



Roof Structure Design Guide

Patrick A. Bodwell

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Verco Decking, Inc.

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Acknowledgments

This design example is the culmination of more than a decade of effort to document a set of best practices based on conversations with steel fabricators, erectors, contractors, and engineers. Many great professionals have contributed suggestions to create the most economical solutions to solve critical design questions.

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1.0 Introduction

Steel roof deck and open web steel joist is the predominate roof system in the United States for large flat roof structures. This system is the preferred choice of developers for its simplicity, strength, fire resistance, and economy. Large distribution warehouses, big-box retail, industrial and single-story commercial buildings all benefit from a steel deck and joist roof structure.

This steel roof deck and joist design example offers guidance for the design professional based on current best practices in order to provide the best possible safe and economical steel roof deck and open web steel joist roof structure. The combinations of products illustrated in this example may not be the optimum for every project, however the underlying methods provide a basis to optimize many other combinations of steel roof deck and joists.

1.1 Design Example

The design example walks through the design of a modest 300 ft x 504 ft warehouse structure starting with basic fire life-safety and continuing through the structural design of the steel roof deck and open web steel joists and girders. The seismic design of the diaphragm and wall anchorage are followed by a wind load analysis to ensure that the building design based on seismic requirements is capable of resisting the wind loading. The resulting design is summarized as a set of roof structural plans, notes, schedules, and details in the Appendix.

Factory Mutual roof assembly considerations are applied to the example building to demonstrate the impact of Factory Mutual compliance on the design. Factory Mutual applies wind load conditions and detailing that goes beyond the minimum life-safety requirements of the building code, with the goal of mitigating property loss.

Very large roof structures require additional design considerations to distribute the design loads and account for the effects of thermal expansion. The design methods used for the modest design example structure are applied to very large roof structures in combination with consideration of thermal expansion. This will demonstrate that thermal expansion joints may be eliminated in many large roof structures through the use of a ductile steel roof diaphragm system. The elimination of thermal expansion joints in the diaphragm simplifies the design process, while providing a safer and more efficient building.

1.2 Cold-Formed Steel Deck Calculations

The design example takes advantage of both Verco and Vulcraft web-based design tools to determine the strength of the steel roof deck and open web steel joist tie plates. These web-based solutions are a large step forward from the catalogs of static load tables historically used for the design of steel roof deck. Summary design tool output tables are inserted in the design example where steel deck strength is required. Readers are encouraged to utilize the web-based design tools referenced throughout the design example to minimize design effort and maximize project economy by creating project specific product data sheets for their next roof structure design.

1.3 Codes and Reference Documents

This example follows the provisions of the International Building Code and referenced standards in force at the time of writing. In addition, the Verco IAPMO-UES product evaluation report is used as the basis of recognition for proprietary products and design methods. The primary reference documents used are:

International Building Code (2018 IBC)

American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures (ASCE 7-16)

American Iron and Steel Institute North American Specification for the Design of Cold-formed Steel Structural Members (AISI S100-16)

Wind Design, 2015 Interim Revision February 2020 (FM 1-28)

Roof Deck Securement and Above-Deck Roof Components January, 2016 Interim Revision February 2020 (FM 1-29)

Approval Standard for Profiled Steel Panels for Use as Decking in Class 1 Insulated Roof Construction, June 2012 (Class Number 4451)

Verco Decking, Inc. IAPMO-UES Evaluation Report (ER-2018))

Steel Deck Institute Standard for Steel Roof Deck (RD-2017)

Steel Deck Institute Manual of Construction, 3rd Edition (MOC3, 2016))

Steel Joist Institute Standard Specifications, 44th Edition (SJI 100-2015)

2.0 Warehouse Building

This design guide addresses a typical mid-sized warehouse building, with concrete tilt-up walls, tube steel columns, open web steel joists & joist girders, and a steel deck roof diaphragm. The principles applied to the design of the roof deck for this structure have direct parallels with similar buildings constructed with off-site pre-cast wall panels, concrete unit masonry walls, and steel (brace) frame systems. The building can be summarized as follows:

| | |
|---------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Building Use: | Warehouse/rack storage and/or manufacturing with optional office space. |
| Roof Structure: | Steel Roof Deck supported by open web steel joists and open web steel joist girders bearing on concrete tilt-up panels and tube steel columns with a minimum 1/4 inch per foot slope to mitigate ponding considerations. |
| Wall Structure: | Cast on-site concrete tilt-up wall panels. |
| Seismic Force Resisting System: | Intermediate precast shear walls (bearing wall system, Reference A5 in ASCE 7 Table 12.2-1) supporting a flexible steel roof deck diaphragm. |
| Fire Protection System: | Automatic sprinkler system. |
| Site Conditions: | 60 foot side-yard clearance on all four sides of building. |

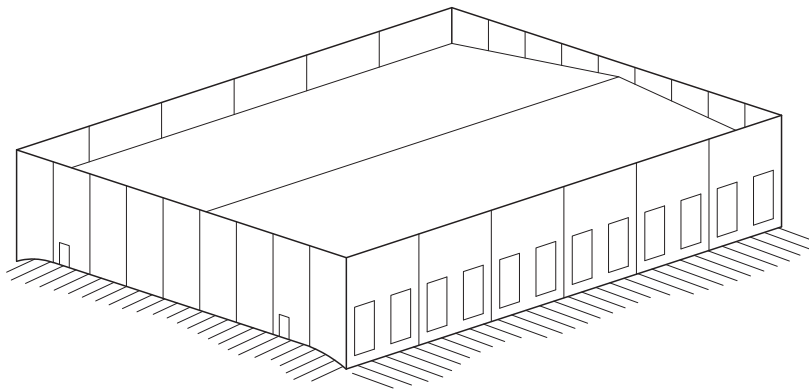
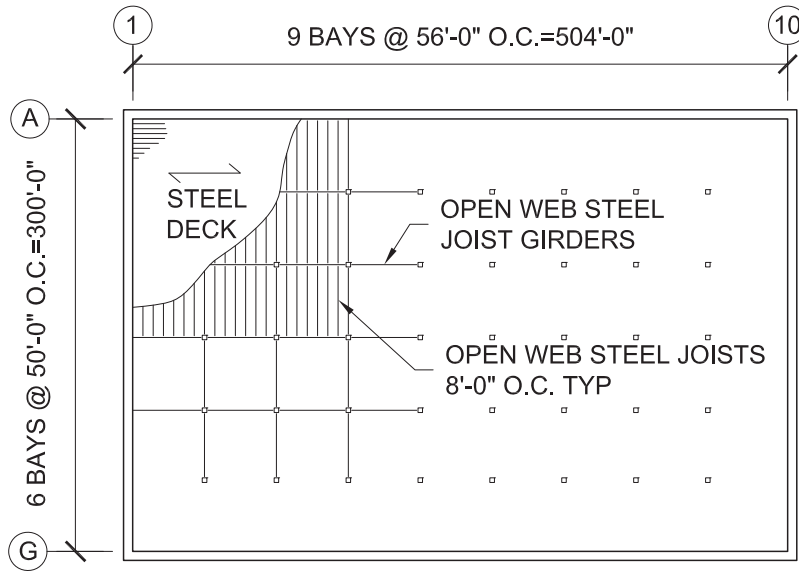


