CFS-NEES
Advancing Cold-Formed Steel Earthquake Engineering

RESEARCH REPORT

CFS-NEES Building Structural Design Narrative

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CFS-NEES - RR01c

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This document was originally released in October, 2011 and amended in April, 2012 to reflect changes to the shearwall analysis to correct an error in the aspect ratio adjustment, match the analysis more closely to the final drawings, manually adjust the stiffness estimates to ensure deflection compatibility along each line of shear and make minor editorial changes to the drawings.

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INTRODUCTION

The NSF sponsored CFS-NEES[1] project R-CR: Enabling Performance-Based Seismic Design of Multi-Story Cold-Formed Steel Structures project was undertaken to study the behavior, particularly seismic behavior, of light-framed structures using cold-formed steel cee-sections as the primary gravity load carrying elements with wood structural panel diaphragms and shearwalls as the primary lateral load resisting system.

Devco Engineering, Inc. was selected to develop design calculations and drawings for the structure based on criteria determined by the research team. Input on the details of design was also sought from industry professionals through the Industry Advisory Board (IAB)[2]. The details developed in the design phase will be studied via component and full-scale shake table testing of the structure.

This report discusses the design of the gravity and lateral systems for the CFS-NEES building. Specific calculations and drawings are attached herewith as appendices for reference.

Design Criteria

Design of the structure was based on a site in Orange County, California. Gravity and lateral loads were determined per the 2009 edition of the International Building Code (IBC) based on this location.

For member sizing, the "North American Specification for the Design of Cold-Formed Steel Structural Members", 2007 edition (AISI S100-07) was used. Member callouts were based on SSMA/SFIA criteria. Shearwall and diaphragm design was based on the "North American Standard for Cold-Formed Steel Framing – Lateral Design", 2007 edition (AISI S213-07).

Wind and seismic forces were determined based on a location at 520 W. Walnut Blvd, Orange, California (latitude 33.8 degrees; longitude -117.86 degrees).

For simplicity, and consistent with industry standards, allowable strength design (ASD) was used for members and connections not part of the lateral force resisting system (LFRS). For design of the LFRS, load and resistance factor design (LRFD) was used.

Architectural Concept

The architectural concept for the CFS-NEES building was developed by the project team. See Appendix 6 for a rendition of the architectural concept.

[1] See www.ce.jhu.edu/cfsnees for details

[2] See www.ce.jhu.edu/cfsnees/advisoryboard.php for member list

Calculation Systems and Notations
Calculations were developed using standards employed at Devco Engineering for page numbering and labeling of attached documents. The following describes the system used:

The particular element design being undertaken is double underlined at the top of the first page associated with the design of that element. The criteria used to size the element, for example loading, span lengths and any special considerations follow. Final member or connection selection is double underlined with an arrow on the right hand side of the page.

Computer printouts or other associated documents related to a specific element design are attached behind the hand calculations for that element. These supplemental documents are referenced by a number inside a hexagon on the hand calculations and the same symbol and number can be found in the upper right hand corner of the related printout.

Where spreadsheet printouts are provided in the appendices, black values are labels, blue values are user inputs and red values are calculated within the spreadsheet.

Software
The following software was used in the development of the calculations:

- AISIWIN version 8, Devco Software, Inc. Used for member sizing of simple span members with uniform loads and axial loads were applicable.
- LGBEAMER version 8, Devco Software, Inc. Used for member sizing of more complex span and load conditions.
- Microsoft Excel: Used to develop spreadsheets for lateral analysis and other general purpose calculation tasks.

Member Nomenclature
Member designations were used per SSMA/SFIA standards.

Appendices
Appendices 1-5 attached contain the following:

Appendix 1: Framing Member Design
Appendix 2: Seismic Lateral Analysis
Appendix 3: Shearwall and Diaphragm Analysis and Design
Appendix 4: Lateral System Design – Supplemental Calculations
Appendix 5: Design Drawings dated 10/31/11
Appendix 6: Architectural concept drawings
Appendix 7: Rigid Diaphragm Analysis

Structural Design Summary

Gravity System

Based on input from the IAB, a 'ledger framing' system was chosen rather than traditional platform framing. According to the IAB, ledger framing which attaches floor and roof joists to the inside flanges of the load-bearing studs via a combination of track and clip angles is currently the dominant method of construction. Studs are broken at the top of each floor level and capped with a track. Walls above are stacked on the lower wall top track. See Appendix 5, details 1 and 2/SF4.40.

Roof Joists

Roof joists were designed as simple span members with uniform loading. End rigidity of the attachment to the stud walls was not considered in the roof joist design. Design loads included 20 psf dead load, 20 psf live load and wind uplift per IBC requirements. Note that for the effective wind area associated with the joist spans for this building, maximum corner wind uplift was calculated at 14.1 psf and thus was not a significant concern in the design.

Roof joist deflection was limited to L/240 for dead load and L/180 for total loads. For distortional buckling, k_ϕ was conservatively taken as zero. Had additional flexural strength been required, the k_ϕ value appropriate for the joists selected and OSB sheathing on the compression flange could have been used.

Based on these loads and a maximum clear span of 22 feet, 1200S200-54 joists at 24 inches on center were selected. The compression flange of the joists was considered to be continuously braced via attachment of sheathing. In accordance with industry standards, two rows of bridging were specified in order to minimize joist rotation.

Because the web height-to-thickness for the selected joists exceeded 200, web stiffeners were required at member ends. Stiffening was accomplished with clip angles screwed to the joist and to the rim (ledger) track. This method transfers the reaction from the joist web to the support in direct shear rather than bearing, thus precluding web crippling failure in the joists.

Rooftop mechanical units each weighing up to 600 lb were anticipated. Design of the joists for support of these units was based on the load being distributed to at least two joists with two 150 lb point loads supported by any individual member. Based on these loads, back-to-back 1200S200-54 joists were specified at mechanical unit supports.

Roof joist design, including sizing of joists at mechanical units and connection of joists to exterior walls can be found in Appendix 1, page R-1. Drawings related to roof joists can be found in Appendix 5, sheets SF4.02, SF4.20 and SF4.40.

Floor Joists

In addition to the standard 18 psf dead load to account for framing, sheathing, flooring and the like, a 15 psf partition load was included to account for partitions that may be moved at various times during the structure's life span. Live load for floor joist design varies by location. For example, the typical live load is 50 psf but 80 psf is required at corridors. As such, joists were designed as simple span members with varying distributed loads. Similar to the roof joists, end rigidity of the connection to the wall was not considered.

Deflection limits of L/240 for total loads and L/360 for live loads were used. For distortional buckling, k_ϕ was conservatively taken as zero. Had additional flexural strength been required, the k_ϕ value appropriate for the joists selected and plywood sheathing on the compression flange could have been used.

Based on the above, 1200S250-97 joists 24 inches on center were selected. The compression flange of the joists was considered to be continuously braced via attachment of sheathing. Two rows of bridging were specified in order to minimize joist rotation. In addition, due to the high end reactions and relatively short bearing length, web stiffeners were required at joist ends. Stiffening was accomplished in the same way as at the roof, but with additional fasteners required for the higher loads.

At the clerestory opening, single track headers were designed to carry floor joist loads to carrier joists on either side of the opening. A 1200T200-68 was chosen for the 8'6" span. Carrier joists were designed for a distributed load equal to one half of that used at typical joists in combination with the concentrated loads from the headers on each side of the opening. Single 1200S350-97 carriers were selected.

Floor joist analysis and design is found in Appendix 1, pages F-1 and F-2. Drawings for floor joists can be found in Appendix 5, sheets SF4.01, SF4.20 and SF4.40.

Load-bearing Walls

For a desired clear height of framing of 8'0" and 12" deep joists, studs were designed as 9 ft. in length. Code prescribed wind loads, when reduced for area, were less than 15 psf. As such, a slightly conservative value of 15 psf wind load was used for stud design.

Studs above the 2nd floor platform were designed to carry wind load in combination with roof dead and live loads. Load combinations per ASCE 7-05 were used. The total gravity load of 440 lb/stud was used based on the roof joist reactions. Gravity loads were applied at the inboard stud flange, resulting in an end eccentricity of 3 inches to the center of the studs. Since walls will receive gypsum board sheathing on at least one flange, k_ϕ for distortional buckling was taken as zero per CFSEI Technical Note G100-08. Based on these criteria, 600S162-33 studs at 24 inches on center were chosen. The studs were acceptable with either sheathing bracing, or discrete bracing near mid-height. Since some tests may be performed without interior sheathing, discrete bridging (noted as CRC, or cold-rolled channel in the calculations) will be required for these tests.

With the stud size known, the connection of the roof joists to the wall was designed. The connection was designed for shear due to gravity loads plus tension due to outward acting wind loads (suction) on the wall studs.

In order to allow the roof diaphragm to extend over the top of the level 2 walls, the parapet was designed as a free-standing cantilever. Track and fasteners were chosen to resist the associated overturning forces.

Walls running perpendicular to the joists transfer out of plane lateral forces to the diaphragm via their connection to the joists. However, for walls parallel to the joists, transferring out of plane wall forces into the diaphragm is accomplished via a direct connection of the wall to the diaphragm sheathing. For plywood to steel connections, allowable screw forces were based on the American Plywood Association publication APA E830D “Technical Note: Fastener Loads for Plywood – Screws”, dated August 2005.

Lower level walls were designed similarly to the upper level walls except that in addition to roof gravity loads, floor gravity loads were also considered. Gravity loads from the roof and wall above were considered concentric. Gravity loads from floor joists were applied at the inboard stud flange, thus introducing an eccentricity of half the stud width or 3 inches. On this basis, 600S162-54 studs @ 24 inches on center with discrete bridging at mid-height were chosen.

With the stud size known, the connection of the floor joists to the wall was designed. The connection was designed for shear due to gravity loads plus tension due to outward acting wind loads (suction) on the wall studs.

At the stair clerestory the carrier joists apply concentrated vertical loads to the 1st floor wall studs. Based on the maximum load from the carrier joists and from the roof and wall above, it was determined that two 600S162-54 studs would be required along with additional fasteners from the rim track to the studs.

At the northwest exit stair, the 2nd floor joists are supported by an interior wall. The interior wall is subjected only to 5 psf partition pressure and does not support roof gravity loads. Accordingly, these studs were sized as 362S162-54 at 24 inches on center with bridging at 48 inches on center.

Additionally at the northwest exit stair, the exterior wall studs span the full 18’ 0” height to the roof joists. These studs support only roof gravity loads. On this basis, the studs were sized as 600S162-54 at 24 inches on center with bridging at 48 inches on center.

Design of structural walls can be found in Appendix 1, pages W-1 through W-5. Drawings depicting the load-bearing walls can be found in Appendix 5, sheets SF4.20, SF4.30 and SF4.40.

<u>2nd Floor Wall Openings</u>

To support loads around window and door openings, headers, sill and jambs were sized. A maximum opening width of 8’ 0” was considered. For windows, openings were considered to be 4’ 0” tall with a sill height of 2’ 6”.

For openings at the 2nd floor, the perimeter rim track or joists were found to have sufficient capacity to carry gravity loads over the opening. As such, no additional gravity header was specified.

Header and sill tracks were sized as 600T150-33 to carry a 15 psf lateral load from jamb-to-jamb. The connection of these members to the jamb studs was designed to support 196 lb of lateral shear. Per AISI S100-07 section E4, The shear capacity of a #10 sheet metal screw in 33-mil steel is 177 lb/screw. As such, (4) #10 as specified is, by observation, adequate.

Jamb studs were sized based on the lateral reactions from the header and sill as well as the eccentric vertical reaction from the rim track or joist above. To account for the eccentric nature of gravity loads, a moment couple was included based on 3 inches of eccentricity and a 12 inch deep member. An option for using two 600S162-33 or a single 600S162-54 jamb was provided. Interconnection of the two-member configuration was designed per AISI S100-07 D1.2.

Design of the jamb/rim track connection considered the concentrated shear due to gravity loads as well as the top of jamb lateral reaction from the jamb analysis. Screw quantity was determined based on minimum 33-mil jambs.

Design of openings in the 2nd floor walls can be found in Appendix 1, pages W-6 and W-7. Framed opening drawings can be found in Appendix 5, sheet SF4.50.

<u>1st Floor Wall Openings</u>
For the long side of the structure, the 1200T200-97 rim track above openings was analyzed and found to be sufficient to carry gravity loads over openings up to 6' 6" in width. For larger openings, two 1200S250-97 were specified. The two 1200S250-97 header members were also specified for openings where clerestory carriers were supported.

For the short side of the structure, the maximum opening was 6' 0" in width. As such, the 1200S250-97 end joist could easily carry the gravity loads over the opening.

Head and sill tracks were sized as 600T150-54 for 15 psf lateral pressures.

Jambs were designed with considerations similar to those at the 2nd level, but with additional gravity loads from the structure above. On this basis, an option for two 600S162-54 or a single 600S200-68 were specified.

For large openings where gravity loads were exceptionally high, rather than rely on the screw shear to support the entire gravity loads, trimmer studs (studs immediately below the header that support header gravity loads as axial loads) were designed to provide a bearing type support for the header. 600S162-54 trimmers in combination with 600S162-54 king, or jamb studs were specified.

Design of openings in the 2nd floor walls can be found in Appendix 1, pages W-8 through W-10. Framed opening drawings can be found in Appendix 5, sheet SF4.50.

<u>Lateral System</u>
Because testing will be based on shake-table simulated seismic forces, the design of the lateral system focused on seismic design.

Lateral forces were determined based on mapped short period spectral response acceleration parameter, S_s, and mapped 1-second spectral response acceleration parameter, S_1 for the location described previously. Site Class D was chosen as is typical for sites in the vicinity of this project. For the office occupancy chosen, I_E = 1.0 was used.

Lateral resistance was provided by wood structural panel shearwalls. For this system, the following parameters were derived from ASCE 7-05 Table 12.2-1:

Response Modification Coefficient, R = 6.5
Overstrength Factor, Ω_0 = 3
Deflection Amplification Factor C_d = 4

The resulting base shear coefficient was calculated as C_s = 0.143.

The effective seismic weight, W used in ASCE 7-05 Eq'n 12.8-1 was based on estimated weights of roof, floor and exterior walls. A 1200 lb allowance for roof top MEP was included. In addition, per ASCE 7-05 section 12.7.2, a 10 psf allowance for partitions was included on the 2nd floor. Reduced seismic weight due to stair openings in the 2nd floor were not considered as the weight of attached stair elements would likely counteract any reduction in floor mass. A total seismic weight of approximately 78 kips was determined; resulting in a seismic base shear force of approximately 11 kips.

The vertical distribution of the calculated shear was based on ASCE 7-05 section 12.8.3. The design shear forces at the roof and 2nd levels were determined to be roughly 6.5 and 4.5 kips respectively.

Calculation of C_s, W and the seismic shear at each level is shown in Appendix 2, page 1 and Appendix 1, sheet L-2.

Shearwalls

Based on the proposed location of windows and doors, shearwall locations were selected on each of the (4) perimeter walls. Both Type I and Type II shearwalls were investigated. However, for this structure, the Type II shearwalls did not, in the opinion of the investigators and the IAB, provide a significant benefit. As such, Type I shearwalls were selected throughout.

The size and location of shearwalls on each side of the building varied. As such, the horizontal distribution of shear was determined based on an estimate of shearwall stiffness. Shearwall stiffness was estimated based on AISI S213-07 Eq'n C2.1.1. Note that shearwall stiffness determined using this method varies with applied load. As such, preliminary estimates of stiffness based on a nominal 1000 lb of shear force were adjusted by trial and error until reasonable deflection compatibility was achieved along each line of shearwalls. Spreadsheets were developed to allow interactive design of the shearwall with changing stiffness. See Appendix 3, sheet 1 for the initial estimate of horizontal shear distribution along each line of shear. Appendix 3, sheet 2 provides the adjusted values of shear distribution as determined by trial and error.

Based on the force distribution, shearwalls were selected per the procedures of AISI S213-07. OSB sheathing was selected on the basis of economy of OSB and on the fact that for 54-mil and heavier framing, a fixed maximum aspect ratio of 2:1 applies to Structural 1 sheathing but not to OSB. The typical 2nd floor stud framing was specified as 33-mil, but in order to meet strength requirements 54-mil chord studs were selected. Also minimum 43-mil top and bottom track were specified. Therefore, shear values applicable to 33-mil (upper level) or 54-mil (lower level) framing members were used. Per Table C2.1-3 of AISI S213-07, for edge fasteners at 6 inches on center, the nominal shear strength of the assembly selected was 700 lb/ft (upper level) and 825 lb/ft (lower level). Analysis of the individual shearwalls is found in Appendix 3, sheet 2.

ASCE 7-05 Table 12.12-1 limits seismic story drift to $0.025h_{sx}$ for the type of structure contemplated where h_{sx} is the story height. Drift was determined based on AISI S213-07 Eq. C2.1-1 and found to be within this limit for each wall. The data indicates that displacement is dominated by the non-linear term and the anchor/hold-down term of Eq. C2.1-1. For the upper level shearwalls, the anchor/hold-down term was estimated as no data was available for the system used that includes strap elongation as well as fastener slip. For the lower level shearwalls, the anchor/hold-down term was based on data published by the hold-down manufacturer. Displacement analysis can be found in Appendix 3, sheet 3. It should be noted that this displacement is based on the stiffness only of the shearwalls and does not account for additional stiffness provided by non-structural wall panels or the rigidity of the wall to floor connections.

For ease of reference in calculations per Appendix 2 shearwalls were labeled based on their location on the structure. For example, shearwall L2N1 is the first Level 2 shearwall (L2) located on the north side (N1). Note that the analysis of shearwall L1E2 was based on a length of 6 feet while drawings indicate an 8 foot length. The 8 foot length was used based on the window locations to avoid an awkward sheathing infill adjacent to the windows. Because the design is somewhat conservative, calculations were not revised.

See Appendix 5, sheets SF4.00, SF4.10, SF4.11 and SF4.30 for shearwall drawings. Appendix 5, sheet SF4.00 indicates the direction of north used in the calculations.

<u>Shearwall Chord Studs</u>

Shearwall chords were designed for load combinations per ASCE 7-05, section 2.3.2 including dead, live and both lateral and vertical seismic loads. Eccentric moment due to both gravity (ledger on inside face of stud) and seismic (shear panels on outside face of stud) loads were included. Chords were sized based on basic LRFD load combinations in addition to the strength requirements of AISI S213-07, C5.1.2.

Chord stud strength was checked at the minimum of the amplified seismic load, or the maximum seismic load the system can deliver as allowed in AISI S213-07. As described above a lower bound value of shearwall nominal strength, based on the field stud thickness and yield point, was conservatively used for evaluating the sheathing and fasteners at the upper level shearwalls. However, for design of chord studs based on the ‘maximum load the system can deliver’, use of this lower bound, field stud, value is unconservative. Therefore, the nominal shearwall strength was increased to the upper bound strength (i.e. strength based on chord stud thickness and yield point as opposed to that of the field studs) for sizing chord studs. Based on this analysis, two 600S162-54 back-to-back chords were selected for both the 1st and 2nd levels. Note that one chord stud beam-column interaction value of 1.028 was calculated. Based on the minor level of the overstress and the presence of sheathing that is unaccounted for in the analysis, this was considered acceptable. Chord analysis can be found in Appendix 3, sheet 4.

Near the northwest exit stair, the shearwall encroached into the balloon framed area at the stair opening. In order to retain the 2-story design typical throughout the remainder of the structure, the portion of the balloon wall used for shear resistance was framed with 'stacked studs'. The tracks between the 1st and 2nd floor wall studs were sized to resist out of plane lateral forces between the edge of the 2nd floor diaphragm and a jamb stud on the opposite end of the shearwall. A full height 2-story jamb stud was sized to resist these out-of-plane forces as well as act as a chord stud for the shearwall. Note that since the outer stud of the jamb/chord is continuous, no chord tie was required between the 2nd and 1st floors in this location. The design of this system can be found in Appendix 4, sheets SW-1 through SW-2. Drawings for the shearwall chord studs can be found in Appendix 5, sheet SF4.30.

<u>Ties and Hold-downs</u>

Shearwall ties and hold-downs were sized in accordance with the requirements of AISI S213-07, C5.1.2. Resisting dead load was reduced for vertical seismic force per ASCE 7-05, 12.4.2.3.

For the 2nd floor ties, a strap system was chosen to transfer forces from the 2nd floor chords to the 1st floor chords. To avoid crushing the plywood that runs between the bottom track at the 2nd floor and the top track of the 1st floor, straps were sized for both compression and tension. An unbraced length, KL = 3 inches was conservatively used for the compression analysis based on a maximum 3 inch vertical spacing between upper and lower fasteners. Strap design considered both yielding of the gross section and fracture of the net section. For net section fracture, area was reduced for a maximum of two screw holes based on the design utilizing two vertical rows of fasteners. Both LRFD level forces as well as the minimum of amplified seismic and maximum seismic force the system can deliver were considered in sizing the straps and fasteners. As described above for chord studs, the nominal shearwall strength for the upper level shearwalls was increased to the upper bound nominal strength, based on the chord stud thickness and yield point, when evaluating the maximum force the system can deliver.

First floor hold-downs were designed for the same load as 2nd floor ties. However, since a proprietary hold-down was selected, data provided by the manufacturer was used for hold-down strength and fastener requirements. Hold-down analysis can be found in Appendix 3, sheet 5 and Appendix 4 sheet SW-3. Tie and hold-down drawings can be found in Appendix 5, sheet SF4.30.

<u>Shear Anchors</u>

Transfer of 2nd floor shear forces to 1st floor shearwalls is accomplished via screw fasteners between the 2nd floor base track and the 1st floor top track. These fasteners pass through the 2nd floor diaphragm. As such, fasteners with spacing to match the edge fasteners for 2nd floor shearwalls were selected.

For the 1st floor shear anchors, the initial design was based on the notion of a concrete foundation and anchors were sized for the maximum in-plane shear force within a shearwall. However, it is likely that this design will be revised in favor of a steel foundation and alternate anchors will be required. Shear anchor analysis can be found in Appendix 4, sheet SW-4. Shear anchors are shown in Appendix 5, sheet SF4.30.

Diaphragms

Per ASCE 7-10, section 12.3.1.1, diaphragms were idealized as flexible.

Roof and floor diaphragms were designed for the higher of the maximum total roof shear and the minimum diaphragm shear required by ASCE 7-05, Eq. 12.10-2. Diaphragm capacity was determined per AISI S213-07, Table D2-1.

On this basis, an unblocked minimum 7/16 inch OSB diaphragm with fasteners at 6 inches on center at supported edges and 12 inches on center in the field was selected for the roof. For the 2nd floor diaphragm, minimum 23/32 inch unblocked structural panels with fastening to match the roof were selected. Note that the 2nd floor diaphragm has holes at the clerestory and exit stairs. The reduced diaphragm length in these areas was accounted for. Roof diaphragm analysis and design can be found in Appendix 3, sheet 6 and Appendix 4, page D-1.

Diaphragm perimeter members were sized for the maximum value of drag force supplied to the shearwalls and diaphragm chord forces based on a beam analogy with the chords acting as the tension and compression elements similar to beam flanges. Diaphragm collectors and chords in structures braced by light framed shearwalls are excluded from overstrength requirements per ASCE 7-05, 12.10.2.1 exception 2. As such, only the LRFD level forces were considered in the design of the collectors, chords and their connections. For both the roof and 2nd floor diaphragms, the typical perimeter members used for gravity support were found to be adequate as collectors and chords. Floor diaphragm analysis and design can be found in Appendix 3, sheet 7 and Appendix 4, page D-2.

Chord and collector splices will be required at the long sides of the buildings due to the length of the walls versus standard lengths of track sections. Based on the calculated chord and collector forces, splices were designed. These calculations can be found in Appendix 4, pages D1 and D2.

The north exit stair creates an opening in the 2nd floor diaphragm. To ensure that diaphragm forces generated in the sub-diaphragm south of the stair opening are transferred to the main 2nd floor diaphragm, tension straps and solid blocking were added. Calculations for the blocking and attachments can be found in Appendix 4, page D3. Drawings for this reinforcing can be seen in Appendix 5, sheet SF4.01.

The 2nd floor clerestory opening also requires reinforcing. Reinforcing design was based, conservatively, on a cantilever beam model for piers on each side of the opening. The moment couple created by the shear at the end of the cantilevered element is transferred into the 2nd floor diaphragm via strap and blocking. Design of the strap and blocking can be found in Appendix 4, page D4. This reinforcing can be seen Appendix 5, sheet SF4.01

Summary

Based on IBC and AISI requirements, calculations and drawings for the CFS-NEES building were produced. The calculations and drawings are included as appendices herewith. As with any structural design, certain engineering judgments were required. Where such judgments were required, they were made based on basic principles of mechanics and standards common to the design of cold-formed steel structures.

References

IBC 2009: “International Building Code”, 2009 edition. International Code Council

ASCE 7-05: ASCE Standard [ASCE/SEI 7-05] “Minimum Design Loads for Buildings and Other Structures.” 2005 edition. American Society of Civil Engineers

AISI S213-07: AISI Standard “North American Standard for Cold-Formed Steel Farming – Lateral Design”, 2007 edition. American Iron and Steel Institute.

AISI S100-07: AISI Standard “North American Specification for the Design of Cold-Formed Steel Structural Members” [NASPEC], 2007 edition. American Iron and Steel Institute.

APA E830D “Technical Note: Fastener Loads for Plywood – Screws”, August 2005. American Plywood Association.

Appendix 1

CFS-NEES
Framing Member Design

April 11, 2012

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CFS-NEES

DEAD LOADS

METAL PANEL WALLS		FLOOR SYSTEM	
Framing	1.0	Framing	2.5
gyp brd 2 sides	5.0	Underlayment	3.0
Metal Panels	1.0	Floor Covering	2.0
Insulation	1.5	Gyp Ceiling	3.0
Misc	1.5	3/4 inch Plywood	2.5
	=====	MEP	3.0
	10.0	Misc.	2.0
			=====
			18.0

MEMBRANE ROOF	
EPDM Membrane	6.0
Insulation	2.0
Sheathing	2.5
Framing	2.5
Ceiling	3.0
MEP	2.0
Misc	2.0
	=====
	20.0

WIND LOAD - IBC 2009 ,

85 mph, Exposure B, I=1.0, Mean Roof Height = 17.0 ft
K_{zt} at Base = 1
K_d = 0.85 , Roof Slope 0.0 degrees
Enclosed Building, GC_{pi} = 0.18

WALL COMPONENTS AND CLADDING per ASCE7-05 Figure 6-11A

GCp by Zone

	Zone 4 (+/-)	Zone 5 (+/-)
10 ft^2	0.90/-0.99	0.90/-1.26
500 ft^2	0.63/-0.72	0.63/-0.72

Wind Pressures (psf) by Zone ()

Height				Windward (4,5)		Leeward (4)		Leeward (5)	
z (ft)	K_z	K_{zt}	q_z (psf)	A=10	A=500	A=10	A=500	A=10	A=500
0 - 17	0.70	1.00	11.01	11.9	10.0	-12.9	-10.0	-15.9	-10.0

PARAPETS

GCp by Case and Zone

	Case A (Zone 4/-2) Front/-Back	Case A (Zone 4 or 5/-3) Front/-Back	Case B (Zone -4/4 or 5) -Front/Back	Case B (Zone -5/4 or 5) -Front/Back
10 ft^2	0.90/-1.80	0.90/-2.80	-0.99/0.90	-1.26/0.90
500 ft^2	0.63/-1.10	0.63/-1.10	-0.72/0.63	-0.72/0.63

Wind Pressures (psf) by Case and Zone ()

Top of Parapet				Case A (4/-2)		Case A (4 or 5/-3)		Case B (-4/4 or 5)		Case B (-5/4 or 5)	
(ft)	K_z	K_{zt-p}	q_{h-p}	A=10	A=500	A=10	A=500	A=10	A=500	A=10	A=500
20	0.70	1.00	11.01	29.7	19.1	40.8	19.1	-20.8	-14.9	-23.8	-14.9

Note that the Pressures are Calculated for 10 and 500 Square Feet Areas. The GCp Values Do Not Always Vary Linearly between these Areas in Figures 6-11A through 6-17. Therefore, Interpolation of These Calculated Values is Not Recommended.

ROOF COMPONENTS AND CLADDING - MONOSLOPE ROOF per ASCE7-05 Figures 6-14A and 6-11B
K_h = 0.70; K_{zt} at roof = 1.00; q_h = 11.01 psf

	Positive Pressure, p (psf)				Negative Pressure, p (psf)			
	A=10		A=100		A=10		A=100	
Zone	GC_p	p	GC_p	p	GC_p	p	GC_p	p
1	0.3	10.0	0.2	10.0	-1.0	-13.0	-0.9	-11.9
2	0.3	10.0	0.2	10.0	-1.8	-21.8	-1.1	-14.1
3	0.3	10.0	0.2	10.0	-2.8	-32.8	-1.1	-14.1

NOTE: Use 15 psf min for components and cladding

SEISMIC LOADS - IBC 2009

Short period spectral acceleration - S_s = 1.39 , 1-Second spectral acceleration - S_1 = 0.50
Building Height - H_r = 17 ft, Site Class = D
Occupancy Category = II , Seismic Design Category = D
F_a = 1.00 , F_v = 1.50 , $S_{MS} = F_a S_s$ = 1.39 , $S_{M1} = F_v S_1$ = 0.75
$\mathbf{S_{DS}}$ = 2/3 S_{MS} **= 0.93** , SD_1 = 2/3 S_{M1} = 0.50

LATERAL FORCE RESISTING SYSTEM - STRENGTH DESIGN LEVEL FORCES

I = 1.0
Bearing Wall Systems
13. Light frame walls sheathed with wood structural panels rated for shear resistance or steel sheets
R = 6.5, Ω_0 = 3.0, C_d = 4.0, C_t = 0.02
Period Exponent x = 0.75, ASCE 7-05 Eq 12.8-7 T = $C_t H_r^x$ = 0.167 Seconds, TL = 16.

ASCE Section 12.8 Equivalent Lateral Force Procedure

ASCE 7-05 Eq 12.8-2 $C_S = S_{DS}$ I/R	= 0.143
ASCE 7-05 Eq 12.8-3 $C_S = S_{D1}$ /(T*R/I)	= 0.459
ASCE 7-05 Eq 12.8-5 C_S	= 0.010

C_S = 0.143

ELEMENTS AND COMPONENTS - ASCE 7-05 Eq 13.3-1 thru 13.3-3
(Results Shown are for Alternate Basic Load Combinations Using ASD Design and are Referenced Equations / 1.4)

Element Type	1	2	3
$a_p/R_p/I_p$	1/2.5/1	1/2.5/1	1.25/1/1
Seismic Coefficients - See Below for Element Types			
z = 0	0.20	0.20	0.33
z = 9	0.21	0.21	0.66
z = 17	0.32	0.32	0.99

Element Type	**Description**
1	Architectural Component or Element - Exterior Nonstructural Wall Elements and Connections - Wall elements
2	Architectural Component or Element - Exterior Nonstructural Wall Elements and Connections - Body of wall panel connections
3	Architectural Component or Element - Exterior Nonstructural Wall Elements and Connections - Fasteners of the connecting system

FLOOR LIVE LOADS

Offices 50 psf
Office corridors above 1st floor 80 psf
Partitions 15 psf

ROOF LOADS:

Roof Live 20 psf

DEFLECTION LIMITS:

Floor L/360 LL; L/240 DL + LL
Roof L/240 LL; L/180 DL + LL

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PROJECT: CFS-NEES | PROJECT NO: 10-277 | DESIGN: RLM | DATE: Dec '10

Roof Joists

Span = 22' Clear –
DL = 20 psf
LL = 20 psf

for WL · $A = \frac{22^2}{3} = 161\ ft^2$

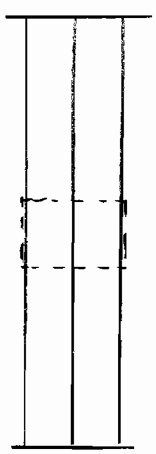
∴ WL ≤ 14.1 psf uplift

(1) ∴ Use 1200S200-54 Joists @ 24" oc
Bridg'g @ 1/3 Pts (Stiffeners Req'd)

Support @ Roof Top Mech Units

Use W = 600 lb ea of (2) Units
est 3'x4' in plan
Duct penetration < 22"

P ≤ 150 lb @ (2) loc'ns any Joist

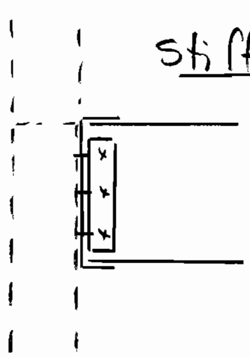


(2) ∴ Use (2) 1200S200-54 B/B Typ
Below Units

∴ Web Stiffeners Req'd @ Bearing Pts

Stiffeners

Py = 1030 lb Max · # scr = 1030/400 = 2.6



∴ Use L1½x1½x54-mil x 0'10"
w/ (3) #10 ea Leg

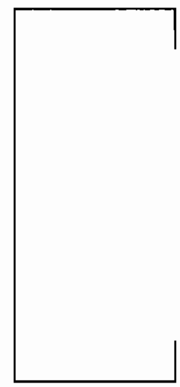
1

2007 North American Specification ASD
DATE: 12/21/2010
CFS-NEES

SECTION DESIGNATION: 1200S200-54 [50] Single

<u>Section Dimensions:</u>

Web Height =	12.000 in
Top Flange =	2.000 in
Bottom Flange =	2.000 in
Stiffening Lip =	0.625 in
Inside Corner Radius =	0.0849 in
Punchout Width =	1.500 in
Punchout Length =	4.000 in
Design Thickness =	0.0566 in



<u>Steel Properties:</u>

Fy =	50.000 ksi
Fu =	65.000 ksi
Fya =	50.000 ksi

ALLOWABLE RAFTER SPANS

<u>INPUT PARAMETERS</u>

Roof Slope 0:12
Bridging Interval for Uplift: THIRD Pt

<u>Inward Loads</u>

Dead Load = 20.0 psf	DL Multiplied by 1.00 for Strength Checks
Live Load = 20.0 psf	LL Multiplied by 1.00 for Strength Checks

<u>Outward Loads (Uplift)</u>

Resisting DL = 12.0 psf	DL Multiplied by 1.00 for Strength Checks
Wind Load = 14.1 psf	WL Multiplied by 1.00 for Strength Checks

Dead Load Deflection Limit = L/240
Total Load Deflection Limit = L/180
Wind Load not modified for deflection calculations
Web Stiffeners Required at Supports
Shear Capacity Based on Unpunched Web
K-phi for Distortional Buckling = 0.00 lb*in/in
Include Torsion? Yes
Torsional Lever Arm to: Web Center

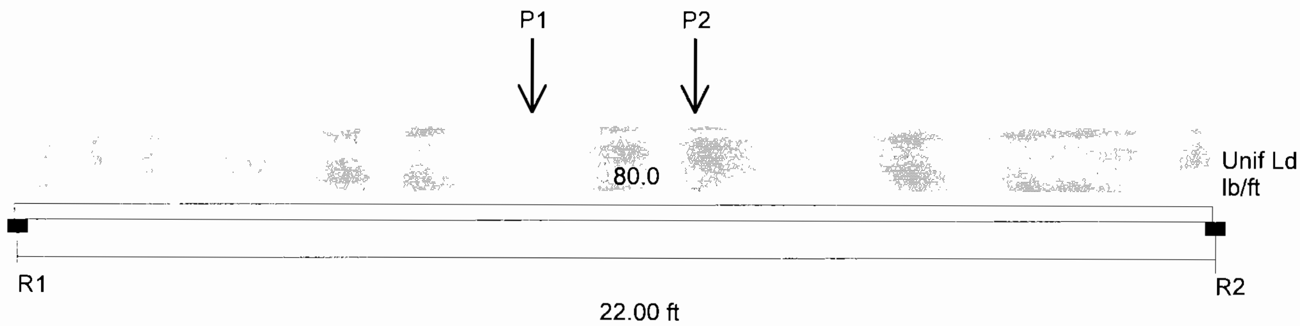
<u>ALLOWABLE RAFTER SPANS - Horizontal Projection</u>

