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Structural design of the 2013 Santa Clara University Solar Decathlon 'radiant house'

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STRUCTURAL DESIGN OF THE 2013 SANTA CLARA UNIVERSITY SOLAR DECATHLON 'RADIANT HOUSE'

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SENIOR DESIGN PROJECT REPORT

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ABSTRACT

The Structural Team for the Santa Clara University Solar Decathlon 2013 Radiant House achieved the task of designing a structurally operative, yet efficient, innovative, and sustainable house that satisfies the needs of everyone from Team Radiant House. The loadings to be considered in the structural design were: dead load, live load, wind load, seismic load, and roof load. The house design implements a classic system of joists, beams, studs, and shear walls to transfer loads to the piers, and finally to the foundations, in combination with a steel framing system to direct loads, increase stiffness and simplify connections for ease of construction, and transportation. The house was designed in softwood timber, with the intent of switching the material to bamboo in the joists and shear wall elements, an intent which was achieved in accordance with the success of other senior design groups. Santa Clara's 2013 Solar Decathlon Radiant House uses the structural engineering as a crucial element in showing new methods of sustainable development and design looked for in the Solar Decathlon Competition.

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INTRODUCTION

The Structural Design Team of Santa Clara University's 2013 Solar Decathlon "Radiant House' was tasked with developing a structurally sound, sustainable, and innovative structural design for the 2013 Solar House. The Solar Decathlon is a competition which emphasizes sustainable, innovative, and affordable design of residential homes though student led teams and designs. "The Solar Decathlon is a key strategy to achieve an important goal for National Renewable Energy Laboratory (NREL) and the U.S. Department of Energy (DOE), namely, to foster development and facilitate widespread adoption of homes that demonstrate solar and energy efficiency technologies in marketable applications, through technology development and key partnerships. The strategy includes fostering excellence in building science education in universities with the goal of equipping future design and construction professionals with the skills necessary to design and build quality high performance homes that are healthy, safe, durable, and energy efficient." (as per SD2013 RFP)

The DOE, and the SCU Solar Decathlon Team, are interested in developing net-zero sustainable housing due primarily to the extremely detrimental effect current energy usage is having on the planet. There is a worldwide need for a movement to greener living. Humans have caused the mass majority of the current planetary warming, along with many other environmental issues, all of which relate to excessive energy consumption in some way. Industrialization, deforestation, and pollution have increased atmospheric concentrations of water vapor, carbon dioxide, methane, and nitrous oxide, all greenhouse gases that help trap heat near Earth's surface. According to NASA's Goddard Institute for Space Studies, average temperatures have climbed 1.4 degrees Fahrenheit around the world since 1880, much of this occurring in recent decades. A report by the IPCC released in April 2007 warned that global warming could lead to large-scale food and water shortages and have catastrophic effects on wildlife. The frequency and magnitude of natural disasters, such as hurricanes and heat waves, will also increase. Clearly, the effect of inefficient energy usage is having an extremely negative effect on the planet.

"The U.S. Department of Energy Solar Decathlon challenges collegiate teams to design, build, and operate solar-powered houses that are cost-effective, energy-efficient, and attractive. The winner of the competition is the team that best blends affordability, consumer appeal, and design excellence with optimal energy production and maximum efficiency." (DOE Solar Decathlon Home Page) The Solar Decathlon is an international competition focusing on ten contests by which the student designed solar houses are judged by. Through the structural design of the Radiant House, this team focused on certain aspects of the engineering, as well as the architecture, contests within the 10 Solar Decathlon Contests. The major features to be considered therein were designing structurally sound structure, assuring each of its three modules can withstand travel to its competition location, determining appropriate module, foundation, and roof connections, and incorporating innovative features. The ultimate goal of the structural engineering of the Solar House was to design for structural integrity, whilst not overdesigning so as to minimize cost, and maximize sustainability and innovation.

The competition location for the 2013 Department of Energy Solar Decathlon is Orange County Great Park in Irvine, California, on October 3–13, 2013. During this 10 day period hundreds of thousands of house visits will be provided to the public.

"For the student competitors, the project is designed to increase education about energy-efficient home design, and to accelerate home research and development… and to achieve the goal of developing and demonstrating solar and energy efficiency technologies in marketable residential applications." (SD2013 RFP) The Santa Clara University 2013 Radiant House should be a viable representation of the direction in which housing need to progress. The structural aspect of the house was designed to be sustainable while maintaining ease of construction and assembly, and, perhaps most importantly, remaining economical.

FIG. 1 - 2013 SOLAR DECATHLON LOCATION – IRVINE, CA

SUMMARY ALTERNATIVE ANALYSIS

The most fundamental and innovative aspect of the structural design of the Solar House was the material. There are three commonly used building materials: steel, concrete, and timber. Concrete is not a viable option for a project of this scale. Along with steel and timber, we considered bamboo as a third building option. The pros and cons of each are summarized in the table below:

Table 1. Structural Materials Comparison

	Pros	Cons
Steel	o High Strength o Recyclable o Full design information	o Expensive o High CO_2 emissions
Timber	o Cheaper than steel o Full design information	o Not particularly sustainable – does not regenerate quickly
Bamboo	o High strength to weight ratio o Strongest growing woody plant on earth o One of the widest ranging habitats o Grows the fastest: some species grow one and a half meters a day → Sustainable	o Very little structural testing and therefore extremely limited design information (extensive testing required) o Variable material properties an dimensions o Not easily cut into consistent product

From this table, it is easy to reach the conclusion, as the Structural Team did, that bamboo would be an extremely sustainable, and likely very structurally sound material, but it would require research, and therefore time and money, before use. Bamboo has the potential to be analyzed and designed with the same ease as timber once it is a more established structural material in the US. Steel is expensive and, though it is recyclable, the processes involved in recycling steel do not lend themselves well to being coined ‘sustainable’. Timber is a less expensive and greener building alternative than steel, though not so much so as bamboo. Timber has adequate strength and design properties for a project of this scale.

Santa Clara University has spent extensive time testing bamboo's properties over the 2011/2012 and 2012/2013 school years. Therefore, the 2013 Solar Decathlon Radiant House was designed as a timber structure, with the intent of shifting to bamboo when the testing was completed and if the results proved it to be an equivalent or better alternative. The testing showed bamboo as a viable alternative to timber, and therefore Bamboo Shear Walls and Floor/Ceiling Joists were implemented. Bamboo has the potential to be a superior, inexpensive, and sustainable building material in the future – by using it in the SCU Solar House bamboo's potential was showcased as an innovative new direction.

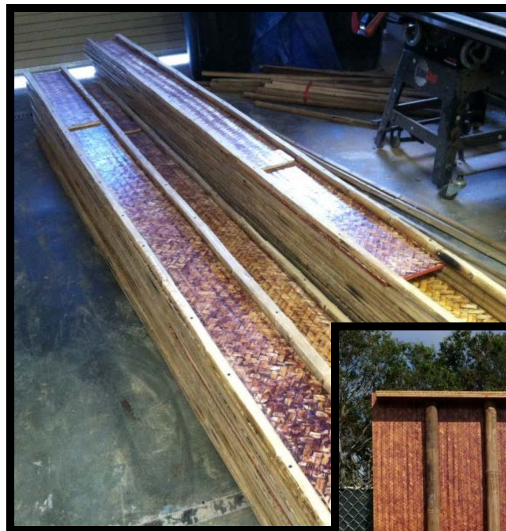

FIG. 2&3 - BAMBOO FLOOR JOISTS AND SHEAR WALLS

In addition to the innovative use of bamboo in the Radiant House, the Structural Team proved innovative in its combinatory approach to the Structural Design. The Radiant House will be initially constructed at Santa Clara University in the spring and summer of 2013. It will then be deconstructed in order to be shipped to the Solar Decathlon Competition Site in Irvine, CA, where it must be reconstructed in a 7 day period

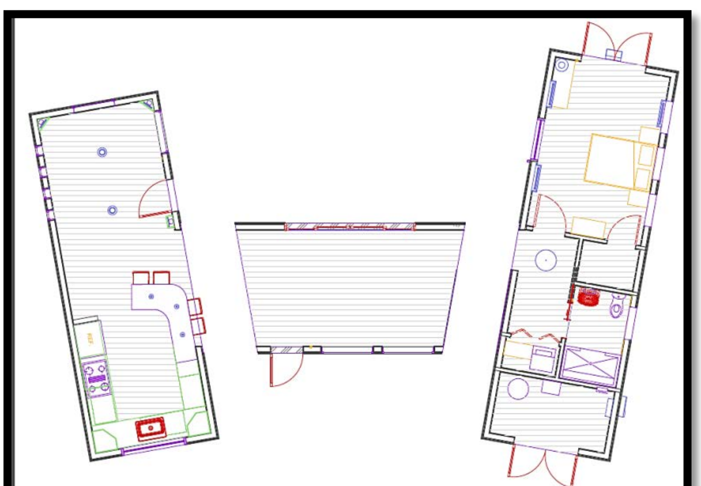

FIG. 4 - 2013 RADIANT HOUSE FLOOR PLAN

before the competition commences on October 3, 2013. At the end of the competition the house must be deconstructed, moved, and reconstructed at least once more in a more permanent location. For this reason it is required at the Radiant House be a modularized structure. The Architectural Team along with the Structural Team determined that a four module system would be most effective: three main modules and a roof module. These modules are to be essentially fully constructed in Santa Clara before shipping.

The Structural Team decided to implement a combined Steel, Bamboo, and Timber structural system in contrast to a Steel Moment Frame system that is the typical industry standard for Modularized Homes today. The Team was committed to using the aforementioned bamboo elements, negating the possibility of using a Steel Moment Frame System, yet due to the modularized design of the house it was determined that some steel elements would be beneficial. The travel and modular assembly aspects of the Radiant House create a need for extra strength and stiffness factors as well as simple connection details in the design –

FIG. 5&6 - 2013 RADIANT HOUSE STEEL ELEMENTS

FIG. 7 - 2013 RADIANT HOUSE MODULE CONNECTION

steel has the ability to achieve this much more effectively than any other material. Without the extra stiffness granted by the steel elements in the design the extra loadings and deflections due to these loadings cases would cause severe damage to the fully finished modular elements. This integrated system makes for an efficient and dynamic structure capable of performing well throughout its stages of mobilization and construction.

"The Solar Decathlon is… a public event designed to increase awareness about energy for residential use. The competition demonstrates that a beautifully and well-designed house can generate enough thermal and electrical energy to meet the needs of a household… A critical long-range outcome of the Solar Decathlon project is the development and demonstration of cost-effective solar-powered homes." (as per SD2013 RFP) The structural engineering of the Solar House plays a strong role in this demonstration. The use of structural bamboo in this design as sustainable and economical option, along with the integrated and dynamic aspect of material usage in the SCU 2013 Radiant House, the structural design is a striking showcase of innovative methods within the residential housing industry.

DESIGN CRITERIA AND STANDARDS

The Structural components to be included in the Structural Design of the Santa Clara 2013 Solar Decathlon Radiant House were required to be in compliance with the Santa Clara Building Code, the Irvine Building Code, and the Solar Decathlon 2013 Rules. Loadings considered were: dead load, live load, wind load, seismic load, and roof load. Bamboo components will be used in place of soft wood where they are demonstrated to comply with the acceptance criteria developed by other Senior Design groups. The following are the codes and standards which the Structural Team followed and utilized.

- → 2009 IBC
- → 2007 NDS
- → AISC Steel Construction Manual
- → ASCE 705
- → DOE Solar Decathlon 2013 Design Requirements

DESCRIPTION OF THE DESIGNED FACILITY

Radiant House was designed using SCU's 2009 Solar Decathlon Refract House Structural Design as a foundation. The design for Refract House included a structural narrative, structural drawings, and addressed the following features: Material Properties, Gravity Loads, Seismic Loads, Wind Loads, Joists and Beams, Columns, Diaphragms, Shear Walls, Moment Frame, Cantilever Column, Foundations, Grade Variability, Deck, Solar Racking, and Overturning Moment.

Using those subjects as a basis, steel moment frames were removed from Radiant House's design due to Professor Aschheim's request that the house be designed solely using shear walls as the primary lateral load resistance. This is because the shear walls being placed in the house were supposed to be bamboo shear walls for innovative purposes. Afterward, it was also determined that the cantilever column was not needed either because that was part of a feature exclusive to Refract House.

After determining what was not needed for Radiant House Structural Design, what was needed was decided. Adding to the list above, the additional features that had to be addressed were: steel frame, connections, carport, awnings, and their respective drawings.

All weights and loads are shown in Appendix A.

Modules

Radiant House is designed into 4 different modules. If looking at a planar view of the house in Appendix B Sheet S-102, 3 modules are distinguishable. The modules are labeled A, B, and C from left to right when the house is facing North. When looking at a side or isotropic view, the fourth module is discernible. This module, which can be seen in Appendix B sheets S-206 and S-301, is the top module which runs across the middle of the other three modules. Modules A and

C are rectangular. Module B is trapezoidal when looking at its planar view but when looking at its side or isotropic view, the module only has two window-covered walls. The top module is essentially a pentahedron without its bottom side.

Gravity Loads

(Refer to Appendix A - page 24)

This was one of the most important features of the house design to address. Knowing the materials and their weights is important because almost of the other features' calculations depended upon house weight. This was also one of the most difficult aspects to figure out. The materials used inside of the house, such as HVAC and exterior finish, were supposed to be chosen by different subgroups within the SCU Solar Decathlon team. Even though the other groups were supposed to choose the materials, they still had not decided what they wanted by the time the structural design had to be started. Using materials the other teams believed would be used inside the house, the house's weights were decided.

In order to simplify the weight, the house was separated into components; flat roof, sloped roof, floors, exterior walls, and interior walls. This allowed for the module weights to be calculated on a linear foot basis.

Seismic Loads

(Refer to Appendix A – page 26 & Appendix B – sheet S-650)

In order to design for seismic loads, the seismic design base shear had to be calculated for Irvine, CA and for Santa Clara, CA. This had to be done in order to calculate for a worst case scenario seismic load. First, the mapped spectral accelerations were determined using the USGS Java Ground Motion Parameter Calculator. Secondly, the occupancy category, II, was given by the Department of Energy. After that, the calculation in the appendix was followed through using the 2010 California Building Code and Seismic Building Codes. In order to be conservative, the response modification coefficient, R, was chosen to be 6.5 for light frame walls with panels for shear resistance. The building period coefficient based on structural system, Ct, was chosen as 0.02 for ‘other structural system’ and its corresponding exponent for fundamental period of vibration calculation, x, was 0.75. Following through with the calculation, the earthquake load, E, for both Irvine and Santa Clara was determined to be 0.2 times the weight of the house.

Using the earthquake load and the total weight of each individual module, the seismic force distribution was obtained. The force was determined in the North to South direction and in the East to West direction. It was obtained for ground level and for roof level.

Wind Load

(Refer to Appendix A – page 30 & Appendix B – sheet S-650)

Using ASCE 7-10, it was determined that the wind conditions in Irvine and in Santa Clara were the same. The basic wind speed as set by the Department of Energy was 85 miles per hour. The enclosure classification is enclosed because the home is an enclosed structure. All other factor values were determined using their respective charts within the ASCE 7-10 book.

Using the values obtained, the velocity pressure, q_z came out to be 14.15. The second velocity pressure, q_h came out to be 13.36. Using these values, the design wind load came out to 12.027 pounds per square foot. Using this value, the shear based on wind loads per each module was determined for the North to South and for the East to West directions.

After comparing the values obtained in each direction for wind and for seismic, the worst case shear in each direction was chosen. For Modules A and C, seismic loads governed in the North to South direction and wind loads governed in the East to West direction. For Module B, seismic loads governed in the East to West direction and wind loads governed in the North to South direction.

Shear Walls

(Refer to Appendix A – page 31 & Appendix B – sheet S-104)

The shear walls in the house were designed using the governing lateral loads. The shear in each direction was distributed by tributary area into the wall closest to each area. Next, the amount of shear walls in each wall was determined based on the amount of space available in each wall to place in a shear wall. The shear force was then distributed into each shear wall to design the strength per shear wall. The plywood and edge staple or nail spacing were chosen based on the shear wall table allocated in the 2009 Wind and Seismic Design Book. They were chosen based on allowable strength.

Following the basic strength design of each shear wall, a few more checks were required. The first check was making sure that the shear wall had a proper holdown. The strength for each holdown was verified using the Simpson Tie-Down Catalog. The next check was overturning to make sure the shear wall would not turn over. The following check was a mudsill to rim joist connection check to make sure that the force from the lateral load would not cause the connection between the top of the house and the shear wall to break or disconnect. This was also the reason why there was a check to see how many clips were required to connect to the top plate.

Diaphragm

(Refer to Appendix A – page 38 & Appendix B – sheets S-103, S-105, S-106)

The floor diaphragm was designed for Modules A, B, and C. The flat roof diaphragm was designed for Modules A and C. Over Module B and the middle of Modules A and C, a sloped diaphragm was designed.

An innovation within Radiant House is shown through the floor diaphragm. The floor diaphragms were designed to transfer the lateral loads to the shear walls and the frame of Radiant

House. The diaphragm is attached to the rim joist that is attached to inside web of the steel channel. This is different because diaphragms are not typically located within the steel frame, but instead above it. Most of the shear walls are attached above the C15x33.9 and the one that separates the mechanical room from the house is above an HSS8x3x3/8.

The roof diaphragms are attached to the top plates of the walls. They also transfer the lateral loads to the shear walls and the frame of Radiant House.

Foundations

(Refer to Appendix A – page 46 & Appendix B – sheets S101, S-501, S-502)

The foundation of the Radiant House was designed using Standard Piers and Seismic Piers from Central Piers Incorporated. Using the weight of each module, the total bearing area needed was calculated. To be more conservative, an overturning calculation was done to determine the number of square feet for footings per side. Using the required amount of area for footings, the number of seismic piers and standard piers were chosen.

Grade Variability

(Refer to Appendix A – page 50 & Appendix B – sheet 601)

The Department of Energy required that the house design have structural calculations that account for the grade variability on the runway where the house will be on display. In order to avoid calculations, piers were chosen that can be adjusted by 2” each. The piers also range in size from 4” up to 36” high. Using the basic grade variability and using profiles to figure out the height of the house at a location on the lot, a pier that would account for the height differences was chosen.

Steel

(Refer to Appendix A – page 56 & Appendix B – SOS Steel Co. sheets)

The loads on the house were small enough that the house could have been designed solely out of timber. Although that is the case, the house was designed using a steel frame. A steel frame was designed to make connecting the modules easier for the group of students who will be assembling, disassembling, and reassembling the house. The steel frame was also used for transportation and transferring loads.

W5x19 **(Refer to Appendix A – page 60 & Appendix B – Sheet D4-D5.1)**

These 6 pieces of steel were originally designed in a lighter size in order to collect the gravity loads from the top module and transfer the loads down to the steel columns that hold them up. The steel size was also chosen because there was a 6 inch width limit in the location that the steel was being placed in. When designing for shear in the house it was discovered that a collector was needed to transfer later loads in the East to West direction. Although the original size and the W5x19 can both transfer the seismic loads throughout the house to the shear wall, the original size did not enable for proper bolting connections. It was because of that that the W5x19 was chosen.

HSS 3.5x3.5x 5/16 **(Refer to Appendix A – page 59 & Appendix B – Sheet D1-D3)**

This size steel was chosen for the columns that hold up the W5x19 collector steel. Steel was chosen as the element to hold the W5x19 up, instead of timber, due to the ease of connections. At first the steel columns were designed to be the element that enabled for inter-modular connections. When designing the steel with the steel company though, it was decided not to use that as the inter-modular connection because it would still be a difficult connection. The steel still remained as the primary gravity catcher of the top module though. A smaller size steel tube would have been able to take the small loads from the top module, but in order to limit eccentric loads and still enable stiffness, the HSS 3.5x3.5x5/16 was chosen for the column design.

HSS 8x3x3/8 **(Refer to Appendix A – page 58 & Appendix B – sheet D6-D8)**

Transportation, insufficient testing on the bamboo joists, and connection to the C15x33.9 led to the decision of having steel beams located every 8 feet within the floor of the house.

Transportation

The trucks being used to transport each module of the house down to Irvine are being donated by an SCU alumnus with a transportation company. The beds of the trucks are only 8 feet wide, which was a conflict with the 12 foot wide base of Radiant House. If placed upon the bed of the truck, the sides of the house would be unsupported and this may cause the house to come apart. In order to support the house, additional steel would need to be placed on the beds of the trucks and then the modules would be placed above it, or steel would need to already be implemented as part of the base of the house. The price of the steel would be a part of the house regardless of whether it was placed inside of the house or not so it was decided to make the house stiffer, which is safer, and add steel to the bottom of the house.

Insufficient Testing

The bamboo joists tested for the house were tested for gravity loads but not for shear loads. This meant that nailing into the bamboo joists for the diaphragm could actually be dangerous. Each diaphragm plywood board is 4'x8' and would need to be nailed into a joist on the 8' side. In order to make the diaphragm load safe, the HSS was designed so that 2 - 2x4's would be bolted onto the top of the HSS steel for the diaphragm to be nailed into.

Ease of connection to C15x33.9

The last step in designing the steel was deciding which shape of steel to use as the support. The size was narrowed down to a W, C, or HSS member. Each had a flat top so that the 2 – 2x4's could be placed above it. Each shape was available in the height limits that were needed in order for the top of the 2-2x4's to align properly with the top of the bamboo joists. Each shape was also able to take the loads placed upon it during transportation. Because steel is priced on a linear foot weight basis, the next step was narrowing down the shape by weight. The most promising shape was the HSS but due to the fact that the ends of the beam would need to be welded to the C15x33.9, it still was not clear which shape would be used. After meeting with S.O.S. Steel Company, their manager said it would not be a difficult connection to weld, so HSS was chosen as the shape.

C15x33.9 **(Refer to Appendix A – page 56 & Appendix B – sheet D6-D8)**

A C15x33.9 was chosen as the base frame main component due to its shape, height, and strength.

Shape

Going with a large HSS shape steel member seemed safest when trying to decide on a shape. The shape would not deflect under torsional loading. Although that is the case, it seemed too difficult to attach the floor joist hangers to the HSS member. This led to the decision to use a W-shape or a C-shape so that a rim joist could be placed inside of it in order to attach the floor joist hangers onto it. A W-shape seemed safest because a channel may bend inwardly and a W-shape would be able to resist loads better. Although a w-shape seemed like a better shape to use, a W-shape didn't allow for a thick rim joist to be attached to its web. The channel also had a flat backside, which enabled for easier inter-modular connections. With a W-shape, an inter-modular connection utilizing the W's at the base of two modules would have proven too difficult or nearly impossible.

Height

The Santa Clara University team wanted to have a standard interior ceiling height. With basic construction it would have been a difficult feature to design for. By using a taller steel size at the base to place the walls of the house above, it was easier to get close to this interior height.

Strength

Using Visual Analysis, it was determined that a W12x19 could be used. This meant that we would have been able to use a much smaller size that would be much cheaper. Once we changed it to a channel, it was established that using a C15x33.9 would be sufficient to carry the strength. The challenge with using the C15x33.9 was that the price would be nearly twice the price of the W12x19. After bringing up the dilemma with the SCU Solar Team, the team decided that paying more for a higher ceiling height was worth the money.

C8x11.5 **(Refer to Appendix A – page 57 & Appendix B – sheet D7)**

The C8x11.5 pieces of steel are used at the base ends of Module B. A channel was chosen so that the backside of the channel could connect easily to the backside of the channel at the base of modules A and C. It was also chosen in order to be able to take the transportation loads. Due to the fact that module B is not as heavy as modules A and C, the channel at its base did not have to be as heavy as the C15x33.9 that is at the ends of the other modules.

L8x4x1/2 **(Refer to Appendix A – page 70 & Appendix B – sheet D10)**

The Top module is 42 feet long and has to be lifted into place after the other three modules are set down. This means that the top module had to have a steel frame as well because there is not a piece of timber 42 feet long that would be thin enough to conceal within the walls of the house. Because the loads are so light on the top module, a large size of steel was not needed. The reason that an angle was chosen for the top module frame was because there had to be a way to connect the W5x19 to the top module. For an easier connection, we chose to bolt through the angle and into the flange of the W5x19. The L8x4x1/4 also had to be strong enough so that the transportation company could attach D-rings to pick points in order to lift it without the module deflecting too much.

C3x3.5 **(Refer to Appendix A – page 74 & Appendix B – sheet D9)**

In order to make the top module stiffer for transportation, steel was added in the areas between where the windows in the module are located. This will enable the top module to deflect less and enable for more security in its strength and stiffness. The steel was not placed in there to take a particular amount of load. It will take some of the roof load though. It will also be used to attach the timber infill to the L8x4x1/2 steel.

L2x2x1/4 **(Refer to Appendix A – page 67 & Appendix B – sheet D10)**

Temporary bracing needed to be added to the modules for transportation. The module that needed temporary bracing was Module B. When it stands alone, this module only has a floor and two walls. These two walls also contain vast amount of windows and glass doors. The module would be unstable during transportation and the windows inside the module would probably break if no bracing was designed. Due to the fact that the actual loads during transportation are unknown, the design load was chosen as 20 kips tension and compression to be conservative. To be conservative, temporary bracing was also designed for Modules A, B, and the top module.

COST ESTIMATE

Item	Budgeted Cost	Actual Cost
Lumber	$20000.00	$26808.29
Standard Piers	$3444.00	$1650.00
Seismic Piers	$2000.00	$1908.00
Steel	$35000.00	$44841.40
Deck	$7000.00	$10811.91
Bamboo		

Estimated Budget versus Actual Cost

When estimating the cost of the house, labor was not adjusted into the budget. The actual cost shown is the cost that the Department of Energy has estimated for certain items within the structure of the house and this price includes labor. For a seismic pier for example, we bought 12 seismic piers and each one costs $105 so the total price should be $1260 but because the piers need to be installed, the Department of Energy added $648 for labor to the cost.

For the deck, the original budgeted cost was estimated for a different house and deck layout. The deck was not as large as the current deck which is roughly 1500 sq. ft., which is larger in area than the 1000 sq. ft. house. The price of the deck also includes decking material such as railing and the bamboo decking that were chosen by the architect of the team. This material adds to the structural cost of the deck. The standard piers for the deck were also included in the cost of the deck instead of under the cost of the standard piers, which is why the actual cost of the standard piers is much lower than the budgeted cost of standard piers.

CONCLUSION

The ultimate objective of the Department of Energy Solar Decathlon competition, and the Santa Clara University Solar Decathlon Team is to educate students and the public about the money-saving opportunities and environmental benefits presented by clean-energy products and design solutions, and to demonstrate the comfort and affordability of homes that combine energy-efficient construction and appliances with renewable energy systems available today. Santa Clara's 2013 house is using the structural engineering as a crucial element in showing new methods of sustainable development and design. By primarily using bamboo, creating a dynamic and integrated design, and using sustainable products where possible throughout the structure, the structural engineering of the Solar House has the ability to showcase sustainable design options.

APPENDIX A – Structural Calculations

APPENDIX B – As-built Construction Drawings and Design Renderings

Radiant House

Project Manual

US Department of Energy

Solar Decathlon 2013

Santa Clara University

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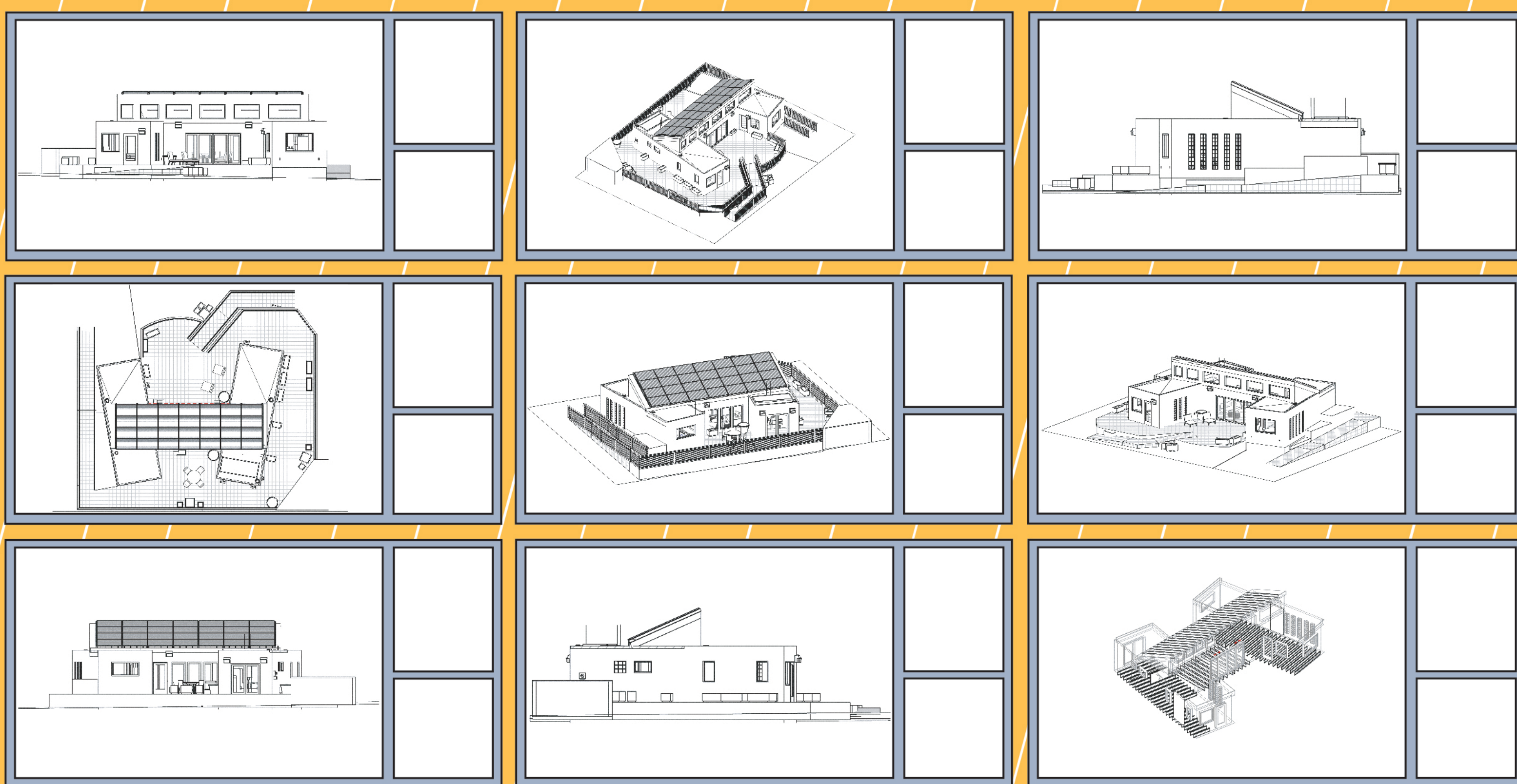


Rules Compliance

Rules Compliance Checklist

Rule #	Rule Name	Content Description	Location
4-2	Construction Equipment	Drawing(s) showing the assembly and disassembly sequences and the movement of heavy machinery on the competition site	O-101
4-2	Construction Equipment	Specifications for heavy machinery	41 20 00 41 23 23 41 62 23
4-3	Ground Penetration	Drawing(s) showing the locations and depths of all ground penetrations on the competition site	S-101
4-4	Impact within the Solar Envelope	Drawing(s) showing the location, contact area, and bearing pressure of every component resting directly within the solar envelope	S-108
4-5	Generators	Specifications for generators (including sound rating)	26 32 13
4-6	Spill Containment	Drawing(s) showing the locations of all equipment, containers, and pipes that will contain liquids at any point during the event	P-102 P-103 P-104 P-901 P-902 M-102 M-103 H-101 FP-01
4-6	Spill Containment	Specifications for all equipment, containers, and pipes that will contain liquids at any point during the event	21 10 00 22 05 00 22 11 13 22 11 16 22 11 23 22 12 00 22 13 00 22 13 53 22 41 16 22 41 23 22 80 08 23 23 23 23 83 16 23 83 33
4-7	Lot Conditions	Calculations showing that the structural design remains compliant even if 18 in. (30.48 cm) of vertical elevation change exists	Pg. 50
4-7	Lot Conditions	Drawing(s) showing shimming methods and materials to be used if 18 in. (30.48 cm) of vertical elevation change exists on the lot	S-501
5-2	Solar Envelope Dimensions	Drawing(s) showing the location of all house and site components relative to the solar envelope	G-201 G-202

5-2	Solar Envelope Dimensions	List of solar envelope exemption requests accompanied by justifications and drawing references	N/A
6-1	Structural Design Approval	List of, or marking on, all drawing and project manual sheets that have been or will be stamped by the qualified, licensed design professional in the stamped structural submission; the stamped submission shall consist entirely of sheets that also appear in the drawings and project manual	Pg. 15
6-2	Finished Square Footage	Drawing(s) showing all information needed by the rules officials to measure the finished square footage electronically	G-101
6-2	Finished Square Footage	Drawing(s) showing all movable components that may increase the finished square footage if operated during contest week	N/A
6-3	Entrance and Exit Routes	Drawing(s) showing the accessible public tour route	G-103
7-1	Placement	Drawing(s) showing the location of all vegetation and, if applicable, the movement of vegetation designed as part of an integrated mobile system	L-101
7-2	Watering Restrictions	Drawing(s) showing the layout and operation of greywater irrigation systems	N/A
8-1	PV Technology Limitations	Specifications for photovoltaic components	26 31 00 48 19 16
8-3	Batteries	Drawing(s) showing the location(s) and quantity of all primary and secondary batteries and stand-alone, PV-powered devices	F-101 E-105 T-102
8-3	Batteries	Specifications for all primary and secondary batteries and stand-alone, PV-powered devices	26 33 13
8-4	Desiccant Systems	Drawing(s) describing the operation of the desiccant system	N/A
8-4	Desiccant Systems	Specifications for desiccant system components	N/A
8-5	Village Grid	Completed interconnection application form	Pg. 115
8-5	Village Grid	Drawing(s) showing the locations of the photovoltaics, inverter(s), terminal box, meter housing, service equipment, and grounding means	E-103 E-201 E-401

8-5	Village Grid	Specifications for the photovoltaics, inverter(s), terminal box, meter housing, service equipment, and grounding means	26 05 19 26 05 26 26 05 33 26 05 83 26 24 16 26 27 26 26 28 16 26 31 00 48 19 16
8-5	Village Grid	One-line electrical diagram	E-611
8-5	Village Grid	Calculation of service/feeder net computed load per NEC 220	E-601
8-5	Village Grid	Site plan showing the house, decks, ramps, tour paths, and terminal box	G-103
8-5	Village Grid	Elevation(s) showing the meter housing, main utility disconnect, and other service equipment	E-201
9-1	Container Locations	Drawing(s) showing the location of all liquid containers relative to the finished square footage	P-102 P-103 P-104
9-1	Container Locations	Drawing(s) demonstrating that the primary supply water tank(s) is fully shaded from direct solar radiation between 9 a.m. and 5 p.m. PDT or between 8 a.m. and 4 p.m. solar time on October 1	P-103 P-901
9-2	Team-Provided Liquids	Quantity, characteristics, and delivery date(s) of all team-provided liquids for irrigation, thermal mass, hydronic system pressure testing, and thermodynamic system operation	N/A
9-3	Greywater Reuse	Drawing(s) showing the layout and operation of greywater reuse systems	N/A
9-4	Rainwater Collection	Drawing(s) showing the layout and operation of rainwater collection systems	N/A
9-6	Thermal Mass	Drawing(s) showing the locations of liquid-based thermal mass systems	H-101
9-6	Thermal Mass	Specifications for components of liquid-based thermal mass systems	N/A
9-7	Greywater Heat Recovery	Drawing(s) showing the layout and operation of greywater heat recovery systems	N/A
9-8	Water Delivery	Drawing(s) showing the complete sequence of water delivery and distribution events	P-101
9-8	Water Delivery	Specifications for the containers to which water will be delivered	22 12 00
9-9	Water Removal	Drawing(s) showing the complete sequence of water consolidation and removal events	P-101
9-9	Water Removal	Specifications for the containers from which water will be removed	22 12 00 22 13 53

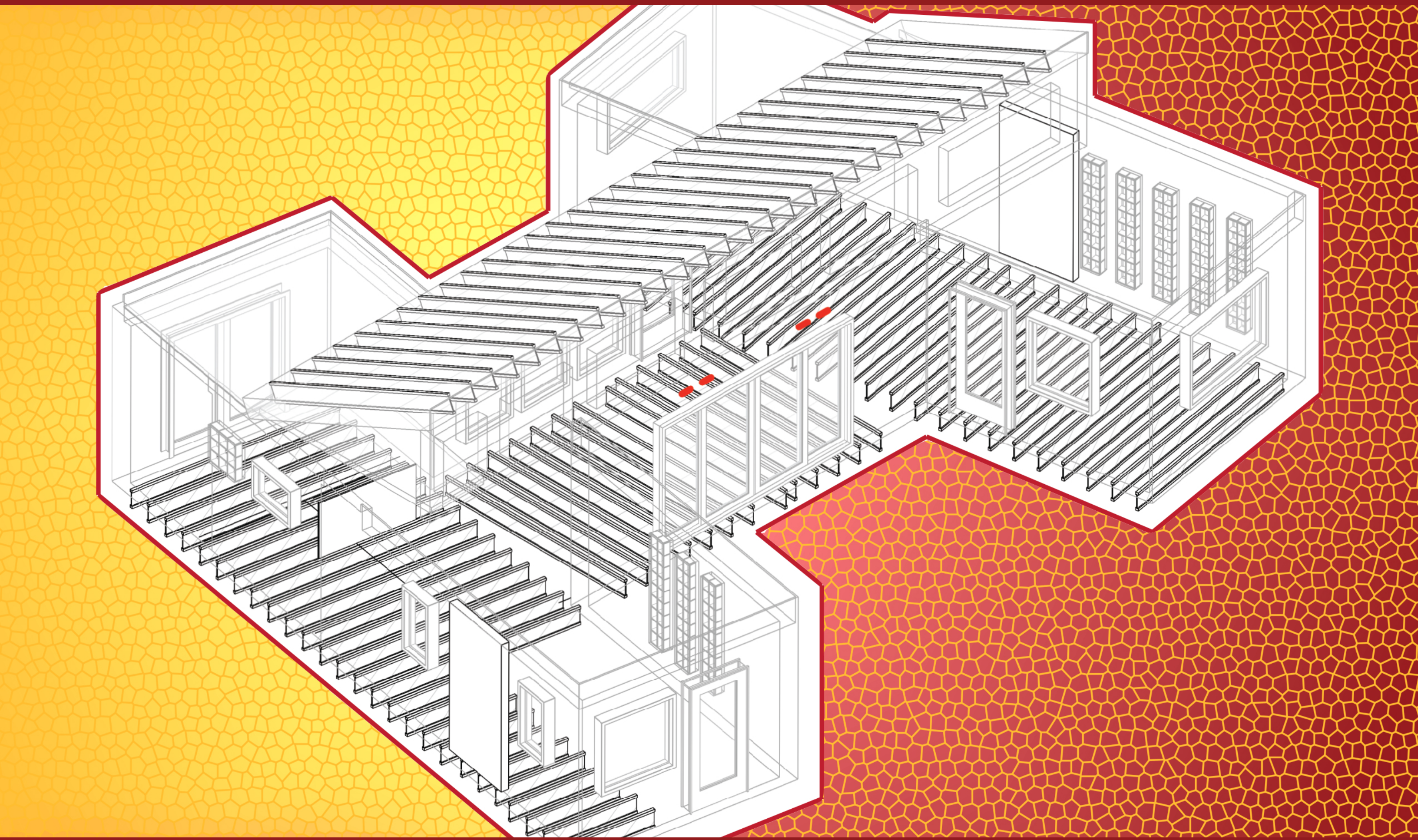
11-4	Public Exhibit	Interior and exterior plans showing entire accessible tour route	G-103 X-101

6-1 Structural Design Approval

The following is a list of the sheets and pages that are stamped in the hard copy by Dr. Mark Aschheim, PE. The sheets and pages can be found in the Construction Drawings and Project Manual at the locations listed below, or in a separate, but attached Stamped Drawing Set and Stamped Structural Calculations.

- Structural Calculations, pages 19-95 of the Project Manual
- Construction drawings, including:
  - S-001 Structural Notes and Symbols
  - S-101 Foundation Plan
  - S-102 Module Separation
  - S-103 Floor Framing Plan
  - S-104 Wall Framing Plan
  - S-105 Lower Roof Framing
  - S-106 Upper Roof Module Framing Plan
  - S-107 Deck Framing Plan
  - S-108 Bearing Pressure Plan
  - S-109 South Awning Plan
  - S-201 Module A Wall Framing
  - S-202 Module A Wall Framing
  - S-203 Module B Wall Framing
  - S-204 Module C Wall Framing
  - S-205 Module C Wall Framing
  - S-206 Roof Elevation
  - S-303 Roof Module Section
  - S-311 Deck Section
  - S-401 Awning Large Scale Plan
  - S-501 Pier Details
  - S-502 Pier Plate Details
  - S-512 Bamboo Gravity Wall Details
  - S-521 I-Joist Details
  - S-531 Connection Details
  - S-532 Timber to Steel Connections
  - S-541 Railing Details
  - S-551 Awning Details
  - S-601 Schedules
- S.O.S. Steel Co. Shop drawings, including:
  - D1
  - D2
  - D3
  - D4
  - D4.1
  - D5
  - D5.1

D6
D7
D8
D9
D10
D11
E1
E1.1
E2
E3
E4



Structural Calculations

STRUCTURAL CALCULATIONS

TABLE OF CONTENTS

STRUCTURAL NARRATIVE

The 2013 Radiant House is the realization of Santa Clara's goal to design a strong, efficient, and innovative symbol of sustainability. Utilizing a conventional system of joists, beams, studs, and shear walls, we implemented an innovative and original design to resist and transfer dead, live, wind, and seismic loads to the foundation.

The approach of our team is to substitute an initial conventional softwood design with one that makes use of a bamboo structural system. Research conducted at Santa Clara University has yielded results affirming bamboo's suitability as a substitute for bamboo joists and gravity walls. The bamboo will be incorporated into the design as a primary structural component of Radiant House.