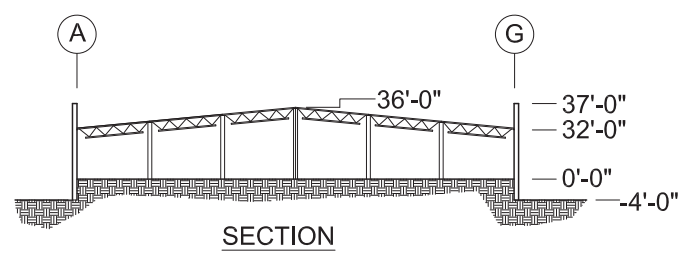
Figure 2.1 Warehouse Building

The warehouse building is based on bay sizes and framing layouts that have been proven to provide an economical balance of structural performance while providing efficient joist spacing for the automatic fire sprinkler systems. The bay size for this example warehouse is 50 ft x 56 ft. The 50 ft long open web steel joists are spaced at a uniform 8 feet o.c. across the building, bearing on the 56 ft long open web steel joist girders. The roof height ranges from 32 ft to 36 ft with a 37 ft parapet height.



ROOF PLAN VIEW

Figure 2.2 Warehouse Building Plan View



SECTION

Figure 2.3 Warehouse Building Cross Section

2.1 Dead Loads

Roof Structure Dead Load:

| | | |
|--------------------------------------------|----------|--|
| Roofing System + re-roof (3 psf + 2 psf) | 5.0 psf | |
| Steel deck | 3.0 psf | |
| <hr/> | | |
| Subtotal for Steel Deck and Roofing System | 8.0 psf | |
| Automatic Sprinklers | 2.0 psf | |
| Miscellaneous | 1.5 psf | |
| Open Web Steel joists | 2.0 psf | |
| <hr/> | | |
| Subtotal for Open Web Steel Joists | 13.5 psf | |
| Open Web Steel Joist Girders | 1.5 psf | |
| <hr/> | | |
| Total Dead Load | 15.0 psf | |

Roof Structure Live Load

| | |
|------------------------|---------------------------------------|
| Uniform Roof Live load | 20.0 psf (reducible) |
| Concentrated Live Load | 300 lbs (over a 2.5 ft x 2.5 ft area) |

Snow Load

Does not apply at this site

Wall Dead Load

| | |
|------------------------|---------|
| Normal Weight Concrete | 150 pcf |
| Wall Thickness | 9¼ inch |
| Wall Weight | 116 psf |

2.2 Deflection Criteria

Building deflection is primarily a serviceability consideration. Deflections are limited for warehouse type structures to prevent damage to non-structural elements of the structure.

The roof structure deflection is due to vertical gravity and wind loads. IBC Table 1604.3 allows for roof structures not supporting a ceiling to deflect up to L/180, however it requires a more stringent L/240 for roof members supporting non-plaster ceilings. Many warehouse structures end up with a portion of the building containing office space with a ceiling. For this reason, a L/240 deflection limit will be set for the joists and girders that may support the future ceiling and the more liberal L/180 for the steel deck that will not support the weight of the future ceiling.

| Member | Joists & Girders | Steel Deck |
|----------------------------|------------------|------------|
| Live Load Deflection Limit | L/240 | L/180 |
| Wind Load Deflection Limit | L/240 | L/180 |

Seismic lateral deflection, building drift, is primarily a structural consideration. When a structure is subject to the design level earthquake it is acceptable for there to be damage to nonstructural elements of the structure, provided that life safety is not compromised. The primary deflection requirement is to limit the shear deflection of the diaphragm to prevent a progressive collapse of the primary structure. To prevent collapse, the P-delta stability coefficient will not exceed 0.10 following the requirements of ASCE 7 Section 12.8.7.

| | |
|----------|------|
| PΔ limit | 0.10 |
|----------|------|

Lateral wind deflection is service level deflection criteria similar to vertical loads. The IBC does not provide a specific requirement. Guidance may be taken from ASCE 7 Appendix Commentary Section CC.2.2 that recommends that a vertical deflection limit between L/600 and L/400 is generally an acceptable range for most structures. For this warehouse structure, the more liberal L/400 deflection limit will be set.

| | |
|---------------------------------|-------|
| Wind Diaphragm Deflection Limit | L/400 |
|---------------------------------|-------|

2.3 Seismic and Wind Parameters

Ontario, California is the location selected for this design example. The following seismic and wind parameters are for this location. These parameters are discussed further in Sections 5 and 6.

Seismic Parameters

$S_s = 1.5g$ (short period)

$S_1 = 0.6g$ (1-second period)

Risk Category II

Site Class D

Wind Parameters

$V = 95$ mph basic wind speed

$K_d = 0.85$ Wind directionality factor

$K_{zt} = 1.0$ Topographic Factor

$K_e = 1.0$ Topographic Factor

Risk Category II

Exposure Category C

Enclosure Classification = Enclosed

3.0 Building Size Limitations

Building area and height limits have been established for all building types to ensure basic life safety in the designs. The requirements are set forth in the provisions of the IBC for general occupancy egress and fire considerations. ASCE 7 also includes limitations for life safety in seismic events based on the lateral system used.

3.1 Area and Height Limits for Fire and Life Safety

The IBC limits the area and height of a building based on a combination of the occupancy type, fire resistance for the building materials and the fire protection system. The common occupancy types that this example building would fall under are storage (warehouse) and/or manufacturing. In addition, a portion of the building is likely to have business (office) usage. Mercantile (retail) often use this same building type.

The last factor to consider to determine the fire and life safety requirements for the structure is whether or not automatic fire sprinklers are used for fire suppression in the building. The building that is part of this example does have an automatic fire sprinkler system.

The occupancy category is determined in accordance with Chapter 3 of the IBC.

The following possible occupancies have been identified for this example:

| | |
|-----------------------------------|------------|
| Moderate Hazard Storage Group S-1 | IBC §311.2 |
| Moderate Hazard Factory Group F-1 | IBC §306.2 |
| Business Group B | IBC §304.1 |
| Mercantile Group M | IBC §309.1 |

For this example, all of the primary structural members are non-combustible by nature, including the concrete walls, steel columns, steel joists and steel deck. However, in the installed condition without a fire proofing coating, the steel does not meet the requirements for a fire rated assembly, therefore it qualifies for a Type II-B rating.

| | |
|----------------------------------|------------------------|
| Fire Resistance Rating Type II-B | IBC §602.2 & Table 601 |
|----------------------------------|------------------------|

Based on the fire resistance rating Type II-B and occupancy categories B, F-1, M, and S-1, the following building limitations are established:

| | |
|------------------------------------------------------------------------|-----------------|
| 55 ft maximum height, not sprinklered | IBC Table 504.3 |
| 75 ft maximum height, sprinklered (controls for this example building) | |

The clearances around the building will also impact the allowable building area. For this example building, there is 60 foot side yard clearance on all 4 sides of the building.

