WY-20/03F

FINAL REPORT

State of Wyoming
Department of Transportation

Design and Performance Evaluation of a Semiflexible Snow Barrier for Avalanche Protection

By:
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16. Abstract

A new type of avalanche risk reducing snow supporting structure called a "snow supporting umbrella" or "SSU" was investigated for its potential effectiveness in preventing natural release of snow avalanches. Because no existing design specifications or other publications address how the geometry, member arrangement, and connection details of a SSU should be selected, a research project to develop the SSU concept and a generic design process was funded by and performed for the Wyoming Department of Transportation (WYDOT). Additional motivation for the project included the need to reinforce a region of the Milepost 151 Avalanche starting zone where small natural snow slab releases have occurred in between two rows of existing rigid steel snow supporting structures. An analysis of the interplay of various design parameters was performed, followed by application of a structural analysis and design process for the design of three SSU to be fabricated and installed at the project site. Due to the lack of redundancy of the single ground anchor foundation of the SSU, a novel load-limiting slip device for the connection between the SSU and foundation was developed and implemented on the fabricated SSU. Three SSU were installed at the site where the previous small avalanches were observed and then monitored over a period of two winter seasons. No subsequent avalanche activity was observed after the row of SSU was added to the facility. Annual inspections of the SSU over two summer periods indicated that the SSU performed as intended and without noticeable distress in any of its elements. Installation of the SSU at the project site required significantly less labor on-site in the steep starting zone area compared to what would be needed to install rigid steel snow bridges or flexible snow nets. A cost comparison using data from four different passive avalanche defense projects in the United States including rigid, flexible, and semiflexible snow supporting structures indicates that the SSU concept has the potential for significant construction cost savings over the other two systems, primarily due to reduction in the number of foundations and the required onsite installation labor.

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APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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INTRODUCTION

1.1 Background

There is a long history of use of constructed avalanche defense systems across Europe, whereas, in the United States the last 50 years of avalanche mitigation has focused on mitigating danger by artificial release of snow avalanches via detonation of hand charges in the starting zone or by impact in the starting zone by projectiles fired by artillery. More recently, the Wyoming Department of Transportation (WYDOT) has embraced the concept of limiting the ability of avalanches to release by retaining the starting zone snowpack in place with snow supporting structures (SSS). This approach is termed "constructed avalanche defense". Strictly speaking, SSS are any type of structural system installed across a potential avalanche starting zone that has a purpose of holding the snow in place so that its release is prevented, and SSS are classified as either rigid, flexible, or semiflexible. In the last decade, WYDOT deployed a series of rigid "snow bridges" at the Milepost 151 Avalanche near Jackson, Wyoming, and this system has eliminated the natural large snow slab avalanches that historically reached U.S. Hwy 89/191 at a frequency of once or twice per winter season. The impact of this strategic move away from active control with explosives has been to eliminate the need for WYDOT maintenance personnel during intense storm periods, since they must no longer monitor conditions at the avalanche site to decide if road closures are prudent. Figure 1 displays a snow bridge (or SSS) installed at the Milepost 151 site.

Figure 1 Snow bridges installed at the Milepost 151 site

Distinctly different, yet with the same aim of retaining large swaths of snowpack in the starting zone, are flexible and semiflexible snow supporting structures that utilize an interwoven fabric of steel netting as the snow supporting surface. Snow nets are flexible systems that utilize the netting spanning between upright poles, which must be anchored with numerous cables to provide system stability (see Figure 2). This system has a primary disadvantage of requiring a relatively complex system of guy cables connected to ground anchors, which results in more intensive construction effort. A relatively new development in Europe has been the semiflexible system that uses a combination of "rigid" crossbeams and wire rope netting but typically requires only one anchor point to ground upslope of the unit. This type of snow supporting structure has been termed a “snow umbrella” and is referred to as a snow supporting umbrella (SSU) herein. Figure 3 provides a photograph of a series of SSU installed in the French Alps region.

Because the SSU designs used in France and Italy are proprietary, no engineering analysis and design guidance exists that is available to the public. Moreover, based on a comprehensive literature review at the time of initiation of this project, no published articles or other reference materials on how the SSU should be designed exist. Therefore, a comprehensive analysis of the behavior and response of the SSU system and development of design guidelines and specifications is needed before this system can be implemented for avalanche hazard mitigation in the United States.

Figure 2 Flexible snow nets in France

Figure 3 Semiflexible "snow umbrella" avalanche mitigation units in France

1.2 Problem Statement

This project provided valuable information on the design and performance of a new lightweight, easily constructible SSS for avalanche starting zones, to be used for the purpose of avalanche risk reduction near transportation facilities within the State of Wyoming. For many decades, the WYDOT has managed the danger to motorists traveling on roadways adjacent to known avalanche paths using active defense measures. These measures typically require winter maintenance staff to forecast the potential for avalanche activity, and when the probability of slides becomes significant, enact road closures and use artillery or explosives to artificially release avalanches. If crews are successful in triggering a slide, snow-moving equipment is brought in to clear the debris before the road is reopened. This approach requires *immense personnel resources* and carries the inherent *risk* involved with detonation of artillery/explosives. The alternative to active control is passive, avalanche starting zone constructed defense that operates stand-alone with no need for WYDOT staff during winter, and that virtually eliminates danger to the traveling public in mountainous areas.

The prototype system being proposed for evaluation was based on a new type of "snow umbrella" used successfully in Europe to retain snowpacks vulnerable to natural release, and consists of wire netting grid supported by only two structural steel cross members and with one foundation per unit. This type of system has the following potential benefits:

- *Lightweight* – less than approximately 500 lbs per unit for the tallest expected unit.
- *Adaptable* to any terrain condition because of three-point bearing/connection to ground.
- Self-reacting structural system requires only one ground anchor foundation per unit, which can be installed with hand tools, and therefore, *minimizes construction time* and *disturbance* to the site.
- Units can be placed in groups side-by-side and sited in irregular patterns to *minimize visual impacts* to the surrounding landscape.
- *Minimal visual impact* to landscape because of the limited number of large components.
- Relatively *easy installation*: can be flown into remote sites in groups and anchored to one ground anchor per unit by hand.
- Expected to be very *cost-effective* based on above characteristics.

There are no design standards, either European or domestic, specifically focused on this new type of SSU, although the Swiss Guide [1] could be used to generate snow loading forces. Additionally, at the Milepost 151 Avalanche site near Jackson, Wyoming, a small region of the slope near the upper reaches of the starting zone lacked sufficient support from the existing rigid SSS and was in need of additional stabilization of the snowpack. In January of 2015, a small natural avalanche released from this area and its debris impacted several SSS below the release zone. Since SSS are not generally designed for dynamic loading from an active avalanche, it is vital to protect the existing SSS from future slides by adding a row of SSU to the release zone noted for the 2015 event.

1.3 **Study Objectives**

This research study was set up to accomplish the following project objectives:

- Develop a general design procedure that could be applied to the design of snow umbrella units for any geographic location and site.
- Produce a site-specific design of snow umbrella units and fabricate and install the units in the region where the snowpack has released and impacted existing snow bridges.

- Monitor the on-going performance and "health" of the array of snow umbrella units by visual evaluations and using electronic instrumentation.
- Provide the requisite fundamental understanding and background information to be used in the development of a U.S. design guide or standard for snow umbrellas.

1.4 Research Tasks

The following task descriptions describe the work activities that were used to achieve the above detailed research outcomes.

1.4.1 Task 1: Development of Design Criteria, SSU Geometry, and Design Procedure

Task 1 involved the creation of design criteria, SSU geometry and connection details, and a generic design procedure to be used for site-specific designs in the future. This task was performed in the first year of the project between the summers 2016 and 2017. Design criteria for SSU addressed the following aspects of their design:

- Snow loading – guidance on the use of the Swiss Guide to generate snow loads due to creep and glide forces as a function of site characteristics, and SSU geometry and configuration.
- Basic structural system layout – how different structural components are arranged and connected in order to create a surface that can support the snowpack effectively.
- Selection of type of structural members (materials, cross-section shape) and their connection details.
- Foundation design criteria and details.

1.4.2 Task 2: Fabrication and Installation of SSU and Instrumentation

Three snow umbrellas were fabricated and installed at the Milepost 151 site between summer and late fall of 2017. Existing instrumentation, which had been used by InterAlpine to monitor snow pressures at the Milepost 151 site for a previous research project, was adapted for use to monitor meteorological site parameters, including snow depth and temperature, and to measure SSU structural deformations. Due to sub-freezing temperatures at the site when the SSUs were installed, in October of 2017, this instrumentation could not be installed until the summer of 2018. Delays in construction in 2017 were a result of the very long fire summer fire season and the resulting lack of any available helicopters to ferry construction equipment up to the site.

1.4.3 Task 3: Data Collection, Analysis, Implementation Recommendations, and Develop Final Deliverables

This task involved collection and reduction of data from the instrumentation, visual inspections of the installed SSU over two summer periods, and development of implementation recommendations based on all of the results of all tasks in the project.

Data from the 2018 – 2019 winters season was downloaded in 2019, and significant problems were noted in the recorded values. Upon inspection of the SSU, during 2019, it was found that much of the instrumentation was damaged during the winter season. Where snow depth sensors had been mounted to steel masts, they were found to be hanging by their electrical wires, and therefore, did not yield any reliable values for snowpack depth adjacent to the SSU. The cellular modem antenna was also ripped from its mounting location and dangling from its electrical wires. Finally, strain gages that had been mounted to the SSU became detached, and therefore, were not able to accurately measure internal strain in the steel central tube. Due to this, no accurate electronic data could be obtained from analysis of the recorded transducer signals. The cause of such extensive damage to the installed equipment is unknown. A previous project at the site, which focused on experimental measurement of performance parameters on the rigid SSS, yielded excellent results and several publications were produced based on the project. Discussions with the WYDOT avalanche technician about whether the 2018 – 2019 winter season was particularly severe did not identify any specific weather event that could be the cause of the observed damage. One explanation that would explain devices being ripped from their supports is a severe icing event that would have added mass and surface area to the exposed transducers, followed by a significant wind event.

1.5 **Outcomes**

This research project provided invaluable information that will help to evaluate the relative merits of SSU as an effective and economical method to mitigate avalanche hazards by arresting snowpack at the starting zone. The work provides insight on how SSUs respond under snow loading and the best practices for their design, including connection details and specifications. The project has additionally prevented further releases of the small region of snowpack at the project site thereby protecting the existing SSS from damage due to dynamic impact loading.

1.6 **Report Organization**

This report includes seven chapters, each of which is described in overview below.

- Chapter 1 provides an introduction to the project, including motivation for it, objectives of the work, and the research tasks identified to accomplish the objectives.
- Chapter 2 provides a detailed consideration of the design features of a snow umbrella, with discussion of how the various geometric aspects of the unit influence its response and design.
- Chapter 3 includes the in-depth structural analysis and design of the final snow umbrella concept selected for advancement.
- Chapter 4 details the foundation analysis and design.
- Chapter 5 documents the construction of the SSU at the project site and describes the instrumentation of one SSU unit.
- Chapter 6 provides a discussion of the comparative construction costs of various types of snow supporting structures.
- Chapter 7 provides a summary of the research work, conclusions, and recommendations for future research work.

1.7 **Copyright and Use by Others**

This report documents a design process for SSU developed by the report author based on established structural engineering principles and in consideration of the snow loads stipulated by the global standard for snow loading within an avalanche starting zone – the “Swiss Guide” (FOEN 2006). While others may adopt the process described herein for the design of snow umbrellas for other sites and projects, the site-specific design presented within the report may only be used by WYDOT. Design details and concepts such as the slip connection are protected by copyright and the author of this report and WYDOT assume no liability for their use by others at other avalanche mitigation project sites.

SSU SYSTEM DEVELOPMENT

1.8 Introduction

The concept of the SSU originates in Europe where this type of structure has been used in Italy and France. The basic function of any snow supporting structure is to retain the snowpack so that release of natural avalanches is prevented. The primary advantage of the SSU is the simplicity of the foundation system. A snow bridge (see Figure 1) may require as little as four and as many as six independent ground anchors for stability, whereas the SSU requires only one ground anchor at the uphill side and two steel plates for bearing on the ground surface at the downhill side of the unit. Since installation of ground anchors represent a significant expense in the overall cost of a constructed passive avalanche defense facility, reduction in the required number of anchors is expected to dramatically lower costs. A basic drawing schematic of a side elevation view of a SSU is shown in Figure 4.

The most comprehensive technical guide document for planning and design of constructed avalanche defense systems is the “Swiss Design Guide” (FOEN 2006). This document provides detailed guidance on the layout of snow supporting structures across an avalanche starting zone, as well as specifications for calculation of snow pressures that act on obstacles within a snowpack positioned on a sloped surface. Because rigid snow supporting structures have the longest history of use for avalanche defense, the Swiss Guide additionally contains more detailed guidance on rigid structure design, as compared to flexible or semiflexible structures. In fact, the SSU, as a concept, does not appear in the Swiss Guide, and therefore, engineers are faced with decision-making with regard to geometry and details of the SSU. Therefore, a systematic evaluation of the SSU concept and the various features that can best achieve the desired goal of avalanche risk reduction at the lowest initial and long-term cost is warranted.

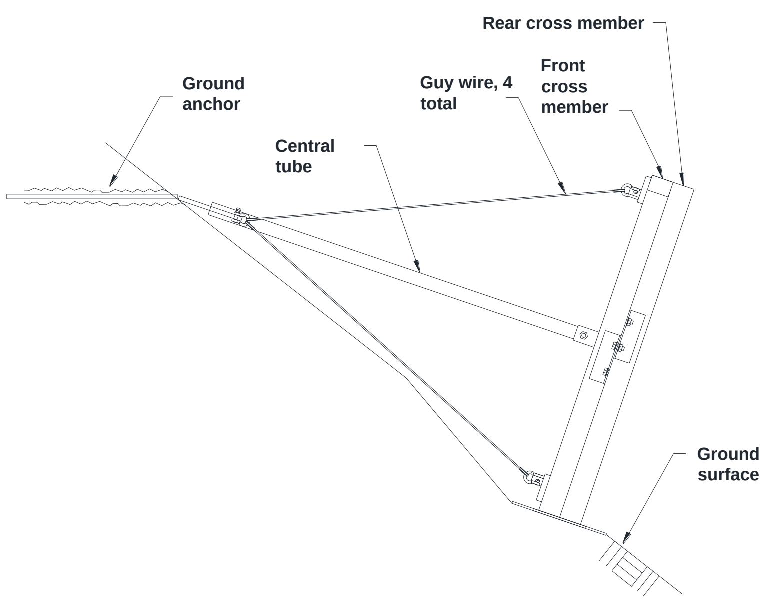


Figure 4 Snow supporting umbrella (SSU) side view

In terms of broad aspects of behavior, the primary objective of any snow supporting structure is to retain the snowpack in place for a given design snowpack depth, and to do so with a low probability of failure. Thus, a SSS can be thought of in a sense as a retaining wall or fence that is subjected to snow pressures – both lateral (or normal to the supporting surface) and vertical. The designer is tasked with providing an efficient structural system that will retain snow adequately and transfer the imparted snow forces into the earth. Simplicity of design and construction, and reliability, in terms of performance, are of utmost importance. With these in mind, the original concept development of the SSU was surely motivated by the desire to minimize foundation "work", or more broadly, reduce construction activities that must take place on the steep slopes of a starting zone. A simple deployment operation where a SSS can be placed with minimal effort into its final position is desired.

Flexible snow nets require a complex system of guy anchor cables to stabilize steel masts in between that span netting (see Figure 2). This requires significant manpower onsite to lay out and install masts and guy cable ground anchors, tensioning in place of span wires in between masts, etc. Rigid snow bridges require very precise ground anchor installation so that the rigid structural steel of the unit mate with foundations with reasonable tolerances. Therefore, neither of these systems achieve the simplicity sought after in an effective constructed defense system. On the contrary, the SSU achieves both minimization of foundations and onsite construction effort since units can be flown via helicopter to the site, lowered into position, and connected to the ground anchor by a simple shackle connection with no additional assembly required onsite.

1.9 **Design Considerations**

1.9.1 Overview

The SSU unit is composed of a system of cross members, central tube, guy cables, and net supporting surface that resemble the form of an inverted umbrella, hence the name of the system. Strung around the four corners of the SSU (ends of cross members, shown as gray in Figure 5 and forming an "X" in the figure) is a perimeter steel cable (magenta in Figure 5 and forming a square around the cross members) to which an infill steel net is connected (red in Figure 5 and forming a series of orthogonal lines in between the perimeter cable sides). The basic SSU unit geometry is depicted in Figure 6, which shows a side elevation view of a SSU while Figure 7 depicts a SSU, with snow pressures, their resultants, and reaction forces.

The overall load path and function of each of the SSU elements is described as follows: Snowpack creep and glide down the slope is prevented by both the infill net surface and to some extent bearing against the cross members. Pressure perpendicular to the infill net cables induces cable tension and each individual infill net cable spans from the perimeter cable to the point where it crosses a cross member, and then again to the opposite side perimeter cable. Because the infill cables are anchored to the perimeter cable, slope parallel pressures acting within the span of the infill net are transferred to the cross members and the perimeter cables. Load path then continues within the perimeter cables to the ends of the cross members that support the perimeter cables, and the cross members thus experience forces perpendicular to the plane that contains the cross members. Additionally, because the infill net cables are placed into tension under the effect of snow pressure, inward pulling forces act on the perimeter cables, which in turn transmit these to the ends of the cross members. By this action, the cross members are thus also subjected to significant axial compression forces that can cause buckling behavior. To reduce the bending moment effects within the cross members, guy anchor cables support the cross members near each outside corner of the net surface, and thereby, a significant portion of the slope parallel snow pressure is transferred directly back to the uphill foundation via tension in the guy anchor cables. Snow pressures acting on the cross members away from the guy anchor cables, either due to the bearing of the infill net over the cross members or through direct snow pressure acting on the cross members, is transmitted via internal shear to the center termination point of the cross members at the central tube. These end shears are transferred to the central tube inducing tension in the central tube, and the load path is up through the central tube as tension force to the uphill anchor point. As shown in Figure 7, the central tube also experiences internal bending moment due to the snow pressure that acts normal to the axis of the tube.

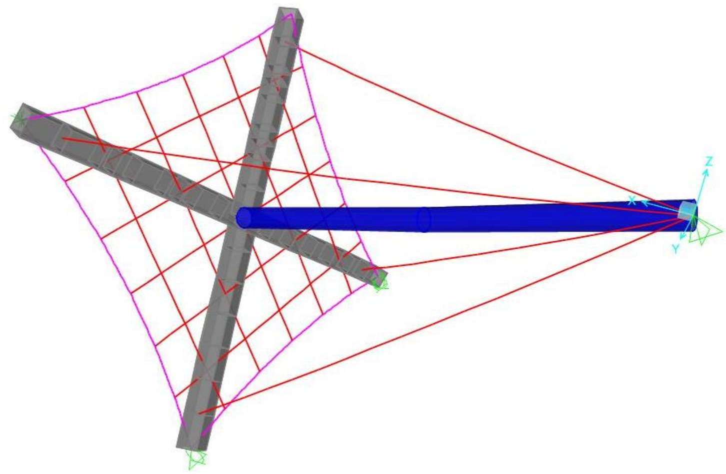


Figure 5 Three-dimensional view of SSU model

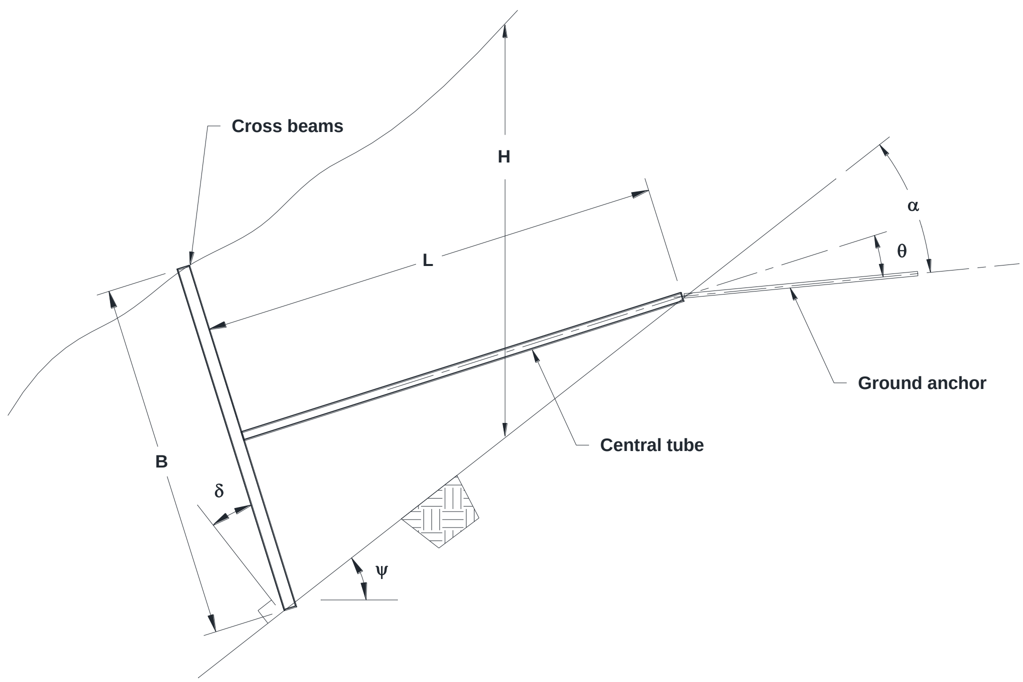


Figure 6 Basic geometry of SSU, foundation, and slope

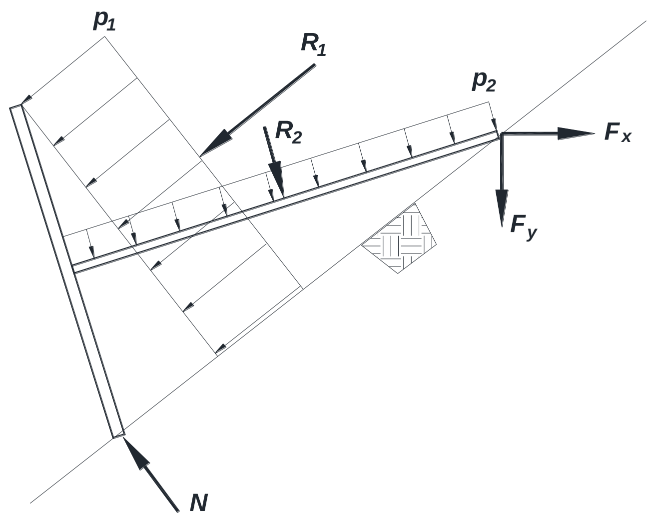


Figure 7 Snow loading and reaction forces

In summary, infill net, perimeter, and guy anchor cables are subjected to tension forces only, while cross members experience axial compression, and internal shear and bending moment, and finally the central tube is subjected to axial tension, and internal shear and bending moment. The tension carried within the central tube and guy anchor cables is transmitted to the earth at the uphill foundation. Because this force is a pulling force, the uphill foundation is required to be an anchor that transmits tension force adequately into the earth.

1.9.2 SSU Geometry

A uniform slope-parallel snow pressure, p_1, acting over the depth of the SSU, is prescribed by the Swiss Guide as a simplification to the complex and nonlinear pressure with depth of snow (Figure 7). This is the primary snow load acting on the SSU, and is responsible for most of the internal SSU force and moment, and external reaction forces. The resultant force of this snow pressure is depicted as R_1,, in Figure 7. Due to the weight of snow and snowpack settlement, the central tube is also subjected to a uniformly distributed snow pressure, p_2, with resultant force R_2. While this snow load effect must be taken into consideration, it influences primarily the internal bending moment and shear within the tube and its design. Additional details on snow pressures used for this project are provided in a later section of the report.

The design snowpack depth is parameter H(Figure 6), and this is selected based on historical records for snowfall and snowpack depth and should typically correspond to a 100-year return period snow depth. At the Milepost 151 study site, the design vertical snow depth is 6.5 feet. Design snowpack depth in turn dictates the cross member dimension B, taking into consideration the slope angle ψ, and the angle between a slope normal and the cross member plane, δ. With reference to Figure 6, B can be found as:

$$B = H\frac{\cos\psi}{cos\delta} \tag{1}$$

The slope angle at the study site varies but at the steepest reaches in the upper starting zone, is approximately ψ = 37 degrees. However, for reasons described later, the slope angle for design of the SSU was taken as ψ = 40 degrees. The angle between the cross members and a normal to the slope depends on the length of the central tube L, relative to one-half the dimension B, by:

$$\tan\delta = \frac{0.5B}{L} \tag{2}$$

Selection of the central tube length L is of critical importance as it influences two different design considerations. The central tube is subjected to vertical snow pressures, and therefore, experiences internal bending stresses. To minimize central tube cross section size, the span length of the tube should be minimized as bending increases in proportion to length squared. However, overturning stability is provided by the compressive ground reaction force N acting on steel bearing pads at the base of each cross member, and as the central tube length decreases, the required soil reaction N increases. This is determined by application of static equilibrium principles and summation of external moments about the uphill end of the central tube:

$$\sum M = 0{:}\ -N\frac{L}{\cos\delta} + R_1\frac{B}{2} + R_2\frac{L}{2} = 0 \tag{3}$$

With all parameters known except the compressive reaction force N, the force can be solved for as:

$$N = \frac{(R_1 B + R_2 L)}{2L}\cos\delta \tag{4}$$

Clearly it is evident that as central tube length L decreases, the magnitude of N increases. This has two implications. Firstly, as the downhill foundation reaction increases, so too must the size of the steel bearing plates due to the need to limit soil pressures to acceptable or allowable values. Thus, to minimize cost of structural steel a small reaction N is desired. This, however, competes with the second influence of reaction N: as N increases, the resultant ground anchor reaction force F (Figure 7) becomes more horizontal, which has positive implications for design of the ground anchor itself, as described in detail below.

