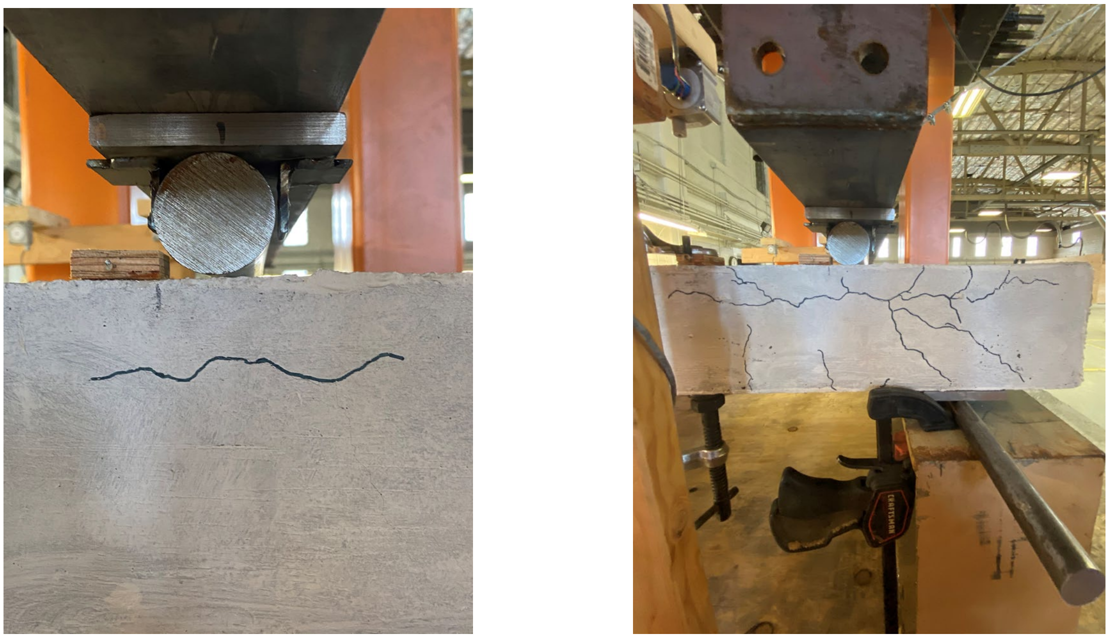


Figure 4-12: Progression of Shear Cracks in Panel 1 (East Side)

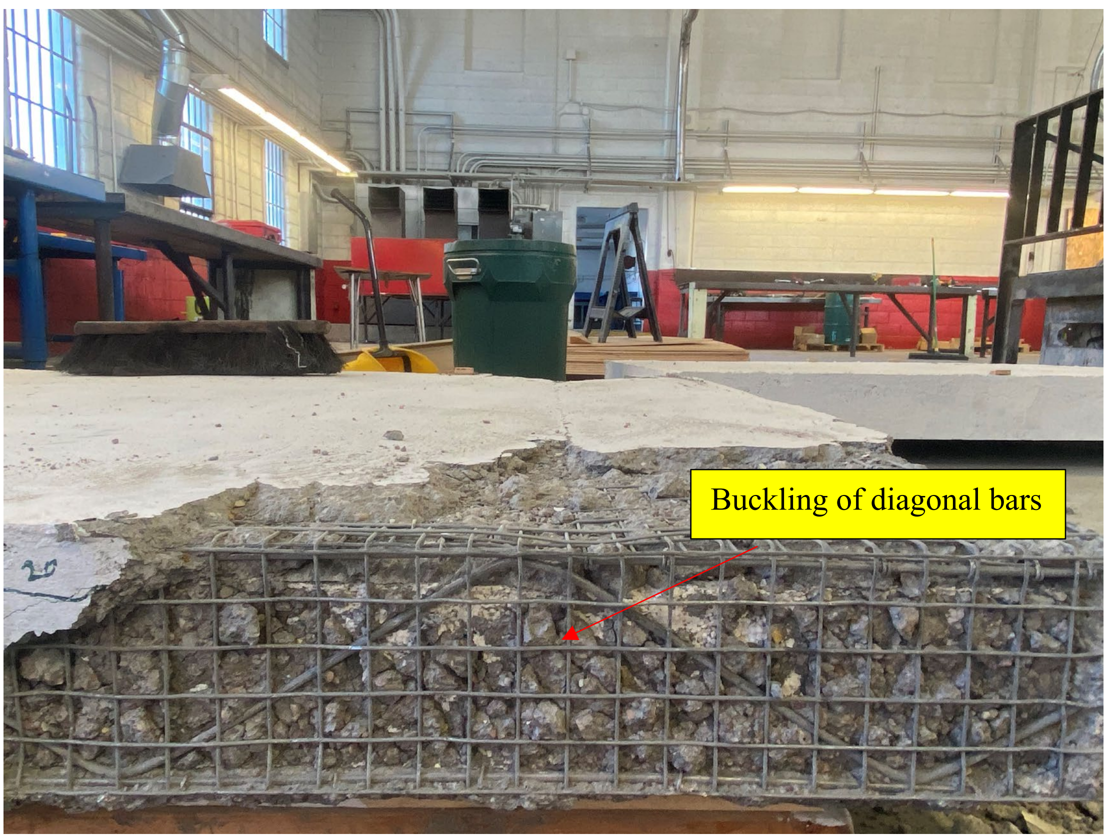


Figure 4-13: Buckling of diagonal bars in Panel 1

The maximum deflection of the panel was observed to be at the point where the load was applied. It was not possible to mount a potentiometer at this location during testing. To determine the deflection of the panel at this point, an approximate method was utilized. The deflected shape of the panel was assumed to be linear and data from the pots mounted in other locations of the panel used. Figure 4-14 and Figure 4-15 provide schematics to calculate the maximum deflection of the panel at the loading point.

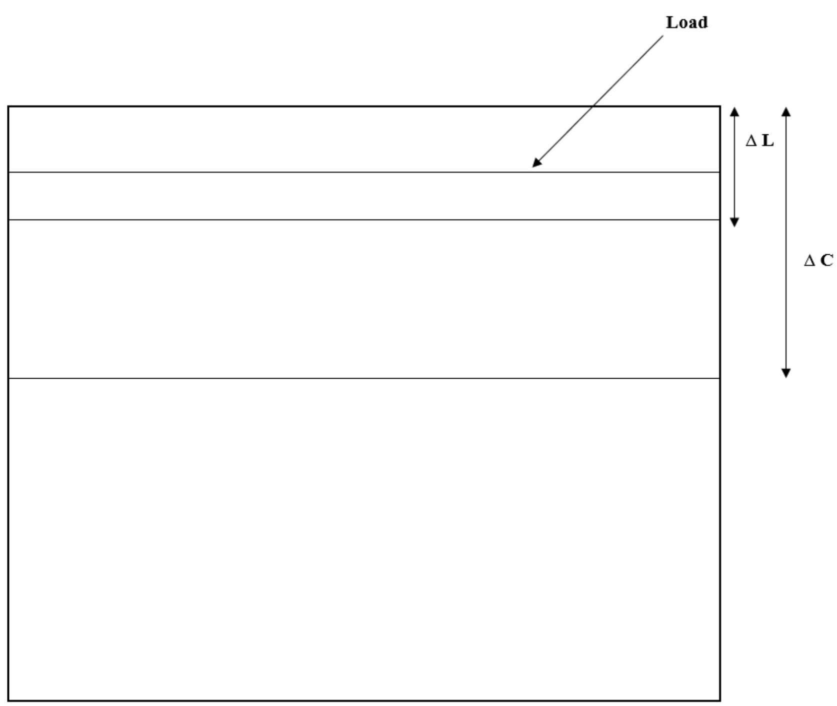


Figure 4-14: Representation of Top view of Shear Test of Panel 1

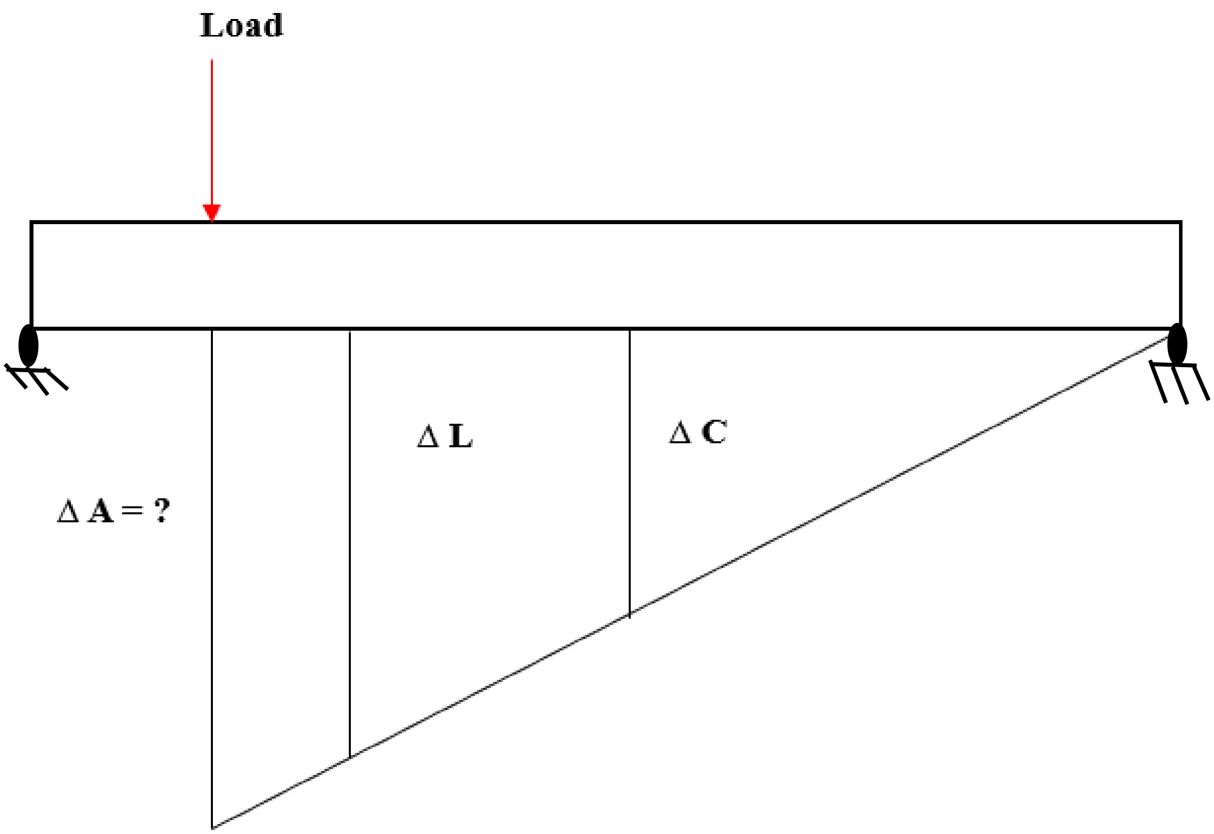


Figure 4-15: Deflection for Panel 1

The ΔL illustrated in Figure 4-15 the average deflection at point where one string pot is attached which is at 18.5 in from the edge of Panel 1. The ΔC is the average deflection at the point where the second-string pot is attached which is at 24.5 in from the edge of the panel. The ΔA is the unknown value of deflection at the point where the load is applied during testing. The load is applied at 13 in from the edge of Panel 1. All the known values of deflections are for the ultimate load of 20080 lb that the panel could withstand.