1-story with automatic sprinklers and 60 foot public ways or yards, Groups B, F, M or S:

| | |
|-----------------------------------------------------|----------------|
| Unlimited Area (controls for this example building) | IBC per §507.4 |
|-----------------------------------------------------|----------------|

2-story with automatic sprinklers and 60 foot public ways or yards, Groups B, F, M or S:

| | |
|----------------|----------------|
| Unlimited Area | IBC per §507.5 |
|----------------|----------------|

The warehouse building in this example is checked to determine if it meets the fire and life safety limitations of the IBC in Section 504.3. The building height is measured from grade to the average roof height. In this case the floor level will be assumed to be 4 ft above grade for loading docks around the warehouse perimeter with a 34 ft average roof height above the floor level. See Figure 2.3 Building Cross Section depicting the elevation of the roof structure relative to the floor level.

Area: 504 ft x 300 ft = 151,200 sf < Unlimited area, therefore acceptable

Height: 34 ft + 4 ft = 38 ft < 75 ft, therefore acceptable

Stories: $1 \leq 1$ or 2 stories for unlimited area, therefore acceptable

Chapter 5 of the IBC provides the complete requirements for maximum area, height, and number of stories for buildings that do not meet the requirements for an unlimited area building.

3.2 Height Limits for Structural Seismic Design

The determination of the building height for seismic life safety design is different than that for fire and life safety design in the IBC. For seismic design the height is taken from the base of the structure to the top of the seismic force resisting system. ASCE 7 defines the base of the structure as “the level at which horizontal seismic ground motions are considered to be imparted on the structure”. The common convention is to tie the wall panels into the concrete floor slab. The floor slab therefore defines the elevation at which the seismic load is transferred into the building.

The Seismic Design Category (SDC) assigned to the building is a primary limiting factor of the maximum building height. This warehouse structure with intermediate precast shear walls, bearing wall system as the seismic force-resisting system, will typically be assigned a SDC of D or higher in high seismic regions. The determination of the SDC is covered in Section 5 of this example. For structures with a SDC of D or higher the maximum building height is limited to 40 feet in ASCE 7 Table 12.2-1. There is an allowance made in Footnote i of the table, increasing the height to 45 feet for single story storage warehouse facilities. The increased 45 ft limit allowance for warehouse structures is within the height limits for storage buildings with Early Suppression Fast Response (ESFR) sprinkler systems.

Height: 34 ft \leq 45 ft, therefore acceptable

ASCE 7 Table 12.2-1(A5)

4.0 Roof Structure Vertical Load Design

Steel roof structures support vertical loads including dead loads including the self-weight of the roof structure, fire sprinkler systems, miscellaneous mechanical/electrical systems, roof live loads, and wind loads. The roof structure may also support snow loads, depending on building location, although that is not applicable to this example building located in Southern California. The load resisted by each member of the roof structure will vary based on the load combination it supports. The steel deck will support its self-weight, the roof assembly, roof live load, and wind load. The open web joist girders support the loads imposed by the steel deck in addition to their self-weight, mechanical systems, fire sprinkler system, and architectural finishes such as ceilings.

The load to each member can be determined using load combinations from one of two methods, Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD). This example uses the ASD load combinations in ASCE 7-16 Section 2.4.1 for the vertical design. The governing load combinations are then used to help determine the appropriate steel deck type and gage and deck fastening system. The load combinations will also be used to specify the appropriate open web steel joists and joist girders.

4.1 Steel Deck Vertical Load Design

The design of steel deck to support vertical loads has several parts that include developing the design loads, checking the strength of the steel deck, and checking deflection for serviceability. This involves determining the dead load, the appropriate live load, calculating the wind loads, and determining the governing load combinations.

The steel deck provides vertical support for the weight of the roof system, roof live loads, and wind loads. For this warehouse building example, the steel deck will not be used to support the miscellaneous ceiling, mechanical, electrical, and fire sprinkler system loads. The various miscellaneous loads mentioned above will be supported by the open web steel joists, open web steel joist girders, and miscellaneous steel members when needed. The design of the deck to resist in-plane seismic or wind driven diaphragm shear loads is addressed in Sections 5 and 7 respectively.

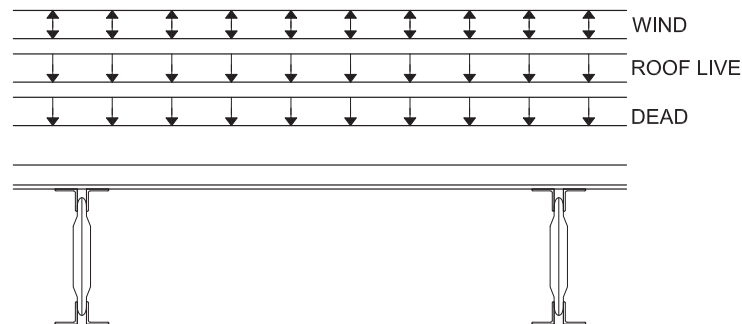


Figure 4.1 Vertical Uniform Loading

The steel deck panels support the uniform vertical dead and live load. The panels act as a series of beams spanning between the open web steel joist framing. The resolution of forces on the steel deck is developed using engineering mechanics for a slender simple or multi-span beam as shown in Figure 4.2.

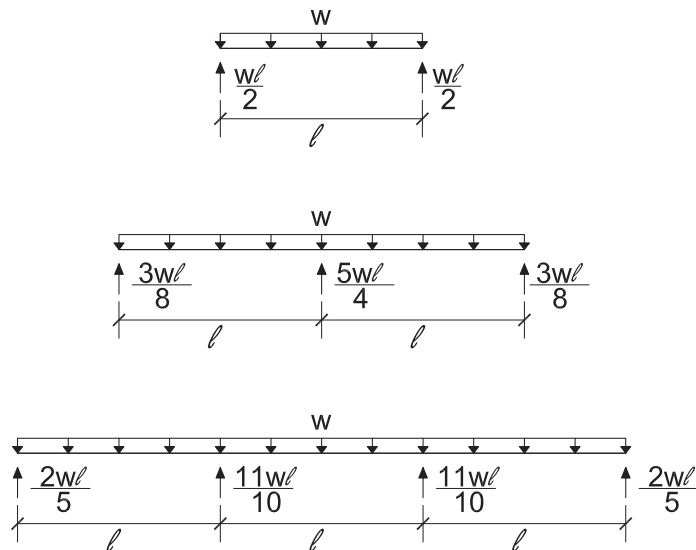


Figure 4.2 Uniform Load Reactions at Supports for 1, 2, and 3 Span Beams

4.1.1 Steel Deck Dead and Roof Live Loads

The steel roof deck dead loads developed in Section 2 are summarized as follows. They are separated for inward wind loading and those to be combined with wind uplift. The dead loads for resisting wind uplift do not consider the allowance for re-roofing that will not be present until a re-roof is added. The minimum gage steel deck self-weight is used for the uplift case to account for areas of the roof with the lightest gage steel roof deck. This reduces the dead load to resist the uplift forces on the steel roof deck. A slightly heavier allowance is used for the steel deck for inward loading to account for roof areas that may have heavier than the minimum gage steel roof deck.