Radiant House will be composed of three base modules and an additional roof module. A perimeter structural steel channel or angle will serve as the backbone to each of these modules, serving to collect all gravity and lateral loads and distribute them into the foundation, which will consist of a series of seismic and standard piers. Roof gravity loads will be collected by bamboo joists and transferred through bamboo gravity walls into the steel channel. While lateral loads at the roof level will transfer through the roof diaphragm into the shear walls and into the perimeter steel channel. The floor joists will be hung such that a minimum clearance of 1- 9/16 inch will be maintained above the bottom of the steel channel. Lateral loads will be collected by floor diaphragms and transferred into the steel perimeter channel.

The sloped roof module consists of a structural steel angle base that transmits roof diagram loads into the top plates of the base modules. The roof angle is 24 degrees and is supported on the low side by the above mentioned structural steel angle and a vierendeel truss at the high end of the slope. The vierendeel truss is created by vertical structural steel channels running between the base angle and a top horizontal steel angle. The vertical steel channels are placed at strategic locations to carry and transfer shear loading due to wind and seismic in the East-West directions and gravity loads, while still allowing windows for natural lighting. The roof slopes from the top steel at the north wall to the base steel on the south side in order to create a slope optimal for solar collection.

As Radiant House will be transported by truck from Santa Clara to Irvine, our greatest concern is maintaining the structural integrity of the completed house during transportation. The significant wind loads expected to act on the house's modules during transportation will require thorough consideration during both the design and transportation phases in order to preserve structural stability. For example, steel beams will be placed parallel to the floor joists and flush with the bottom of the perimeter channel to support and transfer the gravity load to the truck bed during transportation.

Santa Clara's 2013 Radiant House demonstrates the importance of structural engineering's role in sustainable development and design. With its frugal and efficient design and its pioneering use of sustainable products, the structural engineering of Radiant House reaffirms Santa Clara's enduring commitment to sustainability.

BAMBOO STRUCTURAL SYSTEMS NARRATIVE

Santa Clara seeks to implement a bamboo structural system in our 2013 Solar Decathlon house. We will provide a conventional softwood design and then, where sufficient capacity is demonstrated, will propose substitution of the following bamboo structural components:

- *Bamboo gravity walls*: Wall panels will be prefabricated, with hollow section bamboo culms at 16 inches on center, mounted to 4 ft. by 8 ft. woven bamboo panels. The assembly will have an integral bottom plate positioned to allow the wall panel be dropped in place on top of a field-installed bottom plate. Similarly, the lower of two top plates will be preinstalled in the panel assembly. Once the wall panel is in place, the upper of the two top plates will be field installed as a means to integrate the wall panels into the structural system. See Figure 18.
- *Bamboo joists*: Representing our third innovation in this category, our I-shaped bamboo joists consist of a woven bamboo sheet product used as a web, with solid section bamboo culms attached to the web to form flanges. See Figure 17.

These components are being developed in our laboratory in collaboration with a bamboo fabricator in Vietnam. Acceptance criteria for each component has been approved and quality assurance/quality control protocols for each component has also been approved to ensure the bamboo components used in construction adhere to design expectations.

- *Bamboo stud walls*: Stud walls were subjected to axial compression testing to establish design values and determine behavior under loading.
- *Bamboo joists*: Shear and bending tests are underway to establish design values.

Results obtained through testing have been compared to behavior displayed by conventional softwood components and to required demand loads to determine the suitability of bamboo as a substitute for components in Radiant House's structural system. Santa Clara hopes to provide innovation in Radiant House with this bamboo structural system.

See S-512 and S-521 for relevant drawings.

MATERIAL PROPERTIES

Steel

Channels	f_y = 36 ksi
Angles	f_y = 36 ksi
HSS beams	f_y = 46 ksi
W beams	f_y = 36 ksi

Connections

Welds	E70XX
High Strength Bolts	A490
Machine Bolts	A307

Wood Framing

Sawn Lumber

Horizontal Framing:			
	2x6	D.F.	No.2
	4x6	D.F.	No.2
	6x6	D.F.	No.1
	2x12	D.F.	No.1
	4x12	D.F.	No.1
Vertical Framing:			
	2x studs	D.F.	No.2
	4x posts	D.F.	No.2
Mudsills & Ledgers:		D.F.	No.2

GRAVITY LOADS

Flat Roof over the Mech Room			
Roofing	TPO Roofing	1.5	psf
Sheathing	3/4" T&G Plywood	2.2	psf
Framing	TJI 230 Joists @ 24" O.C.	2.1	psf
Insulation	ICYNENE MD-R-200	1.5	psf
Solar Thermal	Free Hot Water	3.3	psf
MEP	Messana Panels	2.8	psf
Parapet		1.7	psf
Misc.		0.9	psf
Dead Load		**16.0**	**psf**
Live Load		**20**	**psf**

Flat Roof Not over the Mech Room			
Roofing	TPO Roofing	1.5	psf
Sheathing	3/4" T&G Plywood	2.2	psf
Framing	TJI 230 Joists @ 16" O.C.	2.1	psf
Insulation	ICYNENE MD-R-200	1.1	psf
MEP	Messana Panels	2.8	psf
Parapet		1.7	psf
Misc.		0.6	psf
Dead Load		**12.0**	**psf**
Live Load		**20**	**psf**

Slope Roof			
PV Panels	Bosch Solar Module c-Si M 60	3.0	psf
Racking System	Sunplanter	1.0	psf
Roofing	TPO Roofing	1.5	psf
Sheathing	3/4" T&G Plywood	2.2	psf
Framing	TJI 230 Joists @ 16" O.C.	2.1	psf
Insulation	ICYNENE MD-R-200	1.5	psf
MEP	Messana Panels	2.8	psf
Misc.		0.9	psf
Dead Load		**15.0**	**psf**
Live Load		**20**	**psf**

Floors			
Finish	Tate ConCore 1500 Posilock	8.8	psf
Sheathing	3/4" T&G Plywood	2.2	psf
Framing	TJI 230 Joists @ 16" O.C.	2.1	psf
Insulation	ICYNENE MD-R-200	1.1	psf
MEP	Conduit, Ducting, and Piping	1	psf
Misc		0.8	psf
Dead Load		**16.0**	**psf**

Live Load		**50**	**psf**

Exterior Wall			
Exterior Finish	Siding	4	psf
Sheathing	15/32" Str1 Plywood	1.7	psf
Studs	2 x 4 @ 16" staggered	2.7	psf
Interior Finish	5/8" EcoRock Gyp	2.5	psf
Insulation	ICYNENE MD-R-200	1.1	psf
Glazing	Windows and Mullion	2	psf
Misc.		1	psf
Dead Load		**15**	**psf**

Exterior Side Wall Of Top Module			
Exterior Finish	Siding	4	psf
Sheathing	15/32" Str1 Plywood	1.7	psf
Studs	2 x 4 @ 16" staggered	2.7	psf
Interior Finish	5/8" EcoRock Gyp	2.5	psf
Insulation	ICYNENE MD-R-200	1.1	psf
Misc.		1	psf
Dead Load		**13**	**psf**

Interior Wall			
Finish	(2) 5/8" EcoRock Gyp	5	psf
Framing	2 x 4 @ 16" single stud	1	psf
Misc.		2	psf
Dead Load		**8**	**psf**

Other Loads			
Steel Foundation Track		33.9	plf
Mechanical Equipment		2000	lb

SEISMIC DESIGN BASE SHEAR

Santa Clara seismic conditions govern over Irvine

Zip Code = 95053
Spectral Response Accelerations S_s and S_1
S_s and S_1 = Mapped Spectral Acceleration Values
Site Class B - F_a = 1.0, F_v = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Centroid Sa (g)		
0.2	1.500	(S_s)	, Site Class D
1.0	0.600	(S_1)	, Site Class D
Period (sec)	**Maximum Sa (g)**		
0.2	1.500	(S_s)	, Site Class D
1.0	0.600	(S_1)	, Site Class D
Period (sec)	**Minimum Sa (g)**		
0.2	1.500	(S_s)	, Site Class D
1.0	0.600	(S_1)	, Site Class D

Occupancy Category			=	II		(ASCE/SEI 7-10, Table 1.5-1)
I			=	1		
h_x			=	18	ft	
R			=	6.5		Light Framed Walls w/ Panels For Shear Resistance
C_t			=	0.02		
X			=	0.75		
T_a	=	$C_t * (h)^x$	=	0.175		
S_s			=	1.5	g	
S_1			=	0.6	g < 0.6g	
F_a			=	1		
F_v			=	1.5		
S_{MS}	=	$F_a * S_s$	=	1.5	g	
S_{M1}	=	$2 * S_{ms}/3$	=	0.9	g	
S_{DS}	=	$F_v * S_1$	=	1	g > 0.50	Seis. Design Cat.: D
S_{D1}	=	$2 * S_{m1}/3$	=	0.6	g	
C_s	=	$S_{ds}/ (R/I)$	=	0.154		(ASCE/SEI 7-10, Eq 12.8-2)
C_s max	=	$S_{d1}/T_a(R/I)$	=	0.528		(ASCE/SEI 7-10, Eq 12.8-3)
C_s min	=	$.044\ S_{DS} * I$	=	0.044		(ASCE/SEI 7-10, Eq 12.8-5)
C_s			=	0.154		(Controls)
V	=	$C_s * W$	=	0.154	W	
ρ			=	1.300		
E	=	ρ * V	=	0.200	W	

SEISMIC DESIGN BASE SHEAR

Santa Clara seismic conditions govern over Irvine

Zip Code = 92618

Spectral Response Accelerations S_s and S_1

S_s and S_1 = Mapped Spectral Acceleration Values

Site Class B - F_a = 1.0, F_v = 1.0

Data are based on a 0.01 deg grid spacing

Period (sec)	Centroid Sa (g)		
0.2	1.470	(S_s)	, Site Class D
1.0	0.520	(S_1)	, Site Class D
Period (sec)	**Maximum Sa (g)**		
0.2	1.500	(S_s)	, Site Class D
1.0	0.546	(S_1)	, Site Class D
Period (sec)	**Minimum Sa (g)**		
0.2	1.401	(S_s)	, Site Class D
1.0	0.498	(S_1)	, Site Class D

Occupancy Category			=	II		(ASCE/SEI 7-10, Table 1.5-1)
I			=	1		
h_x			=	18	ft	
R			=	6.5		Light Framed Walls w/ Panels For Shear Resistance
C_t			=	0.02		
x			=	0.75		
T_a	=	$C_t * (h)^x$	=	0.175		
S_s			=	1.5	g	
S_1			=	0.546	g < 0.6g	
F_a			=	1		
F_v			=	1.5		
S_{MS}	=	$F_a * S_s$	=	1.5	g	
S_{M1}	=	$2 * S_{ms}/3$	=	0.819	g	
S_{DS}	=	$F_v * S_1$	=	1	g > 0.50	Seis. Design Cat.: D
S_{D1}	=	$2 * S_{m1}/3$	=	0.546	g	
C_s	=	$S_{ds}/ (R/I)$	=	0.154		(ASCE/SEI 7-10, Eq 12.8-2)
C_s max	=	$S_{d1}/T_a(R/I)$	=	0.481		(ASCE/SEI 7-10, Eq 12.8-3)
C_s min	=	$.044\ S_{DS} * I$	=	0.044		(ASCE/SEI 7-10, Eq 12.8-5)
C_s			=	0.154		(Controls)
V	=	$C_s * W$	=	0.154	W	
ρ			=	1.300		
E	=	ρ * V	=	0.200	W	

MODULE A

Building Weight:

Level	N-S Seismic Load			E-W Seismic Load		
	Area (sf)	Flat Load (psf)	Weight (lbs.)	Area (sf)	Flat Load (psf)	Weight (lbs.)
Sloped Roof	144	15	2160	144	15	2160
Roof	257	12	3084	257	12	3084
Floor	388	16	6208	388	16	6208
Ext. Wall (N)	117	15	1755	117	15	1755
Ext. Wall (S)	117	15	1755	117	15	1755
Ext. Wall (E)	315	15	4729	315	15	4729
Ext. Wall (W)	210	15	3154	210	15	3154
Roof Side Wall (W)	42	13	540	42	13	540
Roof Wall (N)	70	15	1055	70	15	1055
Top Steel			1171			1171
Total Wt.			25609			25609

E = 5121.9 lbs. 5121.9 lbs.

MODULE A

Force Distribution:

Level	h_x^k (ft)	N-S Seismic Load				E-W Seismic Load			
		w_x (lbs)	$w_x{*}h_x^k$	$w_x{*}h_x^k$ / S($w_i{*}h_i$)	F_x (lbs)	w_x (lbs)	$w_x{*}h_x^k$	$w_x{*}h_x^k$ / S($w_i{*}h_i$)	F_x (lbs)
Roof	18.0	13593	244668	0.89064	**4562**	13593	244668	0.89064	**4562**
Ground	2.5	12017	30041	0.10936	**560**	12017	30041	0.10936	**560**
Total		25609	274710	1	5122	25609	274710	1	5122

MODULE B

Building Weight:

Level	N-S Seismic Load			E-W Seismic Load		
	Area (sf)	Flat Load (psf)	Weight (lbs)	Area (sf)	Flat Load (psf)	Weight (lbs)
Sloped Roof	247	15	3703	247	15	3703
Floor	228	16	3648	228	16	3648
Ext. Wall (N)	199	15	2991	199	15	2991
Ext. Wall (S)	161	15	2421	161	15	2421
Roof Wall (N)	146	15	2196	146	15	2196
Steel			1030			1030
Total Wt.			15989			15989

E = 3197.9 lbs 3197.9 lbs

Module B

Force Distribution:

Level	h_x^k (ft)	N-S Seismic Load				E-W Seismic Load			
		w_x (lbs)	$w_x{*}h_x^k$	$w_x{*}h_x^k$ / S($w_i{*}h_i$)	F_x (lbs)	w_x (lbs)	$w_x{*}h_x^k$	$w_x{*}h_x^k$ / S($w_i{*}h_i$)	F_x (lbs)
Roof	18.0	9523	171414	0.91382	**2922**	9523	171414	0.91382	**2922**
Ground	2.5	6466	16166	0.08618	**276**	6466	16166	0.08618	**276**
Total		15989	187580	1	3198	15989	187580	1	3198

MODULE C

Building Weight:

Level	N-S Seismic Load			E-W Seismic Load		
	Area (sf)	Flat Load (psf)	Weight (lbs)	Area (sf)	Flat Load (psf)	Weight (lbs)
Sloped Roof	144	15	2160	144	15	2160
Roof with water	108	16	1728	108	16	1728
Roof	197	12	2364	197	12	2364
Floor	436	16	6976	436	16	6976
Ext. Wall (N)	117	15	1755	117	15	1755
Ext. Wall (S)	117	15	1755	117	15	1755
Ext. Wall (E)	354	15	5314	354	15	5314
Ext. Wall (W)	319	15	4778	319	15	4778
Interior Walls	276	8	2211	276	8	2211
Roof Side Wall (E)	42	13	540	42	13	540
Mech Room			2000			2000
Roof Wall (N)	70	15	1055	70	15	1055
Bathroom			1200			1200
Steel			1171			1171
Total Wt.			35005			35005
		E =	**7001.0**	**lbs**		**7001.0** **lbs**

MODULE C

Force Distribution:

Level	h_x^k (ft)	N-S Seismic Load				E-W Seismic Load			
		w_x (lbs)	$w_x*h_x^k$	$\frac{w_x*h_x^k}{S(w_i*h_i)}$	F_x (lbs)	w_x (lbs)	$w_x*h_x^k$	$\frac{w_x*h_x^k}{S(w_i*h_i)}$	F_x (lbs)
Roof	18.0	16811	302592	0.87688	**6139**	16811	302592	0.87688	**6139**
Ground	2.5	16995	42486	0.12312	**862**	16995	42486	0.12312	**862**
Total		33805	345079	1	7001	33805	345079	1	7001

WIND LOAD

Per ASCE 7-10

Wind conditions for Santa Clara and Irvine are the same.

Basic Wind Speed	V	=	85	mph
Importance Factor	I	=	1.0	
Exposure Category		=	C	
Velocity Pressure Exposure Coefficient	K_Z	=	0.9	
Velocity Pressure Exposure Coefficient	K_h	=	0.85	
Topographic Factor	K_{Zt}	=	1	
Wind Directionality Factor	K_d	=	0.85	
Gust Effect Factor	G	=	0.85	
Enclosure Classification		=	Enclosed	
Internal Pressure Coefficient	GC_{pi}	=	0.18	
	GC_{pi}	=	-0.18	
Wall External Pressure Coefficients	C_p	=	0.8	Windward Wall
			-0.25	Leeward Wall
			-0.7	Side Wall
Roof External Pressure Coefficients	C_p	=	-0.9	Windward Wall
			-0.5	Leeward Wall
Velocity Pressure	$q_z = (0.00256)(Kz)(Kzt)(Kd)(V^2)(I)=$		14.15	
Velocity Pressure	$q_h = (0.00256)(Kh)(Kzt)(Kd)(V^2)(I)=$		13.36	
Design Wind Load	p = qGCp - qi(Gcpi)	=	12.027	psf

Module A				
Width. N-S	=	32.33	ft	
Width. E-W	=	12	ft	
Height	=	18.00	ft	
Vwind.N-S	=	**2598**	**lbs**	Seismic Governs
Vwind.E-W	=	**7000**	**lbs**	Wind Governs
Module B				
Width. N-S	=	12	ft	
Width. E-W	=	21.10	ft	
Height	=	18.00	ft	
Vwind.N-S	=	**4569**	**lbs**	Seismic Governs
Vwind.E-W	=	**2598**	**lbs**	Wind Governs
Module C				
Width. N-S	=	36.33	ft	
Width. E-W	=	12	ft	
Height	=	18.00	ft	
Vwind.N-S	=	**2598**	**lbs**	Seismic Governs
Vwind.E-W	=	**7866**	**lbs**	Wind Governs

SHEAR WALLS

MODULE A

E-W Wind Load on Shear Walls

Shear Wall	Wall Type	Fx (lbs)	Px = Fx/1.4	b (ft)	h(ft)	Aspect Ratio (h:b)	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
A	Segmented	605	432.3	2.84	9.46	3.326	152.0	475	15/32 Str 1	10d @ 6
B	Segmented	605	432.3	2.84	9.46	3.326	152.0	475	15/32 Str 1	10d @ 6
C	Segmented	485	346.5	2.84	9.46	3.326	121.9	475	15/32 Str 1	10d @ 6
D	Segmented	485	346.5	2.84	9.46	3.326	121.9	475	15/32 Str 1	10d @ 6
H&I	Collector A	2530	1806.9	-	-	-	-	-	-	-
K	Collector B	2289	1635.3	-	-	-	-	-	-	-

N-S Seismic Load on Shear Walls

Shear Wall	Wall Type	Fx (lbs)	Px = Fx/1.4	b (ft)	h(ft)	Aspect Ratio (h:b)	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
E	Segmented	1225	874.9	2.75	9.46	3.439	318.1	340	15/32 Str 1	10d @ 6
F	Segmented	3340	2386.0	7.50	9.46	1.261	318.1	340	15/32 Str 1	10d @ 6
G	Segmented	2281	1629.2	12.00	9.46	0.788	135.8	340	15/32 Str 1	10d @ 6

MODULE B

E-W Seismic Load on Shear Walls

Shear Wall	Wall Type	Fx (lbs)	Px = Fx/1.4	b (ft)	h(ft)	Aspect Ratio (h:b)	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
H	Segmented	3477	2483.3	4.00	9.46	2.365	620.8	665	15/32 Str 1	10d @ 3
I	Segmented	3477	2483.3	4.00	9.46	2.365	620.8	665	15/32 Str 1	10d @ 3
K	Collector B	1461	1043.7	-	-	-	-	-	-	-

N-S Wind Load on Shear Walls

Shear Wall	Wall Type	Fx (lbs)	Px = Fx/1.4	b (ft)	h(ft)	Aspect Ratio (h:b)	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
E&F		2284.38	1631.7	-	-	-	-	-	-	-
N,O&P		2284.38	1631.7	-	-	-	-	-	-	-

MODULE C

E-W Wind Load on Shear Walls

Shear Wall	Wall Type	Fx (lbs)	Px = Fx/1.4	b (ft)	h(ft)	Aspect Ratio (h:b)	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
J	Segmented	1643	1173.8	2.71	9.46	3.492	433.4	510	15/32 Str 1	10d @ 4
K	Perforated	6323	4516.1	-	-	-	-	-	-	-
L	Segmented	344	245.6	2.84	9.46	3.326	86.4	475	15/32 Str 1	10d @ 6
M	Segmented	344	245.6	2.84	9.46	3.326	86.4	475	15/32 Str 1	10d @ 6
H&I	Collector A	2963	2116.1	-	-	-	-	-	-	-

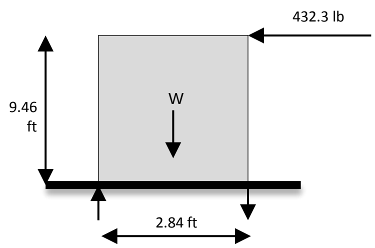
N-S Seismic Load on Shear Walls

Shear Wall	Wall Type	Fx (lbs)	Px = Fx/1.4	b (ft)	h(ft)	Aspect Ratio (h:b)	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
N	Segmented	1784.6	1274.7	4.00	9.46	2.365	318.7	340	15/32 Str 1	10d @ 6
O	Segmented	1784.6	1274.7	4.00	9.46	2.365	318.7	340	15/32 Str 1	10d @ 6
P	Segmented	1784.6	1274.7	4.00	9.46	2.365	318.7	340	15/32 Str 1	10d @ 6
Q	Segmented	3069.5	2192.5	7.67	9.46	1.234	286.0	340	15/32 Str 1	10d @ 6

Perforated Shear Walls

Shear Wall	$\sum L_i$ (ft)	L_{total} (ft)	Opening Height Ratio	Percent Full-Height Sheathing	C_o	v (ASD) (plf)	v. allow (plf)	Plywood (in.)	Edge Nailing (in.)
K	0.00	8.04	h/3	0.75	1	0.0	561.58	15/32 Str 1	10d @ 4

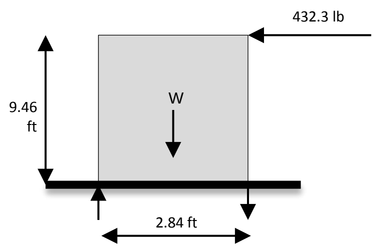
SHEAR WALL - A :



Check Loads:				
P_x =		432.3	lbs	
W^* =		0	lbs	
M_{ot} =		4088.7	lb-ft	
T_{asd} =		1437.79	lbs	
Check Holdown:		HDU2 SDS2.5		
Ta.hd=		3075	lbs	OK
Check Overturning				
W =		0	lbs	
M_r =		8744.5	lb-ft	
M_{ot} =		4088.7	lb-ft	
M_r	>		M_{ot}	OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			27	in.

* Assume weight of wall is 0 to be conservative

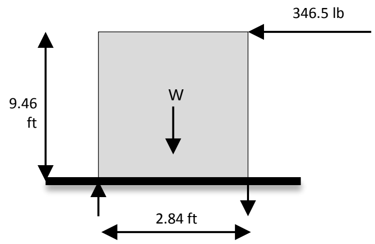
SHEAR WALL - B :



Check Loads:				
P_x =		432.3	lbs	
W^* =		0	lbs	
M_{ot} =		4088.7	lb-ft	
T_{asd} =		1437.79	lbs	
Check Holdown:		HDU2 SDS2.5		
Ta.hd=		3075	lbs	OK
Check Overturning				
W =		0	lbs	
M_r =		8744.5	lb-ft	
M_{ot} =		4088.7	lb-ft	
M_r	>		M_{ot}	OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			27	in.

* Assume weight of wall is 0 to be conservative

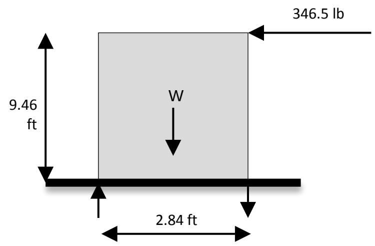
SHEAR WALL - C :



<u>Check Loads:</u>				
P_x =	346.5	lbs		
W^* =	0	lbs		
M_{ot} =	3277.5	lb-ft		
T_{asd} =	1152.51	lbs		
<u>Check Holdown:</u>	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
<u>Check Overturning</u>				
W =	0	lbs		
M_r =	8744.5	lb-ft		
M_{ot} =	3277.5	lb-ft		
M_r	>	M_{ot}		OK
<u>Clips to Top Plate</u>				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			27	in.

* Assume weight of wall is 0 to be conservative

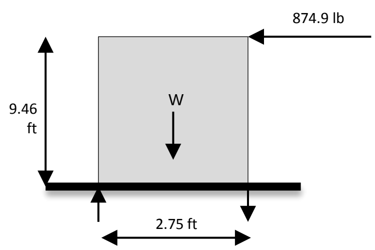
SHEAR WALL - D :



<u>Check Loads:</u>				
P_x =	346.5	lbs		
W^* =	0	lbs		
M_{ot} =	3277.5	lb-ft		
T_{asd} =	1152.51	lbs		
<u>Check Holdown:</u>	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
<u>Check Overturning</u>				
W =	0	lbs		
M_r =	8744.5	lb-ft		
M_{ot} =	3277.5	lb-ft		
M_r	>	M_{ot}		OK
<u>Clips to Top Plate</u>				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			27	in.

* Assume weight of wall is 0 to be conservative

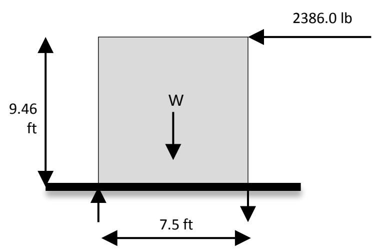
SHEAR WALL - E :



<u>Check Loads:</u>				
P_x =	874.9	lbs		
W^* =	0	lbs		
M_{ot} =	8274.9	lb-ft		
T_{asd} =	3009.04	lbs		
<u>Check Holdown:</u>	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
<u>Check Overturning</u>				
W =	0	lbs		
M_r =	8456.3	lb-ft		
M_{ot} =	8274.9	lb-ft		
M_r	>	M_{ot}		OK
<u>Clips to Top Plate</u>				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			26	in.

* Assume weight of wall is 0 to be conservative

SHEAR WALL - F :

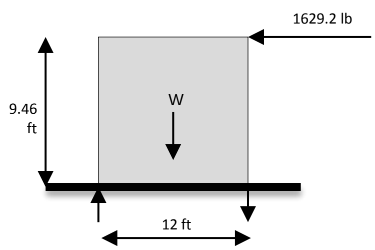


Check Loads:

P_x =	2386.0	lbs		
W^* =	0	lbs		
M_{ot} =	22567.8	lb-ft		
T_{asd} =	3009.04	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	23062.5	lb-ft		
M_{ot} =	22567.8	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			4	
Req'd Spacing			30.00	in.

* Assume weight of wall is 0 to be conservative

SHEAR WALL - G :

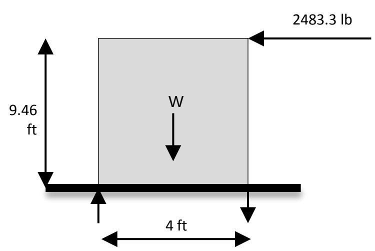


Check Loads:

P_x =	1629.2	lbs		
W^* =	0	lbs		
M_{ot} =	15409.5	lb-ft		
T_{asd} =	1284.12	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	36900.0	lb-ft		
M_{ot} =	15409.5	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			3	
Req'd Spacing			48	in.

* Assume weight of wall is 0 to be conservative

SHEAR WALL - H :

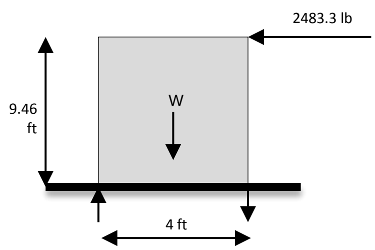


Check Loads:

P_x =	2483.3	lbs		
W^* =	0	lbs		
M_{ot} =	23488.3	lb-ft		
T_{asd} =	5872.07	lbs		
Check Holdown:	HDU8 SDS2.5			
Ta.hd=	6970	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	27880.0	lb-ft		
M_{ot} =	23488.3	lb-ft		
M_r	>	M_{ot}		OK

* Assume weight of wall is 0 to be conservative

SHEAR WALL - I :

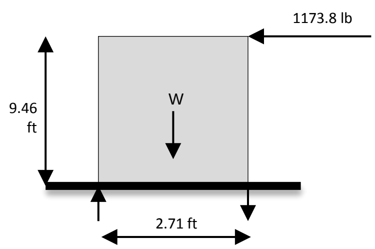


Check Loads:

P_x	=	2483.3	lbs		
W*	=	0	lbs		
M_{ot}	=	23488.3	lb-ft		
T_{asd}	=	5872.07	lbs		
Check Holdown:		HDU8 SDS2.5			
Ta.hd=		6970	lbs		OK
Check Overturning					
W	=	0	lbs		
M_r	=	27880.0	lb-ft		
M_{ot}	=	23488.3	lb-ft		
M_r		>	M_{ot}		OK

* Assume weight of wall is 0 to be conservative

SHEAR WALL - J :

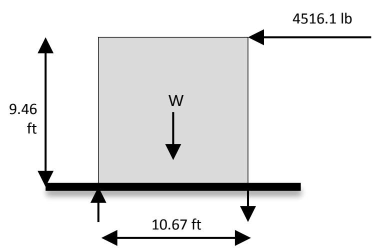


Check Loads:

P_x	=	1173.8	lbs		
W*	=	0	lbs		
M_{ot}	=	11102.6	lb-ft		
T_{asd}	=	4099.42	lbs		
Check Holdown:		HDU4 SDS2.5			
Ta.hd=		4565	lbs		OK
Check Overturning					
W	=	0	lbs		
M_r	=	12363.5	lb-ft		
M_{ot}	=	11102.6	lb-ft		
M_r		>	M_{ot}		OK
Clips to Top Plate					
Clip				A35	
Capacity				695	lbs
Number Req'd				2	
Req'd Spacing				30	in.

* Assume weight of wall is 0 to be conservative

SHEAR WALL - K :

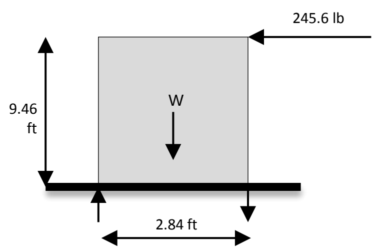


Check Loads:

P_x	=	4516.1	lbs		
T_{asd}	=	$\frac{Vh}{Co\sum Li}$	lbs / lb-ft		
T_{asd}	=	5400.00	lbs		
Check Holdown:		HDU5 SDS2.5			
Ta.hd=		5645	lbs		OK
Clips to Top Plate					
Clip				A35	
Capacity				695	lbs
Number Req'd				7	
Req'd Spacing				21.00	in.

* Assume weight of wall is 0 to be conservative

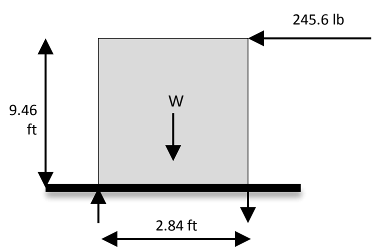
SHEAR WALL - L:



Check Loads:				
P_x =	245.6	lbs		
W^* =	0	lbs		
M_{ot} =	2323.4	lb-ft		
T_{asd} =	817.0	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	8744.5	lb-ft		
M_{ot} =	2323.4	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			26	in.

* Assume weight of wall is 0 to be conservative

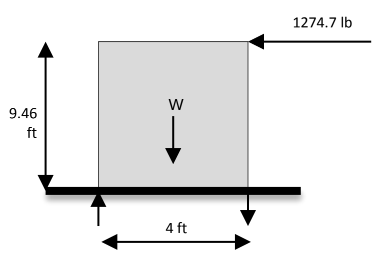
SHEAR WALL - M :



Check Loads:				
P_x =	245.6	lbs		
W^* =	0	lbs		
M_{ot} =	2323.4	lb-ft		
T_{asd} =	817.00	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	8744.5	lb-ft		
M_{ot} =	2323.4	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			26	in.

* Assume weight of wall is 0 to be conservative

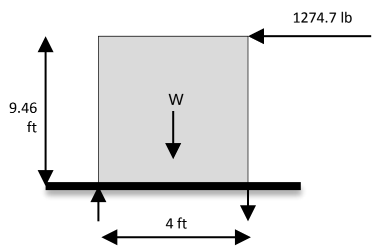
SHEAR WALL - N :



Check Loads:				
P_x =	1274.7	lbs		
W^* =	0	lbs		
M_{ot} =	12056.9	lb-ft		
T_{asd} =	3014.24	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	0.0	lb-ft		
M_{ot} =	12056.9	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			24	in.

* Assume weight of wall is 0 to be conservative

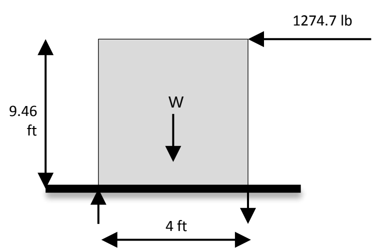
SHEAR WALL - O :



Check Loads:				
P_x =	1274.7	lbs		
W* =	0	lbs		
M_{ot} =	12056.9	lb-ft		
T_{asd} =	3014.24	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	12300.0	lb-ft		
M_{ot} =	12056.9	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			24	in.

* Assume weight of wall is 0 to be conservative

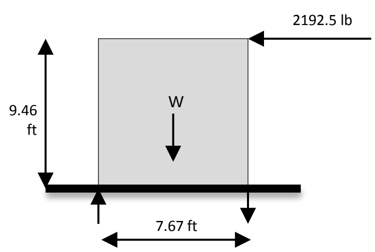
SHEAR WALL - P :



Check Loads:				
P_x =	1274.7	lbs		
W* =	0	lbs		
M_{ot} =	12056.9	lb-ft		
T_{asd} =	3014.24	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	12300.0	lb-ft		
M_{ot} =	12056.9	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			2	
Req'd Spacing			24	in.

* Assume weight of wall is 0 to be conservative

SHEAR WALL - Q :



Check Loads:				
P_x =	2192.5	lbs		
W* =	0	lbs		
M_{ot} =	20737.6	lb-ft		
T_{asd} =	2704.91	lbs		
Check Holdown:	HDU2 SDS2.5			
Ta.hd=	3075	lbs		OK
Check Overturning				
W =	0	lbs		
M_r =	23575.0	lb-ft		
M_{ot} =	20737.6	lb-ft		
M_r	>	M_{ot}		OK
Clips to Top Plate				
Clip			A35	
Capacity			695	lbs
Number Req'd			3	
Req'd Spacing			30	in.

* Assume weight of wall is 0 to be conservative

FLAT ROOF DIAPHRAGM

MODULE A

	Unblocked Diaphragm Design			
Direction 1	Height of Story	H_1	12.00	ft
N-S	Dead Load	D_L	12.00	psf
	Length of Building	L_{B1}	12.33	ft
		L_{B2}	12.00	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	144.32	plf
	Seismic	W_S=0.2*D_L*L_{B1}	29.60	plf
V_{max}	Wind	V_{wmax}=W_L*L_{B2}/2	865.95	lb
	Seismic	V_{smax}=W_s*L_{B2}/2	177.60	lb
v_{wd}	Wind	v_{wd}=V_{wmax}/L_{B1}	70.21	plf
v_{sd}	Seismic	v_{sd}=V_{smax}/L_{B1}	14.40	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	v_{wdnom}=v_{wd}*Ω	**140.42**	**plf**
	v_{sdnom}	v_{sdnom}=v_{sd}*Ω	**28.80**	**plf**
Direction 2	Height of Story	H_1	12.00	ft
E-W	Dead Load	D_L	12.00	psf
	Length of Building	L_{B1}	12.00	ft
		L_{B2}	12.33	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	144.32	plf
	Seismic	W_S=0.2*D_L*L_{B1}	28.80	plf
V_{max}	Wind	V_{wmax}=W_L*L_{B2}/2	890.00	lb
	Seismic	V_{smax}=W_s*L_{B2}/2	177.60	lb
v_{wd}	Wind	v_{wd}=V_{wmax}/L_{B1}	74.17	plf
v_{sd}	Seismic	v_{sd}=V_{smax}/L_{B1}	14.80	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	v_{wdnom}=v_{wd}*Ω	**148.33**	**plf**
	v_{sdnom}	v_{sdnom}=v_{sd}*Ω	**29.60**	**plf**

***Use 3/4" T&G, 8d nails @ 6 inch edge nail spacing**

***Use 15 gauge staples @ 12 inches for field stapling**

FLAT ROOF DIAPHRAGM

MODULE C

	Unblocked Diaphragm Design			
Direction 1	Height of Story	H_1	12.00	ft
N-S	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	16.50	ft
		L_{B2}	12.00	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	144.32	plf
	Seismic	$W_S=0.2*D_L*L_{B1}$	52.80	plf
V_{max}	Wind	$V_{wmax}=W_L*L_{B2}/2$	865.95	lb
	Seismic	$V_{smax}=W_s*L_{B2}/2$	316.80	lb
v_{wd}	Wind	$v_{wd}=V_{wmax}/L_{B1}$	52.48	plf
v_{sd}	Seismic	$v_{sd}=V_{smax}/L_{B1}$	19.20	plf
Nominal Capacities	Ω		2	
	$\mathbf{v_{wdnom}}$	$v_{wdnom}=v_{wd}*\Omega$	**104.96**	**plf**
	$\mathbf{v_{sdnom}}$	$v_{sdnom}=v_{sd}*\Omega$	**38.40**	**plf**
Direction 2	Height of Story	H_1	12.00	ft
E-W	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	12.00	ft
		L_{B2}	16.50	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	144.32	plf
	Seismic	$W_S=0.2*D_L*L_{B1}$	38.40	plf
V_{max}	Wind	$V_{wmax}=W_L*L_{B2}/2$	1190.68	lb
	Seismic	$V_{smax}=W_s*L_{B2}/2$	316.80	lb
v_{wd}	Wind	$v_{wd}=V_{wmax}/L_{B1}$	99.22	plf
v_{sd}	Seismic	$v_{sd}=V_{smax}/L_{B1}$	26.40	plf
Nominal Capacities	Ω		2	
	$\mathbf{v_{wdnom}}$	$v_{wdnom}=v_{wd}*\Omega$	**198.45**	**plf**
	$\mathbf{v_{sdnom}}$	$v_{sdnom}=v_{sd}*\Omega$	**52.80**	**plf**

***Use 3/4" T&G, 8d nails @ 6 inch edge nail spacing**

***Use 15 gauge staples @ 12 inches for field stapling**

FLOOR DIAPHRAGM

MODULE A

	Unblocked Diaphragm Design			
Direction 1	Height of Story	H_1	8.00	ft
N-S	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	32.33	ft
		L_{B2}	12.00	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	96.22	plf
	Seismic	$W_S=0.2*D_L*L_{B1}$	103.47	plf
V_{max}	Wind	$V_{wmax}=W_L*L_{B2}/2$	577.30	lb
	Seismic	$V_{smax}=W_s*L_{B2}/2$	620.80	lb
v_{wd}	Wind	$v_{wd}=V_{wmax}/L_{B1}$	17.85	plf
v_{sd}	Seismic	$v_{sd}=V_{smax}/L_{B1}$	19.20	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	$v_{wdnom}=v_{wd}*\Omega$	**35.71**	**plf**
	v_{sdnom}	$v_{sdnom}=v_{sd}*\Omega$	**38.40**	**plf**
Direction 2	Height of Story	H_1	8.00	ft
E-W	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	12.00	ft
		L_{B2}	32.33	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	96.22	plf
	Seismic	$W_S=0.2*D_L*L_{B1}$	38.40	plf
V_{max}	Wind	$V_{wmax}=W_L*L_{B2}/2$	1555.50	lb
	Seismic	$V_{smax}=W_s*L_{B2}/2$	620.80	lb
v_{wd}	Wind	$v_{wd}=V_{wmax}/L_{B1}$	129.62	plf
v_{sd}	Seismic	$v_{sd}=V_{smax}/L_{B1}$	51.73	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	$v_{wdnom}=v_{wd}*\Omega$	259.25	plf
	v_{sdnom}	$v_{sdnom}=v_{sd}*\Omega$	103.47	plf
Load with cut panel sections				
	Diaphragm Edge Shear	$v_{wdnom} * L_{B2}$	8382.39	lbs
	Length with cut panels	L_{B3}	28.33	ft
	v_{wdnom}	$v_{wdnom} * L_{B2}$	**295.85**	**plf**
	Diaphragm Edge Shear	$v_{sdnom} * L_{B2} / L_{B3}$	3345.42	lbs
	v_{sdnom}	$v_{sdnom} * L_{B2} / L_{B3}$	**118.07**	**plf**

***Use 3/4" T&G, 10d nails @ 6 inch edge nail spacing**

***Use 15 gauge staples @ 12 inches for field stapling**

FLOOR DIAPHRAGM

MODULE B

	Unblocked Diaphragm Design			
Direction 1	Height of Story	H_1	8.00	ft
E-W	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	21.50	ft
		L_{B2}	12.00	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	96.22	plf
	Seismic	W_S=0.2*D_L*L_{B1}	68.80	plf
V_{max}	Wind	V_{wmax}=W_L*L_{B2}/2	577.30	lb
	Seismic	V_{smax}=W_s*L_{B2}/2	412.80	lb
v_{wd}	Wind	v_{wd}=V_{wmax}/L_{B1}	26.85	plf
v_{sd}	Seismic	v_{sd}=V_{smax}/L_{B1}	19.20	plf
Nominal Capacities	Ω		2	
	$\mathbf{v_{wdnom}}$	v_{wdnom}=v_{wd}*Ω	**53.70**	**plf**
	$\mathbf{v_{sdnom}}$	v_{sdnom}=v_{sd}*Ω	**38.40**	**plf**
Direction 2	Height of Story	H_1	8.00	ft
N-S	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	12.00	ft
		L_{B2}	21.50	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	96.22	plf
	Seismic	W_S=0.2*D_L*L_{B1}	38.40	plf
V_{max}	Wind	V_{wmax}=W_L*L_{B2}/2	1034.32	lb
	Seismic	V_{smax}=W_s*L_{B2}/2	412.80	lb
v_{wd}	Wind	v_{wd}=V_{wmax}/L_{B1}	86.19	plf
v_{sd}	Seismic	v_{sd}=V_{smax}/L_{B1}	34.40	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	v_{wdnom}=v_{wd}*Ω	172.39	plf
	v_{sdnom}	v_{sdnom}=v_{sd}*Ω	68.80	plf
Load with cut panel sections				
	Diaphragm Edge Shear	v_{wdnom} * L_{B2}	2068.65	lbs
	Length with cut panels	L_{B3}	8.00	ft
	$\mathbf{v_{wdnom}}$	v_{wdnom} * L_{B2}	**258.58**	**plf**
	Diaphragm Edge Shear	v_{sdnom} * L_{B2} / L_{B3}	825.60	lbs
	$\mathbf{v_{sdnom}}$	v_{sdnom} * L_{B2} / L_{B3}	**103.20**	**plf**

***Use 3/4" T&G, 10d nails @ 6 inch edge nail spacing**

***Use 15 gauge staples @ 12 inches for field stapling**

FLOOR DIAPHRAGM

MODULE C

	Unblocked Diaphragm Design			
Direction 1	Height of Story	H_1	8.00	ft
N-S	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	36.33	ft
		L_{B2}	12.00	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	96.22	plf
	Seismic	W_S=0.2*D_L*L_{B1}	116.27	plf
V_{max}	Wind	V_{wmax}=W_L*L_{B2}/2	577.30	lb
	Seismic	V_{smax}=W_s*L_{B2}/2	697.60	lb
v_{wd}	Wind	v_{wd}=V_{wmax}/L_{B1}	15.89	plf
v_{sd}	Seismic	v_{sd}=V_{smax}/L_{B1}	19.20	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	v_{wdnom}=v_{wd}*Ω	**31.78**	**plf**
	v_{sdnom}	v_{sdnom}=v_{sd}*Ω	**38.40**	**plf**

Direction 2	Height of Story	H_1	8.00	ft
E-W	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	12.00	ft
		L_{B2}	36.33	ft
Applied Load	Wind	W_L=(12.027psf)*H_1	96.22	plf
	Seismic	W_S=0.2*D_L*L_{B1}	38.40	plf
V_{max}	Wind	V_{wmax}=W_L*L_{B2}/2	1747.93	lb
	Seismic	V_{smax}=W_s*L_{B2}/2	697.60	lb
v_{wd}	Wind	v_{wd}=V_{wmax}/L_{B1}	145.66	plf
v_{sd}	Seismic	v_{sd}=V_{smax}/L_{B1}	58.13	plf
Nominal Capacities	Ω		2	
	v_{wdnom}	v_{wdnom}=v_{wd}*Ω	291.32	plf
	v_{sdnom}	v_{sdnom}=v_{sd}*Ω	116.27	plf
Load with cut panel sections				
	Diaphragm Edge Shear	v_{wdnom} * L_{B2}	10584.67	lbs
	Length with cut panels	L_{B3}	32.33	ft
	v_{wdnom}	v_{wdnom} * L_{B2}	**327.36**	**plf**
	Diaphragm Edge Shear	v_{sdnom} * L_{B2} / L_{B3}	4224.36	lbs
	v_{sdnom}	v_{sdnom} * L_{B2} / L_{B3}	**130.65**	**plf**

***Use 3/4" T&G, 10d nails @ 6 inch edge nail spacing**

***Use 15 gauge staples @ 12 inches for field stapling**

TJI ® 230 JOISTS PROVIDED INFORMATION

TJI Depth = 9 1/2 "

ROOF JOISTS

40 PSF Live Load/20 PSF Dead Load > 20 PSF Live Load/16 PSF Dead Load **OK**

L/480 Live Load Deflection

40 PSF Live Load/20 PSF Dead Load

Spacing	Allowable Span		
16" o.c.	16'-8"	>11'	**OK**

L/360 Live Load Deflection (Minimum criteria per code)

40 PSF Live Load/20 PSF Dead Load

Spacing	Allowable Span		
16" o.c.	18'-1"	>11'	**OK**

FLOOR JOISTS

$$\Delta = \frac{22.5wL^4}{EI} + \frac{2.67wL^2}{d \times 10^5} = 0.1583 \text{ in.}$$

w = uniform live load in pounds per linear foot = 66.67

L = span in feet = 11.625

d = out-to-out depth of the joist in inches = 9.5

$EI = 206 \times 10^6$ in^2-lb

L/480 = 0.2906 in.