The ideal application of force to a ground anchor is direct tension without any component of force perpendicular to the axis of the anchor (i.e. lateral loading). This is because forces not aligned with the axis of the anchor induce bending and the anchor must act as a laterally loaded pile. Ground anchors for a SSU could consist of steel reinforcing bars grouted within drilled holes, or alternatively flexible steel cables embedded in the same fashion. In either case, consideration of the capabilities and limits of the drilling operation is critical. The easiest arrangement for drilling is a hole with its axis perpendicular to the ground surface. As the angle of incidence ϕ (Figure 8) between the hole axis and ground surface becomes smaller, drill operations become more challenging as drill bits can tend to “walk” or move laterally parallel to the ground surface. With reference to Figure 8, anchor orientation “A” has the benefit of ease of drilling the hole but the disadvantage of a large component of applied load F perpendicular to the anchor. Case “B” is a ground anchor whose hole will be more difficult to drill but has its axis more aligned with the applied load F, therefore, experiencing less lateral loading and internal bending. Based on conversations with drilling operators and for typical avalanche starting zones slopes, any inclination above a horizontal would be difficult to drill but a horizontal ground anchor should be attainable for most projects.

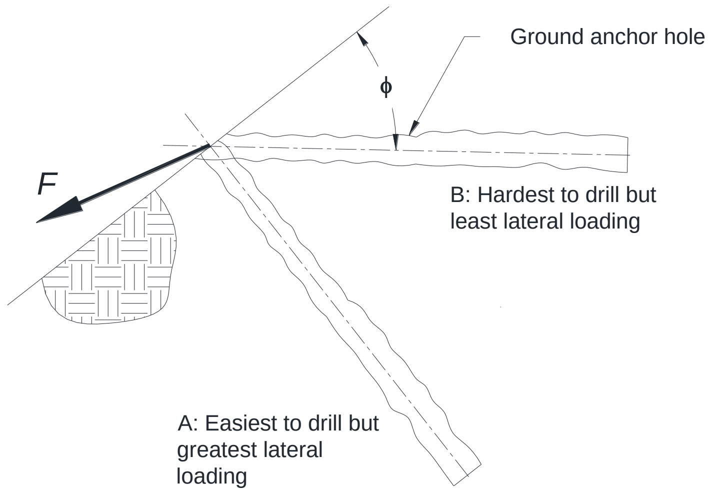


Figure 8 Ground anchor orientation with respect to ground surface

In summary, the "height" B of the SSU is mainly determined based on design snowpack depth and slope angle. The length of the central tube should be minimized in order to minimize tube material (both length and cross section size) and hence cost. However, tube length cannot be so small that the downhill soil reaction N becomes too large, resulting in the need for large steel cross member bearing plates. Finally, all of the above parameters influence to some extent the total resultant snow load applied to the uphill foundation, and final selection of SSU geometry should consider this aspect. Ideally, if possible, geometry should be chosen so that lateral loading of the ground anchor is minimized, i.e. the snow load resultant acting on the uphill foundation should be aligned as much as practical with the anchor axis. A detailed calculation of the resultant snow loading at the uphill foundation is provided later in the report.

1.9.3 Characteristics of Snow Loading on SSU

This section discusses very broadly the snow pressures acting on a structure located within an avalanche starting zone. The primary load effect is slope-parallel pressure, and this pressure varies both spatially and temporally. For a given snowpack depth and slope ground surface condition, the highest pressures will be developed on an isolated structure of finite width (parallel to contour lines) with no other structures nearby. This is because the snowpack tends to move around the edges of the structure and down the slope. The opposite condition exists when a structure is infinitely wide and the snowpack is completely restrained from down slope movement. With this in mind, SSU units that are near the end of a row of units experience larger snow pressures than units located closer to the middle of a row. This phenomenon is called "end-effects", and the implications in design are that end units, or "exterior" units, should be more robust in order to handle the larger end-effect pressures, and "interior" units can utilize smaller member sizes (e.g. smaller cable diameters, or cross member section size). Additional details of how the SSU were designed to account for this effect for this project are found later in this report.

Snowpack in general will tend to move down the slope in a direction where the slope has the highest gradient. Depending on how a row of SSU units is located with respect to this gradient, some degree of snow loading parallel to a contour line is possible. Because the SSU has only a fixed foundation condition at the upper foundation (ground anchor), lateral movement of the bearing pads under the cross members is possible. This is actually a positive design feature of the SSU, small sideways movements can be accommodated if the bearing plates slip but the SSU will always tend to align itself with the resultant movement of the snowpack down the slope. It is likely that some small sideways displacement of the downhill side of the SSU may occur during the first winter season after construction but then should be minimal in subsequent winters.

1.9.4 Single Point Uphill Anchorage

One of the positive features of the SSU is the need for only one ground anchor to provide stability – the single uphill foundation. Ironically, this attribute can also be considered a potential drawback, at least from the standpoint of structural redundancy. The term “redundant” means extra, or repeated, and within the context of structural engineering, the presence of redundancy is a positive characteristic since it implies that failure of one element of the structure does not lead to failure of the entire structure. With the SSU, if the uphill ground anchor were to fail (e.g. fracture of the reinforcing bar or bond slip failure between grout and soil), static equilibrium would no longer be maintained, and the SSU would likely begin to accelerate down the slope. Thus, a single point positive anchorage to the earth represents a non-redundant design, and very careful and conservative design of the ground anchor system would be required to ensure a low probability of failure of the avalanche mitigation system.

A novel concept proposed, and detailed in this report, is the use of a load-limiting “anchor slip device” in between the SSU itself and the ground anchor. This mechanism is analogous to a fuse in an electrical circuit that limits the amperage or voltage that can be applied to a device within the circuit. The basic idea is to devise a slip mechanism, whereby, if a SSU experiences an overload, the mechanism releases in a controlled fashion and limits the force input to the ground anchor. The “controlled slippage” would need to be of small enough magnitude that movement of the SSU down the slope would not be too large.

The anchor slip device proposed in this work consists of a steel wire rope cable that anchors to a steel slip plate at the tip of the central tube, and then to the ground anchor at its other end. Wire rope clips, or “fist grip” clips, are used to apply clamping pressure to the anchor slip cable that lies flat against the steel slip plate, thereby producing a friction connection between the SSU and the ground anchor. If designed correctly, the slip cable would begin to slip through the fist grip at a specific load level that is some percentage smaller than the design ground anchor capacity. As long as slippage occurs at a load level less than that corresponding to foundation failure, static equilibrium can be maintained. The amount of slippage to be expected would depend on the design load slip level. It is envisioned that slippage of an anchor might occur only occasionally over a period of five to ten years. Each summer season, a quick visual inspection of the entire facility would be made to check if any units experienced slippage, and if so, then the SSU would be pulled back up the hill into the correct location using a come-a-long (hand operated winch). The actual magnitude of movement down the slope would be on the order of a couple of inches up to say, two feet. The anchor slip cable would have a tail length (the extra length on the downhill side of the fist grip) of several feet to accommodate the slippage. Specific design details of the anchor slip device are presented in a subsequent chapter of this report.

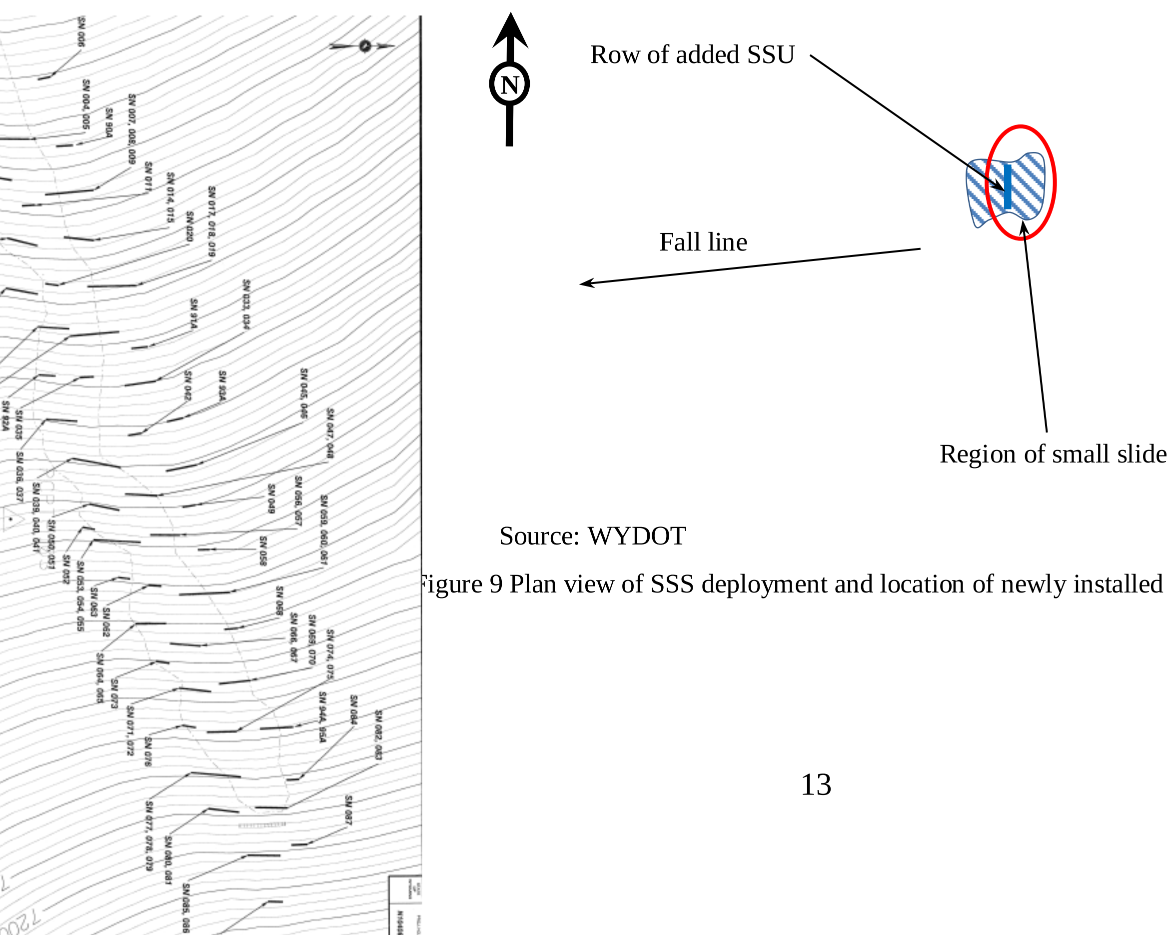
STRUCTURAL ANALYSIS AND DESIGN

1.10 Introduction

Structural design of most of the elements of the SSU follows established structural engineering practice. The analysis needed to accurately determine the distribution of snow loads within the infill cable net and perimeter cable, however, requires a more sophisticated consideration of system behavior. Specifically, distribution of snow loads within the infill net will likely be in two dimensions, which is not easily accounted for using simple hand calculations. Furthermore, the tension force induced in a cable structure is highly sensitive to cable deformation, and quantification of these deformations is very difficult using closed-formed solutions. Despite this, first-order approximations can be made using simplifying assumptions, like one-way cable span behavior, and results of the more basic analysis can be used to check the results of more refined structural models. This chapter of the report describes in detail the location of the new SSU within the Milepost 151 facility, the snow loading used for analysis and design of the SSU, and the structural analysis methods used to arrive at realistic values for the internal force distribution within the SSU unit.

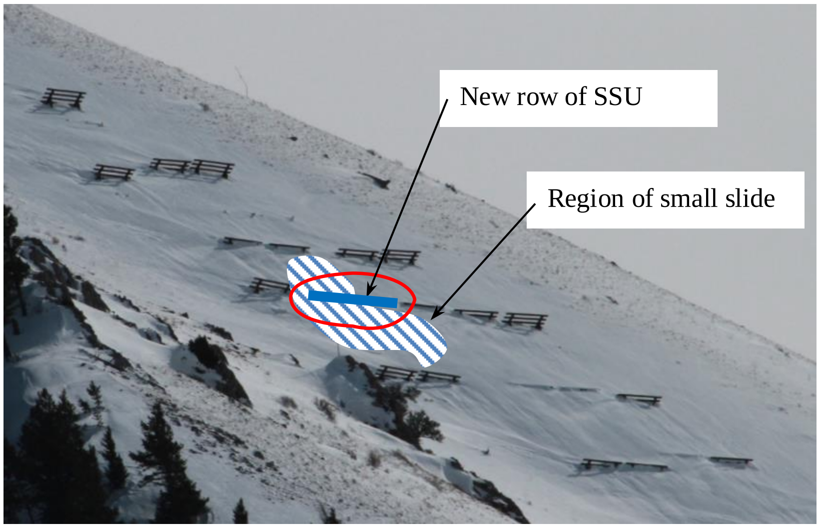
1.11 Location of SSU at Project Site

One of the primary motivations for this project was the need to reinforce a small tile of snowpack in the upper regions of the Milepost 151 Avalanche starting zone. In 2015, a small natural slide was observed to have occurred and avalanche debris impacted a couple of rows of the existing SSS. Although no damage to SSS was observed during inspection in the subsequent summer season, it was recommended that additional structures be added within the region of avalanche release. Figure 9 provides a plan view of the Milepost 151 site with SSS numbers, and the red oval on the right-hand side of the figure highlights the region where the small slide occurred. Figure 10 depicts similar information but overlain on a photograph of the Milepost 151 site.



Source: WYDOT

Figure 9 Plan view of SSS deployment and location of newly installed SSU



Source: WYDOT with modifications by InterAlpine

Figure 10 Photograph Milepost 151 Avalanche site with small slide region and added SSU

1.12 **SNOW LOADING**

The Swiss Guide provides a process for calculation of snow pressures and forces based on site conditions and structure attributes. For complete details of the Swiss Guide provisions, the reader is referred to the Swiss Guide document and the research report *Performance Evaluation of Rigid Snow Supporting Structures at the Milepost 151 Avalanche, Jackson, Wyoming* FHWA WY-18/01F (Hewes, 2017). Only the primary parameters needed for snow pressure calculation and the final pressures are discussed herein.

Rather than developing a SSU unit design that could be used only at the Milepost 151 site, it was decided to establish snow load parameters that would yield snow pressures near the upper limit of the range of possible values. This was considered reasonable since the site-specific snow load parameters for the Milepost 151 site were already close to values that would be considered extreme. Specifically, the ground slope angle was chosen as ψ = 40 degrees instead of the 37-degree slope at the Milepost 151 site. Site elevation was chosen as 8500 feet instead of the approximate 7200 feet of the Milepost 151 site. The final snow load parameter than has a significant influence on magnitude of snow pressure is the glide factor. This parameter accounts for ground surface roughness conditions that influence the extent to which a snowpack can slip over the ground surface. Essentially, for smooth ground conditions, the lack of friction between the bottom of the snowpack, and ground surface results in very high snow pressures. For the Milepost 151 site, the design glide factor was 3.0 but for design of the SSU for this project, a factor of 3.2 was used – this is the largest glide factor stipulated by the Swiss Guide. With the above selected snow parameter values, the snow pressures used for design will result in a SSU unit that could be utilized for many different avalanche mitigation projects within the state of Wyoming.

A detailed spreadsheet with calculations of snow pressures is provided in Appendix A of this report. Two different uniform snow pressures must be considered in the design of any snow supporting structure. Where structures are very wide (along a contour) or where individual units are placed side-by-side in long rows, snow pressures within the “interior” region, i.e. not near the end of a row, are almost an order of magnitude smaller than those that develop near the end of the unit or row. The “end-effect” pressures are assumed to act only over a defined width ΔL of structure nearest its free edge (i.e. no other unit adjacent on one side of the structure). Figure 11 illustrates the concept of end-effects for a SSU with another SSU on its left side and no adjacent unit on its right hand side. Although the actual pressure distribution is complex, a simplified step variation (dashed line in Figure 11) is specified in the Swiss Guide. For the last SSU in a row, an exterior unit the side of the SSU with no adjacent SSU (the right side in Figure 11) would experience the higher end-effect pressures while the opposite side (left side in Figure 11), that is positioned adjacent to another unit, would have the much reduced “interior” pressure (the lowest region of the dashed line in Figure 11). For SSU units that are not an end unit in a row, i.e. they are within the interior region of the row, no end-effect pressures are applied and the basic interior snow pressure is applied for design. Thus, two different SSU unit designs can be used at a given site: SSUs that are intended to be placed inside a row (called “interior units”) and more robust exterior units that will be the last SSU, on each end of the row. Figure 12 illustrates the snow design snow pressures acting on an exterior unit in isometric view.

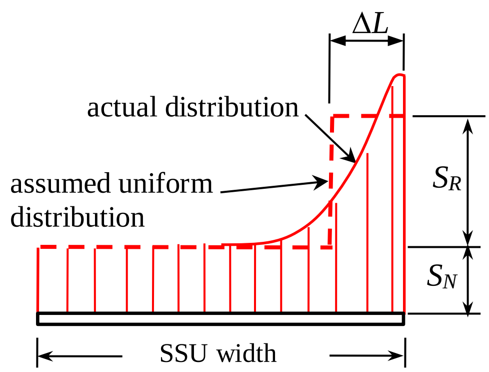


Figure 11 Variation of snow pressure (red) across the width of an isolated SSU

The Swiss Guide stipulates a slope – normal snow pressure for rigid structures but states that this can be neglected for flexible snow nets. No specific discussion of the novel SSU exists in the Swiss Guide. For typical slopes and geometry, the slope – normal pressures are approximately 10 percent to 20 percent of the slope – parallel pressures. The central tube snow pressure p_2 (Figure 7) is found by calculation of the components of the slope – parallel and slope – normal snow loading normal to the axis of the tube. Based on observed excessive bending of central tubes on SSU installed at a small project near Salt Lake City, Utah, it is advisable to increase the Swiss Guide derived central tube loads by a factor of 2.5 in order to avoid under – designing the central tube section size. While the 2.5 increase seems large, the nominal pressure p_2 without this increase is very small and the increase does not result in an unreasonable central tube size. For the Milepost 151 site, the distributed snow loading for the central tube was approximately 20 pounds per foot and this was increased to 50 pounds per foot for final design.

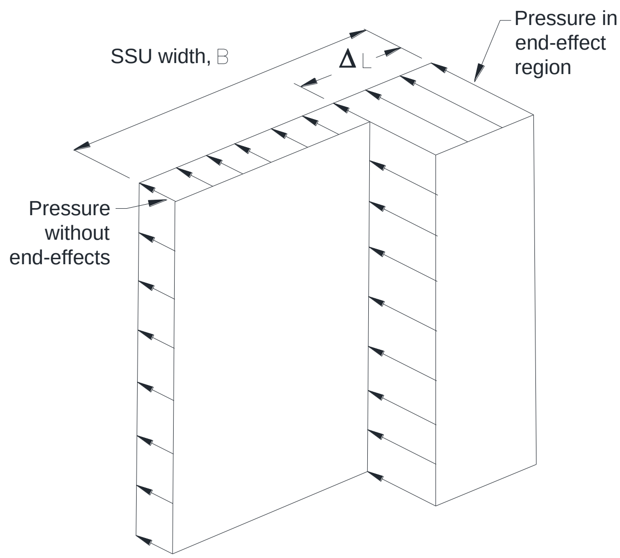


Figure 12 Snow pressure on exterior SSU

1.13 **Overview of Structural Analysis**

The response of a structure composed of or containing cables under transverse loading is highly non-linear in nature. Simplifying assumptions can be made for cable behavior under the snow pressure but they are typically overly conservative. For a single cable of known span, transverse distributed load, and specified vertical sag, tension within the cable can be calculated by simple static equilibrium. However, when a cable is loaded transversely and the elements that support the cable deflect like the SSU cross members will, a closed-form hand solution is not possible. The complexity of system response is explained as follows: an installed sag of the perimeter cable can be achieved easily, but as the SSU begins to support snow loads, the cross members will deflect under the loading applied by the perimeter cables thereby increasing cable sag, which has the effect of reducing cable tension. With the complex three-dimensional loading and SSU structure, straight-forward calculation of this effect is not practical. Moreover, the infill net spans not in one direction but in the two orthogonal directions (spans towards the sides and top and bottom of SSU), and this behavior further complicates analysis. Therefore, while an engineer could conservatively design the SSU with a simplified analysis model, a more robust three-dimensional finite-element computer model can be used to more accurately determine cable and SSU element forces. This is expected to lead to significant reductions in calculated forces and hence smaller required cables and SSU element sizes.

1.13.1 Influence of Cable Sag on Force

The critical parameter that must be established prior to the analysis of a cable is the installed sag, typically measured as a percentage of the cable clear span length (chord length between support points). As sag increases, the cable tension required to support a given force normal to the span decreases rapidly but so does stiffness. While the reduced stiffness associated with large sag can lead to occupant comfort problems in structures used by humans (bridges, walkways in a building, etc.), no such issue is present in the SSU. However, excessive slackness in a cable can lead to excessive deformations during loading and distorted geometry after loading. A common initial cable sag under self-weight value for many structural applications is 5 percent of the span. This was used as a basic starting point for the infill net and perimeter cables. Because the guy anchor cables provide support to the end of the cross members, a relatively small 1 percent initial installed sag was selected for these tension elements so that they engage the cross members quickly once loading begins and the ends of the cross members deflect transversely.

1.13.2 Infill Net Mesh Width

One of the critical design parameters for the infill net is the size of the mesh, the width and height of openings. The Swiss Guide requires a maximum opening of four inches, if no covering of the net with smaller wire netting (e.g. “chicken wire”). Where the infill cable net has a finer mesh of wire net attached to it, a maximum mesh width of eight to ten inches is recommended. Because a mesh size of four inches (no fine wire net attached to infill net) would require significantly more labor and materials to produce, a maximum mesh width of ten inches was selected for the project along with a woven steel galvanized wire finer mesh.

As the mesh width decreases, the induced cable tensile force decreases because each wire supports less area and hence snowpack. With this in mind, the infill net structural analysis conservatively assumed a ten-inch mesh size so that an upper bound to cable forces could be obtained. The final geometry of the infill mesh was a nine-inch mesh width, as shown in Figure 13.

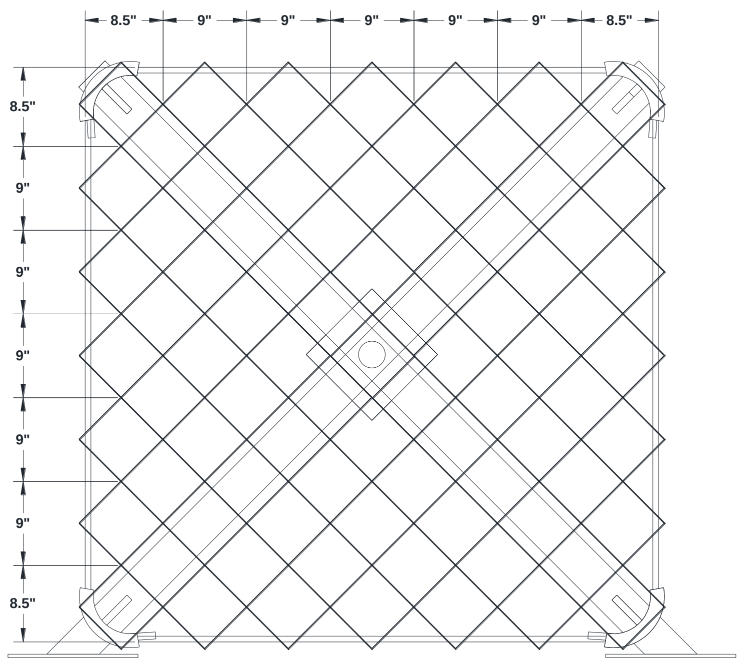


Figure 13 Infill net geometry

1.13.3 Infill Net Analysis and Design

Because it can be difficult to obtain reasonable results in very complex finite-element models, the infill net system was modelled as an independent system from the rest of the elements of the SSU. Two different models were developed, one corresponding to an exterior SSU with end-effect pressures, and one corresponding to an interior SSU, with the much lower basic snow pressure. Snow pressure resultants were determined for intersections of the infill net cables based on tributary area of the “node” and snow pressure acting in that region (Figure 14). In this way, infill net nodes within the end-effect zone receive significantly larger resultant forces as those nodes outside of the end-effect region (Figure 15). The tributary area for interior nodes was simply 10 in x 10 in = 100 in^2 while that for nodes along the perimeter was 5 in x 10 in = 50 in^2. Interior pressure away from the end-effect region was conservatively taken as 1.72 psi while the end-effect pressure used to calculate resultant nodal loading was 8.6 psi (see Appendix A). Cable sizing for the infill net is simply based on the maximum observed tension in any of the infill net elements in the computer model, and details of this maximum force and final infill net cable size are provided in Appendix B. Figure 16 provides an illustration of the deformed shape of the infill net and perimeter wire system under the effects of snow loading.

1.13.4 Perimeter Cable Analysis and Design

The infill net finite-element model, depicted in Figure 14, used pinned support conditions at each corner where the perimeter wire wraps around the end of a cross member. At even this level of refinement in analysis (i.e. a finite-element model of infill net and perimeter cable), this is a conservative modelling assumption because in reality, as the net is loaded and it in turns loads, the perimeter cable, the cross beams, will both shorten axially and deflect in the direction of snow pressure. Thus, the “supports” of the perimeter cable, which are the end of the cross members, will deform, which has the effect of reducing both infill net tension and perimeter cable tension. Nevertheless, some conservatism is appropriate and the maximum tension force in the perimeter cable element in the infill net model was used for sizing of the perimeter wire rope cable. Details of the design calculations are provided in Appendix C.