RAFTER SPACING	Inward Loads	Outward Loads
12.0 in	30' 2"	30' 2"
16.0 in	27' 10"	30' 2"
24.0 in	22' 8"	30' 2"

R-1

2007 NASPEC [AISI S100]

Project:	CFS-NEES	**Date:**	8/5/2011
Model:	Roof Joists with Mech Unit		



Point Loads	**P1**	**P2**
Load(lb)	150	150
X-Dist.(ft)	9.50	12.50

Section : (2) 1200S200-54 Back-to-Back C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 10344.2 Ft-Lb **Moment of Inertia, I =** 32.668 in^4 **Va =** 2754.7 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	6265.0	0.606	6265.0	Full	10344.2	0.606	0.554	L/477

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	264.0	9122.8	0.687

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	1030.0	1.00	NA	0.0	NA	**YES**
R2	1030.0	1.00	NA	0.1	NA	**YES**
P1	150.0	1.50	NA	6170.1	NA	**YES**
P2	150.0	1.50	NA	6175.5	NA	**YES**

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	1030.0	0.0	1.00	0.37	0.00	0.14	NA
R2	1030.0	0.1	1.00	0.37	0.00	0.14	NA
P1	273.2	6170.1	1.00	0.10	0.60	0.37	NA
P2	271.4	6175.5	1.00	0.10	0.60	0.37	NA

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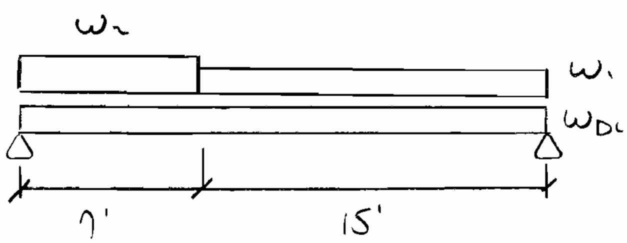
PROJECT: CFS-NEES | **PROJECT NO:** 10-277 | **DESIGN:** RLM | **DATE:** Jan '11

Floor Joists

Span = 22' C/c (max)

DL = 18 psf
Partitions = 15 psf
LL = 50 psf Typ
= 80 psf Corridors (Plan North 7' +/-)

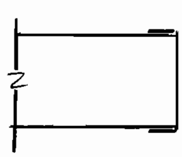


$w_{DL} = 18$ psf
$w_1 = 50 + 15 = 65$ psf
$w_2 = 80$ psf

(1) (2) ∴ Use 1200S250-97 @ 24" oc. Strap + Blk @ 1/3 pts. Stiffeners Req'd

Headers @ Stair Clerestory

Span ≤ 8.5'
$w \le (18+80)\left(\frac{7}{2}\right) = 343$ lb/ft



∴ Use 1200T200-68 Cont

Conn @ Carriers

$R_y = 1458$ lb ∴ Use $V_A = 400$ lb/scr

scr = $1458/400 = 3.6$

∴ Use L2x2x54-mil ∠ x 0'11" min w/ (4) #10 ea leg

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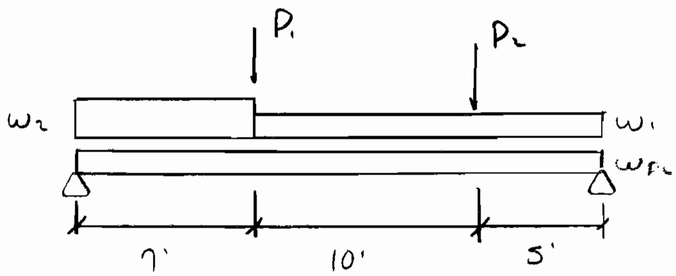
PROJECT: CFS-NEES	PROJECT NO: 10-277	DESIGN: RWM	DATE: Jan '11

Corriess @ Stair Clerestory



$w_{DL} = 18\left(\frac{12}{12}\right) = 18 \text{ lb/ft}$

$w_1 = 65\left(\frac{12}{12}\right) = 65$ "

$w_2 = 80\left(\frac{12}{12}\right) = 80$ "

$P_1 = 1458 \text{ lb} \quad (1190 \text{ lb LL})$

$P_2 = (18+65)\left(\frac{5}{2}\right)\left(\frac{8.5}{2}\right) = 882 \text{ lb} \quad (691 \text{ lb LL})$

4 5 6

∴ Use 1200S250-97 Carriers - ALT
(2) 1200S250-97 Boxed - Stiffeners Req'd

Web Stiffeners

V = 2196 lb max

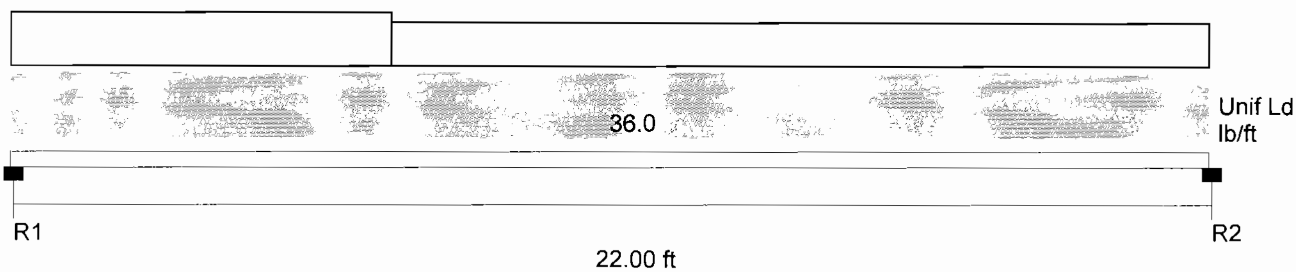
$\# \text{ scr} = \frac{2196}{400} = 5.5$

∴ Use L1½x1½x54-mil L x 0'10" w/
(6) #10 ea Leg

1

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: Typ Floor Joists - DL + LL 24 in oc



Sloped/Partial Loads	**Case**	**X1 ft**	**W(X1) lb/ft**	**X2 ft**	**W(X2) lb/ft**
	1	0.00	160.0	7.00	160.0
	2	7.00	130.0	22.00	130.0

Section : 1200S250-97 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 12568.3 Ft-Lb **Moment of Inertia, I =** 33.835 in^4 **Va =** 8147.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	10414.3	0.829	10414.3	Full	12568.3	0.829	0.913	L/289

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	95.00	264.0	11456.3	0.909

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	**2002.6**	1.00	1617.5	2830.6	0.0	0.64	**YES**
R2	**1859.4**	1.00	1617.5	2830.6	1.8	0.60	**YES**

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	2002.6	0.0	1.00	0.25	0.00	0.06	NA
R2	1859.3	1.8	1.00	0.23	0.00	0.05	NA

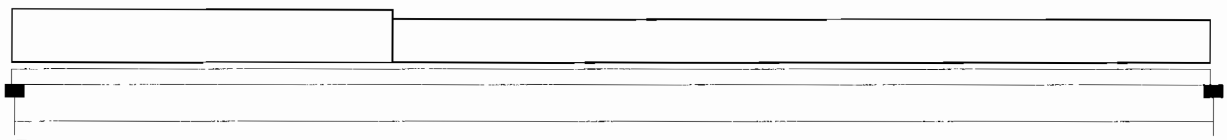
Within Span (Unstiffened)

	Unpunched				Punched			
Span	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.
Center Span	10.80	10414.3	-0.4	0.69	10.80	10414.3	-0.4	0.69

2

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: Typ Floor Joists - LL only 24 in oc



R1 R2
22.00 ft

Sloped/Partial Loads	**Case**	**X1 ft**	**W(X1) lb/ft**	**X2 ft**	**W(X2) lb/ft**
	1	0.00	160.0	7.00	160.0
	2	7.00	130.0	22.00	130.0

Section : 1200S250-97 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 12568.3 Ft-Lb **Moment of Inertia, I =** 33.835 in^4 **Va =** 8147.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	8237.2	0.655	8237.2	Full	12568.3	0.655	0.722	L/365

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	95.00	264.0	11456.3	0.719

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	1606.6	1.00	1617.5	2830.6	0.0	0.52	No
R2	1463.4	1.00	1617.5	2830.6	1.7	0.47	No

Combined Bending and Shear

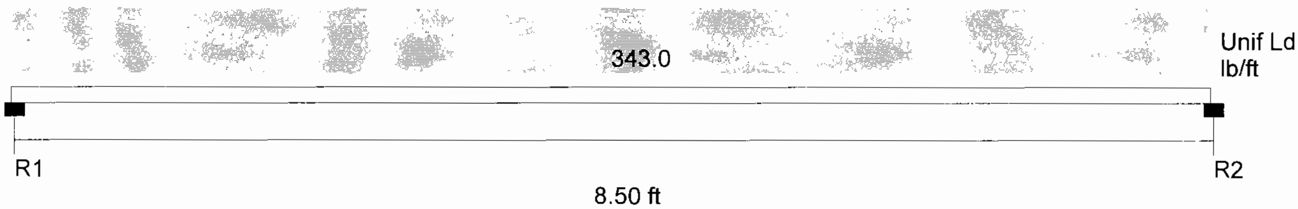
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	1606.6	0.0	1.00	0.20	0.00	0.04	NA
R2	1463.3	1.7	1.00	0.18	0.00	0.03	NA

Within Span (Unstiffened)

	Unpunched				Punched			
Span	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.
Center Span	10.74	8237.2	1.0	0.43	10.74	8237.2	1.0	0.43

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: Floor Header at Stair Clerestory



Section : 1200T200-68 Single Track (X-X Axis) **Fy =** 50.0 ksi
Maxo = 5135.2 Ft-Lb **Moment of Inertia, I =** 18.026 in^4 **Va =** 2712.6 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Ratio
Center Span	3097.7	0.603	3097.7	Full	5135.2	0.603	0.076	L/1346

Combined Bending and Web Crippling

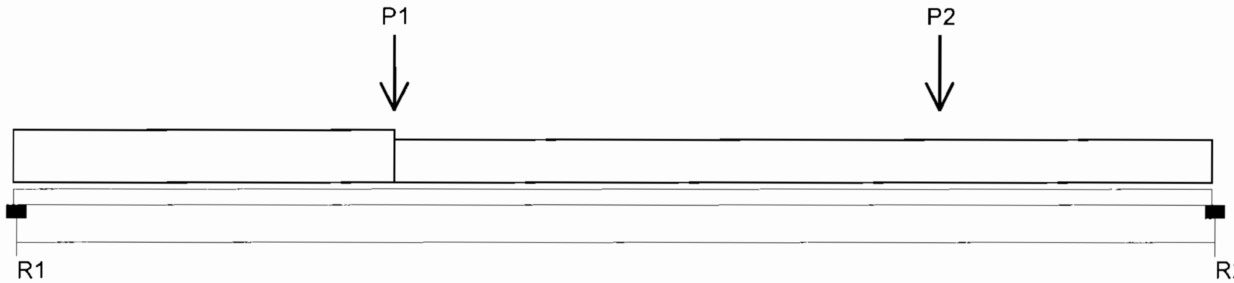
Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	**1457.8**	1.00	573.1	1031.5	0.0	**1.29**	**YES**
R2	**1457.8**	1.00	573.1	1031.5	0.1	**1.29**	**YES**

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	1457.8	0.0	1.00	0.54	0.00	0.29	NA
R2	1457.8	0.1	1.00	0.54	0.00	0.29	NA

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: Floor Carriers at Stair Clerestory - LL Only



Point Loads	**P1**	**P2**
Load(lb)	1190	691
X-Dist.(ft)	7.00	17.00

Sloped/Partial Loads

Case	X1 ft	W(X1) lb/ft	X2 ft	W(X2) lb/ft
1	0.00	80.0	7.00	80.0
2	7.00	65.0	22.00	65.0

Section : 1200S350-97 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 16442.5 Ft-Lb **Moment of Inertia, I =** 43.269 in^4 **Va =** 8147.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	10445.6	0.635	10445.6	Full	16442.5	0.635	0.710	L/372

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	264.0	14236.9	0.734

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	**1771.7**	1.00	1617.5	2830.6	0.0	0.57	**YES**
R2	**1644.3**	1.00	1617.5	2830.6	0.8	0.53	**YES**
P1	1190.0	1.50	4022.0	6636.2	10437.1	0.54	No
P2	691.0	1.50	4022.0	6636.2	7419.6	0.36	No

Combined Bending and Shear

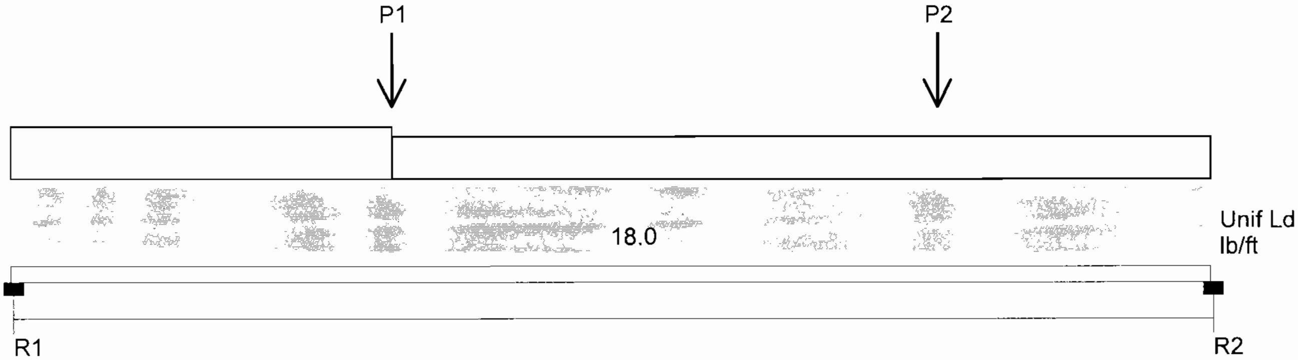
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	1771.7	0.0	1.00	0.22	0.00	0.05	NA
R2	1644.2	0.8	1.00	0.20	0.00	0.04	NA
P1	1213.8	10437.1	1.00	0.15	0.63	0.43	NA
P2	1319.6	7419.6	1.00	0.16	0.45	0.23	NA

Within Span (Unstiffened)

	Unpunched				Punched			
Span	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.
Center Span	7.00	10410.4	1213.8	0.43	7.00	10410.4	1213.8	0.43

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: Floor Carriers at Stair Clerestory - DL + LL



Point Loads	**P1**	**P2**
Load(lb)	1458	882
X-Dist.(ft)	7.00	17.00

Sloped/Partial Loads

Case	X1 ft	W(X1) lb/ft	X2 ft	W(X2) lb/ft
1	0.00	80.0	7.00	80.0
2	7.00	65.0	22.00	65.0

Section : 1200S350-97 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 16442.5 Ft-Lb **Moment of Inertia, I =** 43.269 in^4 **Va =** 8147.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	12986.1	0.790	12986.1	Full	16442.5	0.790	0.887	L/298

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	264.0	14236.9	0.912

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	**2195.8**	1.00	1617.5	2830.6	0.0	0.71	**YES**
R2	**2075.2**	1.00	1617.5	2830.6	0.8	0.67	**YES**
P1	1458.0	1.50	4022.0	6636.2	12963.8	0.67	No
P2	882.0	1.50	4022.0	6636.2	9351.3	0.46	No

Combined Bending and Shear

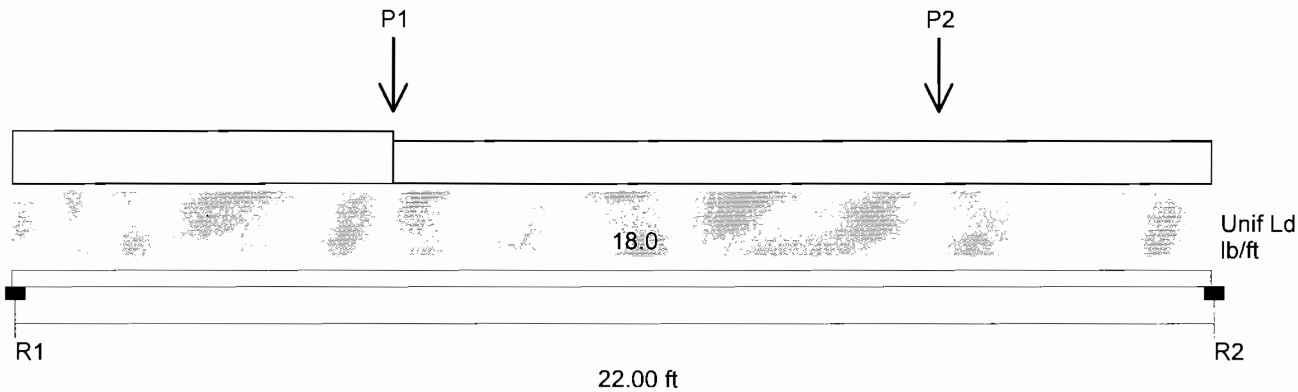
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	2195.8	0.0	1.00	0.27	0.00	0.07	NA
R2	2075.1	0.8	1.00	0.25	0.00	0.06	NA
P1	1512.4	12963.8	1.00	0.19	0.79	0.66	NA
P2	1660.6	9351.3	1.00	0.20	0.57	0.37	NA

Within Span (Unstiffened)

Span	Unpunched Loc'n, X (ft)	Unpunched M(X) (Ft-Lb)	Unpunched V(X) (lb)	Unpunched Intr.	Punched Loc'n, X (ft)	Punched M(X) (Ft-Lb)	Punched V(X) (lb)	Punched Intr.
Center Span	7.00	12930.6	1512.4	0.66	7.00	12930.6	1512.4	0.66

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: Floor Carriers at Stair Clerestory - DL + LL



Point Loads	**P1**	**P2**
Load(lb)	1458	882
X-Dist.(ft)	7.00	17.00

Sloped/Partial Loads

Case	X1 ft	W(X1) lb/ft	X2 ft	W(X2) lb/ft
1	0.00	80.0	7.00	80.0
2	7.00	65.0	22.00	65.0

Section : (2) 1200S250-97 Boxed C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 25136.6 Ft-Lb **Moment of Inertia, I =** 67.669 in^4 **Va =** 16294.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	12986.1	0.517	12986.1	Full	25136.6	0.517	0.567	L/466

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	2195.8	1.00	3235.0	5661.3	0.0	0.35	No
R2	2075.2	1.00	3235.0	5661.3	0.8	0.33	No
P1	1458.0	1.50	8043.9	13272.5	12963.8	0.41	No
P2	882.0	1.50	8043.9	13272.5	9351.3	0.28	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	2195.8	0.0	1.00	0.13	0.00	0.02	NA
R2	2075.1	0.8	1.00	0.13	0.00	0.02	NA
P1	1512.4	12963.8	1.00	0.09	0.52	0.27	NA
P2	1660.6	9351.3	1.00	0.10	0.37	0.15	NA

Within Span (Unstiffened)

	Unpunched				Punched			
Span	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.	Loc'n, X (ft)	M(X) (Ft-Lb)	V(X) (lb)	Intr.
Center Span	7.00	12930.6	1512.4	0.27	7.00	12930.6	1512.4	0.28

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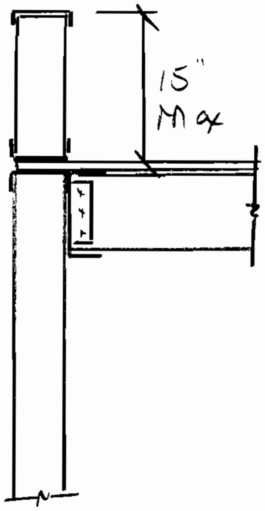
PROJECT: CFS-NEES | **PROJECT NO:** 10.277 | **DESIGN:** RLM | **DATE:** Feb'11

Typ Wall Studs

Upper Level



$H = 9'$ (T.O Joist)
$WL = 15$ psf

$C_{DL} = C_{LL} = 20\left(\frac{22}{2}\right) = 220$ lb/ft
$= 440$ lb/Joist @ 24" oc

$e_x = 3"$

ASCE 7-05 Load Combinations:
D + L
D + W
D + .75L + .75W

Use $k_\phi = 0$ for Distortional Buckling

(1) ∴ Use min 600S162-33 @ 24" oc - sheathing braced or CRC Mid-Ht or 48" oc

Typ Stud/Rafter Conn

$V = 440(1+.75) = 770$ lb , $T = .75(15)+40.8(2)(1.25) = 196$ lb

Note: Joists Do Not necessarily align w/studs

$\# \, SCS = \frac{770}{177} + \frac{196}{84} = 6.7$

∴ Use 1200T200-68 - #10 ea Leg & (7) #10 ea stud

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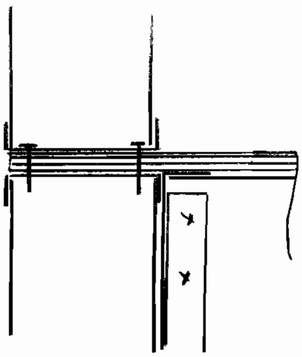
PROJECT: CFS-NEES	PROJECT NO: 10.277	DESIGN: TLM	DATE: Feb '11

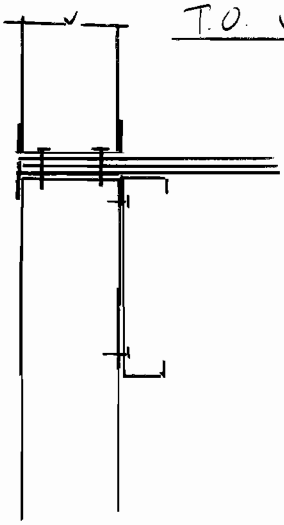
Parapet

WL ≤ 40.8 psf



2 ∴ Use 600S162-33 @ 24" oc. Min
600T150-43 T + B w/ #10 ea leg
+ (2) #10 @ 5" oc ea stud

T.O. Wall Conn @ Joist Parallel



$R_x = 159 + 40.8(2)(1.25) = 261$ lb/stud

Per APA E830: in 78 mil steel
+ 1/2" ply $V_u = 470$ lb/scr

∴ Use $V_A = \frac{470}{3}\left(\frac{.045}{.078}\right) = 91$ lb/scr

$\#scr = \frac{261}{91} = 2.9$ /stud*

∴ Use (2) #10 @ 24" oc. 5" Gage
Min 43 mil Track.
(2) #10 @ Joist ea stud

* Consider (2) @ ply + (1) @ Joist

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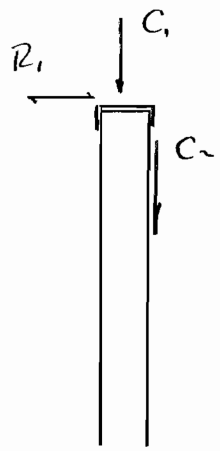
PROJECT: CFS-NEES | **PROJECT NO:** 10-277 | **DESIGN:** RLM | **DATE:** Feb '11

Lower level



$C_1 = \underbrace{380}_{\text{Roof}} + \underbrace{10.25(20)}_{\text{wall}} = 1085$ lb/stud [440 lb LL + 645 lb DL]

$C_2 = \underbrace{18\left(\frac{24}{12}\right)\left(\frac{22}{2}\right)}_{DL} + 1607$ lb $= 2003$ lb/Joist [1607 lb LL + 396 lb DL]

C_2 @ $e_x = 3''$

Case 1 - DL + LL : $C = 3088$ lb

$M_{ecc} = 2003(3) = 6009$ in-lb

$\therefore e_{eff} = \frac{6009}{3088} = 1.95''$

Case 2 - DL + .75LL + .75W (W = 15 psf)

$\therefore \Sigma C = 645 + 396 + .75(440 + 1607)$
$= 2576$ lb

$M_{ecc} = (396 + .75(1607))(3) = 4804$ in-lb

$\therefore e_{eff} = \frac{4804}{2576} = 1.86''$

3 $\therefore$ Use 600S162-54 @ 24" oc ; - Align w/studs above. CRC @ mid-pt
4 or 48" oc (sheathing braced NOT appropriate for axial load in tests)

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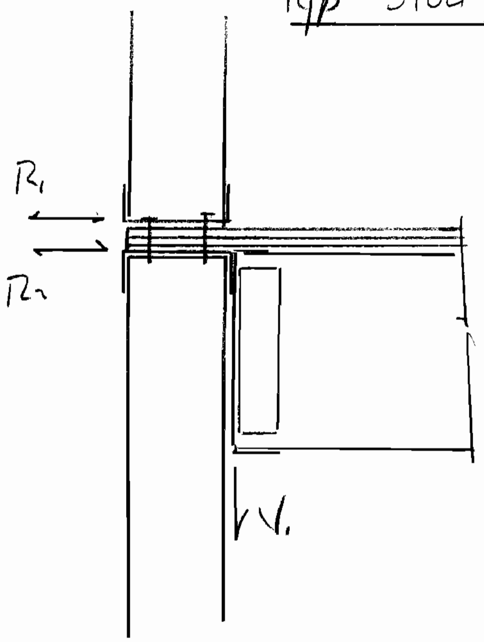
Typ Stud/Joist Conn



$R_1 = 159$ lb
$R_2 = 179/.75 = 239$ lb } unfactored

$V_1 = 2003$ lb @ 24" oc

54 mil studs : #10 screws
$V_A = 400$ lb ; $T_A = 198$ lb

$$\# scs \leq \frac{239}{198} + \frac{396 + 1607(.75)}{400} = 5.2$$

∴ Use Min R00T200-97 Rim Track- (6)#10 ea stud

Posts @ Stair Carrier Joists

$V_1 = 2196$ lb ∴ $\Sigma C = 2196 + 1085 = 3281$ lb

$M_{ecc} = 2196(3) = 6588$ in-lb
$= 549$ Ft-lb

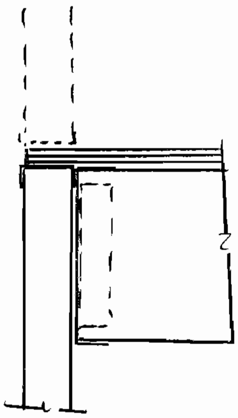
$$\# scs = \frac{3281}{400} = 8.2$$

5 ∴ Use Add'nl 600S162-54 Stud/Post aligned w/ Carriers: (9) #10 Rim Track/stiffener to Stud

Int Brng Wall @ Stair

$C < 2003$ lb ; $e = 1.81"$ (3 5/8 studs)

$W = 5$ psf



6 7 ∴ Use 362S162-54 @ 24" oc. CRC mid Ht or 48" oc. or OK as sheathing braced

Stud/Joist Conn

$$\#scr \le \frac{64}{198} + \frac{2003}{400} : 5.3$$

∴ Use Typ Rim Track + (6) #10 ea stud

Exterior Balloon Wall @ Exit Stair

$H = 18'$ $C = 880$ lb @ $e_x = 3"$

8 ∴ Use 600S162-54 @ 24" oc. CRC 48" oc Max.

Stud/Joist Conn

$M_x = 282 + 82(1.25) = 385$ lb ; $V = 880$ lb

$$\#scr = \frac{385}{198} + \frac{880}{400} = 4.1$$

∴ Use Min (6) #10 ea stud

2007 North American Specification ASD
DATE: 2/15/2011
CFS-NEES

1

SECTION DESIGNATION: 600S162-33 [33] Single

Input Properties:

Web Height =	6.000 in	Design Thickness =	0.0346 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0765 in
Bottom Flange =	1.625 in	Yield Point, Fy =	33.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	33.0 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 9.00 ft — Deflection Limit L/240
Lateral Pressure 15.00 psf — Axial Load 880 lb
Stud Spacing 24.0 in

Check Flexure

Load Multiplier for Flexural Strength = 1.00
Includes Eccentric Axial Load: 880 (lb) with 3 (in) eccentricity
Eccentricity considered One end of stud only
Input Flexural Bracing: Mid-Pt   Cb = 1.00
Fe = 53.3 ksi   Fy = 33.0 ksi   0.56 Fy < Fe < 2.78 Fy
Fc = 30.4 ksi   Sc = 0.585 in^3   Sf = 0.598 in^3   Mn = 1317 Ft-Lb
Mmax = 424 Ft-Lb <= Ma = 789 Ft-Lb (Distortional Buckling Controls)
K-phi for Distortional Buckling = 0 lb*in/in

Check Deflection

Deflection Limit: L/240
Load Multiplier for Deflection = 0.70
Maximum Deflection = 0.095 in   Deflection Ratio = L/1133

Check Shear

Vmax = 159 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 638 lb >= Vmax

Check Web Crippling

Rmax = 159 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.50 in
Ra = 175 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 880 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Max unbraced length, KyLy and KtLt = 54.0 in   Max KL/r = 93
Allowable Pure Axial Load, Pa = 2369 lb : Axial Load Ratio, P/Pa = 0.371
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 44751 lb   Alpha = 0.962
Equation C5.2.1-1 = 0.930

Check Equation C5.2.1-2
Pao = 3244 lb
Equation C5.2.1-2 = 0.808

Maximum Interaction = 0.930 <=1.0

W-5

Cantilevered Sill/Parapet Design

Description: CFS-NEES

Input Data

Design Pressure	40.8 psf	Duration Factor	1 (studs/screws)
Max Stud Spacing	24 in	Duration Factor	1 (anchors)
Wall Height	1.25 ft		
Window Height	0 ft	(Trib at top taken as 1/2 window Ht.)	

Dead Load (assumed centered on stud)

Window	10 psf
Wall	10 psf

Size Stud

Stud Type	600S162-33	Maximum Moment		63.8 Ft-lb/stud
Stud Width (in)	6	Bending Stress, fb		**1.3** (ksi)
S_{xx} (in^3)	0.577	Deflection		0.0008 (in)
I_{xx} (in^4)	1.793		L/	**36876** Ratio

Stud to Track

Gross Tens (lb)	128	Screw Va (lb/scr)	177
Dead Load (lb/leg)	13	No. of Screws Ea Leg	0.6
Net Tens (lb)	115	Va (lb/in) weld	100
		Lreq'd	1.15 in each leg

Track to Structure

Resist Lever Arm at Anchors (in)	5	Anchor Va (lb each)	263
Anchor Rows	2	Anchor Ta (lb each)	109
Row Spacing (in)	16	Interaction Exponent	1.00
DL Resistance Lever Arm (in)	3	Interaction Value	**0.97**
DL Resisting Moment (in-lb)	75		
Tension at Anchor (lb/anchor)	92		
Base Shear (lb/anchor)	34		

Track Plate Bending

Lever Arm - Leg to Anchor (in)	0.5	Track Fy (ksi)	33
Eff Width for Plate Bending	12	Thickness Req'd (in)	**0.0341**

2007 North American Specification ASD
DATE: 2/15/2011
CFS-NEES

SECTION DESIGNATION: 600S162-54 [50] Single

Input Properties:

Web Height =	6.000 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	55.3 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 9.00 ft — Deflection Limit L/240
Lateral Pressure 0.10 psf — Axial Load 3088 lb
Stud Spacing 24.0 in

Check Flexure

Load Multiplier for Flexural Strength = 1.00
Includes Eccentric Axial Load: 3088 (lb) with 1.95 (in) eccentricity
Eccentricity considered One end of stud only
Input Flexural Bracing: Mid-Pt — Cb = 1.00
Fe = 52.7 ksi — Fy = 50.0 ksi — 0.56 Fy < Fe < 2.78 Fy
Fc = 40.9 ksi — Sc = 0.953 in^3 — Sf = 0.953 in^3 — Mn = 3251 Ft-Lb
Mmax = 502 Ft-Lb <= Ma = 1947 Ft-Lb

Check Deflection

Deflection Limit: L/240
Load Multiplier for Deflection = 0.70
Maximum Deflection = 0.054 in — Deflection Ratio = L/2014

Check Shear

Vmax = 57 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2823 lb >= Vmax

Check Web Crippling

Rmax = 57 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.50 in
Ra = 679 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 3088 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Max unbraced length, KyLy and KtLt = 54.0 in — Max KL/r = 95
Allowable Pure Axial Load, Pa = 5098 lb : Axial Load Ratio, P/Pa = 0.606
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 71400 lb — Alpha = 0.917
Equation C5.2.1-1 = 0.887

Check Equation C5.2.1-2
Pao = 8521 lb
Equation C5.2.1-2 = 0.620

Maximum Interaction = 0.887 <=1.0

2007 North American Specification ASD
DATE: 2/15/2011
CFS-NEES

4

SECTION DESIGNATION: 600S162-54 [50] Single

Input Properties:

Web Height =	6.000 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	55.3 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 9.00 ft — Deflection Limit L/240
Lateral Pressure 15.00 psf — Axial Load 2576 lb
Stud Spacing 24.0 in

Check Flexure

Load Multiplier for Flexural Strength = 0.75
Includes Eccentric Axial Load: 2576 (lb) with 1.86 (in) eccentricity
Eccentricity considered One end of stud only
Input Flexural Bracing: Mid-Pt   Cb = 1.00
Fe = 52.7 ksi   Fy = 50.0 ksi   0.56 Fy < Fe < 2.78 Fy
Fc = 40.9 ksi   Sc = 0.953 in^3   Sf = 0.953 in^3   Mn = 3251 Ft-Lb
Mmax = 536 Ft-Lb <= Ma = 1947 Ft-Lb

Check Deflection

Deflection Limit: L/240
Load Multiplier for Deflection = 0.70
Maximum Deflection = 0.079 in   Deflection Ratio = L/1373

Check Shear

Vmax = 179 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2823 lb >= Vmax

Check Web Crippling

Rmax = 179 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.50 in
Ra = 679 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 2576 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Max unbraced length, KyLy and KtLt = 54.0 in   Max KL/r = 95
Allowable Pure Axial Load, Pa = 5098 lb : Axial Load Ratio, P/Pa = 0.505
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 71400 lb   Alpha = 0.931
Equation C5.2.1-1 = 0.727

Check Equation C5.2.1-2
Pao = 8521 lb
Equation C5.2.1-2 = 0.509

Maximum Interaction = 0.727 <=1.0

2007 North American Specification ASD
DATE: 2/15/2011
CFS-NEES

S

SECTION DESIGNATION: 600S162-54 [50] Single

Input Properties:

Web Height =	6.000 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	55.3 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 9.00 ft
Deflection Limit L/240
Lateral Pressure 0.10 psf
Axial Load 3281 lb
Stud Spacing 24.0 in

Check Flexure

Load Multiplier for Flexural Strength = 1.00
Includes Eccentric Axial Load: 3281 (lb) with 2 (in) eccentricity
Eccentricity considered One end of stud only
Flexural Bracing: Full
Mmax = 547 Ft-Lb <= Ma = 2527 Ft-Lb & Ma(distortional) = 2158 Ft-Lb
K-phi for Distortional Buckling = 0 lb*in/in

Check Deflection

Deflection Limit: L/240
Load Multiplier for Deflection = 1.00
Maximum Deflection = 0.059 in
Deflection Ratio = L/1845