L/360 = 0.3875 in.

L/480 & L/360 > Δ **OK**

FLAT ROOF JOIST CALCULATIONS

2x12 Trimmed Joist - DFL No. 1

Length	138	in
Depth	9.75	in
Width	1.5	in
Area	14.63	in^2
S_x	23.77	in^3
I_x	115.86	in^4

Reference Design Values

F_b	1000	psi
F_v	180	psi
E	1700000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_M	C_t	C_F	C_i	C_r
1	1	1	1.075	1	1.15

Structural Analysis :

Dead load	20	psf
Live load	25	psf
W: D+L	60	plf
M_{load}	11902.5	lb-in

O.C. spacing	16	in

Design Calculations

Bending Check

$Depth_{nominal}$	10.5	in
$Width_{nominal}$	2	in
Dn/Wn :	5.25	** Block @ 16" o.c. with TJI*
C_L	1	
F'_b	$F_b*C_M*C_t*C_D*C_r*C_F*C_i*C_L$	
F'_b	1236.25	psi
f_b : M/S	500.8	psi
$f_b<F'_b$?	**OK**	

Deflection Check

Length/360	0.383	in.
L	2.778	lb/in
Δ	$5*L*Length^4/(384*E*I)$	
Δ	0.067	in
Δ < Length/360?	**OK**	

Shear Check

F'v	$F_v*C_D*C_M*C_t*C_i$	
F'v	180	psi
V: wL/2	345	lb
f_v : 3V/2A	35.4	psi
$f_v<F'_v$?	**OK**	

FLOOR JOIST CALCULATIONS

DFL No. 2 - 2x4 Floor Joists in Module B from Lines 2-2.5 and 4.5-5

Length	42	in
Depth	3.5	in
Width	1.5	in
Area	5.25	in^2
S_x	3.06	in^3
I_x	5.36	in^4

Reference Design Values

F_b	900	psi
F_v	180	psi
E	1600000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_M	C_t	C_F	C_i	C_r
1	1	1	1	1	1.15

Structural Analysis

Dead Load	16	psf
Live Load	50	psf
W: D+L	66	psf
W	94.29	plf
M_{load}:	1732.5	in-lb

Worst Case Tributary Area:	5	ft^2

Design Calculations

Bending Check

$Depth_{nominal}$	4	in
$Width_{nominal}$	2	in
D_n/W_n :	2	OK
C_L	1	
F'_b	$F_b*C_M*C_t*C_D*C_r*C_F*C_i*C_L$	
F'_b	1035	psi
f_b : M/S	565.7	psi
$f_b<F'_b$?	OK	

Deflection Check

Length/360	0.117	in.
L	5.556	lb/in
Δ	$5*L*Length^4$ /(384*E*I)	
Δ	0.026	in
Δ < Length/360?	OK	

Shear Check

F'_v	$F_v*C_D*C_M*C_t*C_i$	
F'_v =	180	psi
V: wL/2	165	lb
f_v : 3V/2A	47.1	psi
$f_v<F'_v$?	OK	

FOUNDATIONS

MODULE A

DIMENSIONS	LENGTH:	32.33	FT	
	HEIGHT:	18	FT	(AT HIGHEST POINT)
	WIDTH:	12	FT	
	WIND:	12.027	PSF	

GRAVITY

DEAD LOAD

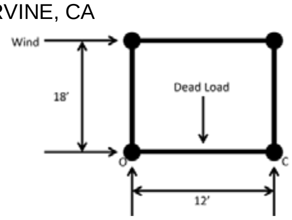
TOTAL MODULE			25609	LBS

LIVE LOAD

FLOOR	50	PSF	19400	LBS
ROOF	20	PSF	8020	LBS
			27420	LBS

1500 PSF BEARING PRESSURE REQUIRES: 19 SQ FT FOR FOOTINGS

LATERAL - FOR IRVINE, CA



WIND:	3028	LBS
DEAD:	27420	LBS

Mo = 0 => c= [(WIND)(18') + (DEAD)(12'/2)]/12'

c= 18251.4 LBS

1500 PSF BEARING PRESSURE REQUIRES: 12.2 SQ FT FOR FOOTINGS PER SIDE

EAST:

	QTY	AREA	
SEISMIC PIERS	2	4	SQ FT
STANDARD PIERS	3	1.8	SQ FT
	13.33	SQ FT	PROVIDED

WEST:

	QTY	AREA	
SEISMIC PIERS	2	4	SQ FT
STANDARD PIERS	3	1.8	SQ FT
	13.33	SQ FT	PROVIDED

26.7	TOTAL SQ FT PROVIDED
1369	PSF PER FOOTING

FOUNDATIONS

MODULE B

DIMENSIONS	LENGTH:	21.17	FT	
	HEIGHT:	18	FT	(AT HIGHEST POINT)
	WIDTH:	12	FT	
	WIND:	12.027	PSF	

GRAVITY

DEAD LOAD

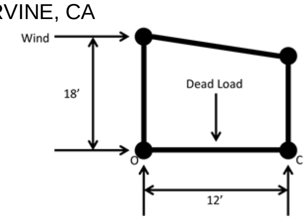
TOTAL MODULE			15989	LBS

LIVE LOAD

FLOOR	50	PSF	11400	LBS
ROOF	20	PSF	4937	LBS
			16337	LBS

1500 PSF BEARING PRESSURE REQUIRES: 11 SQ FT FOR FOOTINGS

LATERAL - FOR IRVINE, CA



WIND:	4159	LBS
DEAD:	16337	LBS

Mo = 0 => c= [(WIND)(18') + (DEAD)(12'/2)]/12'

c= 14407.2 LBS

1500 PSF BEARING PRESSURE REQUIRES: 9.6 SQ FT FOR FOOTINGS PER SIDE

NORTH:

	QTY	AREA	
SEISMIC PIERS	2	4	SQ FT
STANDARD PIERS	1	1.8	SQ FT
	9.78	SQ FT	PROVIDED

SOUTH:

	QTY	AREA	
SEISMIC PIERS	2	4	SQ FT
STANDARD PIERS	1	1.8	SQ FT
	9.78	SQ FT	PROVIDED

19.6	TOTAL SQ FT PROVIDED
1473	PSF PER FOOTING

FOUNDATIONS

MODULE C

DIMENSIONS	LENGTH:	36.33	FT	
	HEIGHT:	18	FT	(AT HIGHEST POINT)
	WIDTH:	12	FT	
	WIND:	12.027	PSF	

GRAVITY

DEAD LOAD

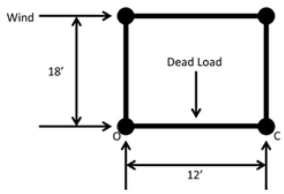
TOTAL MODULE			35005	LBS

LIVE LOAD

FLOOR	50	PSF	21800	LBS
ROOF	20	PSF	8980	LBS
			30780	LBS

1500 PSF BEARING PRESSURE REQUIRES: 21 SQ FT FOR FOOTINGS

LATERAL - FOR IRVINE, CA



WIND:	4760	LBS
DEAD:	30780	LBS

Mo = 0 => c= [(WIND)(18') + (DEAD)(12'/2)]/12'

c= 22529.5 LBS

1500 PSF BEARING PRESSURE REQUIRES: 15.0 SQ FT FOR FOOTINGS PER SIDE

EAST:

	QTY	AREA	
SEISMIC PIERS	2	4	SQ FT
STANDARD PIERS	5	1.78	SQ FT
	16.89	SQ FT	PROVIDED

WEST:

	QTY	AREA	
SEISMIC PIERS	2	4	SQ FT
STANDARD PIERS	5	1.78	SQ FT
	16.89	SQ FT	PROVIDED

33.8	TOTAL SQ FT PROVIDED
1334	PSF PER FOOTING

ANCHORAGE

SEISMIC ANCHOR LOADS

Shear Load (lb) Allowable Design Level			
Module	A	B	C
N-S	**4890**	-	6017
E-W	1558	**4968**	**6181**

Assumed Pullout Design Capacity (per Solar Decathlon rules)
1250 lbs

Shear Load (lb) per Seismic Pier Allowable Design Level			
Module	A	B	C
(lbs)	1223	1242	1545

Assumed Shear Design Capacity (per Solar Decathlon rules)
1500 lbs

Shear Load per Anchor Allowable Design Level			
Module	A	B	C
(lbs)	**1223**	**1242**	**773**

ALLOWABLE SHEAR STRENGTH OF THREADED ROD ANCHOR

AS PER THE DOE, USE A 1" DIAMETER ANCHOR

$R_n = F_nA_b/\Omega$		where $\Omega = 2$
$F_n = F_{nv} =$	20.772	ksi **for A36**
$A_b =$	0.785	in^2
$R_n =$	8157	lbs per anchor

8157 lbs > 1500 lbs **OK**

8157 lbs > 1242 lbs **OK**

PULLOUT STRENGTH - THREADED ROD ANCHOR IN CONCRETE

Embedment length =	36	in.
Pullout Surface Area = π*1.4142*H^2 =	5758	in^2
Shear Strength of Concrete =	800	psi

Force (lbs) = 4606336.85 lbs

4606337 lbs > 1250 lbs **OK**

GRADE VARIABILITY

Standard Piers, fabricated by Central Piers Inc., serve as the footings required to comply with the allowable bearing capacity. See S-101 for the foundation plan and bearing plan. Central Piers Inc. supplies various sized piers for varying ground heights. Central Piers Inc. stocks seismic piers that range from 7”-10” up to 19”-33”. Each Standard Pier has a height adjustment of 2”. Santa Clara plans to bring a set of all of the available sized piers listed on the Footing Adjustability Schedule on S-601 if shimming is needed. Therefore, no additional structural calculations are needed for our method of pier adjustability because an appropriately sized pier will be used where piers are needed.

MOMENT STABILITY

Overturning Moment	=	wind * height		
Resisting Moment	=	weight * base/2		

MODULE A

Overturning Moment	=		29519	lb-ft	
Resisting Moment	=		153656	lb-ft	
OM < RM		OK			
Factor of Safety	=		5.2052659	>1.67	OK

MODULE B

Overturning Moment	=		40550	lb-ft	
Resisting Moment	=		140988	lb-ft	
OM < RM		OK			
Factor of Safety	=		3.4768858	>1.67	OK

MODULE C

Overturning Moment	=		46407	lb-ft	
Resisting Moment	=		210031	lb-ft	
OM < RM		OK			
Factor of Safety	=		4.5258548	>1.67	OK

ROOF MODULE

Overturning Moment	=		24312	lb-ft	
Resisting Moment	=		95548	lb-ft	
OM < RM		OK			
Factor of Safety	=		3.9301247	>1.67	OK

WALL POST DESIGN

Member Information: 4 x 6, No. 1 DF-L

Length	100.75	in
Depth	5.5	in
Width	3.5	in
Area	19.25	in^2

Reference Design Values

F_c	1500	psi
E	1700000	psi
E_{min}	620000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_{M_C}	C_t	C_F	C_i	C_{M_E}	C_{M_Fc}	C_T
1	1	1	1.1	1	1	1	1

Design Calculations

Compression Check

**Worst Case Scenario: Column at A2 or A5*

P_{load}	1300	lbs	
E'_{min}	$E_{min} * C_{M\_E} * C_t * C_i * C_T$	620000	psi
F_{cE}	$0.822 * E'_{min} / ((L/width)^2)$	615	psi
F_c*	$F_c * C_D * C_{M\_Fc} * C_t * C_F * C_i$	1650	psi
C_P	0.338		
F'_c	$F_c * C_D * C_M * C_t * C_F * C_i * C_p$	558.0	psi
f_c	$P_{load}/Area$	67.53	psi
$f_c < F'_c$	**OK**		

WALL STUD DESIGN

Member Information: 2 x 4, No. 2 DF-L

Length	100.75	in
Depth	3.5	in
Width	1.5	in
Area	5.25	in^2

Reference Design Values

F_c	1350	psi
E	1600000	psi
E_{min}	580000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_{M_C}	C_t	C_F	C_i	C_{M_E}	C_{M_Fc}	C_T
1	1	1	1.15	1	1	1	1

Design Calculations

P_{load}	288 lbs		
E'_{min}	$E_{min} * C_{M\_E} * C_t * C_i * C_T$	580000	psi
F_{cE}	$0.822 * E'_{min} / ((L/width)^2)$	106	psi
F_c*	$F_c * C_D * C_{M\_Fc} * C_t * C_F * C_i$	1552.5	psi
C_P	0.067		
F'_c	$F_c * C_D * C_M * C_t * C_F * C_i * C_p$	104.2	psi
f_c	$P_{load}/Area$	54.9	psi
$f_c < F'_c$	**OK**		

Worst Case Scenario: Between A5 & A6

HEADER CALCULATIONS

Header - 4 x 6 DFL No. 1

Length	72	in.
Depth	5.5	in.
Width	3.5	in.
Area	19.25	in^2
S_x	17.65	in^3
I_x	48.53	in^4

Reference Design Values

F_b	900	psi
F_v	180	psi
E	1600000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_M	C_t	C_F	C_i	C_r
1	1	1	1.3	1	1

Structural Analysis

Dead Load	16	psf
Live Load	20	psf
W: D+L	36	psf
W	220	plf
M	11880	in-lb

*(plus 4 plf for 2 - 2x6 plates above header)

Trib. Area of roof over header	36	ft^2

Design Calculations

Bending Check

$Depth_{nominal}$	6	in.
$Width_{nominal}$	4	in.
Dn/Wn	1.5	OK
C_L	1	
F'_b	$F_b*C_M*C_t*C_D*C_r*C_F*C_i*C_L$	
F'_b	1170	psi
f_b : M/S	673.25	psi
$f_b<F'_b$?	OK	

Deflection Check

Length/360	0.2	in.
L	1.667	lb/in
Δ	$5*L*Length^4/(384*E*I)$	
Δ	0.007511	in.
Δ < Length/360?	OK	

Shear Check

F'_v	$F_v*C_D*C_M*C_t*C_i$	
F'_v	180	psi
V: wL/2	660	lb
f_v : 3V/2A	51.43	psi
$f_v<F'_v$?	OK	

*** USE 4X6 HEADERS ON OPENINGS BETWEEN 4-6 ft**

HEADER CALCULATIONS

Header - 2 x 6 DFL No. 2

Length	48	in.
Depth	5.5	in.
Width	1.5	in.
Area	8.25	in^2
S_x	7.56	in^3
I_x	20.80	in^4

Reference Design Values

F_b	900	psi
F_v	180	psi
E	1600000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_M	C_t	C_F	C_i	C_r
1	1	1	1.3	1	1

Structural Analysis

Dead Load	16	psf
Live Load	20	psf
W: D+L	36	psf
W	220	plf
M	5280	in-lb

*(plus 4 plf for 2 - 2x6 plates above header)

Trib. Area of roof over header	24	ft^2

Design Calculations

Bending Check

$Depth_{nominal}$	6	in.
$Width_{nominal}$	2	in.
D_n/W_n	3	OK
C_L	1	
F'_b	$F_b*C_M*C_t*C_D*C_r*C_F*C_i*C_L$	
F'_b	1170	psi
f_b : M/S	698.18	psi
$f_b<F'_b$?	OK	

Deflection Check

Length/360	0.13	in.
L	1.667	lb/in
Δ	$5*L*Length^4/(384*E*I)$	
Δ	0.003462	in.
Δ < Length/360?	OK	

Shear Check

F'_v	$F_v*C_D*C_M*C_t*C_i$	
F'_v	180	psi
V: wL/2	440	lb
f_v : 3V/2A	80.00	psi
$f_v<F'_v$?	OK	

*** USE 2X6 HEADERS ON OPENINGS LESS THAN 4 ft**

STEEL

BOTTOM STEEL CHANNEL : C15X33.9

SUPPORTS AT : 0', 6', 14', 22.33', 30.33', & 36.33'

LOADS

D+L	330.0	plf

PROPERTIES

W	33.9	lb/ft	S_x	42	in^3
A	10	in^2	I_x	315	in^4
d	15	in	l	36.33	ft
b	3.375	in	E	29000000	psi
t	0.625	in	F_y	36	ksi

DEFLECTION

$\Delta_{allowable}$	=	l/480	=	0.076'
Δ_{max}	=	0.009'		**OK**

Max Allowable Uniform Load	=	91.3	klf
Actual Uniform Load	=	0.33	klf
	Max > Actual		**OK**

FLEXURE

Yielding

M_n	=	91.3	kip-ft	
M_{max}	=	2.074	kip-ft	**OK**

SHEAR

V_n	=	77.6	kips	
V_{max}	=	1.888	kips	**OK**

BOTTOM SHORT EDGE STEEL CHANNEL- MODULE B : C8X11.5

SUPPORTS AT : 0', 6', & 12'

LOADS

D + L	216.0	plf

PROPERTIES

W	11.5	lb/ft	S_x	8.14	in^3
A	3.37	in^2	I_x	32.5	in^4
d	8	in	l	6	ft
b	2.25	in	E	29000000	psi
t	0.375	in	F_y	36	ksi

DEFLECTION

Δ_{max}	=	0.007'	=	**OK**
$\Delta_{allowable}$	=	l/480	=	0.013'

Max Allowable Uniform Load	=	17.3	klf
Actual Uniform Load	=	0.216	klf
	Max > Actual		OK

FLEXURE

Yielding

M_n	=	17.3	kip-ft	
M_{max}	=	1.024	kip-ft	**OK**

SHEAR

V_n	=	22.8	kips	
V_{max}	=	0.832	kips	**OK**

STEEL

BOTTOM STEEL: HSS 8X3X3/8

LOADS

Point load during transportation	=	2143.2	lbs

PROPERTIES

b/t	=	5.6
h/t	=	19.9
Cantilevered Length	=	18.6875"

Need to check yielding, flange local buckling, and web local buckling

DEFLECTION

$\Delta_{allowable}$	=	l/480	=	0.039"
Δ_{max}	=	0.0033"	=	**OK**

FLEXURE

Yielding

M_n	=	61.72	kip-ft	
M_{max}	=	3.34	kip-ft	**OK**

SHEAR

$Vn = 0.6F_yA_wC_v$ $A_w = 5.4375$ in^2 $C_v = 1$ $F_y = 46$ ksi

V_n	=	89.87	kips	
V_{max}	=	2.14	kips	**OK**

FLANGE LOCAL BUCKLING

$1.12\sqrt{(E/F_y)}$ = 28.12 > b/t **FLB DOES NOT APPLY**

WEB LOCAL BUCKLING

$2.42\sqrt{(E/F_y)}$ = 60.76 >h/t **WLB DOES NOT APPLY**

STEEL

STEEL COLUMNS : HSS 3.5X3.5X5/16

LOADS

Point Load	=	3	kips

PROPERTIES

W	12.7	lb/ft	S_x	3.34	in^3	r	1.29	in
A_g	3.52	in^2	I_x	5.84	in^4	b/t	9.03	
d	3.5	in	E	29000000	psi	KL/r	81.77	
l	8.79	ft	F_y	46	ksi			

Need check local buckling and flexural buckling

FLEXURAL BUCKLING

When $KL/r < 4.71\sqrt{E/F_y}$:

$$F_{cr} = [\,0.658^{F_y/F_e}\,]\,F_y$$

$$\text{where } F_e = \frac{\pi^2 E}{(KL/r)^2} = 42.81 \text{ ksi}$$

F_{cr} = 29.34 ksi

$P_n = 0.9F_{cr}A_g$ = 92.94 kips > 3 kips **OK**

LOCAL BUCKLING

$1.4\sqrt{(E/I)}$ **=** 59.07 > b/t **NONSLENDER - LB DOES NOT APPLY**

STEEL

TOP MODULE CONNECTOR BEAM : W5X19

LOADS

D + L	=	300.0	plf

PROPERTIES

W	16	lb/ft	S_x	10.2	in^3	L	21.25	ft
A	5.56	in^2	I_x	26.3	in^4	L_b	11.583	ft
A_w	2.24	in^2	I_y	9.13	in^4	Z_x	9.63	in^3
d	5.125	in	E	29000000	psi	r_y	1.28	in.
b_f	5	in	Fy	50	ksi	C_v	1	
t_w	0.4375	in	J_c	0.316	in^4	C_b	1.14	
h_o	4.72	in.						

Need to check yielding and lateral torsional buckling.

DEFLECTION

$\Delta_{allowable}$	=	$l_b/480$	=	0.024'
Δ_{max}	=	0.0133'	=	**OK**

Max Allowable Uniform Load	=	91.3	klf
Actual Uniform Load	=	0.33	klf
	Max > Actual		OK

FLEXURE

Yielding

M_n	=	481.5	kip-in	
M_{max}	=	60.4	kip- in	**OK**

SHEAR

$Vn = 0.6F_yA_wC_v$

V_n	=	67.27	kips	
V_{max}	=	1.738	kips	**OK**

LATERAL - TORSIONAL BUCKLING

L_p	=	$1.76r_y*\surd(E/F_y)$	=		54.25	in	
L_b	=	139	in				
L_r	=	$1.95r_{ts}(E/0.7F_y) *\surd\{(J_c/S_xh_o)+\surd[(J_c/S_xh_o)^2+6.76(0.7F_y/E)^2]\}$					
r_{ts}^2	=	$I_yh_o/2S_x$	→	r_{ts}	=	1.453	
L_r	=	276.25	in.				

When $L_p<L_b<L_r$:

$M_n = C_b \{M_p - [M_p - 0.7F_yS_x]*[(L_b-L_p)/(L_r-L_p)]\} \leq M_p$

M_n =	339.4	k-in	$\leq$	Mp	**OK**
M_n	>	M_{max}			**OK**

STEEL

LATERAL COLLECTOR BEAM : W5X19

LOADS

E-W Seismic and Wind Loads = 7.1 kips

PROPERTIES

b/t	5.85		E	29000	ksi	L	141.81	in
A_g	5.56	in^2	Fy	50	ksi	r_y	1.26	in.
C_w	50.9	in^6	I_x	26.3	in^4	I_y	9.13	in^4
J	0.316	in^4	G	11200	ksi	KL/r	112.55	

Need to check flexural buckling and torsional buckling.

FLEXURAL BUCKLING

When $KL/r < 4.71*\sqrt{(E/F_y)}$:

$F_{cr} = [0.658^{Fy/Fe}] F_y$

where $F_e = \frac{\pi^2 E}{(KL/r)^2}$ = 22.59 ksi

F_{cr} = 19.80 ksi

$P_n = 0.9F_{cr}A_g$ = 99.09 kips $\geq$ 7.1 kips **OK**

TORSIONAL BUCKLING

For doubly symmetric members:

$F_e = [\pi^2 EC_w/(K_zL)^2 + GJ] * 1/(I_x+I_y)$

F_e = 120.34 ksi

F_{cr} = 42.02 ksi

$P_n = 0.9F_{cr}A_g$ = 210.26 kips $\geq$ 7.1 Kips **OK**

STEEL

W5x19 TO COLUMN PLATE CONNECTION

LOADS

W_{wind}	5244	lb	
$W_{seismic}$	5783	lb	* (Seismic Governs)

(2) 1/2" A307 threaded bolt

ALLOWABLE SHEAR STRENGTH

F_{nv}	27	ksi	
A_b	0.196	in^2	
ϕ	0.75		
$R_n = \varphi * F_{nv} * A_b$	3976.08	lbs	
$2 * R_n$	7952.16	lbs	
7952.16 >	5783.00		**OK**

AVAILABLE TENSILE STRENGTH

f_{rv}	13.4	ksi	(WIND LOAD)
F'_{nt}	28.83	≤	**45**

$R_n = \varphi * F'_{nt} * 2A_b =$ 8490 lbs **OK**

W5X19 SPLICE CONNECTION

(Details 2 & 3 on SOS E2)

$A_n * U = A_e$

$\varphi A_e * F_v \geq 7.87^k$

A_e =	0.5625	in^2			
F_v =	21.6	ksi			
$\varphi A_e * F_v$=	9.1125	kips	$\geq$	7.87^k	**OK**

1/2" A325 bolts

$F_n A_b = R_n$ =	10.60	kips			
φR_n =	7.95	kips	$\geq$	7.87^k	**OK**

STEEL

C15X33.9 to C8X11.5 MODULE CONNECTION

MODULE A TO B Detail 4 on SOS E2

LOADS

$W_{wind.}$	5122	lb	* (Wind Governs)
$W_{seismic}$	4569	lb	

1" A307 threaded bolt

ALLOWABLE SHEAR STRENGTH

F_{nv}		27	ksi	
A_b		0.785	in^2	
ϕ		0.75		
$R_n = \varphi * F_{nv} * A_b$		15904.31	lbs	
15904.31	>	5122.00		**OK**

AVAILABLE TENSILE STRENGTH

f_{rv}	6.5	ksi	(WIND LOAD)	
F'_{nt}	44.01	≤	**45**	
$R_n = \varphi * F'_{nt} * A_b =$	25923	lbs		**OK**

STEEL

C15x33.9 to C8x11.5 MODULE CONNECTION

MODULE B TO C Detail 4 on SOS E2

LOADS

$W_{wind.}$	7866	lb	* (Wind Governs)
$W_{seismic}$	7001	lb	

1" A307 threaded bolt

ALLOWABLE SHEAR STRENGTH

F_{nv}	27	ksi	
A_b	0.785	in^2	
ϕ	0.75		
$R_n = \varphi * F_{nv} * A_b$	15904.31	lbs	
15904.31 >	7866.00		**OK**

AVAILABLE TENSILE STRENGTH

f_{rv}	10.0	ksi	(WIND LOAD)
F'_{nt}	36.24	≤	**45**

$R_n = \varphi * F'_{nt} * A_b =$ 21349 lbs **OK**

STEEL

TEMPORARY BRACING : L2X2X1/4

For Transportation

Design Load :	20	kips	in tension or compression

TENSION

A_g	0.944	in^2
$A_e = 0.75A_g$	0.708	in^2

Available Strength in Axial Tension

Yielding

$\varphi_t P_n$	30.6	kips	>	20	kips

Rupture

$\varphi_t P_n$	30.8	kips	>	20	kips

COMPRESSION

$0.45\sqrt{E/F_y}$	12.77
b/t	8

$b/t < 0.45\sqrt{E/F_y} \rightarrow$ nonslender

$P_n = F_{cr}A_g$

KL/r	261.47
$4.71\sqrt{E/F_y}$	133.68

$KL/r > 4.71\sqrt{E/F_y} \rightarrow F_{cr} = 0.877\ F_e$

F_{cr}	3.672	ksi	
P_n	3.466	kips	Per brace

→ Need 6 braces per module B to take 20 kips load in compression and tension

→ Add bracing in other modules for additional support to their existing shear walls

CONNECTIONS

DTT2Z-SDS2.5 HOLDOWN CONNECTION

NORTH SIDE

LOAD

(12.027 PSF)*(7'/2)*(41.5'/2) =		873.46	LBS
(12.027PSF)*(9.2'/2)*(21.1'/2) =		583.67	LBS
	∑	1457.13	LBS

HOLDOWN:

DTT2Z-SDS2.5

2145 LBS > 1457.13 LBS **OK**

SOUTH SIDE

LOAD

(12.027 PSF)*(7')*(41.5'/2) =		1746.92	LBS
(12.027PSF)*(7'/2)*(16.8'/2) =		353.59	LBS
	∑	2100.52	LBS

PERCENT THAT GOES TO HOLDOWN : 51.3%

HOLDOWN:

DTT2Z-SDS2.5

2145 LBS > 1077.56 LBS **OK**

ROOF MODULE

STEEL

ROOF BOTTOM STEEL ANGLE : L8X4X1/2

SUPPORTS AT : 0.1667', 9.75', 31.21', 40.79'

LOADS

$W_{Live\ Load}$	20	psf	120	plf				
$W_{Dead\ Load}$	15.5	psf			Tributary Width	6	ft
D + L	35.5	psf	212.7	plf	Average Wall Height	6.083	ft

PROPERTIES

W	19.6	lb/ft	S_x	7.48	in^2
A	5.8	in^2	I_x	38.6	in^4
d	4	in	l	0.1667	ft
b	8	in	E	29000000	psi
t	0.5	in	Fy	36	ksi

DEFLECTION

Beam	$\Delta_{allowable}$	=	l/480	=	0.537 in
	$\Delta_{max\ (21.46'\ span)}$	=	0.272 in	=	**OK**

Cantilever	$\Delta_{allowable}$	=	l/480	=	0.004 in
	$\Delta_{max(0.1667'\ span)}$	=	0.000	in	**OK**

FLEXURE

Yielding

M_y 215.42 kip-in

M_n	=	1.5*M_y	=	26.93	kip-ft
M_{max}	=		7.63	kip-ft	**OK**

SHEAR

$h/t_w = b/t$ 18.7 < 63.58

V_n	=	.6F_yb*t*C_v	=	57.6	kips
V_{max}	=		2.542	kips	**OK**

LATERAL TORSIONAL BUCKLING

Continuous Lateral Support - N/A

LEG LOCAL BUCKLING

Compact Section - N/A

CHECK INTERNAL BENDING

Load (1' span)

P	=	(35.5 psf * cos (24) + 12.027 psf)*11.25 ft^2	=	500.15	lb
l	8	in			
b	5.625	in			

M_{max}	=	P*b	=	234.45	lb-ft	**OK**
$M_{allowable}$	=	$0.8S_xF_y$	=	2585088	lb-ft	

$\Delta_{allowable}$	=	l/360	=	0.0222	in	
Δ_{max}	=	$(Pb^2/6EI)(3l-b)$	=	0.0201	in	**OK**

STEEL

ROOF BOTTOM STEEL ANGLE : L8X4X1/2

SUPPORTS AT : 0.1667', 9.75', 31.21', 40.79'

LOADS

$W_{Live\ Load}$	120	plf			
$W_{Dead\ Load}$	114.8	plf	Tributary Width	6	ft
D + L	234.8	plf	Average Wall Height	6.083	ft

PROPERTIES

W	19.6	lb/ft	S_x	7.48	in^2
A	5.8	in^2	I_x	38.6	in^4
d	4	in	l	0.1667	ft
b	8	in	E	29000000	psi
t	0.5	in	Fy	36	ksi

DEFLECTION

Beam	$\Delta_{allowable}$	=	l/480	=	0.537
	$\Delta_{max\ (21.46'\ span)}$	=	0.356	=	**OK**

Cantilever	$\Delta_{allowable}$	=	l/480	=	0.004
	$\Delta_{max(0.1667'\ span)}$	=	0.000	in.	**OK**

FLEXURE

Yielding

M_y 215.42 kip-in

M_n	=	$1.5*M_y$	=	26.93	kip-ft
M_{max}	=		8.19	kip-ft	**OK**

SHEAR

$h/t_w = b/t$ 18.7 < 63.58

V_n	=	$.6F_y btC_v/\Omega$	=	57.6	kips
V_{max}	=		2.73	kips	**OK**

LATERAL TORSIONAL BUCKLING

Continuous Lateral Support - N/A

LEG LOCAL BUCKLING

Compact Section - N/A

CHECK INTERNAL BENDING

Load (1' span)

P	=	(35.5 psf * cos (24) + 12.027 psf)*11.25 ft^2 =	500.15	lb
l	8	in		
b	5.625	in		

M_{max}	=	P*b	=	234.45	lb-ft	**OK**
$M_{allowable}$	=	$0.8S_xF_y$	=	2585088	lb-ft	

$\Delta_{allowable}$	=	l/360	=	0.0222	in	
Δ_{max}	=	$(Pb^2/6EI)(3l-b)$	=	0.0201	in	**OK**

STEEL

NORTH WALL ROOF STEEL COLUMNS : C3X3.5

LOADS

$W_{wind\ E\text{-}W}$	12.027	psf	144.32	plf	Tributary Width	12.00	ft
$W_{seismic}$	7499.52	lb	308.20	plf	Beam Length	6.083	ft
$W_{seismic}$	1874.88	lb per column		* (Seismic Governs)			

SECTION PROPERTIES

Weight	3.5	lb/ft	Weight	85.167	lb per column			
A	1.09	in	I_x	1.57	in^4	E	29000000	psi
d	3	in	Z_x	1.24	in^3	F_y	36	ksi
b_f	1.37	in	S_x	1.04	in^3	Fu	58	ksi
t_w	0.132	in	r_x	1.2	in	b/t	5.02	
t_f	0.273	in	L	6.0833	ft	h/t_w	14.5	

DEFLECTION

$\Delta_{allowable}$	=	l/480	=	0.152	in	
$\Delta_{max\ (21.46'\ span)}$	=	$5wL^4/(384EI)$	=	0.0024	in	**OK**

FLEXURE

Yielding Ω_b = 1.67

$M_n = M_p/\Omega = F_yZ_x/\Omega$ =	26.73	k-in =	2.228	k-ft

SHEAR

V_n	=	$.6F_yA_wC_v/\Omega$	=	5.7024	kips	
V_{max}	=			0.93744	kips	**OK**

Ω = 1.5 k_v = 5 C_v = 1

LATERAL TORSIONAL BUCKLING

L_p	147.929	in
L_b	6.083	in
L_b	$\leq$	L_p

→ NO LATERAL TORSIONAL BUCKLING

M_{max}	$= wl^2/12$	=	0.95	kip-ft	**OK**
M_p		=	2.23	kip-ft	

TENSION

P_{max} = 0.1259 kips **OK**

Yielding

$P_n = F_yA_g / \Omega$	=	23.50	kips

Rupture

$P_n = F_uA_e / \Omega$	=	33.27	kips

NORTH WALL ROOF - TIMBER FRAMING

DOUG FIR LARCH 2X6 STUDS

LOADS

$W_{Live\ Load}$	20	psf	120	plf					
$W_{Dead\ Load}$	18	psf	108	plf		fc = P/A		126.7	psi
$V_{wind\ N\text{-}S}$	12.027	psf	55.1	plf		fb = M/S		404.6	psi
D + L +W	50.027	psf	283.1	plf					
					M	254.99	lb-ft		

SECTION PROPERTIES

b	1.5	in	W	1.9	plf	219.213	lb
d	5.5	in	E	1600000	psi	5.219	plf over northwall
A	8.25	in^2	E_{min}	580000	psi		
S_x	7.563	in^3	F_b	900	psi		
I_x	20.8	in^4	F_c	1350	psi		
S_y	2.063	in^4	F_v	180	psi		
I_y	1.547	in^4					
			Grav Tributary Width	6	ft		
			Wind Tributary Width	4.58	ft	MAX TRIB LENGTH	
			Average Wall Height = l	6.08	ft		

BEAM COLUMN ANALYSIS

$$(f_c/F'_c)^2+(F_{b1}/(F'_{b1}*(1-(f_c/F_{cE1}))))+(f_{b2}/(F'_{b2}*(f_c/F_{cE2})-(f_{b1}/F_{bE})^2)) \leq 1.0$$

Adjustment Factors:

C_D	1.6	C_{fu}	1.0
C_{F_Fc}	1.1	C_T	1.0
C_{F_Fb}	1.3	C_i	1.0
C_t	1.0	C_{M_Fb}	1.0
C_r	1.0	C_{M_E}	1.0
C_{M_Fc}	1.0		

STRUCTURAL ANALYSIS

Major Axis Bending

M_{x_Max}	1309.7	lb-ft	(Simple Assumed)
F_{b1}	2078.0	psi	

Minor Axis Bending

F_{b1}	0	psi	(Concentric Axial Force)

Axial Load

P_{axial}	1722.33	lb
F_c	208.77	psi

MEMBER CAPACITIES

Axial Capacity

F'_c $F'_c = F_c\ C_D\ C_M\ C_t\ C_F\ C_i\ C_P$ **1743.95 psi**

E'_{min}	$E'_{min} = E_{min}\ C_M\ C_t\ C_i\ C_T$	580000	psi
F_{cE}	$.822(E'_{min})/(l_e/d)^2$	2706.32	psi

K_e 1 ☐ l_e 6.083

l_e/d 13.27 **OK**

F^*_c	$F^*_c = F_c\ C_D\ C_M\ C_t\ C_F\ C_i$	2376	psi

→ C_p 0.734

Flexural Capacity

F'_b $F'_b = F_b\ C_D\ C_M\ C_t\ C_L\ C_F\ C_i\ C_{fu}\ C_r$ **1872 psi**

CHECK

$(f_c/F'_c)^2+(F_{b1}/(F'_{b1}*(1-(f_c/F_{cE1}))))+(f_{b2}/(F'_{b2}*(f_c/F_{cE2})-(f_{b1}/F_{bE})^2)) \le 1.0$

0.232	≤	1.0	**OK**

TJI ® 230 JOISTS PROVIDED INFORMATION

TJI Depth = 9 1/2 "

ROOF JOISTS

40 PSF Live Load/20 PSF Dead Load > 20 PSF Live Load/16 PSF Dead Load **OK**

L/480 Live Load Deflection

40 PSF Live Load/20 PSF Dead Load

Spacing	Allowable Span		
16" o.c.	16'-8"	>13'	**OK**

L/360 Live Load Deflection (Minimum criteria per code)

40 PSF Live Load/20 PSF Dead Load

Spacing	Allowable Span		
16" o.c.	18'-1"	>13'	**OK**

ROOF CHORD FORCES

Chord Force = M/d = T = C

LOADS

$W_{Live\ Load}$	0	psf	0	plf			
$W_{Dead\ Load}$	18	psf			Chord Depth (d) =	12	ft
D + L	18	psf	216	plf	Length (l) =	43.00	ft

CHORD FORCE ANALYSIS

$M = Wl^2/8 =$ 49923 lb-ft

M/d = 4.16025 kips

MEMBER ALLOWABLE

A36 - L8X4X1/2 - Braced Column Analysis

T_{allow}	188	kips	(Table 5-2) AISC Steel Construction Manual
C_{allow}	34.6	kips	(Table 4-11) AISC Steel Construction Manual

$\text{Chord Force}_{Capacity} =$ 34.6 > $\text{Chord Force}_{Demand} =$ 4.16025 **OK**