To find the value of ΔA from the test results:

Δ L = (Load 1+ Load 2)/2 = (0.34+0.23)/2 = 0.29 in

Δ C = (Center 1 + Center 2)/2 = (0.26 + 0.15)/2 = 0.205 in

$$\frac{\Delta \text{ L}}{18.5} = 0.016$$

$$\frac{\Delta \text{ C}}{24.5} = 0.0084$$

Distance between ΔL and ΔC = 6”

At distance of 6 in , the value of deflection decreases by almost half.

Therefore, ΔA = 0.29 x 2 = 0.58 in (since distance between ΔA and ΔL = 6”)

4.5.2 Panel 2

The dimensions of Panel 2 were 63 in x 49 in. The load was applied at 12.5 in from the edge of panel. The two string pots were attached at 18 in and at the center of panel, respectively. The panel was made on 10/05/2020 and tested on 12/22/2020. The 28-day concrete strength of the specimen was 2800 psi. Figure 4-16 shows the load versus midpoint deflection plot for Panel 2. The ultimate load capacity of the panel was 16800 lb. Between the load cycles of 5 kips and 6 kips, the first crack appeared during the testing. Panel 2 could withstand a maximum load of 16.8 kips, with an average mid-span deflection of 0.2 in.

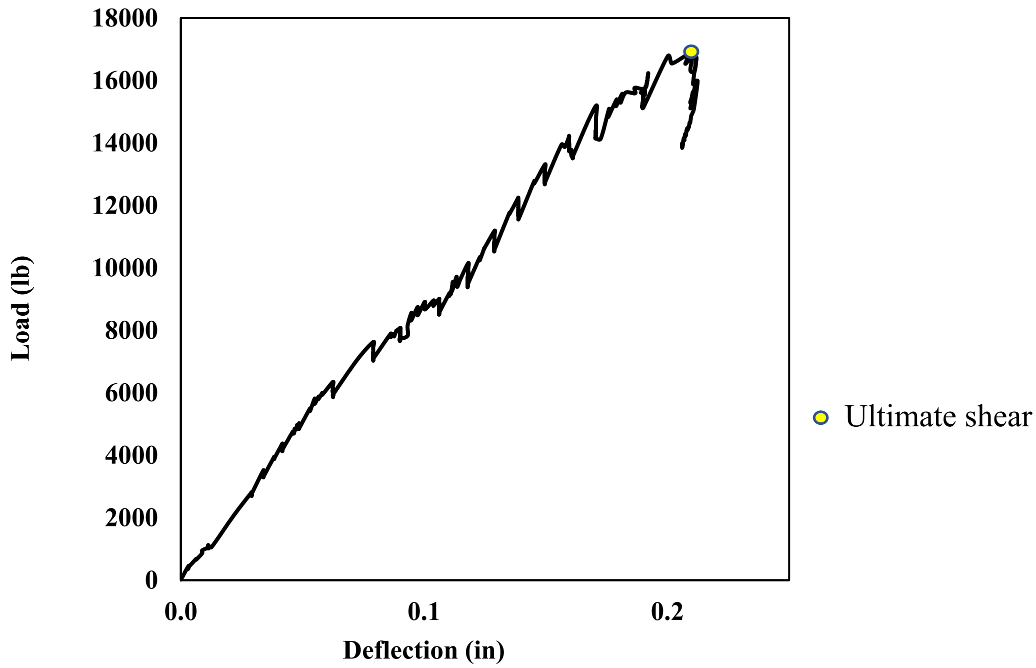


Figure 4-16: Load-Deflection Plot for Panel 2

The mode of failure for Panel 2 was similar to Panel 1 (e.g. shear-compression failure). The first crack (0.1 mm) was formed on the east side. After that, a 0.1 mm width crack appeared on the west side. Then, the number and length of cracks increased along both east and west sides of the panel. The width of cracks on the west side increased to 0.2 mm. Then, the cracks were formed on the bottom wythe of the specimen from the east side. As the load increased, more hairline cracks formed on the bottom side. The cracks went straight across the width of the slab on the bottom wythe and continued to the edge walls. Figure 4-17 shows some of the cracks observed during testing. Similar to Panel 1, not many new cracks were observed after the panel yielded. The panel stopped picking up more force after close to17 kips. However, the panel did not fail completely. Similar to Panel 1, the snapping sound of the wires and chords could be heard towards the end of the experiment. Post failure inspection of the specimen showed buckling of the diagonal bars as can be observed in Figure 4-18: Progression of Shear Cracks in Panel 2 (East Side).

Figure 4-17: Progression of Shear Cracks in Panel 2 (East Side)

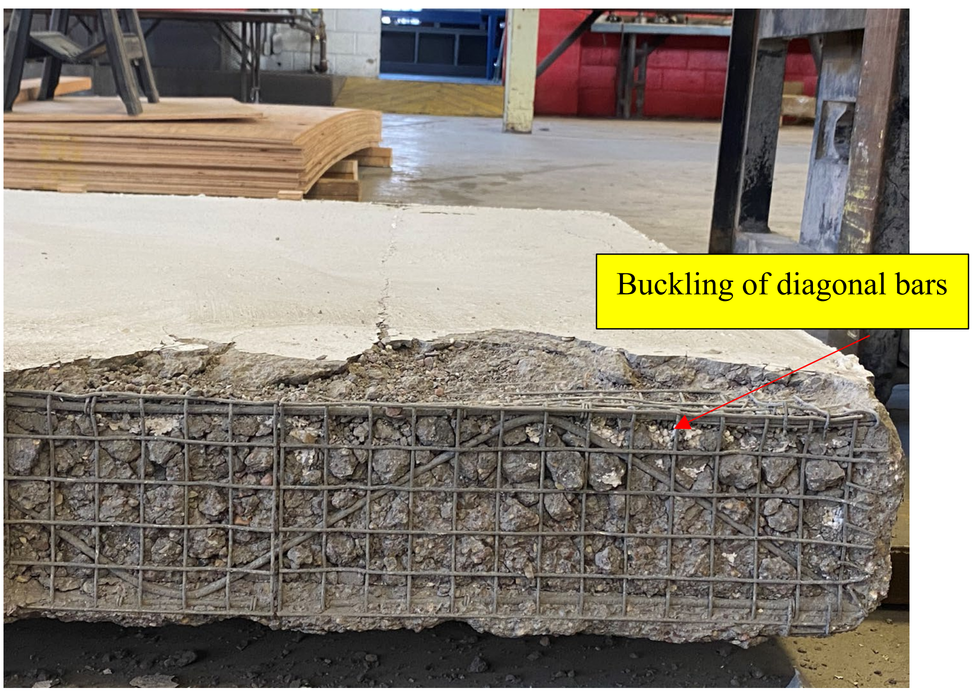


Figure 4-18: Progression of Shear Cracks in Panel 2 (East Side)

To determine the maximum deflection of Panel 2 where the load is applied, a similar procedure as outlined for Panel 1 previously, can be utilized. For Panel 2, ΔL is at 18 in and ΔC is at 24.5 in from the edge of panel. The ΔA is the unknown value of deflection at the point where the load is applied. The load is applied at 12.5 in from the edge of Panel 2. All the known values of deflections are for the ultimate load of 16809 lb for Panel 2. To find the value ΔA from the test:

Δ L = (Load 1+ Load 2)/2 = (0.19+0.29)/2 = 0.24 in

Δ C = (Center 1 + Center 2)/2 = (0.22 + 0.20)/2 = 0.21 in

$$\frac{\Delta\ \mathrm{L}}{18} = 0.013$$

$$\frac{\Delta\ \mathrm{C}}{24.5} = 0.0086$$

Distance between ΔL and ΔC = 6 in

At distance of 6 in, the value of deflection decreases by almost half.

Therefore, ΔA = 0.24 x 2 = 0.48 in (since distance between ΔA and ΔL = 6 in)

4.6 Summary

1. Both slab specimens exhibited high capacity and noticeable deformation when subjected to shear loading. The failure mode was a shear-compression failure with the formation of many hairline cracks before failing. These cracks indicated that there was some distribution of forces in the MR panel until the failure point.
2. The ultimate load capacity for Panel 1 was 20000 lb. Between the load cycles of 8 kips and 9 kips, the first crack appeared. Panel 1 had an average mid-span deflection of 0.4 inch at the ultimate load.
3. The ultimate load capacity for Panel 2 was 16800 lb. Panel 2 had an average mid-span deflection of 0.2 inch at the ultimate load.
4. The maximum deflection for Panel 1 at the loading point was calculated to be 0.58 in. This was calculated to be 0.48 in. for Panel 2
5. In post-testing inspection, buckling of diagonal bars was observed in both panels.

Chapter 5. Simplified Analytical Modeling

In this chapter, experimental values from the testing done previously at Idaho State University (Gurung, 2019) and the shear testing discussed in the previous chapter are used to validate analytical prediction (values) using simplified building code procedures and guidelines. This is discussed for flexural and shear design of MR panels in the subsequent sections. Some guidelines for axial design of the panels are also included.

5.1 Flexural Design

The nominal moment capacity (ϕMn) is calculated using the effective moment equation stated in the section 10.2 of ACI 318 (ACI, 2019). The equation for the nominal moment capacity for a fully composite section is provided in Eqn. (5-1).

$$\phi Mn = 0.85f'c \text{ x } a \text{ x } b(d - a/2) + A's \text{ x } f's(d - d') \qquad (5\text{-}1)$$

Where, A_s= Area of tensile steel (in^2)

A'_s =Area of compression steel (in^2)

d = distance to tension steel (in)

d' = distance to compression steel (in)

f'_s = stress in compression bars (psi)

f'_c =compressive strength of concrete (psi)

a = distance to the neutral axis (in)

L = Span of the panel (ft)

Using Eqn 1, Table 4-2presents moment capacities for MR slab panels of width 4 ft and varying thicknesses and spans. The detailed calculations are given in Appendix A. [Moment Capacity Calculation]

Table 5-1: Moment Capacity of Slab Panels

Compressive Strength f'_c = 3000 psi Stress in compressive bars f'_s = 21750 psi						
Thickness t = 1 in.						
Span (ft.)	**Moment Capacity (lb-in)**	**Average unit weight per ft**	**Moment caused by self-weight (lb-in)**	**Effective Moment capacity (lb-in)**	**Average moment capacity**	
L	M	(lb/ft)	Msw	Mn	(lb-in)	(kip-ft)
8	18942.22	50	4800	14142.22	9333.86	0.78
10	18942.22	50	7500	11442.22	7551.86	0.63
12	18942.22	50	10800	8142.22	5373.86	0.45
14	18942.22	50	14700	4242.22	2799.86	0.23
16	18942.22	50	19200	-	-	-
18	18942.22	50	24300	-	-	-
20	18942.22	50	30000	-	-	-
Thickness t =1.5 in.						
8	34533.37	75	7200	27333.37	18040.02	1.5
10	34533.37	75	11250	23283.37	15367.02	1.28
12	34533.37	75	16200	18333.37	12100.02	1.01
14	34533.37	75	22050	12483.37	8239.02	0.69
16	34533.37	75	28800	5733.37	3784.02	0.32
18	34533.37	75	36450	-	-	-
20	34533.37	75	45000	-	-	-
Thickness t = 2 in.						
8	50124.52	100	9600	40524.52	26746.18	2.23
10	50124.52	100	15000	35124.52	23182.18	1.93
12	50124.52	100	21600	28524.52	18826.18	1.57
14	50124.52	100	29400	20724.52	13678.18	1.14
16	50124.52	100	38400	11724.52	7738.18	0.64
18	50124.52	100	48600	1524.52	1006.18	0.08
20	50124.52	100	60000	-	-	-

The difference between nominal moment capacity and the moment generated by self-weight is the effective moment capacity estimated in the table. The effective moment capacity is multiplied by 0.66 to get the average moment capacity. The value of 0.66 was obtained from full-scale testing of MR panels by Gurung (2019). The blank numbers in the table represent a negative average moment capacity, indicating that the panel's corresponding span is not appropriate for the thickness and width specified, since the moment generated by self-weight exceeds the panel's moment capacity, the values are negative.