Steel Roof Deck Dead Loads (from Section 2):

| | For Inward Loading | For uplift loading |
|---------------------------------|--------------------|--------------------|
| Roof System | 3.0 psf | 3.0 psf |
| Re-roof | 2.0 psf | 0 psf |
| Steel deck | 3.0 psf | 2.0 psf |
| Dead Load for steel deck | 8.0 psf | 5.0 psf |

Steel Roof Deck Live Loads:

| | |
|------------------------|------------------------------------|
| Uniform Roof Live load | 20.0 psf |
| Concentrated Live Load | 300 lb (over a 2½ ft x 2½ ft area) |

Table 4.1 Roof Deck Dead Loads

4.1.2 Wind Loads

Buildings resist both main wind force resisting system (MWFRS) and the components and cladding (C&C) loads. It is unlikely that wind will govern the design for this example building located in Ontario, California, in a high seismic region with a low design wind speed. Even with the assumption that wind will not govern the design of the structure, wind loading will be checked to ensure wind will not govern the design.

This example warehouse building qualifies as a low rise building as defined in ASCE 7 Section 26.2. The building is considered low rise because the building mean roof height does not exceed 60 ft and mean roof height is less than the least horizontal dimension of the building. The mean roof height for wind design for flat roof structures, with $\theta \leq 5^\circ$, is measured from the ground surface adjacent to the building to the eave height.

$$h = 38 \text{ ft (see Figure 2.3)} < 60 \text{ ft} \qquad \text{ASCE 7 §26.2}$$

$$h = 38 \text{ ft} < 300 \text{ ft} = \text{least horizontal dimension}$$

As a low-rise building, the Components and Cladding loading is in accordance with ASCE 7 Section 30.3, and the MWFRS loads may be determined using the envelope procedure in ASCE 7 Chapter 28.

4.1.2.1 Wind Load Parameters

For low rise buildings, both the main wind force resisting system loads, and the components and cladding loads begin with the basic building wind parameters in ASCE 7 Chapter 26. The steps to determine these loads are outlined in ASCE 7 Table 28.2-1, for the Main Wind Force Resisting System, and Table 30.3-1 for Components and Cladding. Both methods are the same for Steps 1 through 5.

Step 1: The warehouse is assigned a Risk Category of II for structures in accordance with ASCE 7 Table 1.5-1 because the occupancy does not meet the requirements to be a Category I, III, or IV occupancy for low risk, substantial risk for human life, or essential facilities.

$$\text{Risk Category} = \text{II} \qquad \text{ASCE 7 Table 1.5-1}$$

Step 2: For this example, the building site is in Ontario, California, which is in a low wind speed region. The basic wind speed is taken from ASCE 7 Figure 26.5-1B.

$$V = 95 \text{ mph} \qquad \text{ASCE 7 Figure 26.5-1B}$$

Step 3: For this example, the wind parameters related to the site will be assumed as follows:

$$\text{Wind directionality factor, } K_d = 0.85 \qquad \text{ASCE 7 Table 26.6-1}$$

$$\text{Exposure Category} = \text{C} \qquad \text{ASCE 7 §26.7}$$

$$\text{Topographic Factor, } K_{zt} = 1.0 \qquad \text{ASCE 7 §26.8}$$

$$\text{Ground Elevation Factor, } K_e = 1.0 \qquad \text{ASCE 7 §26.9}$$

$$\text{Enclosure Classification} = \text{Enclosed} \qquad \text{ASCE 7 §26.12}$$

$$\text{Internal Pressure Coefficient, } GC_{pi} = \pm 0.18 \qquad \text{ASCE 7 Table 26.13-1}$$

Step 4: The velocity pressure coefficients for the roof are determined in accordance with ASCE 7 Table 26.10.1 based on the mean roof height and exposure category or using the formulas in the footnotes of the table. To determine the pressure coefficients, the nominal height of the atmospheric boundary layer, z_g , and the 3-sec gust-speed power law exponent, α , are taken from ASCE 7 Table 26.11-1. These factors are the same for both the MWFRS and C&C loads.

$$z_g = 900 \text{ ft} \qquad \text{ASCE 7 Table 26.11-1}$$

$$\alpha = 9.5 \qquad \text{ASCE 7 Table 26.11-1}$$

For $15 \text{ ft} \leq z \leq z_g$ where, $z = h = 38 \text{ ft}$

$$K_h = K_z = 2.01(z/z_g)^{(2/\alpha)} = 2.01(38/900)^{(2/9.5)} = 1.03 \qquad \text{ASCE 7 Table 26.10-1}$$

Step 5: The velocity pressure is then determined for $z = h = 38$ ft, the mean roof height used for the gable roof.

$$q_h = 0.00256K_zK_{zt}K_dV^2 = 0.00256(1.03)(1.0)(0.85)(95)^2 = 20.3 \text{ psf} \quad \text{ASCE 7 eq. 26.10-1}$$

4.1.2.2 Roof Structure Components and Cladding Zones

The development of the wind load methods for the MWFRS and C&C diverge at Step 6 of the wind pressure determination. The pressure Zones for the external pressure coefficients (GC_p and GC_{pf}) are different for C&C compared to the MWFRS.

Step 6: The external pressure coefficient GC_p for the steel roof deck for this example is developed from Section 30.3 Low-Rise Buildings. This structure is enclosed with a gable end roof therefore the external pressure coefficients are derived from ASCE 7 Figure 30.3-2A. GC_p varies depending on the Zone of the roof with lower outward pressures in the field and progressively higher pressures at the edges and corners of the roof. GC_p also varies with the effective wind area to the component. The width, a , of the edge and corner zones is based on the height of the building and the least width of the building.