SLOPED ROOF DIAPHRAGM

ROOF MODULE

	Unblocked Diaphragm Design			
Direction 1	Height of Story	H_1	6.00	ft
N-S	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	41.06	ft
		L_{B2}	12.00	ft
Applied Load	Wind	$W_L=(12.027psf)*H_1$	72.16	plf
	Seismic	$W_S=0.2*D_L*L_{B1}$	131.40	plf
V_{max}	Wind	$V_{wmax}=W_L*L_{B2}/2$	432.97	lb
	Seismic	$V_{smax}=W_s*L_{B2}/2$	788.38	lb
v_{wd}	Wind	$v_{wd}=V_{wmax}/L_{B1}$	10.54	plf
v_{sd}	Seismic	$v_{sd}=V_{smax}/L_{B1}$	19.20	plf
Nominal Capacities	Ω		2	
	$\mathbf{v_{wdnom}}$	$v_{wdnom}=v_{wd}*\Omega$	**21.09**	**plf**
	$\mathbf{v_{sdnom}}$	$v_{sdnom}=v_{sd}*\Omega$	**38.40**	**plf**
Direction 2	Height of Story	H_1	6.00	ft
E-W	Dead Load	D_L	16.00	psf
	Length of Building	L_{B1}	12.00	ft
		L_{B2}	41.06	ft
Applied Load	Wind	$W_L=(12.027psf)*H_1$	72.16	plf
	Seismic	$W_S=0.2*D_L*L_{B1}$	38.40	plf
V_{max}	Wind	$V_{wmax}=W_L*L_{B2}/2$	1481.55	lb
	Seismic	$V_{smax}=W_s*L_{B2}/2$	788.38	lb
v_{wd}	Wind	$v_{wd}=V_{wmax}/L_{B1}$	123.46	plf
v_{sd}	Seismic	$v_{sd}=V_{smax}/L_{B1}$	65.70	plf
Nominal Capacities	Ω		2	
	$\mathbf{v_{wdnom}}$	$v_{wdnom}=v_{wd}*\Omega$	**246.92**	**plf**
	$\mathbf{v_{sdnom}}$	$v_{sdnom}=v_{sd}*\Omega$	**131.40**	**plf**

***Use 3/4" T&G, 8d nails @ 6 inch edge nail spacing**

***Use 15 gauge staples @ 12 inches for field stapling**

CONNECTIONS

REFER TO PAGE S-531

C15X33.9 TO C8X11.5 INTERMODULAR CONNECTION

LOADS

$W_{wind.}$	3146	lb	* (Wind Governs)
$W_{seismic}$	1857.6	lb	

1/2" A307 threaded bolt

ALLOWABLE SHEAR STRENGTH

F_{nv}	27	ksi	
A_b	0.196	in^2	
ϕ	0.75		
$R_n = \varphi * F_{nv} * A_b$	3976.08	lbs	
3976.08 >	3146.07		**OK**

AVAILABLE TENSILE STRENGTH

frv	16.0	ksi	(WIND LOAD)	
F'nt	22.89	≤	**45**	
$R_n = \varphi * F'_{nt} * A_b =$	3371	lbs		**OK**

NORTH WALL TIMBER STUDS

Simpson Strong-Tie A34 Connection Framing Angle

Max Load =	490	psf	>	50.03	psf	**OK**

JOIST HANGER CONNECTION

Simpson Strong-Tie LSSU135 Sloped Hanger

Max Load =	1275	psf	>	38	psf	**OK**

C3X3.5 TO L8X4X1/2 : TOP & BOTTOM CONNECTION

LOADS

$W_{wind.N-s}$	3146.07	lb	73.164396	plf	*(Wind Governs)
$W_{seismic}$	1857.6	lb	43.2	plf	

Tributary Width	12.00	ft
Wall Height = 'Beam Length'	6.083	ft

Industry Designed Filet Welded Connection

(2) 5/16" Ceramic Coated lag screw

UPLIFT

SA	=	546	ft^2			
$WIND_{lat}$	=	12.027	psf			
$WIND_{perp}$	=	12.027tan(24)		=	5.34	psf
	=	21.86	lb			
UPLIFT	=	2915.64	lb	over whole roof		
UPLIFT	=	208.26	lb	per column end		
GRAVITY	=	96.6	lb	per column end		

CAPACITY

TENSION	=	893	lb	per bolt
SHEAR	=	432	lb	per bolt
T&S	=	447	lb	per bolt

893	lb	>	208	lb	**OK**
431	lb	>	21.86	lb	**OK**

DECK

DECK GIRDER CALCULATIONS

DFL No. 2 - 4x6

Length	96	in
Depth	5.5	in
Width	3.5	in
Area	19.25	in^2
S_x	17.65	in^3
I_x	48.53	in^4

Reference Design Values

F_b	900	psi
F_v	180	psi
E	1600000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_{M_b}	C_t	C_F	C_i	C_r	C_{M_v}
1	0.85	1	1.3	1	1	0.97

Structural Analysis

Dead Load	1.96	psf
Live Load	100	psf
W: D+L	135.95	plf
M_{load}:	13050.88	lb-in

O.C. spacing	16	in

Design Calculations

Bending Check

$Depth_{nominal}$	4	in
$Width_{nominal}$	2	in
Dn/Wn :	2	OK
C_L	1	
F'_b	$F_b*C_M*C_t*C_D*C_r*C_F*C_i*C_L$	
F'_b	994.5	psi
f_b : M/S	739.6	psi
$f_b<F'_b$?	OK	

Deflection Check

Length/360	0.267	in.
L	11.111	lb/in
Δ	5*L*Length^4 /(384*E*I)	
Δ	0.158	in
Δ < Length/360?		OK

Shear Check

F'v	Fv *CD*CM*Ct*Ci	
F'v =	174.6	psi
V: wL/2	543.79	lb
f_v : 3V/2A	42.4	psi
$f_v<F'_v$?	OK	

DECK JOIST CALCULATIONS

DFL No. 2 - 2x6

Length	48	in
Depth	5.5	in
Width	1.5	in
Area	8.25	in^2
S_x	7.56	in^3
I_x	20.80	in^4

Reference Design Values

F_b	900	psi
F_v	180	psi
E	1600000	psi

Adjustment Factors - NDS Table 4.3.1

C_D	C_{M_b}	C_t	C_F	C_i	C_r	C_{M_v}
1	0.85	1	1.3	1	1.15	0.97

Structural Analysis

Dead Load	1.96	psf
Live Load	100	psf
W: D+L	135.95	plf
M_{load}:	3262.72	lb-in

O.C. spacing	16	in

Design Calculations

Bending Check

$Depth_{nominal}$	6	in
$Width_{nominal}$	2	in
Dn/Wn :	3	OK
C_L	1	
F'_b	$F_b*C_M*C_t*C_D*C_r*C_F*C_i*C_L$	
F'_b	1143.675	psi
f_b : M/S	431.4	psi
$f_b<F'_b$?	OK	

Deflection Check

Length/360	0.133	in.
L	11.111	lb/in
Δ	5*L*Length^4 /(384*E*I)	
Δ	0.023	in
Δ < Length/360?	OK	

Shear Check

F'v	Fv *CD*CM*Ct*Ci	
F'v	174.6	psi
V: wL/2	271.89	lb
f_v : 3V/2A	49.4	psi
$f_v<F'_v$?	OK	

DECK SLATS

TIGER DECK

Length	16	in
Depth	3.44	in
Width	0.94	in
Area	3.2336	in^2
S_x	1.85	in^3
I_x	3.19	in^4

Bending Strength (psi @ 12%)	16620
Max Crushing Strength (psi @ 12%)	10320
Weight (lb/cu.ft.)	77

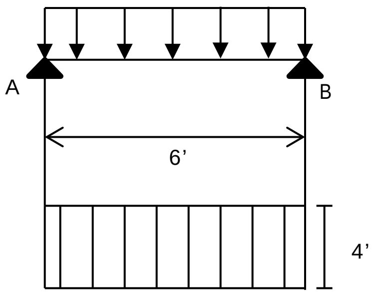
JOIST SPACING (in)	ALLOWABLE LOAD (psf)	MAX DEFLECTION	MAX FLEXURAL STRESS (psi)	Δ
12	40	0.00022	37	L/54096
16	40	0.0007	66	L/22822
19.2	40	0.00145	95	L/13207
24	40	0.00355	148	L/6762
12	60	0.00033	58	L/38085
16	60	0.00105	99	L/15215
19.2	60	0.00218	143	L/6805
24	60	0.00532	224	L/4508
12	90	0.0005	84	L/24043
16	90	0.00158	149	L/10143
19.2	90	0.00327	215	L/5870
24	90	0.00799	335	L/3005
16	*100*	*0.00176*	*165.6*	*L/9090*

THROUGH EXTRAPOLATION:

FLEXURE	165.6	psi	<	335	psi	OK
DEFLECTION	L/9090		<	L/360		OK

DECK FOUNDATION

D	L	Total	Trib B	w
(psf)	(psf)	(psf)	(ft)	(plf)
20.00	100	120.00	4.00	480



w = 480 plf

L = 8.00 ft

$R_A = R_B =$ 1920 LBS

AT 1500$_{PSF}$ BEARING PRESSURE THIS REQUIRES: 1.3 SQ FT FOR FOOTINGS

		QTY	AREA	
A:	SEISMIC PIERS	0	4	SQ FT
	STANDARD PIERS	2	1.8	SQ FT
		3.6	SQ FT	PROVIDED
		533.3	PSF PER FOOTING	

AT 1500$_{PSF}$ BEARING PRESSURE THIS REQUIRES: 1.3 SQ FT FOR FOOTINGS

		QTY	AREA	
B:	SEISMIC PIERS	0	4	SQ FT
	STANDARD PIERS	2	1.8	SQ FT
		3.6	SQ FT	PROVIDED
		533.3	PSF PER FOOTING	

DECK

DECK CONNECTIONS

SLAT TO JOIST:

STANDARD: (2) No. 8 screws per slat per supporting joist

JOIST TO GIRDER:

16 penny nails NOTE: Blocking at ~ 4' intervals for rigidity

GIRDER TO SUPPORT:

Simpson Strong Tie: GLS & GLT

HOUSE LEDGER:

N/A – self supported deck via piers

SUPPORT TO PIER:

Central Piers: Marriage Top (201) to 4x6 In Place Girder

AWNINGS

DESIGN WIND LOAD

ASCE 7-02 6.4.2.2: WIND DESIGN METHOD I

$$P_{net}=\lambda * I * P_{net} * 30$$

BASIC WIND LOAD

Angle (<7°)	4.5	°
Table 6.3 for permeable components & cladding (6.4.3)		
I	1	
λ	1.21	
Basic Wind Speed	90	mph
P_{net30}	**-19.8**	**psf**
(For Zone 2 @ 100 ft^2)		
P_{net30}	-10	psf
(Minimum allowed per 6.4.2.2.1)		
Approximate Effective Area	400	ft^2
P_{net30} =	**-19.8**	**psf**

AWNINGS

MEMBER A (114 IN): HSS 2X2X1/8

PROPERTIES - *AISC Shapes Database*

A_g	0.84	in^2	**Z**	0.584	in^3	**r**	0.761	in
d	2	in	**I**	0.486	in^4	**KL/r**	149.8	in/in
L_{Member}	9.5	ft	**E**	29000000	psi	**Ω**	1.67	
			F_y	46	ksi			

EFFECTIVE AREA

Effective Width	50	in	WIND	-19.8	psf
Effective Length	114	in	*(ASCE 7-02 Wind Design Method I, section 6.4.2.2)*		
A_{eff}	39.6	ft^2			

EFFECTIVE WIND LOAD

1/2 tributary area of wind load (assume aspect ratio 2:1)

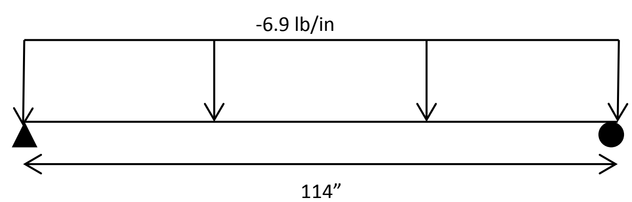
$WIND_{eff} = A_{eff} * WIND$	=	-783.8	lb
$W = WIND_{eff} / \text{Member } L$	=	-6.875	lb/in

DEFLECTION CHECK

$\Delta_{allowable}$	=	L/80	1.43	in	*(conservative)*
Δ_{max}	=	$5WL^4/(384EI)$	1.07	in	

Δ_{max} < $\Delta_{allowable}$ **OK**

FLEXURE



Yielding

M_n	=	F_y*Z	=	26.9	kip-in
M_{allow}	=	Mn/Ω	=	1.3	kip-ft
M_{max}	=	$WL^2/8$	=	0.9	kip-ft

M_{max} < M_{allow} **OK**

FLEXURAL BUCKLING -

Due to tension of shade fabric on beams distributed as axial load on Member A

Assume: *Fabric weight << than tension & wind loads*

75 lb tension/gromet (gromets attached @ 6" O.C. on fabric and weaved into beams)

Tension Force	=	*150*	*plf*
Length of Fabric	=	*139*	*in*
Length of Beam	=	*114*	*in*
Intermediate Braces, Member A	=	*3*	*members*
Total Axial Load on Member A	=	*579*	*lb*
$\boldsymbol{F_{actual}}$	=	***0.689***	***ksi***

When KL/r > $4.71*\sqrt{(E/F_y)}$ = 118.3 in/in

F_{cr} = $0.877*F_e$

where F_e = $\frac{\pi^2 E}{(KL/r)^2}$ = 12.8 ksi

F_{cr} = 11.19 ksi

$\boldsymbol{F_n = F_{cr}/\Omega}$ **= 6.7 ksi**

F_n	=	6.7	ksi
F_{actual}	=	0.689	ksi

F_{actual} < F_n **OK**

AWNINGS

TRUSS BRACE STIFFENER (186 IN): HSS 2X2X1/8

PROPERTIES

A_g	0.84	in^2	**I**	0.486	in^4	**r**	0.761	in
d	2	in	**E**	29000000	psi	KL_{eff}/r	122.2	in/in
Member L	15.5	ft	F_y	46	ksi	**Ω**	1.67	
L_{eff}	7.75	ft						

EFFECTIVE AREA

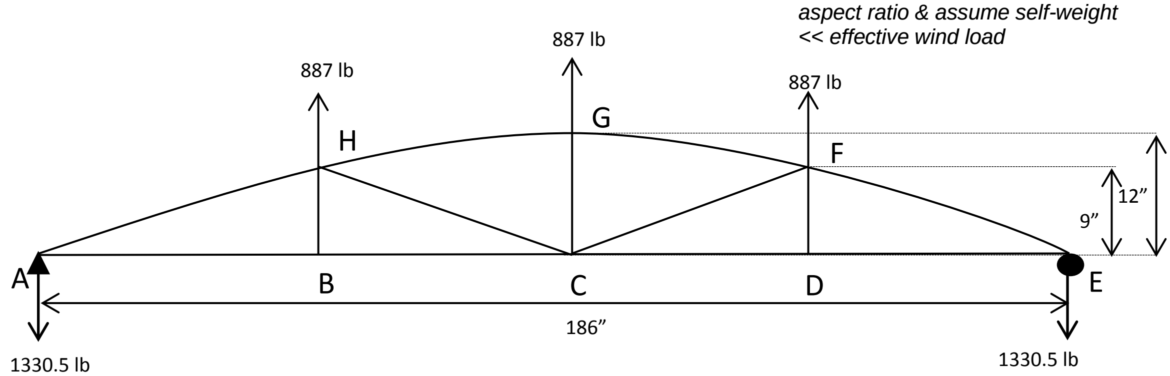
Effective $Width_1$	47	in	WIND	19.8	psf
Effective $Width_2$	57	in	*(NOTE: Positive for upward direction)*		
Member L	186	in	*(ASCE 7-02 Wind Design Method I, section 6.4.2.2)*		
A_{eff}	134.3	ft^2			

EFFECTIVE WIND LOAD

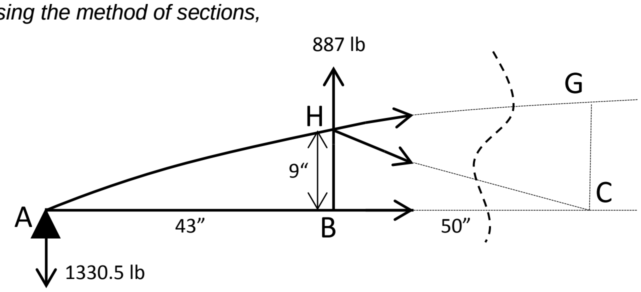
1/2 tributary area of wind load (assume aspect ratio 2:1)

$WIND_{eff} = A_{eff}$*WIND = 2660 lb

NOTE: Conservative for assumed aspect ratio & assume self-weight << effective wind load



Using the method of sections,



Due to Symmetry, end reactions are equal.

$A_y = B_y = (3*887)/2 =$ 1330.50 lb

Axial Load in bottom chord of truss:

$\Sigma F_x = 0 =$ HG + HC_X +BC

$\Sigma M_H = 0 =$ BC (9") + A (43") = BC (9") + (1330.5lb) (43")

BC = -(43"/9")*1330.5lb

$F_{axial} =$ BC = -6356.83 = **6357 lb *(compression)***

FLEXURAL BUCKLING

Due to axial force from uplift

When KL/r		>	*4.71*√ (E/F_y)* =	118.3	in/in	
F_{cr}	=	$0.877 * F_e$				
where F_e		=	$\frac{\pi^2 E}{(KL/r)^2}$	=	19.2	ksi
F_{cr}	=	16.81	ksi			

F_n	=	F_{cr} / Ω	=	10.1	ksi
F_{actual}	=	F_{axial} / A_g	=	7.57	ksi

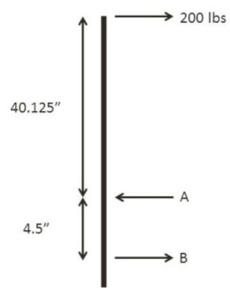
$F_{actual} < F_n$ **OK**

RAILINGS

DESIGN BOLT STRENGTH FOR TYPICAL GUARDRAIL POST

(2) 3/8" A307 threaded bolt

DEMAND TENSILE STRENGTH



LIVE LOAD 200 lb

$\Sigma M_A = 0 \Rightarrow$ B=[LOAD*40.125"]/4.5"

$Bolt_B$= **1961.1** **lb**

$\Sigma F_X = 0 \Rightarrow$ A=B+LOAD

$Bolt_A$= **2161.1** **lb**

ALLOWABLE TENSILE STRENGTH

F_{nt}	45	ksi	(NOMINAL TENSILE STRENGTH)
A_b	0.110	in^2	
Ω	2.00		
$R_n = F_{nt} * A_b / \Omega$	**2485.05**	**lb**	

(AISC Steel Construction Manual EQ. J3-1)

Check		R_n	>	Demand			
2485	lb		>	1961	lb	$Bolt_B$	**OK**
2485	lb		>	2161	lb	$Bolt_A$	**OK**

CONSTRUCTION DRAWINGS

SANTA CLARA UNIVERSITY RADIANT HOUSE

RADIANT HOUSE
SANTA CLARA UNIVERSITY

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U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON

CLIENT: U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON 2013 WWW.SOLARDECATHLON.GOV

MARK	DATE	DESCRIPTION
	08/22/13	DOE AS BUILT SUBMISSION
	08/12/13	AS BUILT REVISIONS
	08/06/13	CD CITY INSPECTION 3
	07/26/13	CD CITY INSPECTION 2
	07/19/13	CD CITY INSPECTION
	05/10/13	CD RESUBMISSION 2
	04/04/13	CD RESUBMISSION
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: Author
CHECKED BY: Checker

SHEET TITLE:
COVER SHEET

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GENERAL STRUCTURAL NOTES

1. GENERAL NOTES AND TYPICAL DETAILS APPLY TO ALL STRUCTURAL FEATURES UNLESS OTHERWISE SHOWN OR NOTED

2. IF CERTAIN FEATURES ARE NOT FULLY SHOWN OR CALLED FOR ON THE DRAWINGS OR SPECIFICATIONS, THEIR CONSTRUCTION SHALL BE OF THE SAME CHARACTER AS FOR SIMILAR CONDITIONS.

3. THE PROJECT SPECIFICATIONS FORM A PART OF THE CONTRACT DOCUMENTS

4. SPECIFICATIONS, CODES AND STANDARDS NOTED IN THE CONTRACT DOCUMENTS SHALL BE OF THE LATEST EDITION UNLESS OTHERWISE NOTED.

5. CONTRACTOR SHALL FIELD VERIFY ALL JOB CONDITIONS AND DIMENSIONS. VARIATIONS THEREOF FROM THE DRAWINGS MUST BE REPORTED TO THE STRUCTURAL ENGINEER. DETAILS INDICATED ON THE DRAWINGS SHALL CONFORM TO BEST PRACTICE AND SHALL BE THE CONTRACTOR'S RESPONSIBILITY.

6. DIMENSIONS SHALL NOT BE SCALED

7. ALL WORK SHALL CONFORM TO THE MINIMUM STANDARDS OF THE FOLLOWING CODES: THE 2010 BUILDING CODE, THE 2013 SOLAR DECATHLON BUILDING CODE, THE INTERNATIONAL RESIDENTIAL CODE (2012 EDITION), AND ANY OTHER REGULATING AGENCIES WHICH HAVE THE AUTHORITY OVER ANY PORTION OF THE WORK, INCLUDING THE STATE OF CALIFORNIA DIVISION OF INDUSTRIAL SAFETY, AND THOSE CODES AND STANDINGS LISTED IN THESE NOTES AND THE SPECIFICATIONS.

8. MANUFACTURED MATERIALS SHALL BE APPROVED BY THE CONTRACTOR PRIOR TO THEIR USE. ALL REQUIREMENTS OF THOSE APPROVALS SHALL BE FOLLOWED.

9. NON-STRUCTURAL FEATURES NOT FULLY SHOWN OR NOTED ON THE STRUCTURAL DRAWINGS INCLUDE BUT ARE NOT LIMITED TO:

   A. ARCHITECTURAL FEATURES
      I. SIZE AND LOCATIONS OF ALL DOOR AND WINDOW OPENINGS.
      II. SIZE AND LOCATION OF ALL NON-BEARING PARTITIONS
      III. SIZE AND LOCATION OF ALL FLOOR DRAINS, SLOPES, DEPRESSED AREAS
      IV. CHANGES IN LEVEL, CHAMFERS, GROOVES, INSERTS, ETC.
      V. SIZE AND LOCATION OF ALL FLOOR AND ROOF OPENINGS
      VI. DIMENSIONS NOT SHOWN IN THE STRUCTURAL DRAWINGS
   B. MECHANICAL, PLUMBING AND ELECTRICAL FEATURES
      I. PIPE RUNS, SLEEVES, HANGERS, TRENCHES, WALL, ROOF AND FLOOR OPENINGS, ETC. NOT SHOWN OR NOTED.
      II. ELECTRICAL CONDUIT RUNS, BOXES, OUTLETS IN WALLS
      III. ANCHORAGE AND BRACING FOR ELECTRICAL, MECHANICAL OR PLUMBING EQUIPMENT
      IV. ANCHOR BOLTS FOR MOTOR MOUNTS
      V. SIZE AND LOCATION OF MACHINE AND EQUIPMENT BASES

10. OPENINGS, POCKETS, ETC. SHALL NOT BE PLACED IN STRUCTURAL MEMBERS UNLESS SPECIFICALLY DETAILED ON THE STRUCTURAL DRAWINGS. NOTIFY THE STRUCTURAL ENGINEER WHEN WORK REQUIRES OPENINGS POCKETS, ETC. IN STRUCTURAL MEMBERS NOT SHOWN ON THE STRUCTURAL DRAWINGS.

11. THE CONTRACTOR SHALL BE RESPONSIBLE FOR COORDINATING THE WORK OF ALL TRADES AND SHALL CHECK ALL DIMENSIONS AND HOLES AND OPENINGS REQUIRED IN STRUCTURAL MEMBERS. ALL DISCREPANCIES SHALL BE CALLED TO THE ATTENTION OF THE ARCHITECT AND SHALL BE RESOLVED BEFORE PROCEEDING WITH WORK.

12. THE CONTRACT DOCUMENTS REPRESENT THE FINISHED STRUCTURE. THEY DO NOT INDICATE THE METHOD OF CONSTRUCTION. THE CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT LIFE AND PROPERTY DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE BUT ARE NOT LIMITED TO BRACING AND SHORING FOR LOADS DUE TO CONSTRUCTION EQUIPMENT AND MATERIALS. OBSERVATION VISITS TO THE SITE BY THE STRUCTURAL ENGINEER SHALL NOT INCLUDE INSPECTION OF THE ABOVE ITEMS.

13. CONTRACTOR SHALL BE RESPONSIBLE FOR ALL SAFETY PRECAUTIONS AND THE METHODS, TECHNIQUES, SEQUENCES OR PROCEDURES REQUIRED TO PERFORM THE CONTRACTORS WORK. THE STRUCTURAL ENGINEER HAS NO SUPERVISORY AUTHORITY OR DIRECT RESPONSIBILITY FOR THE SPECIFIC WORKING CONDITIONS AT THE SITE AND/OR FOR ANY HAZARDS RESULTING FROM THE ACTIONS OF ANY TRADE CONTRACTOR.

14. CONSTRUCTION MATERIALS SHALL BE SPREAD OUT. IF PLACED ON FRAMED FLOORS OR ROOFS. LOAD SHALL NOT EXCEED THE DESIGN LIVE LOAD PER SQUARE FOOT. PROVIDE ADEQUATE SHORING WHERE OVERLOAD IS ANTICIPATED.

15. THE LATERAL SYSTEM OF THE STRUCTURE IS DESIGNED WITH LATERAL RESTRAINT AT EACH LEVEL. STRUCTURAL WALLS OR FRAMES ARE NOT LATERALLY SELF SUPPORTING UNTIL THE ENTIRE DESIGN LATERAL RESTRAINT SYSTEM IS IN PLACE. THE CONTACTOR SHALL PROVIDE TEMPORARY BRACING FOR THE STRUCTURE AND STRUCTURAL COMPONENTS UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE PLANS.

16. UNLESS OTHERWISE NOTED, FOLLOW MANUFACTURER'S RECOMMENDATIONS FOR ALL STRUCTURAL PRODUCTS USED IN THIS PROJECT

17. CONTRACTOR INITIATED CHANGES SHALL BE SUBMITTED IN WRITING TO THE ARCHITECT AND STRUCTURAL ENGINEER FOR APPROVAL PRIOR TO FABRICATION OR CONSTRUCTION OR CONSTRUCTION. CHANGES SHOWN ON SHOP DRAWINGS ONLY WILL NOT SATISFY THIS REQUIREMENT.

18. DRAWINGS INDICATED GENERAL AND TYPICAL DETAILS OF CONSTRUCTION, WHERE CONDITIONS ARE NOT SPECIFICALLY INDICATED BUT ARE OF SIMILAR CHARACTER TO DETAILS SHOWN, SIMILAR DETAILS OF SHALL CONSTRUCTION SHALL BE USED, SUBJECT TO REVIEW AND APPROVAL BY THE ARCHITECT AND THE STRUCTURAL ENGINEER.

INSPECTION, OBSERVATION, AND TESTING

1. THIS SECTION SUMMARIZES THE SPECIFIC REQUIREMENTS OF CHAPTER 17 OF THE 2010 CBC AS THEY APPLY TO THE SPECIAL INSPECTION, STRUCTURAL OBSERVATION AND TESTING OF THE STRUCTURAL PORTIONS OF THE PROJECT.

2. THE TYPES OF WORK LISTED IN THE FOLLOWING TABLE SHALL BE INSPECTED IN ACCORDANCE WITH CBC 1701.5 WHEN INDICATED AS EITHER "CONTINUOUS" OR "PERIODIC". ALL TESTS AND INSPECTIONS SHALL BE PERFORMED BY A CERTIFIED SPECIAL INSPECTION FROM AN INDEPENDENT TESTING AGENCY WHO IS EMPLOYED BY THE OWNER OR AGENT OF THE AGENT AND NOT THE CONTRACTOR.

   A. THE SPECIAL INSPECTOR SHALL OBSERVE THE WORK ASSIGNED FOR CONFORMANCE WITH THE APPROVED DESIGN DRAWINGS AND SPECIFICATIONS.

   B. THE SPECIAL INSPECTOR SHALL FURNISH INSPECTION REPORTS TO THE BUILDING OFFICIAL, THE ENGINEER OF RECORD, AND OTHER DESIGNATED PERSONS. ALL DISCREPANCIES SHALL BE BROUGHT TO THE IMMEDIATE ATTENTION OF THE CONTRACTOR FOR CORRECTION, THEN, IF UNCORRECTED, TO THE PROPER DESIGN AUTHORITY AND TO THE BUILDING OFFICIAL.

   C. THE SPECIAL INSPECTOR SHALL SUBMIT A FINAL SIGNED REPORT STATING WHETHER THE WORK REQUIRING INSPECTION WAS, TO THE BEST OF THE SPECTOR'S KNOWLEDGE, IN CONFORMANCE WITH THE APPROVED PLANS AND SPECIFICATIONS AND THE APPLICABLE WORKMANSHIP PROVISIONS AND STANDARDS OF QUALITY OF THE 2010 CBC

   D. CONTINUOUS AND PERIODICAL SPECIAL INSPECTIONS SHALL BE IN ACCORDANCE WITH CBC 1701.6

3. TYPES OF WORK LISTED IN THE FOLLOWING TABLE SHALL BE OBSERVED DURING PERIODIC SITE VISITS BY THE STRUCTURAL ENGINEER WHEN INDICATED AS "STRUCTURAL OBSERVATION". CONTRACTOR IS RESPONSIBLE FOR NOTIFYING STRUCTURAL ENGINEER 48 HOURS BEFORE WORK IS READY FOR OBSERVATION. THESE VISITS DO NOT CONSTITUTE SPECIAL INSPECTION UNDER SECTION 1701 OF THE CBC

4. IN ADDITION TO INSPECTION BY THE SPECIAL INSPECTOR, THE STRUCTURAL ENGINEER WILL REVIEW THE CONSTRUCTION FOR GENERAL CONFORMANCE WITH THE STRUCTURAL DRAWINGS. THE CONTRACTOR SHALL NOTIFY THE STRUCTURAL ENGINEER AT LEAST FIVE WORKING DAYS PRIOR TO CONCEALING ANY STRUCTURAL ITEMS. THE STRUCTURAL ENGINEER WILL THEN DETERMINE IF A SITE VISIT IS APPROPRIATE. NOTIFICATIONS SHALL INCLUDE REINFORCEMENT AND EMBEDDED ITEMS, PRIOR TO CONCRETE PLACEMENT AND STRUCTURAL FRAMING AND PANEL SHEAR WALL PRIOR TO CONCEALMENT BY FIREPROOFING OR FINISH SURFACES.

5. THE CONTRACTOR SHALL HOLD A PRE-CONSTRUCTION MEETING INVOLVING THE STRUCTURAL ENGINEER AND THE SPECIAL INSPECTOR IN ORDER TO DISCUSS THE SPECIFIC REQUIREMENTS OF THIS SECTION.

DESIGN DATA

1. CODE: 2010 CALIFORNIA BUILDING CODE
   OCCUPANCY CATEGORY: II
   SITE CLASS: D

2. DESIGN LIVE LOADS:

AREA	DESIGN LIVE LOADS	REMARKS
ROOF:	20 PSF	PER SD 2013 BUILDING CODE
FLOORS:	50 PSF	PER SD 2013 BUILDING CODE
DECKS:	100 PSF	PER SD 2013 BUILDING CODE

3. EARHTQUAKE DESIGN LOADS:
   SEISMIC DESIGN CATEGORY: D
   EQUIVALENT LATERAL FORCE PROCEDURE USED
   ZIP CODE 95053

   Fa = 1.0 (FOR SITE CLASS D)
   Ss = 1.5
   Sms = Fa x Ss = 1.5
   Sds = 2 x Sms/3 = 1.000 G

LONGITUDINAL AND TRANSVERSE DIRECTION
R = 6.5
I = 1.0
CS = Sds/(R/I) = 0.154
W = EFFECTIVE SEISMIC DEAD LOAD
V = CS x W = 0.154 W (DESIGN BASE SHEAR)
P = 1.3
E = P x V = 0.200 W

4. WIND DESIGN LOADS:
   BASIC WIND SPEED 85 MPH
   EXPOSURE C

FOUNDATION NOTES

1. CONTRACTOR SHALL CONFORM TO THE REQUIREMENTS OF THE SOLAR DECATHLON OFFICIALS REGARDING SITE PREPARATION AND FOUNDATION

2. METAL PIER STANDS AND PIER AND PIER STAND CAPS SHALL BE MANUFACTURED BY CENTRAL PIER INC. MANUFACTURE AND INSTALLATION SHALL BE IN STRICT ACCORDANCE WITH THE CALIFORNIA STATE SUPPLEMENTAL CERTIFICATION REPORT FOR PIER LISTING NUMBER 186.5

3. ABS PIER PADS SHALL BE MANUFACTURED BY TIE-DOWN ENGINEERING. MANUFACTURE AND INSTALLATION SHALL BE IN STRICT ACCORDANCE WITH THE DETAIL DRAWINGS

4. FOUNDATION TYPE: TEMPORARY SEISMIC PIERS AND STANDARD PIERS FOR MODULAR STRUCTURES.

FOOTING DESIGN VALUES:

FOOTINGS	ALLOWABLE BEARING PRESSURES
SEISMIC PIER	1500 PSF
STANDARD PIER	1500 PSF

LEGEND

- PLYWOOD DIAPHRAGM
- REMOVABLE PLYWOOD ACCESS PANEL
- ST22
- DATUM POINT
- FX — FOOTING
- HOLDOWN
- 4X6 POST
- SHEAR WALL
- GRAVITY WALL
- GROUND PENETRATION HOLE

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U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON

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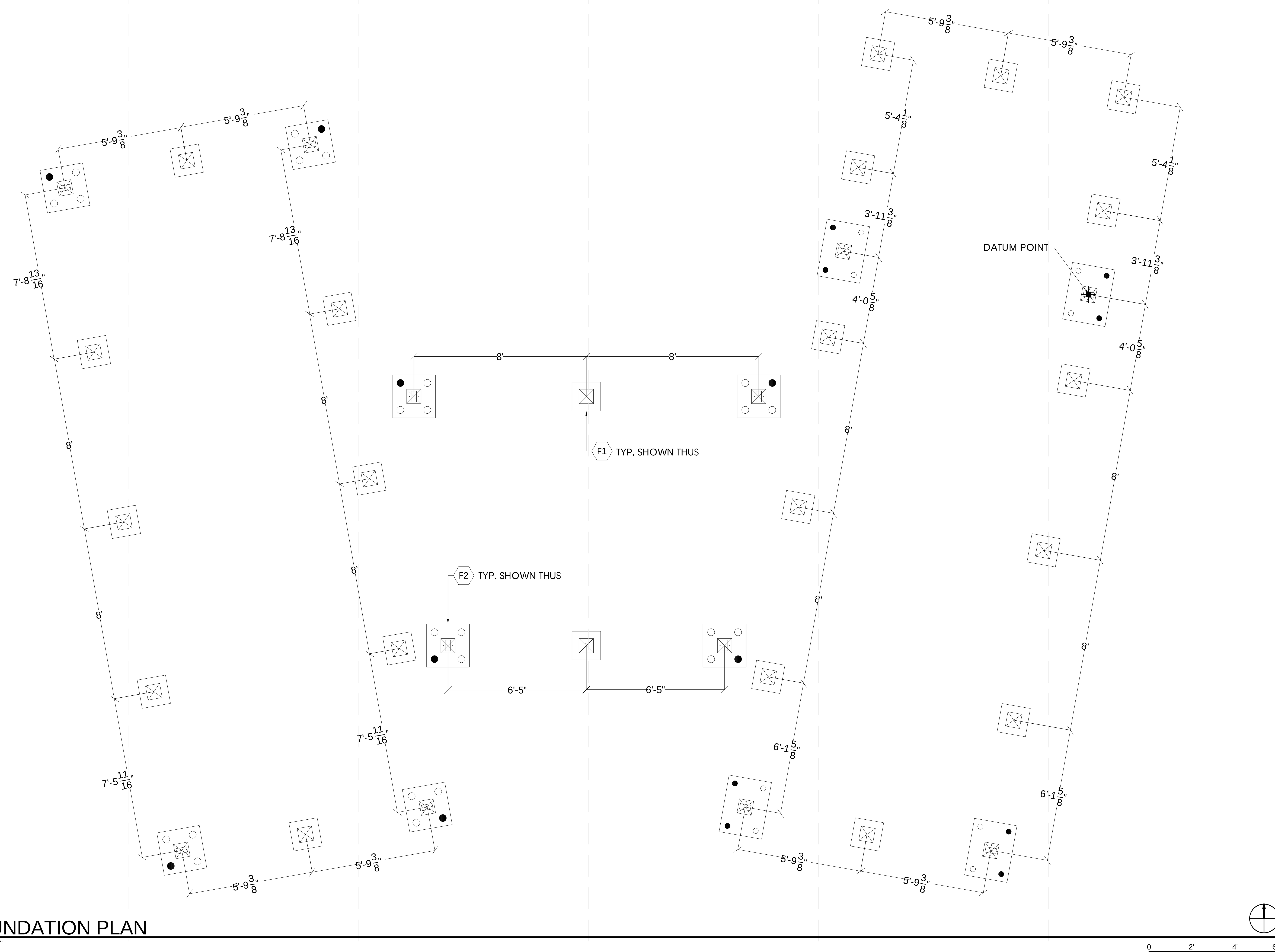
LOT NUMBER: 118
DRAWN BY: ML/KM
CHECKED BY: SCU

SHEET TITLE: STRUCTURAL NOTES AND SYMBOLS

S-001

9/5/2013 3:41:29 PM



A1 FOUNDATION PLAN
3/8" = 1'-0"

GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. S.A.D. FOR ALL DIMENSIONS AND FINISH ELEVATIONS NOT NOTED.
3. S.A.D. FOR ALL FINISHES, FIRE PROTECTION, THERMAL AND MOISTURE PROTECTION, WATERPROOFING, AND ACOUSTICAL REQUIREMENTS FOR ALL CEILINGS, WALLS, FLOORS, ROOF DECKS AND TERRACES.
4. SEE S-601 FOR FOOTING SCHEDULE.
5. PREPARE A LEVEL SURFACE AT THE LOCATION OF EACH PIER TO ENSURE FULL CONTACT FOR THE FOOTING PAD.
6. CONTRACTOR TO SELECT APPROPRIATELY SIZED PIERS BY DETERMINING THE PIER HEIGHT AT EACH SUPPORT LOCATION, MEASURE TO THE TOP OF THE PAD TO THE STEEL CHANNEL TO ENSURE THAT HEIGHT IS NO GREATER THAN 18".
7. FOUNDATION PIERS ARE TO SIT ON TOP OF LOAD RATED ABS PAD. SEE S-501 FOR TYPICAL STANDARD PIER DETAIL.
8. THE MAXIMUM ADJUSTMENT ON THE THREADED ROD ADJUSTER FOR THE STANDARD PIER IS 2" AND THE MAXIMUM ADJUSTMENT OF THE SEISMIC PIER IS 7". WHEN MORE HEIGHT IS NEEDED USE THE NEXT TALLER SIZE PIER SUPPORT.
9. THE STANDARD PIERS TIGHTEN ROD ADJUSTER AGAINST BOTTOM OF STEEL CHANNEL UNTIL SNUG PLUS 1/2 TURN.
10. ALL FOOTINGS FOR DECK SHALL BE SPACED NO FARTHER THAN 6 FEET APART. ALL GIRDERS SHALL HAVE FOOTINGS AT BOTH ENDS.
11. ALL GROUND PENETRATIONS SHALL BE A 38" IN DEPTH TO ALLOW PROPER ATTACHMENT OF 42" THREADED ANCHOR RODS TO THE SEISMIC PIER PLATES.