The experimental data and analytical values can be compared. Table 5-2 provides average experimental results from testing of 10 ft and 18 ft MR panels.

Table 5-2: Average Moment Capacity (Gurung, 2019)

Span (ft)	**Average moment capacity (kip-ft)**
10	13.59
18	9.90

From analytical approach,

Average moment capacity = 5.15 -kip ft (10 ft)

Average moment capacity = 0.21-kip ft (18 ft)

Analytical estimate for the average moment capacity is 38% and 2% of experimental result. This is thought to be appropriate and conservative since the average moment capacity of panels was assumed to be partially composite for the simplified analytical calculation using effective moment capacity for the MR panels as 66% of the effective capacity of a fully composite section.

The following Figure 5-1 shows the relation between span of concrete slab with varying thickness on the values of nominal moment capacity.

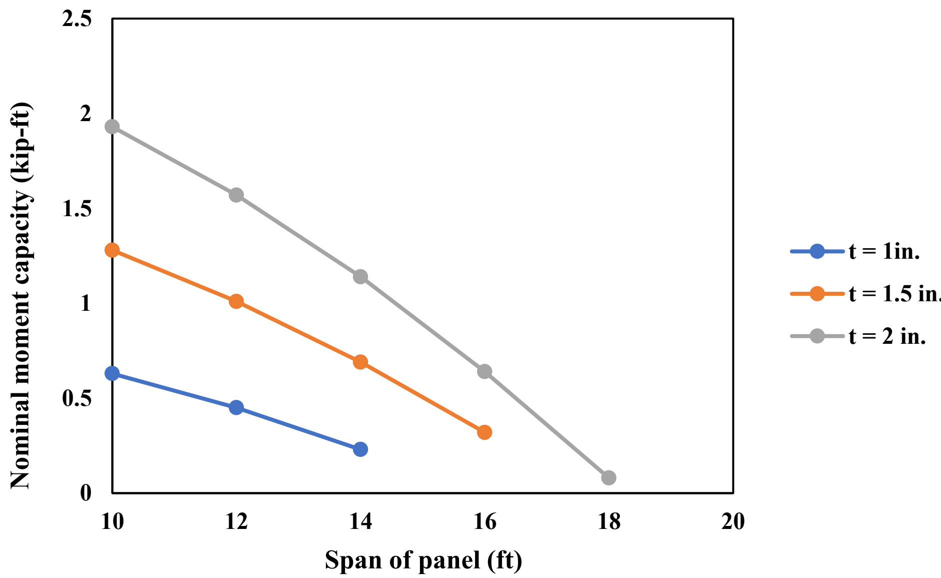


Figure 5-1: Relation between Panel Span and Thickness on its Nominal Moment

The value of nominal moment capacity decreases with increase in the span of panel and increases with increase with thickness of panel.

5.2 Shear Design

The shear capacities are calculated according to ACI 318 (ACI, 2019). As mentioned previously, the shear strength at a section of a member (V_i) is the sum of concrete strength "V_c" and reinforcement strength "V_s".

$$Vi = Vc + Vs \tag{5-2}$$

Results from shear testing of two MR panels are presented in Table 5-3:

Table 5-3: Experimental Shear Strength of MR Panels

Panel	Length (in.)	Width (in.)	Concrete layer Thickness (in.)	Shear strength (lb.)	Compressive strength (psi)
I	60.5	49	2	20000	2800
II	63	49	2	17000	2800

To estimate the shear capacity analytically, the following procedure is proposed.

Shear contribution of the concrete layers as per Figure 5-2:

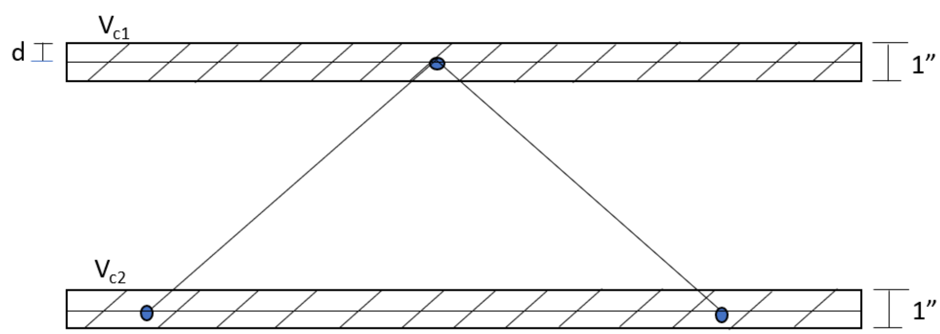


Figure 5-2: Section View of MR Panel

$\varphi V_c = \varphi\, 2\lambda\sqrt{f'c}\, bw\, d$

Where,

for normal weight concrete:

$\lambda = 1$

From the ACI 318 (2019):

$\varphi = 0.75$

From the geometry of the panel:

$d = 0.5"$

As the panel has two concrete layers as shown in Figure 5-2:

$V_c = V_{c1} + V_{c2} = 2V_{c1}$

$\varphi\ V_c = (\varphi\ 2\lambda\sqrt{f'c}\ bw\ d) \times 2$

$\varphi\ V_c = [(0.75)(1)(2)\ (\sqrt{2800})(49)(0.5)] \times 2$

$\varphi\ V_c = 5185$ lb

$\varphi\ V_c = 5185/4 = 1296$ lb/ft

The contribution from EPS core is ignored. The force P as shown in Figure 5-3 is the force equal to the sum of two forces resisted by the truss members. The truss member is represented as the sum of two diagonal Euler buckling loads with fix-fix ends (K=0.5) shown in Figure 5-4.

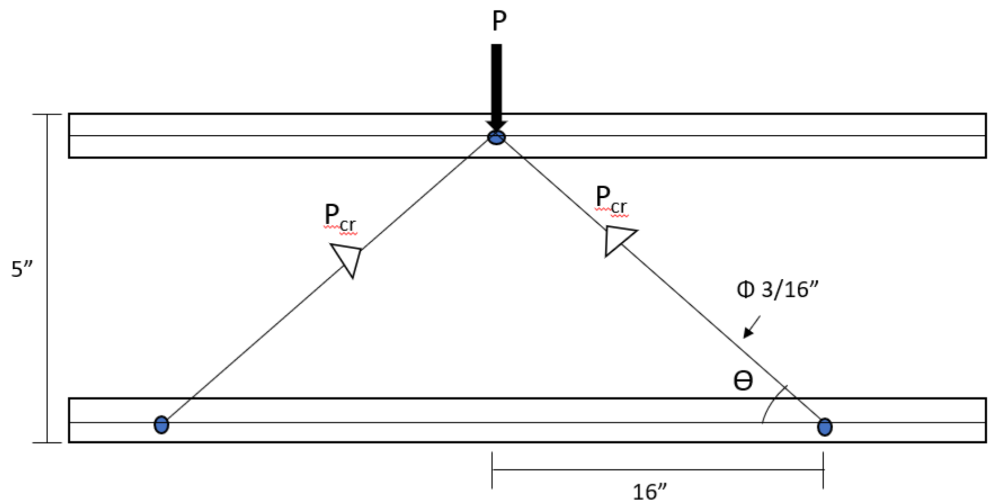


Figure 5-3: Free Body Diagram of a Typical Truss in MR Panels

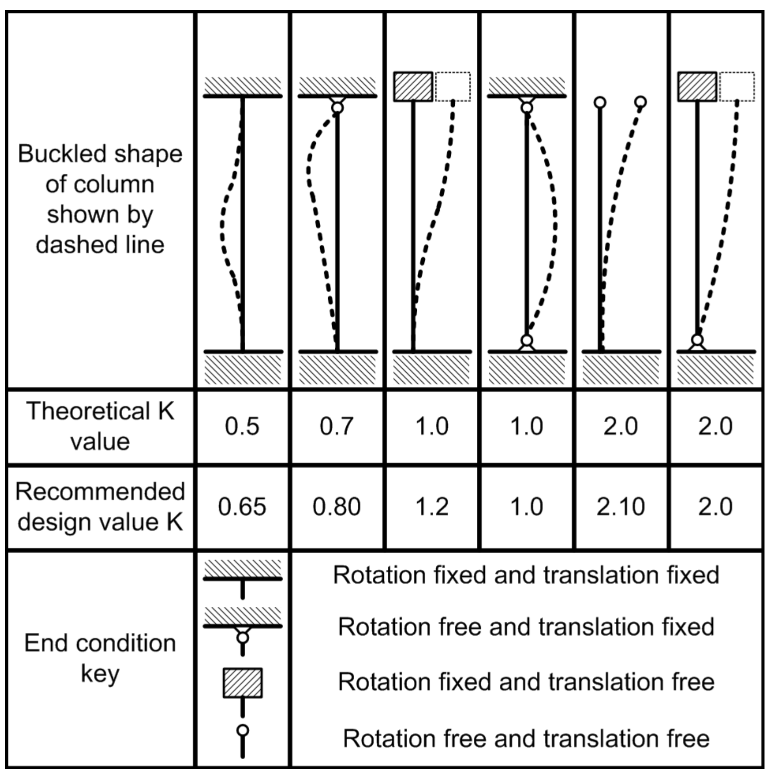


Figure 5-4: Effective length for Euler buckling load

Buckling load of the diagonal is taken as the shear strength of panel. Although it would be conservative, it is a good and safe practice.

For Panel I:

$P_{cr} = \frac{\pi^2 EI}{(KL)^2}$

Where,

P_{cr} = Euler buckling load

Modulus of elasticity of steel (E) = 29000 ksi

Effective length factor (K) = 0.5 (fix-fix end)

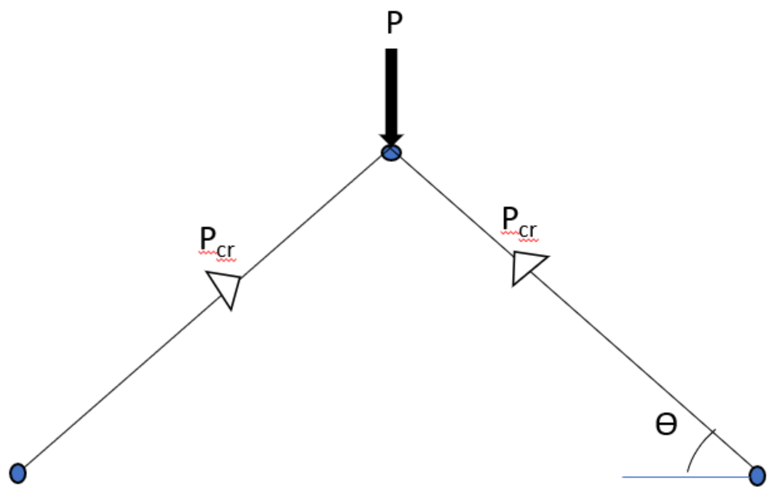
Length of a truss member (L) = $\sqrt{16^2 + 5^2}$ = 16.76 in

For the value of moment of intertia for the panel, the inertia assumed is polar moment of inertia of the circle.

Diameter of member (D) = 3/16"

$I = \frac{\pi D^4}{32} = 1.21 \text{ X } 10^{-4} \text{ in}^4$

$P_{cr} = \frac{\pi^2 (29000)(1000)(1.21 \text{ X } 10^{-4})}{(0.25)(16.76)^2} = 493.2 \text{ lb}$



$\Theta = \tan^{-1}(5/16) = 17.35^{o}$

$+\uparrow\ \Sigma F_y = 0$

$2(P_{cr})\ (\sin 17.35) = P$

$P = 294.2$ lb

For 16” strip @ 6” distance along 49-inch width, there are 9 nodes for truss resistance P.