Check Shear

Vmax = 62 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2823 lb >= Vmax

Check Web Crippling

Rmax = 62 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.50 in
Ra = 679 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 3281 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Max unbraced length, KyLy and KtLt = 48.0 in
Max KL/r = 84
Allowable Pure Axial Load, Pa = 5727 lb : Axial Load Ratio, P/Pa = 0.573
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 71400 lb
Alpha = 0.912
Equation C5.2.1-1 = 0.851

Check Equation C5.2.1-2
Pao = 8521 lb
Equation C5.2.1-2 = 0.638

Maximum Interaction = 0.851 <=1.0

2007 North American Specification ASD
DATE: 2/15/2011
CFS-NEES

SECTION DESIGNATION: 362S162-54 [50] Single

Input Properties:

Web Height =	3.625 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	50.0 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 9.00 ft — Deflection Limit L/120
Lateral Pressure 5.00 psf — Axial Load 2003 lb
Stud Spacing 16.0 in

Check Flexure

Load Multiplier for Flexural Strength = 1.00
Includes Eccentric Axial Load: 2003 (lb) with 1.81 (in) eccentricity
Eccentricity considered One end of stud only
Input Flexural Bracing: Mid-Pt   Cb = 1.00
Fe = 58.0 ksi   Fy = 50.0 ksi   0.56 Fy < Fe < 2.78 Fy
Fc = 42.3 ksi   Sc = 0.462 in^3   Sf = 0.481 in^3   Mn = 1628 Ft-Lb
Mmax = 302 Ft-Lb <= Ma = 975 Ft-Lb

Check Deflection

Deflection Limit: L/120
Load Multiplier for Deflection = 1.00
Maximum Deflection = 0.143 in   Deflection Ratio = L/756

Check Shear

Vmax = 64 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 3372 lb >= Vmax

Check Web Crippling

Rmax = 64 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 634 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 2003 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Max unbraced length, KyLy and KtLt = 54.0 in   Max KL/r = 89
Allowable Pure Axial Load, Pa = 3689 lb : Axial Load Ratio, P/Pa = 0.543
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 21785 lb   Alpha = 0.823
Equation C5.2.1-1 = 0.919

Check Equation C5.2.1-2
Pao = 8210 lb
Equation C5.2.1-2 = 0.554

Maximum Interaction = 0.919 <=1.0

2007 North American Specification ASD
DATE: 2/15/2011
CFS-NEES

SECTION DESIGNATION: 362S162-54 [50] Single

Input Properties:

Web Height =	3.625 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	50.0 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 9.00 ft   Deflection Limit L/120
Lateral Pressure 5.00 psf   Axial Load 2003 lb
Stud Spacing 16.0 in

Check Flexure

Load Multiplier for Flexural Strength = 1.00
Includes Eccentric Axial Load: 2003 (lb) with 1.81 (in) eccentricity
Eccentricity considered One end of stud only
Input Flexural Bracing: Mid-Pt   Cb = 1.00
Fe = 58.0 ksi   Fy = 50.0 ksi   0.56 Fy < Fe < 2.78 Fy
Fc = 42.3 ksi   Sc = 0.462 in^3   Sf = 0.481 in^3   Mn = 1628 Ft-Lb
Mmax = 302 Ft-Lb <= Ma = 975 Ft-Lb

Check Deflection

Deflection Limit: L/120
Load Multiplier for Deflection = 1.00
Maximum Deflection = 0.143 in   Deflection Ratio = L/756

Check Shear

Vmax = 64 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 3372 lb >= Vmax

Check Web Crippling

Rmax = 64 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 634 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 2003 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Axial Bracing = Sheathed per 2007 Wall Stud Std.
1/2 in. shth'g with #6 screws 12 in. oc.   Max KL/r = 75
Allowable Pure Axial Load, Pa = 3222 lb : Axial Load Ratio, P/Pa = 0.622
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 21785 lb   Alpha = 0.823
Equation C5.2.1-1 = 0.998

Check Equation C5.2.1-2
Pao = 8210 lb
Equation C5.2.1-2 = 0.554

Maximum Interaction = 0.998 <=1.0

SECTION DESIGNATION: 600S162-54 [50] Single

Input Properties:

Web Height =	6.000 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	55.3 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Wall Solver Design Data - Simple Span

Wall Height 18.00 ft — Deflection Limit L/240
Lateral Pressure 15.00 psf — Axial Load 880 lb
Stud Spacing 24.0 in

Check Flexure

Load Multiplier for Flexural Strength = 1.00
Includes Eccentric Axial Load: 880 (lb) with 3 (in) eccentricity
Eccentricity considered One end of stud only
Flexural Bracing: KyLy = 48.0 in   Cb = 1.00
Fe = 66.4 ksi   Fy = 50.0 ksi   0.56 Fy < Fe < 2.78 Fy
Fc = 43.9 ksi   Sc = 0.947 in^3   Sf = 0.953 in^3   Mn = 3469 Ft-Lb
Mmax = 1327 Ft-Lb <= Ma = 2077 Ft-Lb

Check Deflection

Deflection Limit: L/240
Load Multiplier for Deflection = 1.00
Maximum Deflection = 0.931 in   Deflection Ratio = L/232

Check Shear

Vmax = 282 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2823 lb >= Vmax

Check Web Crippling

Rmax = 282 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 599 lb >= Rmax, stiffeners not required

Check Axial Interactions

P = 880 lb (Including Axial Load Multiplier)
Axial Loads Multiplied by 1.00 for Interaction Checks
Max unbraced length, KyLy and KtLt = 48.0 in   Max KL/r = 95
Allowable Pure Axial Load, Pa = 4519 lb : Axial Load Ratio, P/Pa = 0.195
K-phi for Distortional Buckling = 0 lb*in/in

Check Equation C5.2.1-1
Cmx = 1.00
Pcr = 17850 lb   Alpha = 0.905
Equation C5.2.1-1 = 0.901

Check Equation C5.2.1-2
Pao = 8521 lb
Equation C5.2.1-2 = 0.742

Maximum Interaction = 0.901 <=1.0

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PROJECT: CFS-NEES	PROJECT NO: 10-277	DESIGN: RWM	DATE: Feb '11

2nd Floor Framed Openings

Max RO width = 8'
Typ RO Height = 4' w/ sill @ 2.5'
WL = 15 psf

Gravity Load Support

$w_y = (20+20)\left(\frac{22}{2}\right) + \underbrace{10\,(1.25+2.5)}_{\text{Wall/parapet}} = 478 \text{ lb/ft}$

(1) ∴ Use min 1200T150-68 Rim Track
Do Not Splice over RO
(Long sides)

@ short sides, Max Span ≤ 4'

$w \leq (20+20)(1) + 10\,(1.25+2.5) = 78 \text{ lb/ft}$

∴ 1200S200-54 end Joist ok

Head & Sill Tracks

$w = 15\left(\frac{6.5}{2}\right) = 49 \text{ lb/ft}$

(2) ∴ Use min 600T150-33 Typ Head & Sill

Conn @ Jambs R_y = 196 lb

∴ Use 600S162-33 x 0'4" Cripple w/ (4) #10

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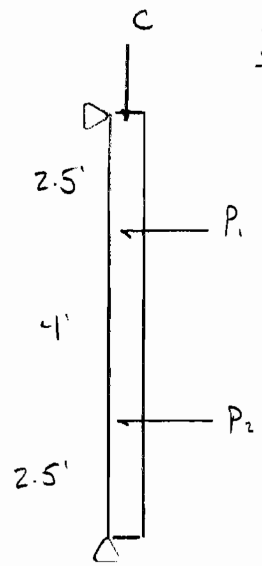
PROJECT: CFS-NEES | **PROJECT NO:** 10-277 | **DESIGN:** RUM | **DATE:** Feb '11



Jambs

$w = 15$ lb/ft

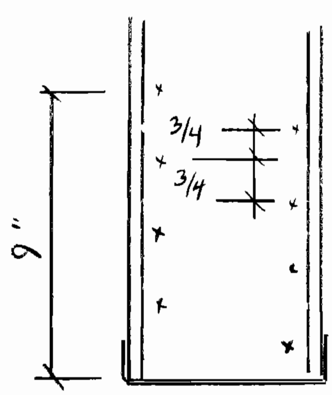
$P_1 = P_2 = 196$ lb

$C = 1912$ lb @ 3" ecc

$\therefore V_e = 1912\left(\frac{3}{12}\right) = 478$ lb @ 12" oc

4 3 : Use (2) 600S162-33 B/B. ALT (1) 600S162-54

B/B Jamb Interconnection (NASPEC D1.2)



Intermediate Connectors:

$P_n = 5256(1.8) = 9460$ lb

$0.025 P_n = 237$ lb

$\therefore$ Use (2) #10 @ 12" oc intermediates

End Connectors:

spaced 4 dia apart for 1.5 × member depth $\therefore$ 4d = 0.76"

$\therefore$ Use (8) #10 @ 1½" oc staggered Typ Top + Bott.

Jamb to Rim Track

$T_x = 210$ lb ; $T_y = 1912$ lb

$\#scr = \frac{210}{84} + \frac{1912}{177} = 13.3$

$\therefore$ Use (14) #10 Typ @ Jambs

Note: Jambs may be superseded by Shearwall Chord members

2007 North American Specification ASD
DATE: 2/17/2011
CFS-NEES

1

SECTION DESIGNATION: 1200T150-68 [50] Single

Input Properties:

Web Height =	12.250 in	Design Thickness =	0.0713 in
Top Flange =	1.500 in	Inside Corner Radius =	0.1070 in
Bottom Flange =	1.500 in	Yield Point, Fy =	50.0 ksi
		Fy With Cold-Work, Fya =	50.0 ksi

Header/Beam Solver Design Data - Simple Span

Header/Beam Span 8.00 ft Deflection Limit L/360
Dead Load = 478.0 lb/ft DL Multiplied by 1.00 for Strength Checks

Check Flexure

Flexural Bracing: Full
Mmax = 3824 Ft-Lb <= Ma = 4957 Ft-Lb & Ma(distortional) = 4957 Ft-Lb
K-phi for Distortional Buckling = 0 lb*in/in

Check Deflection

Deflection Limit: L/360
Maximum Deflection = 0.087 in Deflection Ratio = L/1104

Check Shear

Vmax = 1912 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2713 lb >= Vmax

Check Web Crippling

Rmax = 1912 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 573 lb < Rmax, STIFFENERS REQUIRED

2007 North American Specification ASD
DATE: 2/17/2011
CFS-NEES

SECTION DESIGNATION: 600T150-33 [33] Single

Input Properties:

Web Height =	6.146 in	Design Thickness =	0.0346 in
Top Flange =	1.500 in	Inside Corner Radius =	0.0765 in
Bottom Flange =	1.500 in	Yield Point, Fy =	33.0 ksi
		Fy With Cold-Work, Fya =	33.0 ksi

Header/Beam Solver Design Data - Simple Span

Header/Beam Span 8.00 ft — Deflection Limit L/360
Dead Load = .0 lb/ft — DL Multiplied by 1.00 for Strength Checks
Wind Load = 49.0 lb/ft — WL Multiplied by 1.00 for Strength Checks
WL Multiplied by 1.00 for Deflection Checks

Check Flexure

Flexural Bracing: Full
Mmax = 392 Ft-Lb <= Ma = 499 Ft-Lb & Ma(distortional) = 499 Ft-Lb
K-phi for Distortional Buckling = 0 lb*in/in

Check Deflection

Deflection Limit: L/360
Maximum Deflection = 0.113 in — Deflection Ratio = L/849

Check Shear

Vmax = 196 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 622 lb >= Vmax

Check Web Crippling

Rmax = 196 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 91 lb < Rmax, STIFFENERS REQUIRED

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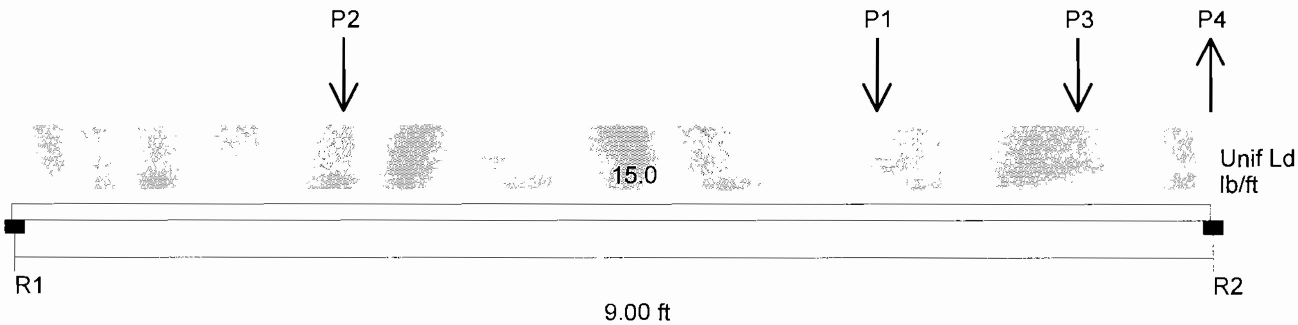
Project: CFS-NEES **Date:** 2/17/2011
Model: 2nd Level Framed Openings - Jambs (Long Sides)



Point Loads	**P1**	**P2**	**P3**	**P4**
Load(lb)	196	196	478	-478
X-Dist.(ft)	6.50	2.50	8.00	9.00

Section : (2) 600S162-33 Back-to-Back C Stud (X-X Axis) **Fy =** 33.0 ksi
Maxo = 1901.3 Ft-Lb **Moment of Inertia, I =** 3.586 in^4 **Va =** 1276.1 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	**Mmax Ft-Lb**	**Mmax/ Maxo**	**Mpos Ft-Lb**	**Bracing (in)**	**Ma(Brc) Ft-Lb**	**Mpos/ Ma(Brc)**	**Deflection (in)**	**Deflection Ratio**
Center Span	957.1	0.503	957.0	Mid-Pt	1901.3	0.503	0.133	L/813

Distortional Buckling Check

Span	**K-phi lb-in/in**	**Lm Brac (in)**	**Ma-d Ft-Lb**	**Mmax/ Ma-d**
Center Span	0.00	108.0	1577.7	0.607

Combined Bending and Web Crippling

Reaction or Pt Load	**Load P(lb)**	**Brng (in)**	**Pa (lb)**	**Pn (lb)**	**Mmax (Ft-Lb)**	**Intr. Value**	**Stiffen Req'd ?**
R1	316.6	1.00	773.6	1547.3	0.0	0.18	No
R2	210.4	1.00	366.5	732.9	0.0	0.25	No
P1	196.0	1.50	950.8	1568.9	957.0	0.41	No
P2	196.0	1.50	950.8	1568.9	742.7	0.34	No
P3	478.0	1.50	950.8	1568.9	682.4	0.48	No
P4	**-478.0**	1.50	391.2	782.5	0.0	0.54	**YES**

Combined Bending and Shear

Reaction or Pt Load	**Vmax (lb)**	**Mmax (Ft-Lb)**	**Va Factor**	**V/Va**	**M/Ma**	**Intr. Unstiffen**	**Intr. Stiffen**
R1	316.6	0.0	1.00	0.25	0.00	0.06	NA
R2	688.4	0.0	1.00	0.54	0.00	0.29	NA
P1	173.0	957.0	1.00	0.14	0.50	0.27	NA
P2	279.3	742.7	1.00	0.22	0.39	0.20	NA
P3	673.4	682.4	1.00	0.53	0.36	0.41	NA
P4	688.4	0.0	1.00	0.54	0.00	0.29	NA

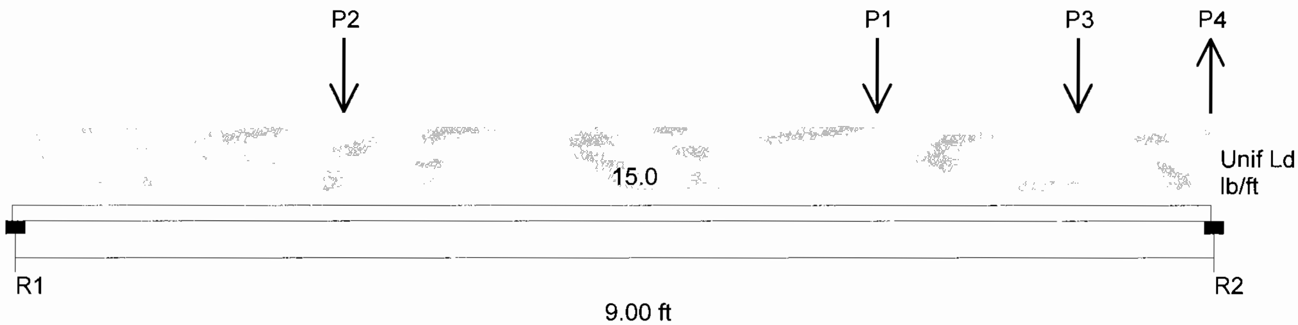
Combined Bending and Axial Load

Span	**Axial Ld (lb)**	**Bracing (in) KyLy**	**Bracing (in) KtLt**	**Max KL/r**	**K-phi (in-lb/in)**	**Lm Brac (in)**	**Allow Ld (lb)**	**P/Pa**	**Intr. Value**
Center Span	1912.0 (c)	Mid-Pt	Mid-Pt	79	0.0	108.0	5256.1 (c)	0.36	1.00

Member Interconnection Spacing = 12.00 in
See NASPEC C4.5 for add'nl interconnection requirements

2007 NASPEC

Project: CFS-NEES **Date:** 2/17/2011
Model: 2nd Level Framed Openings - Jambs (Long Sides) ALT



Point Loads	**P1**	**P2**	**P3**	**P4**
Load(lb)	196	196	478	-478
X-Dist.(ft)	6.50	2.50	8.00	9.00

Section : 600S162-54 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 2527.1 Ft-Lb **Moment of Inertia, I =** 2.860 in^4 **Va =** 2822.9 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	957.1	0.379	957.0	Mid-Pt	2005.5	0.477	0.166	L/649

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	108.0	2158.3	0.443

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	316.6	1.00	598.9	1048.1	0.0	0.27	No
R2	210.4	1.00	482.2	843.8	0.0	0.23	No
P1	196.0	1.50	1403.1	2315.1	957.0	0.30	No
P2	196.0	1.50	1403.1	2315.1	742.7	0.25	No
P3	478.0	1.50	1403.1	2315.1	682.4	0.35	No
P4	-478.0	1.50	518.5	907.4	0.0	0.48	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	316.6	0.0	1.00	0.11	0.00	0.01	NA
R2	688.4	0.0	1.00	0.24	0.00	0.06	NA
P1	173.0	957.0	1.00	0.06	0.38	0.15	NA
P2	279.3	742.7	1.00	0.10	0.29	0.10	NA
P3	673.4	682.4	1.00	0.24	0.27	0.13	NA
P4	688.4	0.0	1.00	0.24	0.00	0.06	NA

Combined Bending and Axial Load

Span	Axial Ld (lb)	Bracing (in) KyLy	Bracing (in) KtLt	Max KL/r	K-phi (in-lb/in)	Lm Brac (in)	Allow Ld (lb)	P/Pa	Intr. Value
Center Span	1912.0 (c)	Mid-Pt	Mid-Pt	95	0.0	108.0	5097.8 (c)	0.38	0.88

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PROJECT: CFS-NEES	PROJECT NO: 10-277	DESIGN: RUM	DATE: Mar '11

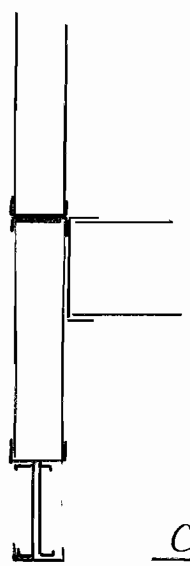
1st Floor Framed Openings

Max RO width = 8'
Typ RO Ht = 4' w/ sill @ 3' (No sill @ Doors)
WL = 15 psf

Gravity load Support

Case 1 - Long sides - No Clerestory Corners

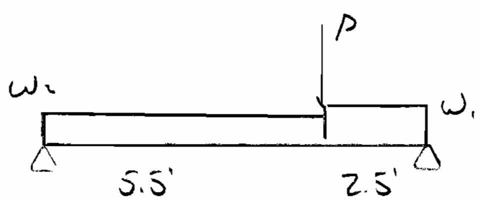
Roof DL + LL :	$\omega = (20+20)(\frac{22}{2})$	=	440 lb/ft
Wall DL :	10 (9 + 1.25 + 2)	=	123 "
Floor DL+LL :	2003 (12/24)	=	1002 "
			1565 lb/ft



(1) : For RO ≤ 6.5' wide 1200T200-97 Rim Track ok. Do Not splice over RO

(2) : For RO > 6.5' wide, Use (2) 1200S250-97 B/B. Stiffeners ea end

Case 2 - Long sides w/ Clerestory Corners



$\omega_1 = 1565$ lb/ft
$\omega_2 = 440 + 123 + 83(\frac{3}{2}) = 771$ lb/ft
$P = 2075$ lb

(3) : Use (2) 1200S250-97 B/B stiffeners ea end

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PROJECT: CFS-NEES | **PROJECT NO:** 10-277 | **DESIGN:** RLM | **DATE:** Mar '11

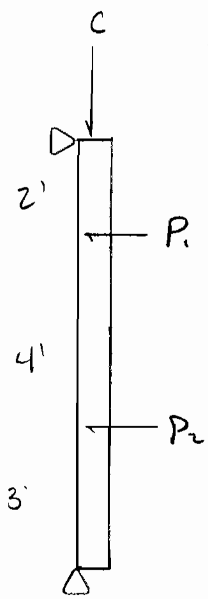
Case 3: Short Sides
May RO ≤ 6' wide. Does not carry floor or roof loads

: 1200S250-97 End Joist ok

Head & Sill Tracks

$\omega \leq 15\left(\frac{9}{2}\right) = 68$ lb/ft

(4) : Use 100T150-54 Typ Head & Sill Tracks



Jambs

Note: ω considered to include Roof DL + LL explicitly. No need to add Conc. axial load from Level 2 Jambs

$\omega = 15$ lb/ft

$P_1 \leq 15\left(\frac{6}{2}\right)\left(\frac{8}{2}\right) = 180$ lb

$P_2 \leq 15\left(\frac{7}{2}\right)\left(\frac{8}{2}\right) = 210$ lb

Case 1 - RO ≤ 6.5' (No B/B hdr)

$C = 5086$ lb

$Me = 1002\left(\frac{6.5}{2}\right)(3) = 9.77$ in-k ⟵ Ve @ 12"

$Ve = \frac{9770}{12} = 814$ lb

(5) (6) : Use (2) 600S162-54 B/B
ALT (1) 600S200-68

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Case 2. Max 8 120 (w/ or w/o Clerestory)

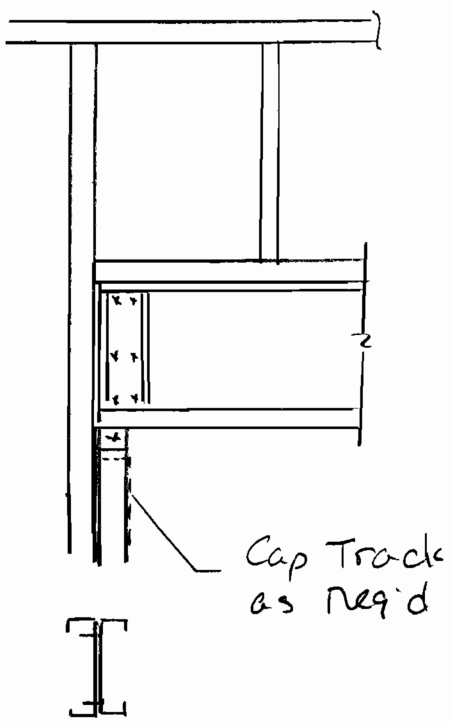
$C_{max} = 1565 \left(\frac{8}{2}\right) = 6260\ lb$ Total

@ Trimmer: $C_T = (1002 + 123)\left(\frac{8}{2}\right) = 4500\ lb$

@ Jamb : $C_J = 1760\ lb$

Note: @ Clerestory Carries Cond'n
$\Sigma C = 6185\ lb$ + $C_J = 1760\ lb$

7 8 : Use 600S162-54 Trimmer +
600S162-54 Jamb



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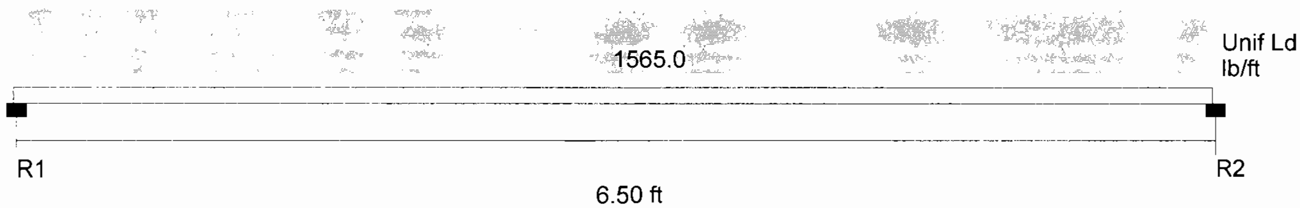
Project: CFS-NEES **Date:** 3/1/2011
Model: 1st Floor Openings - Gravity Case 1 Max 6.5 ft RO



Section : 1200T200-97 Single Track (X-X Axis) **Fy =** 50.0 ksi
Maxo = 9528.9 Ft-Lb **Moment of Inertia, I =** 28.959 in^4 **Va =** 7902.2 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

<u>Flexural and Deflection Check</u>

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Ratio
Center Span	8265.2	0.867	8265.2	Full	9528.9	0.867	0.074	L/1060

<u>Combined Bending and Web Crippling</u>

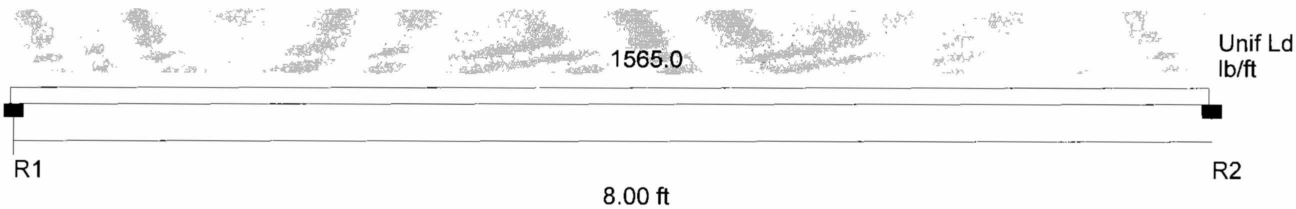
Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	**5086.3**	1.00	1142.2	2056.0	0.0	**2.25**	**YES**
R2	**5086.3**	1.00	1142.2	2056.0	0.0	**2.25**	**YES**

<u>Combined Bending and Shear</u>

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	5086.3	0.0	1.00	0.64	0.00	0.41	NA
R2	5086.3	0.0	1.00	0.64	0.00	0.41	NA

2007 NASPEC

Project: CFS-NEES **Date:** 3/1/2011
Model: 1st Floor Openings - Gravity Case 1 Max 8 ft RO



Section : (2) 1200S250-97 Back-to-Back C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 25136.6 Ft-Lb **Moment of Inertia, I =** 67.669 in^4 **Va =** 16294.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	12520.0	0.498	12520.0	Full	25136.6	0.498	0.072	L/1329

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	96.0	22562.2	0.555

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	6260.0	1.00	7961.2	15922.4	0.0	0.35	No
R2	6260.0	1.00	7961.2	15922.4	0.1	0.35	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	6260.0	0.0	1.00	0.38	0.00	0.15	NA
R2	6260.0	0.1	1.00	0.38	0.00	0.15	NA

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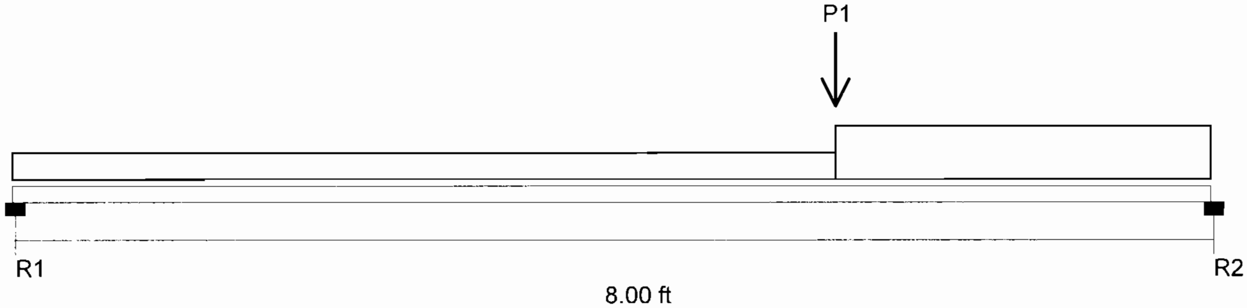
Project: CFS-NEES **Date:** 3/1/2011
Model: 1st Floor Openings - Gravity Case 2



Point Loads	**P1**
Load(lb)	2075
X-Dist.(ft)	5.50

Sloped/Partial Loads	**Case**	**X1 ft**	**W(X1) lb/ft**	**X2 ft**	**W(X2) lb/ft**
	1	0.00	771.0	5.50	771.0
	2	5.50	1565.0	8.00	1565.0

Section : (2) 1200S250-97 Back-to-Back C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 25136.6 Ft-Lb **Moment of Inertia, I =** 67.669 in^4 **Va =** 16294.0 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	10598.3	0.422	10598.3	Full	25136.6	0.422	0.059	L/1617

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	96.0	22562.2	0.470

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	4042.6	1.00	7961.2	15922.4	0.0	0.22	No
R2	6185.4	1.00	7961.2	15922.4	8.1	0.34	No
P1	2075.0	1.50	11809.8	19486.1	10573.7	0.35	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	4042.6	0.0	1.00	0.25	0.00	0.06	NA
R2	6182.2	8.1	1.00	0.38	0.00	0.14	NA
P1	2276.0	10573.7	1.00	0.14	0.42	0.20	NA

Within Span (Unstiffened)

Span	Unpunched Loc'n, X (ft)	Unpunched M(X) (Ft-Lb)	Unpunched V(X) (lb)	Unpunched Intr.	Punched Loc'n, X (ft)	Punched M(X) (Ft-Lb)	Punched V(X) (lb)	Punched Intr.
Center Span	5.50	10563.9	-2276.0	0.20	5.50	10563.9	-2276.0	0.20

SECTION DESIGNATION: 600T150-54 [50] Single

Input Properties:

Web Height =	6.198 in	Design Thickness =	0.0566 in
Top Flange =	1.500 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.500 in	Yield Point, Fy =	50.0 ksi
		Fy With Cold-Work, Fya =	50.0 ksi

Header/Beam Solver Design Data - Simple Span

Header/Beam Span 8.00 ft — Deflection Limit L/360
Dead Load = .0 lb/ft — DL Multiplied by 1.00 for Strength Checks
Wind Load = 68.0 lb/ft — WL Multiplied by 1.00 for Strength Checks
WL Multiplied by 1.00 for Deflection Checks

Check Flexure

Flexural Bracing: Full
Mmax = 544 Ft-Lb <= Ma = 1520 Ft-Lb

Check Deflection

Deflection Limit: L/360
Maximum Deflection = 0.089 in — Deflection Ratio = L/1085

Check Shear

Vmax = 272 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2728 lb >= Vmax

Check Web Crippling

Rmax = 272 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 443 lb >= Rmax, stiffeners not required

5

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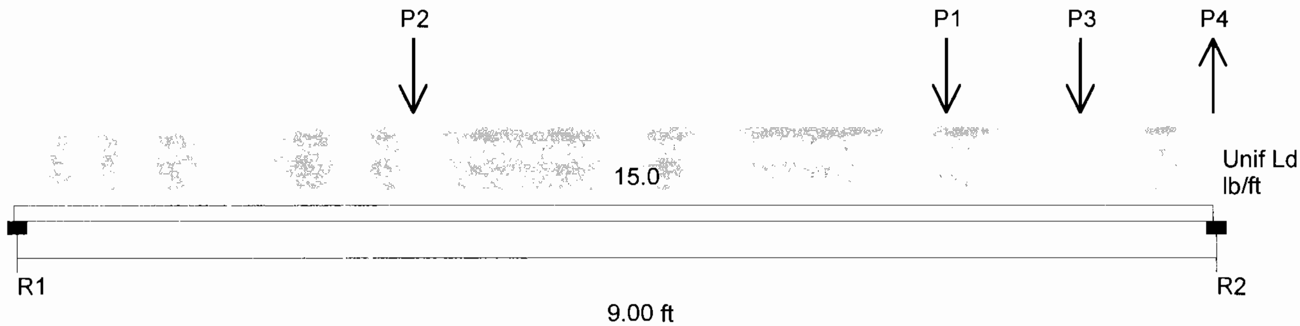
Project: CFS-NEES **Date:** 3/1/2011
Model: 1st Floor Openings - Jambs Case 1



Point Loads	**P1**	**P2**	**P3**	**P4**
Load(lb)	180	210	814	-814
X-Dist.(ft)	7.00	3.00	8.00	9.00

Section : (2) 600S162-54 Back-to-Back C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 5054.2 Ft-Lb **Moment of Inertia, I =** 5.721 in^4 **Va =** 5645.8 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	**Mmax Ft-Lb**	**Mmax/ Maxo**	**Mpos Ft-Lb**	**Bracing (in)**	**Ma(Brc) Ft-Lb**	**Mpos/ Ma(Brc)**	**Deflection (in)**	**Deflection Ratio**
Center Span	1158.1	0.229	1157.9	Full	5054.2	0.229	0.100	L/1078

Distortional Buckling Check

Span	**K-phi lb-in/in**	**Lm Brac (in)**	**Ma-d Ft-Lb**	**Mmax/ Ma-d**
Center Span	0.00	108.0	4316.7	0.268

Combined Bending and Web Crippling

Reaction or Pt Load	**Load P(lb)**	**Brng (in)**	**Pa (lb)**	**Pn (lb)**	**Mmax (Ft-Lb)**	**Intr. Value**	**Stiffen Req'd ?**
R1	337.9	1.00	2860.0	5720.1	0.0	0.05	No
R2	187.1	1.00	1765.2	3530.5	0.0	0.05	No
P1	180.0	1.50	3865.4	6378.0	1157.9	0.16	No
P2	210.0	1.50	3865.4	6378.0	945.4	0.14	No
P3	814.0	1.50	3865.4	6378.0	994.9	0.23	No
P4	-814.0	1.50	1865.1	3730.1	0.0	0.19	No

Combined Bending and Shear

Reaction or Pt Load	**Vmax (lb)**	**Mmax (Ft-Lb)**	**Va Factor**	**V/Va**	**M/Ma**	**Intr. Unstiffen**	**Intr. Stiffen**
R1	337.9	0.0	1.00	0.06	0.00	0.00	NA
R2	1001.1	0.0	1.00	0.18	0.00	0.03	NA
P1	157.1	1157.9	1.00	0.03	0.23	0.05	NA
P2	293.1	945.4	1.00	0.05	0.19	0.04	NA
P3	986.1	994.9	1.00	0.17	0.20	0.07	NA
P4	1001.1	0.0	1.00	0.18	0.00	0.03	NA

Combined Bending and Axial Load

Span	**Axial Ld (lb)**	**Bracing (in) KyLy**	**Bracing (in) KtLt**	**Max KL/r**	**K-phi (in-lb/in)**	**Lm Brac (in)**	**Allow Ld (lb)**	**P/Pa**	**Intr. Value**
Center Span	5086.0 (c)	48.0	48.0	71	0.0	108.0	12971.5 (c)	0.39	0.68