LEGEND

DATUM POINT
FX FOOTING TYPE
GROUND PENETRATION HOLE

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	04/04/13	CD RESUBMISSION
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
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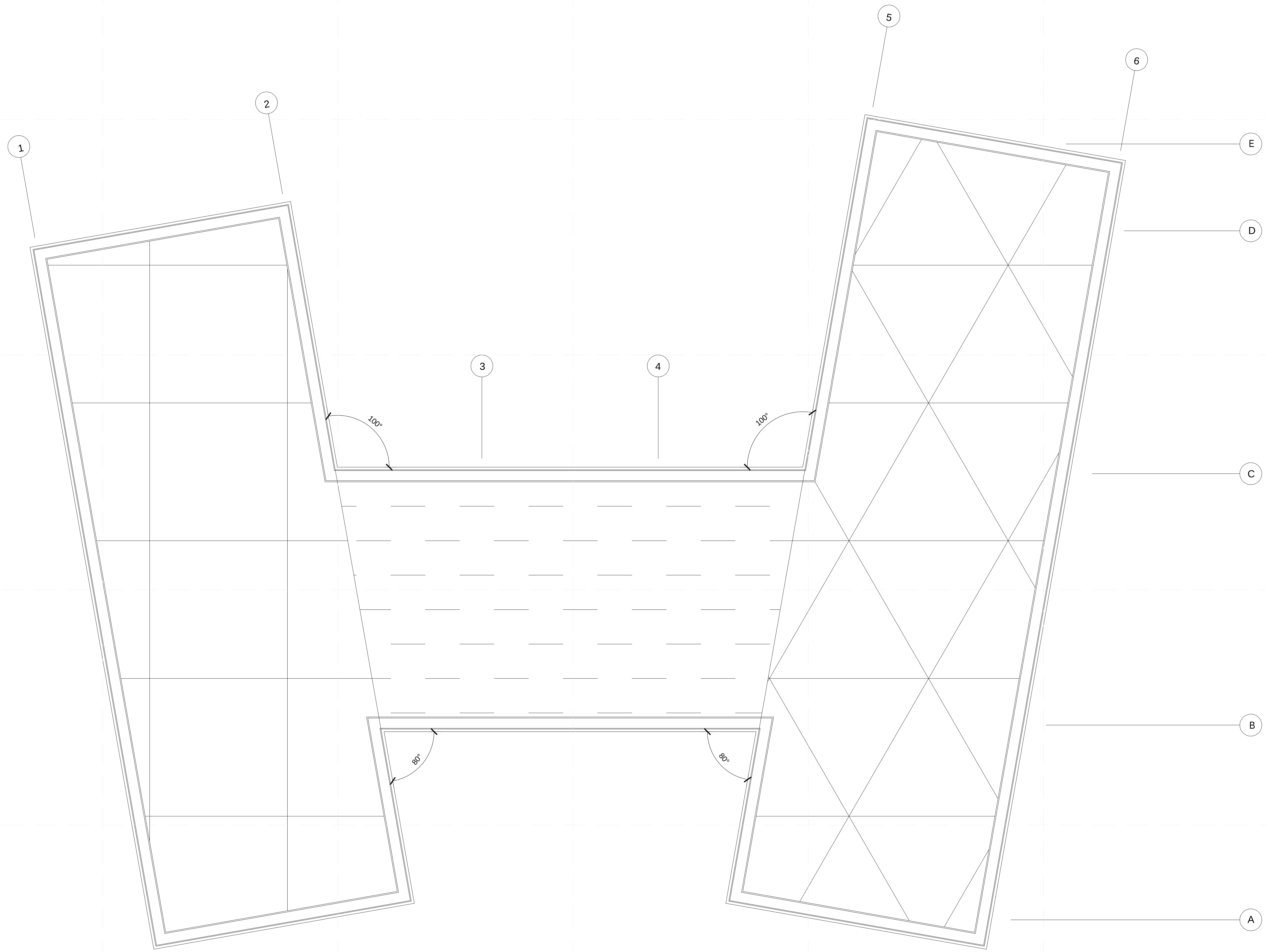
SHEET TITLE:
FOUNDATION PLAN

S-101

9/5/2013 9:53:02 AM



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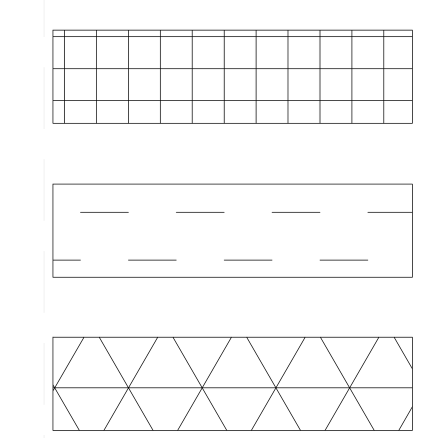

A1 MODULE SEPARATION
3/8" = 1'-0"

9/5/2013 9:57:29 AM

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LOT NUMBER: 118
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CHECKED BY: SCU
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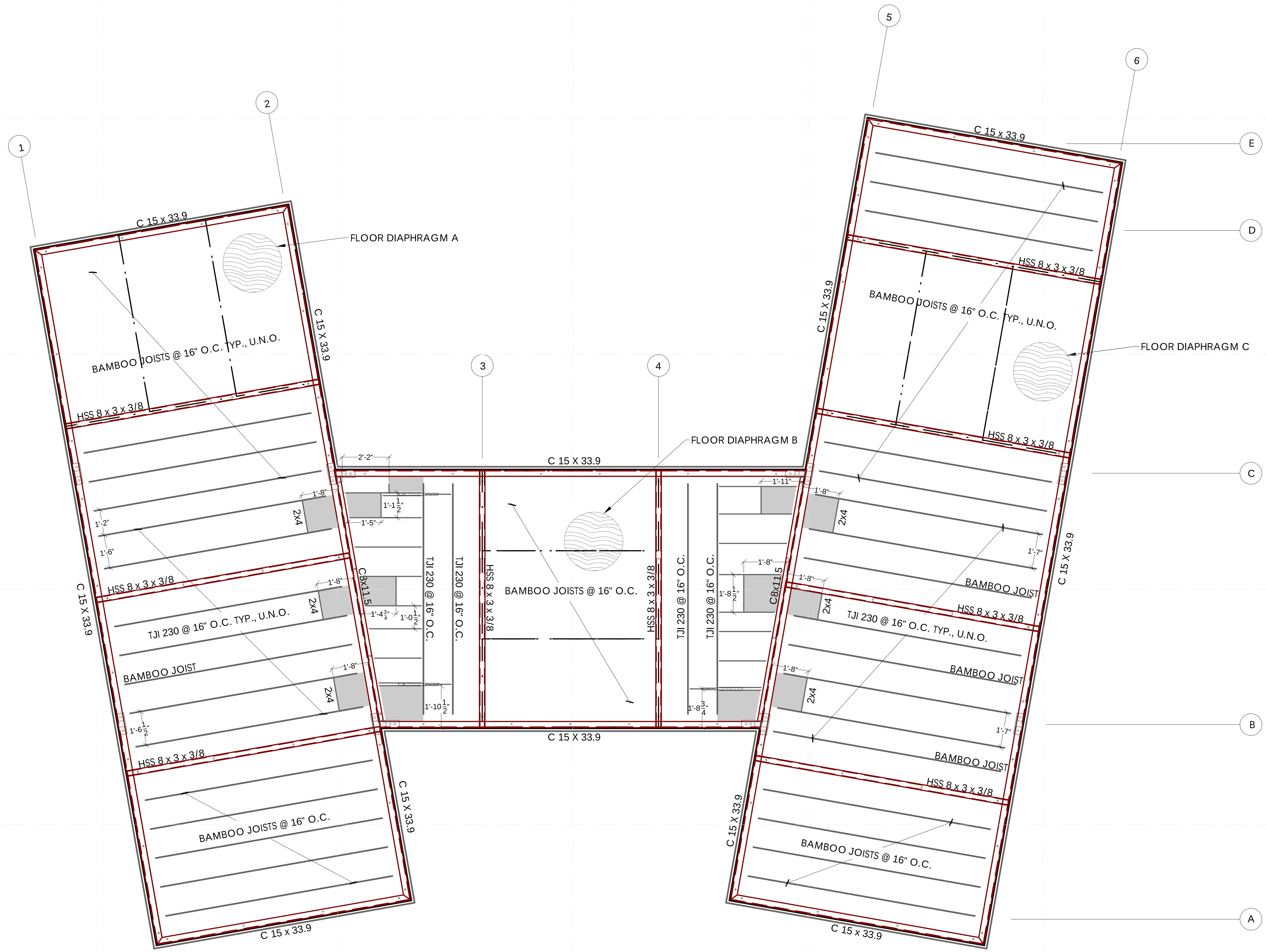
SHEET TITLE:
MODULE SEPARATION

S-102



A1 FLOOR FRAMING PLAN

3/8" = 1'-0"

9/5/2013 9:58:25 AM

GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. SEE S-601 FOR JOIST HANGER SCHEDULE.
3. SEE S-601 FOR DIAPHRAGM NAILING SCHEDULE.
4. ACCEPTABLE TO SUBSTITUTE TJI 230 FOR BAMBOO JOIST AS NECESSARY FOR NAILER.
5. 2 - 2X12 TRIMMED RIM JOIST NAILED TOGETHER SHALL RUN CONTINUOUSLY INSIDE C15X33.9.
6. A 2X4 SHALL RUN CONTINUOUSLY OVER ALL C8X11.5 AND HSS8X3X3/8.
7. ALIGN VERTICAL JOINT IN EACH BAY AS SHOWN FOR PLYWOOD DIAPHRAGM.
8. 2X4 TYP. LINES "2" TO "2.5" AND "4.5" TO "5" SPACING 16" O.C., U.N.O.

LEGEND

- PLYWOOD SUBFLOOR
- REMOVABLE PLYWOOD ACCESS PANEL
- ST22
- PLYWOOD BORDER

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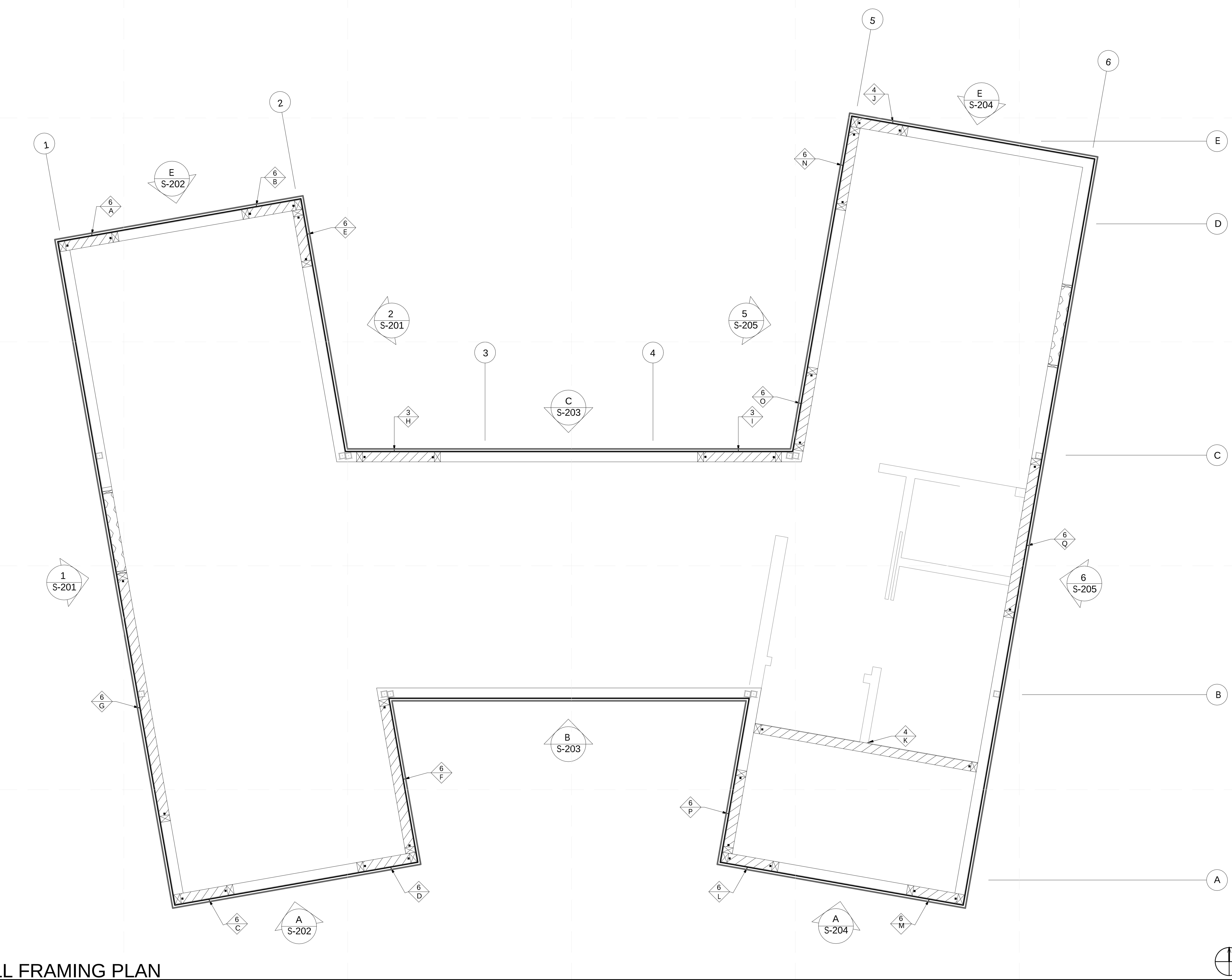
LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
FLOOR FRAMING PLAN

S-103

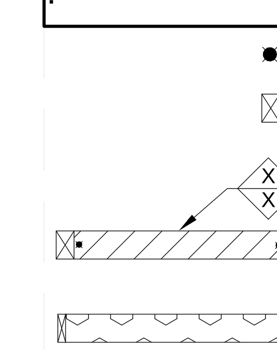
GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. THE TOP NUMBER CORRESPONDING TO A SHEAR WALL INDICATES THE EDGE NAILING OR STAPLING. BOTTOM NUMBER INDICATES THE SHEAR WALL.
3. SEE S-601 FOR SHEAR WALL SCHEDULE.
4. ALL EXTERIOR STUD WALLS ARE STAGGERED STUD 2X4 @ 16" O.C.



A1 WALL FRAMING PLAN
3/8" = 1'-0"

LEGEND



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LOT NUMBER: 118
DRAWN BY: ML
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SHEET TITLE:
WALL FRAMING PLAN

S-104

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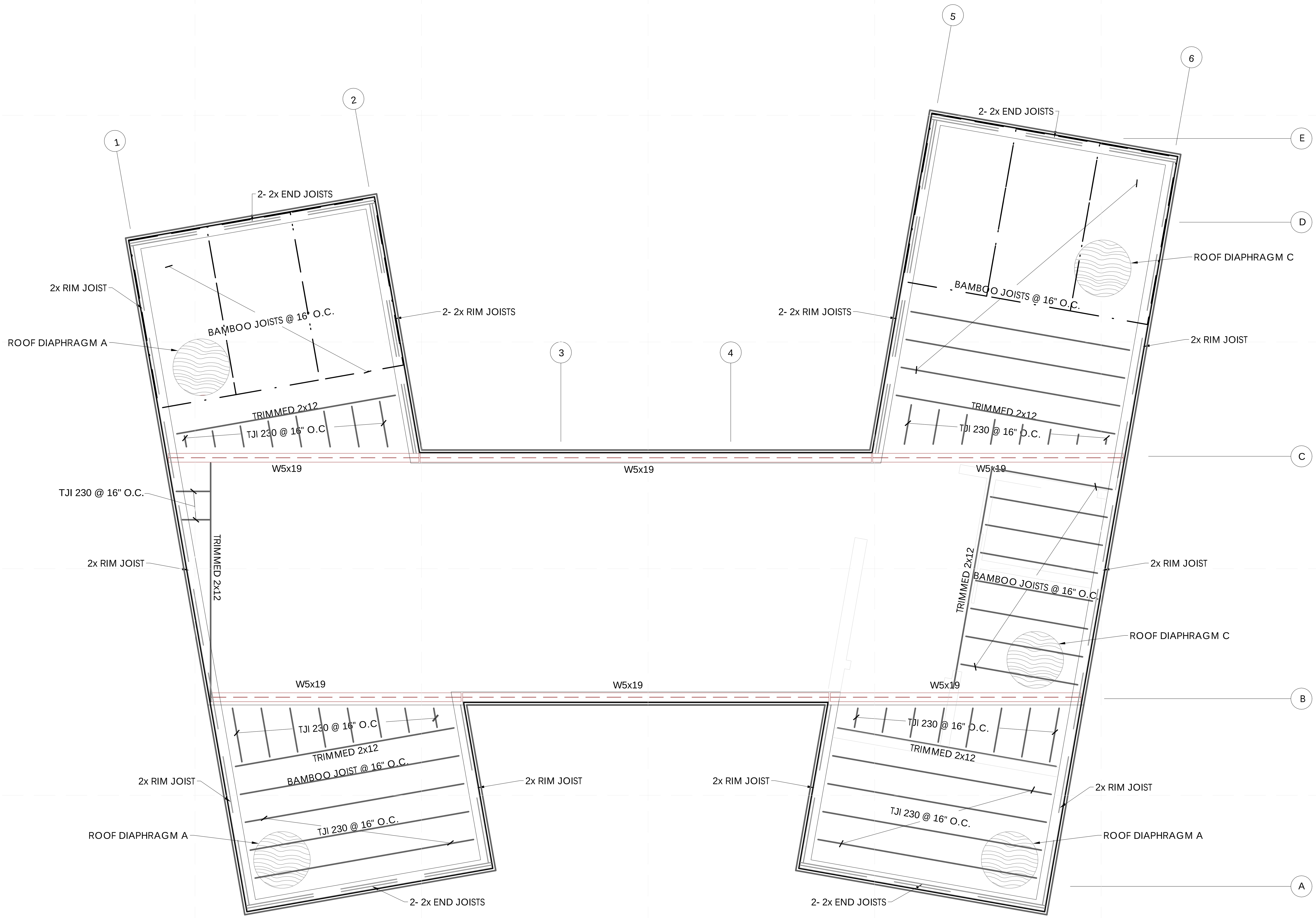

A1 LOWER ROOF FRAMING PLAN

3/8" = 1'-0"

0 2' 4' 6'

9/5/2013 10:00:25 AM

GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. SEE S-601 FOR JOIST HANGER SCHEDULE AND PLYWOOD DIAPHRAGM NAILING SCHEDULE.
3. BLOCK ENDS OF 2X12 JOISTS IN ROOF WHERE IT MEETS 1 - 2X RIM JOIST.
4. ACCEPTABLE TO SUBSTITUTE TJI 230 FOR BAMBOO JOIST AS NECESSARY FOR NAILER.
5. ALIGN VERTICAL JOINT IN EACH BAY AS SHOWN FOR PLYWOOD DIAPHRAGM.
6. ALL 2X RIM AND END JOISTS SHALL BE 9.5" TALL.

LEGEND

ROOF DIAPHRAGM

PLYWOOD BORDER

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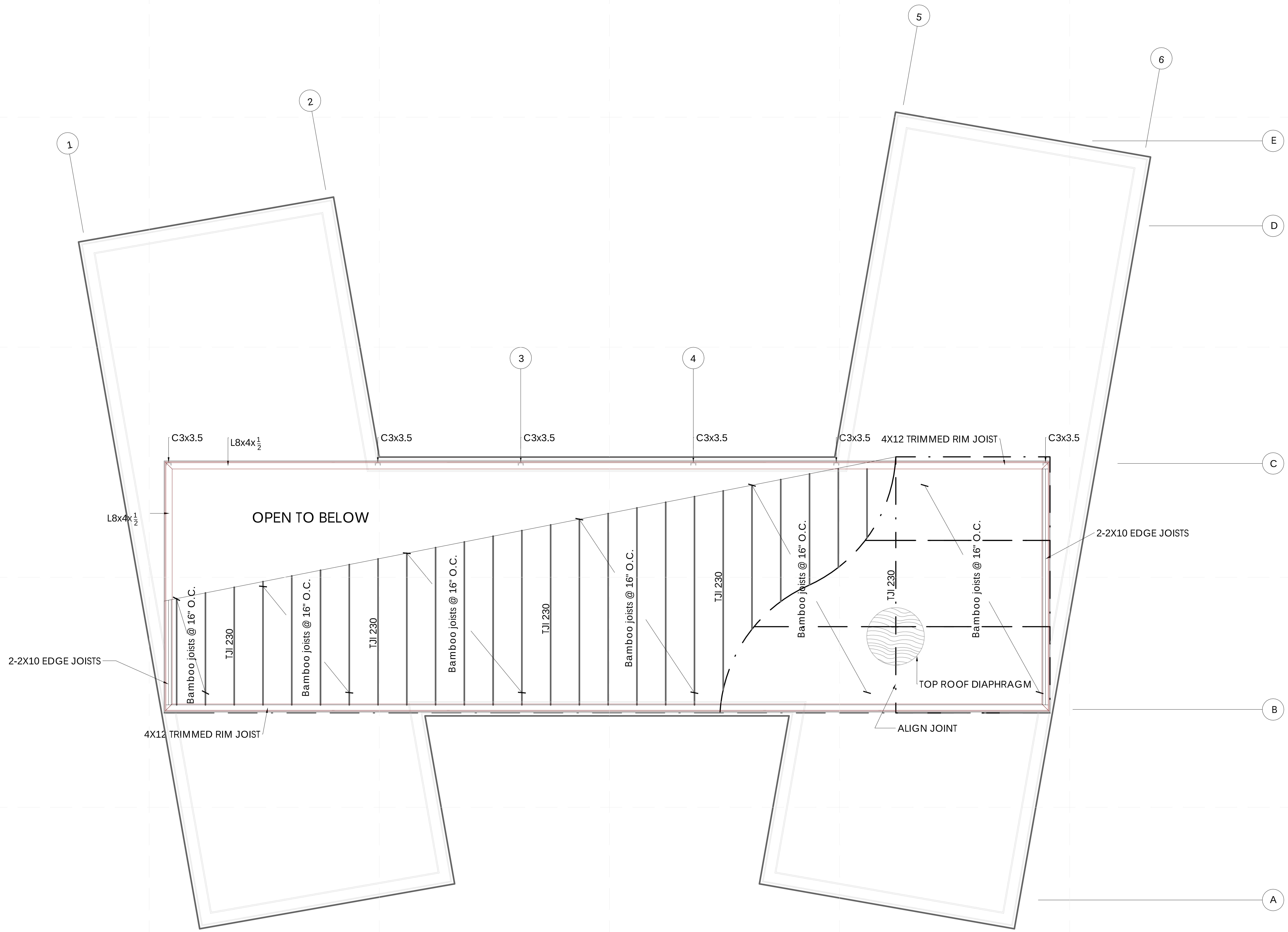
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
LOWER ROOF FRAMING

S-105



A1 UPPER ROOF MODULE FRAMING PLAN

3/8" = 1'-0"

GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. SEE S-601 FOR JOIST HANGER SCHEDULE.
3. SEE S-601 FOR PLYWOOD DIAPHRAGM SCHEDULE.
4. 1 - 4X12 TRIMMED RIM JOIST NAILED TOGETHER SHALL RUN CONTINUOUSLY INSIDE STEEL ANGLE.

LEGEND

ROOF DIAPHRAGM

PLYWOOD BORDER

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	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118

DRAWN BY: ML

CHECKED BY: SCU

SHEET TITLE:
UPPER ROOF MODULE FRAMING PLAN

S-106

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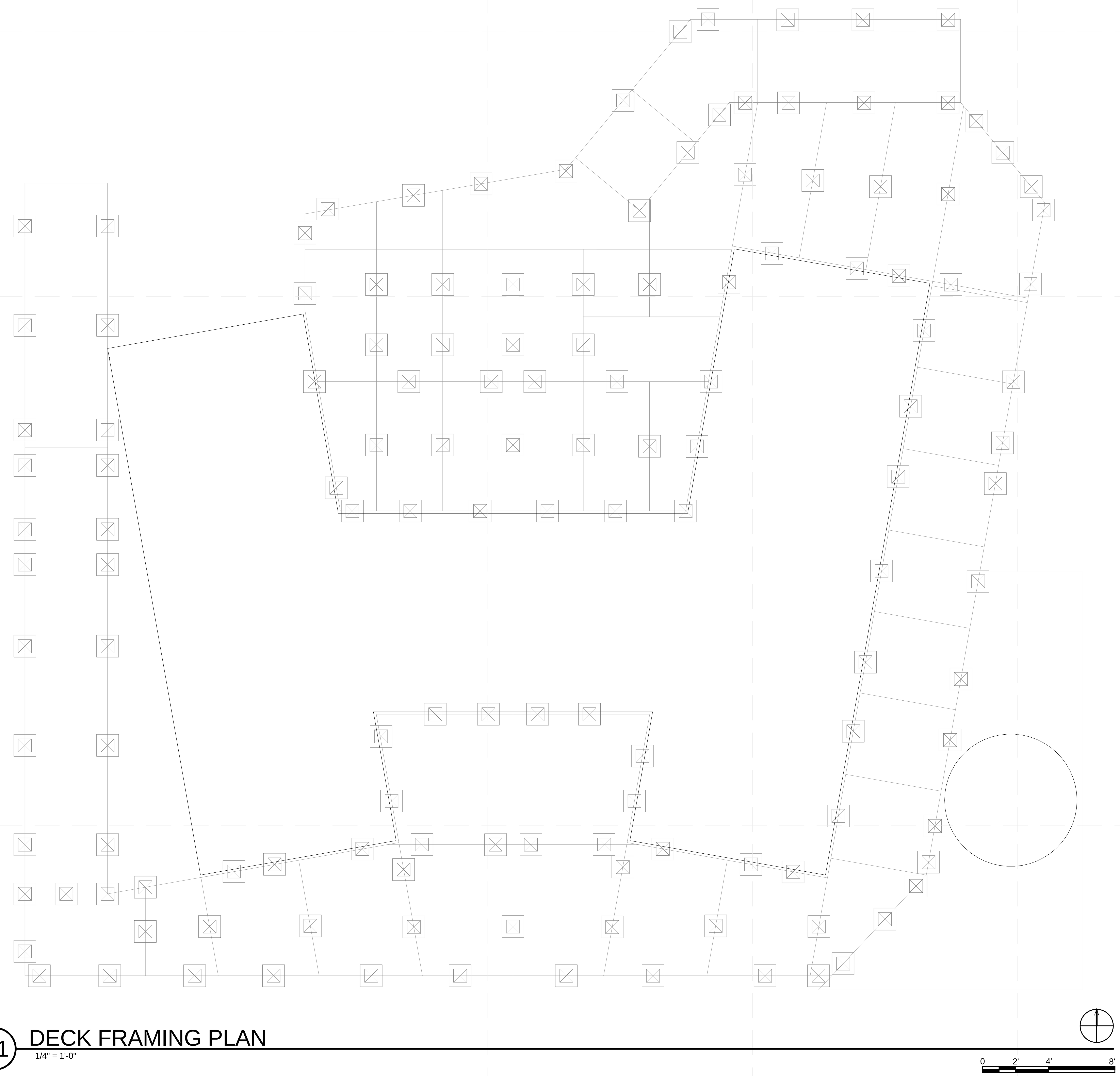
GENERAL SHEET NOTES

1. ALL PIERS SHOWN AND PROVIDED FOR THE DECK ARE STANDARD PIERS.



A1 DECK FRAMING PLAN

1/4" = 1'-0"

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LOT NUMBER: 118

DRAWN BY: ML

CHECKED BY: SCU

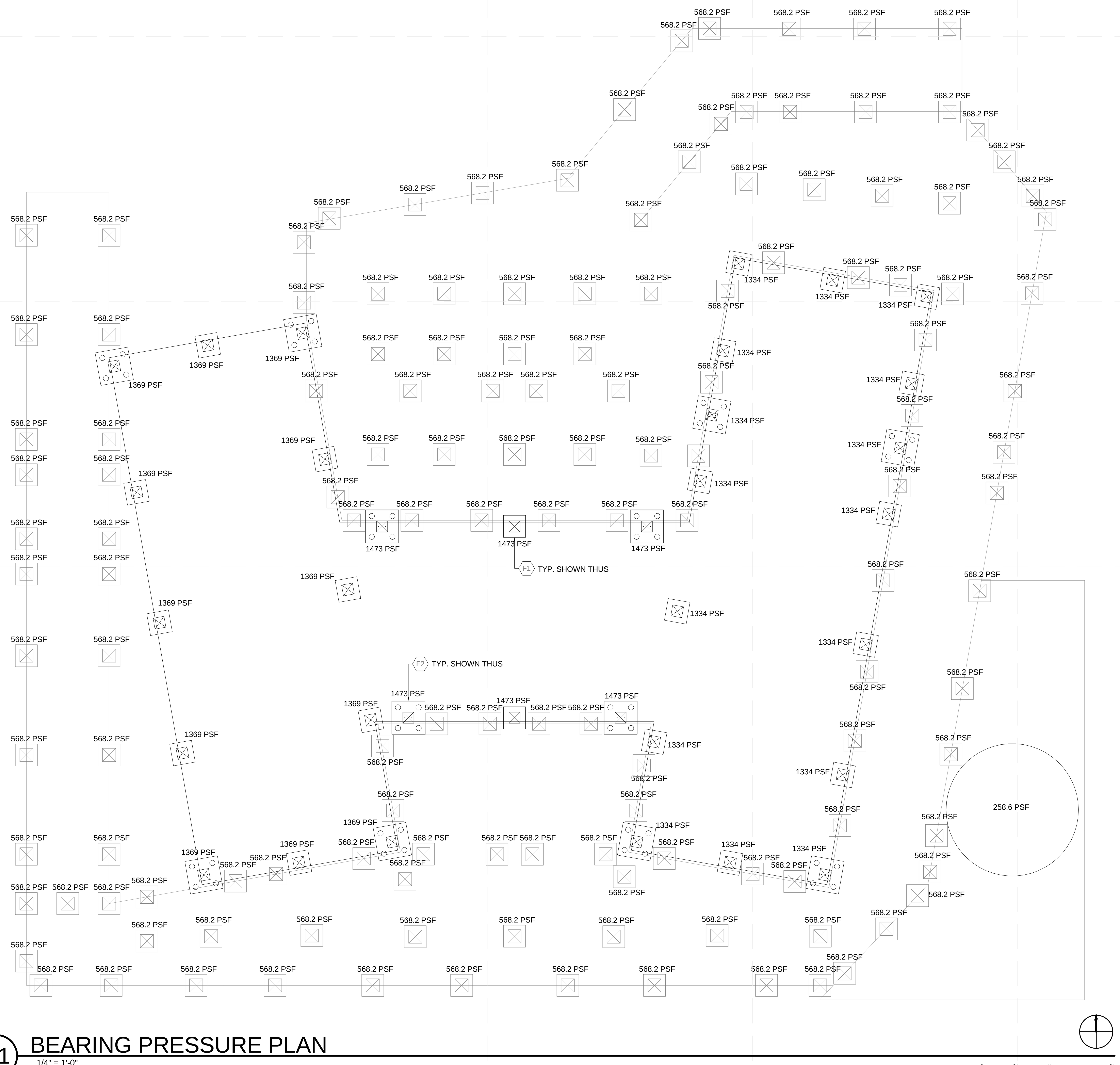
SHEET TITLE:
DECK FRAMING PLAN

S-107

9/5/2013 11:20:29 AM

GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. S.A.D. FOR ALL DIMENSIONS AND FINISH ELEVATIONS NOT NOTED.
3. S.A.D. FOR ALL FINISHES, FIRE PROTECTION, THERMAL, AND MOISTURE PROTECTION, WATERPROOFING, AND ACOUSTICAL REQUIREMENTS FOR ALL CEILINGS, WALLS, FLOORS, ROOF DECKS AND TERRACES.
4. SEE S-601 FOR FOOTING SCHEDULE.
5. BEARING CAPACITY AND FOOTING DESIGN BASED ON GOVERNING CASE OF GRAVITY AND LATERAL FORCES.
6. MAXIMUM BEARING CAPACITY FOR ALL FOOTINGS TO BE 1500 PSF.



A1 BEARING PRESSURE PLAN
1/4" = 1'-0"

0 2' 4' 8'

LEGEND

FX FOOTING TYPE

RADIANT HOUSE
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SUMMERS & SONS ELECTRIC, INC.
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U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON

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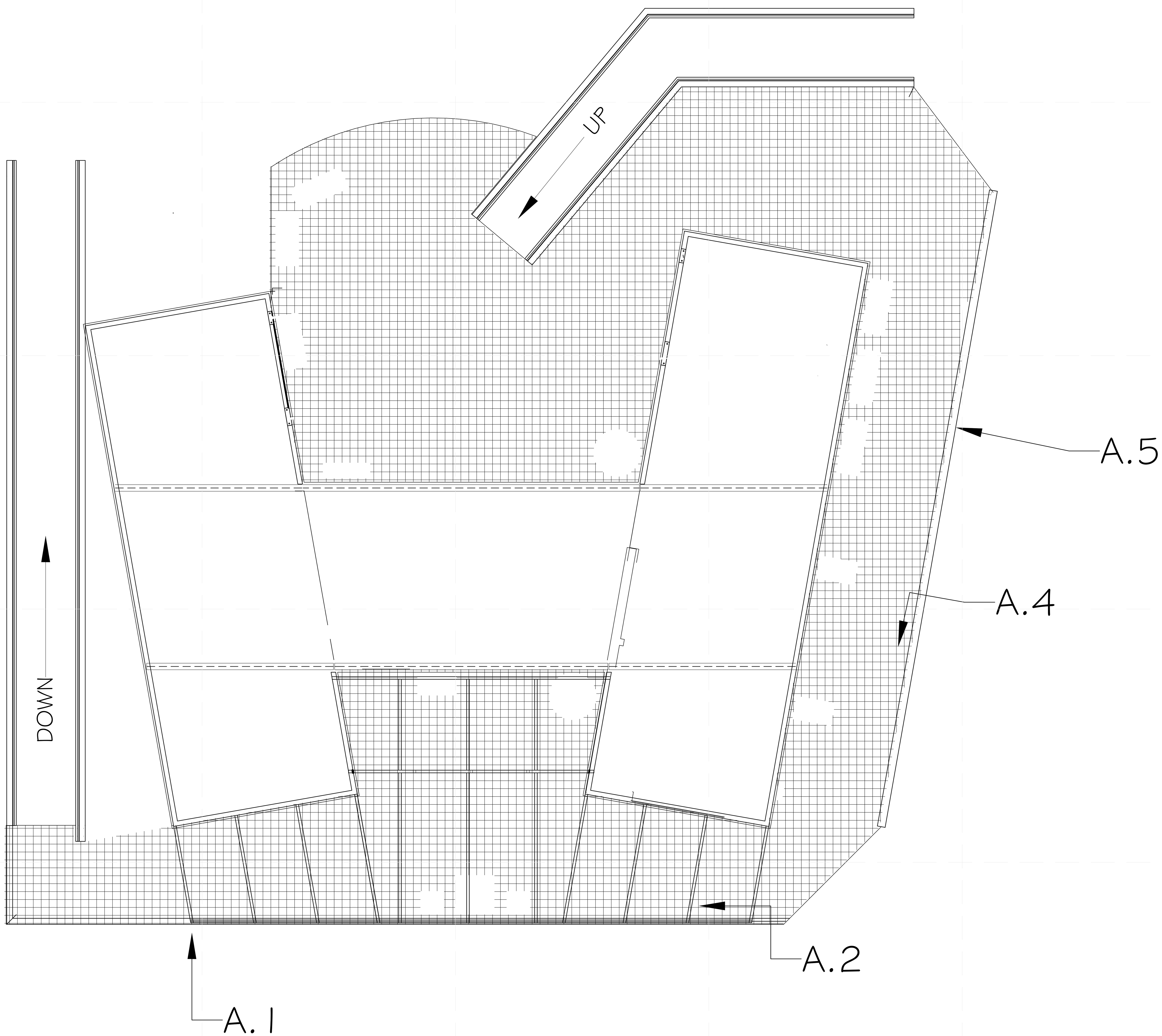
	04/04/13	CD RESUBMISSION
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
BEARING PRESSURE PLAN

S-108

9/5/2013 11:20:48 AM



A1 AWNING GENERAL PLAN
1/4" = 1'-0"

0 2' 4' 8'

9/5/2013 10:04:51 AM

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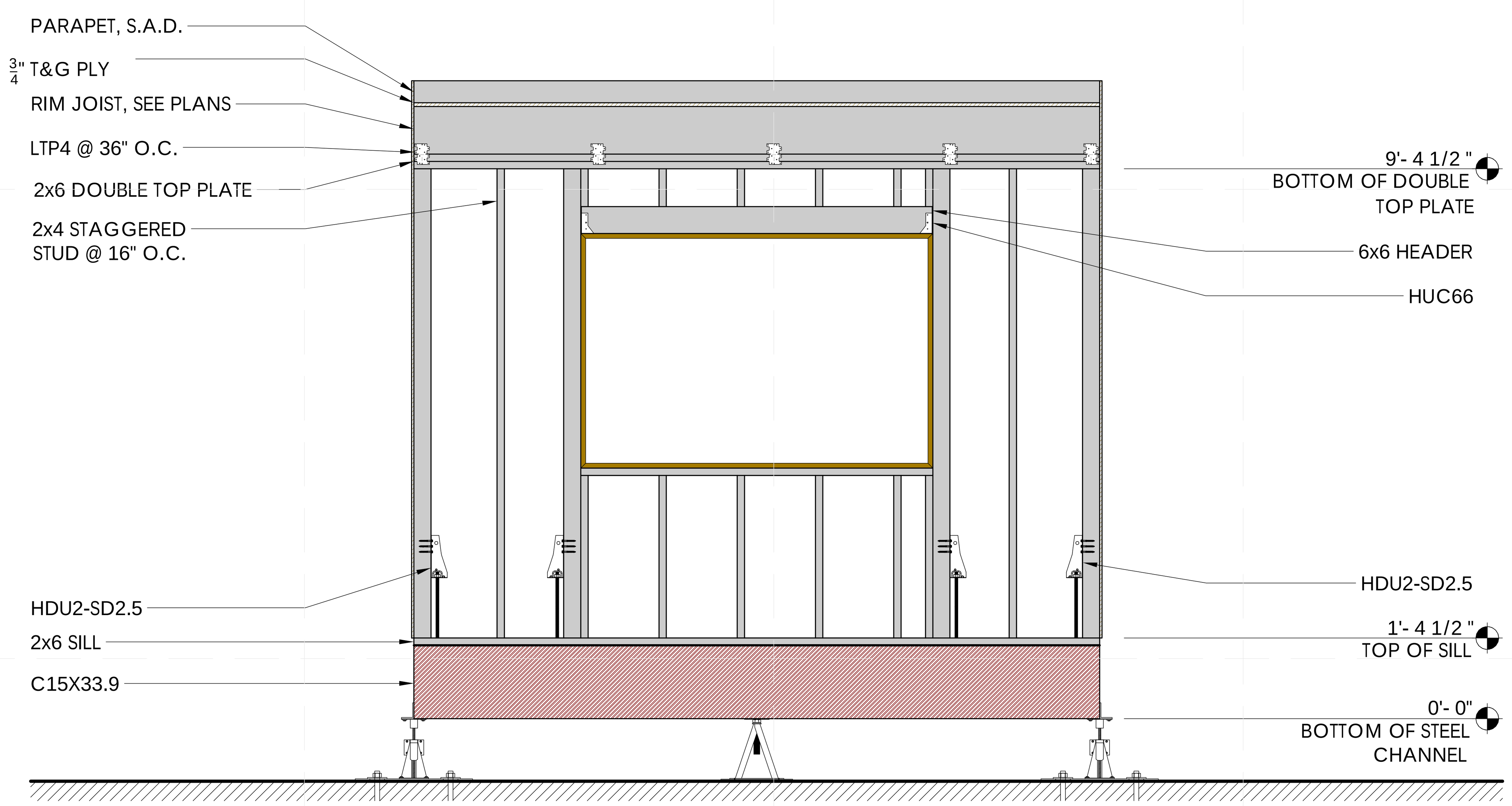
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: AT
CHECKED BY: SCU

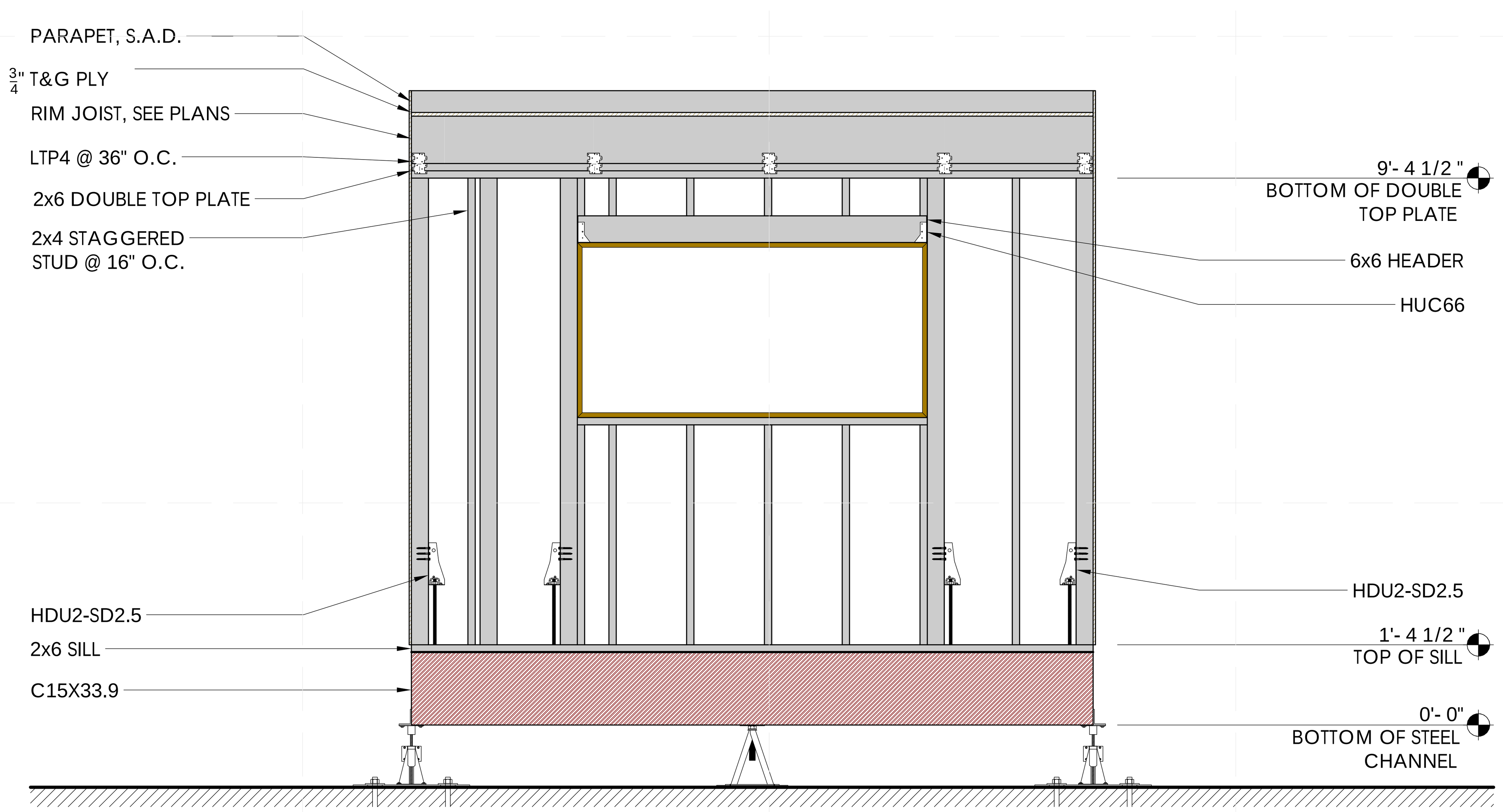
SHEET TITLE:
SOUTH AWNING PLAN

S-109



E MODULE A NORTH WALL ELEVATIONS

1/2" = 1'-0"



A MODULE A SOUTH WALL ELEVATIONS

1/2" = 1'-0"