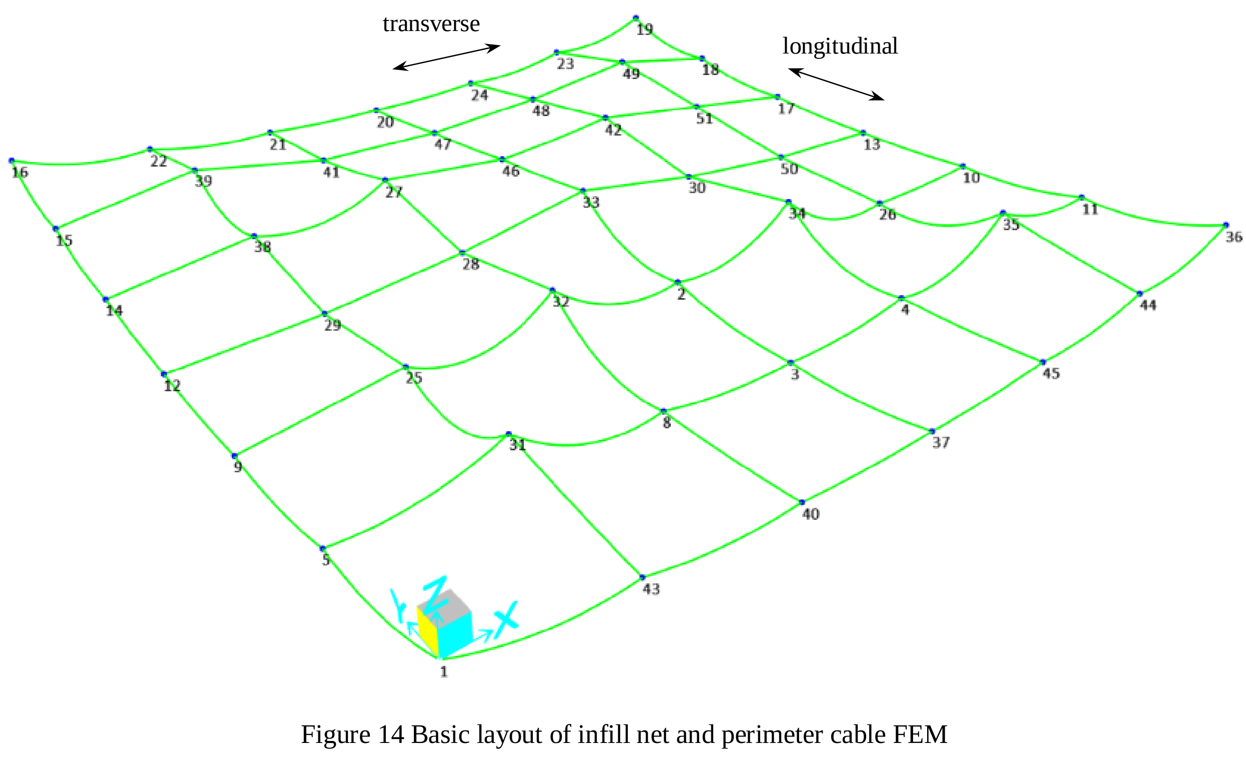


Figure 14 Basic layout of infill net and perimeter cable FEM

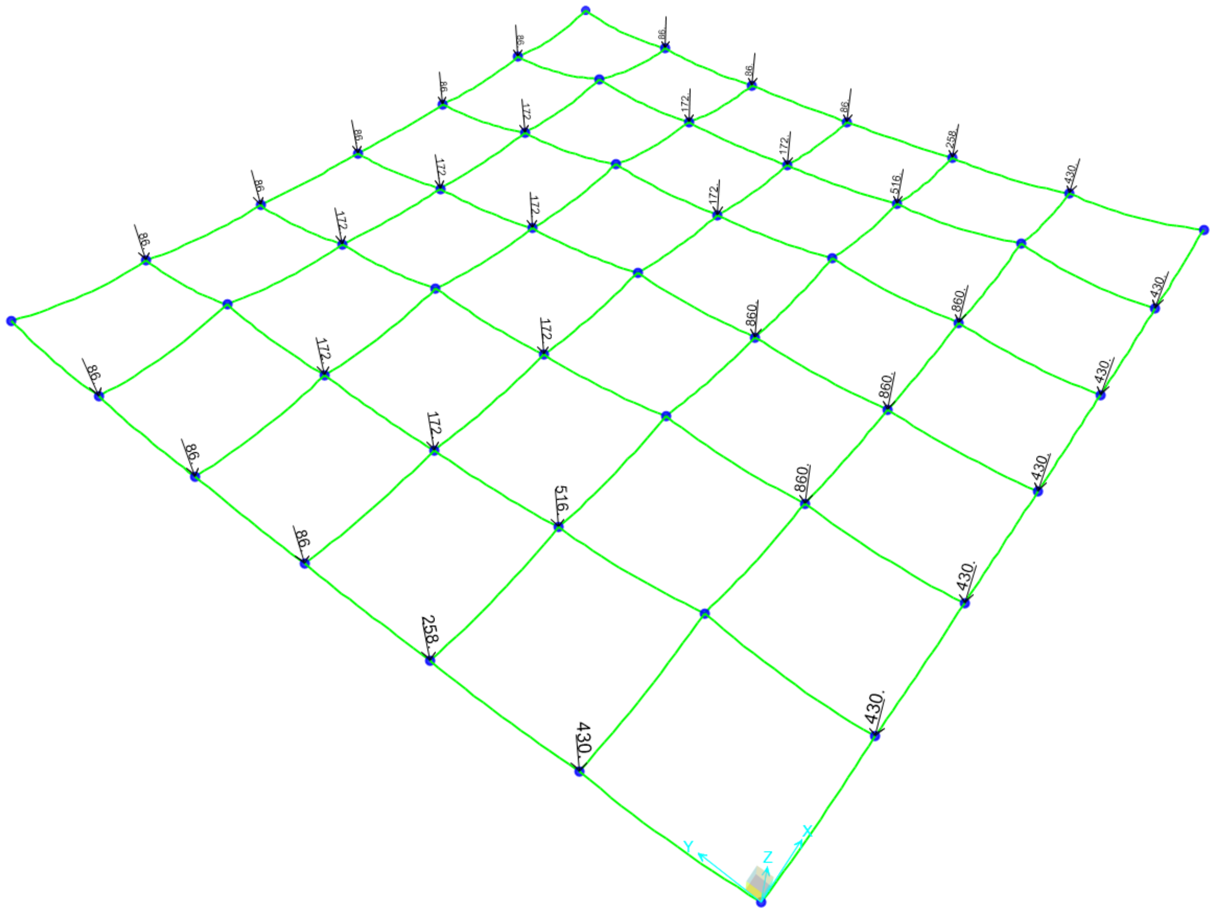


Figure 15 Infill net and perimeter wire rope loading for exterior SSU(forces in lb)

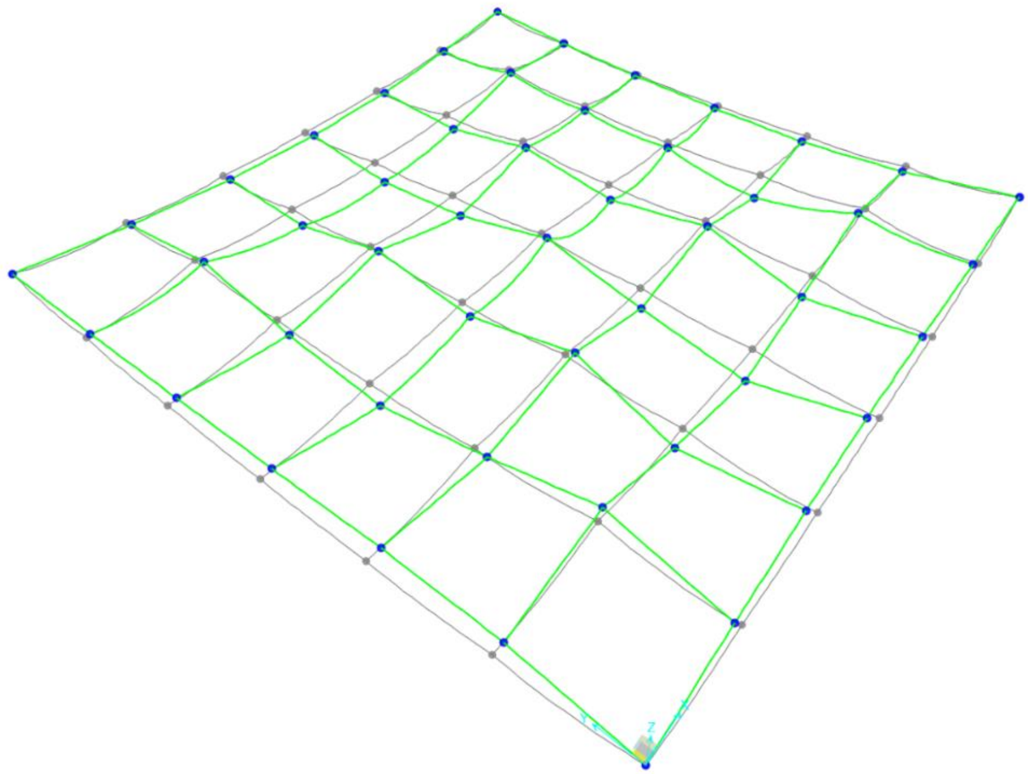


Figure 16 Deformed shape of infill net and perimeter cable model

1.13.5 SSU Analysis and Design

In order to simplify the modelling, a finite-element model of the SSU without the infill net, explicitly modeled was developed (Figure 17). Infill net cable tension forces from the infill net model were applied to the perimeter cable and cross members, as shown in Figure 18. Forces shown in the figure are with respect to a coordinate system where the x-direction is aligned parallel to and down the slope, the y-direction is across the slope (parallel to a contour line), and the z-direction is perpendicular to the slope. Figure 19 illustrates the deformed shape of the SSU unit under the effects of snow pressure. Unfactored (ASD) foundation reaction forces are shown on the model, in Figure 20, for an exterior unit, and again these forces are shown along the same coordinate system, as described above.

Design of the structural steel elements of the SSU was according to standard structural engineering practice in the United States. Specifically, the AISC 360 – 16 design specification was used to size steel cross members, the central tube, and miscellaneous steel plates and fasteners (bolts). Load and resistance factor design (LRFD), with a 1.6 snow load factor, was used. Figure 21 shows a graphical representation of the internal axial force state within the SSU with tension shown as blue (perimeter wire rope, guy anchor cables, and central tube in tension) and compression depicted by red (cross members are in compression). The analysis results of the SSU finite-element model were used to obtain the maximum guy anchor cable tension force, and design calculations are shown in Appendix D for this element of the SSU. Details of the structural steel design can be found in Appendices E and F.

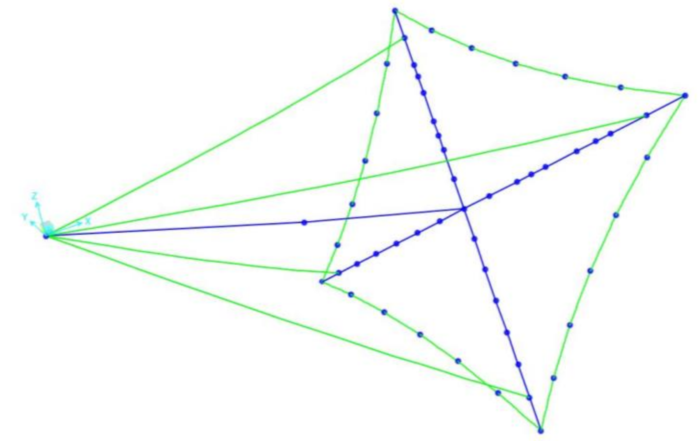


Figure 17 Finite element model of SSU

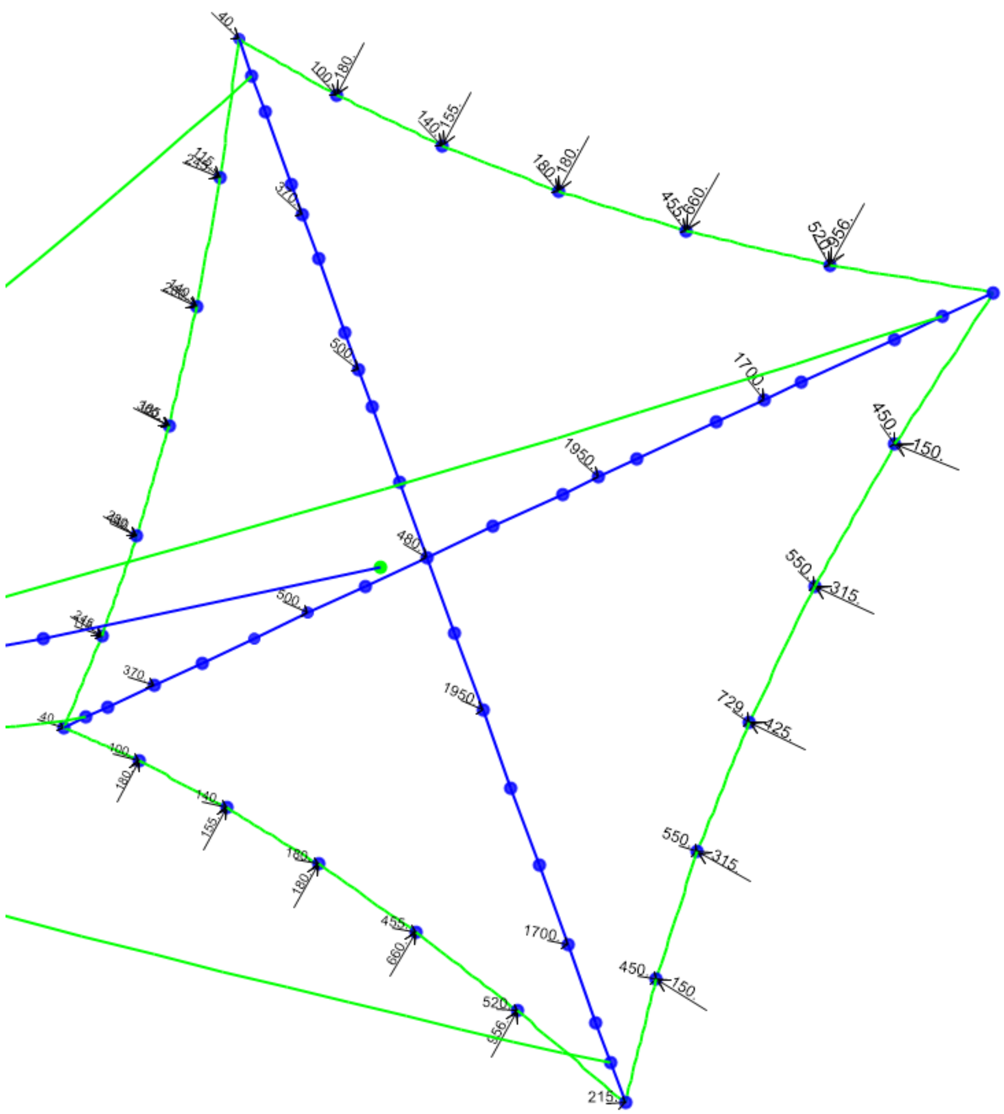


Figure 18 Nodal loads applied to exterior SSU finite-element model

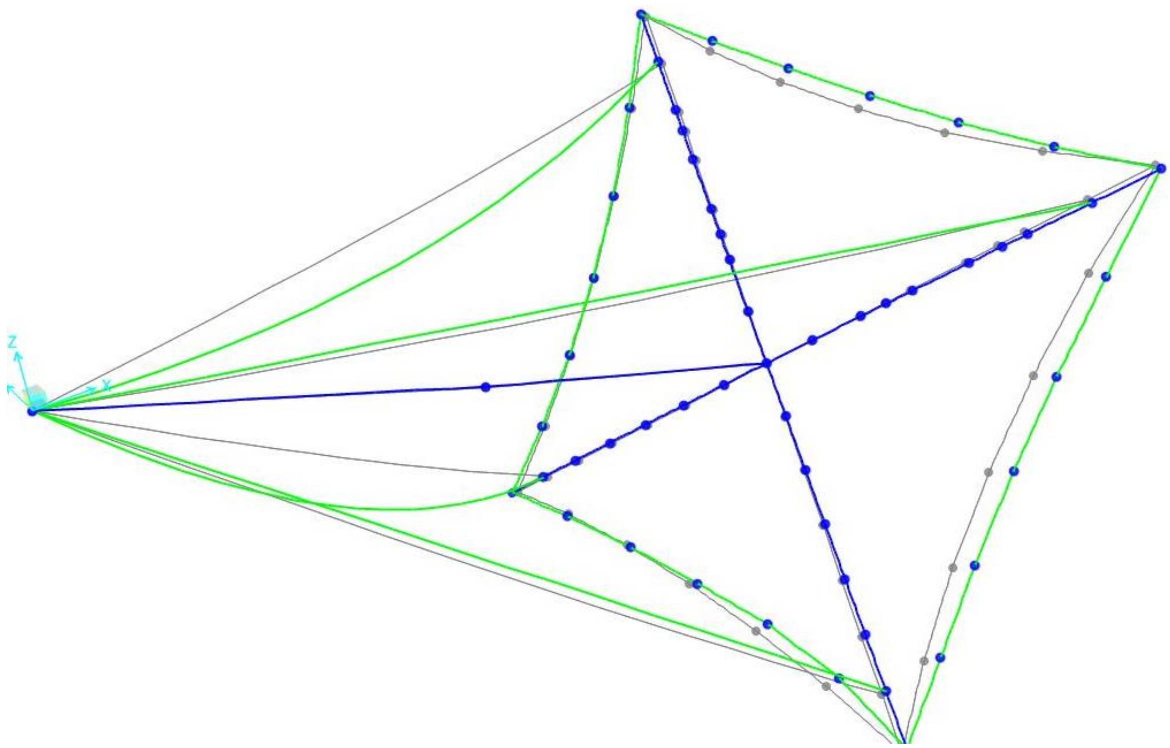


Figure 19 Deformed shape of SSU

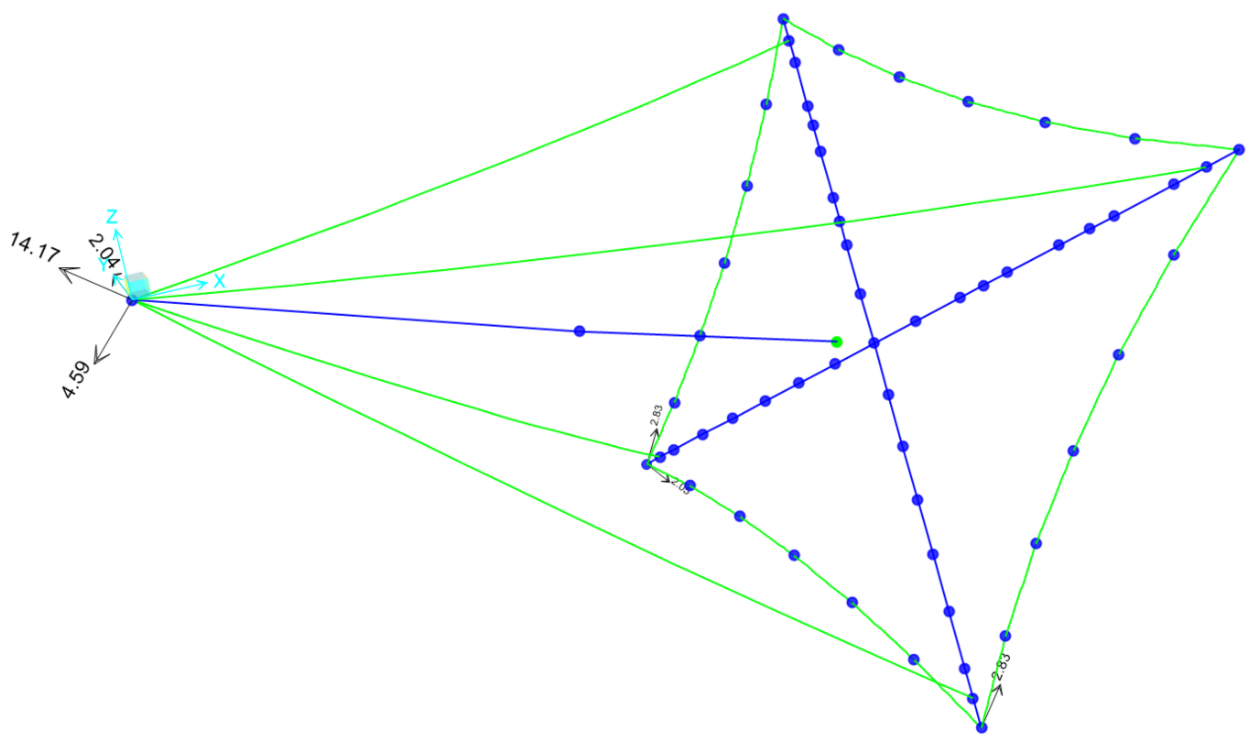


Figure 20 Unfactored foundation reaction forces for exterior SSU (forces in lb/1000)

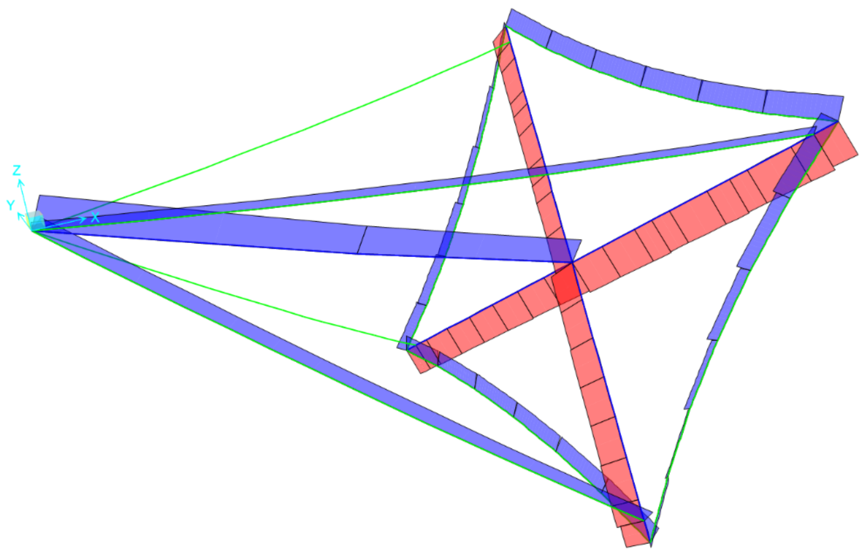


Figure 21 Internal axial force diagram for exterior SSU: red =compression, blue=tension

1.13.6 Sizing of Cables for Strength

Design of cable tension elements follows provisions of the ASCE standard "Structural Applications of Steel Cables for Buildings, ASCE/SEI 19-16". The required cable minimum tensile strength, S_r, is taken as that tension force resulting from the combination of dead load and snow loading (unfactored). The allowable strength of a cable, S_a, is given by:

$$S_a = S_{min} x N_f / \Omega$$

where

S_{Min}= minimum breaking strength of the cable

N_f= fitting reduction factor = 0.80 for wire rope clip

Ω = safety factor = 2.2

1.14 **Foundation Connection Slip Plate**

1.14.1 Background

One of the biggest concerns about the SSU as a concept is the single ground anchor used at the uphill foundation. Typically, the two other systems, flexible snow nets and rigid snow fences, utilize at least several ground anchors, and therefore, possess structural redundancy, which means that failure of one anchor does not necessarily lead to failure of the entire unit. With a single point ground anchor, the SSU does not possess any redundancy, at least with respect to its foundations, failure of the ground anchor will result in instability of the SSU unit and would constitute complete failure of the unit. For example, in Park City, Utah, failure of a ground anchor on one SSU resulted in the unit tumbling down the slope under a high snow load event. Figure 22 shows a photograph of the failed unit during the summertime and the unit can be seen to be flipped upside down with its central tube pointing downhill.

Figure 22 SSU with failed ground anchor near Park City, Utah

Concerns about lack of redundancy and overall system performance resulted in the snow umbrella concept being eliminated as an option during preliminary design of the Interstate-90 (I-90) Snoqualmie Pass avalanche project. Ultimately flexible snow nets were selected and installed on slopes above I-90. However, it is important to note that many different types of structural systems commonly used in society lack redundancy, and yet, with appropriate design and detailing provide a long trouble-free and safe service life.

1.14.2 Slip Connection Concept

The potential for structural failure in general always exists because of the variable nature of both loading and of material properties and member geometry. However, risk of failure is managed by careful selection of load and resistance factors (LRFD design), or factors-of-safety (allowable stress design). One concept for significantly reducing the probability of failure of the single ground anchor is a fuse-like connection between the SSU and the ground anchor. The connection would be designed and calibrated so that at a specified load level, it would slowly release so that the load could no longer increase. One could employ a yielding steel plate that begins elongating once a certain load is applied. However, because of the potential need to accommodate snowpack glide movements on the order of tens of inches when the mechanism releases, such a plate would need to be approximately 200 inches in length. This is not practical. Another concept was identified that consists of a slip connection where a steel cable connects the SSU to the foundation. The cable passes through a clamp that can keep the cable from slipping or pulling through up to a specified slip load and then allows the cable to pull through it once the load is reached. The slip cable would need to be long enough to accommodate the amount of down slope sliding imposed by the snowpack that is overloading the system. Glide movements per winter are on the order of 1 to 2 feet, and this extra "tail length" of the slip cable would be easy to provide without a significant cost.

The basic idea of the slip device is that when activated, the slip cable pulls through the cable clamps and the SSU unit slides down the slope quasi-statically. Snowpack glide in springtime is on the order of inches per day, where the extreme glide condition may take place once or twice during a season. The probability of occurrence of the extreme snow load condition that results in exceedance of the slip load is expected to be very low , and also to be isolated to a small number of units across a starting zone. It is expected that most units will never experience the slip load, and that over the service life of the installation, only a handful of units might experience activation of the slip device. Annual summertime inspections of a SSU facility would include a visual inspection of the slip device on each SSU unit to check for slippage and movement of the unit. When and where this is noted, a unit could easily be pulled back up the slope using a hand operated cable jack (i.e., a "come-a-long").