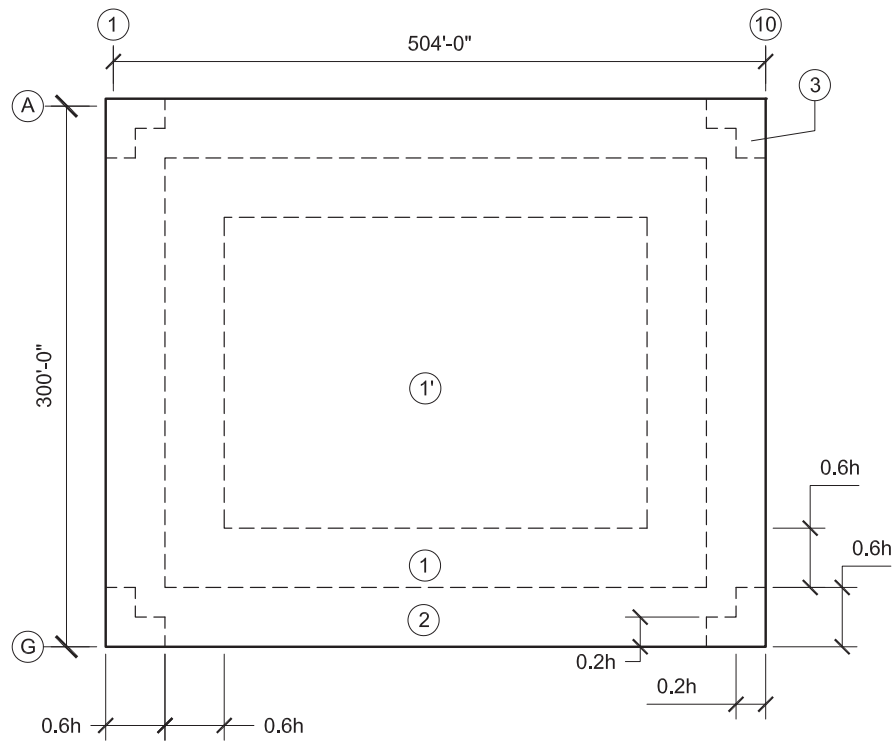


Figure 4.3 Components and Cladding Zones
Reference ASCE Figure 30.3-2A

Zone Dimensions:

ASCE 7 Figure 30.3-2A

$0.6h = 22.8$ ft, therefore use 23 ft width for Zone 1 & 2

$0.6h = 22.8$ ft, therefore use 23 ft length for Zone 3

$0.2h = 7.6$ ft, therefore use 8 ft width for Zone 3

4.1.2.3 Steel Roof Deck Components and Cladding Wind Pressure

Step 6 (continued): The next part of Step 6 is to determine the effective wind area for the steel deck. This area is required to look up the value of GC_p in the diagrams in ASCE 7 Figure 30.3-2A.

The Effective Wind Area is defined in ASCE 7 Section 26.2 as follows.

EFFECTIVE WIND AREA, A: The area used to determine the external pressure coefficient, (GC_p) and (GC_m). For C&C elements, the effective wind area in Figures 30.3-1 through 30.3-7, 30.4-1, 30.5-1, and 30.7-1 through 30.7-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For rooftop solar arrays, the effective wind area in Fig. 29.4-7 is equal to the tributary area for the structural element being considered, except that the width of the effective wind area need not be less than one-third its length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

In this example 1½ in deep PLB-36 steel roof deck with a 36 in coverage width will be supported by open web steel joists at 8 ft on center. The 1½ in deep deck is the most efficient profile for the 8 ft joist spacing and the 36 in width develops higher diaphragm shears for a given attachment pattern compared to narrower 3 in deep roof deck.

To determine the pressure coefficients for the wind load on the steel deck, the effective wind area for the steel roof deck is determined.

$L = 8$ ft, steel deck span between joists

$w = 3$ ft, steel deck sheet width

$w \geq L/3 = 8 \text{ ft}/3 = 2.67$ ft, therefore use 3 ft

Effective Wind Area of steel deck:

$$A_e = L(w) = 8 \text{ ft}(3 \text{ ft}) = 24 \text{ sf}$$

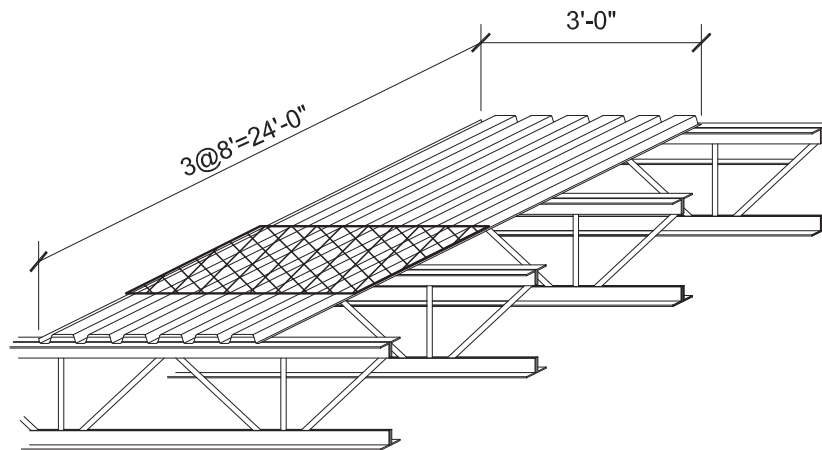


Figure 4.4 Effective Wind Area of Deck

The effective wind area for the steel deck is used to determine the external pressure coefficients (GC_p and GC_{pf}) from the chart in ASCE 7 Figure 30.3-2A. These can be read directly from the chart or can be calculated using the underlying equations that developed the chart which are presented in the commentary. An important consideration for this project is the effect of parapets. For low slope roofs with parapets there is some relief for the high wind uplift pressures in the corner Zone 3. Note 5 in ASCE 7 Figure 30.3-2A provides the basis for this reduction in wind pressure due to parapets.

ASCE 7 Figure 30.3-2A Note 5:

If a parapet equal to or higher than 3 ft is provided around the perimeter of the roof with $\theta \leq 7^\circ$, the negative values of (GC_p) in Zone 3 shall be equal to those for Zone 2, and positive values of (GC_p) in Zone 2 and 3 shall be set equal to those for wall Zones 4 and 5 respectively, in Figure 30.3-1

The top of wall in this example is 37 ft high around the entire structure. On lines A and G the roof elevation is 32 ft creating a 5 ft high parapet. At Lines 1 and 10 the roof elevation slopes from 36 ft at the ridge to 32 ft at the corner. The parapet at on lines 1 and 10 is 3 ft or greater in height until the roof height exceeds 34 ft. Based on the geometry this transition occurs 75 ft from the corner as shown in Figure 4.5. The corner Zone 3 is 23 ft long which falls completely within the length of the parapet exceeding 3 ft high, therefore the relief to use the lower Zone 2 pressure in Zone 3 per Note 5 is appropriate.