So, truss resistance = 9 x 294.2 = 2647.8 lb/ft

The truss resistance value is assumed to be per foot of the panel for a conservative estimate.

$\phi V_n = \phi V_c + \phi V_s$

$= 1296$ lb/ft + 0.75 (2647.8)

$= 3281.85$ lb/ft

From experimental testing,

$\phi V_n = 20000/(49/12) = 4897.96$ lb/ft

Therefore, the analytical estimate for the ultimate shear capacity is 67% of the experimental result. This is thought to be appropriate and conservative since the simplified analytical calculation does not consider nonlinear deformation, strain hardening, or redistribution of forces.

For Panel II:

The value of ϕV_c is same for both panels as they both have same compressive strength of 2800 psi.

The value of P is also same as Panel 1 of 2647.8 lb/ft as they both have the same width of 49 in.

$\phi V_n = \phi V_c + \phi V_s$

$= 3281.85$ lb/ft

From experimental testing,

$\phi V_n = 17000/(49/12) = 4163.27$ lb/ft

The analytical shear strength for Panel 2 is 79% of the experimental result which is conservative and appropriate as explained previously for Panel 1. If the average of the experimental results for Panel 1 and Panel 2 are taken (e.g. 4531 lb/ft), the analytical prediction is close to 72% of the average experimental strength.

5.3 Axial design

The Design Strength requirements of ACI 318-19 §11.5 must be met for the axial ($\phi P_n \geq P_u$) design of wall panels.

Where,

P_u = load from structural analysis

ϕ = 0.65 as per ACI 318 (ACI, 2019) for compression design

$$\phi Pn = (0.65 f'c) Ag \tag{5-3}$$

The value of nominal axial capacity changes with change in compressive strength of concrete, width, and thickness of the panel. For panels that are more than 10 ft in height and are resisting considerable axial loads (e.g. more than 3% of $f'_c A_g$), it is recommended that the out-of-plane buckling of the panel to be considered in structural design. An MR wall panel should not be axially loaded more than 5% of $f'_c A_g$ unless further experimental data becomes available. This is done to prevent axial crushing of the panel during lateral loading, such as earthquakes, if the wall panel is designed as a shear wall.

Chapter 6. Construction Concepts, Sample Design Examples and Recommendations

This chapter presents some concepts and sample calculations for residential buildings made of MR Panels.

6.1 One-Story Residential Building

Figure 6-1 illustrates the plan view for a one-story residential building made of SCIPs. The analyzed system that combines the advantages of tilt up and pre-engineered steel is the SCIP system. SCIPs have composite parts, combining weld and wire trusses, insulation and mesh which are then finished with concrete. Since SCIPs combine both structural performance and thermal insulation in a single package, it could be one of the most attractive methods to build a residential structure.

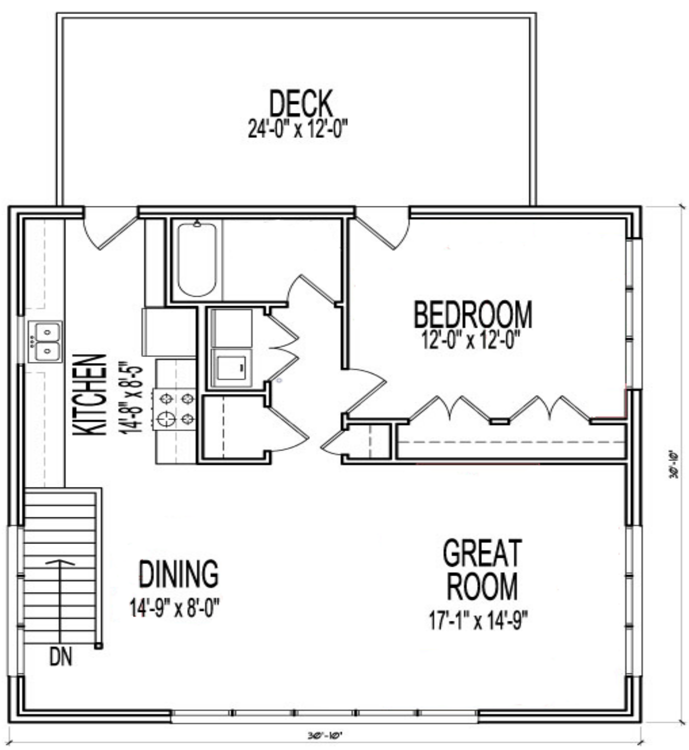


Figure 6-1: Sample One-Story Residential Building

For construction of the house using on-site Shotcreting, first a common metal stud track anchorage with vertical wire tie connection is employed. In this application, SCIPs are used as infill panels in tilt up style structure. As the panels are lightweight and flexible, they can be moved in place with minimal labor and time. SCIPs are initially fixed together using conventional wire tires, then further in the process they are secured permanently just prior to applying concrete. In this case, SCIPs with pre-cut open channels can be used, as this facilitates the ease of placement with any needed electrical or plumbing conduits prior to concrete application. After initial erection, SCIPs are securely fastened together by the process of a highly efficient C clip gun. For instance, panels used in the design can be 4 ft by 15 ft by 6 in. thick. The panel-to-panel joints are spliced with flat mesh to ensure the reinforcement matrix is uninterrupted to avoid cracking. Butt joints are covered with a flat screen mesh and, corners are covered with L-mesh to fit either inside or outside corner dimensions. The openings are covered with cap mesh bent into a U-shape. Expansion joints may be employed for thermal expansion and other reasons (e.g. seismic joints).

On-site adjustments are quick and easy as well. For instance, if the header piece of a doorway needs to be narrowed, the panel is field cut and stripped in exact dimensions and tied in place for a proper fit. Plumbing and other internal fixtures can be placed on-site inside the panels by cutting grooves in EPS cores in those locations. After the panels are erected, it is necessary that prior to the application of concrete skins, masking of the surrounding area is done. Wet mix shotcrete can be produced on-site by a mixer mounted on a loader for small projects, such as a residential house. The standard 1 in. thickness or more can be applied in a single pass using either the dry mix or wet mix process. For larger projects, processes such as getting concrete from batch plants can be employed. Concrete can also be applied by hand for smaller projects. This can be varied, however, depending on the job requirements.

MR panels can be customized by varying the strength of the mix, spacing of trusses and the weight of the mesh. By design, the electrical lines can either be surface mounted or installed inside the panel wall. A Manifold Pex water system is recommended for the MR panel running inside wall or placed in conduit in a modular application. This Manifold PEX system allows for any water connections to be limited to only outside concrete skins.

6.2 Example I: Checking Shear Capacity of a MR Slab Panel

A precast MR slab panel is subjected to an ultimate shear demand of 1500 lb/ft. $f'_c = 5000$ psi, the panel has a width of 4 ft, and concrete layer thickness of 1 in.

$\varphi V_c = \varphi\ 2\lambda\sqrt{f'c}\ bw\ d$

Where,

for normal weight concrete:

$\varphi = 0.75$

$\lambda = 1$

$d = 0.5''$

As the panel has two concrete layers as shown in Figure 6-2

$V_c = V_{c1} + V_{c2} = 2\ V_{c1}$

$\varphi V_c = (\varphi\ 2\lambda\sqrt{f'c}\ bw\ d)\ X\ 2$

$\varphi\ V_c = [(0.75)(1)(2)\ (\sqrt{5000})(4)(12)(0.5)]\ X\ 2$

$\varphi\ V_c = 5091.2$ lb

$\varphi\ V_c = 5091.2/14 = 1272.8$ lb/ft

No contribution from EPS core is assumed for simpler calculation. The force P as shown in figure is force is equal to the sum of two forces resisted by the truss members. The truss member is represented as the sum of two diagonal Euler buckling loads with fix-fix ends. Figure 6-2 shows the clear representation for the calculation for truss resistance.

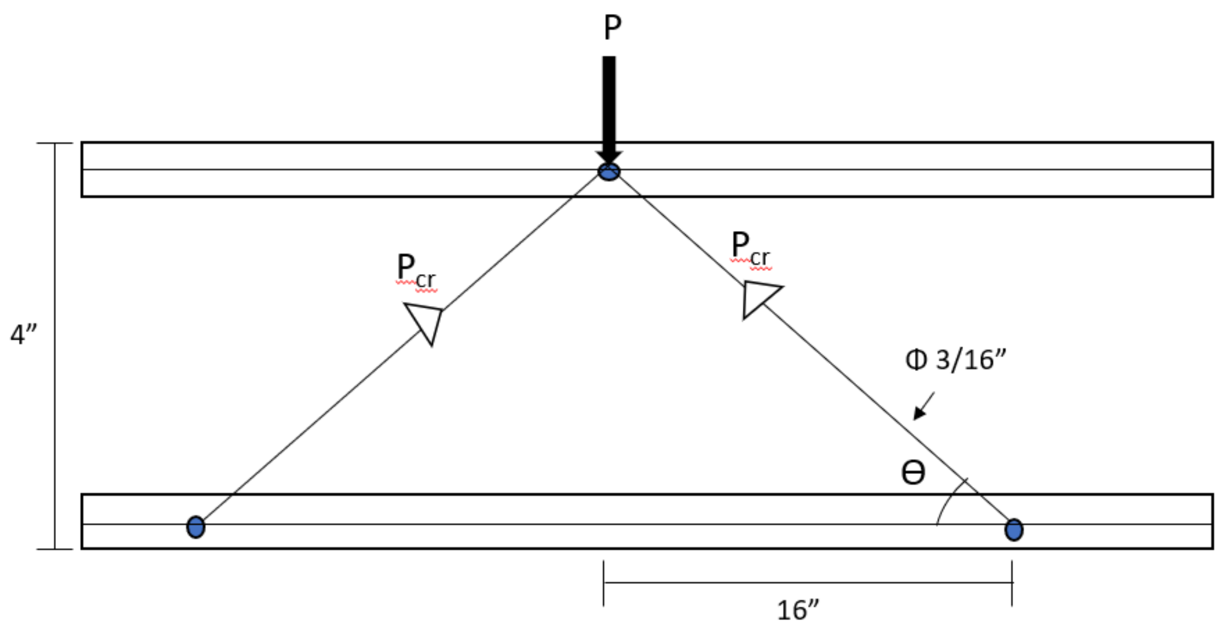


Figure 6-2: Free Body Diagram of the Steel Truss in MR Panel

$P_{cr} = \frac{\pi^2 EI}{(KL)^2}$

Where,

P_{cr} = Euler buckling load

Modulus of elasticity of steel (E) = 29000 ksi

Effective length factor (K) = 0.5 (assuming fix-fix end)

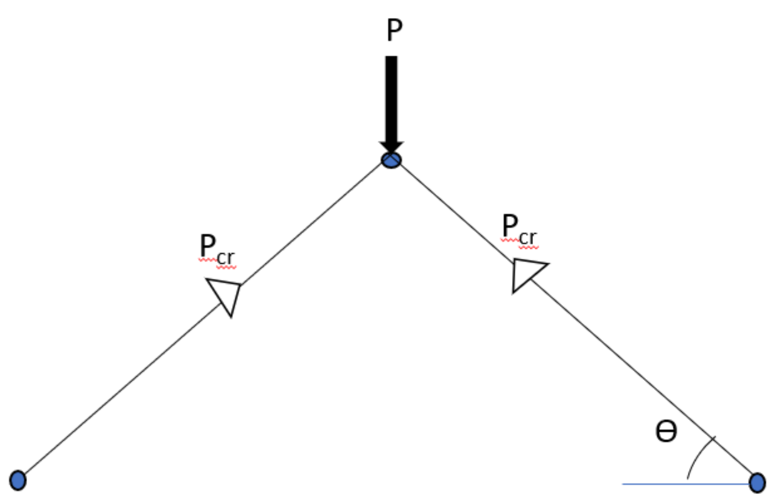
Length of a truss member (L) = $\sqrt{16^2 + 4^2}$ = 16.49 in

For the value of moment of intertia for the panel, the inertia assumed is polar moment of inertia of the circle.