It is important to note that another simple option exists for guarding against ground anchor failure, and that is simply to increase the factor-of-safety used in its design. While this option is simple and would not lead to extreme increase in construction cost, it only adds a margin of safety to the foundation system. The novelty of the slip device is that it limits the total snow load that can be imparted to the entire SSU unit and foundation, thereby also providing for overload prevention of the structural members of the SSU. This is vital because while the chance of exceedance of the 100-year snow depth is relatively small, local regions within a starting zone could certainly experience snow depths greater than the expected maximum depth due to an extreme snowfall event, or snow drifting, etc.

1.14.3 Slip Connection Design

A connection utilizing a steel plate and wire rope fist grip clips was developed for the project. A large diameter wire rope slip cable connects to the foundation on one end with a large thimble eye and shackle, and on its other end it is clamped to a steel plate at the end of the SSU central tube with one-half of a fist grip clip. Figure 23 shows a photograph of a fist grip clip, and Figure 24 illustrates the slip device concept. The fist grips apply a normal clamping force N_{fg}, to the cable at each fist grip location. By friction between the slip cable and the fist grip, and cable and steel plate, a frictional force F_{fr} is developed between the slip cable and slip plate. If the value of the normal force and the coefficient of friction are known, then a force that will cause slip to occur F_{Slip} can be calculated. As long as this force is somewhat less than the calculated ultimate strength of the ground anchor, slip will occur before ground anchor failure happens.

Source: https://atlantic-group.com/product/fist-grip-clips/

Figure 23 Fist grip clip used in slip connection

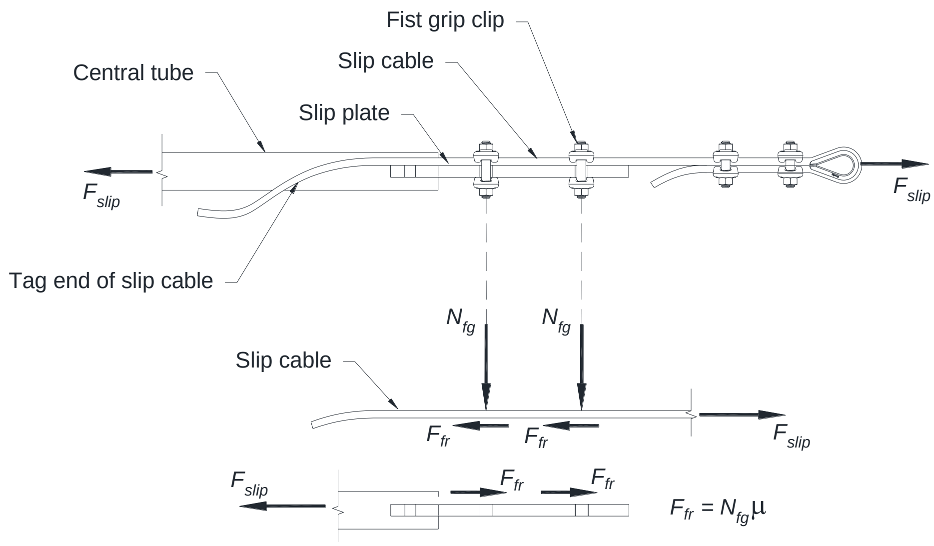


Figure 24 Concept of load-limiting slip device between SSU and ground anchor

Sizing of the slip cable itself follows the same procedures as detailed previously for other cable elements of the SSU, and detailed calculations are provided in Appendix G. The selected cable size was a ¾ inch diameter EIPS 6x19 galvanized wire rope. Selection of the slip force level and design of the connection can best be explained by an analogy of chain links in series where each link has a unique capacity. The design snow force is determined by the nominal snow pressures obtained using the geometry of the SSU and the Swiss Guide snow pressure equations. However, snow pressures used in design can be exceeded if the snowpack depth becomes greater than the design snowpack depth, which statistically is always possible. For example, the design snowpack depth at the project site is 6.5 ft and all SSS have been sized to retain snow up to this vertical depth. However, could a 500-year return period winter occur with a maximum snowpack depth of say 9 ft? If this happened, and all other factors being equal, this would lead to an increase in the resultant snow force of $9^2/6.5^2 = 1.92$ or about a 90 percent increase above the design level (snow pressure varies with snowpack height squared). It is this type of scenario that motivates the use of the slip connection, if the SSU becomes overloaded, then limit the force that can be input to the foundation to some amount less than the capacities of the various failure modes (e.g. ground anchor fracture, or grout column slip with respect to the earth, etc).

Figure 25 depicts the failure modes and their ultimate (failure) capacities for elements in series in between and including the slip device and the foundation (details on the various capacities are provided in the Appendices of this report). If failure of the foundation is to be avoided, the slip device slip force F_{Slip} must be less than the smallest of the capacities of all of the other elements in the chain. The design snow force (i.e. unfactored nominal snow load) is obtained from the resultant of the forces F_1 and F_2 previously given in Table 1. For the exterior SSU, this is approximately 15.0 kip. If a slip force of 50 percent greater than the design resultant snow load is desired, then the slip device should be designed to “let go” (i.e. slip cable begins to slip through fist grip clip) at 1.5 x 15 = 22.5 kip.

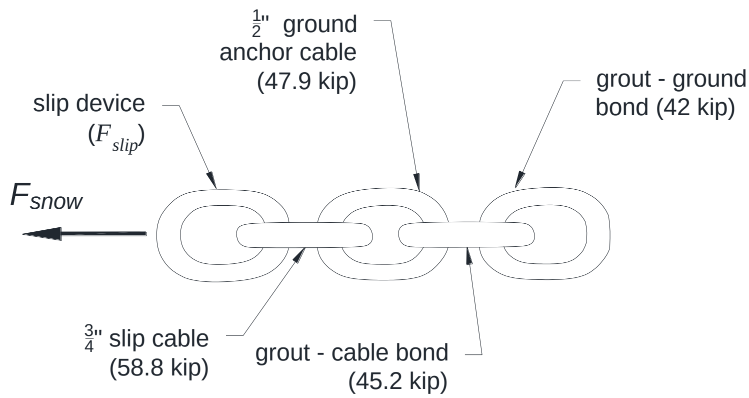


Figure 25 Series of capacities of critical elements between SSU and foundation

The two parameters that must be decided on for design of the slipping mechanism itself are the number of fist grip clips that should be used and the torque to which the bolts on the clip should be tightened. This requires some back-calculation of fist grip clamping force and implied coefficient of friction from the given manufacturer's data for number of clips required to develop the ultimate breaking strength of the given cable size. Clamping force induced by a fist grip clip N_{fg} can be back calculated from the required torque per bolt on the clip and the following equation:

$$T = \frac{KDP}{12} \text{ [ft-lb]} \tag{5}$$

where,

K= Torque Coefficient (=0.10 waxed/lubricated bolts/nuts, = 0.20 plain bolts/nuts, =0.25 for hot-dipped galvanized bolts/nuts)

D = Nominal bolt diameter [in]

P = Bolt clamping force [lb]

For a ¾ in diameter fist grip, the manufacturer's required bolt torque value is 225 ft-lb. Therefore, assuming hot-dipped galvanized bolts and solving equation (6) for the corresponding clamping force P:

$$P = \frac{12\,T}{KD} = \frac{12\ x\ 225}{0.25\ x\ 0.75} = 14{,}400\ lb \tag{6}$$

Each fist grip clip has two bolts per unit, so the total clamping force per clip is double the above amount, or 28,800 lb. To develop the full (minimum) breaking strength of a ¾ in diameter wire rope, three fist grip clips are required per the manufacturer. Wire rope clips in general transfer force by friction, and with reference to Figure 26, the actual amount of force that must be transferred by the clamping action across the two ends of the wire rope is based on transferring only one-half of the ultimate breaking strength of the cable, as determined by the free-body diagram, in Figure 26, and static equilibrium. By standard principles of physics and engineering, the friction force F_{fr} is simply the clamping force N_{fg} times coefficient of friction μ (equation 7). The back-calculated coefficient of friction μ can be found by setting the total friction force (number of fist grip clips, n, times friction force of a single clip, F_{fr}) equal to one-half the breaking strength of given cable size (equation 8).

$$F_{fr} = N_{fg}\,\mu \text{ [lb]} \tag{7}$$

$$nN_{fg}\,\mu = 0.5\,F_{UTS} \quad \longrightarrow \quad \mu = \frac{0.5F_{UTS}}{nN_{fg}} \text{ [no dimensions]} \tag{8}$$

The minimum ultimate breaking strength of ¾ in EIPS bright wire rope 6x19 or 6x37 class is F_{UTS} = 58,800 lb. Therefore, the back-calculated coefficient of friction is:

$$\mu = \frac{0.5 \; x \; 58{,}800 \; lb}{3 \; x \; 28{,}800} = 0.34$$

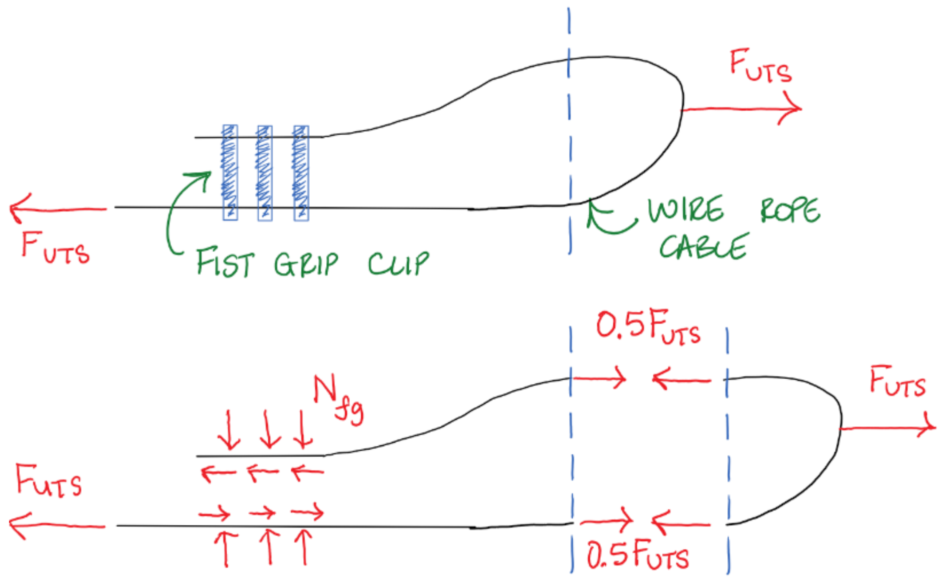


Figure 26 End termination of a wire rope and forces transferred by fist grip clips

To calculate the number of fist grip clips and their required bolt torque, the force to be transferred (F_{Slip}) is substituted for F_{UTS} in equation (8) and the total clamping force $n \; x \; N_{fg}$ can be solved for:

$$nN_{fg} = \frac{0.5 \; F_{Slip}}{\mu} = \frac{0.5 \; x \; 22.5 \; kip}{0.34} = 33.1 \; kip \quad \text{(total clamping needed)}$$

If only two fist grip clips are used in the slip device, then each must apply a clamping force of one-half of the above total clamping force or 16.6 kip. Since each fist grip uses two bolts, each bolt must provide one-half of this value or 8.3 kip. The required torque for each bolt of the fist grip is found by equation (6) as:

$$T = \frac{0.25 \; x \; 0.75 \; x \; 8{,}300}{12} = 130 \; ft - lb$$

The above describes the basic design process for the slip connection, but it is important to make the following notes. For a large-scale application of the SSU concept, the actual slip capacity of a given slip device should be verified by testing of several test specimens to verify the needed to torque to achieve the desired slip force in the connection. This sort of testing could be done a relatively simple manner and compared to the overall cost of a project, would be negligible. It is envisioned that for the specific slip cable size used for the project, a bolt torque versus slip force graph could be constructed so that interpolation for the desired slip force could be made.

1.15 Detailed SSU Structural Drawings

Detailed engineering plans were developed based on the above described design process and calculations. The drawings presented in this section are comprehensive and can be used for soliciting bids from contractors for production of SSU. A general description of all of the elements of the SSU, and how modifications might be made when the system is applied to a large-scale project, is provided below.

All structural steel on the SSU was specified as weathering steel in order to eliminate the need for future maintenance (if painted), and also because the natural weathered color would blend in with the surrounding landscape better than a galvanized steel finish. The main structural steel elements of the SSU consist of two HSS square tube members that form an “X” and are joined where they meet with a rigid connection (Figure 27). These two elements are connected to a circular steel pipe that is called the central tube and that links the cross members with the uphill ground anchor foundation (Figure 28 and 29). Ideally, the connection between the cross members and the central tube should be a “pinned” connection, a connection that does not transmit any bending moment from the cross beams to the central tube. This is accomplished by the use of an oversized section of pipe into which the central tube inserts where it meets the center of the cross members. A single horizontal bolt through the oversized pipe and central tube allows for rotation of the end of the central tube about a horizontal axis, and the bolt transmits snow load from the cross members to the central tube (i.e. the tube can rotate in a vertical plane, pivoting around the bolt at its ends) (Figure 35).

Wrapped around the corners of the x-shaped cross member system is a perimeter cable that engages with curved angle cable deviators (Figure 40 and 41), and the perimeter cable is spliced using wire rope clips along one side of the SSU (Figure 38 and 39). It is noted that a better solution in terms of reduced cost for a large-scale application of the SSU system is swaging of the splice connection. This process requires specialized equipment for large diameter wire rope but when producing the perimeter cable in large quantities, economy of scale should make this the preferred perimeter cable splice connection. The perimeter cable is anchored to the top end of each cross member using a smooth steel dowel that is welded to the deviator and using a wire rope clip. This connection helps to prevent the “X” formed by the cross members from collapsing due to vertical snow loads (closing like a pair of scissors).

The infill net is anchored continuously along all sides of the SSU by wrapping the infill net cable around the perimeter cable. For this specific project, no existing (i.e. off-the-shelf) wire rope nets of the appropriate size were available, and therefore, the nets were manufactured by the project team. However, for a large-scale project it is envisioned that infill nets could be sized and ordered from a manufacturer at a much-reduced cost compared to this project. The infill net is draped over the cross members on the uphill side of the SSU.

Guy anchor cables span from each end of a cross member to the uphill foundation connection (slip) plate, and these provide support to the cross members thereby reducing moment demands (Figures 28, 29, 36, and 37). The guy anchor cables are currently detailed using thimble eye terminations and connected to the SSU steel with shackles. The anchor cable ends are secured with “turnbacks” and wire rope clips. For a large-scale application of the system, the guy anchor cables should be detailed with swage fittings to clamp the dead end of the anchor cables. This will result in cost savings as compared to wire rope clips.

The slip connection plate is a ¾ in thick structural steel plate welded to the uphill end of the central tube (Figure 34). Holes are drilled through the plate for shackles to connect to the guy anchor cables and additional holes for the fist grip clips are provided (Figure 42).

1.15.1 General Notes

LOADS

- All snow loads determined by the “Defense Structures in Avalanche Starting Zones, Technical Guidelines as an Aid to Enforcement” document published by the WSL Swiss Federal Institute for Snow and Avalanche Research, Davos, Switzerland.
- Glide factor, N= 3.2
- Slope inclination angle, $\psi \leq$ 40 degrees
- Elevation of starting zone ≤ 8,500 ft.
- Slope solar aspect of west – southwest.

STRUCTURAL STEEL

- All design and construction of structural steel shall conform to the requirements of the American Institute for Steel Construction Specification for Structural Steel Buildings 2016 Edition (AISC 360-16) and the Wyoming Department of Transportation Standard Specifications for Road and Bridge Construction.
- All construction and tolerances shall conform to the AISC Code of Standard practice for Steel Buildings and Bridges.
- All steel shall be corrosion resistant weathering steel and meet or exceed the following specifications:
  a. Angle shapes: ASTM A588 (F_y= 50 ksi)
  b. Square or rectangular tube (HSS): ASTM A847 (F_y= 50 ksi)
  c. Pipe (P): ASTM A847 (F_y= 50 ksi)
- All bolted connections shall conform to the Research Council on Structural Connections Specifications for Structural Joints Using High Strength Bolts current edition
- All bolted connections shall use hot-dipped galvanized ASTM A325 bolts and be pretensioned unless noted otherwise. All ASTM A307 bolts shall be installed snug tight.
- All welded connections shall be welded in accordance with the American Welding Society’s Structural Welding Code for Structural Steel (AWS D1.1).
- All welding shall be performed with E70XX low hydrogen electrodes compatible with weathering steel unless noted otherwise.
- No holes, notches, or other penetrations through structural steel shall be permitted without prior approval of the engineer-of-record.

STRUCTURAL CABLES

- All wire rope shall be United States origin, galvanized, and be constructed with an independent wire rope core (IWRC).
- All wire rope shall be constructed in accordance with U.S. Federal Specification RR-W-410.
- All wire rope hardware shall be installed according to manufacturer's recommendations.
- All fittings and hardware for steel cables shall be galvanized.
- Perimeter steel cables shall be manufactured to achieve a 5 percent sag of the span between adjacent deviators when cross members and cable are in their final position. The span is taken as the distance of a chord tangent to the radiused deviator cable bearing surface.
- Guy anchor cables shall be manufactured to achieve a 1 percent sag of their span when the cross members and guy anchor cables are in their final installed position. Their span is the distance between centerline of holes in structural steel to which the guy cables attach at each end.
- Fist grip bolts and nuts at the slip plate shall be tightened to a torque of 130 ± 5 ft-lb.
- Swage fittings appropriate for the cable size may be substituted for all wire rope clips shown on the plans. Swage fittings shall be installed according to manufacturer's recommendations. Indicated turn back distances may be changed for swaged fitting use according to manufacturer's recommendations.

GROUT MATERIALS AND GROUND ANCHORS

- Grout materials shall conform to the WYDOT Standard Specifications for Road and bridge Construction – 2010 Edition.
- Neat grout shall use Type I or Type II cement. Neat cement grout shall be mixed at a rate of not more than six gallons of water per (94-lb) sack of cement and have a minimum 28-day compressive strength of 4,000 psi. The grout shall be continuously agitated and delivered to the hole free of lumps and unmixed cement. The delivery pressure shall be limited to avoid soil heave.
- Centralizing spacers that center the ground anchor cable in the bore hole shall be used at a spacing along the ground anchor of not more than three feet, with the first and last centralizer within six inches of either end of the bore hole.
- Each ground anchor shall be grouted in one continuous operation and at ambient temperature of not less than 40 degrees Fahrenheit and not more than 100 degrees Fahrenheit.