Member Interconnection Spacing = 12.00 in
See NASPEC C4.5 for add'nl interconnection requirements

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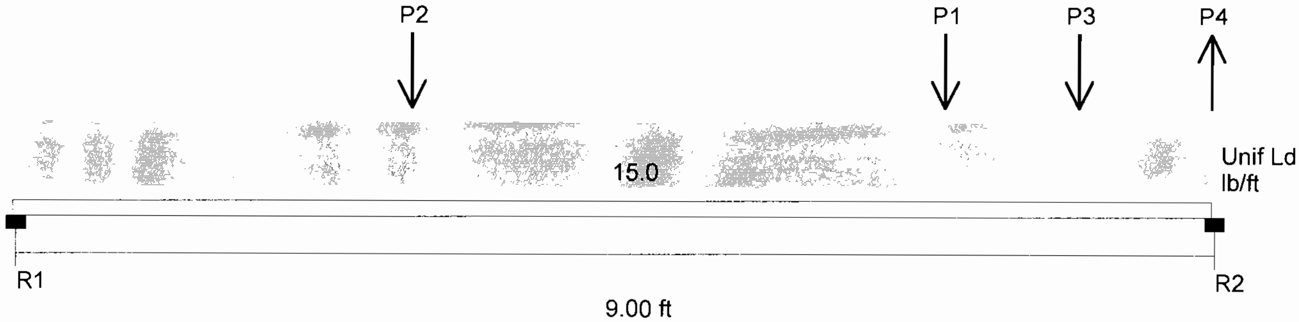
Project: CFS-NEES **Date:** 3/1/2011
Model: 1st Floor Openings - Jambs Case 1 ALT



Point Loads	**P1**	**P2**	**P3**	**P4**
Load(lb)	180	210	814	-814
X-Dist.(ft)	7.00	3.00	8.00	9.00

Section : 600S200-68 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 3642.4 Ft-Lb **Moment of Inertia, I =** 4.101 in^4 **Va =** 5350.3 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Ratio
Center Span	1158.1	0.318	1157.9	Full	3642.4	0.318	0.140	L/772

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	108.0	3307.5	0.350

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	337.9	1.00	914.4	1600.2	0.0	0.19	No
R2	187.1	1.00	816.0	1428.0	0.0	0.12	No
P1	180.0	1.50	2152.1	3551.0	1157.9	0.24	No
P2	210.0	1.50	2152.1	3551.0	945.4	0.21	No
P3	814.0	1.50	2152.1	3551.0	994.9	0.37	No
P4	-814.0	1.50	872.9	1527.5	0.0	0.48	No

Combined Bending and Shear

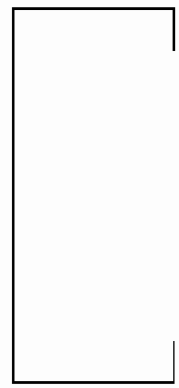
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	337.9	0.0	1.00	0.06	0.00	0.00	NA
R2	1001.1	0.0	1.00	0.19	0.00	0.04	NA
P1	157.1	1157.9	1.00	0.03	0.32	0.10	NA
P2	293.1	945.4	1.00	0.05	0.26	0.07	NA
P3	986.1	994.9	1.00	0.18	0.27	0.11	NA
P4	1001.1	0.0	1.00	0.19	0.00	0.04	NA

Combined Bending and Axial Load

Span	Axial Ld (lb)	Bracing (in) KyLy	KtLt	Max KL/r	K-phi (in-lb/in)	Lm Brac (in)	Allow Ld (lb)	P/Pa	Intr. Value
Center Span	5086.0 (c)	48.0	48.0	66	0.0	108.0	9978.5 (c)	0.51	0.90

2007 North American Specification ASD
DATE: 3/1/2011
SFOBB Toll Ops Bldg

SECTION DESIGNATION: 600S162-54 [50] Single



Section Dimensions:

Web Height =	6.000 in
Top Flange =	1.625 in
Bottom Flange =	1.625 in
Stiffening Lip =	0.500 in
Inside Corner Radius =	0.0849 in
Punchout Width =	1.500 in
Punchout Length =	4.000 in
Design Thickness =	0.0566 in

Steel Properties:

Fy =	50.000 ksi
Fu =	65.000 ksi
Fya =	55.318 ksi

ALLOWABLE AXIAL LOADS

INPUT PARAMETERS

Overall Stud Length = 7 ft
Load has not been modified for load type or duration
Member Configuration: SINGLE MEMBER

K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

TOTAL ALLOWABLE AXIAL LOADS (lb)

WEAK AXIS BRACING	MAXIMUM KL/r	CONCENTRIC LOADING	LOADED THROUGH WEB
48 in	84	5727	2792
MID Pt	74	6349	2992
THIRD Pt	49	7609	3292

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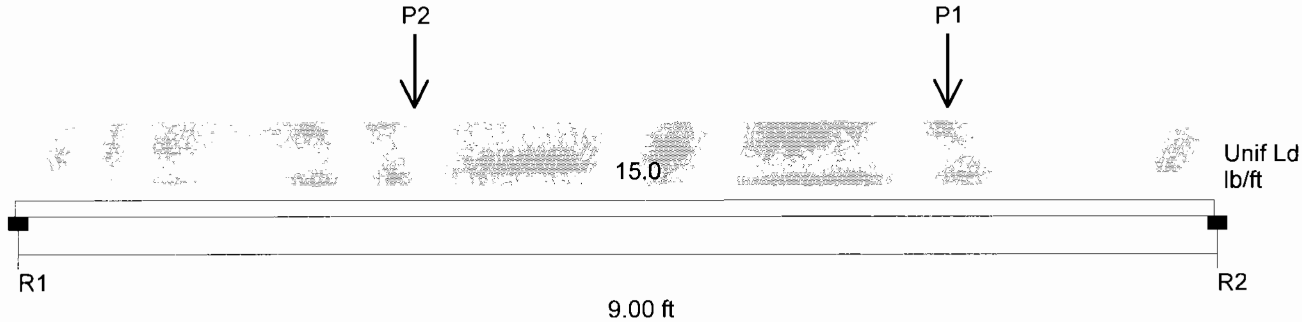
Project: CFS-NEES **Date:** 3/1/2011
Model: 1st Floor Openings - Jambs Case 2



Point Loads	**P1**	**P2**
Load(lb)	180	210
X-Dist.(ft)	7.00	3.00

Section : 600S162-54 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 2527.1 Ft-Lb **Moment of Inertia, I =** 2.860 in^4 **Va =** 2822.9 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	675.0	0.267	675.0	Full	2527.1	0.267	0.117	L/925

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	108.0	2158.3	0.313

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	247.5	1.00	598.9	1048.1	0.0	0.21	No
R2	277.5	1.00	598.9	1048.1	0.0	0.24	No
P1	180.0	1.50	1403.1	2315.1	525.5	0.20	No
P2	210.0	1.50	1403.1	2315.1	674.4	0.24	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	247.5	0.0	1.00	0.09	0.00	0.01	NA
R2	277.5	0.0	1.00	0.10	0.00	0.01	NA
P1	247.5	525.5	1.00	0.09	0.21	0.05	NA
P2	202.7	674.4	1.00	0.07	0.27	0.08	NA

Combined Bending and Axial Load

Span	Axial Ld (lb)	Bracing (in) KyLy	Bracing (in) KtLt	Max KL/r	K-phi (in-lb/in)	Lm Brac (in)	Allow Ld (lb)	P/Pa	Intr. Value
Center Span	1760.0 (c)	48.0	48.0	84	0.0	108.0	5726.8 (c)	0.31	0.64

Appendix 2

Seismic Lateral Analysis

CFS-NEES

Seismic Analysis (LFRS) per ASCE 7-10

Occ. Category	II			
I_e =	1.0			
S_s =	1.39	F_a =	1.0	(Table 11.4-1)
S_1 =	0.50	F_v =	1.5	(Table 11.4-2)
Site Class	D			
h	18 (ft)			

$S_{MS} = F_a S_S$ =	1.39	(Eq. 11.4-1)
$S_{M1} = F_v S_1$ =	0.75	(Eq. 11.4-2)
$S_{DS} = 2/3*S_{MS}$ =	0.927	(Eq. 11.4-3)
$S_{D1} = 2/3*S_{M1}$ =	0.500	(Eq. 11.4-4)

Bearing Wall System

Light-frame (cold-formed steel) walls sheathed with wood structural panels or steel sheet.

Table 12.2-1

R	6.5	$V = C_s W$		(Eq. 12.8.1)
Ω_0	3	**C_s =**	**0.143**	(Eq. 12.8-2)
C_d	4			
Max Ht.	65 ft.			

C_t =	0.02	C_{smax} =	0.440	(Eq. 12.8-3)
x =	0.75	C_{smin} =	0.01	(Eq. 12.8-5)
T_a =	0.175 (sec)			
T_L =	12 (sec)			

Base and Structural Level Shear, V Calculation

Building Dimensions			**Unit Weights**		
Width (E-W)	49.75	(ft)	Roof	20	(psf)
Length (N-S)	23.00	(ft)	Floor	18	(psf)
			Walls	10	(psf)
$H_{1\text{-}2}$	9.00	(ft)	Partitions	10	(psf)
$H_{2\text{-}R}$	9.00	(ft)	Rooftop MEP		
Parapet	1.25	(ft)		1200	(lb) Total

Clerestory (2nd Floor)

Width (E-W)	8.50	(ft)	C.G. (SW Corner = 0,0; X = East, Y = North)			
Length (N-S)	10.00	(ft)	X =	34.5	Y =	10

Element Masses

Roof	22885	(lb)			
Rooftop MEP	1200	(lb)			
2nd Floor DL	32039	(lb) - includes partitions			
Clerestory	0	(lb) - includes partitions			**Exclude**
Lower Walls	6548	(lb)	Considers only top half of these walls		
Upper Walls	13095	(lb)			
Parapet	1819	(lb)	**Overall Base Shear**		
Total Mass, W	**77585**	(lb)	V =	0.143	* W
			V =	**11061**	(lb)

Vertical Distribution (12.8.3)

k = 1 (Period less than 0.5 sec)

Level	**w_x (lb)**	**h_x (ft)**	**$w_x h_x^k$**	**C_{vx}**	**F_x (lb)**
Roof	32451	18.00	584123	0.590	**6524**
2nd	45134	9.00	406206	0.410	**4537**
			990329		

Notes:

Roof w_x based on Roof DL, Rooftop MEP, Parapet and 1/2 of Upper Walls

2nd Level wx based on 2nd Floor DL, Less Clerestory, + 0.5 x (Upper Walls + Lower Walls)

Appendix 3

Shearwall and Diaphragm Analysis and Design

Shearwall Relative Stiffness - For Horizontal Distribution

Based on AISI S213-07 C2.1.1

V =	1000	(lb)	Nominal value for determining relative stiffness
E_s =	2.95E+07	(psi)	
Gt =	77500	(lb/in)	Based on IBC Table 2305.2.2, taken conservatively for OSB. Also in the 2005 NDS Manual, Table M9.2.4 (page 65
ρ =	1.05	Constant 1.85 for ply, 1.05 for OSB	
ω_4 =	1.0	Constant for wood structural panels	
β =	660	Constant 810 for Plywood, 660 for OSB	

Upper SW	b (ft)	v (lb/ft)	h(ft)	A_c (in^2)[a]	Fast'nr Spc, s (in)	t_{stud} (in)[b]	ω_1 (in)	ω_2 (in)	ω_3	δ_v [c] (in)	T @ δ_v [c] (lb)	δ_v' [d] (in)	d (in) Cant. Bend	d (in) Shth Shr.	d (in) Nonlinear	d (in) anchors	$\Sigma\delta$ (in/kip)	Est. %V
L2S1	4	250	9	0.69	6	0.033	1.00	1.00	1.061	0.093	3705	0.056	0.0179	0.0276	0.1522	0.13	0.3248	0.277
L2S2	5	200	9	0.69	6	0.033	1.00	1.00	0.949	0.093	3705	0.045	0.0115	0.0221	0.0871	0.08	0.2020	0.446
L2S3	4	250	9	0.69	6	0.033	1.00	1.00	1.061	0.093	3705	0.056	0.0179	0.0276	0.1522	0.13	0.3248	0.277
L2N1	12	83	9	0.69	6	0.033	1.00	1.00	0.612	0.093	3705	0.019	0.0020	0.0092	0.0098	0.01	0.0351	0.687
L2N2	8	125	9	0.69	6	0.033	1.00	1.00	0.750	0.093	3705	0.028	0.0045	0.0138	0.0269	0.03	0.0770	0.313
L2W1	4	250	9	0.69	6	0.033	1.00	1.00	1.061	0.093	3705	0.056	0.0179	0.0276	0.1522	0.13	0.3248	0.191
L2W2	4	250	9	0.69	6	0.033	1.00	1.00	1.061	0.093	3705	0.056	0.0179	0.0276	0.1522	0.13	0.3248	0.191
L2W3	7	143	9	0.69	6	0.033	1.00	1.00	0.802	0.093	3705	0.032	0.0058	0.0158	0.0376	0.04	0.1007	0.617
L2E1	6	167	9	0.69	6	0.033	1.00	1.00	0.866	0.093	3705	0.038	0.0080	0.0184	0.0552	0.06	0.1381	0.358
L2E2	8	125	9	0.69	6	0.033	1.00	1.00	0.750	0.093	3705	0.028	0.0045	0.0138	0.0269	0.03	0.0770	0.642

Notes:
a. Chord area based on (2) 600S162-33
b. Defined as framing 'designation' thickness (use minimum deliverable)
c. Need to determine actual value based on selected hold-downs and/or component tests

Lower SW	b (ft)	v (lb/ft)	h(ft)	A_c (in^2)[a]	Fast'nr Spc, s (in)	t_{stud} (in)[b]	ω_1 (in)	ω_2 (in)	ω_3	δ_v [c] (in)	T @ δ_v [c] (lb)	δ_v' [d] (in)	d (in) Cant. Bend	d (in) Shth Shr.	d (in) Nonlinear	d(in) anchors	$\Sigma\delta$ (in/kip)	Est. %V
L1S1	4	250	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.05	0.0109	0.0169	0.0930	0.12	0.2419	0.278
L1S2	5	200	9	1.13	6	0.054	1.00	0.61	0.949	0.234	9785	0.04	0.0070	0.0135	0.0532	0.08	0.1512	0.444
L1S3	4	250	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.05	0.0109	0.0169	0.0930	0.12	0.2419	0.278
L1N1	12	83	9	1.13	6	0.054	1.00	0.61	0.612	0.234	9785	0.02	0.0012	0.0056	0.0060	0.01	0.0263	0.688
L1N2	8	125	9	1.13	6	0.054	1.00	0.61	0.750	0.234	9785	0.03	0.0027	0.0084	0.0164	0.03	0.0579	0.312
L1W1	4	250	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.05	0.0109	0.0169	0.0930	0.12	0.2419	0.192
L1W2	4	250	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.05	0.0109	0.0169	0.0930	0.12	0.2419	0.192
L1W3	7	143	9	1.13	6	0.054	1.00	0.61	0.802	0.234	9785	0.03	0.0036	0.0097	0.0230	0.04	0.0757	0.615
L1E1	6	167	9	1.13	6	0.054	1.00	0.61	0.866	0.234	9785	0.04	0.0049	0.0113	0.0337	0.05	0.1037	0.358
L1E2	8	125	9	1.13	6	0.054	1.00	0.61	0.750	0.234	9785	0.03	0.0027	0.0084	0.0164	0.03	0.0579	0.642

Notes:
a. based on (2) 600S162-54
b. Defined as framing 'designation' thickness (use minimum deliverable)
c. Need to determine actual value based on selected hold-downs and/or component tests
d. Note that the above distribution is an estimate only since displacement is not linear with v

Design Shearwalls (Type I)

Total Seismic Shear - Upper Level **6524** (lb)
From Seismic Lateral Analysis.xlsx

Upper SW	%V	V (lb)	w (ft)	v (lb/ft)	Sheathing	Fastener Edge Spc (in)	Table[1] v_n (lb/ft)	h (ft)	Aspect Ratio	Factor 2w/h	Adjusted v_n (lb/ft)	ϕ	ϕv_n	$v/(\phi v_n)$
L2S1	**0.295**	962	4	241	7/16" OSB	6	700	9	2.25	0.889	622	0.6	373	**0.644**
L2S2	**0.410**	1337	5	267	7/16" OSB	6	700	9	1.80	1.000	700	0.6	420	**0.637**
L2S3	**0.295**	962	4	241	7/16" OSB	6	700	9	2.25	0.889	622	0.6	373	**0.644**
L2N1	**0.640**	2088	12	174	7/16" OSB	6	700	9	0.75	1.000	700	0.6	420	**0.414**
L2N2	**0.360**	1174	8	147	7/16" OSB	6	700	9	1.13	1.000	700	0.6	420	**0.350**
L2W1	**0.237**	773	4	193	7/16" OSB	6	700	9	2.25	0.889	622	0.6	373	**0.518**
L2W2	**0.237**	773	4	193	7/16" OSB	6	700	9	2.25	0.889	622	0.6	373	**0.518**
L2W3	**0.526**	1716	7	245	7/16" OSB	6	700	9	1.29	1.000	700	0.6	420	**0.584**
L2E1	**0.400**	1305	6	217	7/16" OSB	6	700	9	1.50	1.000	700	0.6	420	**0.518**
L2E2	**0.600**	1957	8	245	7/16" OSB	6	700	9	1.13	1.000	700	0.6	420	**0.583**
			Max	267					2.25					**0.644**

Total Seismic Shear - Lower Level **11061** (lb)
From Seismic Lateral Analysis.xlsx (Includes Upper Level Shear)

Lower SW	%V	V (lb)	b (ft)	v (lb/ft)	Sheathing	Fastener Edge Spc (in)	Table[1] v_n (lb/ft)	h (ft)	Aspect Ratio	Factor 2w/h	Adjusted v_n (lb/ft)	ϕ	ϕv_n	$v/(\phi v_n)$
L1S1	**0.295**	1631	4	408	7/16" OSB	6	825	9	2.25	0.889	733	0.6	440	**0.927**
L1S2	**0.410**	2267	5	453	7/16" OSB	6	825	9	1.80	1.000	825	0.6	495	**0.916**
L1S3	**0.295**	1631	4	408	7/16" OSB	6	825	9	2.25	0.889	733	0.6	440	**0.927**
L1N1	**0.640**	3539	12	295	7/16" OSB	6	825	9	0.75	1.000	825	0.6	495	**0.596**
L1N2	**0.360**	1991	8	249	7/16" OSB	6	825	9	1.13	1.000	825	0.6	495	**0.503**
L1W1	**0.236**	1305	4	326	7/16" OSB	6	825	9	2.25	0.889	733	0.6	440	**0.742**
L1W2	**0.236**	1305	4	326	7/16" OSB	6	825	9	2.25	0.889	733	0.6	440	**0.742**
L1W3	**0.528**	2920	7	417	7/16" OSB	6	825	9	1.29	1.000	825	0.6	495	**0.843**
L1E1	**0.400**	2212	6	369	7/16" OSB	6	825	9	1.50	1.000	825	0.6	495	**0.745**
L1E2	**0.600**	3318	8	415	7/16" OSB	6	825	9	1.13	1.000	825	0.6	495	**0.838**
			Max	453					2.25					**0.927**

Notes:
1. 'Table' Shearwall capacity based on AISI S213-07, Tabel C2.1-3 unadjusted for aspect ratio
2. Upper level shearwall capacity based on 33-mil perimeter members
3. Lower level shearwall capacity based on 54-mil perimeter members
4. Sheathing screw size No. 8 Typ

Shearwall Displacements - Type I SW analysis only.

Based on AISI S213-07 C2.1.1

E_s = 2.95E+07 (psi)
Gt = 77500 (lb/in) Based on IBC Table 2305.2.2, taken conservatively for OSB. Also in the 2005 NDS Manual, Table M9.2.4 (page 65)
ρ = 1.05 Constant 1.85 for ply, 1.05 for OSB
ω_4 = 1.0 Constant for wood structural panels
β = 660 Constant 810 for Plywood, 660 for OSB

Upper SW	b (ft)	v (lb/ft)	h(ft)	A_c $(in^2)^a$	Fast'nr Spc, s (in)	t_{stud} $(in)^{b,e}$	ω_1 (in)	ω_2 (in)	ω_3	δ_v [c] (in)	T @ δ_v [c] (lb)	δ_v' [d] (in)	d (in) Cant. Bend	d (in) Shth Shr.	d (in) Nonlinear	d(in) anchors	$\Sigma\delta$ (in)	C_d	$\Delta = C_d\delta e$	Δ/h_{sx}
L2S1	4	241	9	0.69	6	0.033	1.00	1.00	1.061	0.1	5000	0.043	0.017	0.027	0.141	0.097	0.282	4	1.129	0.010
L2S2	5	267	9	0.69	6	0.033	1.00	1.00	0.949	0.1	5000	0.048	0.015	0.030	0.156	0.087	0.287	4	1.150	0.011
L2S3	4	241	9	0.69	6	0.033	1.00	1.00	1.061	0.1	5000	0.043	0.017	0.027	0.141	0.097	0.282	4	1.129	0.010
L2N1	12	174	9	0.69	6	0.033	1.00	1.00	0.612	0.1	5000	0.031	0.004	0.019	0.043	0.023	0.089	4	0.358	0.003
L2N2	8	147	9	0.69	6	0.033	1.00	1.00	0.750	0.1	5000	0.026	0.005	0.016	0.037	0.030	0.088	4	0.353	0.003
L2W1	4	193	9	0.69	6	0.033	1.00	1.00	1.061	0.1	5000	0.035	0.014	0.021	0.091	0.078	0.204	4	0.818	0.008
L2W2	4	193	9	0.69	6	0.033	1.00	1.00	1.061	0.1	5000	0.035	0.014	0.021	0.091	0.078	0.204	4	0.818	0.008
L2W3	7	245	9	0.69	6	0.033	1.00	1.00	0.802	0.1	5000	0.044	0.010	0.027	0.111	0.057	0.204	4	0.818	0.008
L2E1	6	217	9	0.69	6	0.033	1.00	1.00	0.866	0.1	5000	0.039	0.010	0.024	0.094	0.059	0.187	4	0.749	0.007
L2E2	8	245	9	0.69	6	0.033	1.00	1.00	0.750	0.1	5000	0.044	0.009	0.027	0.103	0.050	0.188	4	0.754	0.007
																		Max	**1.150**	**0.011**

Notes: a. based on (2) 600S162-33
b. Defined as framing 'designation' thickness (use minimum deliverable)
c. Estimate only pending component test results.
d. Scaled to actual tension load
e. Studs conservatively taken as 33-mil. Chord studs are 54-mil and tracks are 43-mil.

Lower SW	b (ft)	v (lb/ft)	h(ft)	A_c $(in^2)^a$	Fast'nr Spc, s (in)	t_{stud} $(in)^b$	ω_1 (in)	ω_2 (in)	ω_3	δ_v [c] (in)	T @ δ_v [c] (lb)	δ_v' [d] (in)	d (in) Cant. Bend	d (in) Shth Shr.	d (in) Nonlinear	d(in) anchors	$\Sigma\delta$ (in)	C_d	$\Delta = C_d\delta e$	Δ/h_{sx}
L1S1	4	408	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.088	0.018	0.028	0.248	0.198	0.490	4	1.962	0.018
L1S2	5	453	9	1.13	6	0.054	1.00	0.61	0.949	0.234	9785	0.098	0.016	0.031	0.274	0.176	0.496	4	1.984	0.018
L1S3	4	408	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.088	0.018	0.028	0.248	0.198	0.490	4	1.962	0.018
L1N1	12	295	9	1.13	6	0.054	1.00	0.61	0.612	0.234	9785	0.063	0.004	0.020	0.075	0.048	0.147	4	0.586	0.005
L1N2	8	249	9	1.13	6	0.054	1.00	0.61	0.750	0.234	9785	0.054	0.005	0.017	0.065	0.060	0.148	4	0.591	0.005
L1W1	4	326	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.070	0.014	0.022	0.158	0.158	0.353	4	1.411	0.013
L1W2	4	326	9	1.13	6	0.054	1.00	0.61	1.061	0.234	9785	0.070	0.014	0.022	0.158	0.158	0.353	4	1.411	0.013
L1W3	7	417	9	1.13	6	0.054	1.00	0.61	0.802	0.234	9785	0.090	0.010	0.028	0.196	0.115	0.350	4	1.399	0.013
L1E1	6	369	9	1.13	6	0.054	1.00	0.61	0.866	0.234	9785	0.079	0.011	0.025	0.165	0.119	0.320	4	1.279	0.012
L1E2	8	415	9	1.13	6	0.054	1.00	0.61	0.750	0.234	9785	0.089	0.009	0.028	0.181	0.100	0.319	4	1.274	0.012
																		Max	**1.984**	**0.018**

Notes: a. based on (2) 600S162-54
b. Defined as framing 'designation' thickness (use minimum deliverable)
c. Based on data from Simpson Strong Tie for S/HDU6 holddown and 54-mil chords.

Chord Stud Design Forces

$\Omega_0 =$ 3.0

$S_{DS} =$ 0.927

Upper Level Shearwalls

SW	v (lb/ft)	h (ft)	C_{seis} (lb)	C_{DL} (lb)	C_{LL} (lb)	P_{ui} (lb)	P_{ue} (lb)	Factored ΣC_u	M_u (in-lb)	$\Omega_0 * C_{seis}$ (lb)	factored[3] C (lb)	Max Load SW Can Deliver v_n (lb/ft)	C_{max} (lb)	factored[3] C (lb)	factored[3] M (in-lb)
L2S1	241	9	2165	770	770	1452	2165	3617	2140	6495	**7947**	622	5600	**7052**	12445
L2S2	267	9	2407	920	770	1660	2407	4067	2244	7222	**8882**	700	6300	**7960**	13921
L2S3	241	9	2165	550	440	982	2165	3147	3550	6495	**7477**	622	5600	**6582**	13854
L2N1	174	9	1566	590	440	1037	1566	2603	1585	4697	**5735**	700	6300	**7337**	15788
L2N2	147	9	1321	1100	440	1744	1321	3065	-1268	3963	**5707**	700	6300	**8044**	13668
L2W1	193	9	1739	0	0	0	1739	1739	5218	5218	**5218**	622	5600	**5600**	16800
L2W2	193	9	1739	0	0	0	1739	1739	5218	5218	**5218**	622	5600	**5600**	16800
L2W3	245	9	2206	0	0	0	2206	2206	6618	6618	**6618**	700	6300	**6300**	18900
L2E1	217	9	1957	0	0	0	1957	1957	5872	5872	**5872**	700	6300	**6300**	18900
L2E2	245	9	2202	0	0	0	2202	2202	6606	6606	**6606**	700	6300	**6300**	18900
		Max	**2407**					**4067**	**6618**	7222	**8882**		**6300**	**8044**	**18900**

Upper Level Shearwalls - Interactions

SW	Proposed Chord	I_{xx} (in^4)	P_{Ex} (kips)	LRFD Check α_x	ϕP_n (lb)	ϕM_{nx} (in-lb)	Int'xn (C5.2.2-1)	Strength Check α_x	P_n (lb)	M_{nx} (in-lb)	Int'xn (C5.2.2-1)
L2S1	(2) 600S162-54	5.72	142.78	0.975	19846	77855	0.210	0.951	23348	86506	0.453
L2S2	(2) 600S162-54	5.72	142.78	0.972	19846	77855	0.235	0.944	23348	86506	0.511
L2S3	(2) 600S162-54	5.72	142.78	0.978	19846	77855	0.205	0.954	23348	86506	0.450
L2N1	(2) 600S162-54	5.72	142.78	0.982	19846	77855	0.152	0.949	23348	86506	0.507
L2N2	(2) 600S162-54	5.72	142.78	0.979	19846	77855	0.138	0.944	23348	86506	0.512
L2W1	(2) 600S162-54	5.72	142.78	0.988	19846	77855	0.156	0.961	23348	86506	0.442
L2W2	(2) 600S162-54	5.72	142.78	0.988	19846	77855	0.156	0.961	23348	86506	0.442
L2W3	(2) 600S162-54	5.72	142.78	0.985	19846	77855	0.197	0.956	23348	86506	0.498
L2E1	(2) 600S162-54	5.72	142.78	0.986	19846	77855	0.175	0.956	23348	86506	0.498
L2E2	(2) 600S162-54	5.72	142.78	0.985	19846	77855	0.197	0.956	23348	86506	0.498
						Max	**0.235**			**Max**	**0.512**

Notes:
1. Factored $\Sigma C = 1.2D + E + L$, per ASCE 7-10 2.3.2 load combinations
2. Load combinations include $0.2S_{DS}$ term on dead load
3. Factored C is with dead and live loads factored per ASCE 7-10 12.4.3.2, including 0.5 factor for LL < 100 psf.
4. Where chords are also jambs, add'nl dead and live load are considered.
5. L2S2 considers 150 lb MEP weight assuming units ~ 1/3 pts of roof each side
6. P_{ui} = axial at inside face of stud, P_{ue} = axial load at outside face of stud.
7. Properties and capacities from AISIWIN v8 with K_yL_y and K_tL_t = 48"

Lower Level Shearwalls

SW	v (lb/ft)	h (ft)	C_{seis} (lb)	C_{DL} (lb)	C_{LL} (lb)	P_{ui} (lb)	P_{ue} (lb)	Factored ΣC_u	M_u (in-lb)	$\Omega_0 * C_{seis}$ (lb)	factored[3] C (lb)	Max Load SW Can Deliver v_n (lb/ft)	C_{max} (lb)	factored[3] C (lb)	factored[3] M (in-lb)
L1S1	408	9	5836	1672	2915	1882	3671	9610	5368	17508	**21282**	733	12200	**15974**	14155
L1S2	453	9	6489	1221	1485	500	4081	8923	10744	19466	**21900**	825	13725	**16159**	20774
L1S3	408	9	5836	1452	2585	1895	3671	9140	5326	17508	**20812**	733	12200	**15504**	14114
L1N1	295	9	4220	1492	1155	1180	2655	6865	4423	12661	**15306**	825	13725	**16369**	18734
L1N2	249	9	3561	2002	1155	1180	2240	6912	3178	10683	**14034**	825	13725	**17076**	18734
L1W1	326	9	4676	308	0	0	2937	5103	8810	14028	**14455**	733	12200	**12627**	19800
L1W2	326	9	4676	308	0	0	2937	5103	8810	14028	**14455**	733	12200	**12627**	19800
L1W3	417	9	5960	308	0	0	3754	6387	11263	17881	**18308**	825	13725	**14152**	22275
L1E1	369	9	5275	308	0	0	3318	5702	9955	15826	**16253**	825	13725	**14152**	22275
L1E2	415	9	5935	308	0	0	3733	6362	11199	17805	**18231**	825	13725	**14152**	22275
		Max	**6489**					**9610**	**11263**	19466	**21900**		**13725**	**17076**	**22275**

Lower Level Shearwalls - Interactions

SW	Proposed Chord	I_{xx} (in^4)	P_{Ex} (kips)	LRFD Check α_x	ϕP_n (lb)	ϕM_{nx} (in-lb)	Int'xn (C5.2.2-1)	Strength Check α_x	P_n (lb)	M_{nx} (in-lb)	Int'xn (C5.2.2-1)
L1S1	(2) 600S162-54	5.72	142.78	0.933	19846	77855	0.558	0.888	23348	86506	0.868
L1S2	(2) 600S162-54	5.72	142.78	0.938	19846	77855	0.597	0.887	23348	86506	0.963
L1S3	(2) 600S162-54	5.72	142.78	0.936	19846	77855	0.534	0.891	23348	86506	0.847
L1N1	(2) 600S162-54	5.72	142.78	0.952	19846	77855	0.406	0.885	23348	86506	0.946
L1N2	(2) 600S162-54	5.72	142.78	0.952	19846	77855	0.391	0.880	23348	86506	0.977
L1W1	(2) 600S162-54	5.72	142.78	0.964	19846	77855	0.374	0.912	23348	86506	0.792
L1W2	(2) 600S162-54	5.72	142.78	0.964	19846	77855	0.374	0.912	23348	86506	0.792
L1W3	(2) 600S162-54	5.72	142.78	0.955	19846	77855	0.473	0.901	23348	86506	0.892
L1E1	(2) 600S162-54	5.72	142.78	0.960	19846	77855	0.421	0.901	23348	86506	0.892
L1E2	(2) 600S162-54	5.72	142.78	0.955	19846	77855	0.471	0.901	23348	86506	0.892
						Max	**0.597**			**Max**	**0.977**

Notes:
1. Factored $\Sigma C = 1.2D + E + L$, per ASCE 7-10 2.3.2 load combinations
2. DL and LL include DL and LL from Upper Level, including wall Dead Load
3. Load combinations include $0.2S_{DS}$ term on dead load
4. Factored C is with dead and live loads factored per ASCE 7-10 12.4.3.2, including 0.5 factor for LL < 100 psf.
5. Where chords are also jambs, add'nl dead and live load are considered.
6. L2S2 considers 150 lb MEP weight assuming units ~ 1/3 pts of roof each side
7. Properties and capacities from AISIWIN v8 with K_yL_y and K_tL_t = 48"

Ties and HoldDowns

Ω_0 = 3.0

S_{DS} = 0.927

Ties - Upper to Lower Level — **Tension at End Ties**

SW	v (lb/ft)	h (ft)	C_{seis} (lb)	Roof DL (lb/ft)	Wall DL (lb/ft)	T_{seis} (lb)	Factored DL (lb)	T_{net} (lb)	$\Omega_0 * T_{seis}$ (lb)	factored[2] T_{net} (lb)	Max Load SW Can Deliver v_n (lb/ft)	T_{max} (lb)	factored[2] T_{net} (lb)
L2S1	241	9	2165	220	92	2165	446	1719	6495	**6050**	622	5600	**5154**
L2S2	267	9	2407	220	92	2407	557	1850	7222	**6665**	700	6300	**6651**
L2S3	241	9	2165	220	92	2165	446	1719	6495	**6050**	622	5600	**5951**
L2N1	174	9	1566	220	92	1566	1338	228	4697	**3359**	700	6300	**6651**
L2N2	147	9	1321	220	92	1321	892	429	3963	**3071**	700	6300	**6651**
L2W1	193	9	1739	20	92	1739	160	1579	5218	5058	622	5600	5674
L2W2	193	9	1739	20	92	1739	160	1579	5218	5058	622	5600	5674
L2W3	245	9	2206	20	92	2206	280	1926	6618	6338	700	6300	6374
L2E1	217	9	1957	20	92	1957	240	1717	5872	5631	700	6300	6374
L2E2	245	9	2202	20	92	2202	320	1882	6606	6285	700	6300	6374
		Max	**2407**					**1926**		**6665**			**6651**

SW	LRFD $T_{u\text{-}net}$ (lb)	Strap Tension F_y (ksi)	W (in)	t (in)	ϕT_n (lb)	$\phi T_{fract\text{-}net}$	$T_u / \phi T_n$	KL (in)	r (in)	KL/r	F_e (ksi)	Strap Compression λ_c	F_n (ksi)	P_n (lb)	ϕP_n (lb)	$\Sigma C_u / \phi P_n$	factored C_{max}/P_n
L2S1	1719	50	4.00	0.1017	18306	17948	**0.096**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.443**	**0.734**
L2S2	1850	50	4.00	0.1017	18306	17948	**0.103**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.498**	**0.829**
L2S3	1719	50	4.00	0.1017	18306	17948	**0.096**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.386**	**0.685**
L2N1	228	50	4.00	0.1017	18306	17948	**0.013**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.319**	**0.597**
L2N2	429	50	4.00	0.1017	18306	17948	**0.024**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.376**	**0.594**
L2W1	1579	50	4.00	0.1017	18306	17948	**0.088**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.213**	**0.543**
L2W2	1579	50	4.00	0.1017	18306	17948	**0.088**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.213**	**0.543**
L2W3	1926	50	4.00	0.1017	18306	17948	**0.107**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.270**	**0.656**
L2E1	1717	50	4.00	0.1017	18306	17948	**0.096**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.240**	**0.611**
L2E2	1882	50	4.00	0.1017	18306	17948	**0.105**	3.00	0.0294	102	27.88	1.34	23.61	9603	8162	**0.270**	**0.656**

SW	V_n (lb/scr)	Fasteners # Scr (LRFD)	# Scr (Ω_0)
L2S1	1520	**4.8**	**4.6**
L2S2	1520	**5.4**	**5.2**
L2S3	1520	**4.1**	**4.3**
L2N1	1520	**3.4**	**3.8**
L2N2	1520	**4.0**	**3.8**
L2W1	1520	**2.3**	**3.4**
L2W2	1520	**2.3**	**3.4**
L2W3	1520	**2.9**	**4.1**
L2E1	1520	**2.6**	**3.9**
L2E2	1520	**2.9**	**4.1**

Notes:
1. Factored DL = (0.9 - 0.2S_{DS})*DL*(SW Length)/2 : Per ASCE 7-10 12.4.2.3
2. Factored T_{net} is amplified T_{seis} less factored DL (DL factored as noted above)
3. For compression, applied loads come from Chord Force Analysis
4. Factored Cmax/Pn is based on the amplified or maximum load SW can deliver chord forces
5. V_n for screws conservatively taken as 3.75 x the screw shear ultimate (based on minimum from several mfrs). V_n for the assembly per NASPEC E4 is > listed value
6. ϕ for screw connections = 0.5 per NASPEC E4
7. Values are conservatively based on Type I shearwalls.
8. Fracture on net section taken through width less two screw diameters. Fu = 65 ksi for Fy = 50 ksi; Fu = 45 ksi for Fy = 33 ksi

Lower Level Shearwalls

SW	v (lb/ft)	h (ft)	C_{seis} (lb)	DL (lb/ft)	Wall DL (lb/ft)	T_{seis} (lb)	Factored DL (lb)	T_{net}[2] (lb)	$\Omega_0 * T_{seis}$ (lb)	factored[3] T_{net} (lb)	Max Load SW Can Deliver v_n (lb/ft)	T_{max} (lb)	factored[3] T (lb)
L1S1	408	9	5836	198	81	3671	399	4991	11012	**16663**	733	6600	**11355**
L1S2	453	9	6489	198	81	4081	498	5433	12244	**18411**	825	7425	**13577**
L1S3	408	9	5836	198	81	3671	399	4991	11012	**16663**	733	6600	**12152**
L1N1	295	9	4220	198	81	2655	1196	1686	7964	**10127**	825	7425	**12879**
L1N2	249	9	3561	198	81	2240	798	1871	6719	**8993**	825	7425	**13278**
L1W1	326	9	4676	18	81	2937	142	4375	8810	13727	733	6600	12132
L1W2	326	9	4676	18	81	2937	142	4375	8810	13727	733	6600	12132
L1W3	417	9	5960	18	81	3754	248	5433	11263	17353	825	7425	13551
L1E1	369	9	5275	18	81	3318	212	4823	9955	15374	825	7425	13586
L1E2	415	9	5935	18	81	3733	283	5332	11199	17202	825	7425	13516
		Max	**6489**					**5433**		**18411**			**13586**

Lower Level Shearwalls

SW	Holddown	ϕT_n (lb)	$T_u/\phi T_n$	T_n (lb)	Factored T/Tn
L1S1	S/HDU6	9785	**0.510**	15005	**0.757**
L1S2	S/HDU6	9785	**0.555**	15005	**0.905**
L1S3	S/HDU6	9785	**0.510**	15005	**0.810**
L1N1	S/HDU6	9785	**0.172**	15005	**0.675**
L1N2	S/HDU6	9785	**0.191**	15005	**0.599**
L1W1	S/HDU6	9785	**0.447**	15005	**0.809**
L1W2	S/HDU6	9785	**0.447**	15005	**0.809**
L1W3	S/HDU6	9785	**0.555**	15005	**0.903**
L1E1	S/HDU6	9785	**0.493**	15005	**0.905**
L1E2	S/HDU6	9785	**0.545**	15005	**0.901**

Notes:
1. Factored DL = (0.9 - 0.2S_{DS})*DL*(Wall Length)/2 : Per ASCE 7-10 12.4.2.3
2. T_{net} includes T_{net} from Upper Level
3. Factored T_{net} is amplified T_{seis} less factored DL (DL factored as noted above). Includes factored T_{net} from Upper Level
4. Holddowns listed are by Simpson Strong-Tie with capacities provided by the manufacturer.