9/5/2013 10:03:17 AM

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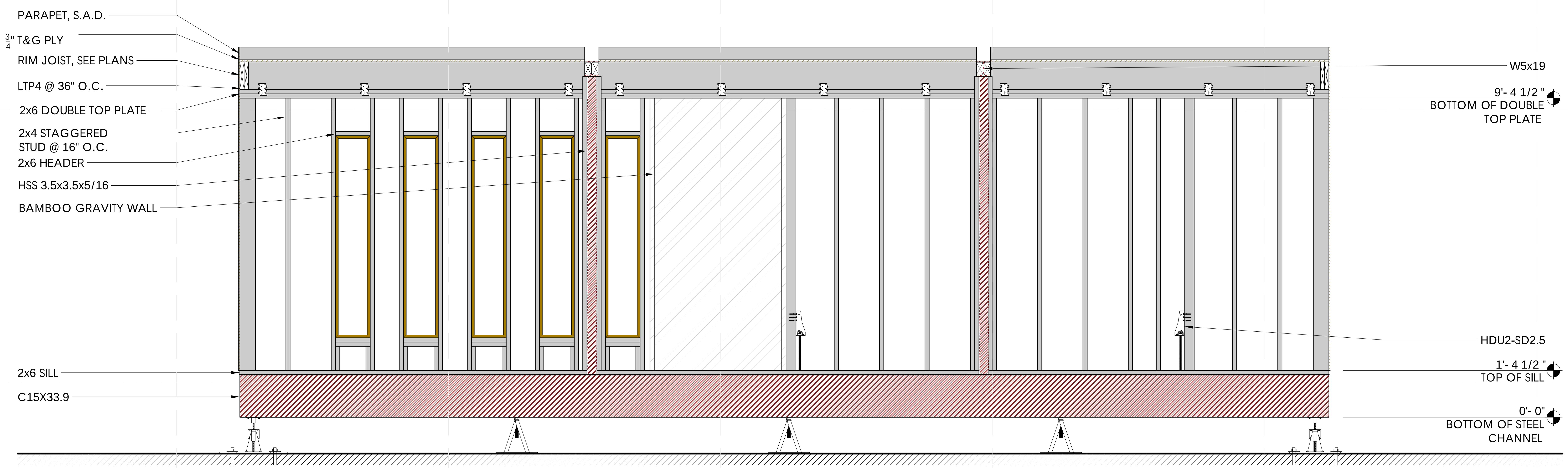
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

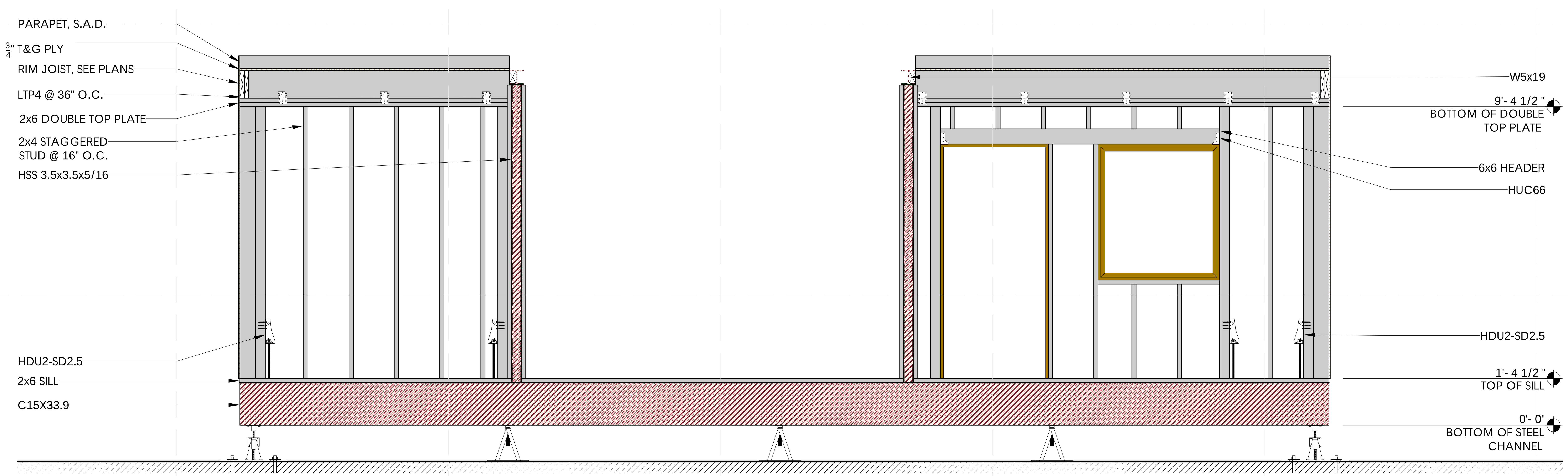
SHEET TITLE:
MODULE A WALL FRAMING

S-201



1 MODULE A WEST WALL ELEVATIONS

1/2" = 1'-0"



2 MODULE A EAST WALL ELEVATIONS

1/2" = 1'-0"

9/5/2013 10:05:37 AM

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U.S. DEPARTMENT OF ENERGY
SOLAR DECATHLON

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WWW.SOLARDECATHLON.GOV

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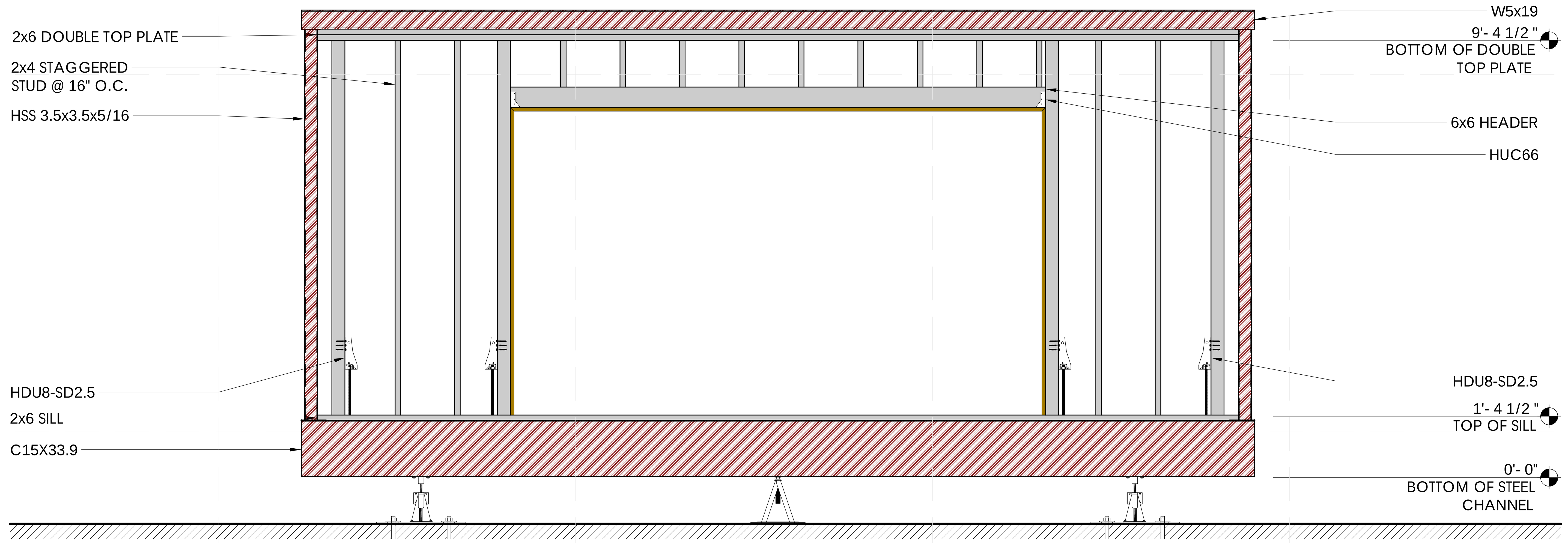
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
MODULE A WALL FRAMING

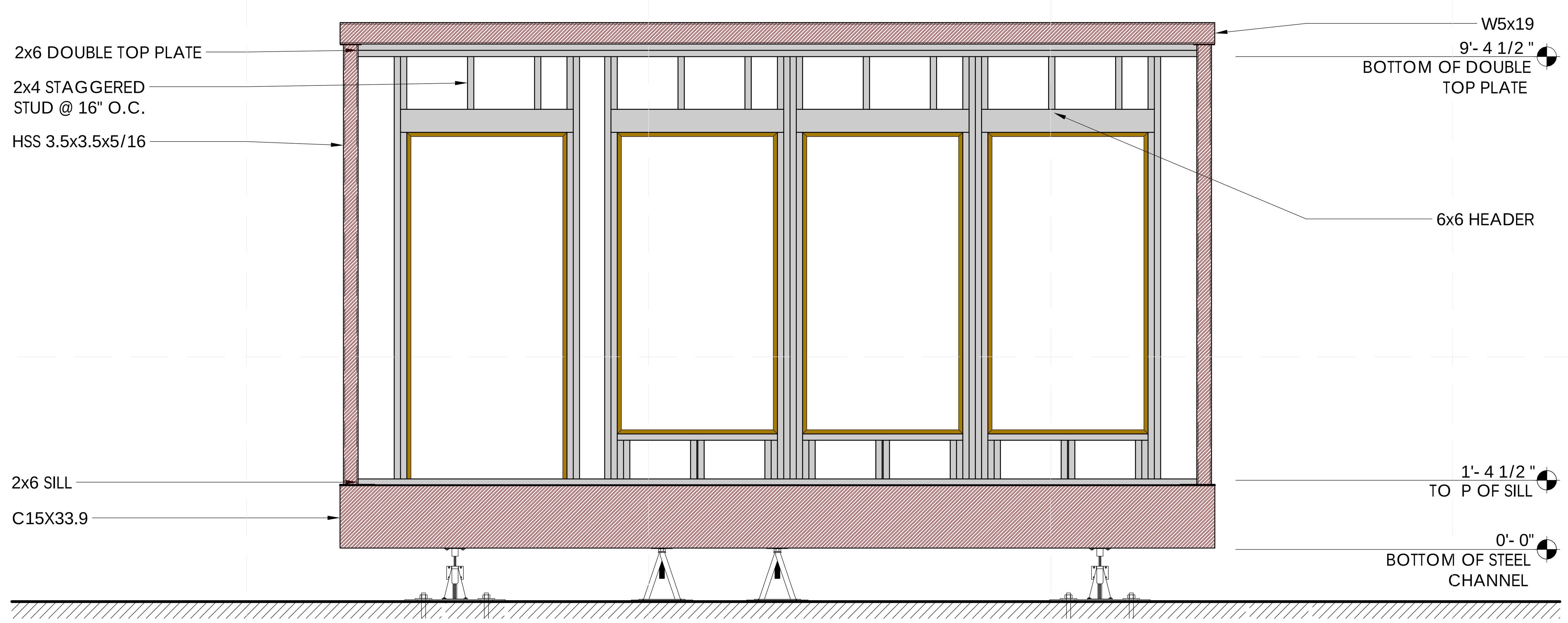
S-202



C MODULE B NORTH WALL ELEVATIONS

1/2" = 1'-0"

0 1" 2" 3" 4"



B MODULE B SOUTH WALL ELEVATIONS

1/2" = 1'-0"

0 1" 2" 3" 4"

9/5/2013 10:06:20 AM

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	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
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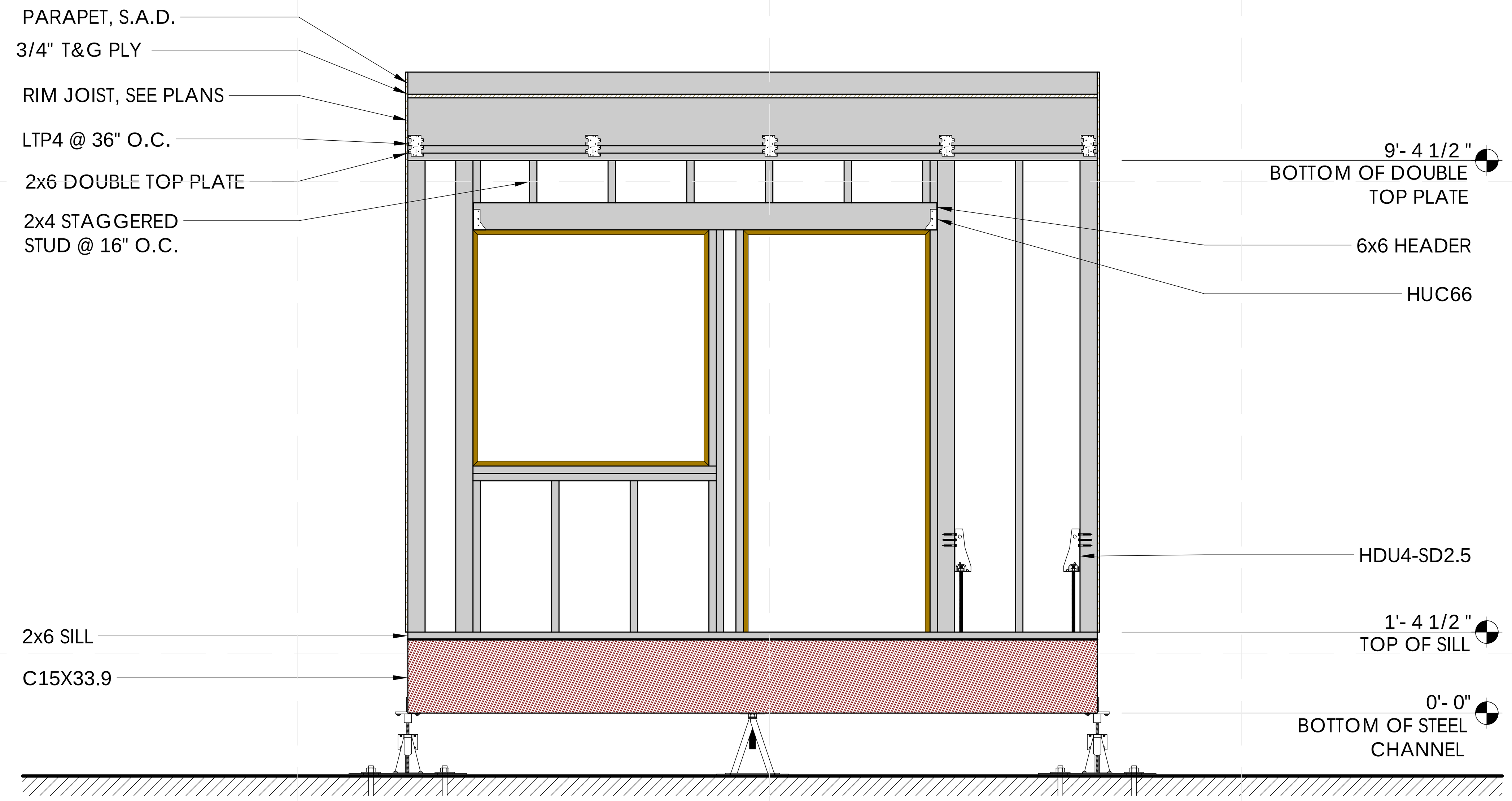
LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
MODULE B WALL FRAMING

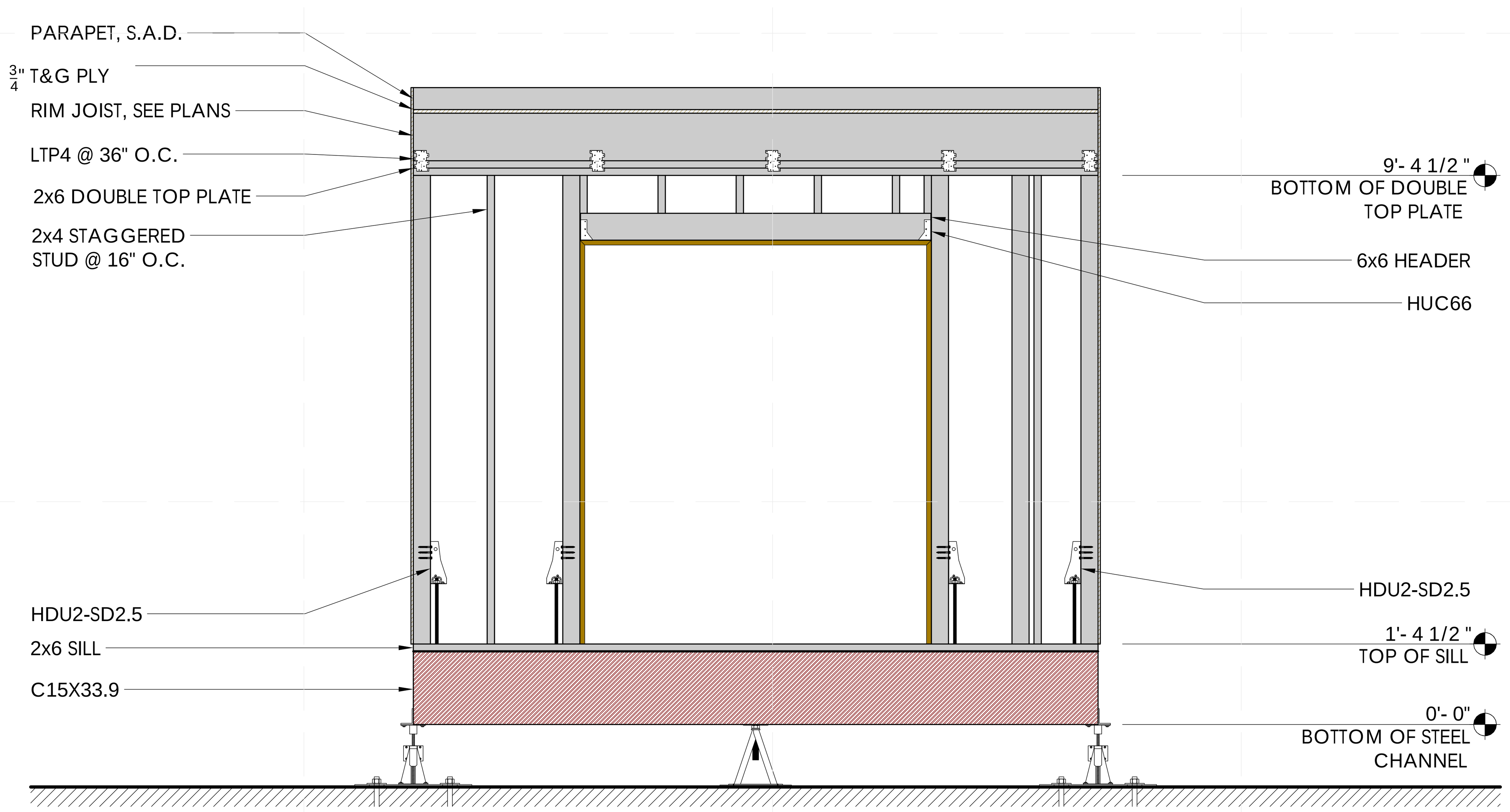
S-203



E MODULE C NORTH WALL ELEVATIONS

1/2" = 1'-0"

0 1" 2" 3" 4"



A MODULE C SOUTH WALL ELEVATIONS

1/2" = 1'-0"

0 1" 2" 3" 4"

9/5/2013 10:07:09 AM

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U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON

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	11/20/12	DD RESUBMISSION
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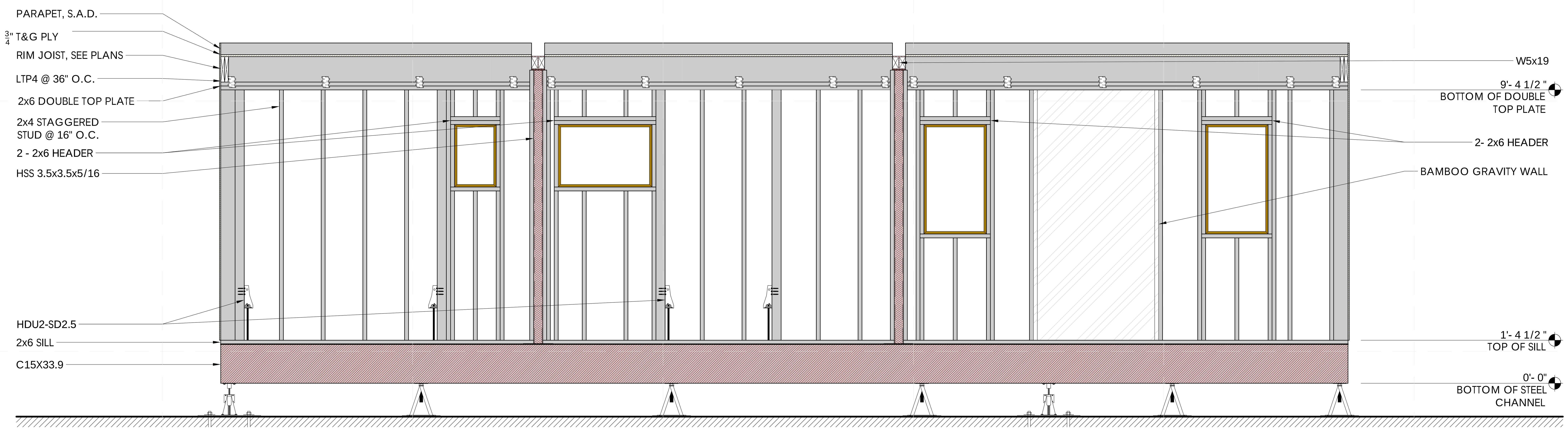
LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

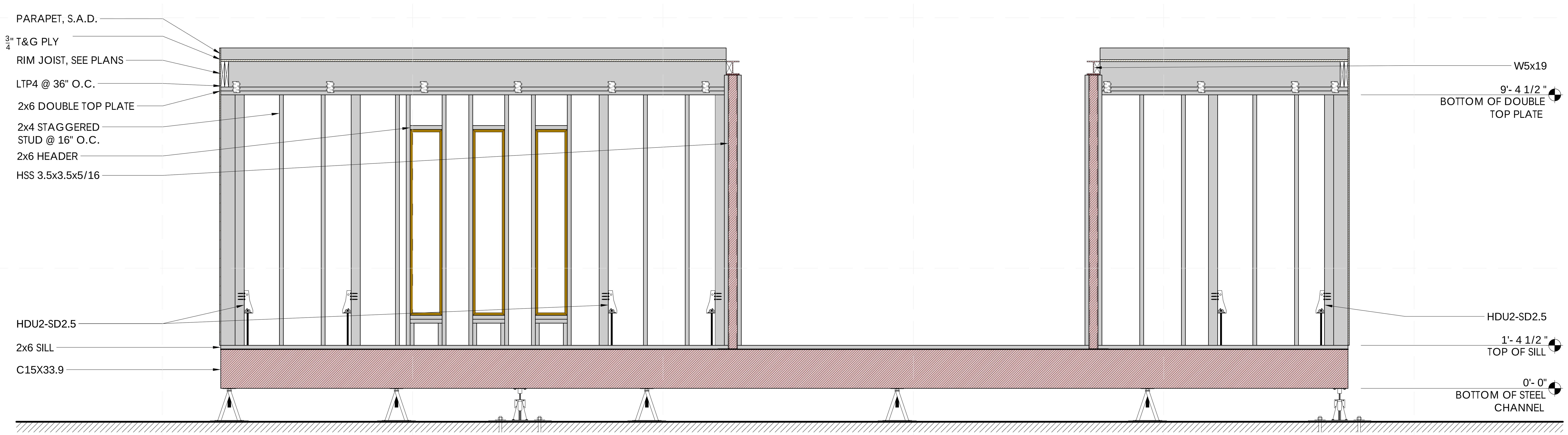
SHEET TITLE:
MODULE C WALL FRAMING

S-204



6 MODULE C EAST WALL ELEVATION

1/2" = 1'-0"



5 MODULE C WEST WALL ELEVATIONS

1/2" = 1'-0"

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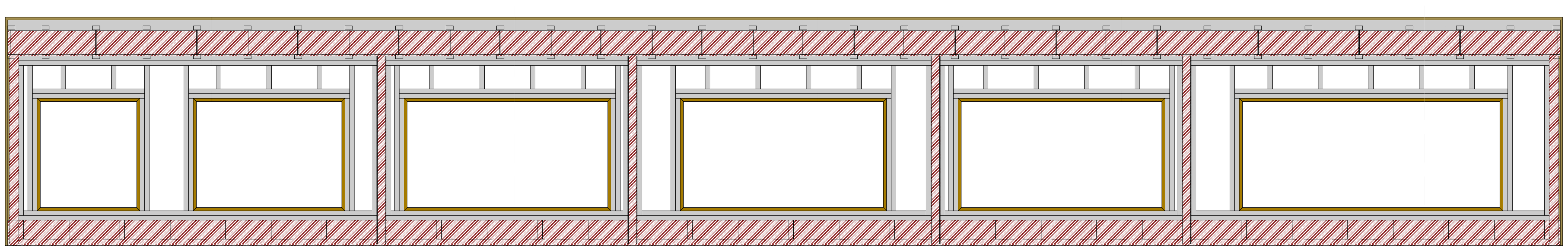
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
MODULE C WALL FRAMING

S-205

9/5/2013 10:08:00 AM



A1 ROOF MODULE

1/2" = 1'-0"

0 1' 2' 4'

9/5/2013 3:09:05 PM

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U.S. DEPARTMENT OF ENERGY
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	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118

DRAWN BY: KM

CHECKED BY: SCU

SHEET TITLE:

ROOF MODULE FRAMING

S-206

GENERAL SHEET NOTES

1. REFER TO GENERAL STRUCTURAL NOTES & DETAILS FOR GENERAL REQUIREMENTS OF CONSTRUCTION, MATERIALS, AND REQUIREMENTS OF CONSTRUCTION OBSERVATIONS.
2. S.A.D. FOR ALL DIMENSIONS AND FINISH ELEVATIONS NOT NOTED.
3. S.A.D. FOR ALL FINISHES, FIRE PROTECTION, THERMAL AND MOISTURE PROTECTION, WATERPROOFING, AND ACOUSTICAL REQUIREMENTS FOR ALL CEILINGS, WALLS, FLOORS, ROOF DECKS, AND TERRACES.

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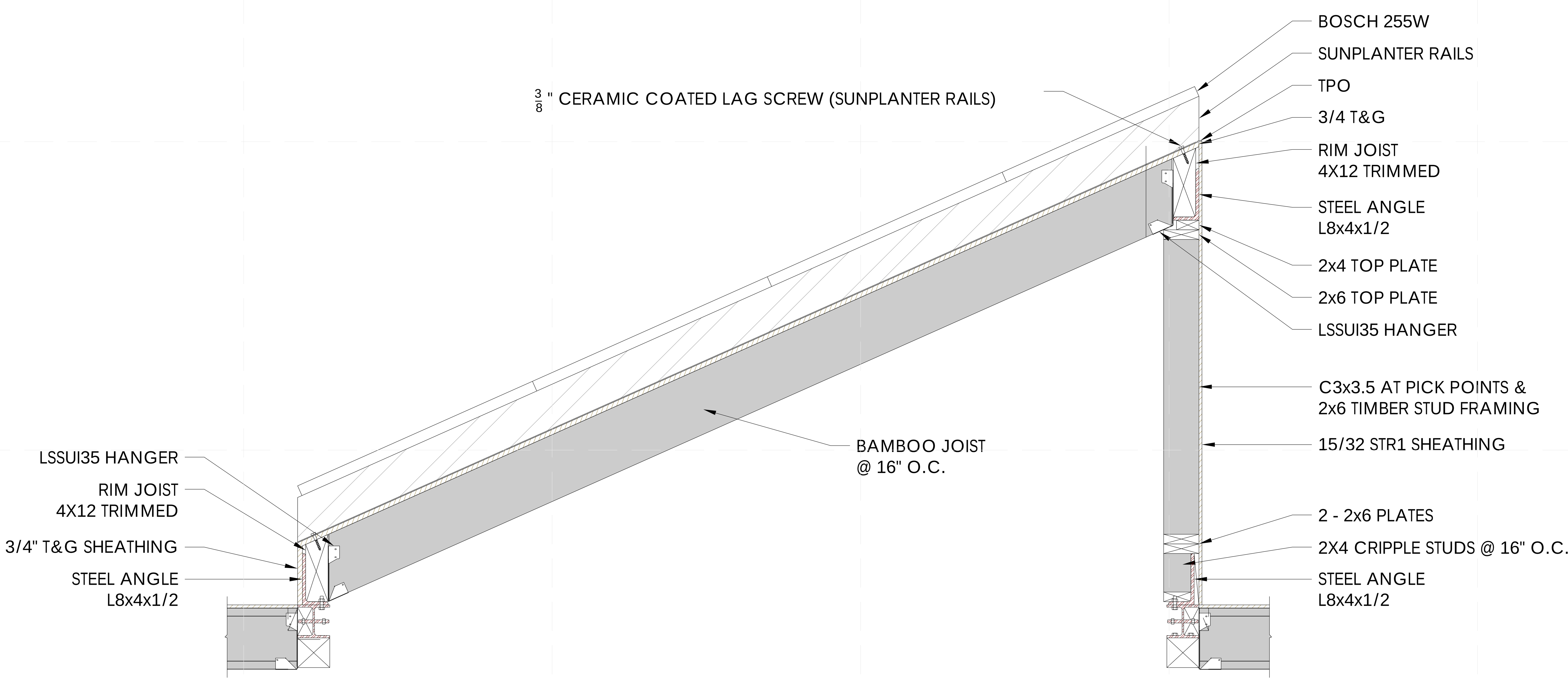

1 ROOF MODULE SECTION
1" = 1'-0"

0 1/2' 1' 2'

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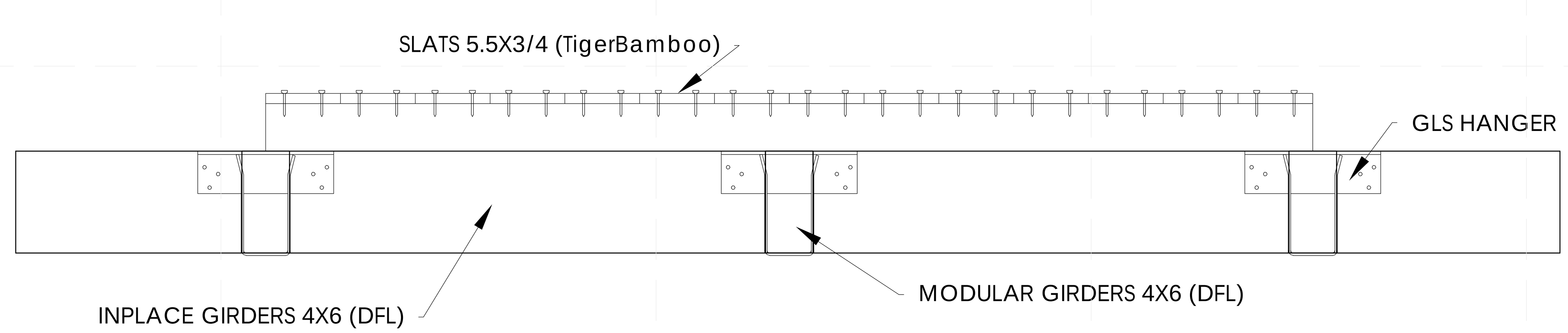
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
ROOF MODULE SECTION

S-301

9/5/2013 10:09:20 AM



A1 DECK SECTION

1 1/2" = 1'-0"

0 1/2" 1' 1 1/2'

9/5/2013 10:14:28 AM

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	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118

DRAWN BY: KM

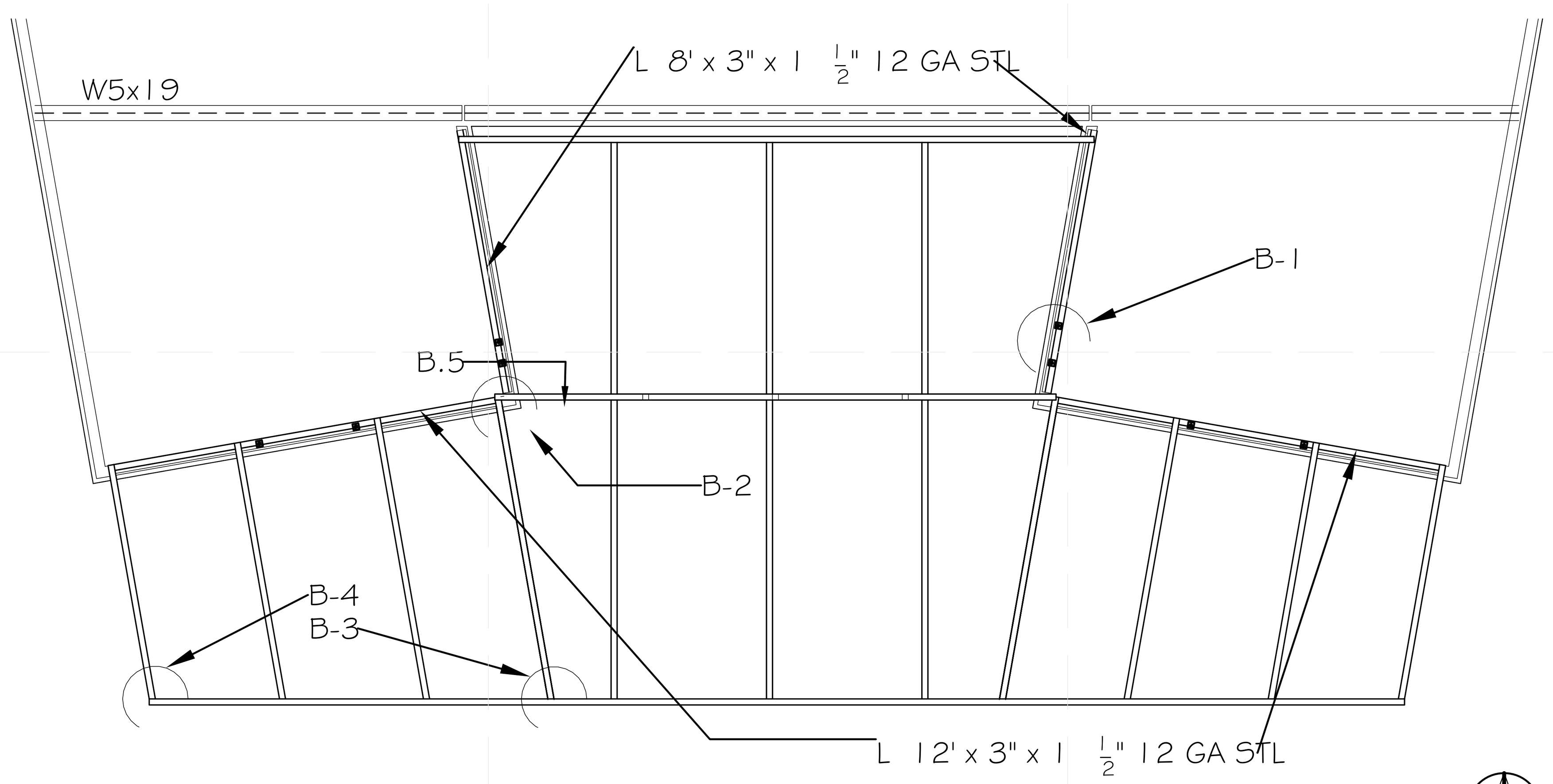
CHECKED BY: SCU

SHEET TITLE:
DECK SECTION

S-311

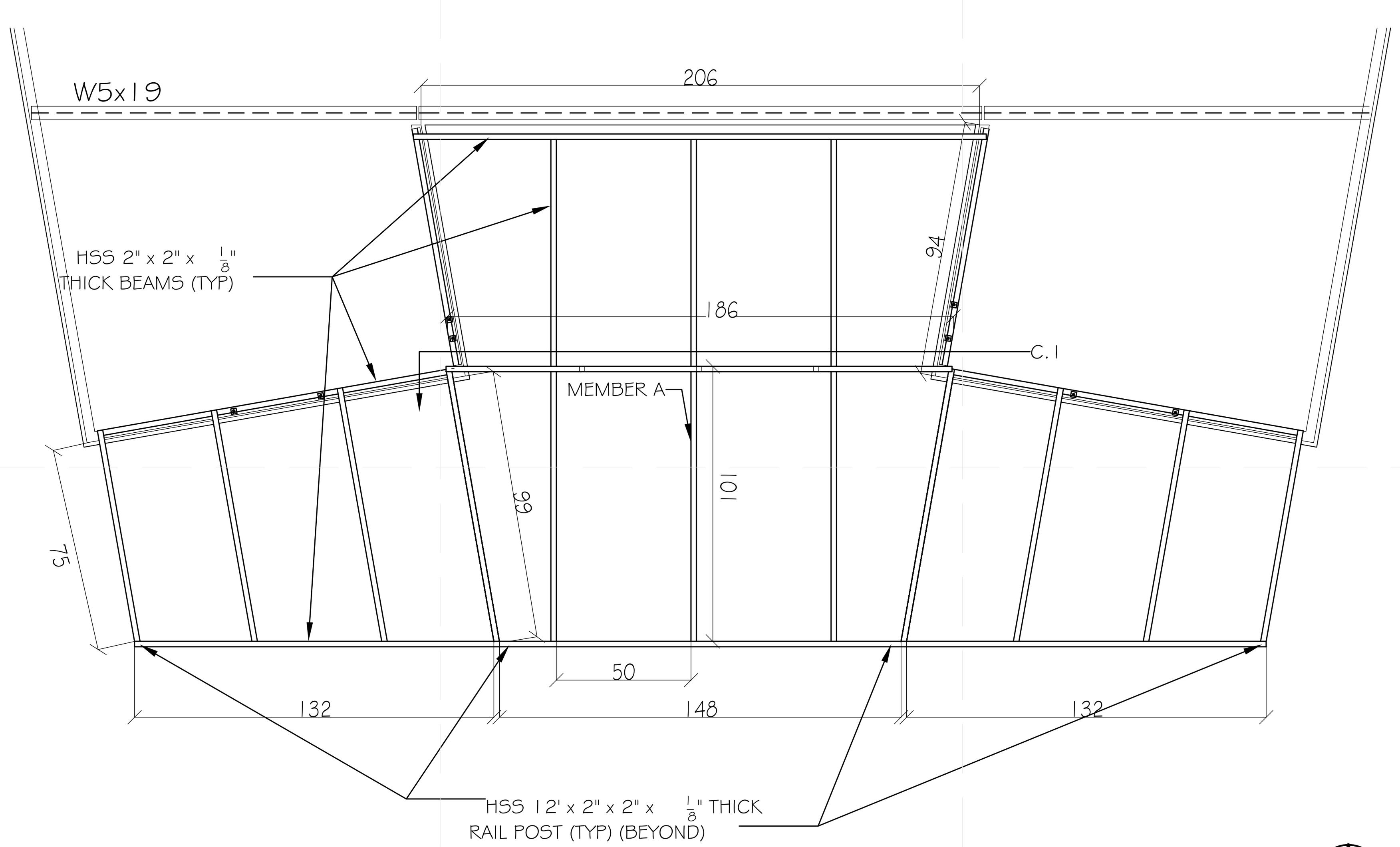
GENERAL SHEET NOTES

1. DECK JOISTS RUN NORTH/SOUTH, DECK BEAMS RUN EAST/WEST



C1 AWNING DETAIL CALLOUTS PLAN

1/4" = 1'-0"



A1 AWNING LARGE SCALE PLAN

1/4" = 1'-0"

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	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118