1.15.2 SSU Elevation Views

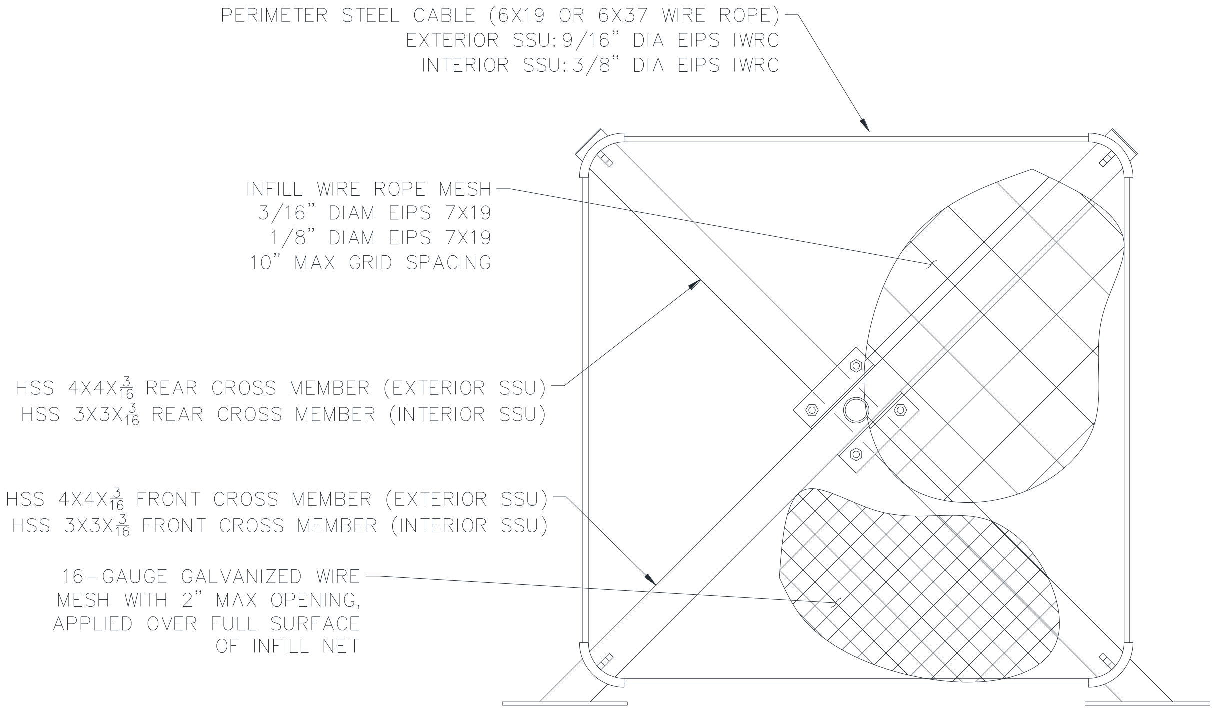


Figure 27 Front elevation view of SSU

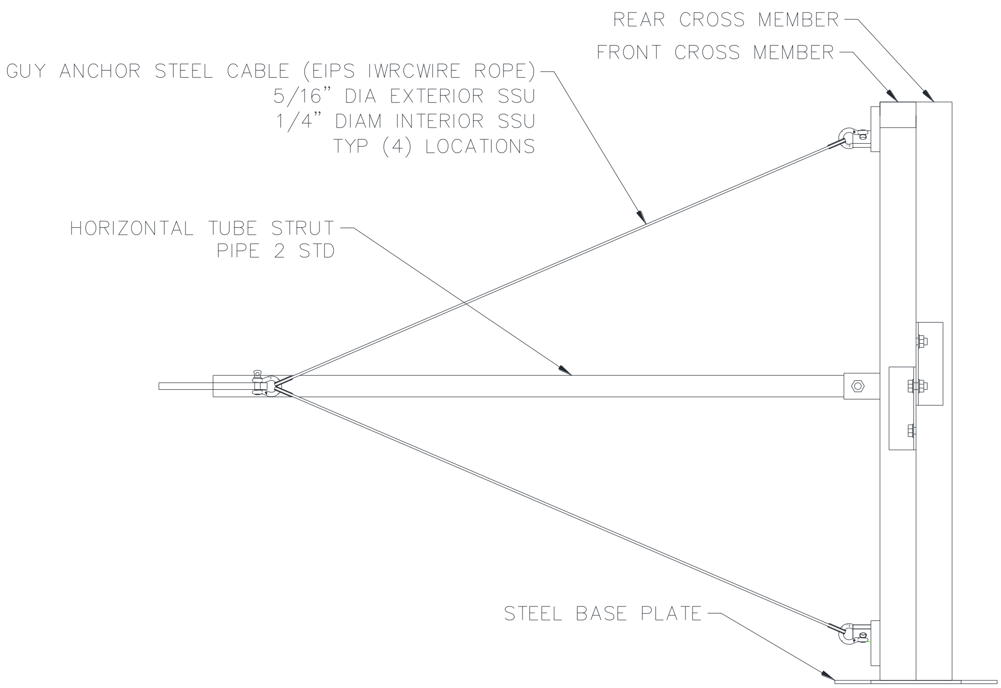


Figure 28 Side elevation view of SSU

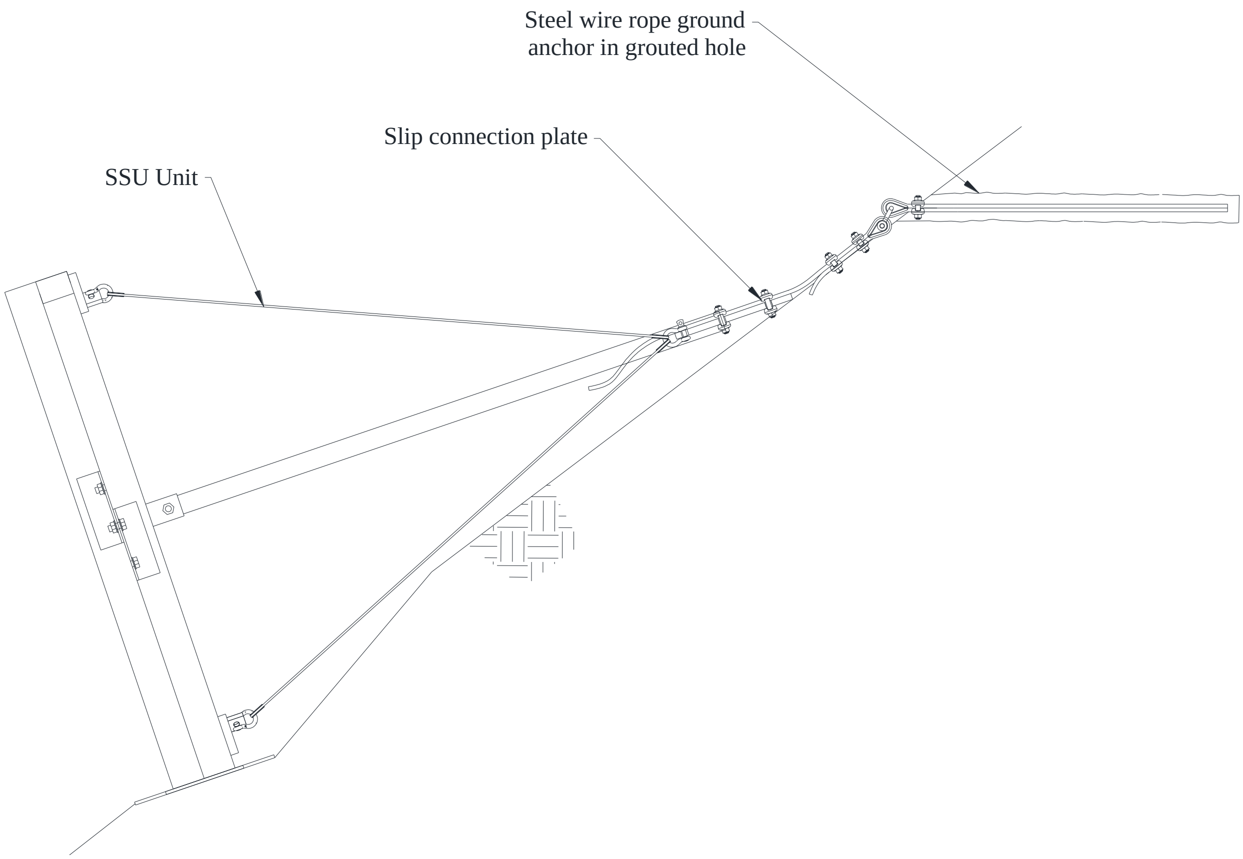


Figure 29 Side elevation view of the SSU unit and foundation installed on slope

1.15.3 *Cross Member Details*

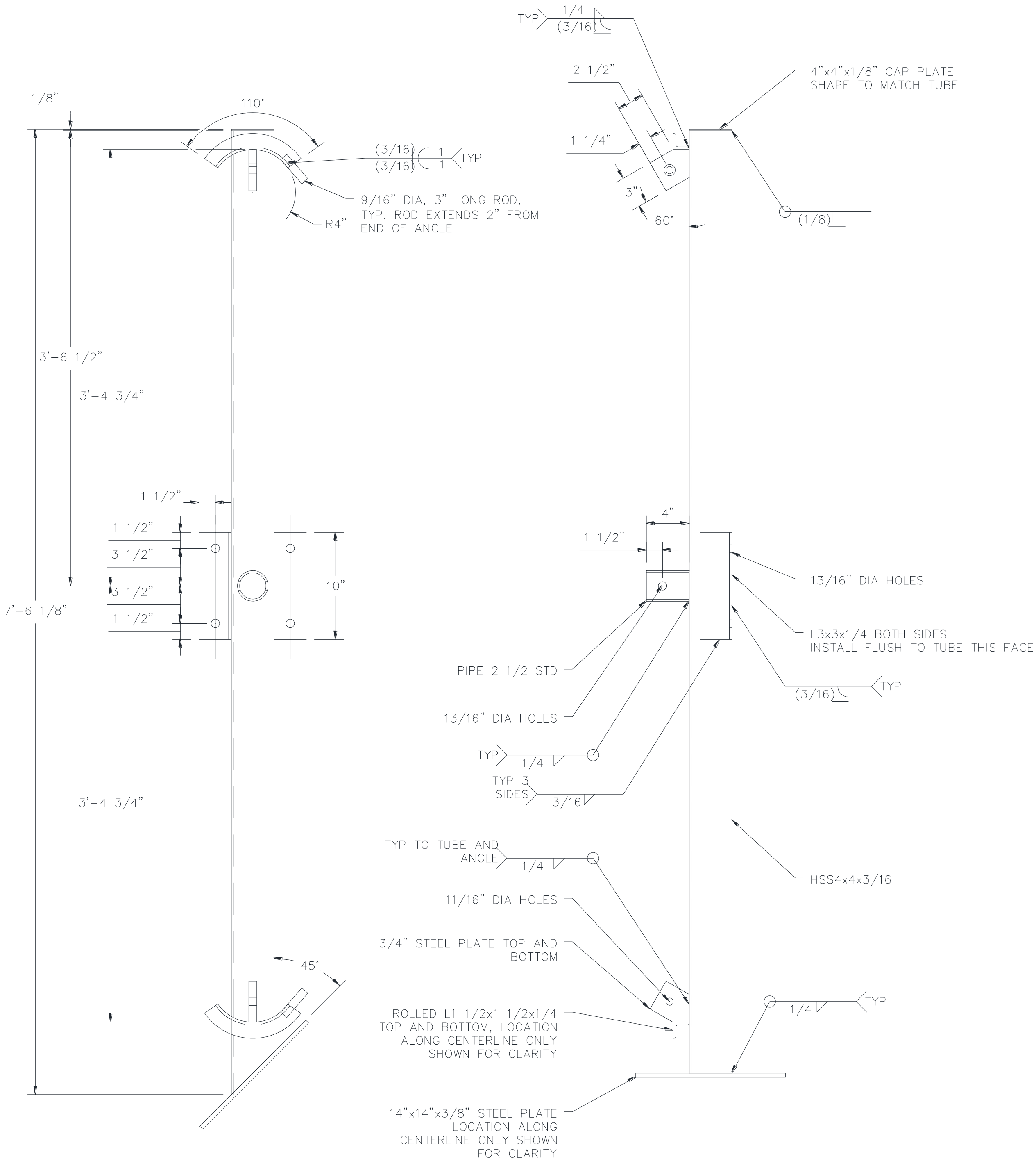


Figure 30 Front cross member detailing for exterior SSU

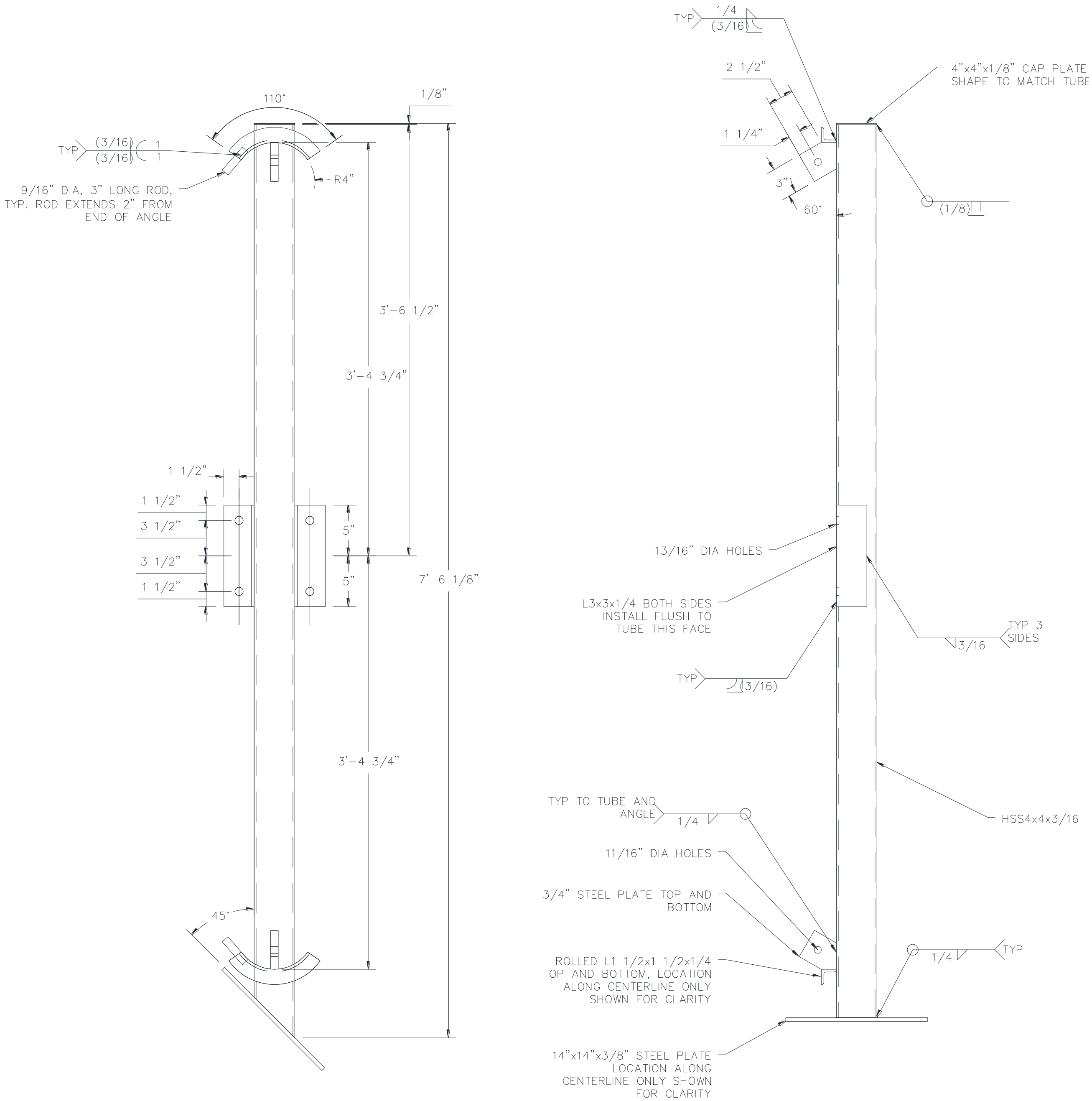


Figure 31 Rear cross member detailing for exterior SSU

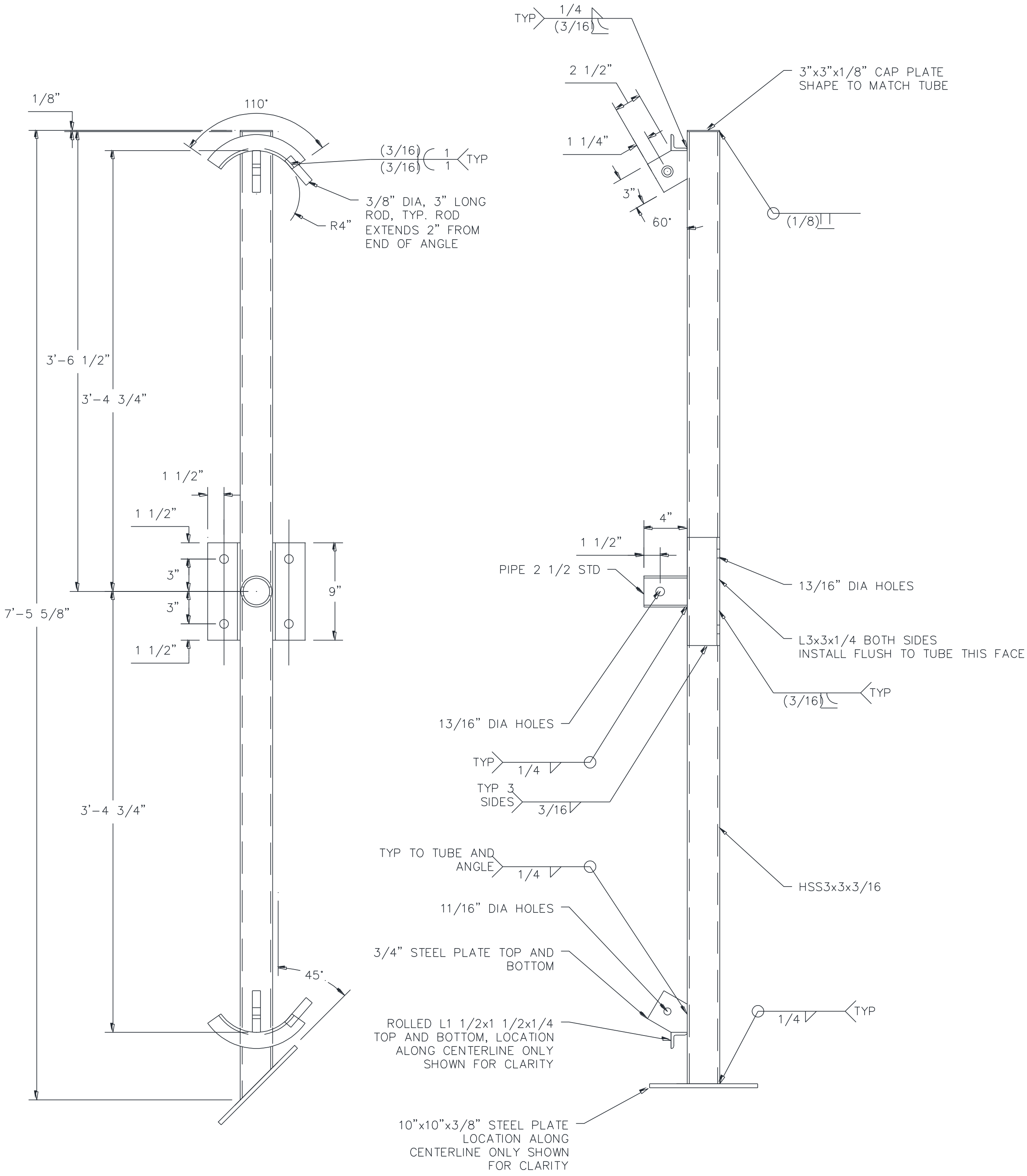


Figure 32 Front cross member detailing for interior SSU

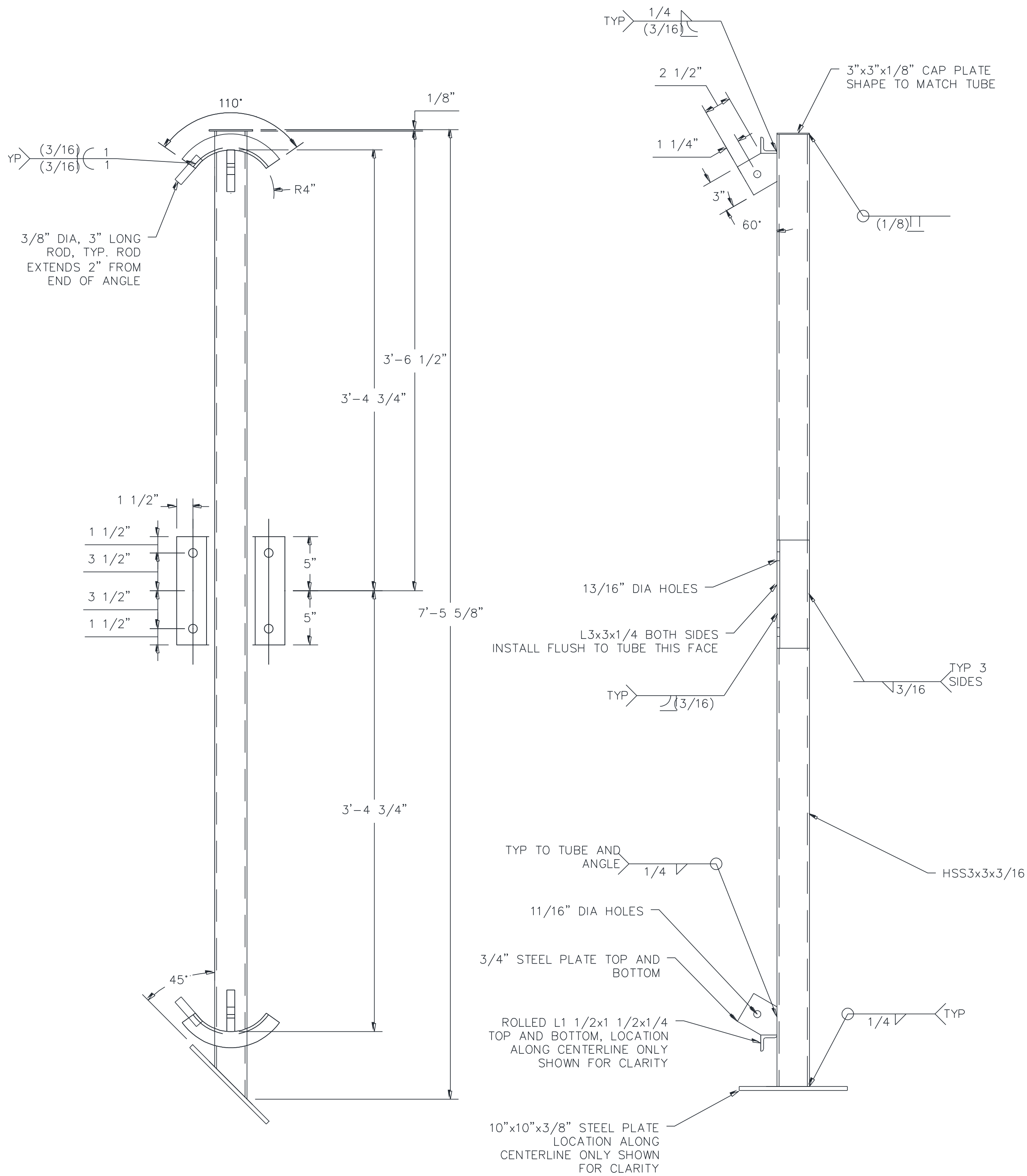


Figure 33 Rear cross member detailing for interior SSU

1.15.4 Central Tube and Slip Plate Details

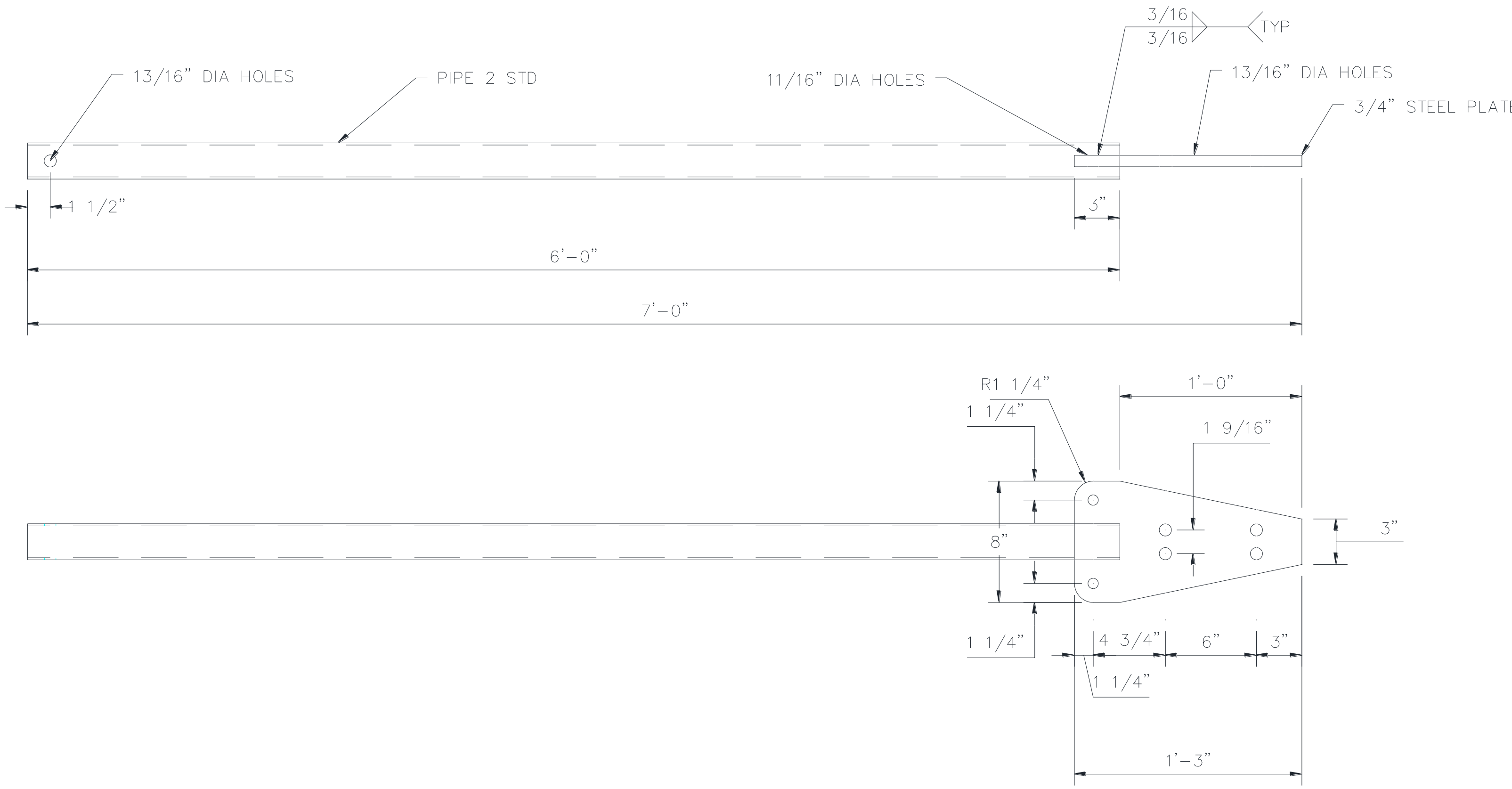


Figure 34 Central tube detailing for SSU

1.15.5 Miscellaneous Details

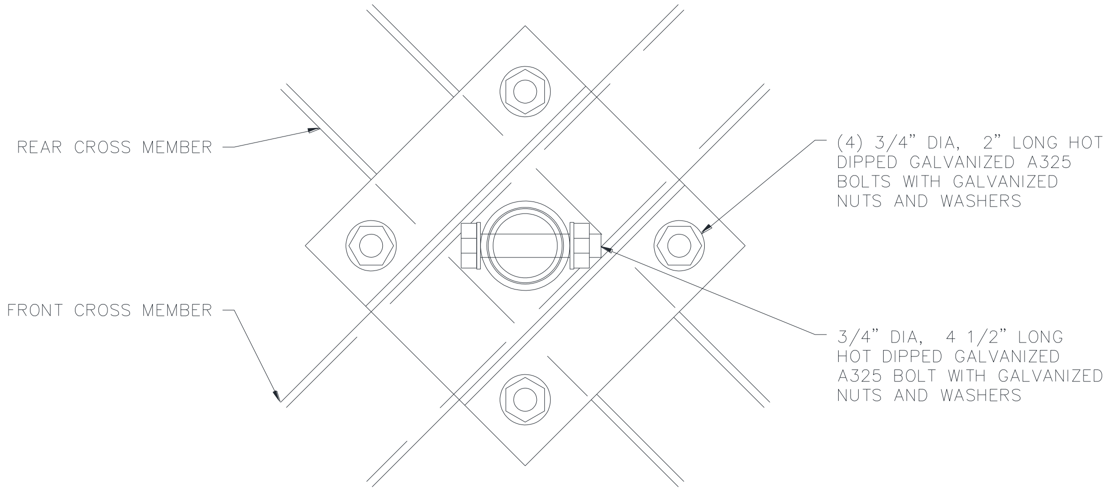


Figure 35 Center connection details for SSU

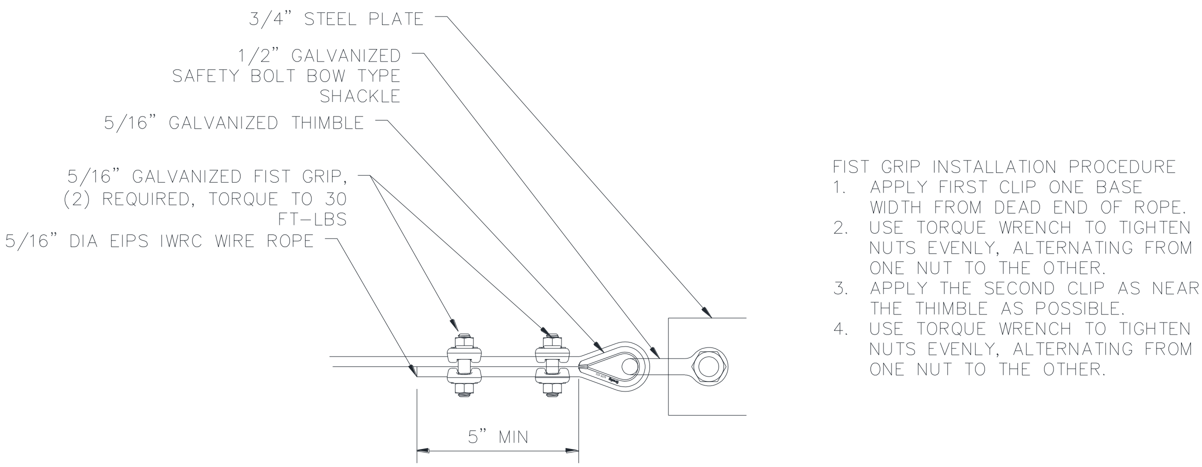


Figure 36 Guy wire rope end connection detail for exterior SSU

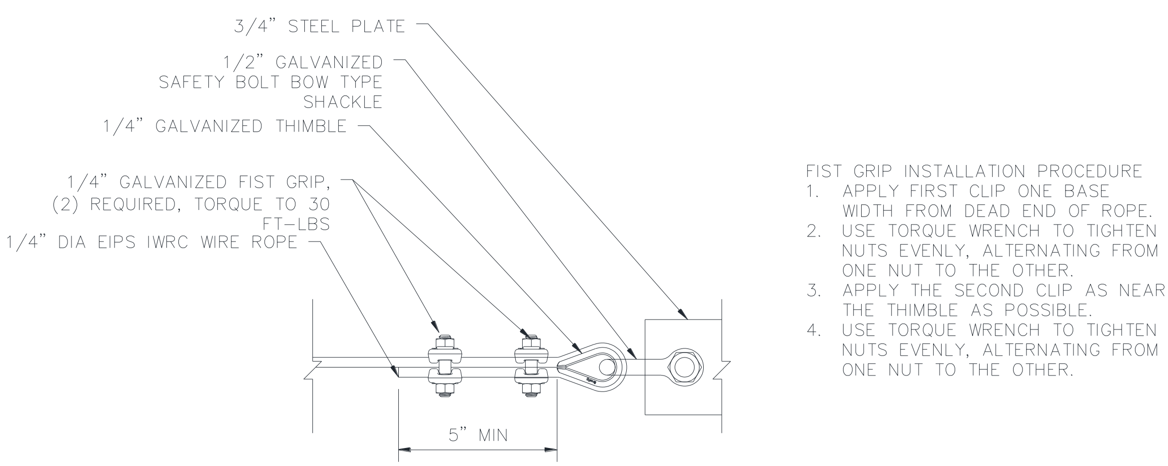


Figure 37 Guy wire rope end connection detail for interior SSU

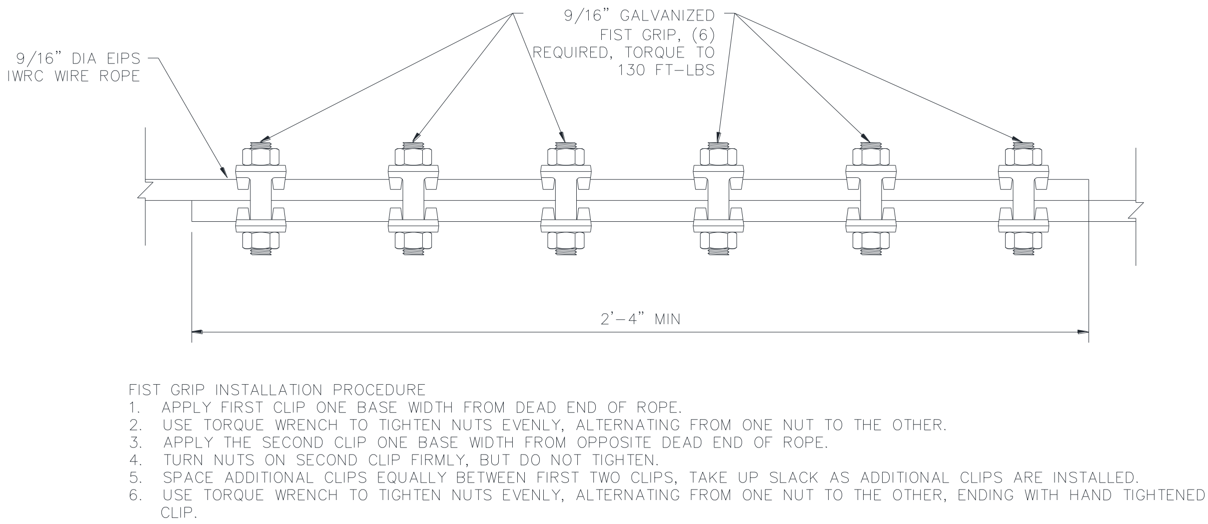


Figure 38 Perimeter wire rope splice detail for exterior SSU

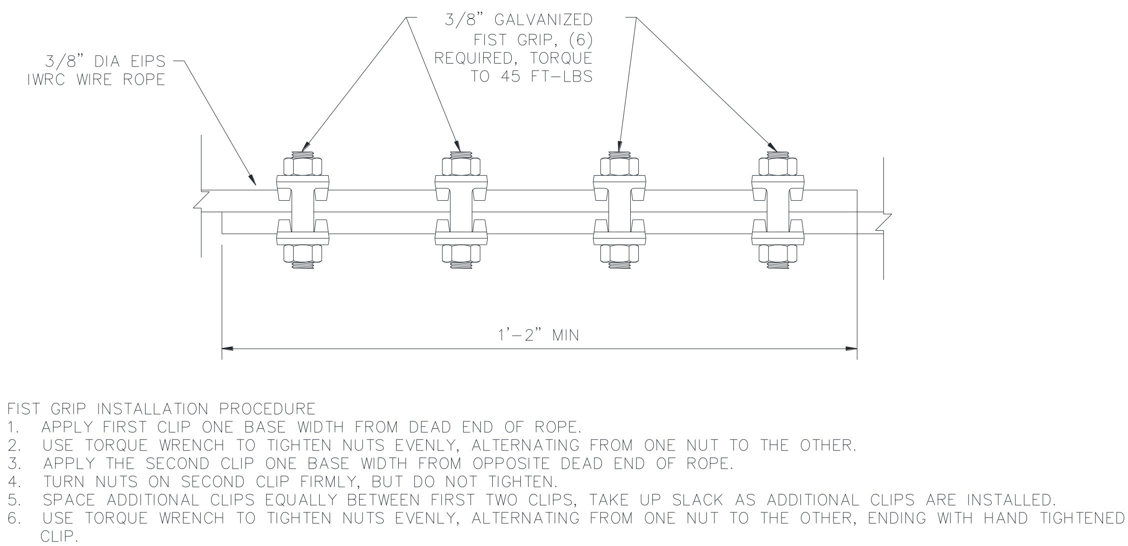


Figure 39 Perimeter wire rope splice detail for interior SSU

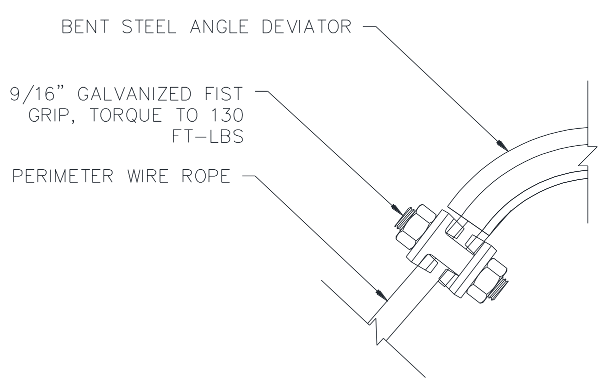


Figure 40 Perimeter wire rope deviator at cross member for exterior SSU

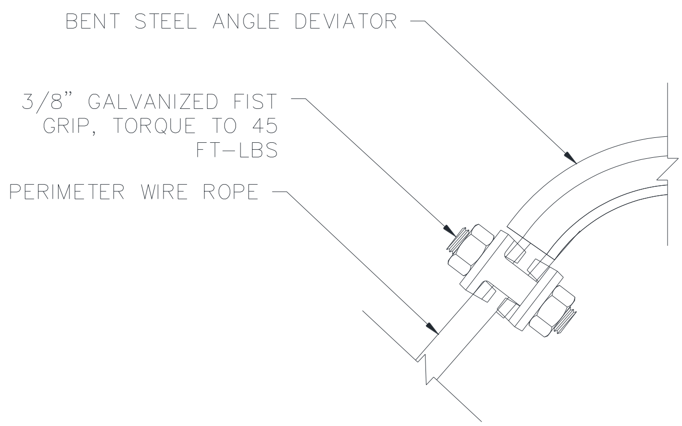


Figure 41 Perimeter wire rope deviator at cross member for interior SSU

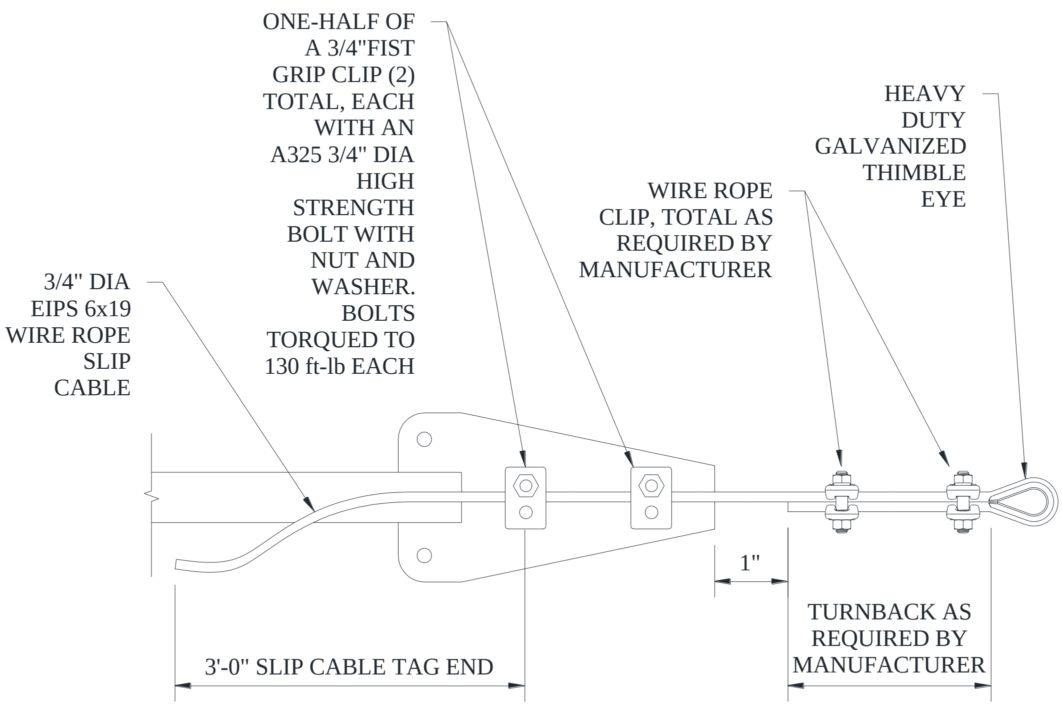


Figure 42 Slip connection details

FOUNDATION DESIGN

1.16 **Lower Foundation Design**

The bottom of each cross member is welded to a steel base plate that bears directly on the ground surface. Friction is neglected between the steel plate and ground, and therefore, the foundation reaction will be perpendicular to the base plate and is always compressive in nature. Forces at the lower foundation were calculated as described previously and also using the finite-element model of the SSU. Width sizing of the square base plates is straight-forward and is based on limiting soil compression stress to some specified allowable bearing stress. Because the SSU is completely insensitive to settlements of the lower foundation, a relatively high basic allowable bearing stress of 2500 psf was assumed. Full detailed calculations of base plate sizing are provided in Appendices H and I.

1.17 **Upper Foundation Design**

The basic orientation of the upper foundation ground anchor with respect to the slope was discussed in detail previously in this report. Additionally, the need to consider the angle formed between the resultant foundation load F and the axis of the ground anchor was presented. The critical aspect for design of the upper foundation is the extent to which it is loaded laterally because small diameter rigid ground anchors do not tolerate significant bending effects. Upper foundation reaction forces were obtained from the finite-element models for both the exterior and interior SSU units. Loads applied to the ground anchor are equal and opposite to reactions, and Table 1 presents unfactored (e.g. allowable stress design, "ASD") forces. With reference to Figure 43, it can be seen that forces $F_1$and F_2 both induce axial tension in the ground anchor. Using the geometry of Figure 43 and by equations 9 and 10 below, horizontal and vertical components of these forces (T and N_U respectively) were calculated and these are also provided in Table 1. Force T represents the design axial tension force that must be considered while force N_U represents the design lateral loading.

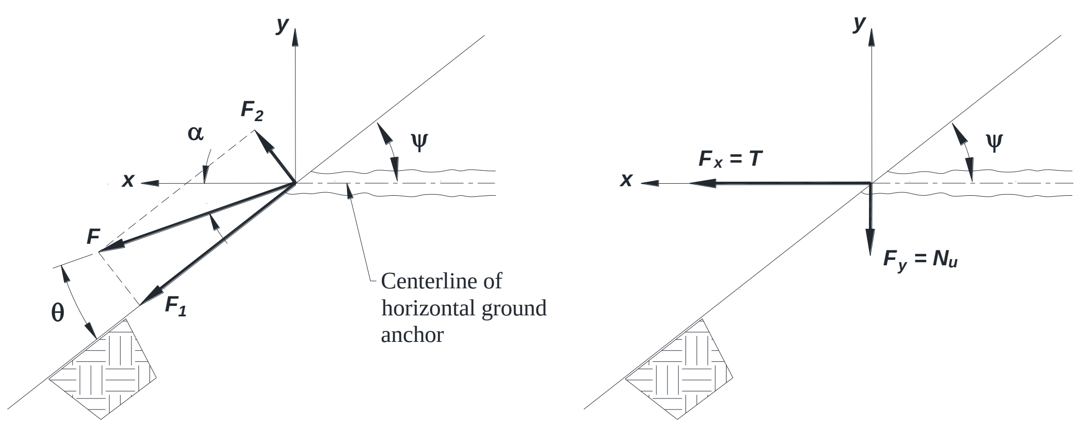


Figure 43 Upper foundation reactions along slope (LHS) and in horizontal and vertical (RHS)

From Table 1, the lateral loading that must be resisted for the exterior SSU is 5.6 kip, while the same for the interior units is approximately 3 kips. These are very significant forces that would be difficult to resist by the ground anchor alone. During preliminary design, sizing of hollow-core micropile (HCM) bars was performed and two inch diameter high-strength steel HCM bars with five inch diameter grout annulus were required. This was deemed extremely unacceptable based on cost construction considerations.

$$T = F_1 \cos\psi + F_2 \sin\psi \tag{9}$$

$$N_U = -F_1 \sin\psi + F_2 \cos\psi \tag{10}$$

Table 1 Upper foundation unfactored forces

SSU	F_1 (kip)	F_2 (kip)	θ (Deg)	T (kip)	N_U (kip)
Exterior	14.2	4.6	18.0	13.8	-5.60
Interior	7.1	2.1	16.5	6.80	-2.96

Alternative types of ground anchors were thus considered and flexible steel cable anchors were selected due to their ability to generally tolerate small off-axis tension loads. With reference to Figure 43, the angle between the resultant foundation load F and the ground slope is θ and the angle change between the axis of the ground anchor and the resultant force that must be accommodated is slope angle ψ minus angle θ. Because values of θ are relatively small (see Table 1), a very abrupt angle change on the order of 23 degrees would occur where the flexible steel cable exits the grout column. Due to concerns about over straining the steel cable material at this location, it was decided to utilize a small precast concrete footing that could be lifted by two persons to resolve the downward force N_U. Footing geometry was selected based on the desire to achieve a uniform bearing pressure underneath the footing (i.e. ground anchor tension force T, resultant foundation load F, and footing reaction force N_U are concurrent). Because concrete is weak in tension and due to the relatively large tensile forces, a steel plate embedded within the footing was used to transmit the resultant snow load to the ground anchor (Figure 44). Details of the footing design are provided in Appendix J.

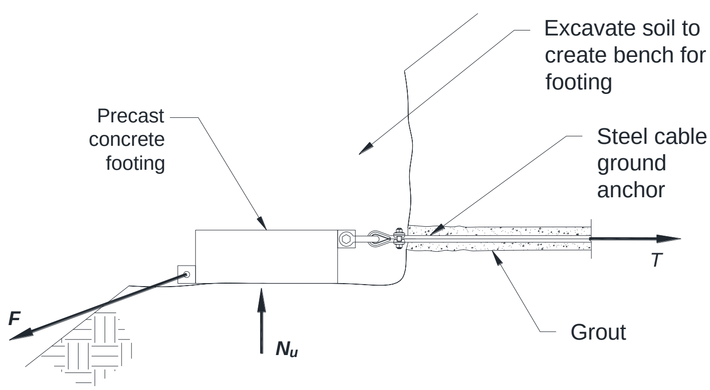
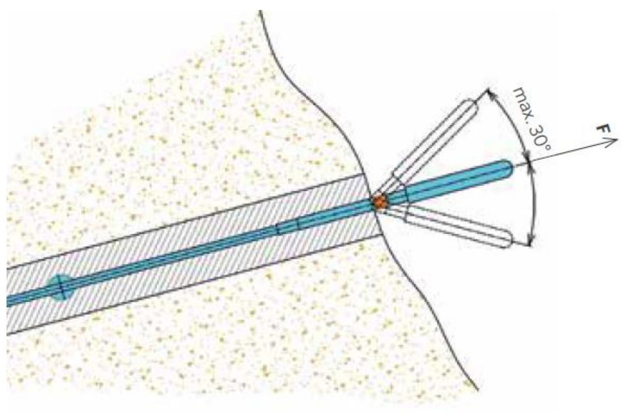


Figure 44 Precast concrete footing to accommodate lateral loading on upper foundation

Additional research on other concepts for the upper foundation was performed after installation of the SSU at the project site was completed. A new type of proprietary flexible ground anchor termination has recently been developed by GEOBRUGG AG (www.geobrugg.com), a Switzerland-based company specializing in rock fall and avalanche hazard mitigation. This now available ground anchor system, which includes both a hardened end loop termination and the steel wire rope cable can accommodate up to a 30 degree off-axis loading of the ground anchor without any reduction in its strength. Figure 45 shows a photograph of the end region of the spiral rope anchor, and Figure 46 illustrates the off-axis loading limits of the system. With reference to Figure 44, the angle between the resultant snow load F, and the ground anchor axis will be $\alpha = \psi - \theta$. For starting zone slope angles between 30 and 40 degrees (the most common range of slopes with avalanche issues), and with angle θ relatively constant irrespective of slope angle, the off-axis angle will typically be approximately 15 to 25 degrees.

Source: www.geobrugg.com

Figure 45 Geobrugg spiral rope ground anchor



Source: www.geobrugg.com

Figure 46 Off-axis loading capability of the Geobrugg flexible ground anchor system

Comparing the two upper foundation options, one with a concrete footing and the other with the Geobrugg (or similar) system,– it is readily evident that the Geobrugg option will require less overall labor to install, and thus, will likely be significantly less expensive compared to the concrete footing option. Elimination of costs for production of footings, transport to the site (both on the roadway and up to the construction zone), and excavation of the soil bench will certainly reduce overall cost of a SSU facility even with a moderately priced spiral rope anchor. It is important to note that other manufacturers have also developed this system, including French company Avaroc (www.avaroc.com), and therefore, multiple options exist for competitive bid purposes. However, the technology associated with this flexible wire rope end terminations is simple, and it is believed that a engineered design could be developed relatively easily, and manufactured domestically so that import fees and delays related to FHWA's Buy America policy could be avoided.