Diameter of member (D) = 3/16"

$I = \frac{\pi D^4}{32} = 1.21 \text{ X } 10^{-4} \text{ in}^4$

$P_{cr} = \frac{\pi^2 (29000)(1000)(1.21 \text{ X } 10^{-4})}{(0.25)(16.49)^2} = 509.5 \text{ lb}$



$\Theta = \tan^{-1}(4/16) = 14.04^{o}$

$+\uparrow \ \Sigma F_y = 0$

$2(P_{cr})\ (\sin 17.35) = P$

$P = 247.2$ lb

For 16’ strip @ 6” distance along 4 ft, there are 9 nodes for truss resistance P.

So, truss resistance = 9 x 247.2 = 2224.8 lb/ft

This value is assumed to be per feet.

$\phi V_n = \phi V_c + \phi V_s$

$= 1272.8$ lb/ft $+ 0.75\ (2224.8)$ psi

6.3 Example II: Calculating Flexural Capacity of a MR Slab Panel

The calculation below provides a step-by-step procedure on how to calculate the flexural capacity of a panel with certain material properties and geometry.

$\phi M_n = 0.85\ f'_c * a * b\ (d - a/2) + A'_s * f'_s (d\text{-}d')$

Where, A_s= Area of tensile steel (in^2)

A'_s =Area of compression steel (in^2)

d = distance to tension steel (in)

d’ = distance to compression steel (in)

f_y = yield strength for steel (psi)

f'_s = stress in compression bars (psi)

f'_c =compressive strength of concrete (psi)

a = distance to the neutral axis (in)

L = Span of panel (ft)

Self-weight	130	lb/ft
Depth (h)	6	in
f'_c	5000	Psi
f_y	60000	Psi
d (distance to tension steel)	5.5	In
d'(distance to compression steel)	0.5	In
b(width)	4	Ft
As	0.5	in^2
A's	0.5	in^2
A	β1.c	
f's	E_s. ((cd’) *0.003/c))	
Es	29000000	Psi
Height(span)	28.5	Ft
C	0.18	In
ϕMn = 0.85 f 'c$*a*b\ (d-a/2)$ + $A's*f's$(d-d’)		lb-in

β_1=	0.85			
$A_s.f_y = 0.85.f'_c.b.\beta_1.c + (A'_s.E_s.((c-d')*0.003/c)))$				
From this equation:				
c =	0.34	In		
a =β_1.c	0.29	In		

Thickness = 6 in		
d = 5.75in d' = 0.25 in		
Span = 28.5 ft		
f's = 43500.00 psi		
ϕMn =	323843.58	lb-in
Average unit weight of panel per linear ft =	300.00	lb/ft
Moment caused by self-weight =	365512.5	lb-in
Effective moment capacity =	-41668.92	lb-in
Average effective moment capacity =	27501.49	lb-in
Average effective moment capacity =	2.29	kip-ft

6.4 Example III: Calculating Axial Capacity of a MR Wall Panel

A wall panel is subjected to an ultimate axial load of 30 kip. The building is in a seismic region. If $f'_c = 5000$ psi, concrete thickness = 3 in on each side, panel height = 10 ft, and the panel is 4 ft wide, calculate the design axial capacity of the panel and indicate if it will be able to resist the axial loads.

$P_f = f'_c \times A_g$

For $f'_c = 5000$ psi, $A_g = 4 \times 6 \times 12 = 288$ in^2

$P_f = 1440000$ lb

$\phi\ P_n = 0.65 \times 1440000 = 936000$ lb

$P_u = 45.1$ kip

The Design Strength requirements of ACI 318-19 §11.5 is that:

$\phi\ P_n \geq P_u$ or 45.1 kip > 30 kip (OK)

Check axial load for % of $f'_c\ A_g$ since the building is in a seismic region:

30,000 / (5000 x 288) x 100 = 2.08% < 3% (OK)

Check the height of the panel for out-of-plane buckling:

Height = 10 ft (OK)

6.5 Recommendation

6.5.1 LRFD design philosophy

Load and Resistance Factor Design (LRFD) is a reliability-based design philosophy, which explicitly takes into account the uncertainties that occur in the determination of loads and strengths. LRFD design should be used for the design of MetRock panels. Safety in any engineering design is assumed when the demands placed on components and materials are less than what is supplied, so that the following basic equation is satisfied:

Demand < Supply

Another way of stating this same principle with respect to structural engineering is that the effect of the loads must be less than the resistance of the materials, so that the following requirement is met:

Load < Resistance

When a particular loading or a combination of loadings reach the component or material resistance, safety margins approach zero and the potential for failure exists. The goal of the basic design equation is to limit the potential for failure to the lowest probability practical for a given situation (Bridges and structures, 2021).

A graphical representation of the LRFD philosophy is presented in Figure 6-3. The overlap of the two bell curves in Figure 6-3 represents the region for which the limit state has been exceeded. The reliability index (β) quantifies the structural reliability or in other words, the risk that a design component has insufficient resistance and that a specific limit state will be reached. A graphical representation of β is shown in Figure 6-4.

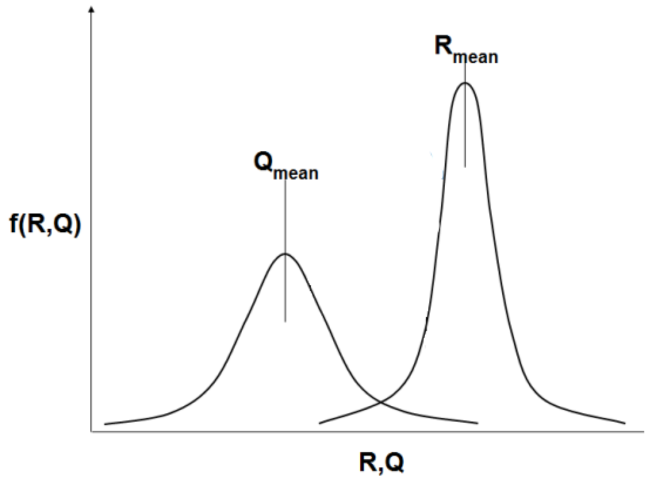


Figure 6-3: Bell Curves Illustrating Distribution of Load (Q) and Resistance (R)

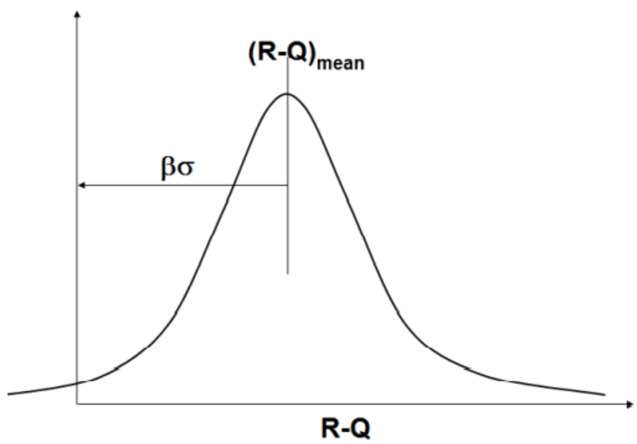


Figure 6-4: Graphical Definition of Reliability Index, β

Reliability index (β) = $\frac{\ln(R-Q)}{\sqrt{(Vr^2+Vq^2)}}$

Where,

$Vr = \sigma_r/R$

$Vq = \sigma_q/Q$

where σ denotes the standard deviation (φ)

6.5.2 Proposed resistance factors

A lesser resistance factor (φ) is applied to loads and materials whose behavior is less well-known and cannot be as accurately predicted. In this manner, greater knowledge of some resistances and loadings can be accounted for, allowing more efficient design of panels while still applying appropriate levels of safety to those resistances and loads which are more ambiguous. The nominal design strength is obtained by multiplying the strength by resistance factor (φ) to the design strength. The calculations in the thesis are on the capacity side; therefore, more uncertainty can be taken into account in terms of using more conservative resistance factors (e.g., instead of 0.9 for flexure, we can propose 0.8). Similarly, for shear a more conservative reduction factor can be used such as 0.65 instead of 0.75. For axial, reduction factor of 0.55 can be used instead of 0.65. As research is conducted and the knowledge base increases, load and resistance factors can be altered to account for the greater certainty, or in some cases, greater uncertainty of loads or resistances.

6.5.3 Uncertainty quantification

During any experimental or analytical study, there is always the possibility of error. Two types of errors are described below.

6.5.3.1 Systemic error

System error is caused by a problem with the measurement equipment. This includes instruments with hard-to-read scales, improperly calibrated instruments, and instruments that are not being used correctly. Systemic errors typically result in measurements that are precise but not accurate, the equipment may provide the same value every time a reading is taken, but the value is not accurate. The possible sources of systemic errors in the shear testing of MetRock panels in this research are discussed as follows.

6.5.3.1.1 Measurement of dimensions of panels

All the measuring instruments have a certain degree of potential error or uncertainty associated with them. The measuring tape used to measure the length, width and depth of panels also had some errors. So, the actual values of length, width and depth of panels may be more or less than the measured values. The panels were measured more than once in order to increase accuracy.

6.5.3.1.2 Concrete compressive strength

Frictional forces between the ends of a compression specimen and the loading plates greatly affect the observed compressive strength and failure of the specimen. Variability in water-to-cement ratio in concrete batches can also affect the compressive strength. To reduce error in concrete compressive strength, more samples were taken during construction of MetRock panels.

6.5.3.1.3 Potentiometer sensitivity

The calculated deflection for Panel 1 and Panel 2 are 0.58 in and 0.48 in, respectively. But the potentiometer used in the study had a sensitivity of +/- 1%. So, the deflections may be slightly more or less 1%. Appendix B provides the certification sheet and other details for the type of potentiometer used in the research.

6.5.3.1.4 Load cell sensitivity

The uncertainty of load cell was about +/-5% of its 225,000 lbf capacity (Appendix C). The maximum load in shear testing of Panel 1 and Panel 2 were 20,000 lb and 17,000 lb, respectively. The average maximum shear strength for the two panels was 18,000 lb. Since the load cell had a sensitivity of +/- 5%, the actual maximum shear strength for the panels tested in this research was either 18,900 lb or 17,100 lb.

In structural engineering testing, it is common to adopt a more conservative approach when quantifying the strength of a structural element using large-scale testing. This approach is specifically relevant for shear behavior of concrete elements. A shear failure is a brittle failure that can trigger partial of full collapse instantaneously. Building codes in the United States and other parts of the world treat shear failure as one of the collapse modes that is likely to result in loss of life. It is recommended that the shear resistance of Panel 2, should be taken as the maximum shear strength for the panels until further research becomes available. If a lower bound value is used for the load cell sensitivity, the actual maximum shear strength for Panel 2 would be 16,150 lb which is the recommended value in this research.

6.5.3.2 Random error

Random error is caused due to unpredictable and uncontrollable factors that can affect the experiment or computational model. Random error is unavoidable but can be reduced by making several measurements. The percentage of random error in the analytical models presented for shear strength of the tested panels can be estimated as follows. In this calculation, the experimental value has been chosen as 16,150 lb as explained before.