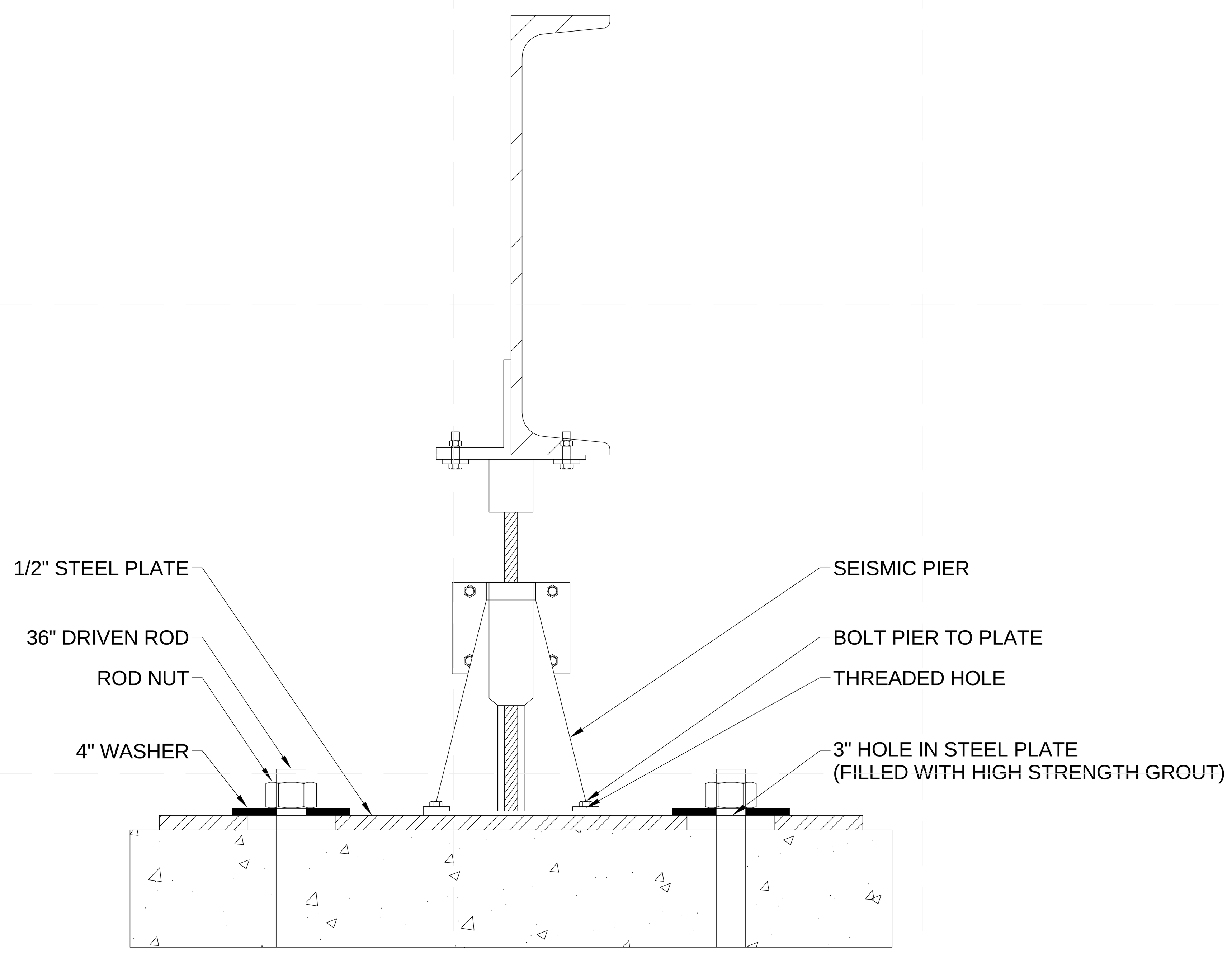
DRAWN BY: AT

CHECKED BY: SCU

SHEET TITLE: AWNING LARGE SCALE PLAN

S-401

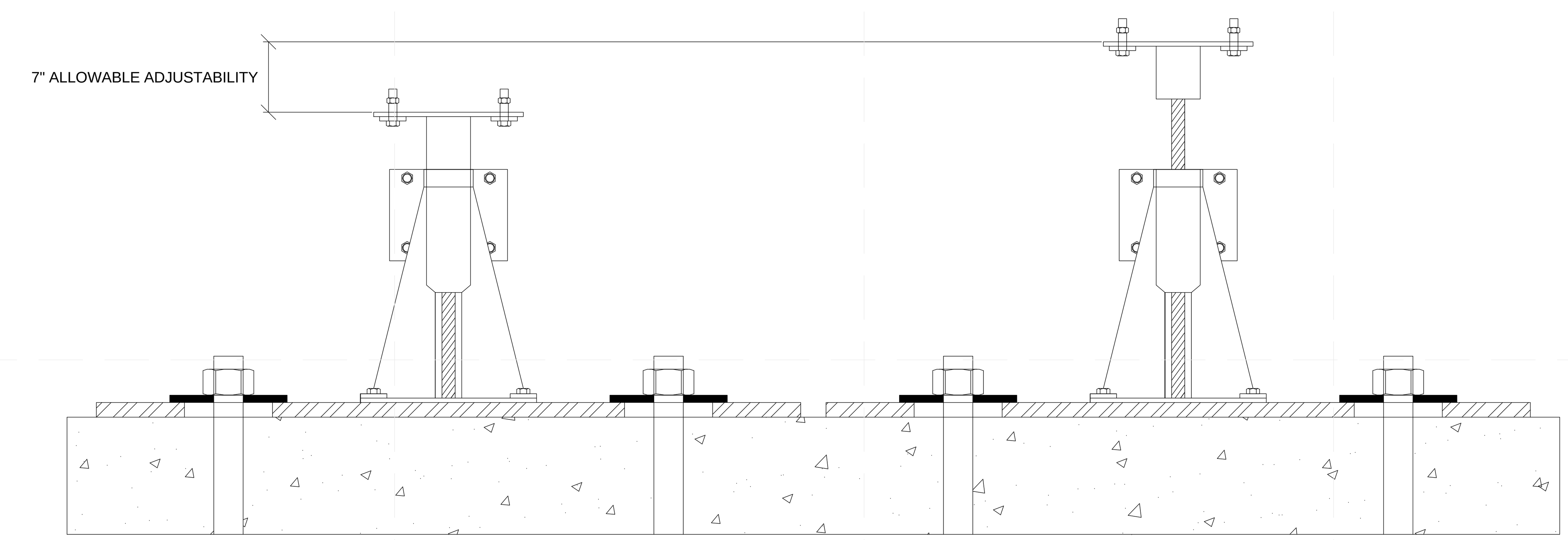
9/5/2013 10:13:00 AM



C1 SEISMIC PIER DETAIL

3" = 1'-0"

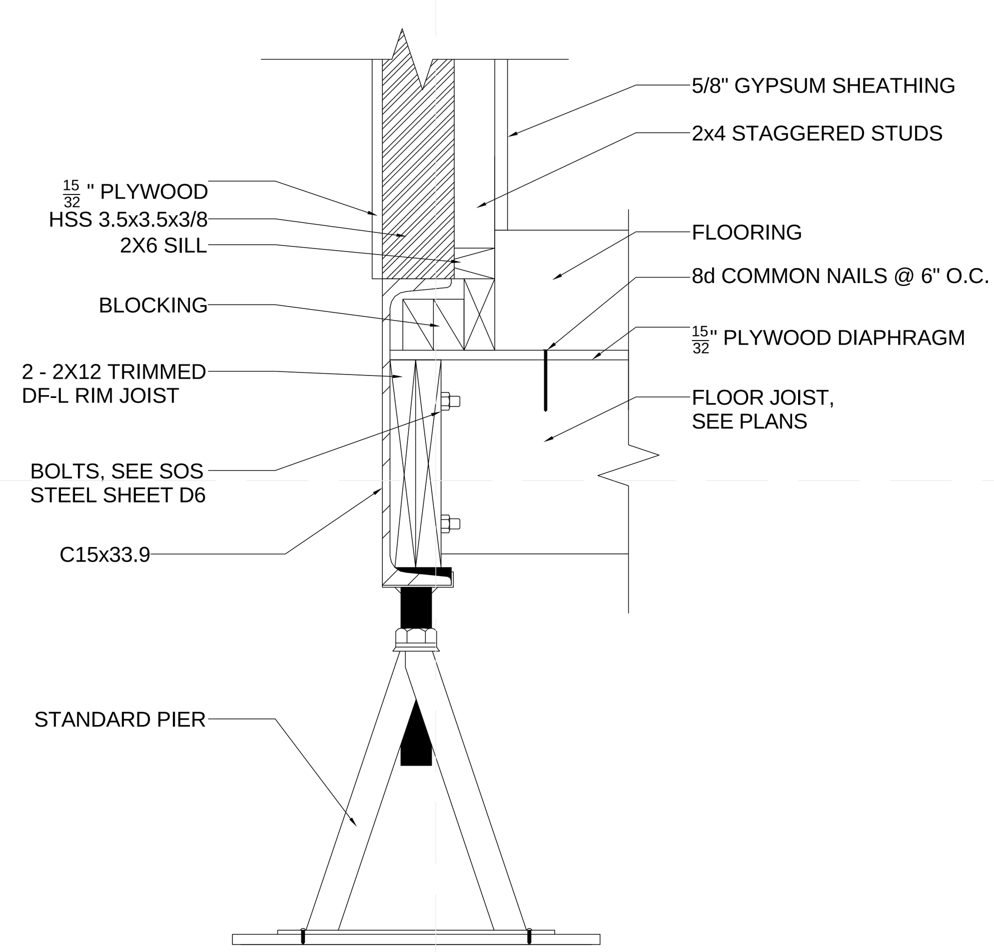
0 3" 6" 1'



C3 SEISMIC PIER ADJUSTABILITY

3" = 1'-0"

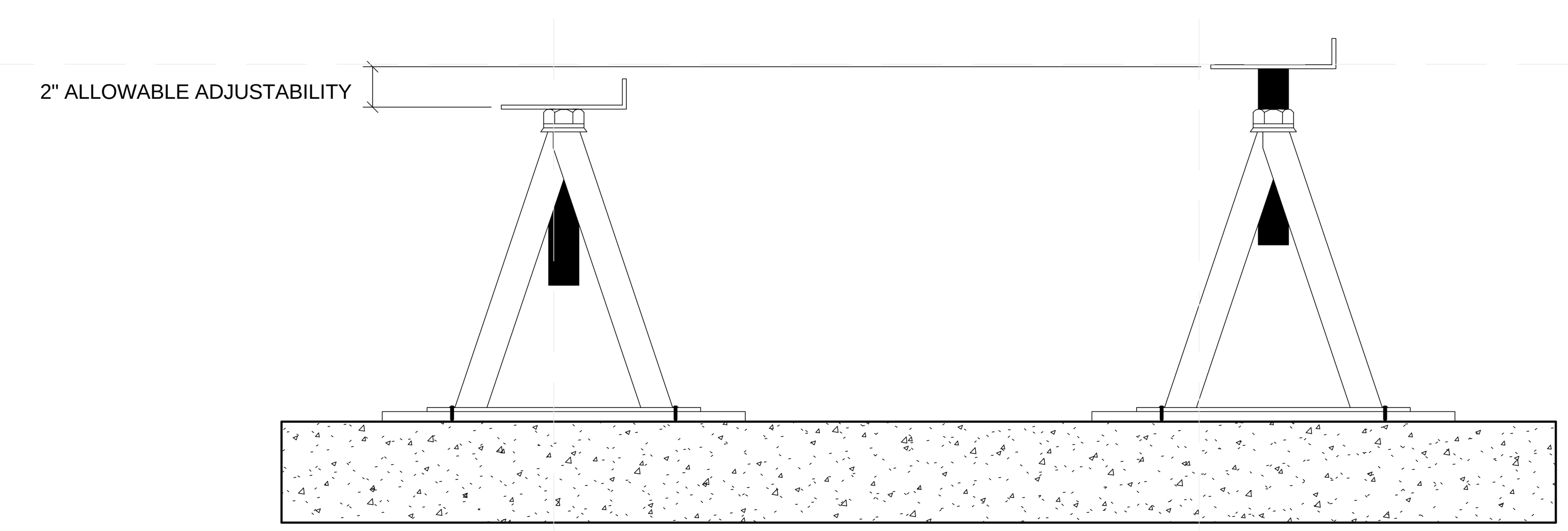
0 3" 6" 1'



A1 STANDARD PIER DETAIL

1 1/2" = 1'-0"

0 1/2" 1' 1 1/2'



A3 STANDARD PIER ADJUSTABILITY

1 1/2" = 1'-0"

0 1/2" 1' 1 1/2'

9/5/2013 10:19:32 AM

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U.S. DEPARTMENT OF ENERGY
SOLAR DECATHLON

CLIENT: U.S. DEPARTMENT OF ENERGY
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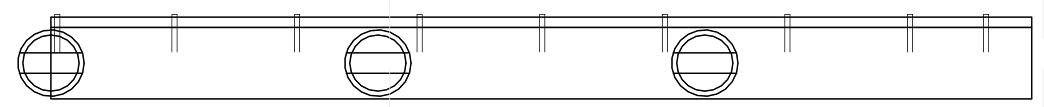
LOT NUMBER: 118
DRAWN BY: ML/EC
CHECKED BY: SCU

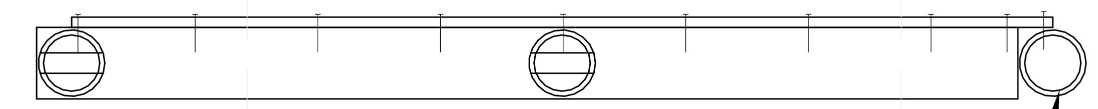
SHEET TITLE:
PIER DETAILS

S-501

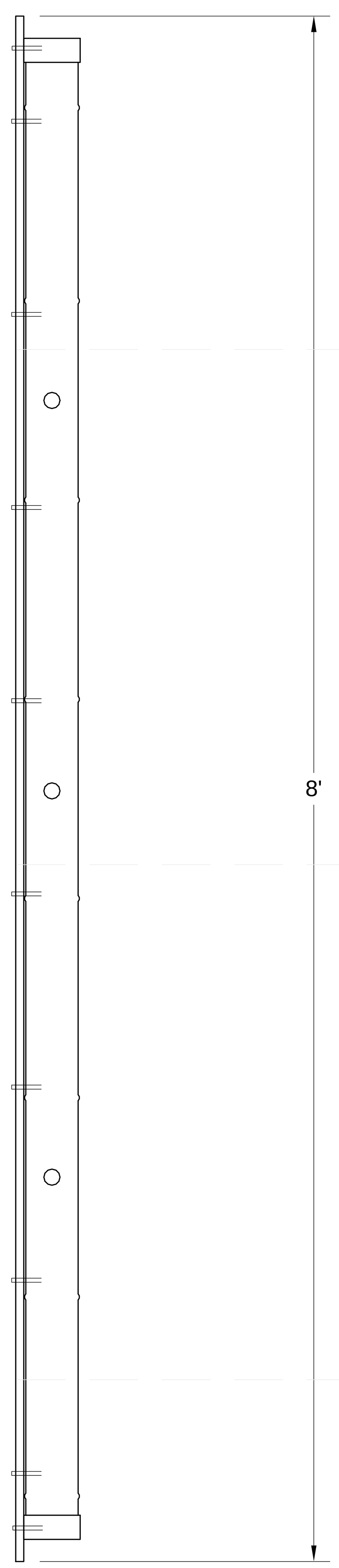
TYPICAL 4' BAMBOO PANEL



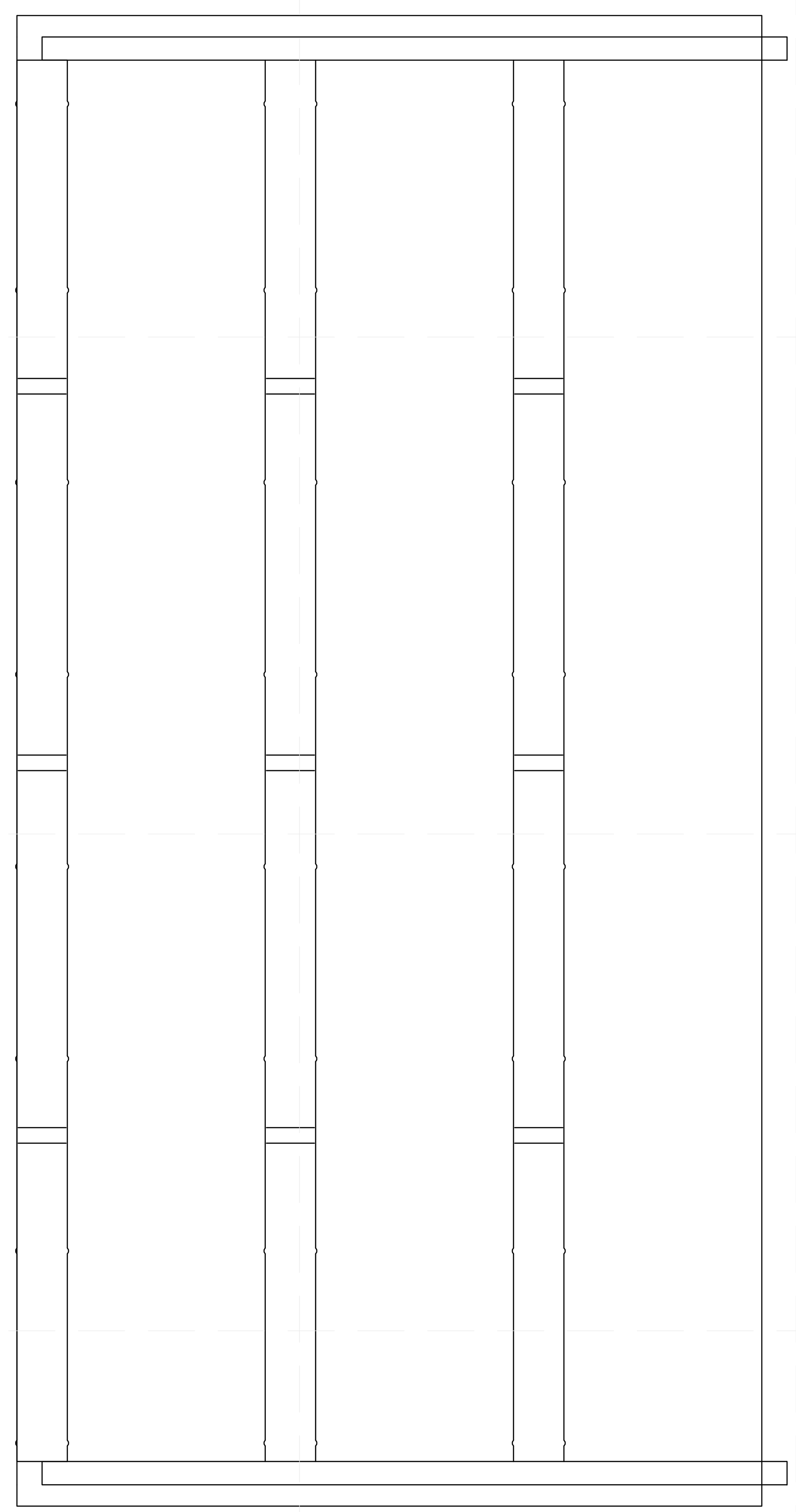
TOP SECTION



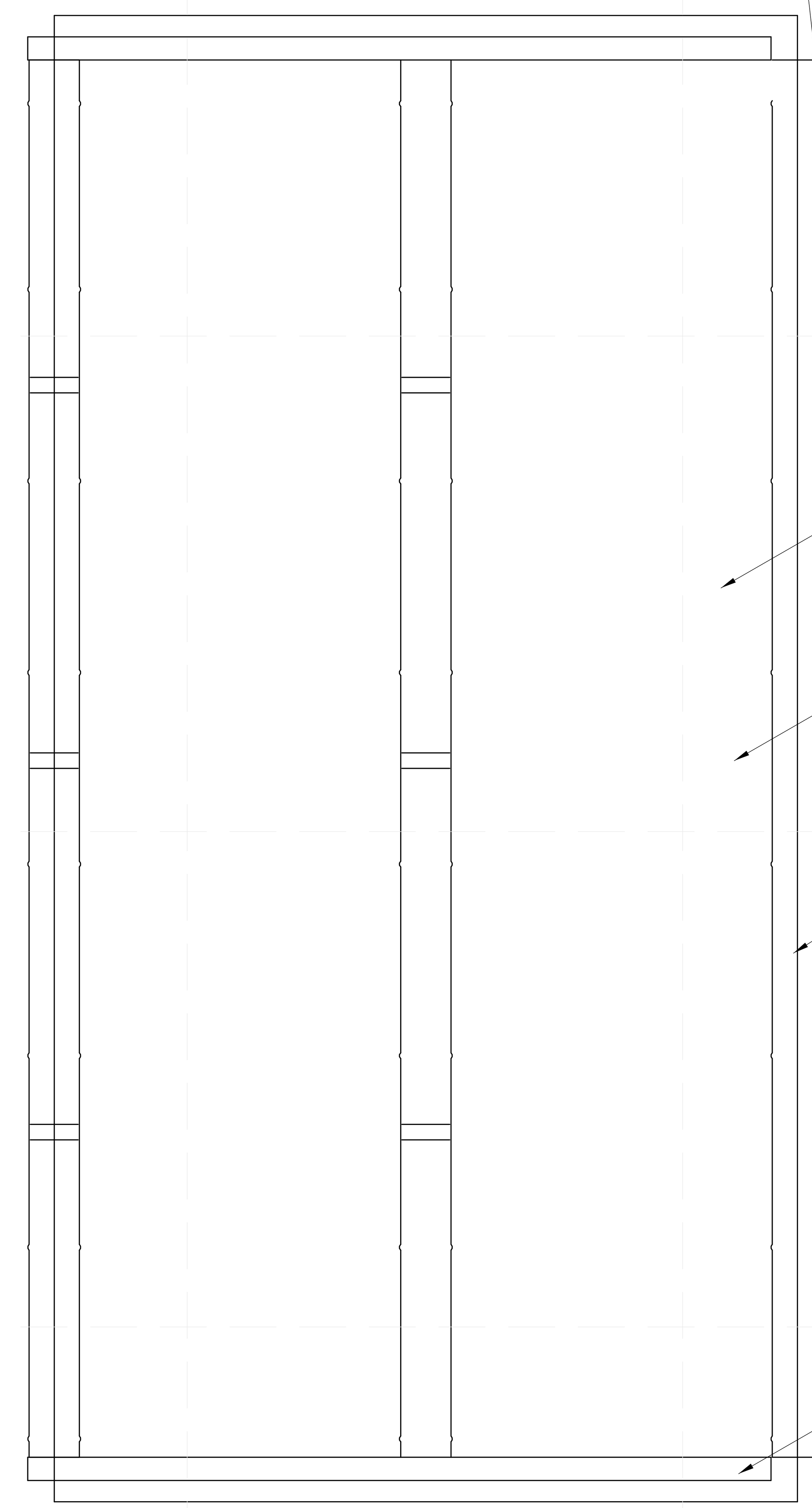
TOP SECTION DURING MUSHROOM GROWTH AND SHIPMENT



LEFT ELEVATION



FRONT ELEVATION FOR FINAL USE



FRONT ELEVATION DURING MUSHROOM GROWTH AND SHIPMENT

B1 GRAVITY WALLS

1 1/2" = 1'-0"

0 1/2' 1' 1 1/2'

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	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118

DRAWN BY: AJG

CHECKED BY: SCU

SHEET TITLE:

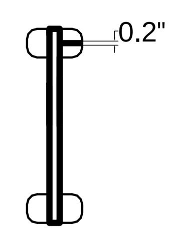
BAMBOO GRAVITY WALL DETAILS

S-512

9/5/2013 10:22:37 AM

GENERAL SHEET NOTES

1. NOT FOR CONSTRUCTION



BISCUIT CONNECTION
(B-B)

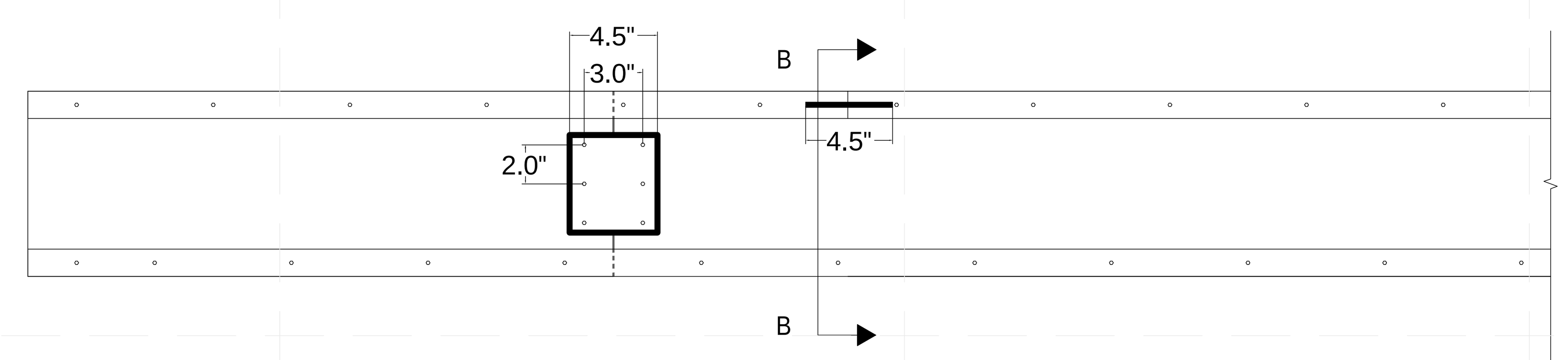


PLATE AND FLANGE SPLICE

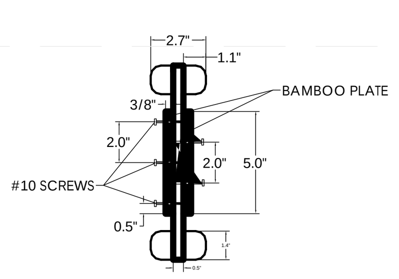
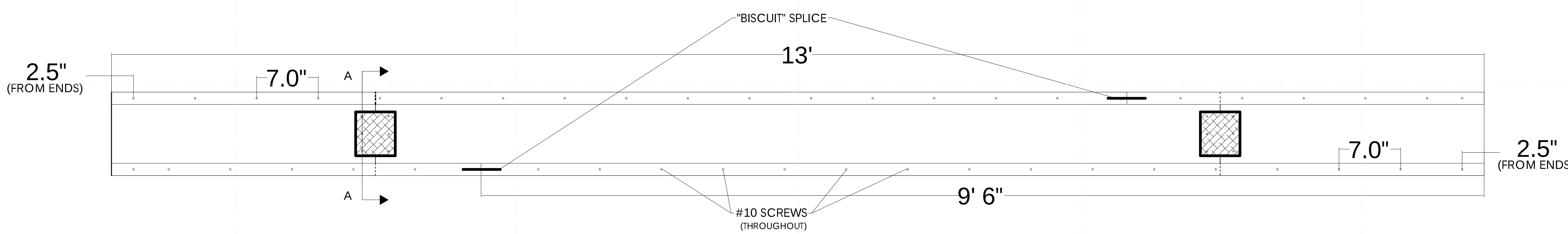


PLATE CONNECTION
(A-A)



FULL JOIST

B1 I JOIST DETAILS

1 1/2" = 1'-0"

0 1/2" 1' 1 1/2'

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	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

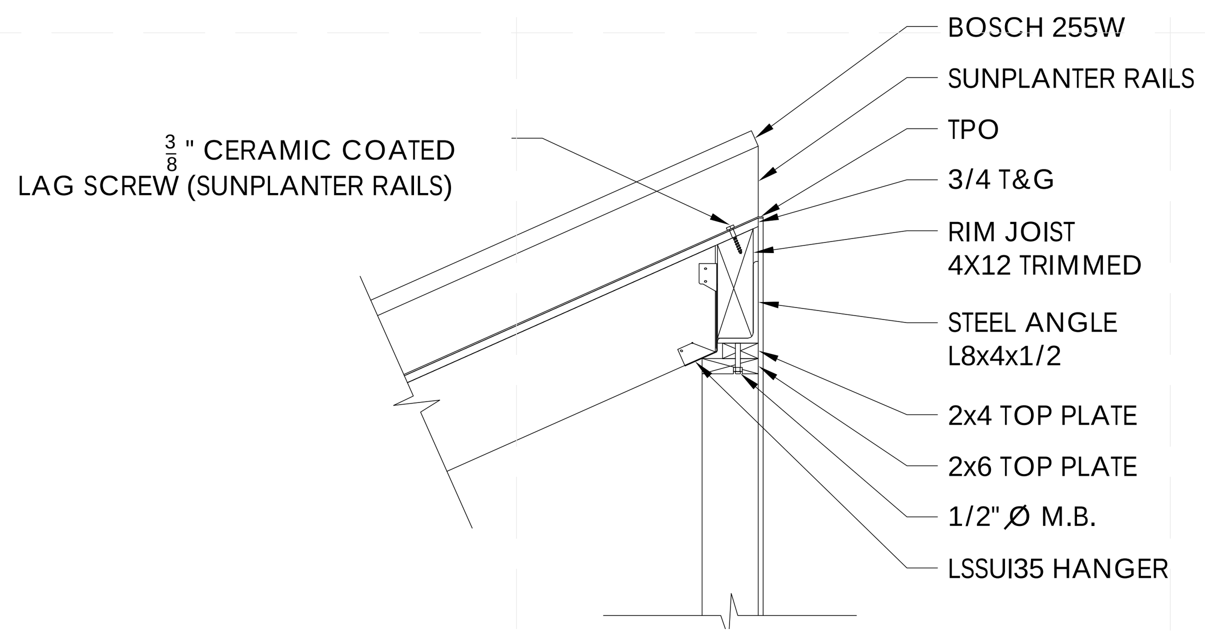
LOT NUMBER: 118
DRAWN BY: DS
CHECKED BY: SCU

SHEET TITLE:
I JOIST DETAILS

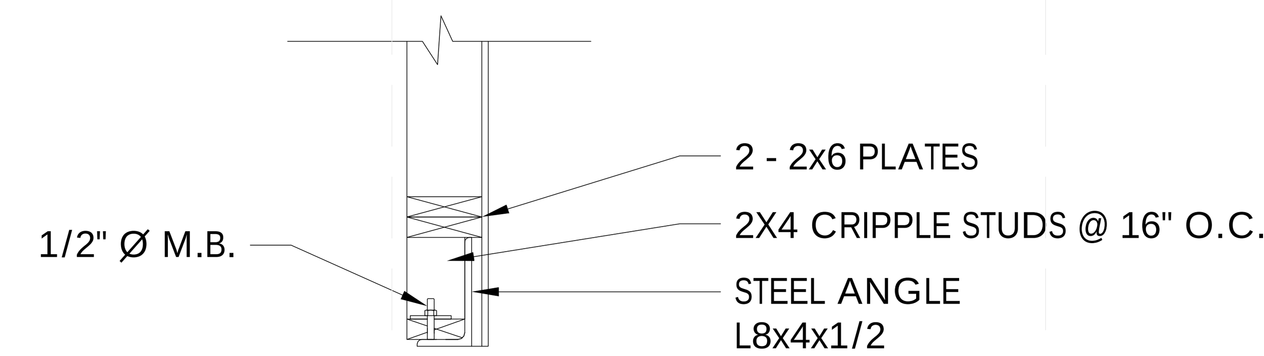
S-521

GENERAL SHEET NOTES

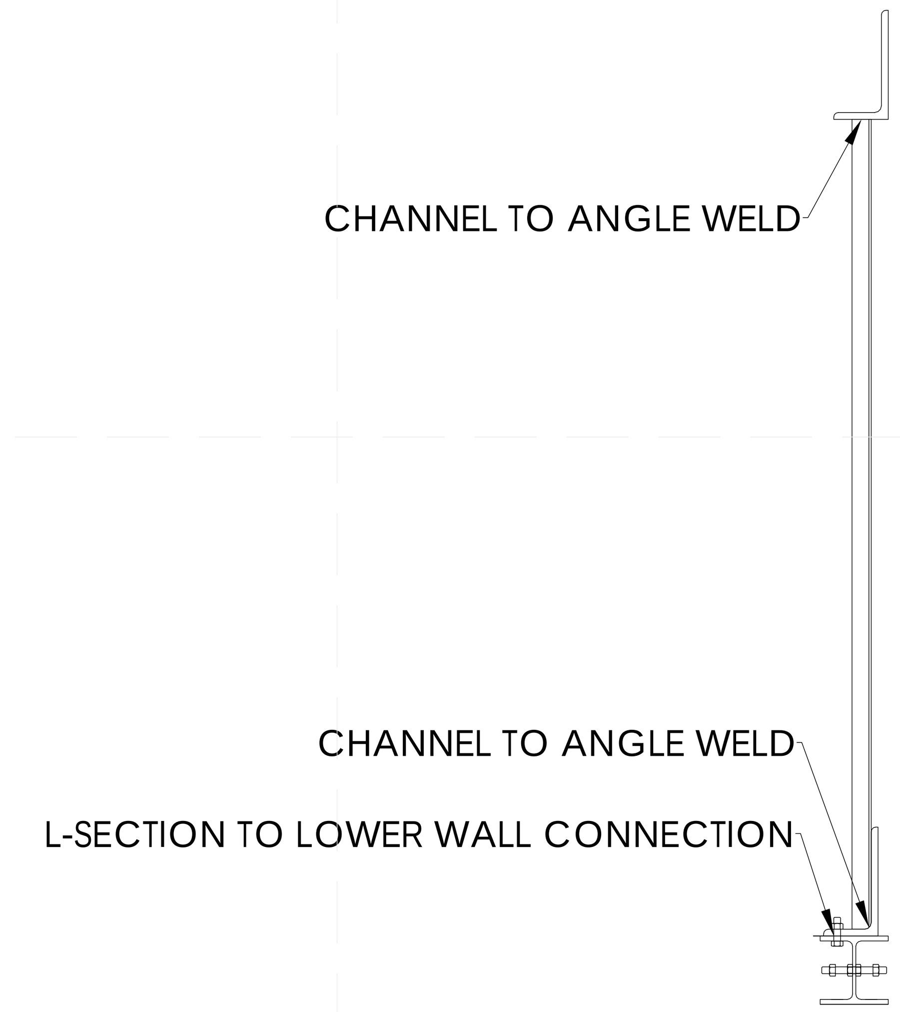
1. REFER TO SOS STEEL SHEET D9 FOR CORRESPONDING LOCATIONS OF DETAIL D2.
2. DETAIL D5 IS LOCATED AT B2, B5, C2 AND C5. REFER TO SHEET S-107 FOR GRID PLAN.



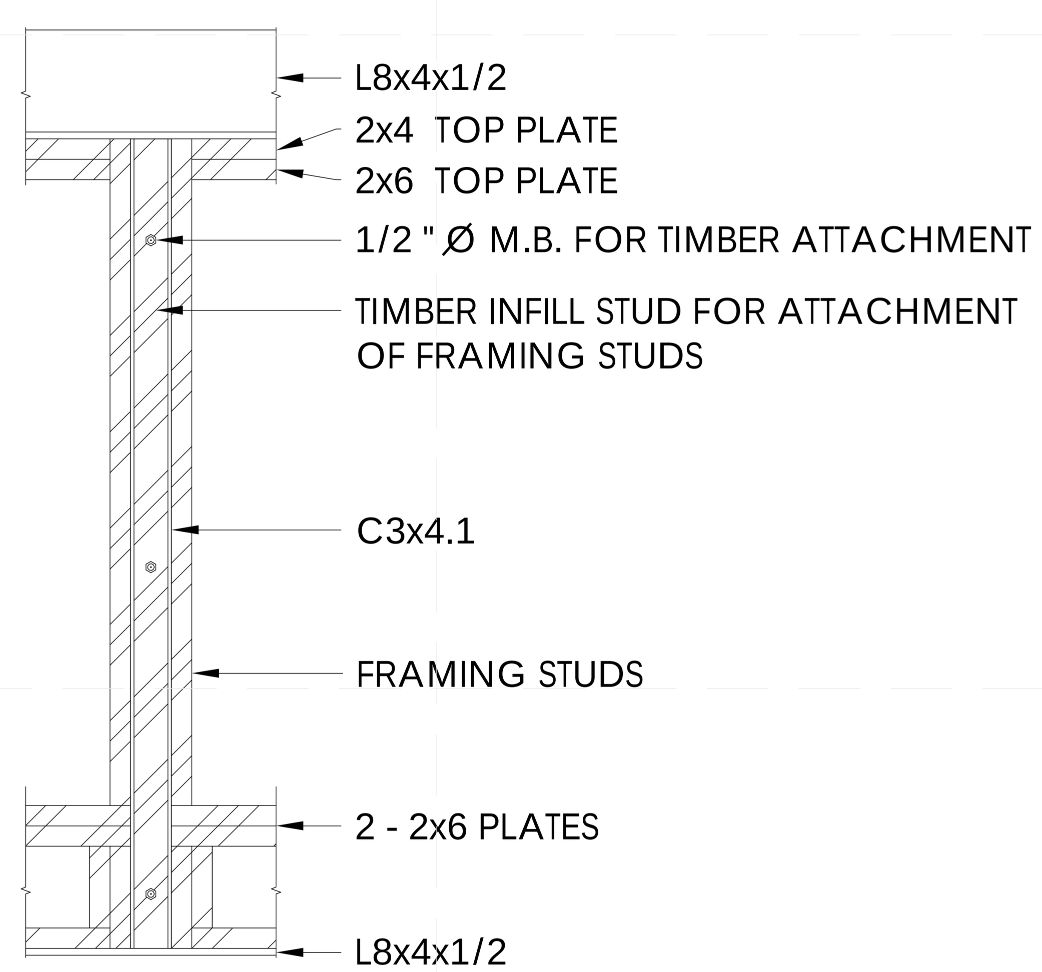
D1 ROOF TOP CONNECTION DETAIL
3/4" = 1'-0"



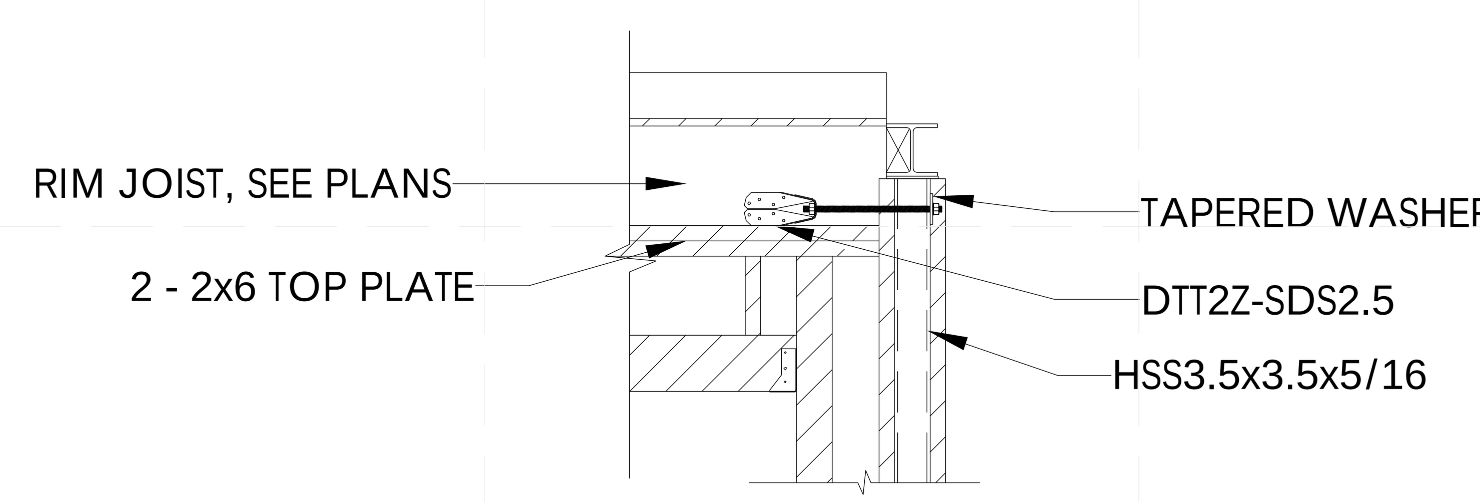
D2 ROOF TIMBER TO ANGLE CONNECTION
1" = 1'-0"



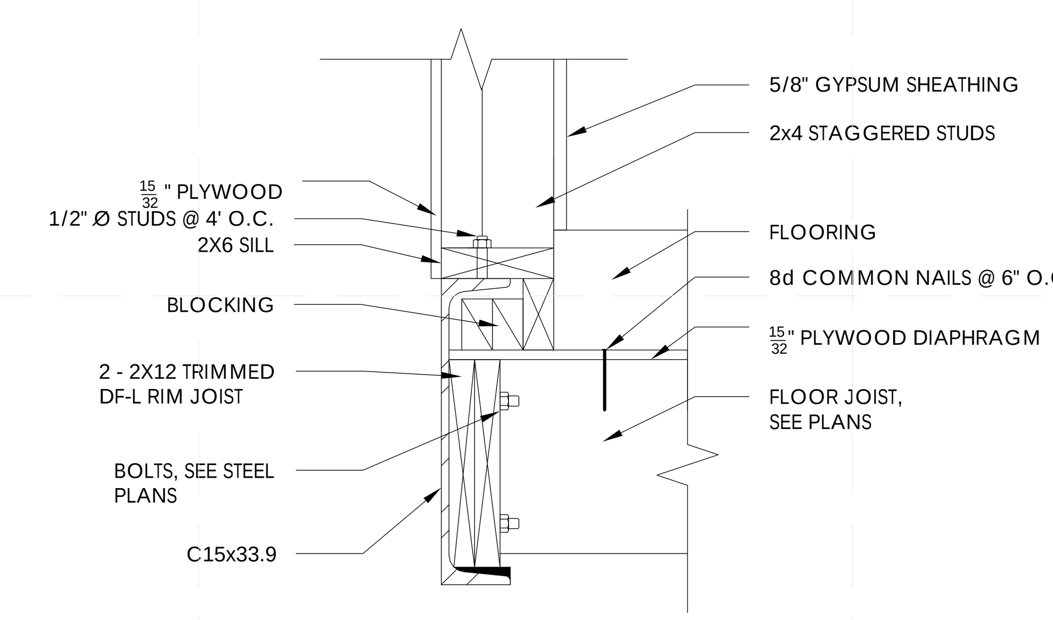
D3 ROOF STEEL CONNECTIONS
1" = 1'-0"



D4 ROOF TIMBER TO CHANNEL CONNECTION
1" = 1'-0"



D5 COLUMN TO TOP PLATE CONNECTION
3/4" = 1'-0"



D6 SILL PLATE TO STEEL CONNECTION
1 1/2" = 1'-0"

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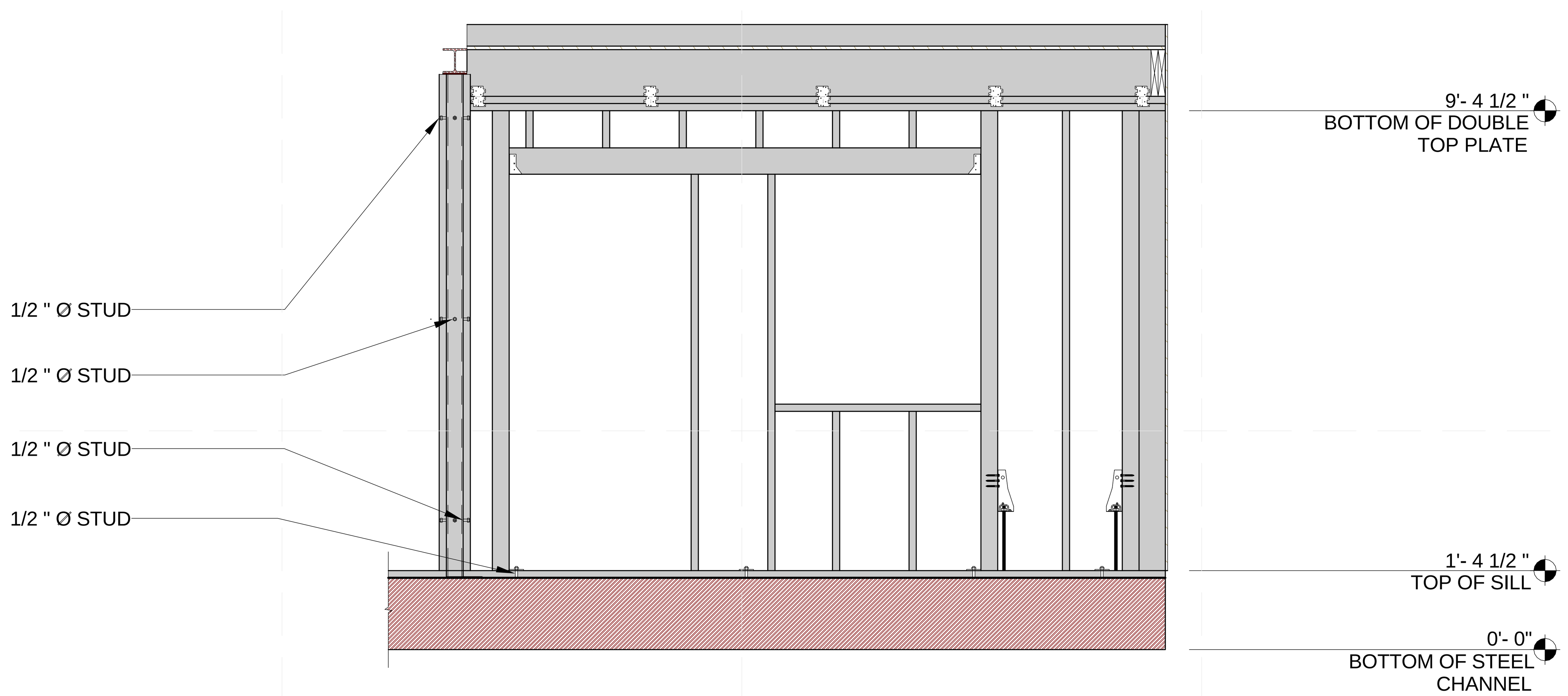
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
CONNECTION DETAILS