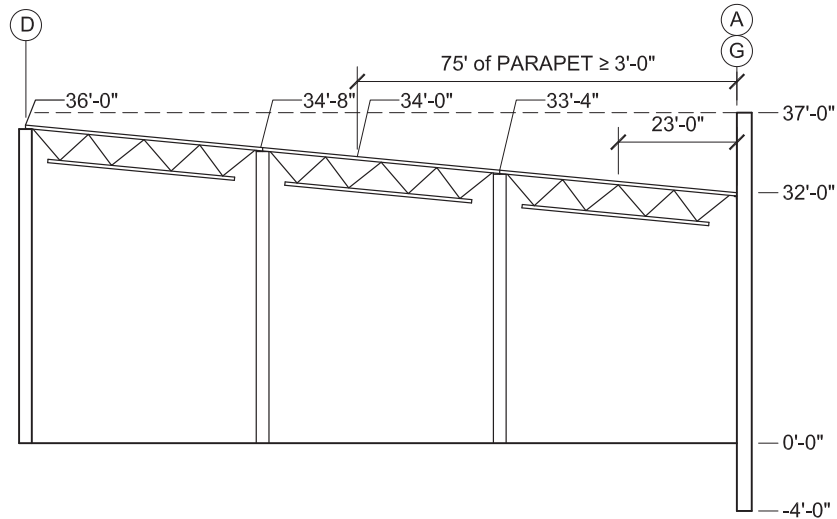


Figure 4.5: Parapet Height

Steel Roof Deck External Pressure Coefficients

ASCE 7 Figure 30.3-2A

| Zone | 1' | 1 | 2 | 3 | 1' & 1 | 2 & 3 |
|--------|-------|-------|-------|-------|--------|-------|
| GC_p | -0.90 | -1.54 | -2.10 | -2.10 | +0.26 | +0.93 |

Table 4.1 Roof Deck C&C External Pressure Coefficients, GC_p

Step 7: The wind design pressure, p , for the steel deck is then determined based on the velocity pressure, internal pressure coefficient, and external pressure coefficients in accordance with ASCE 7 Section 30.3.2. The wind design pressure, p , shall not be less than 16 psf in accordance with ASCE 7 Section 30.2.2.

$$p = q_h [(GC_p) - (GC_{pi})]$$

ASCE 7 eq. 30.3-1

For Zone 1:

$$-p_1 = 20.3 [(-1.54) - (-0.18)] = -34.9 \text{ psf (outward)}$$

$$+p_{1' \& 1} = 20.3 [(+0.26) - (-0.18)] = +8.9 \text{ psf (inward)} \leq +16 \text{ psf, therefore } +16 \text{ psf governs}$$

$$+p_{2 \& 3} = 20.3 [(+0.93) - (-0.18)] = +22.6 \text{ psf (inward)} > +16 \text{ psf, therefore } +22.6 \text{ psf governs}$$

Zone 1', 2, and 3 outward wind pressures are determined in a similar manner and are summarized in Table 4.2.

| Surface | 1' | 1 | 2 | 3 | 1' & 1 | 2 & 3 |
|---------|-------|-------|-------|-------|--------|-------|
| p (psf) | -21.9 | -34.9 | -46.2 | -46.2 | +16.0 | +22.6 |

Table 4.2 Roof Deck C&C External Pressures

4.1.3 Load Combinations for Steel Roof Deck Uniform Loads

Allowable stress design will be used in this example for the vertical out-of-plane design of the steel roof deck. In general, for out-of-plane design of steel deck, both ASD and LRFD methods provide efficient design. The allowable stress design governing load combinations are determined in accordance with ASCE 7 Section 2.4 for inward (+) and outward uplift (-) wind loads.

For the inward wind vertical load check, ASD load combinations are used considering the full weight of the roof with re-roof for inward wind in combination with gravity, and the minimum roof weight in combination with uplift. For the uplift deflection check the additional wind reduction of 0.42 is applied to the components and cladding wind pressure following Note f of IBC Table 1604.3. The weight of the steel roof deck is taken as zero in accordance with Note g of Table 1604.3

The minimum roof structure weight should be considered in combination with the wind uplift. The original design assumptions presented in Section 2 included an allowance for re-roofing. This added re-roofing weight will not be present when the building is first occupied therefore should be excluded from the net uplift consideration.

Load Combinations for Strength:

3. $D + (L_r \text{ or } S \text{ or } R)$

$$8 + 20 = 28.0 \text{ psf}$$

4. $D + 0.6W$

$$8 + 0.6(16) = 17.6 \text{ psf (wind Zones 1' and 1)}$$

$$8 + 0.6(22.6) = 21.6 \text{ psf (wind Zones 2 and 3)}$$

6. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$

$$8 + 0.75(0) + 0.75((0.6(16.0)) + 0.75(20) = 30.2 \text{ psf (wind Zones 1' and 1)}$$

$$8 + 0.75(0) + 0.75((0.6(22.6)) + 0.75(20) = 33.2 \text{ psf (wind Zones 2 and 3) - controls inward}$$

7. $0.6D + 0.6W$

$$0.6(5) + 0.6(-21.9) = -10.1 \text{ psf (wind Zone 1')}$$

$$0.6(5) + 0.6(-34.9) = -18.0 \text{ psf (wind Zone 1)}$$

$$0.6(5) + 0.6(-46.2) = -24.7 \text{ psf (wind Zones 2 and 3) - controls outward}$$

Load Combinations for Deflection:

$$L_r = 20 \text{ psf}$$

$$W = 0.42(16.0) = 6.4 \text{ psf (inward, Zones 1' and 1)}$$

$$W = 0.42(22.6) = 9.5 \text{ psf (inward, Zones 2 and 3)}$$

$$W = 0.42(-21.9) = -9.2 \text{ psf (outward, Zones 1')}$$

$$W = 0.42(-34.9) = -14.7 \text{ psf (outward, Zones 1)}$$

$$W = 0.42(-46.2) = -19.4 \text{ psf (outward, Zone 2 and 3) - controls outward}$$

$$D + L_r = 0 + 20 = 20 \text{ psf}$$

4.1.4 Steel Roof Deck Inward Uniform Load Design

Verco simplifies the design of steel roof deck to resist uniform vertical loads by providing a maximum allowable uniform load web-based design tool that generates uniform load tables and supporting detailed calculations. The web-based design tools are available on the Verco website.

<https://vercodeck.com/design-tools/>

To begin the design process, an initial deck type, gage and attachment pattern needs to be selected. Designing the deck for uniform loads is the first in a series of steps that will continue through, concentrated load design, diaphragm shear design and designing wall bracing anchorage. To design for uniform loads, the following inputs are selected in the web-based design tool for this initial design step.

Design Tool Inputs:

| | |
|-----------------------------------------------------|----------------------------------------------------------------------|
| Design Method: | ASD |
| Deck Type: | PLB-36 |
| Deck Gage: | 22 (minimum thickness) |
| Deck Grade: | Grade 50 (ASTM A653 or A1008) |
| Uniform Load Deflection Limit: | L/180 |
| Support Member Grade: | A572 GR50 (Note: The angles used in OWSJ construction are Grade 50.) |
| Minimum End Bearing of Steel Deck on Supports: | 2 in |
| Minimum Interior Bearing of Steel Deck on Supports: | 4 in |

The minimum attachment pattern for PLB-36 steel deck to supports of 36/4 is chosen to minimize the number of fasteners to supports. For wind uplift loading, the open web steel joist top chord thickness impacts the pull-out strength of the Hilti fasteners. Hilti X-HSN 24 powder actuated fasteners (PAF) require a minimum 1/8 in support member. Therefore, a minimum 1/8 in support steel thickness is chosen that will lead to the least pull-out strength. The angles Vulcraft uses for the top chord of a 50 ft joist would be 1/8 in or thicker. For very short joist, the designer should note on the plans that the joist top chord angles are to be 1/8 in minimum. End lapped steel deck provides the most economical attachment by eliminating double rows of fasteners at butted sheet ends compared to a single row of fasteners through the end lapped deck.