6.5.3.3 Calculating percentage error for the shear testing

$$\%\ \text{error} = \frac{|\text{Experimental value} - \text{Analytical value}|}{\text{Experimental value}}\ \text{X}\ 100\%$$

Analytical shear strength value (ϕV_n) = 3281.85 lb/ft = (3281.85 X 4.083 ft) = 13.4 kips

$$\% \text{ error} = \frac{|16.15 - 13.4|}{16.15} \text{X } 100\%$$

$$= 17\ \%$$

It should be noted that the analytical model presented in this research takes the initial buckling of a diagonal as the failure point for the panel. The model does not consider inelastic deformation in concrete, steel mesh, and diagonals under shear loading which are difficult parameters to capture, unless a more sophisticated finite element is used. If the effects, including strain hardening from steel components, are considered, the % error will reduce to a value smaller than 17%. For the simplified shear analysis presented in this research, which is on the conservative side, the 17% random error between analytical and experimental results under shear loading (non-ductile behavior) is justifiable.

Chapter 7. Conclusions

The goal of this study was to present MetRock SCIPs and evaluate its structural characteristics and validity from the previously large-scale testing. New shear tests were performed to supplement experimental data on MetRock SCIPs.

Experimental results showed that the average effective moment capacity for the MR panels was 66% of the effective capacity of a fully composite section (Gurung, 2019). For the panel of higher compressive strength and thickness, the higher slab span can be chosen for effective MetRock SCIP slab design. With the increase in span of panel, the value of average effective moment capacity for the MR panels decreases until a certain length (in this case 16 ft) after which the value is negative and no longer valid.

The shear tests showed shear-compressive failure of the MR panels. The patterns of load-deflection curve for both panels were similar. Both showed increase in deflection with increase in load until a certain point (ultimate shear point), following that the panels quickly lost strength and failed at the loading point. The maximum deflection was at the point where the load was applied for both panels. The panel also showed buckling of diagonal bars during post-testing inspection. The buckling load of the diagonal bars is critical for shear capacity calculation as it will be the maximum shear contributed by a diagonal. The experimental value of shear capacity is more than the analytical value for both panels which means the shear capacity values expected are less than the values of test (e.g. 72%) which are conservative, but appropriate until further testing data becomes available.

Although the alternative precast methodology utilized to produce panels in this study differs from typical MR SCIP construction, it may be a cost-effective construction method. All the precast specimens were simple to make, handle, and put together. They did not need specialized labor, making construction relatively inexpensive and simple. MR panels also have the potential to be made in the form of modular units in a precast yard. Some concepts for construction and connections were presented.

7.1 Future work

- Produce and compare outcomes of a variety of analytical and numerical prediction models.
- Detailed finite element analysis of panels under flexural, axial, and shear loading.
- Validation of adequacy of connection concepts for modular SCIPs that are proposed in this research.
- Shake table testing of a one-story building with modular SCIPs (e.g. system level testing) and comparing results against component testing.
- Inclusion of modular SCIPs in building standards.

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Appendix A

Moment capacity calculation

Self-Weight	130 lb/ft	
Depth(h)	Varies	
F'_c	3000	Psi
f_y	60000	Psi
d (distance to tension steel)	Varies	In
d'(distance to compression steel)	Varies	In
b(width)	4	Ft
As	0.522	in^2
A's	0.522	in^2
A	β1.c	
f's	E_s.((cd')*0.003/c))	
Es	29000000	Psi
L(span)	Varies	Ft
C	0.18	In
$\phi Mn =$	$0.85\ f'_c * a * b\ (d - a/2) + A's * f'_s(\text{d-d'})$	

β_1=	0.85	
$A_s.f_y = 0.85.f'_c.b.\beta_1.c + (A'_s.E_s.((c\text{-}d')*0.003/c)))$		
From this equation		
c =	0.19	In
a=β_1.c	0.162	In

	Thickness	1	In
	d=0.75in d' = 0.25 in		
	Span=	8	Ft
f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	4800.00	lb-in	
Effective moment capacity	14142.22	lb-in	
average effective moment capacity for the MR panels	9333.86	lb-in	
	0.78	kip-ft	
	Thickness	1	in
	d=0.75in d' = 0.25 in		
	Span=	10	Ft

f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	7500.00	lb-in	
Effective moment capacity	11442.22	lb-in	
average effective moment capacity for the MR panels	7551.86	lb-in	
	0.63	kip-ft	
	d=0.75in d' = 0.25 in		
	Span =	12	Ft
f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	10800.00	lb-in	
Effective moment capacity	8142.22	lb-in	
average effective moment capacity for the MR panels	5373.86	lb-in	
	0.45	kip-ft	
	Span=	14	Ft
f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	14700.00	lb-in	
Effective moment capacity	4242.22	lb-in	
average effective moment capacity for the MR panels	2799.86	lb-in	
	0.23	kip-ft	
	Span=	16	Ft
f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	19200.00	lb-in	
Effective moment capacity	-257.78	lb-in	
average effective moment capacity for the MR panels	-170.14	lb-in	
	-0.01	kip-ft	
	Span=	18	Ft
f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	24300.00	lb-in	

Effective moment capacity	-5357.78	lb-in	
average effective moment capacity for the MR panels	-3536.14	lb-in	
	-0.29	kip-ft	
	Span=	20	Ft
f's(psi)	21750.00		
ϕMn	18942.22	lb-in	
Average unit weight of panel per linear ft	50.00	lb/ft	
Moment caused by self-weight	30000.00	lb-in	
Effective moment capacity	-11057.78	lb-in	
average effective moment capacity for the MR panels	-7298.14	lb-in	
	-0.61	kip-ft	

	Thickness	1.5	In
	d=1.25in d' = 0.25 in		
	Span=	8	Ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	7200.00	lb-in	
Effective moment capacity	27333.37	lb-in	
average effective moment capacity for the MR panels	18040.02	lb-in	
	1.50	kip-ft	
	Thickness	1.5	in
	d=1.25in d' = 0.25 in		
	Span=	10	ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	11250.00	lb-in	
Effective moment capacity	23283.37	lb-in	
average effective moment capacity for the MR panels	15367.02	lb-in	
	1.28	kip-ft	
	d=1.25in d' = 0.25 in		
	Span =	12	ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	16200.00	lb-in	

Effective moment capacity	18333.37	lb-in	
average effective moment capacity for the MR panels	12100.02	lb-in	
	1.01	kip-ft	
	Span=	14	Ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	22050.00	lb-in	
Effective moment capacity	12483.37	lb-in	
average effective moment capacity for the MR panels	8239.02	lb-in	
	0.69	kip-ft	
	Span=	16	ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	28800.00	lb-in	
Effective moment capacity	5733.37	lb-in	
average effective moment capacity for the MR panels	3784.02	lb-in	
	0.32	kip-ft	
	Span=	18	ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	36450.00	lb-in	
Effective moment capacity	-1916.63	lb-in	
average effective moment capacity for the MR panels	-1264.98	lb-in	
	-0.11	kip-ft	
	Span=	20	ft
f's(psi)	21750.00		
ϕMn	34533.37	lb-in	
Average unit weight of panel per linear ft	75.00	lb/ft	
Moment caused by self-weight	45000.00	lb-in	
Effective moment capacity	-10466.63	lb-in	
average effective moment capacity for the MR panels	-6907.98	lb-in	
	-0.58	kip-ft	

	Thickness	2	in
	d=1.75in d' = 0.25 in		
	Span=	8	ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	9600.00	lb-in	
Effective moment capacity	40524.52	lb-in	
average effective moment capacity for the MR panels	26746.18	lb-in	
	2.23	kip-ft	
	Thickness	2	in
	d=1.75in d' = 0.25 in		
	Span=	10	ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	15000.00	lb-in	
Effective moment capacity	35124.52	lb-in	
average effective moment capacity for the MR panels	23182.18	lb-in	
	1.93	kip-ft	
	d=1.75in d' = 0.25 in		
	Span=	12	ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	21600.00	lb-in	
Effective moment capacity	28524.52	lb-in	
average effective moment capacity for the MR panels	18826.18	lb-in	
	1.57	kip-ft	
	Span=	14	ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	29400.00	lb-in	
Effective moment capacity	20724.52	lb-in	
average effective moment capacity for the MR panels	13678.18	lb-in	
	1.14	kip-ft	

	Span=	16	ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	38400.00	lb-in	
Effective moment capacity	11724.52	lb-in	
average effective moment capacity for the MR panels	7738.18	lb-in	
	0.64	kip-ft	
	Span=	18	ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	48600.00	lb-in	
Effective moment capacity	1524.52	lb-in	
average effective moment capacity for the MR panels	1006.18	lb-in	
	0.08	kip-ft	
	Span=	20	Ft
f's(psi)	21750.00		
ϕMn	50124.52	lb-in	
Average unit weight of panel per linear ft	100.00	lb/ft	
Moment caused by self-weight	60000.00	lb-in	
Effective moment capacity	-9875.48	lb-in	
average effective moment capacity for the MR panels	-6517.82	lb-in	
	-0.54	kip-ft	

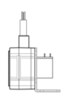
Appendix B

Potentiometer

This compact stringpot with “voltage divider” output, provides ease-of-use and flexibility for measurement ranges up to 50 inches. Made of rugged polycarbonate, the SP1 fits in small spaces, doesn’t need perfect alignment and ships with a stainless steel mounting bracket to let the user easily orient this sensor in just about any direction imaginable.

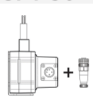
The SP1 is available with a connector, mating plug and sensor cover to protect against IP67 (wet) environments and a lower cost, “open” sensor version priced for both the budget conscious single piece user and the OEM alike.

Ordering Information



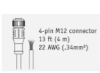
includes sensor & mounting bracket.

Part Number	full stroke range	accuracy	max. acceleration	measuring cable tension (± 25%)	cycle life
SP1-4	4.75 in (120 mm)	1.00%	15 g	7oz. (1,9 N)	2.5M
SP1-12	12.5 in (317 mm)	.25%	15 g	7oz. (1,9 N)	500K
SP1-25	25 in (635 mm)	.25%	15 g	7oz. (1,9 N)	500K
SP1-50	50 in (1270 mm)	.25%	15 g	7oz. (1,9 N)	250K



includes sensor, mounting bracket & mating connector.

Part Number	full stroke range	accuracy	max. acceleration	measuring cable tension (± 25%)	cycle life
SP1-4-3	4.75 in (120 mm)	1.00%	11 g	11.5 oz. (3,2 N)	2.5M
SP1-12-3	12.5 in (317 mm)	.25%	11 g	13.9 oz. (3,7 N)	500K
SP1-25-3	25 in (635 mm)	.25%	11 g	11.5 oz. (3,2 N)	500K
SP1-50-3	50 in (1270 mm)	.25%	11 g	11.5 oz. (3,2 N)	250K



Optional Cordset

Part No.

9036810-0040 for all SP1-xx-3 versions, an optional 13-ft. cordset with 4-pin M12 connector

SP1

Compact String Pot • Voltage Divider

Linear Position to 50 inches (1270 mm)

Rugged Polycarbonate Enclosure • IP67 Optional

Mounting Bracket & Optional Sensor Cover w/Connector

IN STOCK for Quick Delivery!