Roof Diaphragm

Total Roof Shear	**6524**	(lb)
Min Shear	6014	(lb) per ASCE 7-10 Eq. 12.10-2
Roof Width	49.75	(ft - long dimension)
Roof Depth	23.00	(ft- short dimension)
Max shear, v	142	(lb/ft)

Sheathing: Min 7/16" OSB, unblocked, No. 8 screws 6" oc edges and 12" oc field.

v_n =	565	(lb/ft)	per AISI S213 Table D2-1
ϕv_n =	339	(lb/ft)	**OK**

Max diaphragm drag force to shearwall: **2088** (lb) - not amplified

Chord Forces:

Equivalent uniform lateral load (N-S controls) =	131	(lb/ft)
Max 'beam' moment	40571	(Ft-lb)
Max chord forces, C_u/T_u	**1764**	(lb) - not amplified

Rim Track: 1200T200-68 KL = 24" ϕP_n = 6724 (lb) **OK**

2nd Floor Diaphragm

Total Roof Shear	4537	(lb)
Min Shear	6434	(lb) per ASCE 7-10 Eq. 12.10-1
Min Shear	**8365**	(lb) per ASCE 7-10 Eq. 12.10-2

Min End Dimensions: Short Sides 19.5 (ft)
Long Sides 34.75 (ft)

Max shear, v 214 (lb/ft)

Sheathing: Min 23/32 CD-CC Structural Panels, unblocked, No. 8 screws 6" oc edges and 12" oc field.

v_n = 555 (lb/ft) per AISI S213 Table D2-1

ϕv_n = 333 (lb/ft) **OK**

Max diaphragm drag force to shearwall: **3539** (lb) - not amplified

Chord Forces:

Equivalent uniform lateral load (N-S) = 168 (lb/ft)
Equivalent uniform lateral load (E-W) = 364 (lb/ft)
Max 'beam' moment 52019 (Ft-lb)
Max chord forces, C_u/T_u **2262** (lb) - not amplified

Rim Track: 1200T200-97 KL = 24" ϕP_n = 12289 (lb) **OK**

Shear at Edges of Openings

Case 1: Short direction, forces N-S V(x) = (8365/2)-168*x

Location	x (ft)	V (lb)	Shear Length (ft)	v (lb/ft)
Edge of Exit Stair	15	1660	19.5	85
West Clerestory Edge	31	1030	13.25	78
East Clerestory Edge	38.75	2333	13.25	176

Case 2: Long direction, forces E-W V(y) = (8365/2)-364*y

Location	y (ft)	V (lb)	Shear Length (ft)	v (lb/ft)
Edge of Exit Stair	3.5	3594	34.75	103
North Clerestory Edge	7.75	2879	42	69
East Clerestory Edge	17.5	1240	42	30

Appendix 4

CFS-NEES
Lateral System Design
Supplemental Calculations

April 11, 2012

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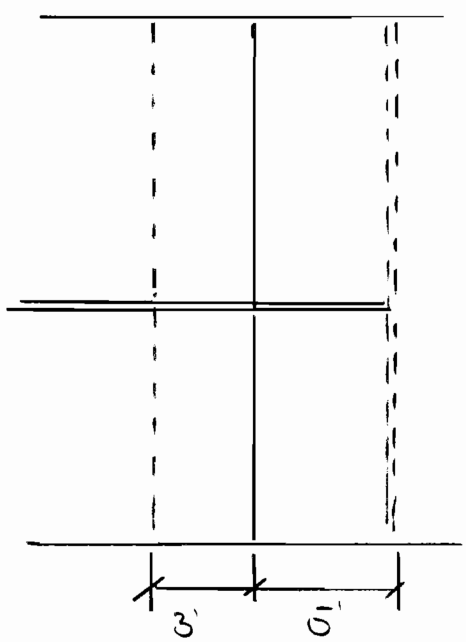
PROJECT: CFS-NEES | PROJECT NO: 10-277 | DESIGN: RLM | DATE: Apr '11

North Shearwall Chords @ Balloon Framing

Shearwall L2N2 + L1N2.



Check Level 1 Top Track

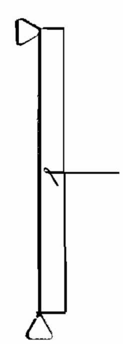
$w = 15(9) = 135$ lb/ft

Span = 5'

(1) ∴ 600T150 54 ok - Do Not splice w/in length of shearwall

600T150 43 B.O. Lvl 2

Clerestory 'Jamb'



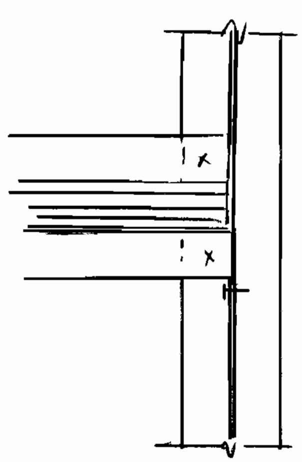
$H = 18'$; $w = 15$ lb/ft

$P = 338$ lb @ 9' AFF

$C \approx 440$ lb

(2) ∴ 600S162-54 Full Ht ok

Conn @ Transition



$V = 338$ lb

∴ Use (2) #10 Upper + Lower to Full Ht

(Note: to keep Top + Bott Track Cont- use strip of floor sheathing Btwn Top + Bott Track

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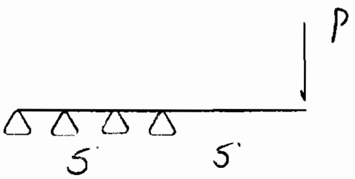
PROJECT: CFS-NEES	PROJECT NO: 10-277	DESIGN: RLM	DATE: MC5'11

Check Chord Capacity

Check Cantilever Track for $K_x L_x$ bracing

$P_{br} = 0.01 P_n$ ∴ $P_n = \frac{19846}{0.85} = 23{,}350$ lb (3)

∴ $P_{br} = 233$ lb



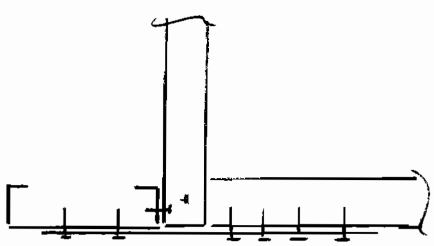
$\beta_{br} = \frac{2(4 - 2/1)}{108}$ 23.35 $= 0.865$ k/in

@ $P = 233$ lb $\Delta = \frac{L}{703} = 0.171"$

∴ $\beta = \frac{233}{0.171} = 1.365$ k/in ok

∴ 600T150-54 / 600T150-43 Combined ok for strength + stiffness

Joist Tie @ edge of Clerestory



$T_x = 1121$ lb (brace force)

\# SCS = 1121/405 = 2.8

∴ Use Min 1½" x 54-mil Strap x 1'0" - (3) #10 to stud + to Joist

Chord

For chord @ $K_y L_y$ = 9', see Shearwall analysis + design spread sheet for Typ chord design

2007 North American Specification ASD
DATE: 4/1/2011
CFS-NEES

1

SECTION DESIGNATION: 600T150-54 [50] Single

Input Properties:

Web Height =	6.198 in	Design Thickness =	0.0566 in
Top Flange =	1.500 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.500 in	Yield Point, Fy =	50.0 ksi
		Fy With Cold-Work, Fya =	50.0 ksi

Header/Beam Solver Design Data - Simple Span

Header/Beam Span 5.00 ft — Deflection Limit L/360
Dead Load = .0 lb/ft — DL Multiplied by 1.00 for Strength Checks
Wind Load = 135.0 lb/ft — WL Multiplied by 1.00 for Strength Checks
WL Multiplied by 0.70 for Deflection Checks

Check Flexure

Flexural Bracing: Full
Mmax = 422 Ft-Lb <= Ma = 1520 Ft-Lb & Ma(distortional) = 1520 Ft-Lb
K-phi for Distortional Buckling = 0 lb*in/in

Check Deflection

Deflection Limit: L/360
Maximum Deflection = 0.019 in — Deflection Ratio = L/3207

Check Shear

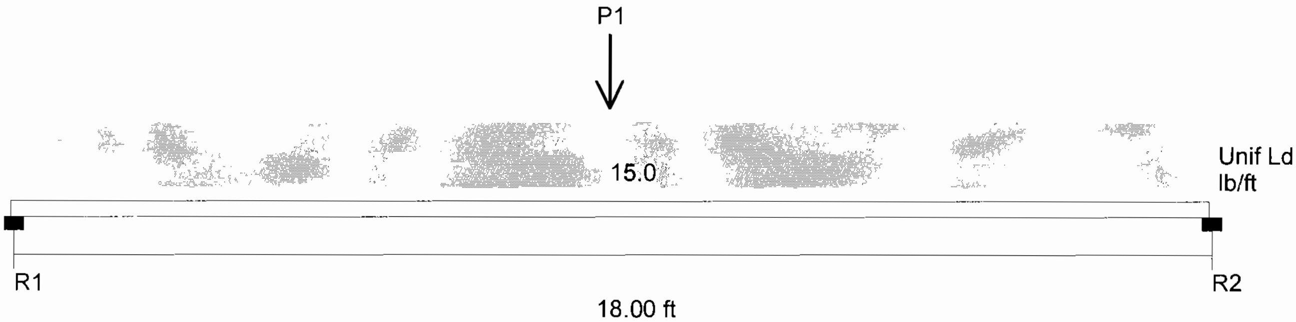
Vmax = 338 lb (Including Flexural Load Multiplier)
Shear capacity not reduced for punchouts near ends of member
Va = 2728 lb >= Vmax

Check Web Crippling

Rmax = 338 lb (Including Flexural Load Multiplier)
Web Crippling capacity not reduced for punchouts near ends of member
End Bearing Length = 1.00 in
Ra = 443 lb >= Rmax, stiffeners not required

2001 NASPEC w/2004 Supplement

Project: CFS-NEES **Date:** 4/1/2011
Model: North Clerestory 'Jamb'



Point Loads	**P1**
Load(lb)	338
X-Dist.(ft)	9.00

Section : 600S162-54 Single C Stud (X-X Axis) **Fy =** 50.0 ksi
Maxo = 2527.1 Ft-Lb **Moment of Inertia, I =** 2.860 in^4 **Va =** 2822.9 lb

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	2128.5	0.842	2128.5	Full	2527.1	0.842	0.883	L/245

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	304.0	1.00	598.9	1048.1	0.0	0.26	No
R2	304.0	1.00	598.9	1048.1	0.0	0.26	No
P1	338.0	1.50	1403.1	2315.1	2128.5	0.64	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	304.0	0.0	1.00	0.11	0.00	0.01	NA
R2	304.0	0.0	1.00	0.11	0.00	0.01	NA
P1	169.3	2128.5	1.00	0.06	0.84	0.71	NA

Combined Bending and Axial Load

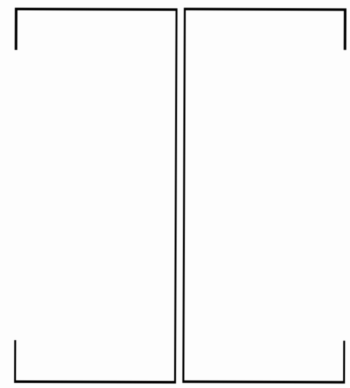
Span	Axial Ld (lb)	Bracing (in) KyLy	Bracing (in) KtLt	Max KL/r	Allow Ld (lb)	P/Pa	Intr. Value
Center Span	440.0 (c)	60.0	60.0	105	4007.3 (c)	0.11	0.95

2007 North American Specification LRFD
DATE: 4/1/2011
CFS-NEES

SECTION DESIGNATION: 600S162-54 [50] (2) Back-to-Back

Section Dimensions:

Web Height =	6.000 in
Top Flange =	1.625 in
Bottom Flange =	1.625 in
Stiffening Lip =	0.500 in
Inside Corner Radius =	0.0849 in
Punchout Width =	1.500 in
Punchout Length =	4.000 in
Design Thickness =	0.0566 in



Steel Properties:

Fy =	50.000 ksi
Fu =	65.000 ksi
Fya =	55.318 ksi

MAXIMUM FACTORED AXIAL LOADS, Pu

INPUT PARAMETERS

Overall Stud Length = 9 ft
Member Configuration: (2) BACK-TO-BACK MEMBERS

K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

TOTAL FACTORED AXIAL LOADS, Pu (lb)

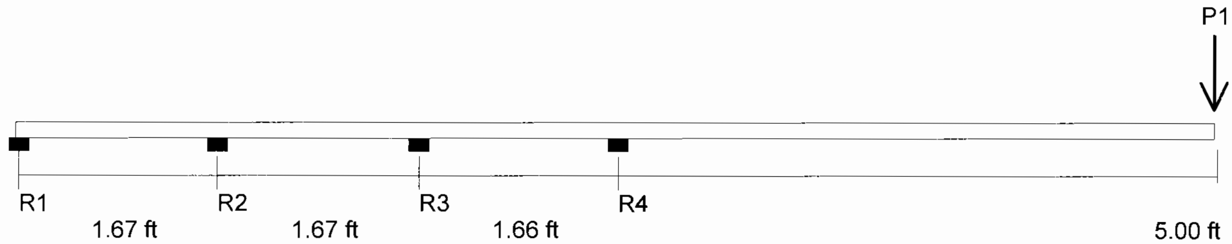
WEAK AXIS BRACING	MAXIMUM KL/r	CONCENTRIC LOADING
48 in	71	19846
MID Pt	80	18383
THIRD Pt	55	22528

Note: For (2) Back-to-Back Members, Individual Members
Must be Adequately Interconnected

4

2001 NASPEC w/2004 Supplement

Project: CFS-NEES **Date:** 4/1/2011
Model: North Clerestory - Track as Bracing for Chord



Point Loads	**P1**
Load(lb)	233
X-Dist.(ft)	10.00

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Built-Up Section:

Section Number	Section	Ixx (in^4)	% of Total Ixx	Area (in^2)	% of Total Area
1	600T150-54 (50)	2.400	55.9%	0.509	55.7%
2	600T150-43 (33)	1.890	44.1%	0.405	44.3%

Overall Member Inputs:

Span	Flexural Bracing (in)	Load (lb)	Axial KyLy (in)	KtLt (in)
Left Span	Full	0	N.A.	N.A.
Center Span	Full	0	N.A.	N.A.
Right Span	Full	0	N.A.	N.A.
Right Cant.	Full	0	N.A.	N.A.

Reaction and Point Load Data:

	R1	R2	R3	R4	P1
Load (lb)	-46.4	278.3	-1120.3	1121.4	233.0
Brng (in)	1.00	1.00	1.00	1.00	1.50

Analysis Summary:

	Flexure		Web Crippling	Shear & Bending			Axial	
Section	**Defl.**	**M/Ma**	**Stiffen Req'd**	**V/Va**	**Unstiffened**	**Stiffened**	**P/Pa**	**Combined**
600T150-54 (50)	L/703	0.429	No	0.18	0.22	0.00	0.00	0.00
600T150-43 (33)	L/703	0.658	YES	0.28	0.51	0.00	0.00	0.00

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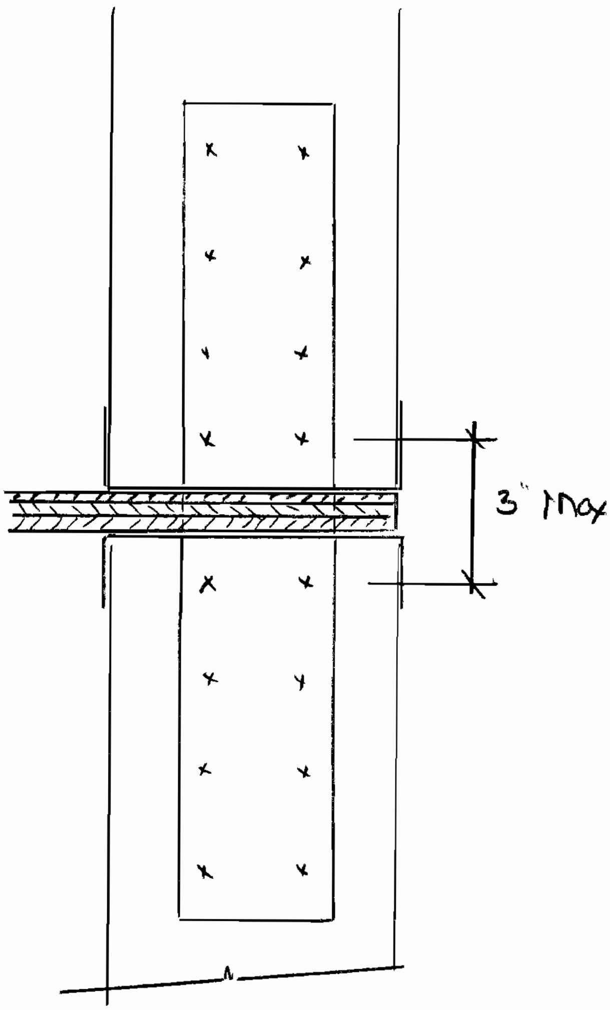
PROJECT: CFS-NEES | PROJECT NO: 10-277 | DESIGN: RLM | DATE: Apr '11

Shearwall Ties - Level 2-1



Max Load System Can Deliver:

Compression Controls

See Spreadsheet for strap + fastener Design

Note: Fasteners also to meet Requirements of NASPEC D1.2

Level 1 Hold-downs

- T_u max (max force system can deliver)
- see spreadsheet for Hold-downs

5/8" ϕ A325 Bolt $\phi T_n = 20.7^k$ ok

(1) ∴ Use Hold-down per spreadsheet + 5/8" ϕ A325 Bolts

SIMPSON

S/HDU *Holdowns*

The S/HDU series of holdowns combines performance with ease of installation. The pre-deflected geometry virtually eliminates material stretch, resulting in low deflection under load. Installation using self-drilling tapping screws into the studs reduces installation time and saves labor cost.

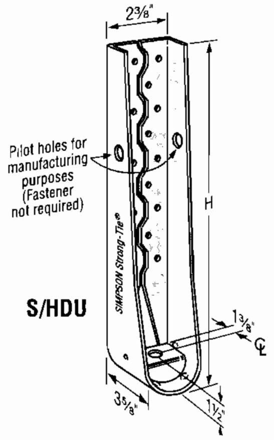
MATERIAL: 118 mil (10 ga)

FINISH: Galvanized

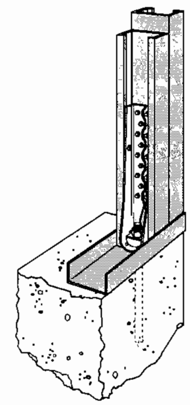
INSTALLATION: • Use all specified fasteners. See General Notes.

• Use #14 screws to fasten to studs

CODES: See page 8 for Code Listing Key Chart.



S/HDU



Typical S/HDU Installation

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Model	H	Fasteners: Fdn Anchor Dia [1]	Fasteners: Stud Fasteners	Stud Member Thickness [4]	ASD: Tension Load	ASD: Deflection at ASD Load [7]	LRFD: Tension Load	LRFD: Deflection at LRFD Load [7]	Nominal Tension Load [8]	Code Ref.
S/HDU4	7⅞	⅝	6-#14	2-33 (2-20ga)	2320	0.093	3705	0.149	5685	FC1
				2-43 (2-18ga)	3825	0.115	6105	0.190	9365	
				2-54 (2-16ga)	3970	0.093	6345	0.156	9730	
				Steel Fixture	4470	0.063	7165	0.103	12120	
S/HDU6	10⅜	⅝	12-#14	2-33 (2-20ga)	4895	0.125	8495	0.250	10470	
				2-43 (2-18ga)	6125	0.119	9690	0.250	15460	
				2-54 (2-16ga)	6125	0.108	9785	0.234	15005	
				Steel Fixture	5995	0.060	9580	0.136	14695	
S/HDU9	12⅞	⅞	18-#14	2-33 (2-20ga)	6965	0.103	11125	0.189	13165	
				2-43 (2-18ga)	9255	0.125	15485	0.250	21810	
				2-54 (2-16ga)	9990	0.106	15960	0.225	24480	
				Steel Fixture	12715	0.125	20510	0.177	31455	
S/HDU11	16⅝	⅞	27-#14	2-33 (2-20ga)	6965	0.103	11125	0.189	13165	
				2-43 (2-18ga)	9595	0.096	15330	0.162	23515	
				2-54 (2-16ga)	9675	0.110	15460	0.158	23710	
		⅞ w/ heavy hex nut	27-#14	2-43 (2-18ga)[6]	11100	0.125	17500	0.250	24955	
				2-54 (2-16ga)[6]	12175	0.125	19445	0.243	29825	
				Steel Fixture[6]	12945	0.111	20680	0.163	31715	

1. Designer shall specify the foundation anchor material type, length, embedment and configuration. Tabulated loads may exceed anchor bolt ASTM A36 or A307 tension capacities.
2. See pages 26–30 for anchor bolt options.
3. See page 21 for anchor bolt retrofit options.
4. Stud design by Specifier. Tabulated loads are based on a minimum studs thickness for fastener connection.
5. ¼" self-drilling tapping screws can be substituted for #14.
6. Heavy hex nut is required to achieve the table loads for S/HDU11.
7. Deflection at ASD and LRFD Loads includes fastener slip, holdown elongation and anchor bolt elongation (L=4").
8. Nominal Tension Load is based on the average ultimate (peak) load from tests. AISI Lateral Design standard requires holdown to have nominal strength to resist lesser of amplified seismic load or the maximum force the system can deliver.

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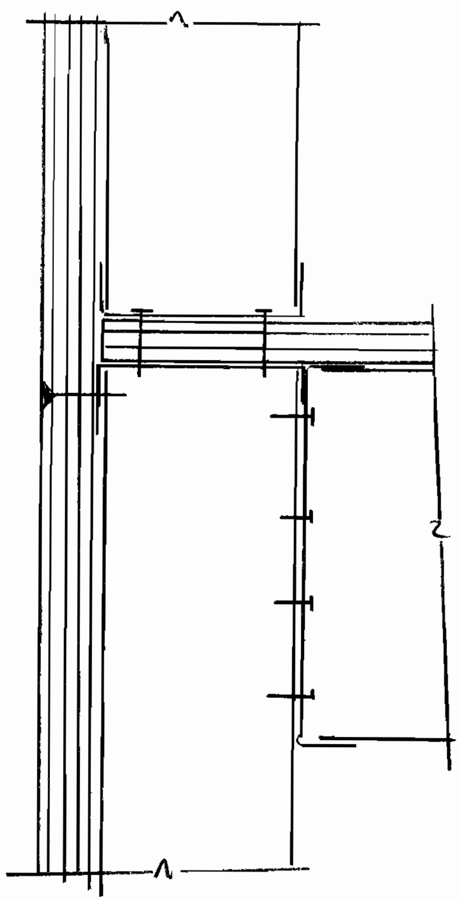
Shear Wall Shear Anchors

Level 2 - Level 1



$v \le 450$ lb/ft

∴ Use sheathing edge fasteners @ Level 1 Top Track

Level 1 - Fdn $v \le 550$ lb/ft (LRFD level)
$= 393$ lb/ft (ASD level)

1/4" ϕ self-Drilling Screw. $\phi V_n = 1242$ lb
$V_A = 828$ lb

1 Nominal screw strength (Hilti Kwik-Pro)
$P_n = 2440$ lb ∴ $P_A = \frac{2440}{1.25 \cdot (3)} = 650$ lb/ea

∴ Use 1/4" self-Drilling Screw @ 12" oc - Hilti Kwik Pro or equiv

ICC Evaluation Service, Inc.
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Business/Regional Office ■ 5360 Workman Mill Road, Whittier, California 90601 ■ (562) 699-0543
Regional Office ■ 900 Montclair Road, Suite A, Birmingham, Alabama 35213 ■ (205) 599-9800
Regional Office ■ 4051 West Flossmoor Road, Country Club Hills, Illinois 60478 ■ (708) 799-2305

DIVISION: 05—METALS
Section: 05090—Metal Fastenings

REPORT HOLDER:

HILTI, INC.
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TULSA, OKLAHOMA 74146
(800) 879-8000
www.us.hilti.com

EVALUATION SUBJECT:

HILTI KWIK-PRO SELF-DRILLING SCREWS

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2006 *International Building Code®* (IBC)
- 1997 *Uniform Building Code™* (UBC)

Property evaluated:

Structural

2.0 USES

The Hilti Kwik-Pro Self-drilling Screws are used to connect cold-formed steel members to cold-formed steel members.

3.0 DESCRIPTION

The Hilti Kwik-Pro Self-drilling Screws are self-drilling tapping screws complying with ASTM C 1513, and are case-hardened from carbon steel conforming to ASTM A 510, Grade 1022. The screws have a hex washer head and have an electroplated zinc coating complying with ASTM F 1941, or a proprietary coating. Table 1 provides screw designations, sizes and descriptions of point styles. Screws are supplied in boxes of individual screws, or in collated plastic strips.

4.0 DESIGN AND INSTALLATION

4.1 Design:

Allowable fastener loads using Allowable Stress Design (ASD) for pull-out, pull-over, and shear (bearing) capacity are provided in Tables 2, 3 and 5, respectively. Instructions on how to calculate Load Resistance Factor Design (LRFD) capacities are found in the footnotes of these tables. Table 4 presents the nominal and allowable fastener tension and shear strengths for the screws. For connections subject to tension, the least of the allowable pull-out, pullover, and tension fastener strength of screws found in Tables 2, 3, and 4, respectively, must be used for design. For connections subject to shear, the lesser of the allowable shear fastener strength and shear (bearing) found in Tables 4 and 5, respectively, must be used for design. Connections subject to combined tension and shear loading must be designed in accordance with Section E4.5 of the AISI – NAS.

The values in the tables are based on a minimum spacing between the centers of fasteners of three times the diameter of the screw, and a minimum distance from the center of a fastener to the edge of any connected part as follows:

1. In jurisdictions adopting the IBC: 1.5 times the diameter of the screw. When the distance to the end of the connected part is parallel to the line of the applied force, the allowable shear fastener strength determined in accordance with Section E4.3.2 of Appendix A of the AISI – NAS must be considered.
2. In jurisdictions adopting the UBC: three times the diameter of the screw. If the connection is subjected to shear force in one direction only, the minimum edge distance must be 1.5 times the diameter of the screw in the direction perpendicular to the force.

Screw thread length and point style are to be selected on the basis of thickness of the fastened material and thickness of the supporting steel, respectively, in accordance with the manufacturer's published installation instructions.

4.2 Installation:

Installation of the Hilti Kwik-Pro Self-drilling Screws must be in accordance with the manufacturer's published installation instructions and this report. The manufacturer's published installation instructions are to be available at the jobsite at all times during installation.

The screws must be installed perpendicular to the work surface using a variable speed screw gun set to not exceed 2,500 rpm. The screw must penetrate through the supporting steel with a minimum of three threads protruding past the back side of the supporting steel.

5.0 CONDITIONS OF USE

The Hilti Kwik-Pro Self-drilling Screws described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Fasteners are to be installed in accordance with the manufacturer's published installation instructions and this report. If there is a conflict between the manufacturer's published installation instructions and this report, this report governs.

5.2 The allowable loads specified in Section 4.1 are not to be increased when the fasteners are used to resist wind or seismic forces.

5.3 The utilization of the nominal strength values contained in this evaluation report, for the design of cold-formed steel diaphragms, is outside the scope of this report.

***Corrected May 2007**

5.4 Drawings and calculations verifying compliance with this report and the applicable code must be submitted to the code official for approval. The drawings and calculations are to be prepared by a registered design professional when required by the statutes of the jurisdiction in which the project is to be constructed.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Tapping Screw Fasteners (AC118), dated December 2006.

7.0 IDENTIFICATION

Hilti Kwik-Pro Self-drilling Screws are marked with an "H" on the top of the heads, as shown in Figure 1. Packages of Hilti Self-drilling Screws are labeled with the report holder's name (Hilti, Inc.), the fastener type and size, and the evaluation report number (ESR-2196).

TABLE 1—HILTI KWIK-PRO SELF-DRILLING TAPPING SCREWS

Description	Designation	Nominal Diameter (in.)	Nominal Screw Length (in.)	Head Style[1]	Point (Number)	Coating[2]
S-MD 10-16 X 5/8 HWH #3	#10-16	0.190	5/8	HWH	3	Zinc
S-MD 10-16 X 3/4 HWH #3	#10-16	0.190	3/4	HWH	3	Zinc
S-MD 10-16 X 3/4 HHWH #3	#10-16	0.190	3/4	HHWH	3	Zinc
S-MD 10-16 X 1 HWH #3	#10-16	0.190	1	HWH	3	Zinc
S-MD 10-16 X 1-1/4 HWH #3	#10-16	0.190	1-1/4	HWH	3	Zinc
S-MD 10-16 X 1/1/2 HWH #3	#10-16	0.190	1-1/2	HWH	3	Zinc
S-MD 12-14X3/4 HWH #3	#12-14	0.216	3/4	HWH	3	Zinc
S-MD 12-14 X 1 HWH #3	#12-14	0.216	1	HWH	3	Zinc
S-MD 12-14 X 1 1/2 HWH #3	#12-14	0.216	1-1/2	HWH	3	Zinc
S-MD 12-14 X 2 HWH #3	#12-14	0.216	2	HWH	3	Zinc
S-MD 1/4-14 X 3/4 HWH #3	1/4-14	0.250	3/4	HWH	3	Zinc
S-MD1/4-14 X 1 HWH #3	1/4-14	0.250	1	HWH	3	Zinc
S-MD 1/4-14 X 1-1/2 HWH #3	1/4-14	0.250	1-1/2	HWH	3	Zinc
S-MD 1/4-14 X 2 HWH #3	1/4-14	0.250	2	HWH	3	Zinc
S-MD 12-24 X 7/8 HWH #4	#12-24	0.216	7/8	HWH	4	Zinc
S-MD 12-24 X 1-1/4 HWH #4	#12-24	0.216	1-1/4	HWH	4	Zinc
S-MD 12-24 X 1-1/4 HWH #5	#12-24	0.216	1-1/4	HWH	5	Zinc
S-MD 12-24 X 1-1/4 HWH #5 Kwik Cote	#12-24	0.216	1-1/4	HWH	5	Kwik-Cote
S-MD 12-24 X 2 HWH #5 Kwik Cote	#12-24	0.216	2	HWH	5	Kwik-Cote
S-MD 12-24 X 3 HWH #5 Kwik Cote	#12-24	0.216	3	HWH	5	Kwik-Cote
S-MD 10-16 X 7/8 M HWH Collated	#10-16	0.190	7/8	HWH	1	Zinc
S-MD 12-14 X 1 M HWH Collated	#12-14	0.216	1	HWH	1	Zinc
S-MD 10-16 X 3/4 M HWH3 Collated	#10-16	0.190	3/4	HWH	3	Zinc
S-MD 12-24 X 7/8 M HWH4 Collated	#12-24	0.216	7/8	HWH	4	Zinc
S-MD 10-16 X 7/8 HWH Pilot Point	#10-16	0.190	7/8	HWH	1	Zinc
S-MD 12-14 X 1 HWH Stitch	#12-14	0.216	1	HWH	1	Zinc
S-MD 1/4-14 X 7/8 HWH Stitch Kwik Seal	1/4-14	0.250	7/8	HWH	1	Kwik-Cote
S-MD 8-18 X 1/2 HWH #2	#8-18	0.164	1/2	HWH	2	Zinc
S-MD 8-18 X 3/4 HWH #2	#8-18	0.164	3/4	HWH	2	Zinc
S-MD 10-16 X 1/2 HWH #2	#10-16	0.190	1/2	HWH	2	Zinc
S-MD 10-16 X 3/4 HWH #2	#10-16	0.190	3/4	HWH	2	Zinc
S-MD 10-16 X 1 HWH #2	#10-16	0.190	1	HWH	2	Zinc
S-MD 12-14 x 3/4 HWH #3 Kwik Seal	#12-14	0.216	3/4	HWH	3	Kwik-Cote
S-MD 12-14 x 1 HWH #3 Kwik Seal	#12-14	0.216	1	HWH	3	Kwik-Cote
S-MD 12-14 X 1-1/4 HWH #3 Kwik Seal	#12-14	0.216	1-1/4	HWH	3	Kwik-Cote
S-MD 12-14 X 1 -1/2 HWH #3 Kwik Seal	#12-14	0.216	1-1/2	HWH	3	Kwik-Cote
S-MD 12-14 X 2 HWH #3 Kwik Seal	#12-14	0.216	2	HWH	3	Kwik-Cote
S-MD 1/4-14 X 3/4 HWH #3 Kwik Seal	1/4-14	0.250	3/4	HWH	3	Kwik-Cote
S-MD 1/4-14 x 1 HWH #3 Kwik Seal	1/4-14	0.250	1	HWH	3	Kwik-Cote
S-MD 1/4-14 X 1-1/2 HWH #3 Kwik Seal	1/4-14	0.250	1-1/2	HWH	3	Kwik-Cote

For **SI**: 1 inch = 25.4 mm.