1.18 **Ground Anchor Design**

Design of the ground anchor followed established engineering principles for ground anchor systems. The Federal Highway Administration (FHWA) has published an advisory circular that provides a succinct summary of the issues related to ground anchor design (FHWA, 1999). Four primary considerations in the design of ground anchors must be addressed: capacity of the steel cable itself, grout-to-ground bond capacity, anchor cable-to-grout bond capacity, and selection of grout material. These are addressed in-turn subsequently, and it is noted that the detailed design information provided herein applies only to this specific project site as subsurface ground conditions can vary widely based on geographic location. A engineered site-specific geotechnical design will be required for other locations where WYDOT may choose to apply the SSU system to mitigate avalanche danger.

1.18.1 Anchor Cable Sizing

Selection of the diameter for the ground anchor cable is based on minimum breaking strength values provided by manufacturers of wire rope and desired factor-of-safety. From Table 1, the maximum ground anchor tensile load is approximately 14 kips. Appendix K provides calculations for wire rope diameter based on the tensile strength of the cable. Additionally, cable sizing must also consider the capacity of the ground anchor based on grout-to-cable bond and this is discussed in the next section. Finally, a dual-cable ground anchor arrangement was selected because of the ease with which to create the connection between the ground anchor and precast concrete footing. With a dual cable anchor, the cable is looped around a heavy-duty thimble eye at the ground surface, thereby, creating a simple means for connecting a shackle to the ground anchor.

1.18.2 Grout – Cable Bond Capacity

An ultimate grout strand bond capacity of approximately 300 psi can typically be achieved in a grout column embedded within the ground. For rock, a significantly higher capacity could be attained. Based on this value and a factor-of-safety of 3.0, an allowable bond stress of 100 psi is used for selecting the anchor cable length. Appendix L provides the detailed calculations for the required ground anchor length based on this failure mode.

1.18.3 Grout – Ground Bond Capacity

The capacity of the grout-to-ground bond is based on numerous factors and a detailed discussion of this topic is beyond the scope of this report. Past engineering work at the site (design of the existing 87 rigid SSS) assumed an ultimate grout-to-ground bond strength of 53 psi. For the size of ground anchor cable selected for the project (as discussed previously), and with a desire to achieve 0.75 inch grout cover over the cable, a 2.5 inch diameter bore hole was chosen. The required length of ground anchor embedment was determined based on neglecting the first three feet of embedment into the slope due to concerns about the quality of the grout-to-ground bond in this disturbed region. The required minimum total length of ground anchor into the slope is $L_b = 12$ ft. past the face of the excavated region, and detailed calculations are provided in Appendix M. Figure 47 below illustrates the length of embedment (bonded length) of the ground anchor.

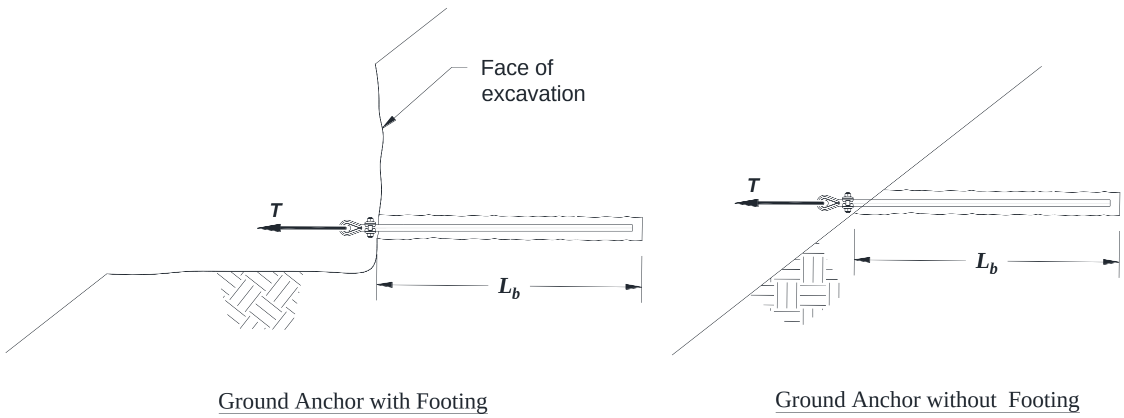


Figure 47 Ground anchor cable embedment (bonded length), L_B

1.18.4 Grout Materials

The quality of bond between the grout and ground, and between the grout and steel cable is related to the quality of the grout materials and the grouting operation. Typically a neat cement grout with water-to-cement ratio of 0.4 to 0.55 is used. Because of the low total number of ground anchors to be grouted for the project and for ease of use, a bagged proprietary high-strength non-shrink grout was utilized. The product, US Spec RA Grout, has a specified 28-day compression strength of 15,000 psi and detailed information is provided in Appendix N.

CONSTRUCTION AND INSTRUMENTATION

1.19 Fabrication of SSU Structural Steel

Upon completion of the site-specific design of SSU for the Milepost 151 location, structural plans were submitted to fabricators for bidding. A company located near Salt Lake City, Utah responded with a competitive bid and was selected for the project. The company had significant experience providing structural steel for ski industry projects, and thus, was best suited for the work. The fabricated steel parts of the SSU were shipped disassembled to Jackson, Wyoming, where the units were assembled by InterAlpine Engineers personnel (Figure 49).

The infill wire rope nets and perimeter cables were also manufactured by InterAlpine since ordering such a small quantity of these custom items from a manufacturer would be costly. An assembly jig was built using steel tubing and the perimeter wire rope was installed along the outside of the frame of the jig. Then, the infill net was built integral with the perimeter cable using a movable loom and proceeding in rows (Figure 50). The two ends of the infill net wire rope cable were connected to each other using a splice connection and wire rope clips.

The infill net and perimeter cable assembly was then placed over the SSU and the perimeter cable tensioned to achieve the correct sag and then spliced using the detail shown previously in Chapter 3. For a large-scale project where the perimeter wire rope is spliced using a swage fittings connection, the following process is envisioned. Because of the size of the swage fittings for the perimeter cable, the splicing of the perimeter cable needs to occur in the factory, and while the cable is not on the SSU. This makes installation onto an assembled SSU difficult, if not impossible. However, if the two cross members are only partially bolted so that they can rotate with respect to each other like blades on scissors, the infill net and perimeter cable can easily be placed onto the collapsed frame, as illustrated in Figure 48. For a square, the perimeter is four times the length of a side or $4B$. If the SSU frame is collapsed into a rectangle, and in the limit the cross members are parallel to one another, their length is $\sqrt{2}B$, which is about $1.4B$, much less than the perimeter cable length. So, a prefabricated perimeter cable with swaged splice can easily be hung onto the SSU. Once the perimeter cable is draped over the corner deviators, then the cross members can be pulled back into their final 90 degree alignment and then be bolted together.

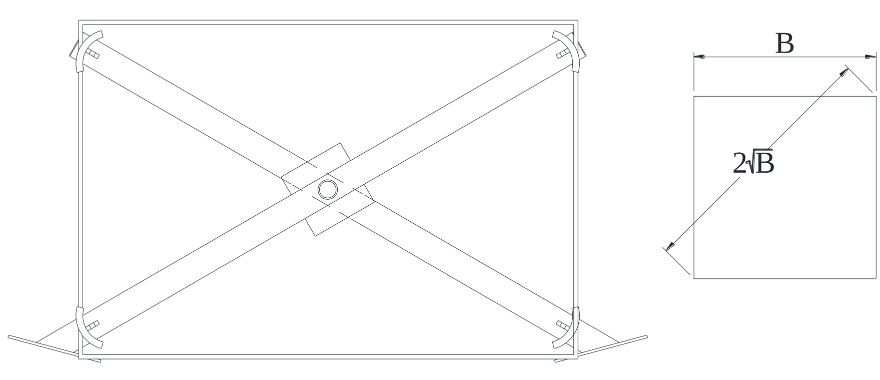


Figure 48 Collapsed SSU cross members allowing for perimeter cable installation

1.20 Foundation Construction

Ground anchor cables were fabricated by InterAlpine using wire rope stock and hardware. Precast concrete footings were also produced by InterAlpine and transported to the project site. A contractor specializing in drilling was used to drill the three ground anchor holes utilizing a portable handheld drill rig that required the use of a large air compressor. This air compressor weighed approximately 1000 lb. and was flown up to the site by helicopter. Figure 51 provides a photograph of the drilling operation in progress, while Figure 52 provides a photo of a completed hole. Ground anchor cables were fitted with centralizers at a spacing of approximately 3 ft along the length of the wire rope cable in order to center the ground anchor in the hole. A one-half inch diameter grout tube was bundled with the cables and extended the length of the cables so that grouting of the hole would proceed from the back towards the front of the hole. An air vent tube was also taped to the anchor cables so that any air could escape as grout filled the hole. The ground anchor assembly was then inserted into its hole to the appropriate depth and the front of the hole was sealed using expansive gap filling foam. Figure 53 provides a photograph of the anchor cable inserted into the hole but prior to sealing the opening, while Figure 54 shows the anchor cable and sealed opening with the grout and air vent tubes exiting the hole.