Complete Specifications

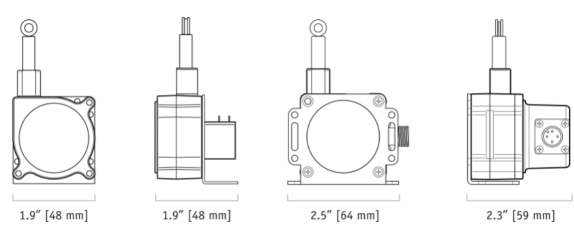
Available Stroke Ranges	0-4.75, 0-12.5, 0-25, 0-50 inches
Output Signal	voltage divider (potentiometer)
Accuracy	±0.25 to ±1.00% (see ordering info)
Repeatability	± 0.05% full stroke
Resolution	essentially infinite
Measuring Cable	0.019-in. dia. nylon-coated stainless steel
Measuring Cable Tension	see ordering information
Maximum Cable Acceleration	see ordering information
Enclosure Material	Polycarbonate
Sensor	plastic-hybrid precision potentiometer
Weight, max. (includes bracket)	.4 lbs (.19 kg)

Electrical

Input Resistance	10K ohms, ±10%
Power Rating, Watts	2.0 at 70°F derated to 0 at 250°
Recommended Maximum Input Voltage	30 V (AC/DC)
Output Signal Change Over Full Stroke Range	94% ±4% of input voltage
Electrical Connection, SP1-xx	solder terminals
Electrical Connection, SP1-xx-3	4-pin, M12 connector

Environmental

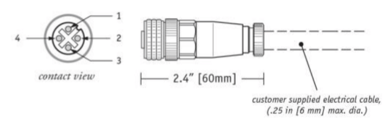
Enclosure	IP 50 (SP1-xx), IP67 (SP1-xx-3)
Operating Temperature, SP1-xx	0° to 160°F (-18° to 70°C)
Operating Temperature, SP1-xx-3	-40° to 160°F (-40° to 70°C)
Vibration	up to 10 g to 2000 Hz maximum



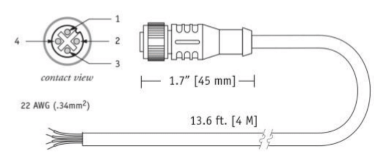
Electrical Connection

Signals	solder terminals	pin	pin - colorcode
+in	#3 (cw)	1	1 - brown
common	#1 (ccw)	2	2 - white
+out	#2 (s)	3	3 - blue
n/c		4	4 - black

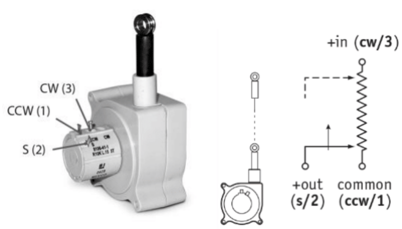
field installable connector (included)



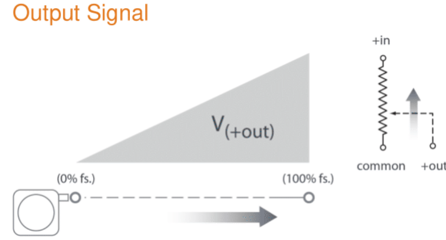
cordset, part no. 9036810-0040 (optional)



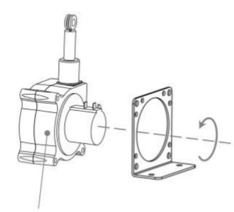
Terminal/Pin Location (SP1-xx)



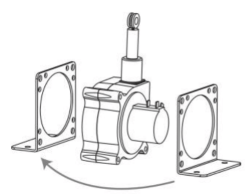
Output Signal



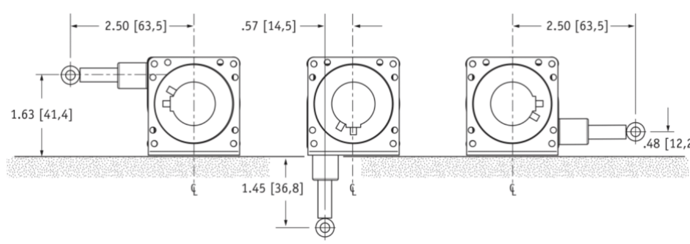
Mounting Options for SP1-xx:



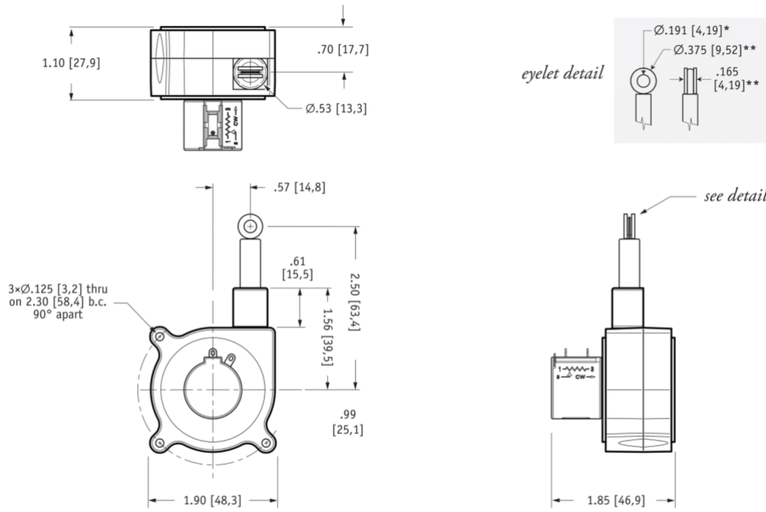
To change measuring cable direction simply remove the 3 bracket attaching screws and rotate sensor body to desired direction.



For added flexibilty, mounting bracket can easily be switched to the opposite side.



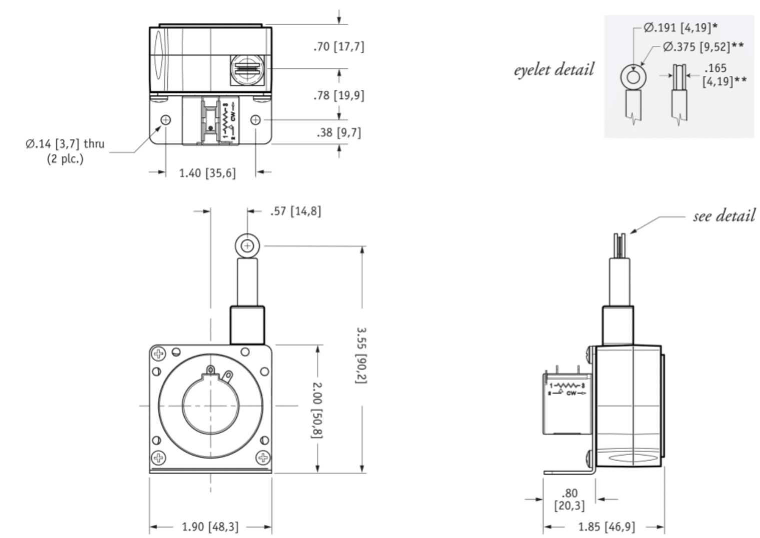
SP1-xx w/o Mounting Bracket



DIMENSIONS ARE IN INCHES [MM]
tolerances are 0.04 IN. [1,0 MM] unless otherwise noted.

* tolerance = +.005 –.001 [+0,1 –0,0]
** tolerance = +.005 –.005 [+0,1 –0,1]

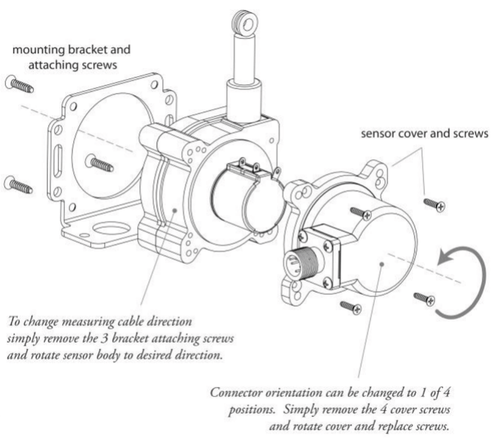
SP1-xx w/ Mounting Bracket

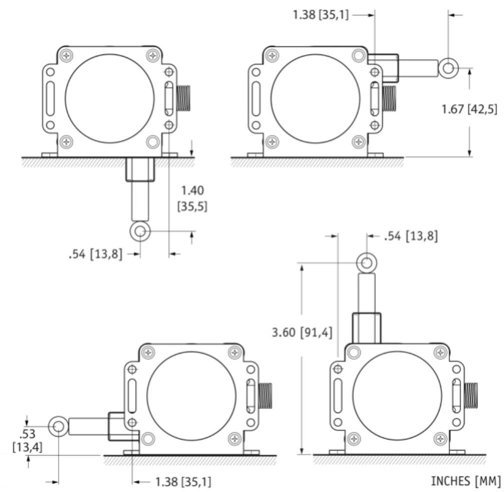


DIMENSIONS ARE IN INCHES [MM]
tolerances are 0.04 IN. [1,0 MM] unless otherwise noted.

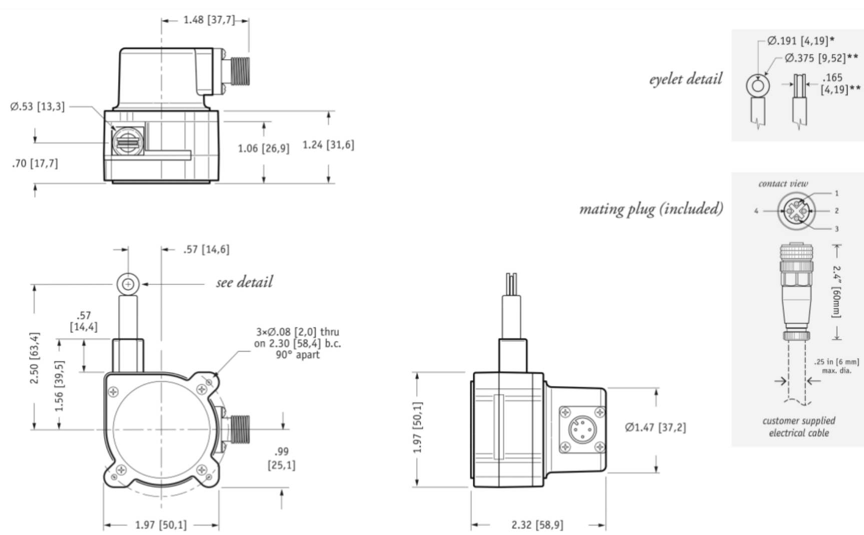
* tolerance = +.005 –.001 [+0,1 –0,0]
** tolerance = +.005 –.005 [+0,1 –0,1]

Mounting Options for SP1-xx-3





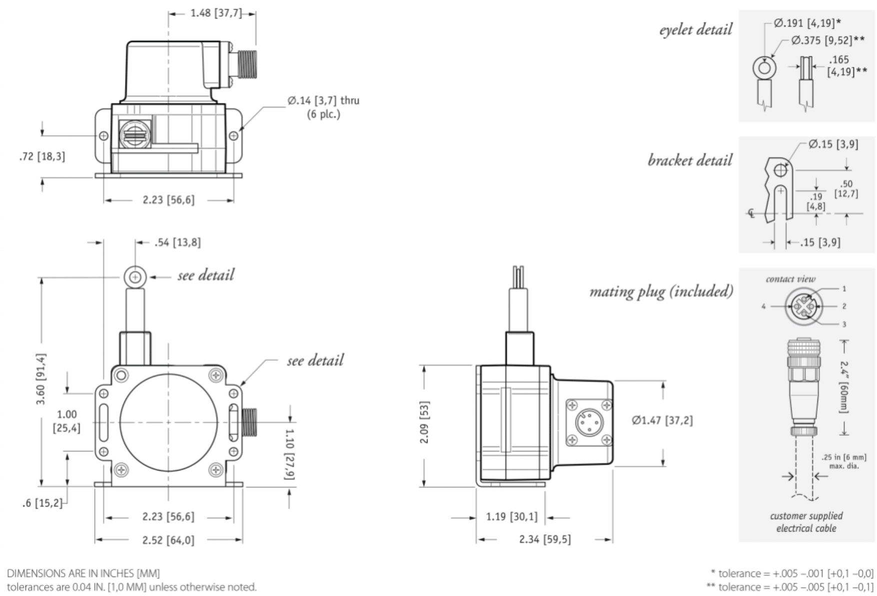
SP1-xx-3 w/o Mounting Bracket



DIMENSIONS ARE IN INCHES [MM]
tolerances are 0.04 IN. [1,0 MM] unless otherwise noted.