[1]Head configuration abbreviations are as follows; HWH = Hex Washer Head. HHWH = High Hex Washer Head.
[2]For coating, Zinc = ASTM F 1941; Kwik-Cote = Proprietary coating.

TABLE 2—ALLOWABLE TENSILE PULL-OUT LOADS (P_{NOT}/Ω), pounds-force [1, 2, 3, 4, 5]

Steel F_u = 45 ksi Applied Factor of Safety, Ω = 3.0								
Screw Designation	Nominal Diameter (in.)	Design thickness of member not in contact with the screw head (in.)						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#8-18	0.164	75	100	125	157	188	220	282
#10-16	0.190	87	116	145	182	218	254	327
#12-14, #12-24	0.216	99	132	165	207	248	289	373
1/4-14	0.250	115	153	191	239	287	333	430

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]For tension connections, the lower of the allowable pull-out, pullover, and tension fastener strength of screw found in Tables 2, 3, and 4, respectively must be used for design.
[2]ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.
[3]The allowable pull-out capacity for other member thicknesses can be determined by interpolating within the table.
[4]To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
[5]For F_u = 65 ksi steel, multiply values by 1.44.

TABLE 3—ALLOWABLE TENSILE PULL-OVER LOADS (P_{NOV}/Ω), pounds-force [1, 2, 3, 4, 5]

Steel F_u = 45 ksi Applied Factor of Safety, Ω = 3.0									
Screw Designation	Washer Head Diameter (in.)	Design thickness of member in contact with the screw head (in.)							
		0.030	0.036	0.048	0.060	0.075	0.090	0.105	0.135
#8-18	0.335	225	271	363	453	567	680	790	1020
#10-16	0.399	268	323	430	540	673	807	943	1210
#12-14, #12-24	0.415	279	337	447	560	700	840	980	1260
1/4-14	0.500	336	407	540	677	843	1010	1180	1520

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]For tension connections, the lower of the allowable pull-out, pullover, and tension fastener strength of screw found in Tables 2, 3, and 4, respectively must be used for design.
[2]ANSI/ASME standard screw head diameters were used in the calculations and are listed in the tables.
[3]The allowable pull-over capacity for other member thicknesses can be determined by interpolating within the table.
[4]To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
[5]For F_u = 65 ksi steel, multiply values by 1.44.

TABLE 4—FASTENER STRENGTH OF SCREW

Screw Designation	Diameter (in.)	Allowable Fastener Strength[4]		Nominal Fastener Strength (tested)	
		Tension (P_{ts}/Ω)[1] (lb)	Shear (P_{ss}/Ω)[2,3] (lb)	Tension, P_{ts} (lb)	Shear, P_{ss} (lb)
#8-18	0.164	335	390	1000	1170
#10-16	0.190	455	405	1370	1215
#12-14	0.216	775	625	2325	1880
#12-24	0.216	1300	760	3900	2285
1/4-14	0.250	1525	815	4580	2440

For **SI**: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]For tension connections, the lower of the allowable pull-out, pullover, and tension fastener strength of screw found in Tables 2, 3, and 4, respectively must be used for design.
[2]For shear connections, the lower of the allowable shear fastener strength and allowable shear (bearing) found in Tables 4 and 5, respectively must be used for design.
[3]See Section 4.1 for fastener spacing and end distance requirements.
[4]To calculate LRFD values, multiply the allowable fastener strengths by the ASD safety factor of 3.0 and multiply again by the LRFD Φ factor of 0.5.

TABLE 5—ALLOWABLE SHEAR (BEARING) CAPACITY OF COLD-FORMED STEEL, lb [1, 2, 3, 4, 5]

Steel F_u = 45 ksi Applied Factor of Safety, Ω = 3.0									
Screw Designation	Diameter (in.)	Design thickness of member in contact with screw head, (in.)	Design thickness of member not in contact with the screw head (in.)						
			0.036	0.048	0.060	0.075	0.090	0.105	0.135
#8	0.164	0.036	174	239	239	239	239	239	239
		0.048	174	268	319	319	319	319	319
		0.060	174	268	373	400	400	400	400
		0.075	174	268	373	497	497	497	497
		0.090	174	268	373	497	597	597	597
		0.105	174	268	373	497	597	697	697
		0.135	174	268	373	497	597	697	897
#10	0.190	0.036	188	277	277	277	277	277	277
		0.048	188	289	370	370	370	370	370
		0.060	188	289	403	463	463	463	463
		0.075	188	289	403	563	577	577	577
		0.090	188	289	403	563	693	693	693
		0.105	188	289	403	563	693	807	807
		0.135	188	289	403	563	693	807	1040
#12	0.216	0.036	200	309	315	315	315	315	315
		0.048	200	308	420	420	420	420	420
		0.060	200	308	430	523	523	523	523
		0.075	200	308	430	600	657	657	657
		0.090	200	308	430	600	787	787	787
		0.105	200	308	430	600	787	920	920
		0.135	200	308	430	600	787	920	1180
1/4 in.	0.250	0.036	215	340	363	363	363	363	363
		0.048	215	331	467	487	487	487	487
		0.060	215	331	463	607	607	607	607
		0.075	215	331	463	647	760	760	760
		0.090	215	331	463	647	850	910	910
		0.105	215	331	463	647	850	1060	1060
		0.135	215	331	463	647	850	1060	1370

For **SI**: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]The lower of the allowable shear fastener strength and shear bearing found in Tables 4 and 5, respectively must be used for design.
[2]ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables
[3]The allowable bearing capacity for other member thicknesses can be determined by interpolating within the table.
[4]To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
[5]For F_u = 65 ksi steel, multiply values by 1.44.

FIGURE 1—HILTI HEX WASHER HEAD SELF-DRILLING SCREW

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PROJECT: CFS-NEES | PROJECT NO: 10-277 | DESIGN: RLM | DATE: Apr '12

West Shearwall Chords @ Balloon Framing

Shearwalls L2W3 & L1W3
Similar to Cond'n @ North L2N2 + L1N2 (pg SW-1 + SW-2)

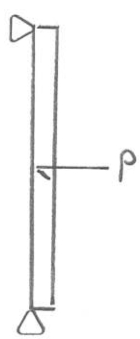
Level 1 Top Track

$\omega \le 15\ (9) = 135$ lb/ft
Span $= 3'0''$

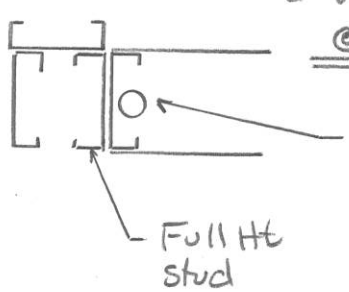
∴ Per pg SW-1 use 600T150-54 - Do Not Splice w/in shearwall Length

Clerestory Jamb

$H = 18'$ $\omega \le 15$ lb/ft ∴ $P \le 135\ (3/2) = 203$ lb @ 9'
$C < 440$ lb (Does Not Support Joists)



∴ 600S162-54 Full Ht & Connection @ Transition per pg SW-1 ok



For 3' span, Cantilever Track ok for $K_x L_x$ bracing of stud per pg SW-2

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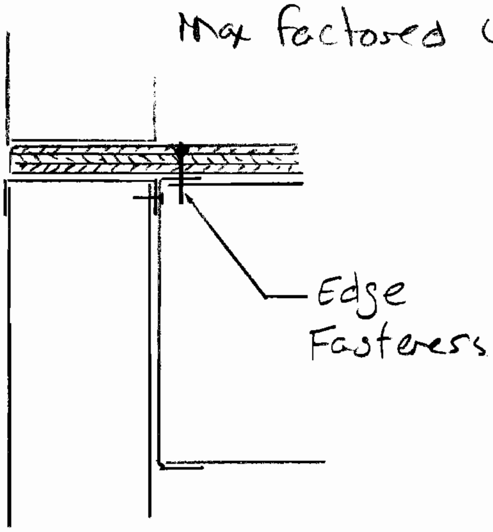
PROJECT: CFS-NEES	PROJECT NO: 10-277	DESIGN: RLM	DATE: May '11

Roof Diaphragm

See spreadsheet for shear analysis

Chords + Collectors

Max factored load = 2241 lb



(1) 1200T200-68 Rim Track
$\phi P_n = 6724$ lb @ $KL = 24"$

splice:

(2) $\#scs = \frac{2241}{801} = 2.8$

$\therefore$ By Insp use 1200S200-54 x 1'6" Nested - (4)#10 to web + (2) #10 ea leg ea side of splice

Rim Track to Top Track

w/in zone of shearwall:

$v_u = 284$ lb/ft max

ϕP_n for #10 = 375 lb/scs (3)

$\therefore$ Use #10 @ 12" oc Rim Track to Top Track

Note: for consistency with 2nd floor design rim track is used as collector/chord. At 2nd floor, tracks have large web penetrations for shearwall ties

2007 North American Specification [AISI S100] LRFD
DATE: 5/16/2011
CFS-NEES

SECTION DESIGNATION: 1200T200-68 [50] Single

Section Dimensions:

Web Height =	12.250 in
Top Flange =	2.000 in
Bottom Flange =	2.000 in
Inside Corner Radius =	0.1070 in
Design Thickness =	0.0713 in

Steel Properties:

Fy =	50.000 ksi
Fu =	65.000 ksi
Fya =	50.000 ksi

MAXIMUM FACTORED AXIAL LOADS, Pu

INPUT PARAMETERS

Overall Stud Length = 2 ft
Member Configuration: SINGLE MEMBER

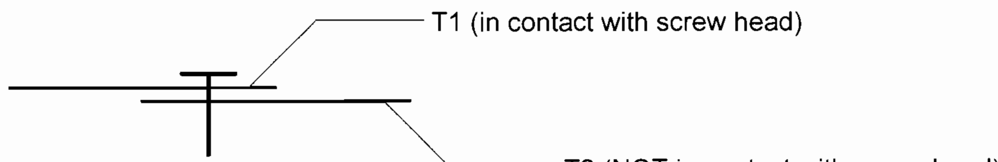
TOTAL FACTORED AXIAL LOADS, Pu (lb)

WEAK AXIS BRACING	MAXIMUM KL/r	CONCENTRIC LOADING	LOADED THROUGH WEB
NONE	47	14940	6724
MID Pt	24	16035	7069
THIRD Pt	16	16246	7134

2

2007 North American Specification [AISI S100] LRFD
DATE: 5/16/2011

CFS-NEES



Screw Connection Input Parameters

T1 = 0.0713 in   Fu(1) = 65 ksi   Edge Dist = NA
T2 = 0.0566 in   Fu(2) = 65 ksi   Edge Dist = NA
Screw Diameter = #10 (0.190 in)
Screw Head Diameter = 0.3125

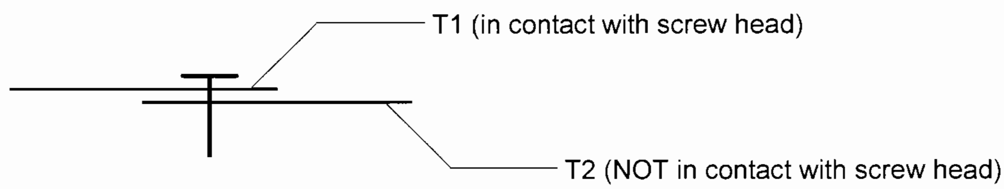
Results

	Nominal Pn (lb)	ASD Pn/Omega (lb)	LRFD phi x Pn (lb)	Min Req'd Screw Strength, Pss (lb)
Shear	1602.4	534.1	801.2	2003.0
Pullout (T2)	594.2	198.1	297.1	2715.5
Pullver (T1)	2172.4	724.1	1086.2	2715.5

Notes:
1. Pullout values assume screw fully penetrates T2
2. Minimum edge distance = 1.5d = 0.285 (in)

2007 North American Specification [AISI S100] LRFD
DATE: 5/16/2011

CFS-NEES



Screw Connection Input Parameters

T1 = 0.0713 in   Fu(1) = 65 ksi   Edge Dist = NA
T2 = 0.0451 in   Fu(2) = 45 ksi   Edge Dist = NA
Screw Diameter = #10 (0.190 in)
Screw Head Diameter = 0.3125

Results

	Nominal Pn (lb)	ASD Pn/Omega (lb)	LRFD phi x Pn (lb)	Min Req'd Screw Strength, Pss (lb)
Shear	789.0	263.0	394.5	986.3
Pullout (T2)	327.8	109.3	163.9	2715.5
Pullver (T1)	2172.4	724.1	1086.2	2715.5

Notes:
1. Pullout values assume screw fully penetrates T2
2. Minimum edge distance = 1.5d = 0.285 (in)

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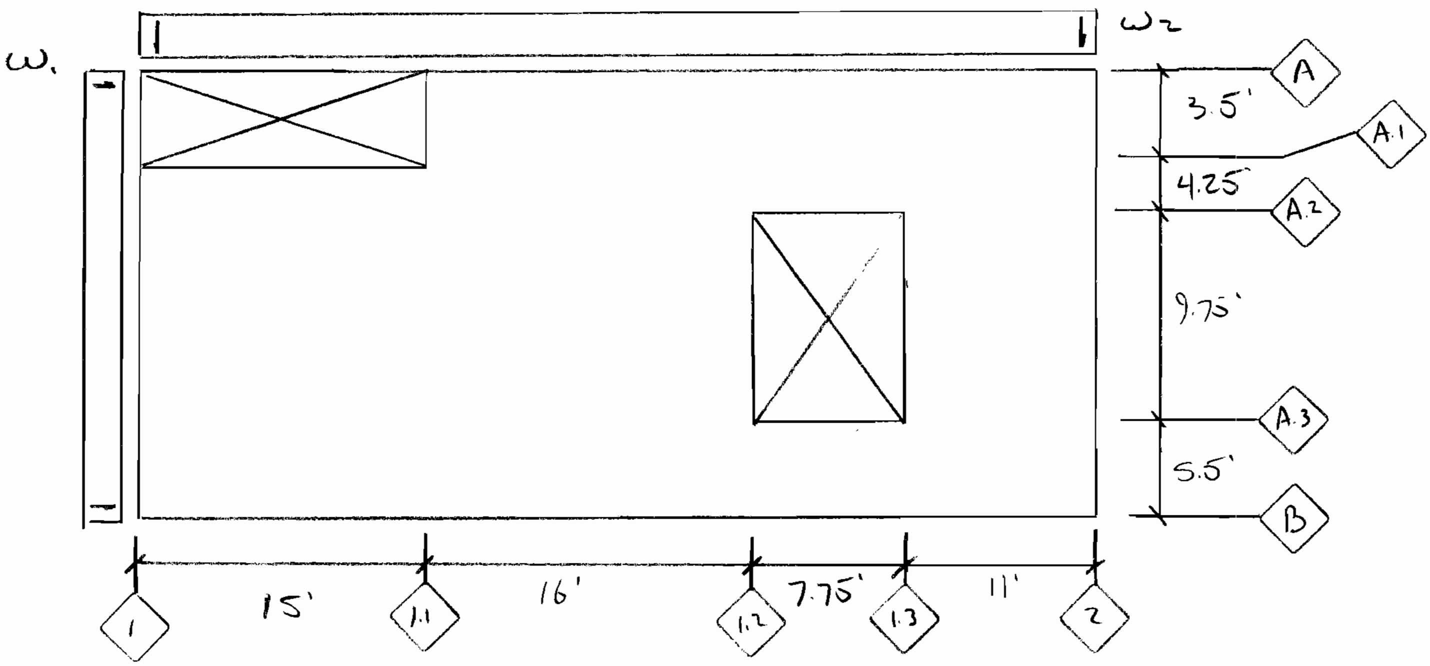
PROJECT: CFS-NEES | **PROJECT NO:** 10-277 | **DESIGN:** RLM | **DATE:** May '11

2nd Floor diaphragm

Use min 23/32 struct. Rated 48/24 panels. T + G
Face grain ⊥ to Joists



estimate $\omega = \frac{V}{L}$: $V = 8365$ lb (Shearwall Analysis and Design.xlsx)

$\therefore \omega_1 = \frac{8365}{23} = 364$ lb/ft

$\omega_2 = \frac{8365}{49.75} = 168$ lb/ft

$\therefore$ @ ea Line of Shear $V = \frac{8365}{2} = 4183$ lb

Chords/Drags Max $C_u = 3804$ lb $\therefore \#scr = \frac{3804}{801} = 4.7$

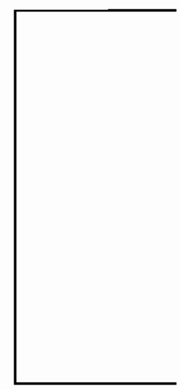
1 : 1200T200-97 Rim Track o/k. $\phi P_n = 12289$ lb
Splice per Roof Rim Track

2007 North American Specification [AISI S100] LRFD
DATE: 5/20/2011

SECTION DESIGNATION: 1200T200-97 [50] Single

Section Dimensions:

Web Height =	12.356 in
Top Flange =	2.000 in
Bottom Flange =	2.000 in
Inside Corner Radius =	0.1526 in
Design Thickness =	0.1017 in



Steel Properties:

Fy =	50.000 ksi
Fu =	65.000 ksi
Fya =	50.000 ksi

MAXIMUM FACTORED AXIAL LOADS, Pu

INPUT PARAMETERS

Overall Stud Length = 2 ft
Member Configuration: SINGLE MEMBER

TOTAL FACTORED AXIAL LOADS, Pu (lb)

WEAK AXIS BRACING	MAXIMUM KL/r	CONCENTRIC LOADING	LOADED THROUGH WEB
NONE	48	28951	12289
MID Pt	24	31225	12882
THIRD Pt	16	31663	12995

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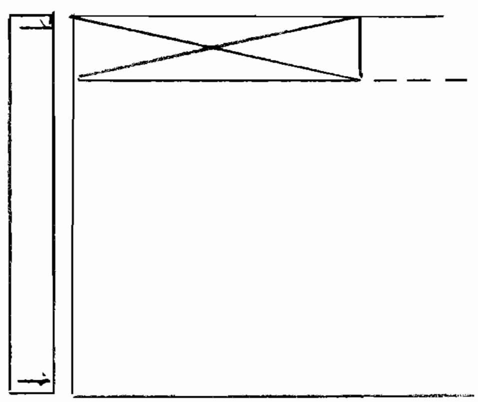
PROJECT: CFS-NEES | PROJECT NO: 10-277 | DESIGN: RLM | DATE: May '11

Reinforcing @ Exit Stair Ro



Tributary Seismic mass:

Walls: $W_W = 10(9)(23 + 2(15))$
$= 4770$ lb

Floor $W_F = (18+10)(23)(15)$
$= 9660$ lb

$\Sigma W = 14430$ lb

$\therefore \Delta V = \frac{14430}{45134}(8365) = 2674$ lb

$\therefore$ Total Drag @ edge of Ro
$V_D \approx 2674/2 = 1337$ lb

ϕV_n for diaphragm $= 333$ lb/ft

$\therefore L_{req'd} = \frac{1337}{333} = 4.0'$

$\therefore$ Use (3) Bays = 72" of solid Blk'g & 1½" x 54-mil strap. Extend strap min 12" over rim Track - (4) #10 strap to Track. Diaphragm edge screws Typ @ strap/Blk'g

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PROJECT: CFS-NEES | **PROJECT NO:** 10-277 | **DESIGN:** RLM | **DATE:** May '11

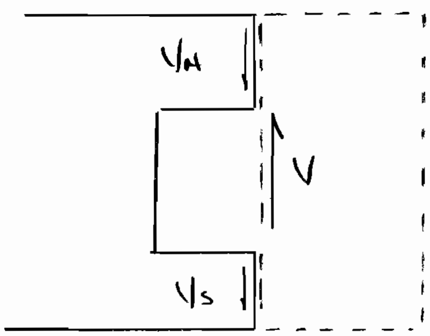
Reinforcing @ Clerestory

For Loads N-S:

$$V_{\langle 1.2 \rangle} = \frac{8365}{2} - 168(18.75) = 1033 \text{ lb}$$

$$V_{\langle 1.3 \rangle} = \frac{8365}{2} - 168(11) = 2335 \text{ lb}$$

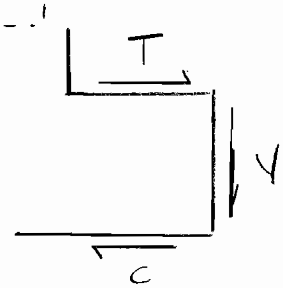
Consider Piers N & S of Opening



distribute V based on pier length

$$\therefore V_N = 2335\left(\frac{7.75}{13.25}\right) = 1366 \text{ lb}$$

$$V_S = 2335\left(\frac{5.5}{13.25}\right) = 969 \text{ lb}$$



$$\therefore C_N/T_N = 1366\left(\frac{7.75}{7.75}\right) = 1366 \text{ lb}$$

$$C_S/T_S = 969\left(\frac{7.75}{5.5}\right) = 1366 \text{ lb}$$

For diaphragm $v_n = 333$ lb/ft

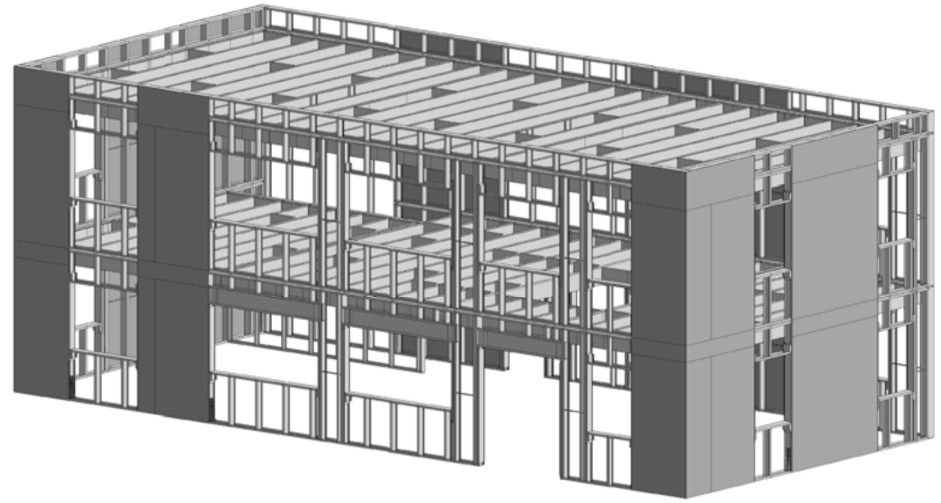
$$L_{req'd} = \frac{1366}{333} = 4.1'$$

$\therefore$ Use strap + Solid Blk'g for min (3) Joist Bays ea corner. Edge screws Typ @ Block'g

Appendix 5

Design Drawings

CFS NEES
JOHNS HOPKINS UNIVERSITY



ABBREVIATIONS FOR COLD-FORMED GENERAL NOTES

A.D. - ARCHITECTURAL DRAWINGS
ADD'L - ADDITIONAL
ALT. - ALTERNATE
BM. - BEAM
B.O. - BOTTOM OF
BLD'G - BUILDING
BLK'G - BLOCKING
BTWN. - BETWEEN
CL. OR CLR. - CLEAR
CLG. - CEILING
COL. - COLUMN
CONC. - CONCRETE
CONN. - CONNECTION
CONT. - CONTINUOUS
CRC - COLD ROLLED CHANNEL
C.W. - CURTAINWALL
DBL. - DOUBLE
DEF'L - DEFLECTION
DIAG. - DIAGONAL
DIM - DIMENSION
DIV ANG OR DA - DIVERTER ANGLE
DWG - DRAWING
EA. - EACH
E.D. - EDGE DISTANCE
EL. OR ELEV. - ELEVATION
(E) - EXISTING
E.O.D. - EDGE OF DECK
E.O.R. - ENGINEER OF RECORD
E.O.S. - EDGE OF SLAB
EQ. - EQUAL
F.O. - FACE OF
FLG - FLANGE
FLR. - FLOOR
F.S. - FAR SIDE
GA - GAUGE
G.C. - GENERAL CONTRACTOR
HDR - HEADER
HGT. - HEIGHT
HORIZ OR HOR. - HORIZONTAL
HSS - HOLLOW STRUCTURAL SECTION
HWC - HIGH WIND CORNER. PER UBC, AREA EXTENDING FROM BUILDING CORNERS 10 FEET OR 0.1 TIMES THE LEAST WIDTH OF THE BUILDING, WHICHEVER IS LESS.
I.L.O. - IN LIEU OF
INV. - INVERTED
JT. - JOINT
LG. - LONG
LOC'N - LOCATION
LLH - LONG LEG HORIZONTAL
LLV - LONG LEG VERTICAL
L.V.F. - LOW VELOCITY FASTENER (SEE GENERAL NOTES FOR SIZE & TYPE).
LVL. - LEVEL
LWC - LIGHT WEIGHT CONCRETE
MAX. - MAXIMUM
MFG - MANUFACTURER
MIN. - MINIMUM
(N) - NEW
N.B.D. - NOT BY DEVCO
N.T.S. - NOT TO SCALE
N/A - NOT APPLICABLE
N.S. - NEAR SIDE
NWC - NORMAL WEIGHT CONCRETE
O/ - OVER
O.C. - ON CENTER
O.H. - OPPOSITE HAND
O.H.D. - OVERHEAD DOOR
OPN'G - OPENING
OWJ - OPEN WEB JOIST
PC. - PIECE
PERP. - PERPENDICULAR
PT - POINT
REINF. - REINFORCING
REF. - REFERENCE
REQ'D - REQUIRED
R.F.I. - REQUEST FOR INFORMATION
R.O. - ROUGH OPENING
S.D. - STRUCTURAL DRAWINGS
SECT. - SECTION
SIM. - SIMILAR
SPCL BRK - SPECIAL BRAKE
SQ. - SQUARE
STL - STEEL
SW - SHEARWALL
T&B - TOP & BOTTOM
T.O. - TOP OF
TYP. - TYPICAL
U.N.O. - UNLESS NOTED OTHERWISE
VERT. - VERTICAL
W.B. - WEDGE BOLT
WDW. - WINDOW
WF - WIDE FLANGE
W/ - WITH
W/IN - WITHIN
W/O - WITHOUT
W.P. - WORK POINT

COLD-FORMED STEEL GENERAL NOTES

1. DESIGN CRITERIA
   2009 IBC
   OCCUPANCY CATEGORY II
   WIND: 85 MPH (3 SECOND GUST), EXP. B, I=1.0
   SEISMIC: S_{DS} = 0.93
   SEISMIC SITE CLASS D, IE=1.0
   SEISMIC DESIGN CATEGORY D
   S_S = 1.39
   S_1 = 0.50
   R = 6.5
   C_d = 4
   WΩ = 3

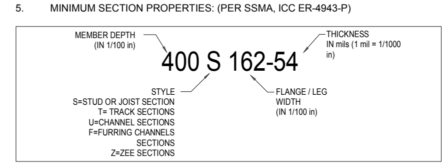
   DEF'L. LIMITS : FLOOR = L/360 LL; L/240 DL + LL
   ROOF = L/240 LL; L/180 DL + LL
   DESIGN LIVE LOADS:
   FLOOR: OFFICES = 50 psf
   OFFICE CORRIDORS ABOVE 1ST FLOOR = 80 psf
   PARTITIONS = 15 psf
   ROOF: ROOF LIVE = 20 psf

2. ALL COLD FORMED STEEL STUDS, JOIST, TRACK & MISC. SHAPES MILL CERTIFIED STEEL TO MEET:
   A. ASTM A1003 ST GRADE 50, TYPE H 54-97 mil GALV. STEEL
   B. ASTM A1003 ST GRADE 33, TYPE H 18-43 mil GALV. STEEL
   EXTERIOR MEMBERS - G60 MINIMUM

3. ALL STEEL STUDS, JOIST & TRACK SHALL HAVE A LEGIBLE LABEL, STAMP OR EMBOSSMENT, AT A MAXIMUM OF 48" O.C., INDICATING THE MANUFACTURER'S NAME, LOGO OR INITIALS, ICC EVALUATION SERVICE REPORT NUMBER, THE MATERIAL BASE METAL THICKNESS (UNCOATED) IN .001 in. AND THE YIELD STRENGTH IF DIFFERENT THAN 33 ksi.

4. MILL CERTIFICATES FROM THE COIL PRODUCER SHALL BE MADE AVAILABLE IF REQUESTED. MILL CERTIFICATE TO INCLUDE AS A MINIMUM THE CHEMICAL COMPOSITION, YIELD STRENGTH, TENSILE STRENGTH, ELONGATION, AND COATING THICKNESS.

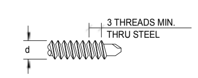
5. MINIMUM SECTION PROPERTIES: (PER SSMA, ICC ER-4943-P)



MINIMUM DELIVERABLE THICKNESS (mils)	GAUGE	DESIGN THICKNESS (INCHES)
33	20	.0346
43	18	.0451
54	16	.0566
68	14	.0713
97	12	.1017
118	10	.1242

6. STUDS AND TRACKS THAT COMPRISE A HEADER, STRONGBACK OR SILL SHALL NOT BE SPLICED.

7. SCREW VALUES USED IN DESIGN MEET 2007 "NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS" (ASI S100-07) SECTION E4 FOR SCREW CONNECTIONS. SCREWS TO CONFORM TO SAE J78.



THE NOMINAL STRENGTH OF THE SCREWS TESTED IN ACCORDANCE WITH SECTION F1.1(a) OF THE ASI S100-07. SHALL NOT BE LESS THAN :

SHEAR			TENSION		
#8	#10	1/4"	#8	#10	1/4"
1860	2003	2440	641	743	4580

8. STUDS SHALL BE SEATED SQUARELY WITH MAX. 1/8" GAP BETWEEN END OF STUDS AND WEB OF TOP AND BOTTOM TRACKS TYPICAL AT ALL BEARING WALLS.