The grouting operation was performed using a hand operated portable grout pump and hand mixed grout (Figure 55). Because of the relatively cold ambient temperatures during the grouting operation, warmed water was used for the mix water. Grout was mixed in a five-gallon bucket per the manufacturer’s recommendations using a hand drill motor and a paddle mixer. Total volume of grout pumped was monitored to check that at least the anticipated grout volume was achieved, and this was confirmed for each of the three holes. Grouted anchors cured for approximately one month prior to installation of the SSU, and no pull-testing of the anchors was performed.

1.21 SSU Installation at Site

SSU units and the precast concrete footings were flown into the site by helicopter, approximately one month after the completion of the ground anchors. Installation of each unit proceeded quickly with a total time from the start to finish of less than one hour. This confirmed the benefit of rapid installation with minimal work on-site for “erection” of the SSU. A unit was simply lowered to its location and a shackle was used to connect the slip cable to the precast footing (Figures 56 and 57). The minimum required crew on the slope for this operation could be as little as two persons, and there would be one person at the location where units are being transported from by the helicopter. Thus, the construction effort related to installation of the SSU units themselves is expected to be reduced compared to rigid SSS or snow nets. Figure 58 shows the final installed state of the three snow supporting structures, while Figures 59 and 60 provide a close-up view of the slip plate, slip cable, and footing.

Figure 49 Photograph of SSU on the valley floor prior to lifting onto the site via helicopter

Figure 50 Infill net and perimeter wire rope system during fabrication

Figure 51 Drilling operation of ground anchor holes

Figure 52 Ground anchor hole after drilling operation

Figure 53 Foundation anchor cable inserted down the drilled hole and with grout tube

Figure 54 Foundation anchor cable sealed in drilled hole with grout and air vent tube

Figure 55 Foundation cable anchor hand grout pump operation

Figure 56 Foundation anchor cable attached to small precast concrete footing

Figure 57 Photograph of precast concrete footing and SSU attachment slip cable

Figure 58 Three snow supporting umbrellas installed at the Milepost 151 project site

Figure 59 View of the anchor slip plate, cable and the small concrete footing

Figure 60 SSU foundation connection slip plate

1.22 Instrumentation Array

Existing electronic instrumentation from the previous research project at the site was adapted for use for measurement of the response of one SSU. Data acquisition equipment, snow depth sensors, glide shoes, and vibrating wire strain gages were inspected and repaired before installation for this research project. A tower was installed next to the southernmost SSU for the snow depth sensor (Figure 61), glide shoes were placed on either side of the SSU (Figure 63), and two vibrating wire strain gages were installed on the central tube of the southernmost SSU (Figure 62). The system was calibrated and began recording data in the fall of 2018.

During January of 2019, InterAlpine made several repeated attempts to connect with the data acquisition system remotely via the cellular modem but without success. WYDOT avalanche personnel hiked up to the project site during the summer of 2019, and subsequently, alerted InterAlpine staff about damage to the instrumentation array. Upon making a site visit, InterAlpine found the instrumentation to be significantly damaged including snow depth sensors, cell modem antenna, and strain gages. The cell modem wiring was ruptured, which explained the inability to connect to the system remotely, and both snow depth sensors experience significant deformation so that the sensor no longer point towards the ground surface. Figures 64 through 66 illustrate the condition of these instruments on the site visit. Because of the above failure of most of the instrumentation, the data recorded by the data acquisition system was essentially useless. For example, the snow depth sensor data displayed a “jump” in distance to the ground at the time that the sensor became damaged and subsequent readings were erratic and meaningless (sensor pointing off into space). Because the strain gages became dislodged from the central tube, internal axial strain data for the central tube was not accurate. Therefore, no comprehensive analysis of recorded data was performed beyond the initial review that revealed the spurious data.

Figure 61 Snow depth sensor mounted to mast next to snow supporting umbrella

Figure 62 Vibrating wire strain installed on SSU central tube

Figure 63 Glide shoe to measure snowpack movement along slope

Figure 64 Severely damaged snow depth sensor after winter 2018 - 2019

Figure 65 Cellular modem antenna damage from winter 2018 – 2019

Figure 66 Backup snow depth sensor damage from winter 2018 - 2019

1.23 SSU Performance and Condition Assessment

The performance of the three SSU was observed over a two winter period and the units were inspected during the summer after each winter. No additional avalanche releases in the region of previous slides were observed during the observation period. During the second winter of operation, it was observed from the valley floor that SSS and the SSU were completely buried (obscured) by the snowpack. Based on this and the excellent condition of the SSU units themselves as observed during the summer of 2019, it is reasonable to conclude that the SSU have performed effectively and as expected.

A final visual inspection of all elements of each SSU was conducted in 2019 and all components appeared to be in very good condition and without any visual signs of distress. The slip connection was also evaluated and no slippage of the slip cable through the fist grip clamps was observed, thus indicating that an overload condition was not experienced in either winter of operation. This is an item that should be checked each year when the site is free of snow (i.e. summertime). The infill net, guy anchor cables, and perimeter cable all appeared to be in excellent condition. For future inspections, all wire rope clips should be inspected to check for signs of slippage of cables at their end termination connections.

COST COMPARISON

1.24 Introduction

One of the most significant benefits of the SSU avalanche defense system is the potential for cost savings compared to other forms of starting zone constructed defense systems. Specifically, the other two main types of snow supporting structures, rigid snow fences and flexible snow nets, have been used extensively in Europe, and to some extent in the United States but both require intensive labor in the field to install. The SSU concept is relatively new and little cost data, and experience domestically exists, while more information is available for the other types of SSS. Included in this section of the report is a construction cost estimate comparison between the three types of SSS based on one domestic project for rigid snow bridges, one domestic project for snow nets, and two recent domestic projects for SSU. The given costs do not include pre-construction engineering design fees, they represent only construction costs.

1.25 Snow Supporting Structure Projects

1.25.1 Jackson South Snow Supporting Structures

The Milepost 151 Snow Supporting Structures avalanche mitigation project was funded and managed by WYDOT in 2012. A total of 87 rigid snow bridges (fences) were installed in the starting zone of the Milepost 151 site immediately above US 89/191 just south of Jackson, Wyoming. Each steel SSS unit required six hollow-core micropile foundations and unit width was 12.0 ft measured parallel to a slope contour line, with design retained snow height of 6.5 ft. The project also included a re-forestation component which included construction of soil benches and planting of evergreens along the benches.

- Original engineer's (InterAlpine LLC) construction cost estimate: $2,200,000 USD
- Winning bid: $2,300,000 USD
- Change orders: $100,000 USD
- Total lineal feet of snow fence: 1044 ft.
- Cost per lineal foot = $\frac{\$2{,}400{,}000}{1044\ ft} = \$2300/ft$
- Cost per square foot of snowpack retained: = $\frac{\$2{,}400{,}000}{87\ x\ 12\ x\ 6.5} = \$353/ft^2$
- Construction cost index multiplier: 1.315
- Adjusted cost per lineal foot $= 1.315\ x\ \$2300/ft$ = $3026/*ft*
- Adjusted cost per square foot of snowpack retained: = $1.315\ x\ \$353/ft^2$ = $465/$ft^2$

1.25.2 I-90 Snoqualmie Pass Snow Nets

The Washington State DOT has been conducting work on Interstate 90 at Snoqualmie Pass to improve wintertime east, west mobility along this critical transportation corridor. Snoqualmie Pass has a long history of significant avalanche activity, and part of the improvements along the route consisted of avalanche hazard mitigation along three slide paths using flexible snow nets. Three different snow net heights were used in the project: $B =$ 9.8 ft (1247 lineal feet total), 11.5 ft (1148 lineal feet total), and 13.1 ft (1941 lineal feet total) Listed below are some of the cost details as obtained from the avalanche consultant, and all numbers are approximate. It is noted that the project also included work to add features to the starting zone that would artificially roughen the terrain, thereby reducing snowpack glide down the slope. The project was constructed in 2014.

- Original engineer's construction cost estimate: \$10,000,000 USD
- Winning bid: \$6,000,000 USD
- Change orders: \$2,900,000 USD
- Total lineal feet of snow net: 4336 ft.
- Cost per lineal foot = $\frac{\$8,900,000}{4336\ ft} = \$2052/ft$
- Cost per square foot of snowpack retained: = $\frac{\$8,900,000}{1941x13.1+1148x11.5+1247x9.8} = \$175/ft^2$
- Construction cost index multiplier: 1.25
- Adjusted cost per lineal foot $= 1.25\ x\ \$2052/ft$ = \$2565/*ft*
- Adjusted cost per square foot of snowpack retained: =$1.25\ x\ \$175/ft^2$ = \$219/ft^2

1.25.3 Park City Mountain Resort Project

A project to install 214 SSU at the Park City Mountain Resort is currently planned for summer 2020 construction and bids for the project have been received. The project entails three different active slide paths that endanger a private road within a gated exclusive private residential development. A total of 64 $B = 4.0$ m (13.1 ft) tall and 150 B = 3.5 m (11.5 ft) tall units are planned for the project. Unit width is 3.45 m (11.3 ft) with a 0.5 m (1.64 ft) space in between in unit. The SSU being purchased for the project are being sourced from Italy.

- Original engineer's construction cost estimate: ≈ $\$2{,}000{,}000\ USD$
- Winning bid for SSU units (all elements above ground): \$700,000 USD
- Low bid for SSU installation including foundations: \$299,000 USD
- High bid for installation including foundations: \$3,500,000 USD
- Average bid for installation including foundations: \$1,328,500 USD
- Total lineal feet of snow net: 2422 ft.
- Total estimated project construction cost: \$2,030,000 USD
- Budgeted contingency: \$150,000
- Cost per lineal foot = $\frac{\$2,030,000+\$150,000}{2422\ \ ft} = \$900/ft$
- Cost per square foot of snowpack retained: = $\frac{\$2,030,000}{11.3x(64x13.1+150x11.5)} = \$70/ft^2$

1.25.4 Red Cloud Deer Valley Resort

Six SSU units were installed above a private residence in the Deer Valley Ski resort area during the fall of 2019. SSU height was 3.5 m (11.5 ft) and width was 3.45 m (11.3 ft) with a 0.5 m (1.64 ft) space in between units. The total construction cost was approximately \$45,000. The SSU purchased for the project were sourced from Italy.

- Cost per lineal foot = $\frac{\$45,000}{6\ x\ 11.3\ \ ft} = \$665/ft$

- Cost per square foot of snowpack retained: $= \frac{\$45{,}000}{6x11.3x11.5} = \$58/ft^2$

1.26 Discussion of Cost Data

Costs per lineal foot and per square foot of installed SSU for the rigid SSS near Jackson, Wyoming, and the snow nets in Washington State were increased to account for inflation using *RSMeans Construction Cost Indexes*. Clearly the rigid steel SSS and the flexible snow nets have significantly higher per lineal foot costs as compared to the snow umbrella system. Even if costs associated with the reforestation work for the Jackson, Wyoming, project and ground surface roughening for the Snoqualmie Project were removed, it is likely that installed cost for these systems would still be markedly higher than the SSU system. However, both of these projects were performed for State Departments of Transportation, and typically public works projects have additional work items that are required to be performed that may not be included in a privately funded project.

A cost per square foot of snowpack retained (vertical plane immediately behind the unit) was also calculated for all the projects. The rigid steel SSS of the Jackson, Wyoming project are the highest, but this is not expected because the design snow depth was only about one-half of that for all of the other projects. A significant portion of construction cost is the installation of ground anchor foundations, which does not scale proportionally with structure height. As unit height increases, so too must the foundation "size" but much of the foundation installation cost is fixed. The Snoqualmie Project per square foot cost is likely a better comparative case as the snow nets on that project were approximately the same height as those used for both of the SSU projects.

Despite not being able to account explicitly for all of the differences between the various avalanche hazard mitigation projects, it can be confidently stated that due to the reduction in the number of required foundations for the SSU system, construction cost for this system is expected to be lower than snow nets or rigid steel frames. Moreover, significantly less labor on the slope is required to install the SSU unit and this will too ultimately lead to a lower overall cost of this system compared to rigid SS and snow nets.

1.27 Non-Economic Benefits

There are additional benefits to the SSU that are not economic in nature. The reduced amount of labor that is required in the field where units are being installed means that construction risk is reduced for the SSU. Moreover, a SSU unit can easily be repaired or replaced by simply releasing the connection to the ground anchor and flying it out with a helicopter. This is not as easily performed for either of the other two types of SSS. Finally, the SSU presents less hazard to wildlife as compared to snow nets as big game have been become ensnared in snow nets. The net surface of a snow net is almost horizontal on its uphill anchor point and animal legs can easily get caught in the netting fabric. The SSU net on the other hand is almost vertical, and therefore, does not present the same hazard. Moreover, snow nets are typically installed in long continuous rows without any break. SSU can be installed with a small gap in between each unit, thereby allowing for large game to move freely in between rows of SSU. The SSU system therefore reduces overall danger to wildlife foraging in or migrating through the area.

SUMMARY AND CONCLUSIONS

This research project investigated a new type of avalanche mitigation structure to be installed in an avalanche starting zone in series of individual units. This novel concept represents a natural step in the evolution of the design of snow supporting structures that hold a snowpack in place, thereby preventing the release of a natural avalanche. Because snow loading on a structure located within a starting zone area can be quite large, snow supporting structures have often required numerous foundations for stability. Existing technologies for snow supporting structures also require intensive effort to erect on the slope, presenting risk to construction personnel. The new concept for snowpack support resembles an upside-down umbrella and has been called a “snow supporting umbrella” (SSU). The system relies on a single ground anchor embedded in the slope and is loaded in tension by the snow loads acting on the SSU. The downhill supports on the SSU unit consist of steel bearing plates that simply rest on the ground surface. The SSU concept represents a significant reduction in foundation costs and therefore overall construction costs. Moreover, because of the simple foundation arrangement of a single point uphill ground anchor and bearing plates at the downhill side of the unit, units can be placed into the starting zone by helicopter fully assembled and with minimal effort by crews on the ground. A single steel shackle connects each SSU unit to its ground anchor foundation. They are placed side-by-side in multiple rows across the starting zone.

Snow loads acting on a structure placed in an avalanche starting zone are calculated by a European technical guide that is the world-wide standard for this practice. The most difficult aspect of the determination of snow effects is establishment of the site-specific snow load parameters including slope angle, glide factor, and site elevation. A straight-forward analysis process that requires the use of advanced analysis software for calculation of the complex distribution of snow forces within the SSU was presented. Simplifications in modelling allow for a step-wise determination of SSU cable maximum tension forces. Initial sag of the various cable elements in the SSU is a critical parameter that influences the final cable tension in the loaded configuration. Structural steel elements in the SSU unit are designed according to standard practice for structural engineering in the United States.

Other design professionals and avalanche consultants have expressed concerns about the lack of redundancy of the SSU foundation system. A very simple and cost-effective slip mechanism was developed that acts as a fuse so that foundation loading is limited to a specified slip load value. The concept utilizing existing materials and hardware and relies on friction created between the slip cable and the slip plate at the uphill SSU ground anchor. There are other options for addressing the lack of redundant foundation system and these include increasing the geotechnical factors-of-safety used in design of the ground anchors. Another option is to provide a “dummy” or blank ground anchor in between each of the ground anchors of adjacent SSU units. A structural cable could be run along a contour line and through the ground anchor shackle connections including the unused ground anchor so that if a given ground anchor fails, the snow loads can be redistributed to other ground anchors in its vicinity. In fact, in addition to serving the aforementioned purpose, the dummy or sacrificial ground anchors could be used for proof-load testing purposes to verify grout-to-ground bond strengths.

The SSU unit design developed specifically for this project and the Milepost 151 Avalanche is lightweight and simple to fabricate and assemble. Interior units which make up the majority of units used for a project weight less than 200 lb each and can be maneuvered on a slope easily by two persons. Exterior units which are designed for the higher end-effect pressures are only used at the ends of a row of SSU units and weigh approximately 300 lb. Taller SSU units that are intended for deeper snowpack depths can now be designed relatively easily and are likely to still be light-weight compared to rigid snow bridges. The SSU developed and detailed in this report can also be utilized at other avalanche sites as long as the starting zone slope inclination is 40 degrees or less, and the site elevation is 8500 ft or less. Sites with one of these parameters above the limits may still utilize the design but a quick calculation of snow pressure using the Swiss Guide would be required to verify that the snow loads are within the limits of the design.

A number of inefficiencies in the fabrication and construction of the SSU for this project existed but could be easily remedied on future larger-scale projects that implement the technology. Most of the inefficiency resulted from the small number of units being produced, three, and the fact that economies of scale almost always exist in the production of products and materials. For example, hand fabrication of the infill snow nets was required since a custom order to a manufacturer would have been needed to obtain the appropriate net size for the SSU unit. Perimeter cables and guy anchor cables were produced using wire rope clips for splices and the turnback (dead) end where thimbles were used as end terminations, but swage fitting can serve the same the purpose at a cheaper cost than wire rope clips.

The SSU unit design developed in this work, and put into use at the Milepost 151 site, has effectively prevented any additional natural avalanche releases over a two winter season period. The performance in service has thus been excellent with no noted issues in any of the elements of the SSU. Because of simple connection to the ground, if repairs to any features of a SSU are ever needed, a unit can be quickly removed from the slope and flown to a level area where work can be more easily accomplished. This cannot be said for other existing snow supporting structure technologies. Furthermore, the SSU presents less risk to big game than snow nets in which the legs of game can become entangled more easily.

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APPENDIX A

Calculation of snow loads per the Swiss Technical Guidelines for Avalanche Defense Structures.

WYDOT Snow Umbrellas Snow Loads by Swiss Guide

Geometry and Loading Parameters

H_k :	6.50	ft.	=	1.98	m	vertical SSS height
D :	4.979	ft.	=	1.52	m	SSU surface length (normal to slope)
B :	5.30	ft.	=	1.62	m	SSU surface length (along SSU grate)
L_{SSU} :	5.00	ft.	=	1.52	m	length of SSS (across slope width)
A :	20.0	ft.	=	6.10	m	distance between ends of adjacent SSU
Z :	8500	ft.	=	2591	m	elevation
ψ :	40	deg	=	0.70	rad	slope angle
δ :	20	deg	=	0.35	rad	angle btwn SSS surface and normal to slope
N :	3.2					glide factor: largest value in Swiss Guide
η :	5.0					strut tranverse load factor
ρ :				0.270	t/m^3	snow density (basic value increased by fc factor)
g :	32.808	ft/sec^2		10.0	m/sec^2	gravity (rounded up)
a :	0.35					force perpindicular to slope factor, (0.35 <= a <= 0.50)
f_c :	1.22					snow density increase with elevation factor
f_s :	0.8					flexible snow supporting structure factor
LC2 :	1.30					Load case two pressure increase factor = 1/0.77

Basic Loads for Infinite Length Supporting Surface (NO end-effects)

S'_N :	15.3	kN/m	=	1048	lbs/ft	snow pressure in the line of slope
S'_Q :	2.0	kN/m	=	137	lbs/ft	snow pressure component normal to the slope
G :	1.3	kN/m	=	86	lbs/ft	weight of snow prism for SSS surface not normal to slope
G'_N :	0.43	kN/m	=	29	lbs/ft	component of weight of snow prism parallel to slope
G'_Q :	1.18	kN/m	=	81	lbs/ft	component of weight of snow prism normal to slope
R'_N :	15.73	kN/m	=	1078	lbs/ft	total snow load parallel to slope (normal to SSS surface)
R'_Q :	3.18	kN/m	=	218	lbs/ft	total snow load normal to slope (parallel to SSS surface)
R' :	16.05	kN/m	=	1100	lbs/ft	total snow load acting on SSS surface
ε_R :	11.4	deg.	=	0.199	rad	angle between the resultant and line of slope
p_{Int} :				$\boxed{1.4}$	psi	uniform snow pressure without end-effects
				1.50	psi	final design pressure (unfactored) without end-effects

End-effect Loads

f_R :	5.00					end effect factor (Eqn 22, p. 61)
S'_R :	76.5	kN/m	=	5242	lbs/ft	end effect loading (Eqn 21, p. 61)
ΔL :	0.51	m	=	1.66	ft.	length of end effect region (Eqn 23, p. 61)
R'_N :	92.24	kN/m	=	6320	lbs/ft	total snow load parallel to slope (normal to SSS surface)
R'_Q :	3.18	kN/m	=	218	lbs/ft	total snow load normal to slope (parallel to SSS surface)
R' :	92.29	kN/m	=	6324	lbs/ft	total snow load acting on SSS surface
ε_R :	2.0	deg.	=	0.034	rad	angle between the resultant and line of slope
p_{Ext} :				$\boxed{8.3}$	psi	uniform snow pressure in the end-effect region
				8.61	psi	final design pressure (unfactored) with end-effects

APPENDIX B

Sizing of infill net wire rope steel cables. Design is by the ASCE 19-16 *Structural Applications of Steel Cables for Buildings*. 6x19 IWRC galvanized wire rope is assumed.

S_{Min} = Minimum required allowable tension capacity of cable

$S_{Min} = \frac{T \; x \; \Omega}{N_f}$ where:

T = cable unfactored tension force

Ω = 2 (factor-of-safety)

N_f = fitting reduction factor =0.8 for wire rope clips (see ASCE 19-16)

Exterior SSU Units

Maximum infill net wire rope tension, T = 1,225 lb (Infill Net Exterior FE Model)

$$S_{Min} = \frac{1{,}225 \; lb \; x \; 2}{0.8} = 3{,}063 \; lb$$

From Wire Rope capacity chart (next page), 3/16 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 4,200 lb = 3,780 lb

Use 3/16" diameter EIPS 7x19 galvanized wire rope (A.k.a. "aircraft cable")

Interior SSU Units

Maximum infill net wire rope tension, T = 350 lb (Infill Net Interior FE Model)

$$S_{Min} = \frac{350 \; lb \; x \; 2}{0.8} = 875 \; lb$$

From Wire Rope capacity chart (next page), 1/8 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 2,000 lb = 1,800 lb. Note that a practical lower limit on size has been deemed at 1/8 in and therefore not using the 3/32 in cable that would appear to have sufficient strength.

Use 1/8" diameter EIPS 7x19 galvanized wire rope (A.k.a. "aircraft cable")

APPENDIX C

Sizing of perimeter wire rope steel cables. Design is by the ASCE 19-16 *Structural Applications of Steel Cables for Buildings*. 6x19 IWRC galvanized wire rope is assumed.

S_{Min} = Minimum required allowable tension capacity of cable

$S_{Min} = \frac{T \times \Omega}{N_f}$ where:

T = cable unfactored tension force

Ω = 2 (factor-of-safety)

N_f = fitting reduction factor =0.8 for wire rope clips (see ASCE 19-16)

Exterior SSU Units

Maximum perimeter wire rope tension, T = 10,116 lb (Infill Net Exterior FE Model)

$$S_{Min} = \frac{10{,}116\ lb \times 2}{0.8} = 25{,}290\ lb$$

From Wire Rope capacity chart (next page), 9/16 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 16.8 x 2000 lb = 30,240 lb

Use 9/16” diameter EIPS 6x19 galvanized wire rope cable

Interior SSU Units

Maximum perimeter wire rope tension, T = 4255 lb (Infill Net Interior FE Model)

$$S_{Min} = \frac{4255\ lb \times 2}{0.8} = 10{,}640\ lb$$

From Wire Rope capacity chart (next page), 3/8 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 7.55 x 2000 lb = 13,590 lb

Use 3/8” diameter EIPS 6x19 galvanized wire rope cable

6x19 Wire Rope Capacities Chart

Diameter (in.)	FIBER CORE Approx. wt./ft. (lb.)	FIBER CORE Minimum breaking force (tons of 2,000 lb.) IPS	FIBER CORE Minimum breaking force (tons of 2,000 lb.) XIP®	IWRC Approx. wt./ft. (lb.)	IWRC Minimum breaking force (tons of 2,000 lb.) IPS	IWRC Minimum breaking force (tons of 2,000 lb.) XIP®	IWRC Minimum breaking force (tons of 2,000 lb.) XXIP®
3/16	0.059	1.55	1.71				
1/4	0.105	2.74	3.02	0.116	2.94	3.40	
5/16	0.164	4.26	4.69	0.18	4.58	5.27	
3/8	0.236	6.10	6.72	0.26	6.56	7.55	8.30
7/16	0.32	8.27	9.10	0.35	8.89	10.2	11.2
1/2	0.42	10.7	11.8	0.46	11.5	13.3	14.6
9/16	0.53	13.5	14.9	0.59	14.5	16.8	18.5
5/8	0.66	16.7	18.3	0.72	17.9	20.6	22.7
3/4	0.95	23.8	26.2	1.04	25.6	29.4	32.4
7/8	1.29	32.2	35.4	1.42	34.6	39.8	43.8
1	1.68	41.8	46.0	1.85	44.9	51.7	56.9
1 1/8	2.13	52.6	57.8	2.34	56.5	65.0	71.5
1 1/4	2.63	64.6	71.1	2.89	69.4	79.9	87.9
1 3/8	3.18	77.7	85.5	3.50	83.5	96.0	106
1 1/2	3.78	92.0	101	4.16	98.9	114	125
1 5/8	4.44	107	118	4.88	115	132	146
1 3/4	5.15	124	137	5.67	133	153	169
1 7/8	5.91	141	156	6.50	152	174	192
2	6.72	160	176	7.39	172	198	217
2 1/8	7.59	179	197	8.35	192	221	244
2 1/4	8.51	200	220	9.36	215	247	272
2 3/8				10.4	239	274	
2 1/2				11.6	262	302	
2 5/8				12.8	288	331	
2 3/4				14.0	314	361	
2 7/8				15.3	341	392	
3				16.6	370	425	
3 1/8				18.0	399	458	
3 1/4				19.5	429	492	
3 3/8				21.0	459	529	
3 1/2				22.7	491	564	
3 5/8				24.3	523	602	
3 3/4				26.0	557	641	
3 7/8				27.7	591	680	
4				29.6	627	720	
4 1/8				31.7	658	757	
4 1/4				33.3	694	799	
4 3/8				35.4	734	844	

Available galvanized at 10% lower strengths, or in equivalent strengths on special request.