S-531



D1 TIMBER TO STEEL CONNECTIONS

1/2" = 1'-0"

0 1' 2' 3'

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	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118

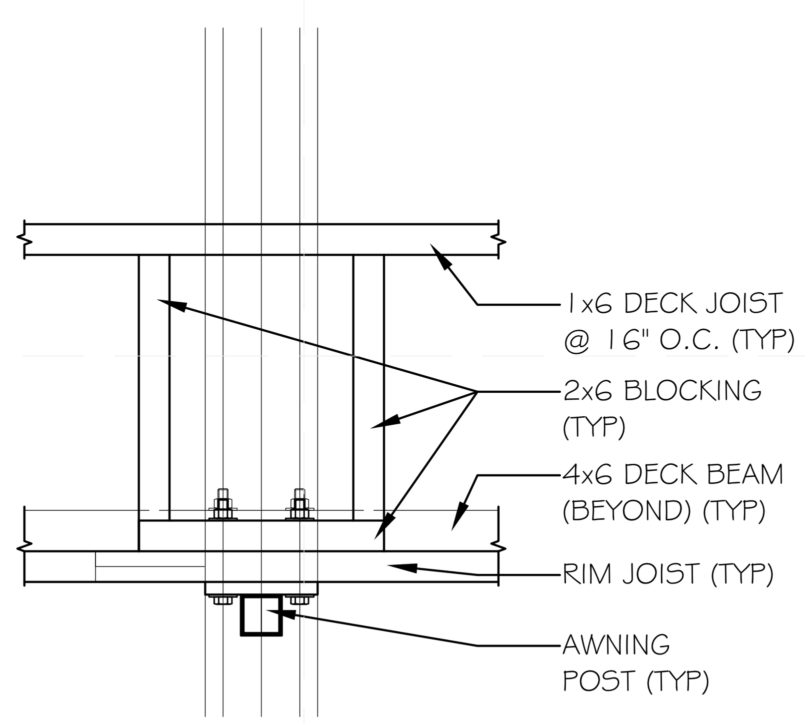
DRAWN BY: ML

CHECKED BY: SCU

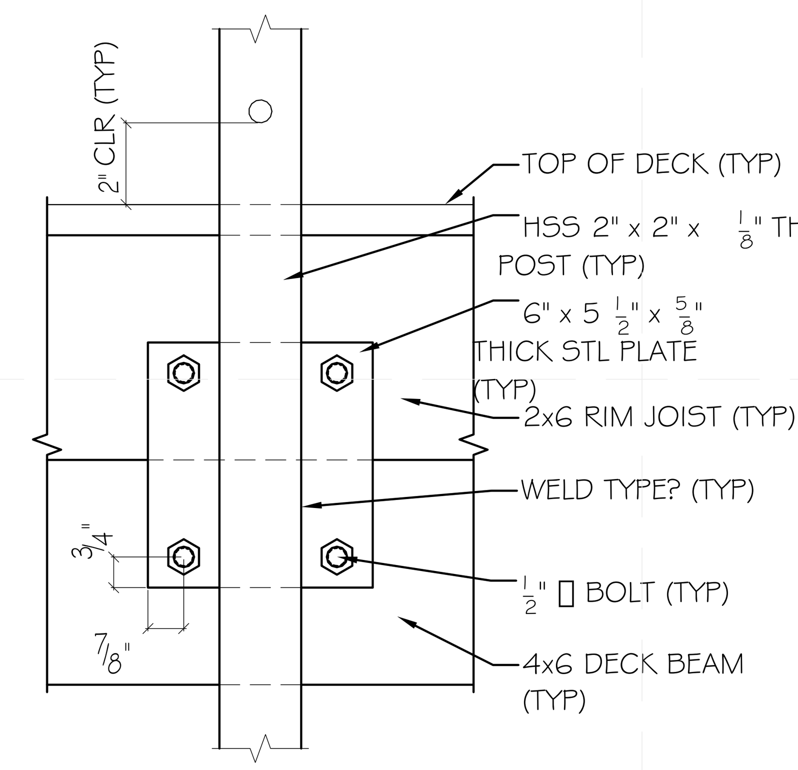
SHEET TITLE:
TIMBER TO STEEL CONNECTIONS

S-532

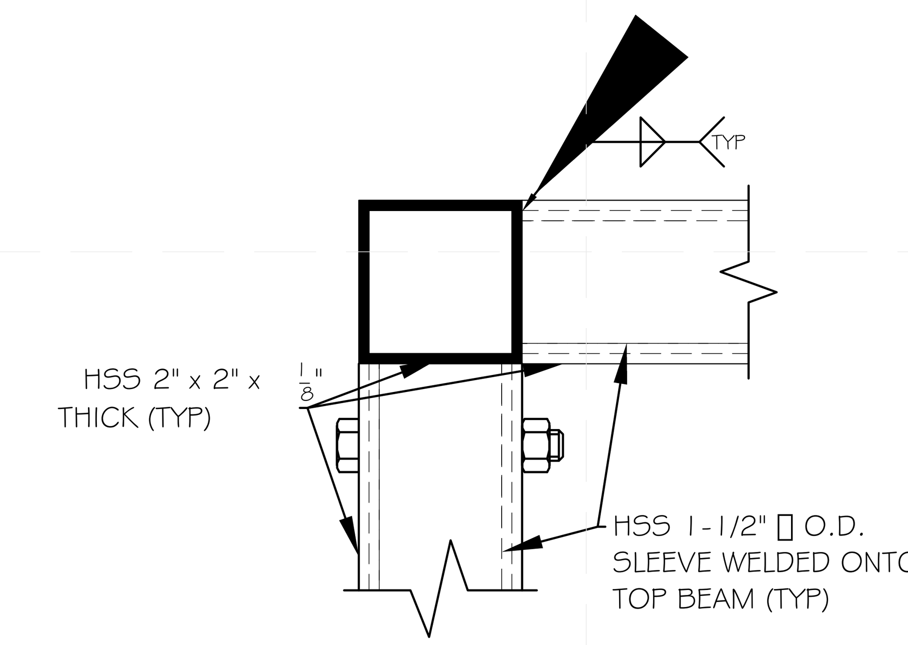
9/5/2013 10:24:23 AM



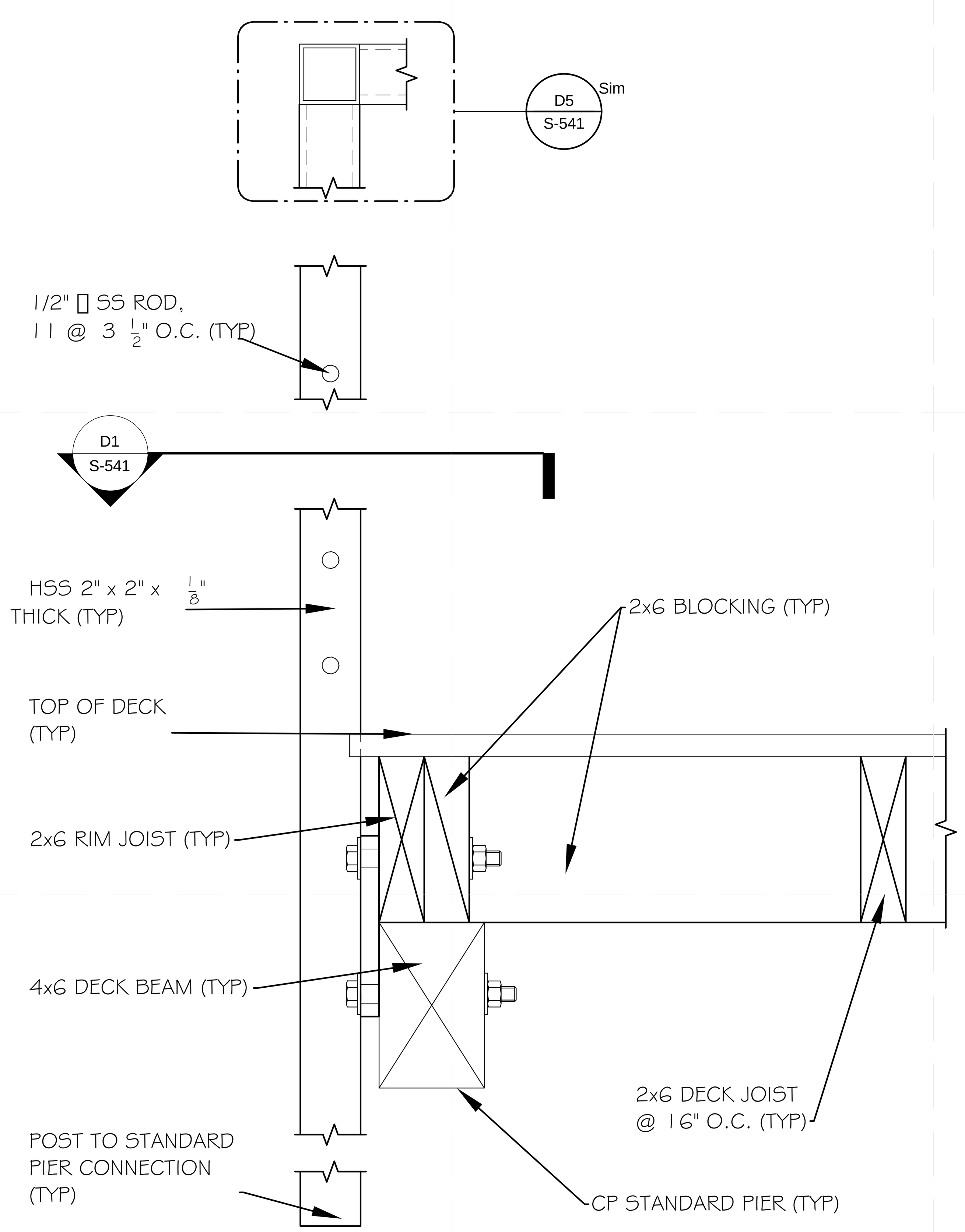
D1 TYP.AWNING POST CONNECTION W/ PARALLEL JOISTING PLAN DETAIL
1 1/2" = 1'-0"



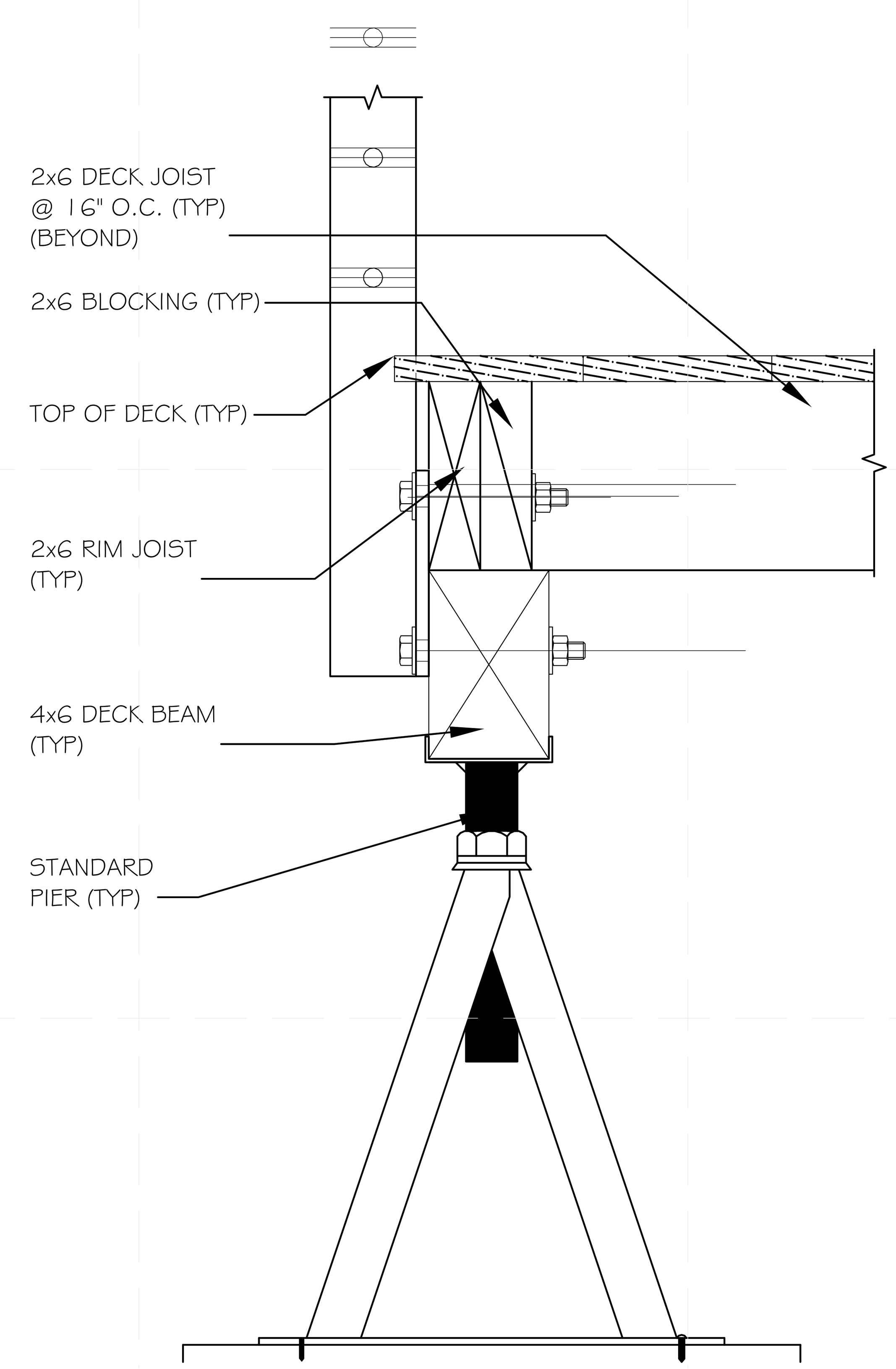
D3 TYP. AWNING POST TO DECK CONNECTION
3" = 1'-0"



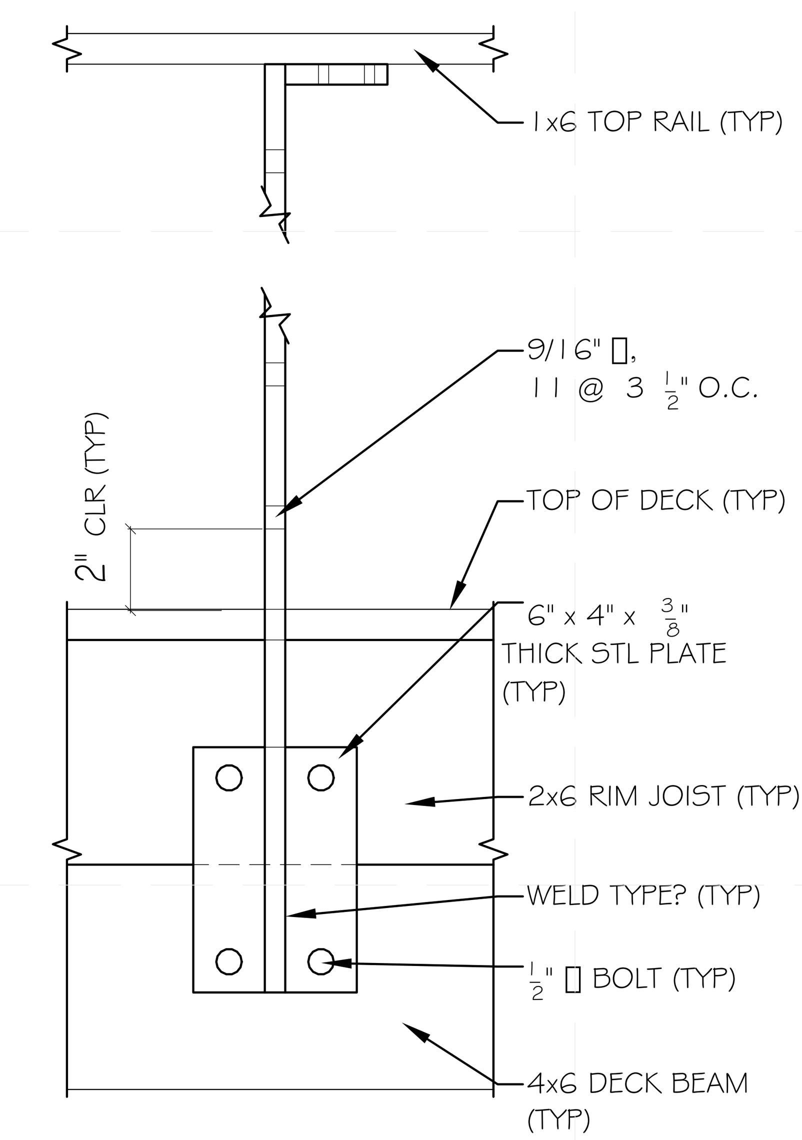
D5 AWNING POST TO TOP BEAM CONNECTION SECTION DETAIL
6" = 1'-0"



A1 AWNING POST SECTION W/ PARALLEL JOISTING
3" = 1'-0"



A3 TYP.GUARDRAIL CONNECTION W/ PERPENDICULAR
3" = 1'-0"



A5 TYP.GUARD RAIL CONNECTION ELEVATION
3" = 1'-0"

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	02/14/13	CD SUBMISSION
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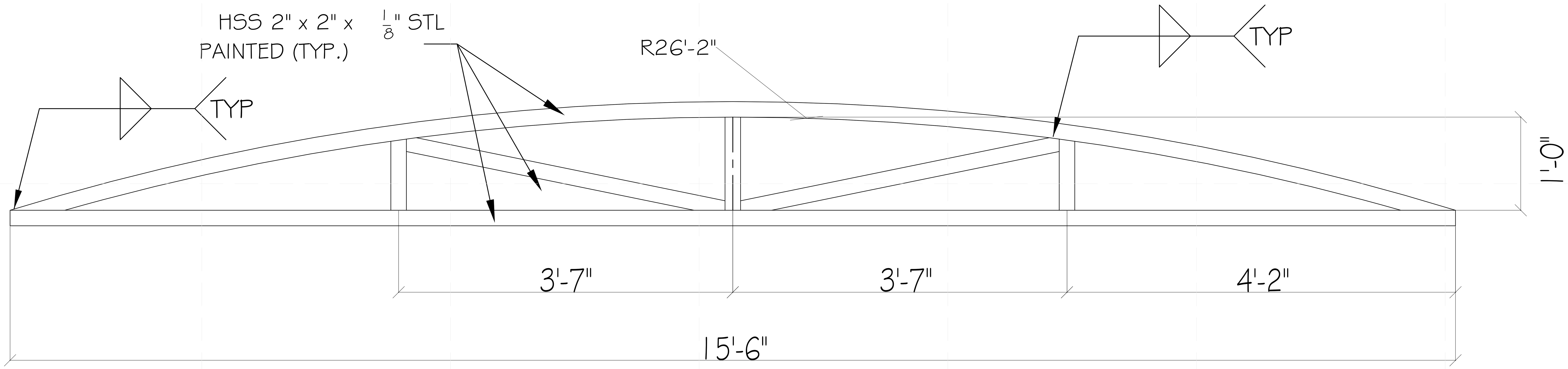
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: AT/PT
CHECKED BY: SCU

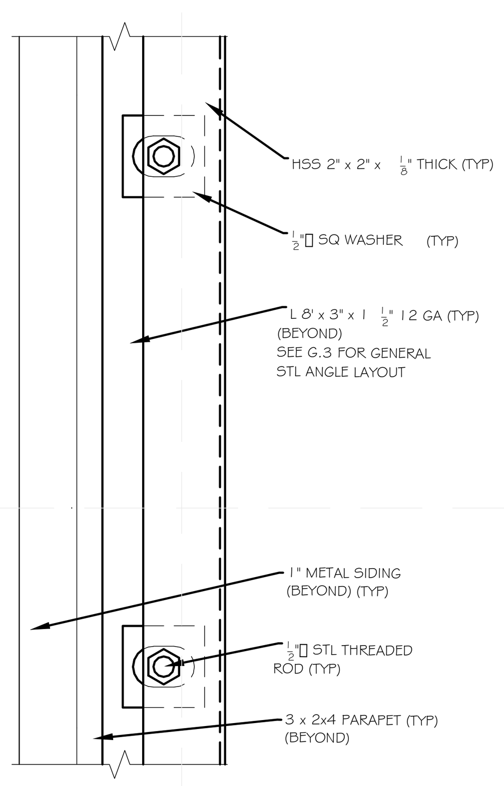
SHEET TITLE:
RAILING DETAILS

S-541

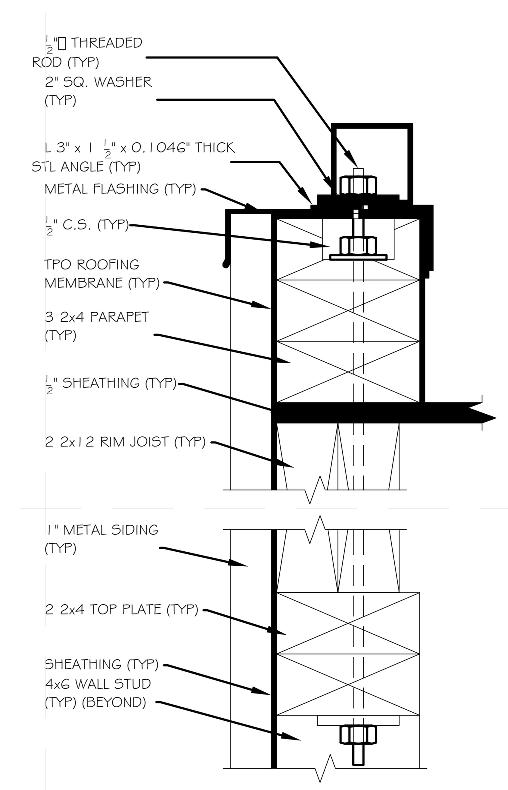
9/5/2013 10:40:12 AM



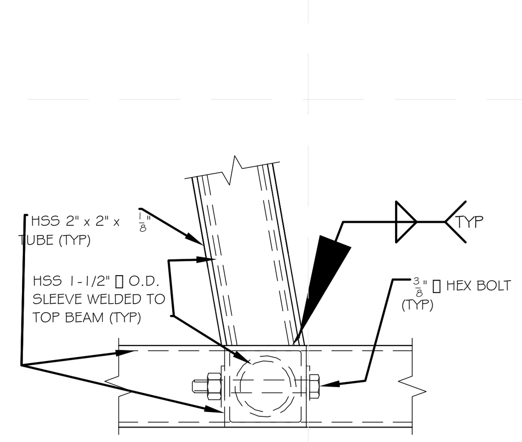
D1 TRUSS ELEVATION
1 1/2" = 1'-0"



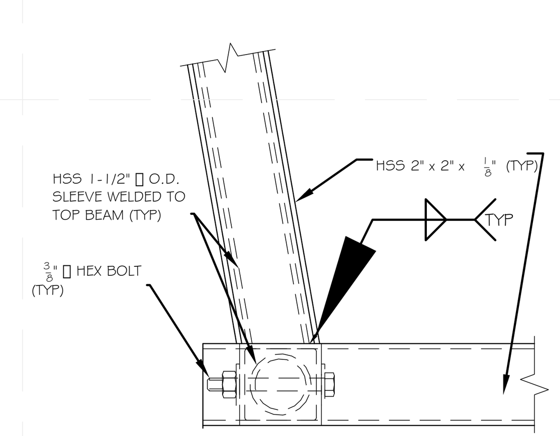
B1 TRUSS TO STL ANGLE CONNECTION PLAN DETAIL
3" = 1'-0"



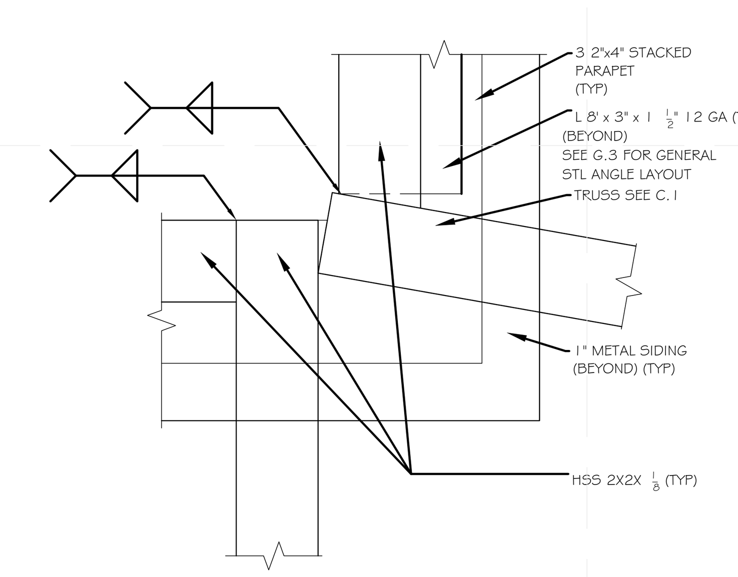
B3 SECTION DETAIL TYPICAL STEEL ANGLE CONNECTION TO ROOF STRUCTURE
3" = 1'-0"



A1 TYPICAL BRACE MEMBER TO TOP BEAM CONNECTION PLAN DETAIL
3" = 1'-0"



A3 TYPICAL END CONNECTION OF BRACE MEMBER AND TOP BEAM PLAN DETAIL
3" = 1'-0"



A5 BRACE MEMBER CONNECTION TO HOUSE PLAN DETAIL
3" = 1'-0"

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	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: AT
CHECKED BY: SCU
COPYRIGHT: NONE: PROJECT IS PUBLIC DOMAIN

SHEET TITLE:
AWNING DETAILS

S-551

9/5/2013 10:52:40 AM

SHEAR WALL SCHEDULE						
WALL	MIN. BASE WIDTH	CAPACITY (PLF)	SHEATHING	EDGE NAILING	SHEAR CLIPS	VERTICAL HOLDOWN
A	2'-10 1/8"	475.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 27" O.C.	HDU2-SDS2.5
B	2'-10 1/8"	475.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 27" O.C.	HDU2-SDS2.5
C	2'-10 1/8"	475.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 27" O.C.	HDU2-SDS2.5
D	2'-10 1/8"	475.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 27" O.C.	HDU2-SDS2.5
E	2'-9"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 26" O.C.	HDU2-SDS2.5
F	7'-5"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 30" O.C.	HDU2-SDS2.5
G	12'-0"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @48" O.C.	HDU2-SDS2.5
H	4'-0"	665.0	15/32" STRUCTURAL 1	10d @ 3	-	HDU8-SDS2.5
I	4'-0"	665.0	15/32" STRUCTURAL 1	10d @ 3	-	HDU8-SDS2.5
J	2'-8 1/2"	510.0	15/32" STRUCTURAL 1	10d @ 4	A35 CLIPS @ 30" O.C.	HDU4-SDS2.5
K	10'-9 1/4"	715.0	15/32" STRUCTURAL 1	10d @ 4	A35 CLIPS @ 21" O.C.	HDU5-SDS2.5
L	2'-8 3/4"	475.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 26" O.C.	HDU2-SDS2.5
M	2'-8 3/4"	475.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 26" O.C.	HDU2-SDS2.5
N	4'-0"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 24" O.C.	HDU2-SDS2.5
O	4'-0"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 24" O.C.	HDU2-SDS2.5
P	4'-0"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 24" O.C.	HDU2-SDS2.5
Q	7'-8"	340.0	15/32" STRUCTURAL 1	10d @ 6	A35 CLIPS @ 30" O.C.	HDU2-SDS2.5

NOTES

1. INTERMEDIATE (FIELD) NAILING SHALL BE 10d @ 12" O.C..
2. ALL NAILS SHALL BE COMMON WIRE GAUGE.
3. SHEAR WALL PLYWOOD SHALL BE FULL HEIGHT.

PLYWOOD DIAPHRAGM NAILING SCHEDULE				
AREA	SHEATHING	EDGE NAILING	INTERMED. STAPLING	REMARKS
A	3/4" T&G	8d @ 6" O.C.	15gauge at 12" O.C.	UNBLOCKED
B	3/4" T&G	8d @ 6" O.C.	15gauge at 12" O.C.	UNBLOCKED
C	3/4" T&G	8d @ 6" O.C.	15gauge at 12" O.C.	UNBLOCKED
Top	3/4" T&G	8d @ 6" O.C.	15gauge at 12" O.C.	UNBLOCKED

NOTES

1. PLYWOOD IS TO BE GLUED TO JOISTS USING LOCTITE PREMIUM ADHESIVE.

TYPICAL JOIST HANGER SCHEDULE					
LOCATION	MEMBER SIZE	SUPPORT	HANGER SERIES	REMARKS	ATTACHMENT
ROOF	TJI 230 (9.5"), BAMBOO JOISTS	TOP PLATE & RIM JOIST	IUS 2.37/9.5	-	8 - SD#9X1.5 INTO FACE
ROOF	TJI 230 (9.5")	STEEL BEAM & LEDGER	LSSUI35	FOR SKEWED JOISTS	9 - 10d INTO TOP & 7 - 10dX1.5 INTO FACE
ROOF	TJI 230 (9.5")	2X12 TRIMMED JOIST	ITS2.37/9.5	FOR SKEWED JOISTS	4 - 10d INTO TOP & 2 - 10dX1.5 INTO FACE
ROOF	TRIMMED 2X12	TOP PLATE & RIM JOIST	LUS28	FOR USE ON SIDE WITH 2-2X HEADER	6 - 10d INTO HEADER & 4 - 10d INTO JOIST
ROOF	TRIMMED 2X12	STEEL BEAM & LEDGER	LUS210	-	8 - 8d INTO HEADER & 4 - 10d INTO JOIST
TOP ROOF	TJI 230 (9.5"), BAMBOO JOISTS	4X12 TRIMMED RIM JOIST	LSSUI35	FOR SLOPED JOISTS	9 - 10d INTO TOP & 7 - 10dX1.5 INTO FACE STIFFENER REQUIRED
TOP ROOF	2-2X10 EDGE JOISTS	4X12 TRIMMED RIM JOIST	HUC210-2	FOR SLOPED EDGE JOISTS	18-16d INTO HEADER & 10-10d INTO JOIST
FLOOR	TJI 230 (9.5")	2- 2X12 TRIMMED RIM JOIST	IUS 2.37/9.5	RIM JOIST RUNS INSIDE THE C15X33.9	8 -10d INTO FACE
FLOOR	2X4	TJI 230 OR 2x4 LEDGER	JPF24	2X4 LEDGER RUNS OVER THE C8X11.5	2 - 10d INTO TOP & 2 - 10d INTO JOIST
WALLS	6X6 HEADER	2x6 STUD OR 4x6 POST	HUC66	FOR 6X6 HEADERS OVER OPENINGS	MIN. 8 - 16d INTO HEADER & 4 - 16d INTO JOIST

NOTES

1. ACTUAL SIZE OF THE TRIMMED 2X12 IS 1.5X9.5.
2. FOR SKEWED LSSUI HANGERS, THE INNER MOST FASTENERS ON THE ACUTE ANGLE SIDE ARE NOT INSTALLED.
3. OK TO SUBSTITUTE FASTENERS WITH FASTENERS OF EQUAL OR GREATER STRENGTH.

FOOTING ADJUSTABILITY SCHEDULE			
TYPE	MIN HEIGHT	MAX HEIGHT	ADJUSTABILITY
SEISMIC PIER	7"	11"	4"
SEISMIC PIER	11"	18"	7"
STANDARD PIER A	4"	6"	2"
STANDARD PIER B	6"	8"	2"
STANDARD PIER C	8"	10"	2"
STANDARD PIER D	10"	12"	2"
STANDARD PIER E	12"	14"	2"
STANDARD PIER F	14"	16"	2"
STANDARD PIER G	16"	18"	2"

NOTES

1. VARIOUS SIZED PIERS PROVIDED TO ACCOMODATE 18" SITE VARIABILITY.
2. SEE S-101 FOR FOUNDATION PLAN.
3. REFER TO FOOTING SCHEDULE.

FOOTING SCHEDULE			
MARK	TYPE	SIZE	MOUNTING OBJECT
F1	STANDARD PIER	16" X 16"	STEEL CHANNEL
F2	SEISMIC PIER	24" X 24"	STEEL CHANNEL

NOTES

1. FOR TYPICAL DETAILS SEE S-501 FOUNDATION DETAILS.
2. SEE S-101 FOR FOUNDATION PLAN.
3. SEE FOOTING ADJUSTABILITY SCHEDULE FOR FOOTING SIZES AND GRADE VARIABILITY.

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U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON

CLIENT: U.S. DEPARTMENT OF ENERGY
SOLAR DECATHLON 2013
WWW.SOLARDECATHLON.GOV

MARK	DATE	DESCRIPTION
	08/22/13	DOE AS BUILT SUBMISSION
	08/12/13	AS BUILT REVISIONS
	08/06/13	CD CITY INSPECTION 3
	07/26/13	CD CITY INSPECTION 2
	07/19/13	CD CITY INSPECTION
	05/10/13	CD RESUBMISSION 2
	04/04/13	CD RESUBMISSION
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
SCHEDULES

S-601

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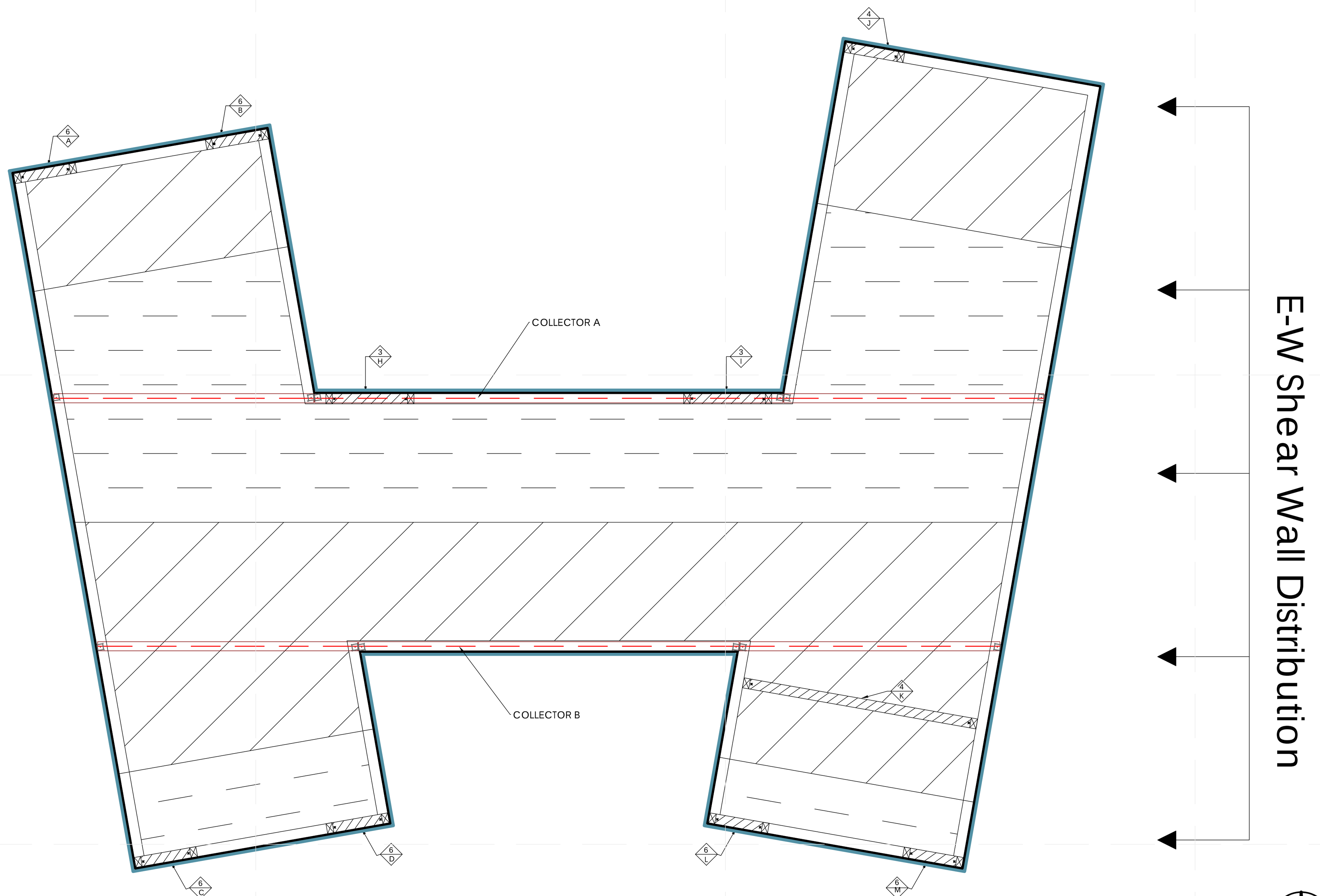
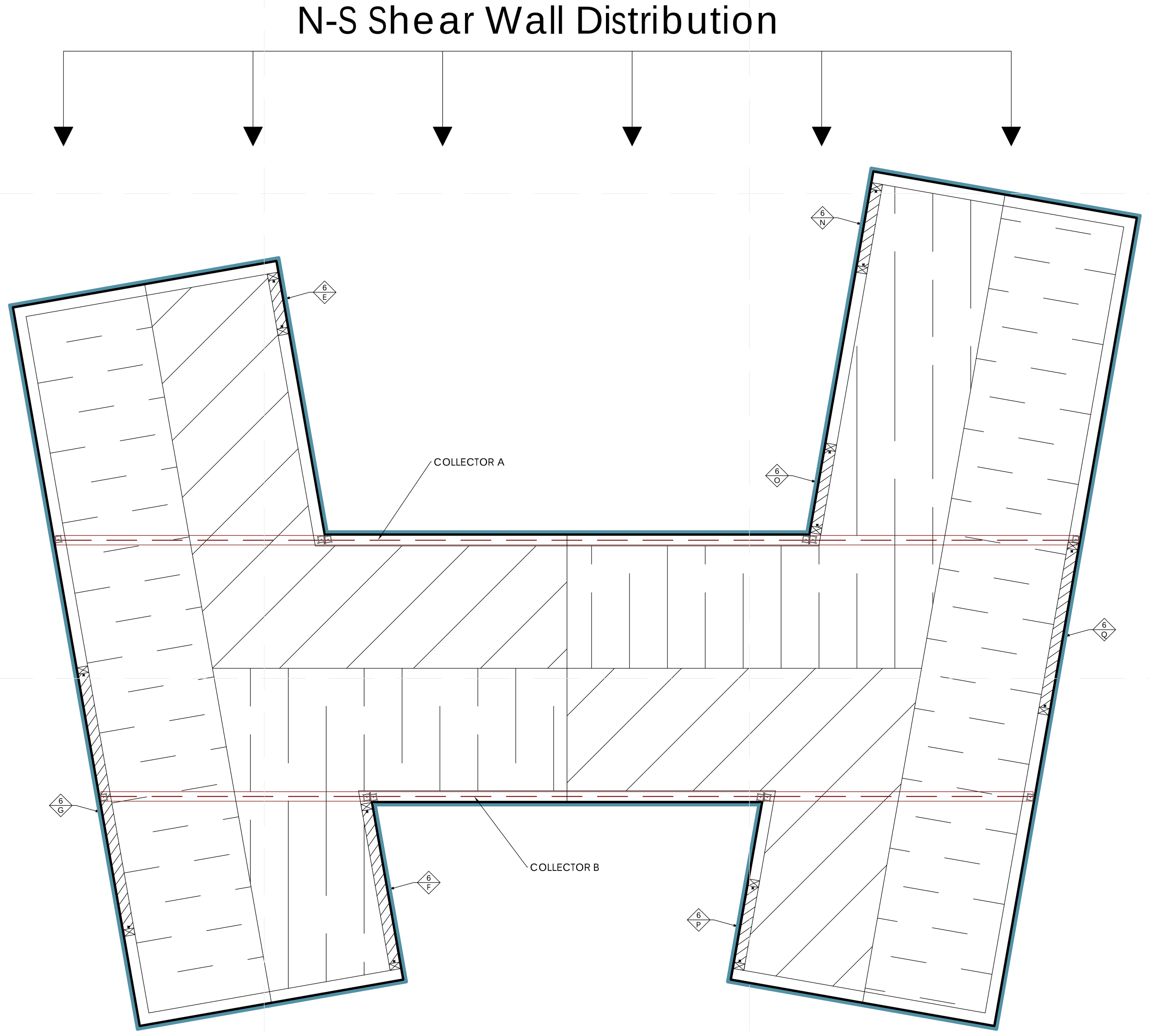


C1 E-W shear wall distribution
3/16" = 1'-0"

N-S Shear Wall Distribution



A1 N-S Shear Wall Distribution
3/16" = 1'-0"

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MARK	DATE	DESCRIPTION
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	08/12/13	AS BUILT REVISIONS
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	04/04/13	CD RESUBMISSION
	02/14/13	CD SUBMISSION
	11/20/12	DD RESUBMISSION
	10/11/12	DD SUBMISSION

LOT NUMBER: 118
DRAWN BY: ML
CHECKED BY: SCU

SHEET TITLE:
SHEAR WALL DISTRIBUTION

S-650

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