PLB-36 Deck Attachment:

| | |
|---------------------------------------------------|--------------------|
| Deck Sheet End Condition: | End Lapped Deck |
| Fastener Type: | Hilti X-HSN 24 PAF |
| Minimum open web steel joist top chord thickness: | 1/8 in |

A minimum attachment pattern will be selected for this portion of the design. Assuming this minimum pattern is acceptable for wind uplift, then any heavier attachment pattern will be adequate for wind uplift.

| | |
|-----------------------------------|------|
| Deck End Connection Pattern: | 36/4 |
| Deck Interior Connection Pattern: | 36/4 |

To generate the table shown in Figure 4.6, the following table parameters are entered:

| | |
|----------------------|--------|
| Start Table at Span: | 5.5 ft |
| Spans Increment: | 0.5 ft |

The span increment input is not critical for this project, since the steel joists are all at 8 ft on center. For projects where the spacing varies, this input could be used to create a single table that would cover all the required conditions.

Based on these inputs, the Verco Steel Deck Uniform Load tool can be used to generate a custom allowable uniform load table for this project. See Figure 4.6 for summary output based on the inputs for this project. Complete detailed calculations supporting the summary in Figure 4.6 may be generated using the web-based Steel Deck Uniform Load design tool. The loads shown in the table are the maximum uniform load the steel roof deck can support for the given spacing of the support members and how many spans a given sheet of deck spans. These values are based on bending, web shear, web crippling, fastener capacity for strength, and deflection.

22 Gage PLB™-36 Grade 50

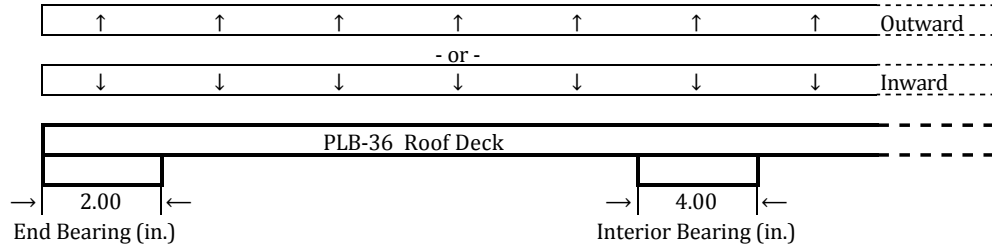
Uniform Allowable Load Table, ASD (psf)

For End Lapped Deck



36/4 Connection Pattern to Supports with
Hilti X-HSN 24 PAF

Support Member A572 GR50
 $0.13 \leq t_2 \text{ (in.)} \leq 0.375$



Inward Uniform Allowable Load Table, ASD (psf)

| Span | Span | 5'-0" | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" |
|------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 1 | Wn/Ω | 140 | 116 | 98 | 83 | 72 | 62 | 55 | 49 |
| | L/180 | 124 | 94 | 72 | 57 | 45 | 37 | 30 | 25 |
| 2 | Wn/Ω | 148 | 122 | 103 | 88 | 76 | 66 | 58 | 52 |
| | L/180 | - | - | - | - | - | - | - | - |
| 3 | Wn/Ω | 184 | 152 | 128 | 110 | 95 | 83 | 73 | 64 |
| | L/180 | - | - | - | - | 92 | 75 | 62 | 52 |

Outward (Uplift) Uniform Allowable Load Table, ASD (psf)

| Span | Span | 5'-0" | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" |
|------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 1 | Wn/Ω | 150 | 124 | 104 | 89 | 77 | 67 | 59 | 52 |
| | Rn/Ω | 120 | 109 | 100 | 93 | 86 | 80 | 75 | 71 |
| | L/180 | - | 101 | 78 | 61 | 49 | 40 | 33 | 27 |
| 2 | Wn/Ω | 139 | 115 | 97 | 82 | 71 | 62 | 55 | 48 |
| | Rn/Ω | 96 | 88 | 80 | 74 | 69 | 64 | 60 | 57 |
| | L/180 | - | - | - | - | - | - | - | - |
| 3 | Wn/Ω | 172 | 143 | 120 | 103 | 89 | 77 | 68 | 60 |
| | Rn/Ω | 109 | 99 | 91 | 84 | 78 | 73 | 68 | 64 |
| | L/180 | - | - | - | - | - | 70 | 57 | 48 |

Steel Deck Properties

| t | Fy | wdd | Id+ | Id- | Se+ | Se- | Mn+/Ω | Mn-/Ω | Vn/Ω |
|--------|-----|------|----------------------|----------------------|----------------------|----------------------|-----------|-----------|--------|
| in | ksi | psf | in. ⁴ /ft | in. ⁴ /ft | in. ³ /ft | in. ³ /ft | lbs-ft/ft | lbs-ft/ft | lbs/ft |
| 0.0299 | 50 | 1.90 | 0.178 | 0.192 | 0.176 | 0.188 | 439 | 469 | 2688 |

Where: $W \leq Wn/\Omega$

- W = Required strength of the governing ASD load combination
- Wn/Ω = Allowable strength governed by the steel deck
- Rn/Ω = Allowable strength governed by connection tension

Steel Deck Uniform V1.0.4 in accordance with AISI S100-16 and AISI S310-16.

Figure 4.6 Verco Uniform Load Web-Tool Summary Page



The maximum uniform load generated by the design tool which the steel roof deck can support based on bending, web shear, and web crippling is compared to the governing inward load combination 6 for the area tributary to the sheet of the deck. The next step is to compare the governing results from the load combinations in Section 4.1.3 above to the steel deck capacities in the design tool summary table to confirm the deck selected is able to support the required loads. The inward loads, outward loads, and loads for deflection will need to be checked. The capacities for inward and outward loads may not be the same, so they should be checked separately.

The steel roof deck span for inward load is the clear span between edges of supporting members. The open web steel joists for a warehouse typically have a 5 in or wider top chord therefore a 7'-7" span would be appropriate for design. It is conservative to use the center to center spacing of the roof framing for the inward vertical load design. For simplicity this conservative center to center span will be used in this example.

Inward Loading Strength Check:

Load Combination 6 for Zones 2 and 3 controlled inward loading.