Source: Wire Rope handbook, Union Rope Company; www.unionrope.com

APPENDIX D

Sizing of guy anchor wire rope steel cables. Design is by the ASCE 19-16 *Structural Applications of Steel Cables for Buildings*. 6x19 IWRC galvanized wire rope is assumed.

S_{Min} = Minimum required allowable tension capacity of cable

$S_{Min} = \frac{T \ x \ \Omega}{N_f}$ where:

T = cable unfactored tension force

Ω = 2 (factor-of-safety)

N_f = fitting reduction factor =0.8 for wire rope clips (see ASCE 19-16)

Exterior SSU Units

Maximum guy anchor wire rope tension, T = 3,305 lb (Exterior SSU FE Model)

$$S_{Min} = \frac{3{,}305 \ lb \ x \ 2}{0.8} = 8{,}263 \ lb$$

From Wire Rope capacity chart (previous page), 5/16 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 5.27 ton x 2000 lb = 9,486 lb

Use 5/16” diameter EIPS 6x19 or 7x19 galvanized wire rope

Interior SSU Units

Maximum guy anchor wire rope tension, T = 1600 lb (Interior SSU FE Model)

$$S_{Min} = \frac{1600 \ lb \ x \ 2}{0.8} = 4000 \ lb$$

From Wire Rope capacity chart (previous page), 1/4 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 6,800 lb = 6,120 lb. Note that a practical lower limit on size has been deemed at 1/4 in.

Use 1/4” diameter EIPS 6x19 or 7x19 galvanized wire rope

7x19 Wire Rope Capacity Chart

7 x 19 - GALVANIZED AIRCRAFT CABLE: PVC								
DIAMETER (IN)		WEIGHT						BREAKING STRENGTH LB
INNER CABLE	OUTER CABLE	INNER CABLE		PVC COATING		TOTAL		
		LB/1000'	KG/1000'	LB/1000'	KG/1000'	LB/1000'	KG/1000'	
3/32	1/8	17.4	7.89	3.2	1.44	20.6	9.33	1,000
3/32	5/32	17.4	7.89	7.2	3.26	24.6	11.15	1,000
1/8	5/32	29.0	13.15	4.1	1.84	33.1	14.99	2,000
1/8	3/16	29.0	13.15	9.0	4.06	38.0	17.21	2,000
5/32	7/32	45.0	20.41	10.8	4.90	55.8	25.31	2,800
5/32	9/32	45.0	20.41	25.1	11.40	70.1	31.81	2,800
3/16	1/4	65.0	29.48	12.6	5.70	77.6	35.18	4,200
3/16	5/16	65.0	29.48	28.8	13.05	93.8	42.53	4,200
1/4	5/16	110.0	49.90	16.2	7.33	126.2	57.23	7,000
1/4	3/8	110.0	49.90	35.9	16.30	145.9	66.20	7,000
5/16	3/8	173.0	78.47	19.8	8.96	192.8	87.43	9,800
5/16	7/16	173.0	78.47	43.1	19.55	216.1	98.02	9,800
3/8	7/16	243.0	110.22	23.3	10.60	266.3	120.82	14,400
3/8	1/2	243.0	110.22	50.3	22.80	293.3	133.02	14,400

Source: www.blairwirerope.com

APPENDIX E

Design check of cross members for interior SSU units using interaction equation for biaxial bending and axial compression. AISC Specification Chapters E, F, and H apply.

Calculation of Axial Compression Capacity

L_C = Effective unbraced length of cross member = 7.1 ft

r = radius of gyration of cross member section = 1.14 in

$\frac{L_C}{r} = \frac{7.1\ ft\ x\ 12\ in/ft}{1.14} = 74.7\ \approx 75$

$\phi_C F_{cr} =$ critical buckling stress $= 28.4\ ksi\ (per\ AISC\ SCM\ Table\ 4-14)$

$\phi_C P_n = \phi_C F_{cr} A_g = 28.4\ ksi\ \ x\ 1.89\ in^2 = 53.6\ kip\ (compression)$

Check which interaction formula applies

$P_U =$ factored axial compression force from loading $= 9.20$ kip

$$\frac{P_U}{\phi_C P_n} = \frac{9.2}{53.6} = 0.17\ < 0.2 \quad \therefore \ \text{Use equation H1} - 1\text{b}$$

Calculate Flexural Capacity

$\phi_b M_n = \phi_b\ F_y\ Z_x$ (AISC Specification Chapter F)

$\phi_b M_n = 0.9\ x\ 46\ x\ 1.97\ in^3 = 81.5\ kip - in$ (this flexural capacity applies to both direction of bending since section is symmetric)

Check Interaction Equation

$M_{Ux} =$ factored bending moment about x − axis from loading $= 39$ kip − in

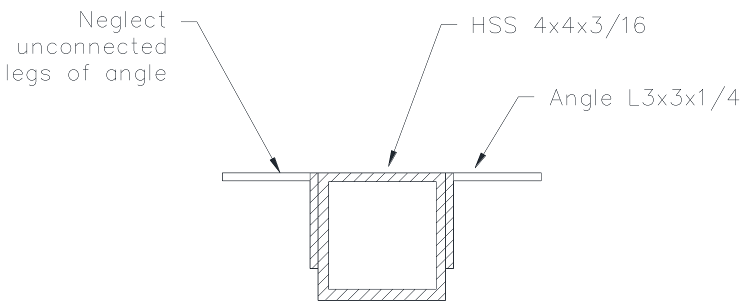
$M_{Uy} =$ factored bending moment about x − axis from loading $= 23$ kip − in

Interaction check: $\frac{P_U}{2\ \phi_c P_n} + \left(\frac{M_{Ux}}{\phi_b M_n} + \frac{M_{Uy}}{\phi_b M_n}\right) = \frac{9.2}{2\ x\ 53.6} + \left(\frac{39}{81.5} + \frac{23}{81.5}\right) = 0.932 < 1.0$

Therefore HSS3x3x 3/16 is sufficient

APPENDIX F

Design check of cross members for exterior SSU units using interaction equation for biaxial bending and axial compression. AISC Specification Chapters E, F, and H apply. Stress checks within the cross members suing the basic HSS section properties are below unity except in the vicinity right near the center tube connection. In this region, the cross members are strengthened by the presence of the angle shapes that are welded along each side. Therefore, section properties used for calculation of capacity can incorporate the positive influence of the angles. Note that only one side of each angle actually welded to the HSS cross member is used for influence on section properties.



Cross Member Section at Center

Section Properties without Addition of Angles

$I_x = I_y = 6.21 \text{ in}^4$ $A_g = 2.58 \text{ in}^2$ $Z_x = Z_y = 3.67 \text{ in}^3$ (per ASIC SCM part 1)

Section Properties with Addition of Angles

$I_x = I_y = 7.33 \text{ in}^4$ $A_g = 4.00 \text{ in}^2$ $Z_x = Z_y = 4.79 \text{ in}^3$ (by hand calculations)

Calculation of Axial Compression Capacity

L_C = Effective unbraced length of cross member = 7.1 ft

r = radius of gyration of cross member section = 1.55 in (from AISC SCM Part 1)

$$\frac{L_C}{r} = \frac{7.1\ ft\ x\ 12\ in/ft}{1.55} = 55$$

$$\phi_C F_{cr} = \text{ critical buckling stress} = 33.8\ ksi\ (per\ AISC\ SCM\ Table\ 4-14)$$

$$\phi_C P_n = \phi_C F_{cr} A_g = 33.8\ ksi\ \ x\ 2.58\ in^2 = 87.2\ kip\ (compression)$$

Check which interaction formula applies

$$P_U = \text{ factored axial compression force from loading} = 19.6 \text{ kip}$$

$$\frac{P_U}{\phi_C P_n} = \frac{19.6}{87.2} = 0.22 > 0.2 \quad \therefore \text{ Use equation H1} - 1\text{a}$$

Calculate Flexural Capacity

$\phi_b M_n = \phi_b\, F_y\, Z_x$ (AISC Specification Chapter F)

$\phi_b M_n = 0.9\ x\ 46\ x\ 4.79\ in^3 = 198\ kip - in$ (this flexural capacity applies to both direction of bending since section is symmetric)

Check Interaction Equation

$M_{Ux} = \text{ factored bending moment about x} - \text{axis from loading} = 111 \text{ kip} - \text{in}$

$M_{Uy} = \text{ factored bending moment about x} - \text{axis from loading} = 46 \text{ kip} - \text{in}$

Interaction check: $\frac{P_U}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{Ux}}{\phi_b M_n} + \frac{M_{Uy}}{\phi_b M_n}\right) = \frac{19.6}{87.2} + \frac{8}{9}\left(\frac{46}{198} + \frac{111}{198}\right) = 0.93 < 1.0$

Therefore HSS4x4x 3/16 is sufficient

APPENDIX G

Sizing of slip cable. Design is by the ASCE 19-16 *Structural Applications of Steel Cables for Buildings*. 6x19 IWRC galvanized wire rope is assumed.

S_{Min} = Minimum required allowable tension capacity of cable

$S_{Min} = \frac{T\ x\ \Omega}{N_f}$ where:

T = cable unfactored tension force

Ω = 2 (factor-of-safety)

N_f = fitting reduction factor =0.8 for wire rope clips (see ASCE 19-16)

Exterior SSU Units

Maximum slip cable wire rope tension, T = 15,000 lb (Resultant of F_1 and F_2 from Table 1)

$$S_{Min} = \frac{15{,}000\ lb\ x\ 2}{0.8} = 41{,}250\ lb$$

From Wire Rope capacity chart (next page), 3/4 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 29.4 x 2000 lb = 52,920 lb

Use 3/4 ” diameter EIPS 6x19 galvanized wire rope cable

APPENDIX H

Sizing of exterior SSU cross member steel base plates – plan dimensions and material thickness.

Cross member foundation plate soil reaction force from simplified model (see Section 2.2.2)

$$N = \frac{(R_1 B \; + \; R_2 L)}{2L} \cos\delta$$

$$R_1 = \; p_{end} \; x \; \Delta L \; x \; B + \; p_{int} \; x \; (B - \Delta L) x \; B$$

$$R_1 = \; 8.6 \; psi \; x \; 1.66 \; ft \; x \; 5 \; ft \; x \; \frac{12^2 in}{1^2 ft} + \; 1.5 \; psi \; x \; (5 - 1.66) x \; 5 \; x \; \frac{12^2 in}{1^2 ft} = 13{,}920 \; lb$$

$$R_2 = \; 100 \; \frac{lb}{ft} \; x \; 7 \; ft = 700 \; lb$$

$$N = \frac{(13920 \; lb \; x \; 5 \; ft \; + \; 700 \; lb \; x \; 7 \; ft)}{2 \; x \; 7 \; ft} \cos 21$$

$N = \; 5000 \; lb$ (total reaction at downhill foundation; two bearing plates)

$N = \; 5440 \; lb$ from finite-element computer model output

Square bearing plate dimensions

Assumed allowable bearing pressure σ_A = 2000 psf (SSU completely unaffected by settlement)

$$\sigma_A = \frac{N}{b^2} \longrightarrow \quad b = \sqrt{\frac{N}{\sigma_A}} = \sqrt{\frac{5440 \; x \; 0.5}{2000}} = \; 1.166 \; ft = 13.99 \; in$$

Plate Thickness Sizing

Plate cantilevers from edge of HSS cross member

Assume outside edge of plate is bearing on a rock and therefore soil load is applied at edge of plate (this is conservative) so that distance is 5 in to face of HSS

Plate material yield stress, assume F_y = 50 ksi

$$M = \; F \; x \; distance = 0.5 \; x \; 5444 \; lb \; x \; 5 \; in = \; 13{,}610 \; lb - in$$

Trial plate thickness, t = 0.375 in

Allowable moment capacity of base plate, $M_A = \frac{Z_x F_y}{\Omega}$

Safety factor, $\Omega = 1.67$

Plastic section modulus, $Z_x = \frac{b\, t^2}{4} = \frac{14\ in\ x\ 0.375^2}{4} = 0.4921\ in^3$

$$M_A = \frac{0.4921\ in^3\ x\ \ 50{,}000\ psi}{1.67} = 14{,}736\ lb - in$$

Use 3/8 in thick cross member base plate

APPENDIX I

Sizing of interior SSU cross member steel base plates – plan dimensions and material thickness.

Cross member foundation plate soil reaction force from simplified model (see Section 2.2.2)

$$N = \frac{(R_1 B \ + \ R_2 L)}{2L} \cos \delta$$

$$R_1 = \ p_{end} \ x \ \Delta L \ x \ B + \ p_{int} \ x \ (B - \Delta L) x \ B$$

$$R_1 = \ 1.7 \ psi \ x \ 5 \ ft \ x \ 5 \ ft \ x \ \frac{12^2 in}{1^2 ft} = 6120 \ lb$$

$$R_2 = \ 100 \ \frac{lb}{ft} \ x \ 7 \ ft = 700 \ lb$$

$$N = \frac{(6120 \ lb \ x \ 5 \ ft \ + \ 700 \ lb \ x \ 7 \ ft)}{2 \ x \ 7 \ ft} \cos 21$$

$N = \ 2367 \ lb$ (total reaction at downhill foundation; two bearing plates)

$N = \ 5440 \ lb$ from finite-element computer model output

Square bearing plate dimensions

Assumed allowable bearing pressure σ_A = 2000 psf (SSU completely unaffected by settlement)

$$\sigma_A = \frac{N}{b^2} \longrightarrow \ b = \sqrt{\frac{N}{\sigma_A}} = \sqrt{\frac{2400 \ x \ 0.5}{2000}} = \ 0.775 \ ft = 9.3 \ in$$

Use 10 in x 10 in steel base plate

Plate Thickness Sizing

So that efficiency in fabrication can be achieved, use the same plate thickness as used for the exterior SSU base plates.

Use 3/8 in thick cross member base plate

APPENDIX J

Sizing of the precast concrete upper foundation footing is based on an allowable bearing stress of 2500 psf. It is felt that this provides a conservative, lower bound estimate for allowable stress because small settlement of the footing will not negatively influence the ground anchor system .

$\sigma = \frac{N}{A}$ where:

$N =$ unfactored compression force acting on footing

A = plan area of square footing

$$A \geq \frac{N}{\sigma_{Allowable}} = \frac{5600\ lb}{2500\ psf} = 2.24\ ft^2$$

$$b \geq \sqrt{2.24\ ft^2 =}\ 1.49\ ft = 17.9\ in$$

Footing thickness, t = 7.0 inch

Use 18 in *x* 18 in *x* 7 in footing

(4) #4 GD60 straight reinforcing bars each way with 2 inch cover over ends; mat centered

4,000 psi 28-day concrete strength

APPENDIX K

Sizing of ground anchor wire rope steel cables. Design is by the ASCE 19-16 *Structural Applications of Steel Cables for Buildings* with modification of the factor-of-safety for geotechnical design considerations. 6x19 IWRC galvanized wire rope is assumed.

S_{Min} = Minimum required allowable tension capacity of cable

$S_{Min} = \frac{T \ x \ \Omega}{N_f}$ where:

$T =$ cable unfactored tension force

Ω = 2 (factor-of-safety)

N_f = fitting reduction factor =0.8 for wire rope clips (see ASCE 19-16)

Exterior SSU Units

Maximum perimeter wire rope tension, T = 14,000 lb

$$S_{Min} = 14{,}000 \ lb \ x \ 3 = 42{,}000 \ lb$$

From Wire Rope capacity chart, 1/2 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 13.3 x 2000 lb = 23,940 lb. Two cables per ground anchor → 47,880 lb capacity

Use 1/2" diameter EIPS 6x19 galvanized wire rope cable looped to form two legs

Interior SSU Units

Maximum perimeter wire rope tension, T = 6,800 lb

$$S_{Min} = 6{,}800 \ lb \ x \ 3 = 20{,}400 \ lb$$

From Wire Rope capacity chart, 3/8 inch diameter EIPS galvanized has a minimum breaking strength of 0.9 x 7.55 x 2000 lb = 13,590 lb. Two cables per ground anchor → 27,180 lb capacity

Use 3/8" diameter EIPS 6x19 galvanized wire rope cable looped to form two legs

APPENDIX L

T = Unfactored tensile load applied to ground anchor = 14.0 kip

Ω = Factor-of-Safety for ground anchor pullout = 3.0

d = Diameter of the ground anchor cable = 0.5 inch

$q_U =$ Ultimate grout − strand bond strength = 100 psi

L_B = Required ground anchor bond length based on grout – ground bond

$$L_B = \frac{T\ x\ \Omega}{2\pi d\ x\ 12"\ x\ q_U} = \frac{14{,}000\ lb\ x\ 3}{2\ x\ \pi\ x\ 0.5"\ \text{x}\ \frac{12"}{ft} x\ 100psi} = 5.6\ ft \quad \therefore \quad L_B = 3 + 5.6\ \approx 9\ ft.$$

Therefore, total ground anchor length = 9 ft based on grout – strand bond capacity

APPENDIX M

T = Unfactored tensile load applied to ground anchor = 14.0 kip

Ω = Factor-of-Safety for ground anchor pullout = 3.0

d_{Hole} = Diameter of the drilled ground anchor hole = 2.5 inch

$q_U =$ Ultimate grout – ground bond strength = 50 psi

L_B = Required ground anchor bond length based on grout – ground bond

$$L_B = \frac{T\ x\ \Omega}{\pi d_{Hole}\ x\ 12"\ x\ q_U} = \frac{14{,}000\ lb\ x\ 3}{\pi\ x\ 2.5"\ \mathrm{x}\ \frac{12"}{ft} x\ 50psi} = 8.9\ ft \quad \therefore \quad L_B = 3 + 8.9\ \approx 12\ ft.$$

Therefore, total ground anchor length = 12 ft based on grout – ground bond capacity

APPENDIX N

RA Grout

High Flow, Non-Aggregate, Non-Shrink Anchoring Grout

DESCRIPTION

RA Grout is a blend of specialty cements and proprietary admixtures. This material is designed to provide maximum flow, shrinkage compensation and extended working times in an aggregate free formulation where clearances are minimal. **RA Grout** is non-metallic and non-corrosive.

USES

RA Grout is ideal for a wide variety of applications that include:

- Grouting of tight clearances between precast segments, beams, columns, fissures and cracks in rocks
- Anchor bolts, soil nails, rock and ground anchors, dowels and rods where sanded grouts restrict complete encapsulation
- Pumping applications and maximizing anchorages

BENEFITS

- Extreme fluidity: Can be pumped into areas that are virtually inaccessible with standard non-shrink grouts
- Working time: Extended for maximum pumping range
- Strength: Attains high compressive strengths at specified water ratios
- Thixotropic: High flow restored by agitation
- Corrosion Protection: Encapsulates tendons, bolts or bars to protect from corrosion
- Consistent: Strict Quality Control testing and standards

STANDARDS

RA Grout has been specifically formulated to meet and exceed the testing requirements of ASTM C1107 and Corp of Engineers CRD C621. When tested in accordance with ASTM C827, **RA Grout** yields a controlled, positive expansion. Meets or exceeds PTI DC35.1-14, Section 6.11.

SURFACE PREPARATION

All surfaces in contact with **RA Grout** shall be free of dirt, oil, grease, laitance and other contaminants that may act as bondbreakers. All unsound concrete should be removed to ensure a good bond. Smooth, dense surfaces need to be mechanically abraded to provide necessary bonding requirements. Mechanically prepare the substrate to a minimum CSP 5 following ICRI Guideline 310.2R to allow proper bonding. ACI recommends that the area to be grouted should be saturated for 24 hours before placement. Remove any standing water. Substrate should be saturated, surface dry (SSD). Maintain contact areas between 40°F (4°C) and 90°F (32°C) prior to grouting and during initial curing period.

FORMING

Method of forming must provide for rapid, continuous grout placement. For pourable grout, construct forms to retain grout without leakage. Forms should be coated with US SPEC form release for easy removal.

MIXING

For larger batches, use a mortar mixer with rotating blades. For smaller batches, use a heavy duty 1/2" (15 mm) (or larger) low-speed, corded drill and mixing paddle #6 per ICRI Technical Guideline 320.5. Pre-wet mixer and empty excess water. Place 3/4 of the required 7.75 quarts of cool, clean potable water in mixer, then add dry material. Mix on low RPM for a total of 3 to 5 minutes, adding the remaining water, until a homogenous mixture is achieved. When using a mortar mixer higher RPMs may be necessary to achieve a homogenous mixture. Mix only enough grout that can be placed within working time. For placements greater than 3" depth, **RA Grout** must be extended 30% by weight of powder, with clean, washed and dried 3/8" (1 cm) pea gravel. Do not blend excess water as this will cause bleeding leading to segregation and sedimentation. Do not use any other admixtures or additives.

PLACING

Grout should be placed using established procedures according to American Concrete Institute recommendations. Mechanical vibration may cause segregation. When necessary, provide vent holes. **RA Grout** must be 100% encapsulated to prevent cracking.

FINISHING & CURING

Follow standard ACI curing practices. Do not disturb formwork or grout for 24 hours.

STORAGE

Normal cement storage and handling practices should be observed. Store in an interior, cool, dry place. Shelf life is one year in original, unopened container.

LIMITATIONS

In addition to limitations already mentioned, please note the following. Do not apply when the surface or ambient temperature is below 40°F (4°C) or expected to fall below 40°F (4°C) within 48 hours. When grouting at minimum temperatures, ensure surfaces in contact with grout do not fall below 40°F (4°C) until final set has been achieved and grout has reached 3,000 psi. 3,000 psi compressive strength is typically achieved within 48 hours at a 40°F (4°C) constant although this should be verified on site as needed. Do not apply over surfaces that are frozen or contain frost. Do not apply over any active faults or cracks in the substrate without addressing any movement that may occur. Do not use as a patching or overlay mortar or in unconfined areas. Not recommended for heavy-duty precision grouting of machinery base plates or crane rails. Setting time will speed up in hot weather and slow in cold weather. For hot and cold weather applications, contact your US SPEC manufacturer's representative.

Source: https://www.usspec.com/products/product/ra-grout/

RA Grout

High Flow, Non-Aggregate, Non-Shrink Anchoring Grout

PHYSICAL PROPERTIES*

All Physical Property testing performed in laboratory conditions of 73.5 ± 3.5°F (23 ± 2°C) and a relative humidity no less than 50% unless otherwise determined by the test method or specification. All results represent **RA Grout** with 7.75 quarts water unless listed otherwise. Tests are conducted under standardized conditions for comparative purposes, and results may not be representative of performance under field conditions.

Property and Test Method	Results		
Compressive Strength ASTM C942 per 4.4.2	**1 Day** 4,500 psi (31.02 MPa)	**7 Days** 11,000 psi (75.84 MPa)	**28 Days** 15,000 psi (103.42 MPa)
Rate of Set ASTM C953 per 4.4.1	**Working Time** 2:30		**Set** 8:00
Bond Strength ASTM C882	**1 Day** 900 psi (6.20 MPa)	**7 Days** 1,600 psi (11.03 MPa)	**28 Days** 1,800 psi (12.41 MPa)
Fluidity ASTM C939	**Test** Flow Cone		**Efflux Time** <30 seconds
30 Minutes Fluidity Following 30 Seconds of Remixing	**Test** Flow Cone		**Efflux Time** <30 seconds
Wick Induced Bleed ASTM C940 modified per 4.4.6.1	**Age** 4 Hours		**Percent Bleed** 0%
Volume Change ASTM C1090	**1 Day** .02%		**28 Days** .03%
Chloride Ion Test ASTM C1152	**% Cl by Weight of Cementitious Grout** .07%		
Permeability ASTM C1202	**Age** 28 Days	**Applied Voltage** 30 V	**Charge Passed** <2,500 coulombs
Accelerated Corrosion Test	**Time** 1,000 hours		**RA Grout** No corrosion
Schupack Pressure Bleed ASTM C1741	**Gelman Pressure** 20 psi 30 psi 50 psi		**Percent Bleed** 0% 1.0% 1.1%
Inclined Tube Test EN 445 per 4.4.9	**Age** Immediately after Mixing 30 min after mixing with 30 sec remix		**Percent Bleeding** 0.0% 0.0%

DANGER

This product contains Crystalline Silica (CAS# 14808-60-7) and Portland Cement (CAS# 65997-15-1). Harmful if swallowed. Causes skin irritation. Causes serious eye damage. May cause an allergic skin reaction. May cause cancer. May cause respiratory irritation. Causes damage to organs through prolonged or repeated exposure. Do not eat, drink or smoke when using this product. Wash hands thoroughly after handling. Contaminated work clothing must not be allowed out of the workplace. Obtain special instructions before use. Do not handle until all safety precautions have been read and understood. Wear protective gloves/protective clothing/eye protection/face protection. Use only outdoors or in a well-ventilated area. Do not breathe the dust.

CALIFORNIA PROPOSITION 65: This product contains Crystalline Silica, Quartz (CAS# 14808-60-7) and may also contain other chemicals known to the State of California to cause cancer, birth defects or other reproductive harm.

FIRST AID

If swallowed: Immediately call a poison center/doctor. Rinse mouth. If in eyes: Rinse cautiously with water for several minutes. Remove contact lenses, if present and easy to do. Continue rinsing. Immediately call a poison center/doctor. If on skin: Wash with plenty of water. Take off contaminated clothing and wash it before reuse. If skin irritation or rash occurs: Get medical advice/attention. If inhaled: Remove person to fresh air and keep comfortable for breathing. Call a poison center/doctor if you feel unwell.

MANUFACTURER/TECHNICAL SERVICE

Contact your US SPEC manufacturer's representative for the most current product information. Always read and follow the warnings and instructions on the most current technical data sheets, available online at www.usspec.com.

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Source: https://www.usspec.com/products/product/ra-grout/