SHEET INDEX

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PROJECT: CFS - NEES
PROJECT LOCATION:
CLIENT: JOHNS HOPKINS UNIVERSITY

SHEET TITLE: COVER SHEET

JOB NO. 10-277
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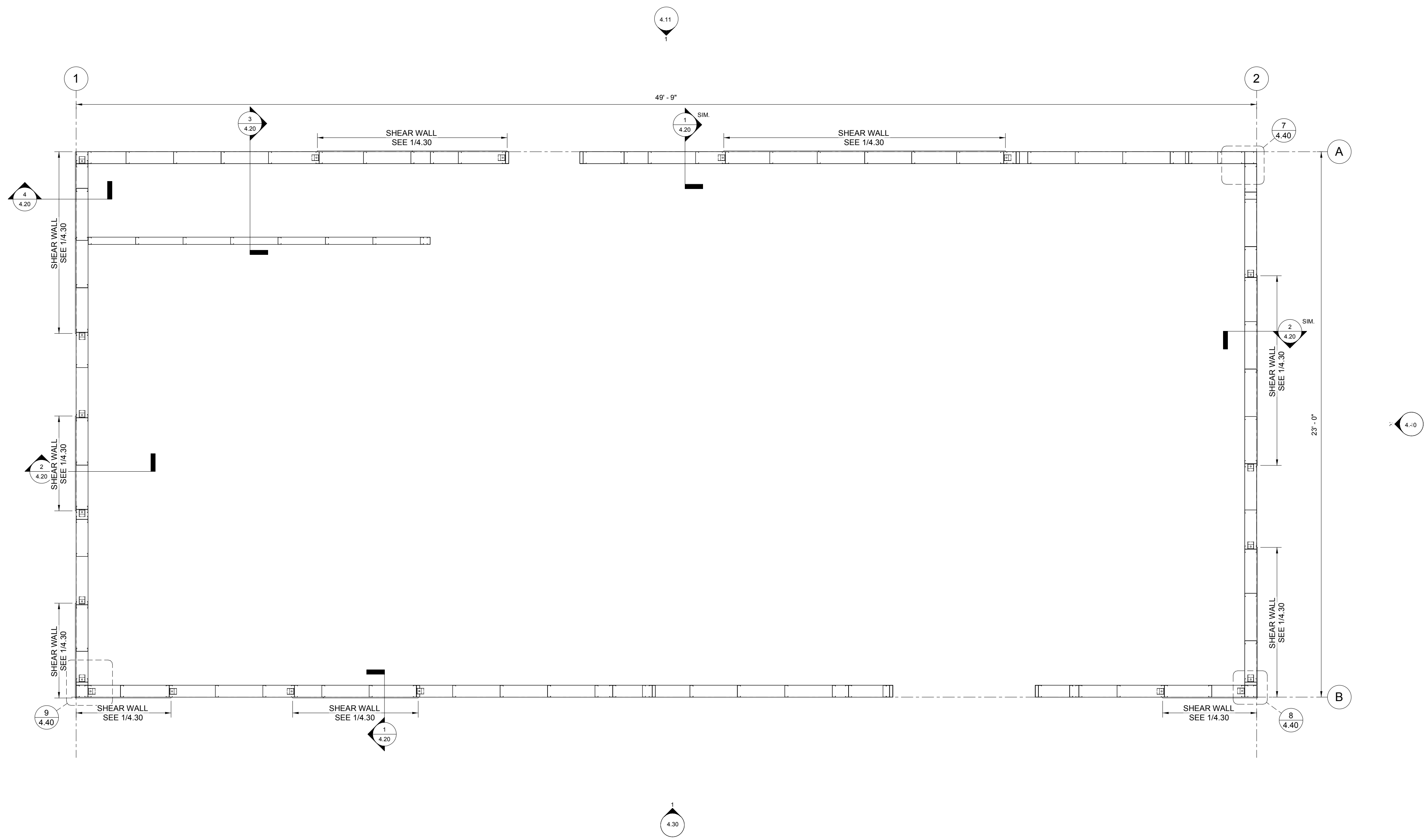
DRAWING: SF 1.00



1 LEVEL 1 LAYOUT PLAN
1/2" = 1'-0"

PROJECT NORTH

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PROJECT LOCATION:
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LEVEL 1 LAYOUT PLAN

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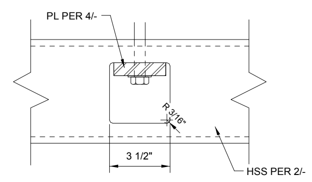
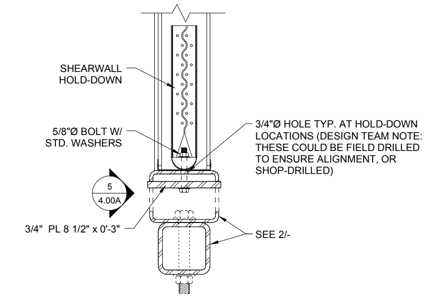
DRAWING:
SF 4.00

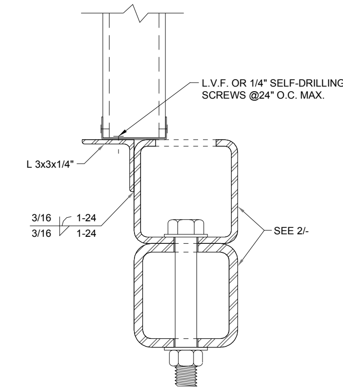
5/17/2012 11:20:38 AM



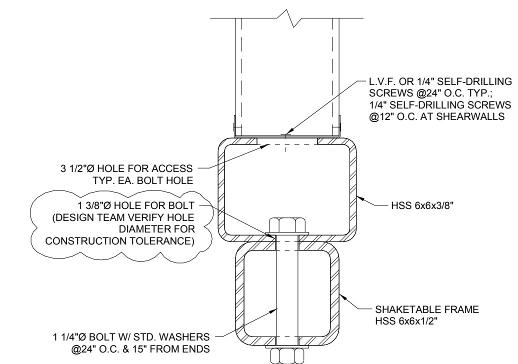
5 SHEARWALL HOLD-DOWN STIFFENER
3" = 1'-0"



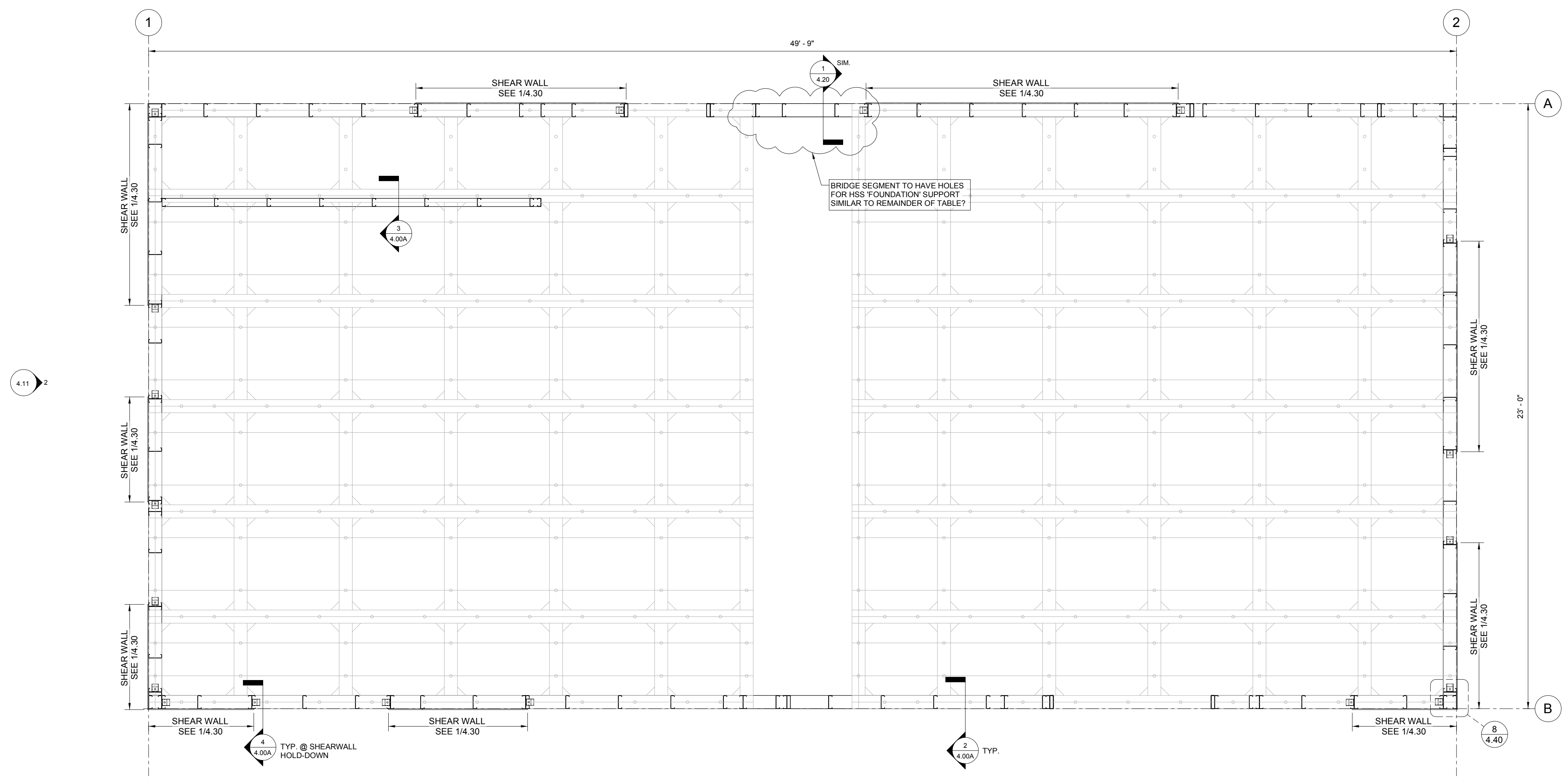
4 HSS FRAME @ SHEARWALL HOLD-DOWN
1 1/2" = 1'-0"



3 HSS FRAME @ INT. BEARING WALL
3" = 1'-0"



2 HSS FRAME @ EXTERIOR WALL
3" = 1'-0"



1 FOUNDATION PLAN
1/2" = 1'-0"

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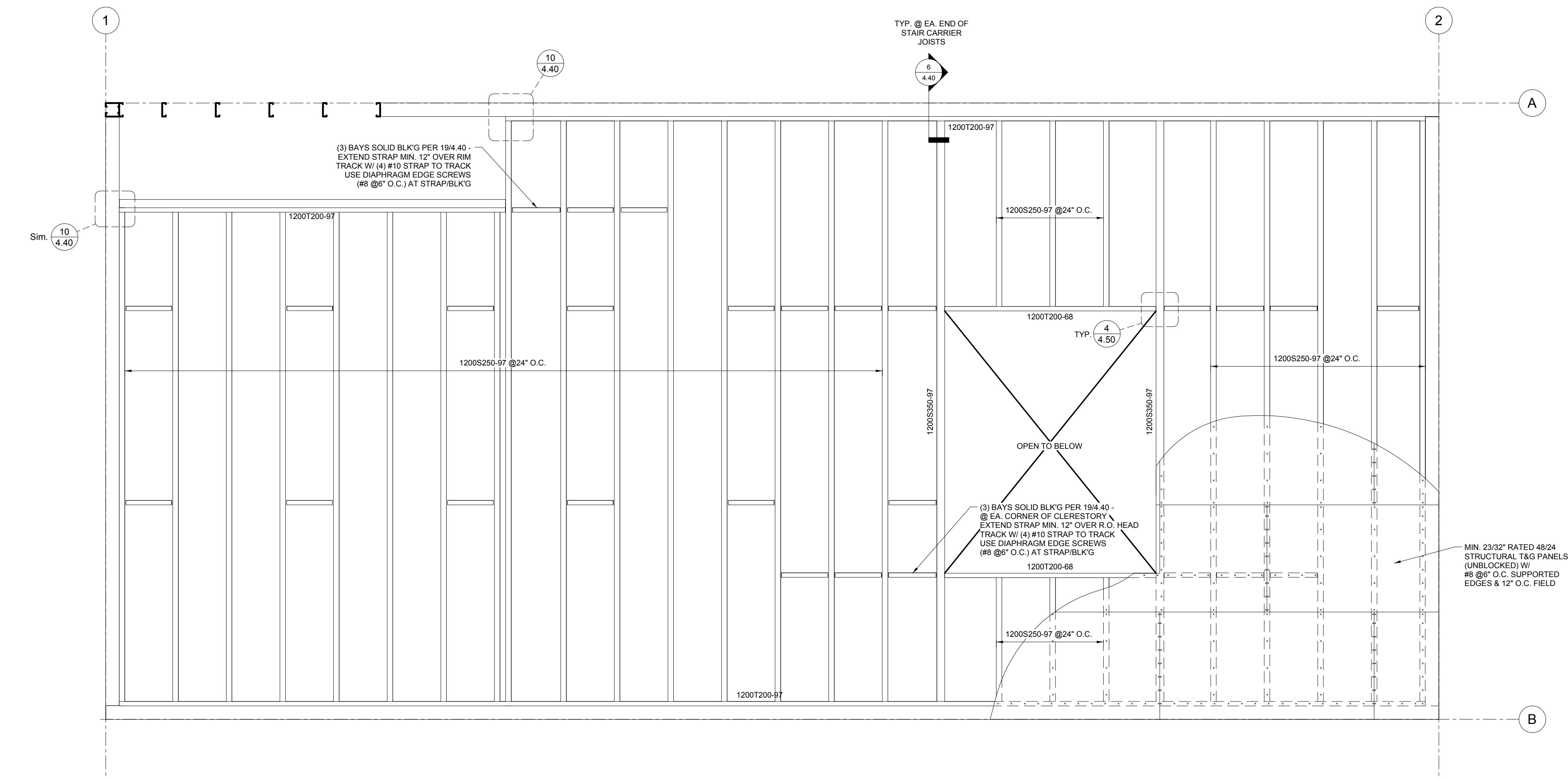

1 FLOOR JOISTS LAYOUT
1/2" = 1'-0"

PROJECT NORTH

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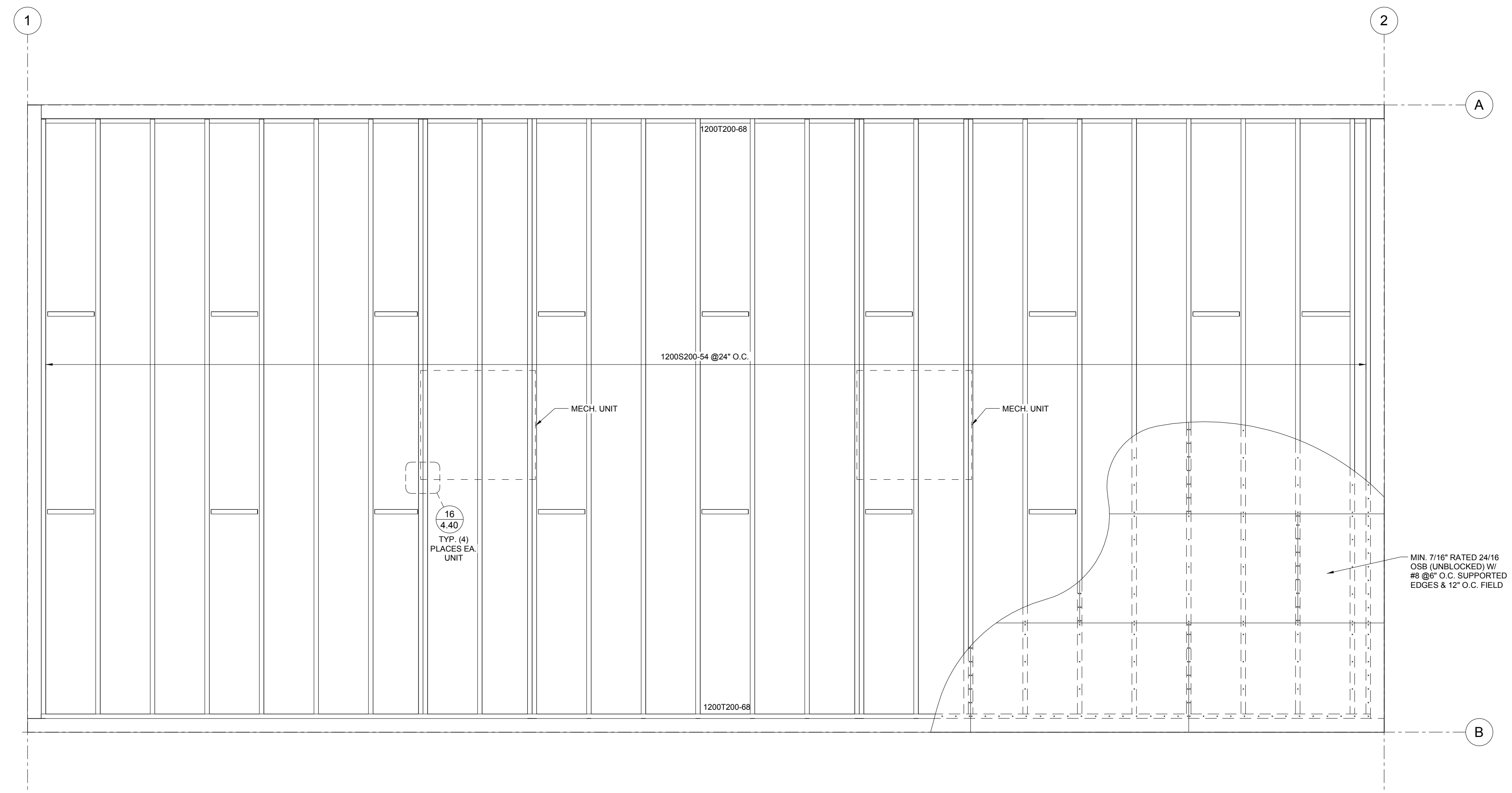

1 ROOF JOISTS LAYOUT
1/2" = 1'-0"

PROJECT NORTH

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ROOF JOIST LAYOUT PLAN

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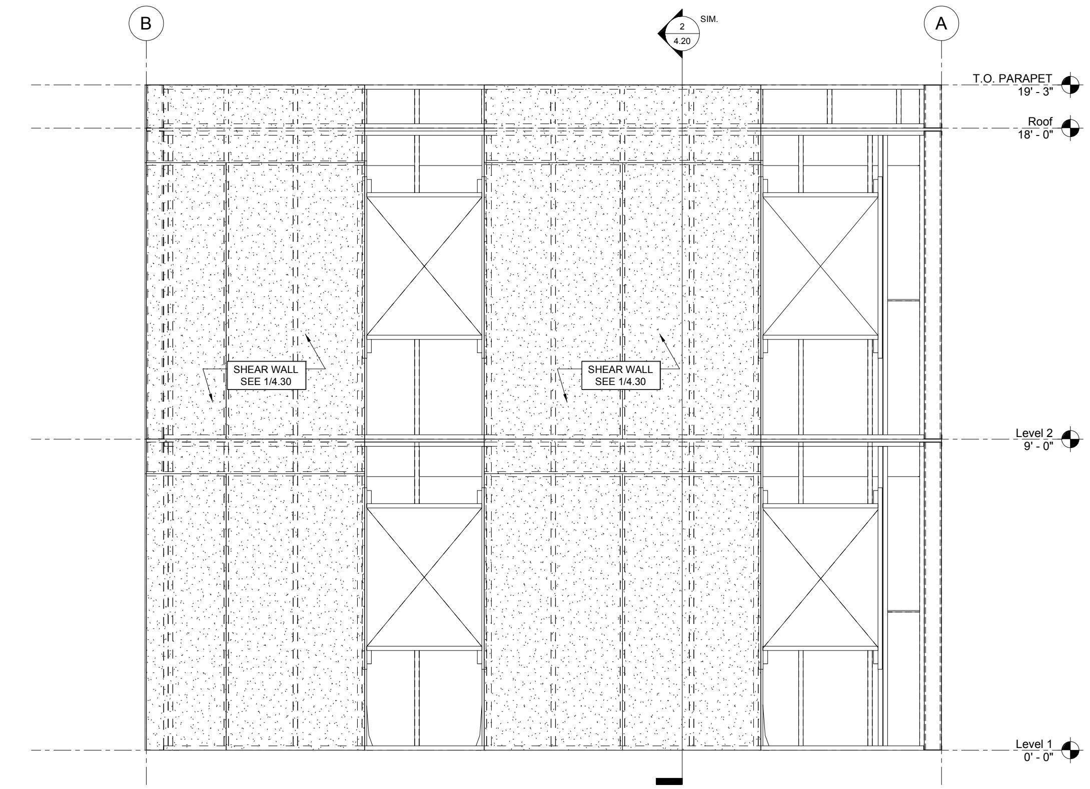
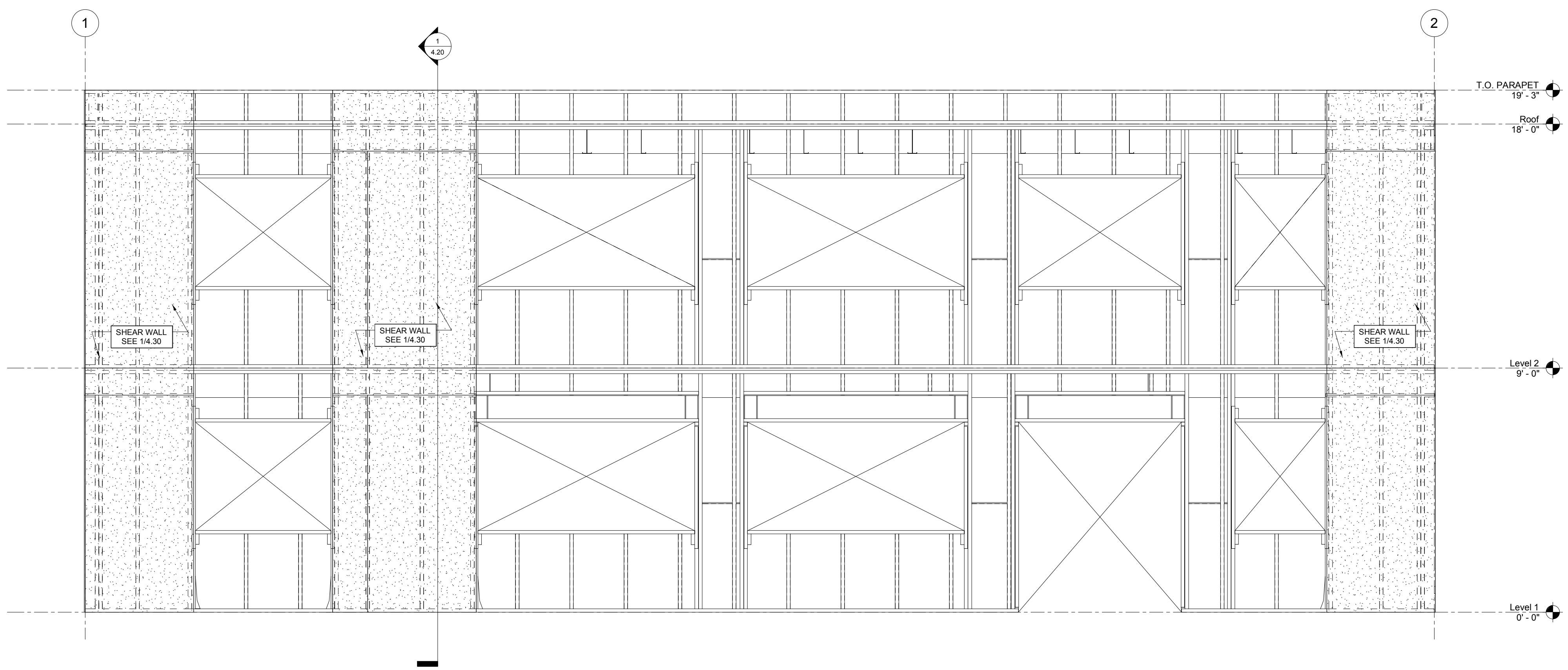


2 EAST ELEVATION
1/2" = 1'-0"



1 SOUTH ELEVATION
1/2" = 1'-0"

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DRAWN BY: DEVCO

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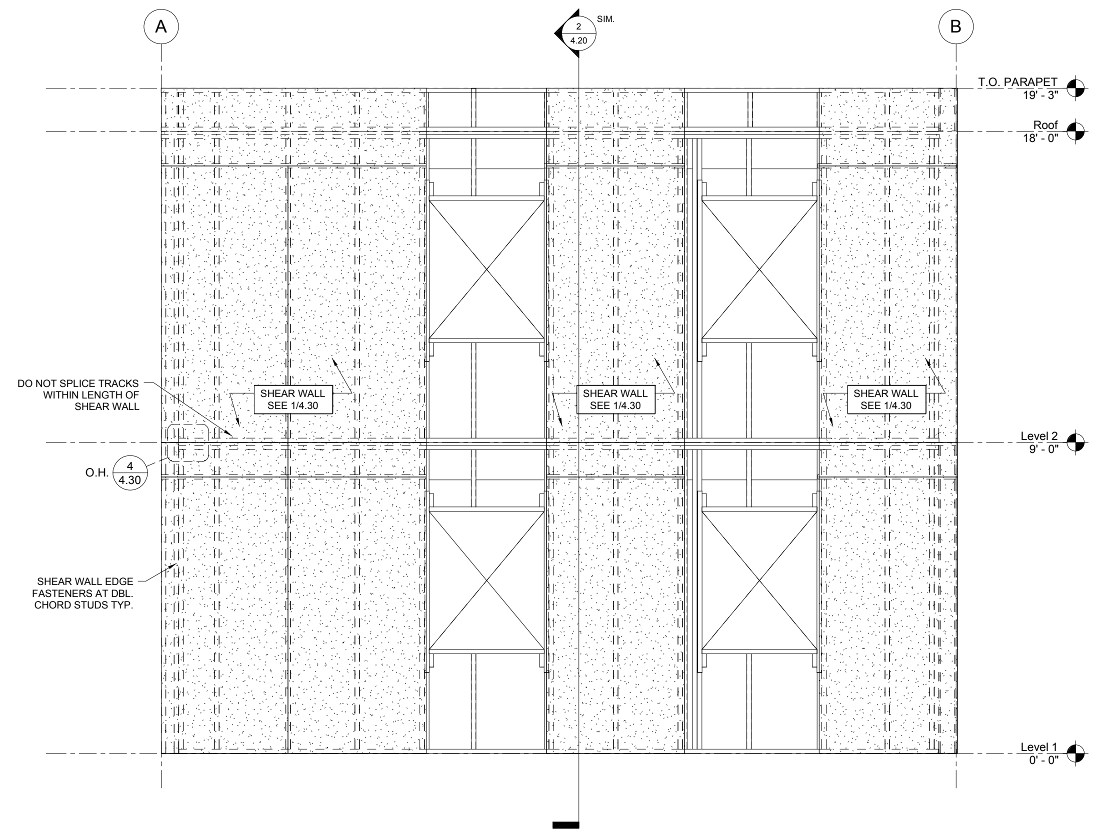
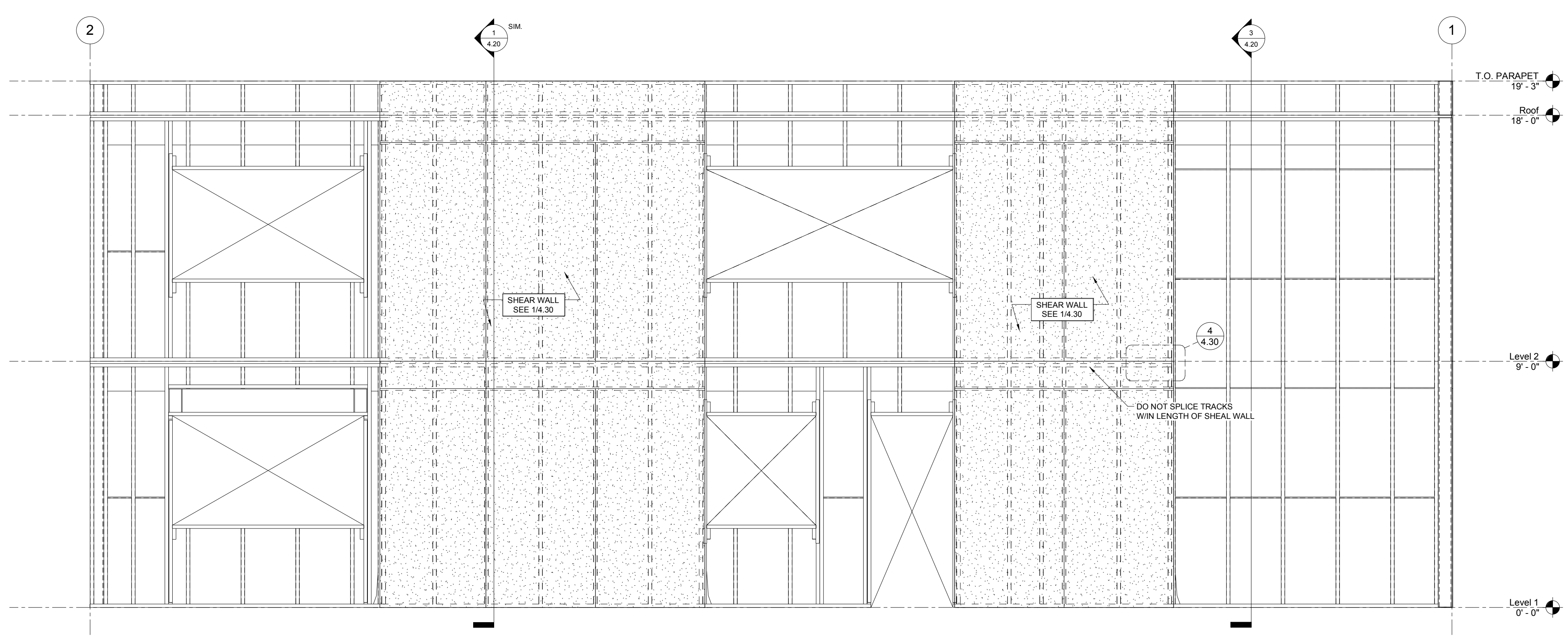


2 WEST ELEVATION
1/2" = 1'-0"



1 NORTH ELEVATION
1/2" = 1'-0"

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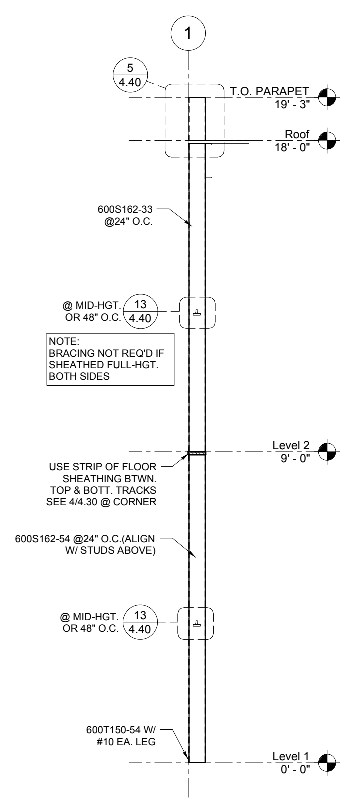
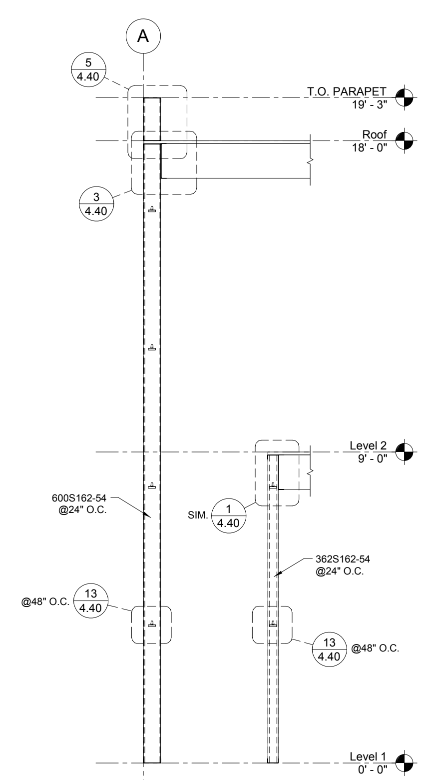
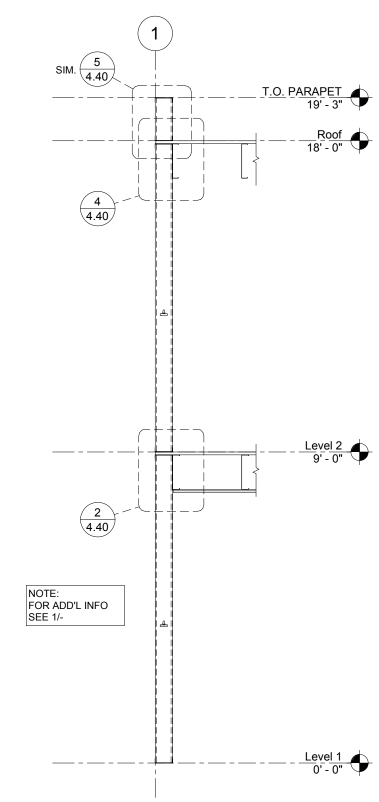


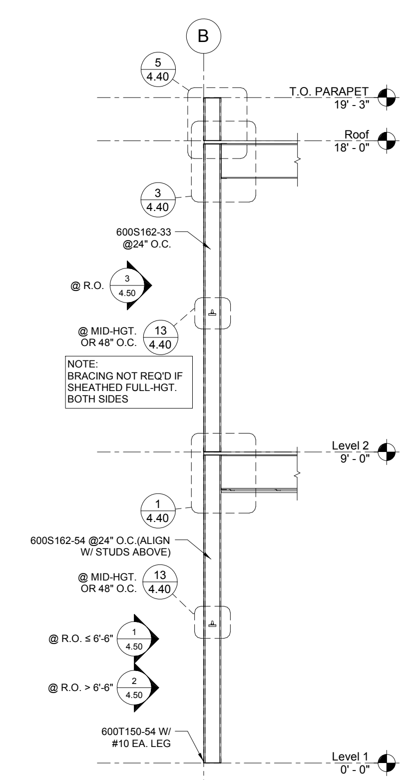
4 WALL SECTION 4
1/2" = 1'-0"



3 WALL SECTION 3
1/2" = 1'-0"



2 WALL SECTION 2
1/2" = 1'-0"



1 WALL SECTION 1
1/2" = 1'-0"

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CFS - NEES

PROJECT LOCATION:

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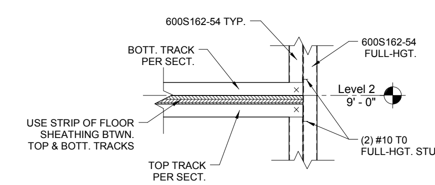
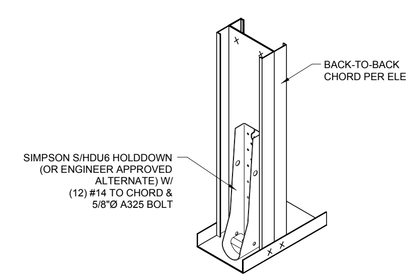
SHEET TITLE:
WALL SECTIONS

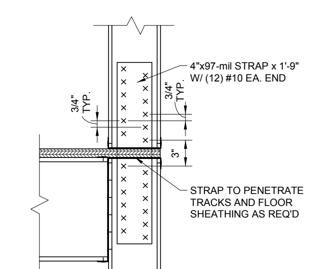
JOB NO.
DRAWN BY:
DRAWING:
SF 4.20



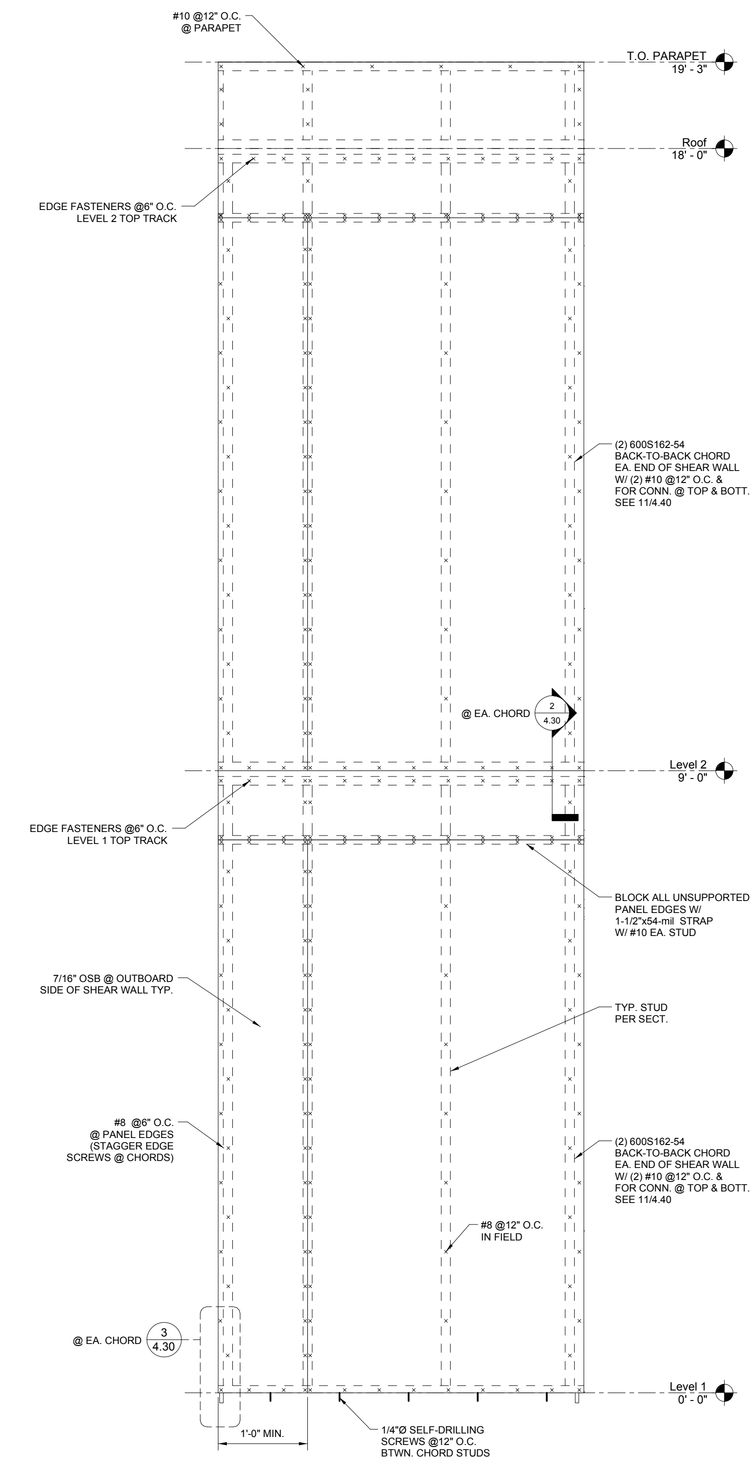
4 ENLARGED VIEW @ SHEAR WALL
1 1/2" = 1'-0"



3 SHEAR WALL HOLDOWNS
1 1/2" = 1'-0"



2 SHEAR WALL LEVEL 2 TIE
1 1/2" = 1'-0"



1 SHEAR WALL TYPE I ELEVATION
1" = 1'-0"

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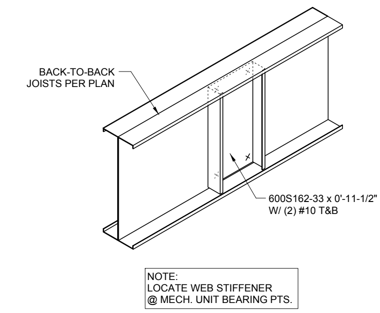
SHEET TITLE: SHEAR WALL ELEVATIONS

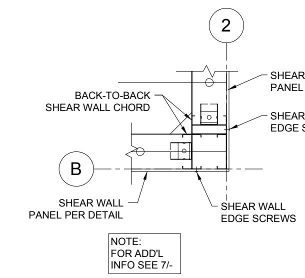
JOB NO.