* tolerance = +.005 –.001 [+0,1 –0,0]
** tolerance = +.005 –.005 [+0,1 –0,1]

SP1-xx-3 w/ Mounting Bracket



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SP1 12/01/2015

Appendix C

Load cell

interface
FORCE MEASUREMENT SOLUTIONS.

LOAD CELL CALIBRATION CERTIFICATION

CONDITION: FINAL
MODEL: 2165FQX-225K SERIAL: 1043656 BRIDGE: A CAPACITY: 225 Klbf
PROCEDURE: C-1257 Mounting Per Interface Installation Instruction 15-5

INPUT RESISTANCE: 350.8 Ohm OUTPUT RESISTANCE: 350.7 Ohm
ZERO BALANCE: 0.096 %RO

TEST CONDITIONS

TEMPERATURE: 73 °F HUMIDITY: 28% EXCITATION: 10 VDC

TRACEABILITY

FORCE STANDARD:	STD-11	NIST#:	684/288317-16	DUE: 15-AUG-2018
STANDARD INDICATOR:	BRD297	NIST#:	5-B2P21-40-1	
TEST INDICATOR:	BRD298	NIST#:	5-B2P21-40-1	

SHUNT CALIBRATION

	Shunt (+/- .01%)	Output	Straight Line Conversion	Connections*
TENSION	60.0 KOhm	-1.45756 mV/V	155.88 Klbf	-Out to +Exc
COMPRESSION	60.0 KOhm	1.45648 mV/V	157.36 Klbf	-Out to -Exc

*For models wired with +Sense, - Sense or -SCal leads, resistor connections are actually to these leads in place of +Exc, -Exc or -Out respectively.

PERFORMANCE

	Rated Output	SEB Output	Nonlinearity	Hysteresis	SEB
TENSION	-2.10439 mV/V	-2.10381 mV/V	-0.044 %FS	0.032 %FS	±0.028 %FS
COMPRESSION	2.08361 mV/V	2.08259 mV/V	0.010 %FS	-0.075 %FS	±0.049 %FS

STATIC ERROR BAND (SEB) The band of maximum deviations of the ascending and descending calibration points from a best fit straight line through zero OUTPUT. It includes the effects of NONLINEARITY, HYSTERESIS, and nonreturn to MINIMUM LOAD.

TEST LOAD APPLIED (Klbf)	RECORDED READINGS (mV/V) Tension	Compression
0	.00000	.00000
45	- .42045	.41680
90	- .84098	.83359
135	-1.26170	1.25037
180	-1.68282	1.66704
225	-2.10439	2.08361
90	- .84165	.83202
0	- .00005	.00004

Interface Inc. certifies that all calibration measurements are traceable to NIST. Estimated uncertainty of measurements is 0.040% RDG. Results relate to serial 1043656 only.
DO NOT REPRODUCE THIS REPORT except in full or with Interface Inc. written approval.

TECHNICIAN : Richard Cratty

CALIBRATION DATE : 09-JUN-2018

APPROVED : Mark Clewley - Calib. Tech./Final Insp.

7401 E. Butherus Dr. • Scottsdale, Arizona 85260 • 480.948.5555 • www.interfaceforce.com

F16-3D-1017 Rev. A

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FORCE MEASUREMENT SOLUTIONS.

LOAD CELL CALIBRATION CERTIFICATION

CONDITION: FINAL
MODEL: 2165FQX-225K SERIAL: 1043656 BRIDGE: B CAPACITY: 225 Klbf
PROCEDURE: C-1257 Mounting Per Interface Installation Instruction 15-5

INPUT RESISTANCE: 350.6 Ohm OUTPUT RESISTANCE: 350.5 Ohm
ZERO BALANCE: -0.062 %RO

TEST CONDITIONS

TEMPERATURE: 73 °F HUMIDITY: 28% EXCITATION: 10 VDC

TRACEABILITY

FORCE STANDARD:	STD-11	NIST#:	684/288317-16	DUE: 15-AUG-2018
STANDARD INDICATOR:	BRD297	NIST#:	5-B2P21-40-1	
TEST INDICATOR:	BRD295	NIST#:	5-B2P21-40-1	

SHUNT CALIBRATION

	Shunt (+/- .01%)	Output	Straight Line Conversion	Connections*
TENSION	60.0 KOhm	-1.45630 mV/V	155.92 Klbf	-Out to +Exc
COMPRESSION	60.0 KOhm	1.45612 mV/V	156.77 Klbf	-Out to -Exc

*For models wired with +Sense, - Sense or -SCal leads, resistor connections are actually to these leads in place of +Exc, -Exc or -Out respectively.

PERFORMANCE

	Rated Output	SEB Output	Nonlinearity	Hysteresis	SEB
TENSION	-2.10246 mV/V	-2.10143 mV/V	-0.069 %FS	0.050 %FS	±0.049 %FS
COMPRESSION	2.08861 mV/V	2.08980 mV/V	0.080 %FS	-0.051 %FS	±0.057 %FS

STATIC ERROR BAND (SEB) The band of maximum deviations of the ascending and descending calibration points from a best fit straight line through zero OUTPUT. It includes the effects of NONLINEARITY, HYSTERESIS, and nonreturn to MINIMUM LOAD.

TEST LOAD APPLIED (Klbf)	RECORDED READINGS (mV/V) Tension	Compression
0	.00000	.00000
45	-.41927	.41889
90	-.83954	.83711
135	-1.26007	1.25471
180	-1.68103	1.67181
225	-2.10246	2.08861
90	-.84060	.83604
0	-.00002	.00004

Interface Inc. certifies that all calibration measurements are traceable to NIST. Estimated uncertainty of measurements is 0.040% RDG. Results relate to serial 1043656 only.
DO NOT REPRODUCE THIS REPORT except in full or with Interface Inc. written approval.

TECHNICIAN : Richard Cratty CALIBRATION DATE : 09-JUN-2018

APPROVED : Mark Clewley - Calib. Tech./Final Insp.

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F16-3D-1017 Rev. A

LOAD CELL CALIBRATION CERTIFICATION

interface
FORCE MEASUREMENT SOLUTIONS.

CONDITION: FINAL
MODEL: 2165FQX-225K SERIAL: 1043656 BRIDGE: A CAPACITY: 225 Klbf
PROCEDURE: C-1257 Mounting Per Interface Installation Instruction 15-5

INPUT RESISTANCE: 350.8 Ohm OUTPUT RESISTANCE: 350.7 Ohm
ZERO BALANCE: 0.096 %RO

TEST CONDITIONS

TEMPERATURE: 73 °F HUMIDITY: 28% EXCITATION: 10 VDC

TRACEABILITY

FORCE STANDARD: STD-11 NIST#: 684/288317-16 DUE: 15-AUG-2018
STANDARD INDICATOR: BRD297 NIST#: 5-B2P21-40-1
TEST INDICATOR: BRD298 NIST#: 5-B2P21-40-1

SHUNT CALIBRATION

	Shunt (+/- .01%)	Output	Straight Line Conversion	Connections*
TENSION	60.0 KOhm	-1.45756 mV/V	155.88 Klbf	-Out to +Exc
COMPRESSION	60.0 KOhm	1.45648 mV/V	157.36 Klbf	-Out to -Exc

*For models wired with +Sense, - Sense or -SCal leads, resistor connections are actually to these leads in place of +Exc, -Exc or -Out respectively.

PERFORMANCE

	Rated Output	SEB Output	Nonlinearity	Hysteresis	SEB
TENSION	-2.10439 mV/V	-2.10381 mV/V	-0.044 %FS	0.032 %FS	±0.028 %FS
COMPRESSION	2.08361 mV/V	2.08259 mV/V	0.010 %FS	-0.075 %FS	±0.049 %FS

STATIC ERROR BAND (SEB) The band of maximum deviations of the ascending and descending calibration points from a best fit straight line through zero OUTPUT. It includes the effects of NONLINEARITY, HYSTERESIS, and nonreturn to MINIMUM LOAD.

TEST LOAD APPLIED (Klbf)	RECORDED READINGS (mV/V) Tension	Compression
0	.00000	.00000
45	-.42045	.41680
90	-.84098	.83359
135	-1.26170	1.25037
180	-1.68282	1.66704
225	-2.10439	2.08361
90	-.84165	.83202
0	-.00005	.00004

Interface Inc. certifies that all calibration measurements are traceable to NIST. Estimated uncertainty of measurements is 0.040% RDG. Results relate to serial 1043656 only.
DO NOT REPRODUCE THIS REPORT except in full or with Interface Inc. written approval.

TECHNICIAN : Richard Cratty CALIBRATION DATE : 09-JUN-2018

APPROVED : Mark Clewley - Calib. Tech./Final Insp.

LOAD CELL CALIBRATION CERTIFICATION

interface
FORCE MEASUREMENT SOLUTIONS.

CONDITION: FINAL
MODEL: 2165FQX-225K SERIAL: 1043656 BRIDGE: B CAPACITY: 225 Klbf
PROCEDURE: C-1257 Mounting Per Interface Installation Instruction 15-5

INPUT RESISTANCE: 350.6 Ohm OUTPUT RESISTANCE: 350.5 Ohm
ZERO BALANCE: -0.062 %RO

TEST CONDITIONS

TEMPERATURE: 73 °F HUMIDITY: 28% EXCITATION: 10 VDC

TRACEABILITY

FORCE STANDARD:	STD-11	NIST#:	684/288317-16	DUE: 15-AUG-2018
STANDARD INDICATOR:	BRD297	NIST#:	5-B2P21-40-1	
TEST INDICATOR:	BRD295	NIST#:	5-B2P21-40-1	

SHUNT CALIBRATION

	Shunt (+/- .01%)	Output	Straight Line Conversion	Connections*
TENSION	60.0 KOhm	-1.45630 mV/V	155.92 Klbf	-Out to +Exc
COMPRESSION	60.0 KOhm	1.45612 mV/V	156.77 Klbf	-Out to -Exc

*For models wired with +Sense, - Sense or -SCal leads, resistor connections are actually to these leads in place of +Exc, -Exc or -Out respectively.

PERFORMANCE

	Rated Output	SEB Output	Nonlinearity	Hysteresis	SEB
TENSION	-2.10246 mV/V	-2.10143 mV/V	-0.069 %FS	0.050 %FS	±0.049 %FS
COMPRESSION	2.08861 mV/V	2.08980 mV/V	0.080 %FS	-0.051 %FS	±0.057 %FS

STATIC ERROR BAND (SEB) The band of maximum deviations of the ascending and descending calibration points from a best fit straight line through zero OUTPUT. It includes the effects of NONLINEARITY, HYSTERESIS, and nonreturn to MINIMUM LOAD.

TEST LOAD APPLIED (Klbf)	RECORDED READINGS (mV/V) Tension	Compression
0	.00000	.00000
45	-.41927	.41889
90	-.83954	.83711
135	-1.26007	1.25471
180	-1.68103	1.67181
225	-2.10246	2.08861
90	-.84060	.83604
0	-.00002	.00004

Interface Inc. certifies that all calibration measurements are traceable to NIST. Estimated uncertainty of measurements is 0.040% RDG. Results relate to serial 1043656 only.
DO NOT REPRODUCE THIS REPORT except in full or with Interface Inc. written approval.

TECHNICIAN : Richard Cratty

CALIBRATION DATE : 09-JUN-2018

APPROVED : Mark Clewley - Calib. Tech./Final Insp.

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