DRAWN BY:

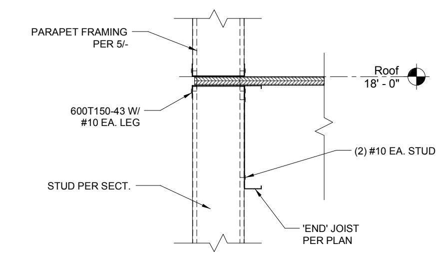
DRAWING: SF 4.30



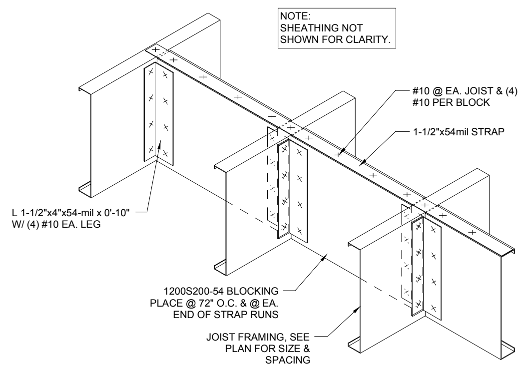
16 JOIST WEB STIFFENER @ MECH. UNIT
1 1/2" = 1'-0"



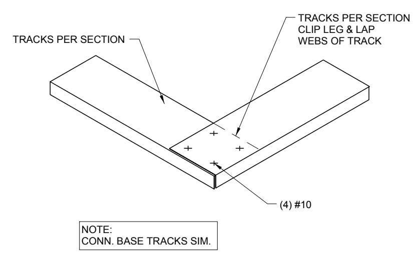
8 CORNER LAYOUT - SE CORNER
1" = 1'-0"



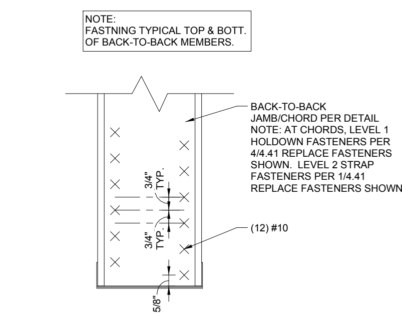
4 ENLARGED SECTION @ ROOF - JOIST PARALLEL
1 1/2" = 1'-0"



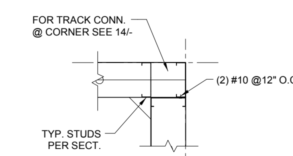
19 JOIST SOLID BLOCKING DETAIL
1 1/2" = 1'-0"



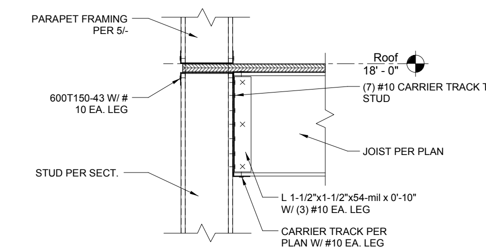
15 TRACK LAP @ CORNER
N.T.S.



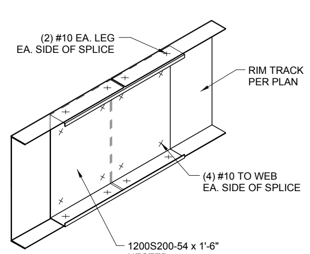
11 JAMB / SHEAR WALL CHORD ENDS
3" = 1'-0"



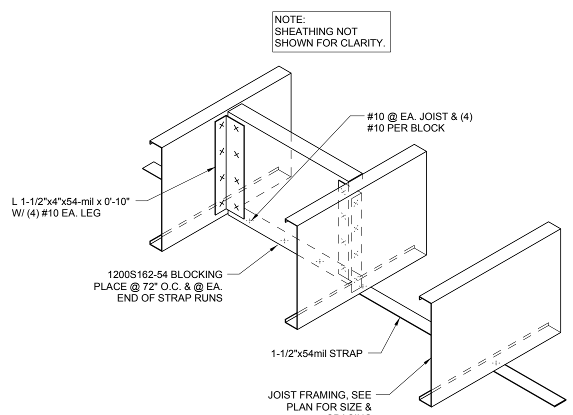
7 CORNER LAYOUT - TYP.
1" = 1'-0"



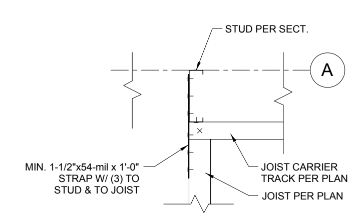
3 ENLARGED SECTION @ ROOF - JOIST PERP.
1 1/2" = 1'-0"



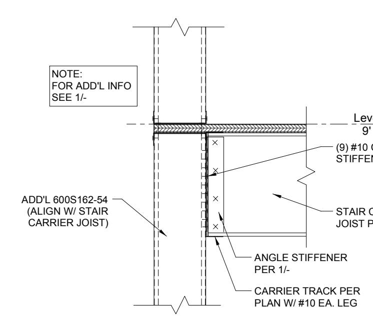
18 RIM TRACK SPLICE
1 1/2" = 1'-0"



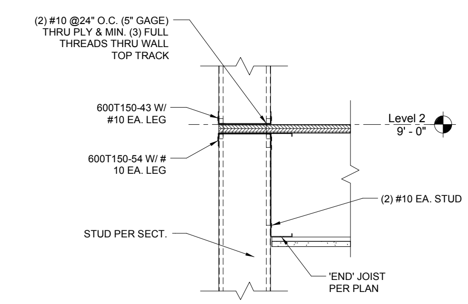
14 JOIST BLOCKING DETAIL
1 1/2" = 1'-0"



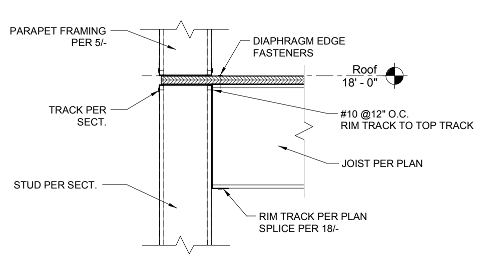
10 JOIST TIE @ EDGE OF CLERESTORY
1 1/2" = 1'-0"



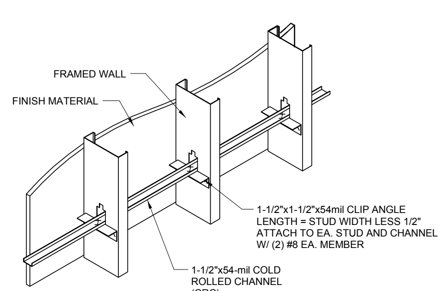
6 ENLARGED SECTION @ STAIR CARRIER
1 1/2" = 1'-0"



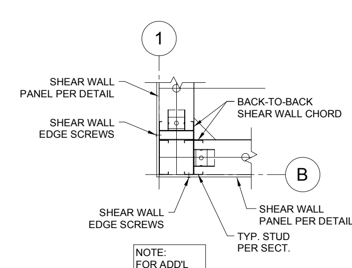
2 ENLARGED SECTION @ 2ND FLOOR - JOIST PARALLEL
1 1/2" = 1'-0"



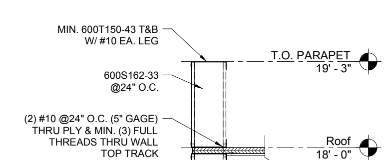
17 ENLARGED SECTION @ ROOF DIAPHRAGM EDGE
1 1/2" = 1'-0"



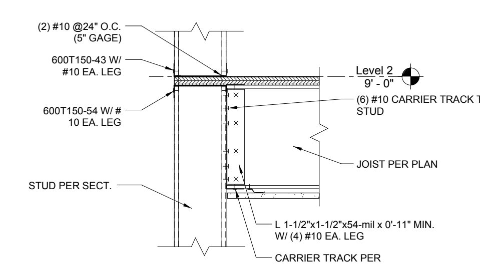
13 WALL BRACING - CRC
N.T.S.



9 CORNER LAYOUT - SW CORNER
1" = 1'-0"



5 ENLARGED SECTION @ PARAPET
1" = 1'-0"



1 ENLARGED SECTION @ 2ND FLOOR - JOIST PERP.
1 1/2" = 1'-0"

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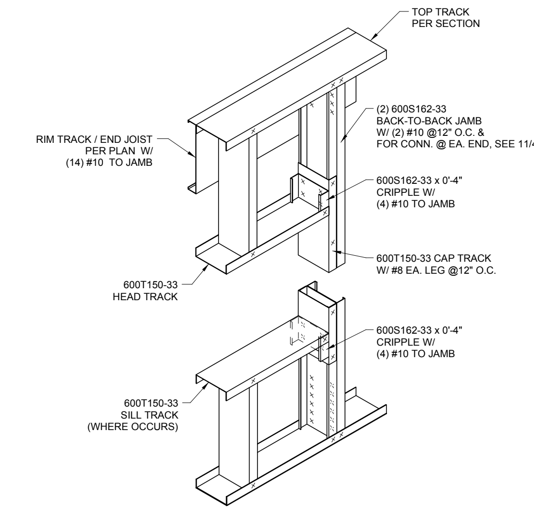
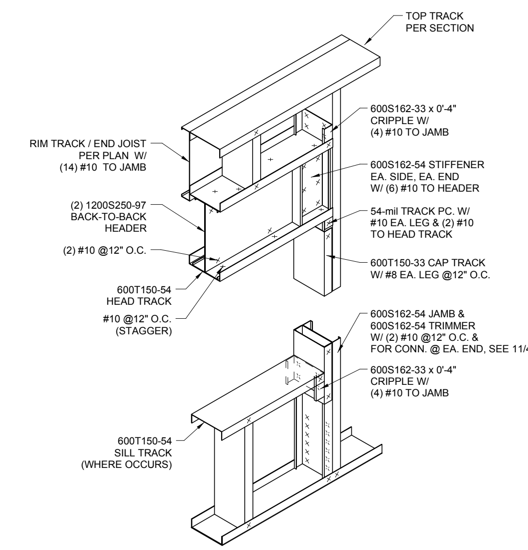
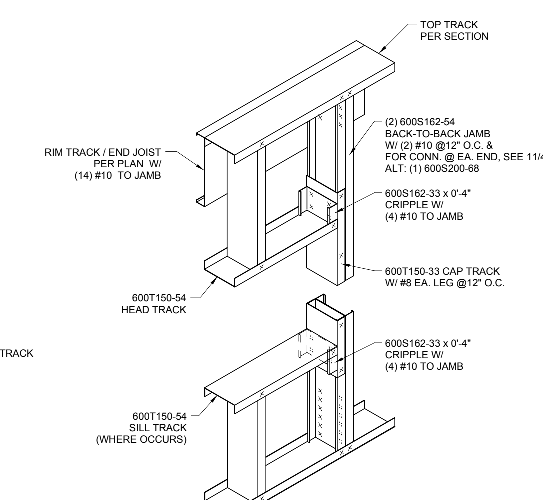


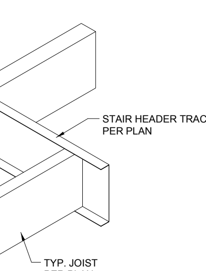
3 LEVEL 2 FRAMED OPENING (R.O. ≤ 8'-0" WIDE)
1" = 1'-0"



2 LEVEL 1 FRAMED OPENING (R.O. > 6'-6" WIDE)
1" = 1'-0"



1 LEVEL 1 FRAMED OPENING (R.O. ≤ 6'-6" WIDE)
1" = 1'-0"



4 STAIR HEADER CONN.
1" = 1'-0"

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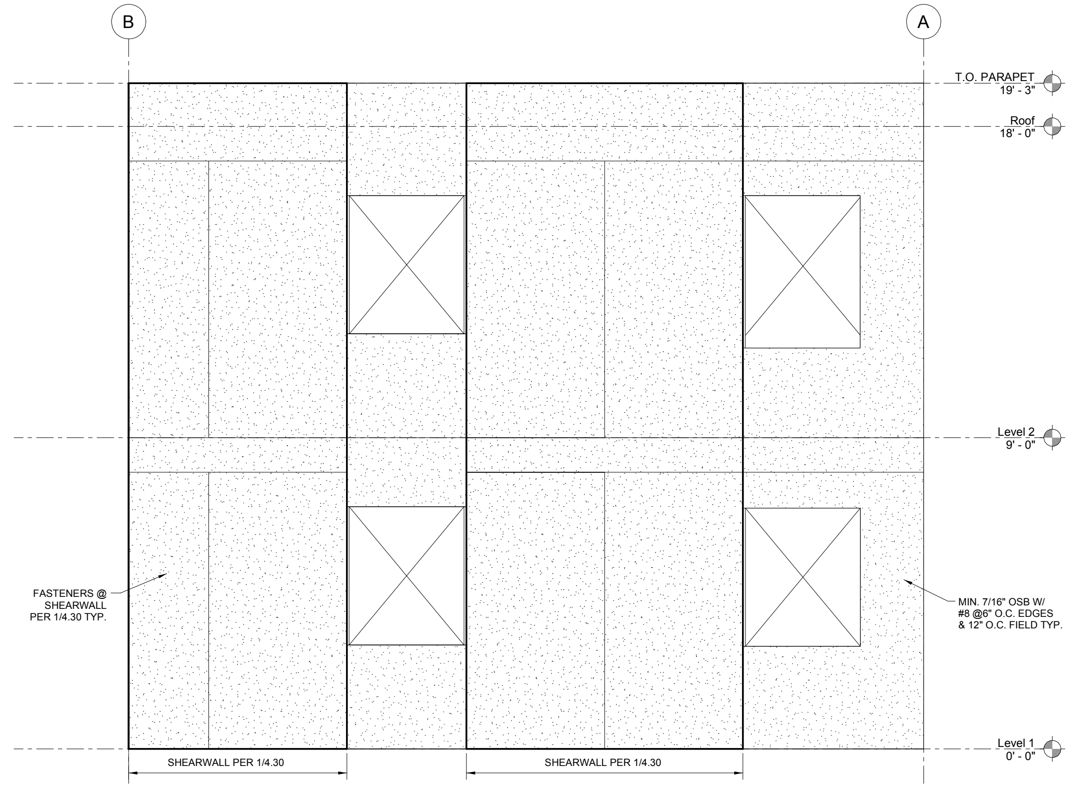
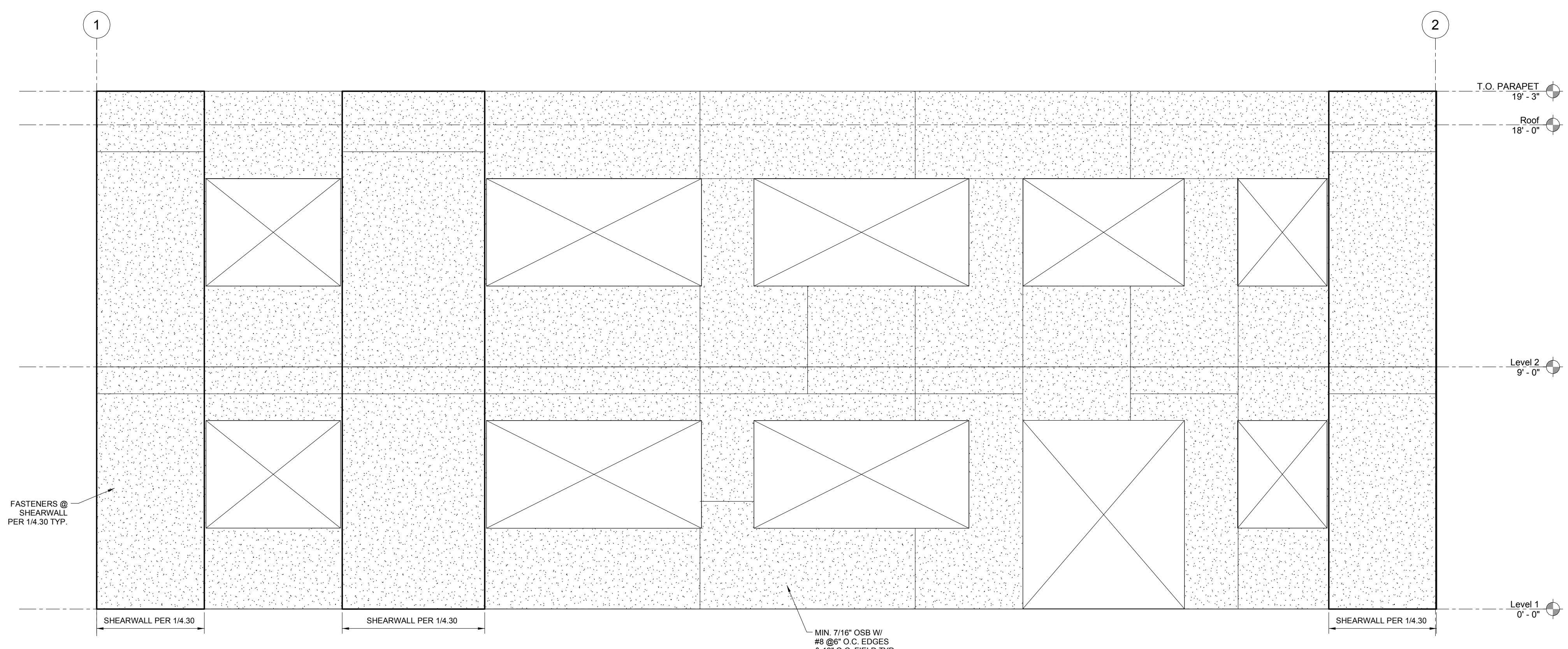


2 SHEATHING ELEVATION - EAST
1/2" = 1'-0"



1 SHEATHING ELEVATION - SOUTH
1/2" = 1'-0"

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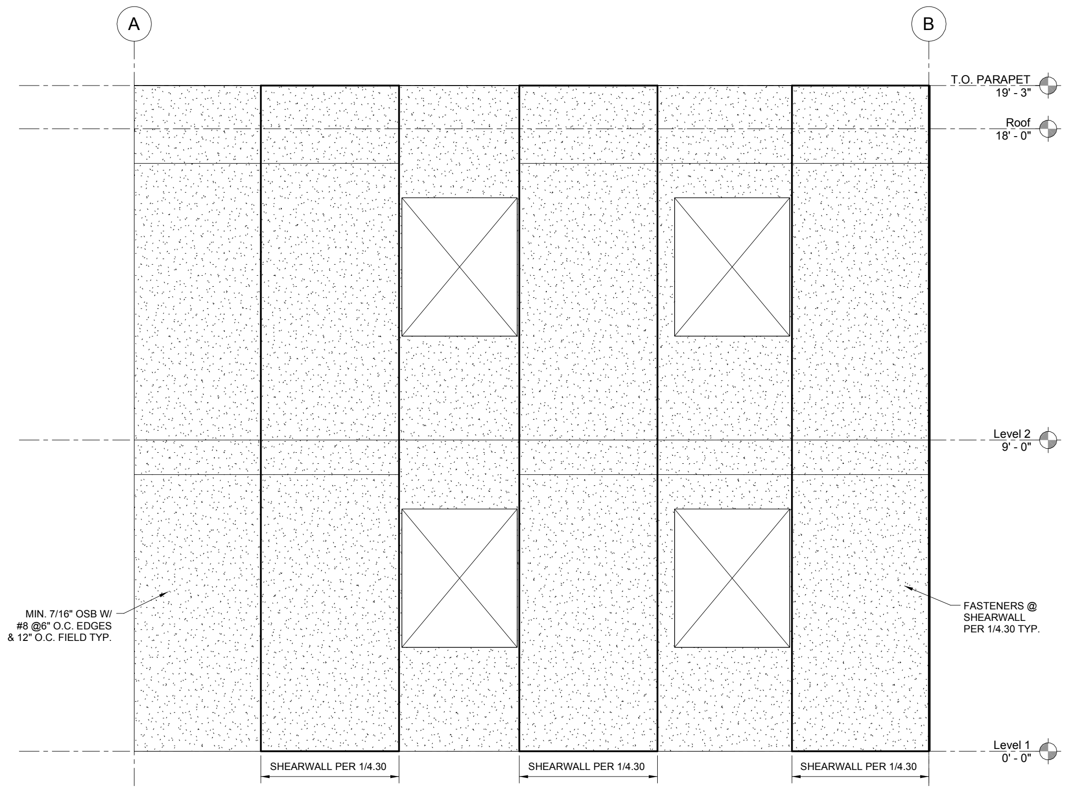
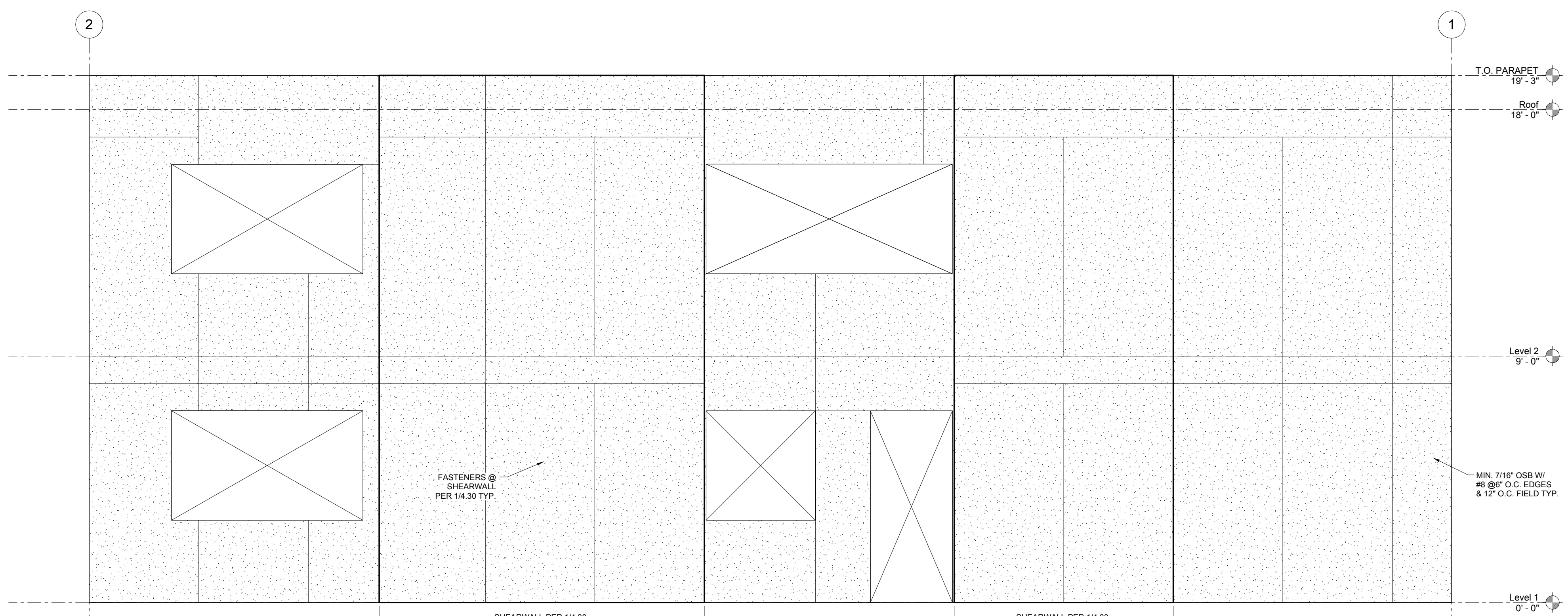
SHEATHING ELEVATIONS

JOB NO.
DRAWN BY:
DRAWING:
SF 5.10



2 SHEATHING ELEVATION - WEST
1/2" = 1'-0"



1 SHEATHING ELEVATION - NORTH
1/2" = 1'-0"

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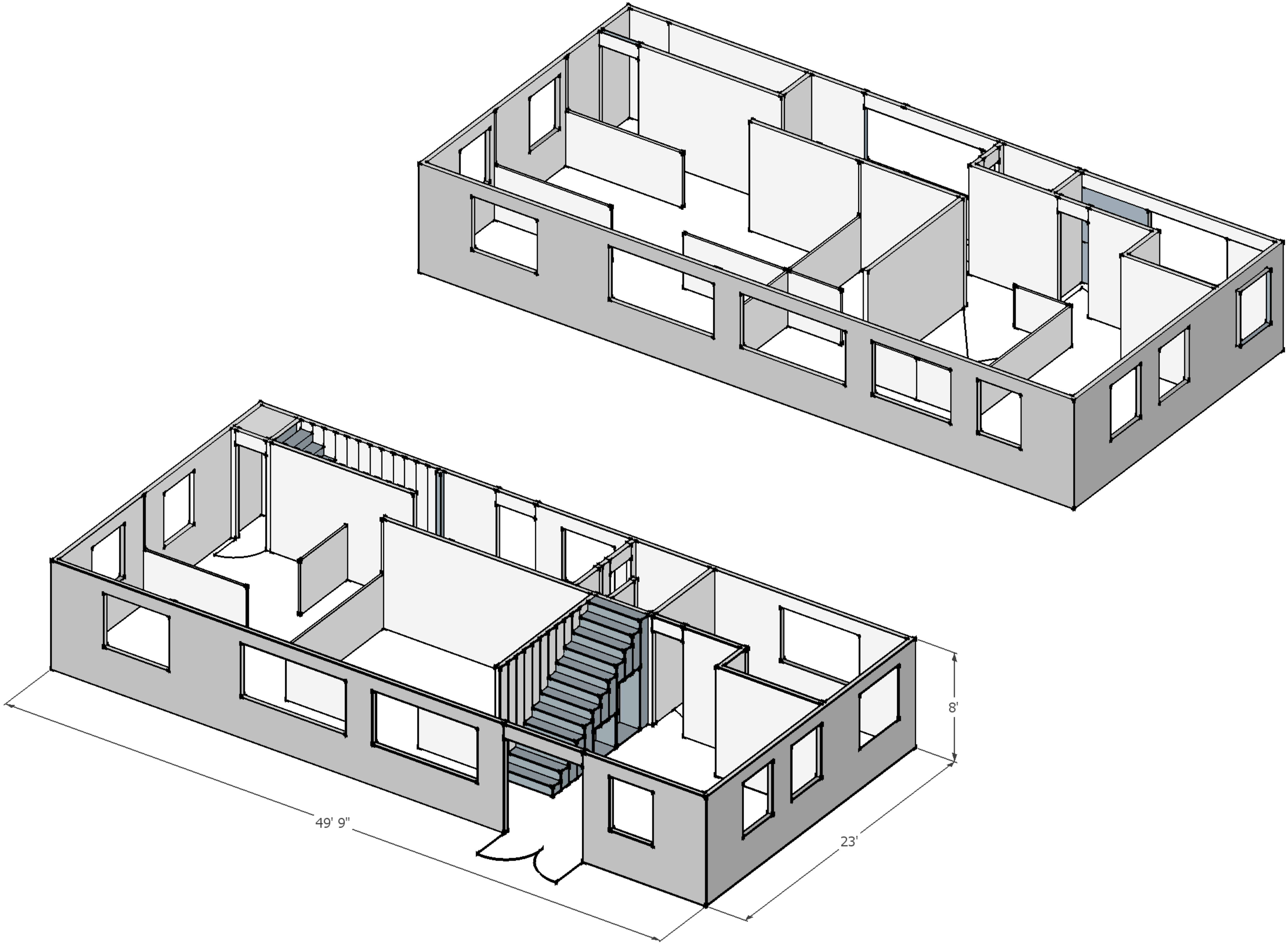
Appendix 6

Architectural Concept



8'
49' 9"
23'

Appendix 7

Rigid Diaphragm Analysis

As discussed in the design narrative, the lateral force resisting system was designed based on an idealized flexible diaphragm. This is consistent with ASCE 7-10, section 12.3.1.1 which states that “Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:” Condition c of this provision includes structures of light-frame construction where there is no concrete or similar topping over wood structural panels and where all of the shearwalls meet the drift requirements of ASCE 7-10, Table 12.12-1. The structure considered herein meet these criteria.

However, based on industry input and for purposes of comparison, an idealized rigid diaphragm analysis was performed. The rigid diaphragm analysis was performed using spreadsheets to determine the load distribution to each shearwall. The analysis included the calculated offset between center of mass and center of stiffness as well as an additional 5% offset for ‘Accidental Torsion’ per ASCE 7-10, section 12.8.4.2.

The shearwall stiffness was estimated as the stiffness calculated for the flexible diaphragm design. Note that this stiffness is an estimate only and could be fine tuned via an iterative procedure to produce more precise results.

The analysis shows that the significant stiffness difference between the North and South shearwall lines results in much higher forces in the North shearwall. The approximately 41% increase in force, however, does not appear to overstress the shearwall or its anchorages.

For the East and West shearwalls, the real and accidental torsion result in a maximum 7.4% increase in shearwall forces. This increase does not appear to overstress the shearwalls or their anchorages.

The rigid diaphragm analysis is presented below.

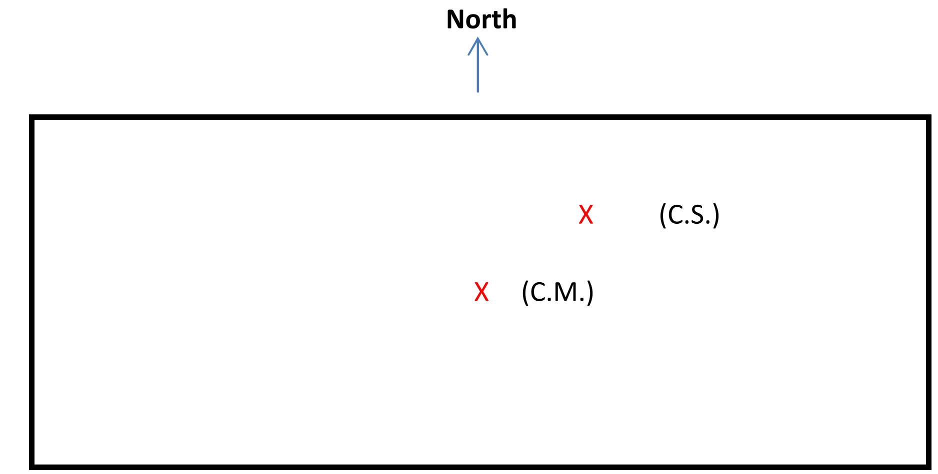
Rigid Diaphragm Analysis

Note: Shearwall stifness is based on the calculated deflection at the applied load based on flexible diaphragm analysis.
Actual stiffness is non-linear and would require an iterative solution to find an exact solution.

Roof Diaphragm

Upper Level SW	b (ft)	v(lb/ft)	V (lb)	Δ (in)	Overall Stiffness (k/ft)	Relative Stiffness
L2S1	4	241	962	0.282		
L2S2	5	267	1337	0.287	138	0.141
L2S3	4	241	962	0.282		
L2N1	12	174	2088	0.089	440	0.450
L2N2	8	147	1174	0.088		
L2W1	4	193	773	0.204		
L2W2	4	193	773	0.204	191	0.196
L2W3	7	245	1716	0.204		
L2E1	6	217	1305	0.187	208	0.213
L2E2	8	245	1957	0.188		

Building Width	23	(ft)
Building Length	49.75	(ft)
Center of Stiffness:	5.5	ft. from North Wall
	23.8	ft. from East Wall
Center of Mass	11.5	ft. from North Wall
	24.88	ft. from East Wall
5% accidental offset	1.15	in N-S direction
	2.49	in E-W direction



Shearwall Polar Moment of Inertia

Side	South	North	West	East
Dist to CS	17.5	5.5	25.9	23.8

J_{SW} = 302541 (k-ft)

E-W Accelerations

V = 6524 (lb)

M_t = 46.76 (ft-k)

Maximum Lever Arm (CM to CS) + 5% Offset = 7.17 (ft)

Resultant ShearWall Forces

SW Line	South	North	West	East
Force (lb)	1929	4595	-767	767
Rigid/Flex	**0.591**	**1.409**	**NA**	**NA**

(equals ratio of rigid diaphragm force along noted line to flexible force along same line)

N-S-W Accelerations

V = 6524 (lb)

M_t = 23.12 (ft-k)

Maximum Lever Arm (CM to CS) + 5% Offset = 3.54 (ft)

Resultant ShearWall Forces

SW Line	South	North	West	East
Force (lb)	185	-185	3503	3021
Rigid/Flex	**NA**	**NA**	**1.074**	**0.926**

(equals ratio of rigid diaphragm force along noted line to flexible force along same line)

Floor Diaphragm

Upper Level SW	b (ft)	v(lb/ft)	V (lb)	Δ (in)	Overall Stiffness (lb/in)	Relative Stiffness
L1S1	4	408	1631	0.490		
L1S2	5	453	2267	0.496	135	0.137
L1S3	4	408	1631	0.490		
L1N1	12	295	3539	0.147	451	0.459
L1N2	8	249	1991	0.148		
L1W1	4	326	1305	0.353		
L1W2	4	326	1305	0.353	189	0.192
L1W3	7	417	2920	0.350		
L1E1	6	369	2212	0.320	208	0.212
L1E2	8	415	3318	0.319		

Center of Stiffness:	5.3	ft. from North Wall
	23.7	ft. from East Wall
Center of Mass	11.5	ft. from North Wall
	24.88	ft. from East Wall
5% accidental offset	1.15	ft. from North Wall
	2.49	ft. from East Wall

Shearwall Polar Moment of Inertia

Side	South	North	West	East
Dist to CS	17.7	5.3	26.1	23.7

J_{SW} = 299706 (k-ft)

E-W Accelerations

V = 4537 (lb)

M_t = 33.38 (ft-k)

Maximum Lever Arm (CM to CS) + 5% Offset = 7.36 (ft)

Resultant ShearWall Forces

SW Line	South	North	West	East
Δ Force (lb)	1310	3227	-548	548
Total	3238	7822	-1315	1315
Rigid/Flex	**0.586**	**1.414**	**NA**	**NA**

(equals ratio of rigid diaphragm force along noted line to flexible force along same line)

N-S-W Accelerations

V = 4537 (lb)

M_t = 16.76 (ft-k)

Maximum Lever Arm (CM to CS) + 5% Offset = 3.69 (ft)

Resultant ShearWall Forces

SW Line	South	North	West	East
Δ Force (lb)	134	-134	2434	2103
Total	318	-318	5936	5124
Rigid/Flex	**NA**	**NA**	**1.073**	**0.927**

(equals ratio of rigid diaphragm force along noted line to flexible force along same line)