Max. Uniform Load > Required Uniform Load

Single Span: 60 psf > 33.2 psf, therefore acceptable at an 8'-0" span

Double Span: 63 psf > 33.2 psf, therefore acceptable at an 8'-0" span

Triple Span: 79 psf > 33.2 psf, therefore acceptable at an 8'-0" span

This project is a warehouse where deck does not support a ceiling below the roof structure. The deflection will be limited to L/180 for roof live load or wind load following the requirements of IBC Table 1604.3 for construction "not supporting a ceiling". For steel members the dead load is permitted to be taken as zero in the dead plus live load combination in accordance with Note g of IBC Table 1604.3, therefore the L/120 deflection for dead plus live will not govern.

Inward Loading Deflection Check:

L_r controls inward loading.

Max. Uniform Load > Required Uniform Load

Single Span: 35 psf > 20 psf, therefore acceptable

Double Span: 63 psf > 20 psf, therefore acceptable

Note: Deflection does not control with Double Span condition, so strength load used.

Triple Span: 65 psf > 20 psf, therefore acceptable

4.1.5 Steel Roof Deck Inward Concentrated Roof Live Load Design

Steel roof deck is subject to a concentrated roof live load for roof maintenance workers of 300 lb distributed over a 2½ ft x 2½ ft area in addition to the uniform dead load, following the provisions of IBC Section 1607.4. This concentrated load becomes a 48 psf live load over the specified area. This translates into a 48 plf load applied over a 2½ ft length for steel roof decks that are designed on a per foot width basis, as shown in Figure 4.7a. The design of the steel roof deck will be conservative if the 300 lb is converted to a line load perpendicular to the deck span of 300 lb / 2½ ft = 120 plf as shown in Figure 4.7b which will result in a higher bending moment than the 48 plf distributed load. This line load will be used to check the steel roof deck for strength.

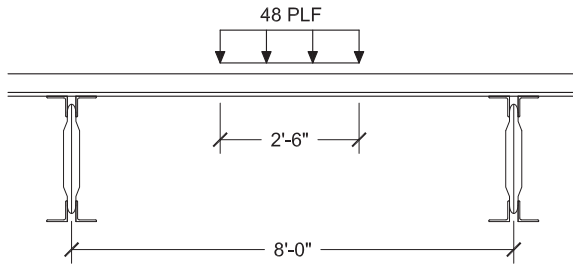


Figure 4.7a Concentrated Roof Load for Maintenance Workers

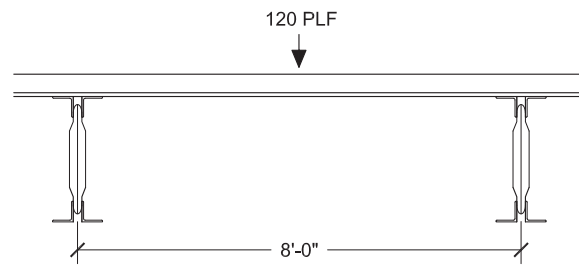


Figure 4.7b Equivalent Line Load for Maintenance Workers

Roof Deck Loads:

| | |
|-------------------------|------------------------------------|
| Roof + Re-Roof: | 8.0 psf |
| Concentrated Live Load: | 48 psf over a 2½ ft length |
| | or |
| | 120 plf across width of steel deck |

Load combination 3 will be used for the concentrated load. The load combinations including wind will not be checked based on the assumption that for safety the maintenance workers would not be present on the roof during high wind events.

3. $D + (L_r \text{ or } S \text{ or } R)$

4.1.5.1 Steel Roof Deck Concentrated Load Design for Strength

Available Strength from IAPMO ER-2018 for 22 gage PLB-36 steel deck.

Bending:

$$M_{a+} = M_{n+} / 1.67 = 0.733 \text{ kip-ft/ft} / 1.67 = 0.439 \text{ kip-ft/ft}$$

$$M_{a-} = M_{n-} / 1.67 = 0.783 \text{ kip-ft/ft} / 1.67 = 0.442 \text{ kip-ft/ft}$$

Vertical component of web shear:

$$V_a = V_n / 1.60 = 4.3 \text{ kip/ft} / 1.60 = 2.7 \text{ kip/ft}$$

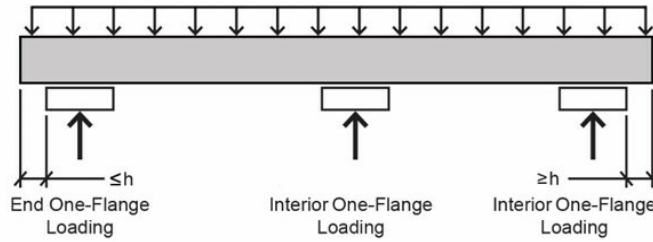
Web Crippling for 2 inch end bearing and 4 inch interior bearing from Verco web-based web crippling tool summary in Figure 4.8.

$$P_{a\text{end}} = 934 \text{ plf}$$

$$P_{a\text{int}} = 1670 \text{ plf}$$

Detailed calculations supporting the web crippling table in Figure 4.8 may be generated using the web-based Web Crippling design tool.

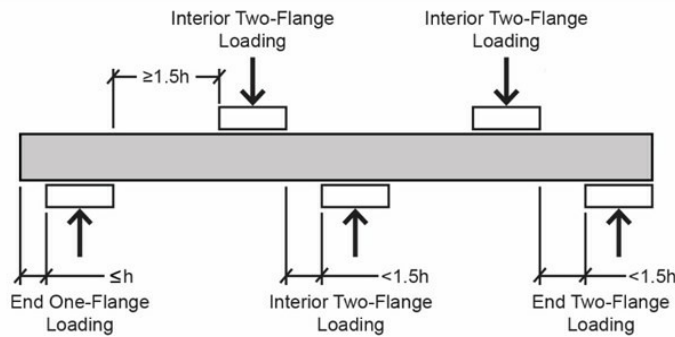
One Flange Loading for Uniform Load



Reaction Allowable Strength at Supports Based on Web Crippling for One Flange Loading, ASD (plf)

| | Bearing | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 3.50 | 4.00 | 4.50 |
|----------|---------|------|------|------|------|------|------|------|------|
| End | Rn/Ω | 751 | 850 | 934 | 1008 | 1075 | 1137 | 1163 | 1163 |
| Interior | Rn/Ω | 1143 | 1271 | 1378 | 1473 | 1558 | 1637 | 1670 | 1670 |

One or Two Flange Loading for Concentrated Loads



Reaction Allowable Strength at Supports Based on Web Crippling for Two Flange Loading, ASD (plf)

| | Bearing | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 3.50 | 4.00 | 4.50 |
|----------|---------|------|------|------|------|------|------|------|------|
| End | Rn/Ω | 811 | 893 | 962 | 1022 | 1077 | 1128 | 1149 | 1149 |
| Interior | Rn/Ω | 1379 | 1549 | 1693 | 1819 | 1933 | 2038 | 2082 | 2082 |

Steel Deck Properties

| | Fy | wdd | Se+ | Se- | Id+ | Id- | Mn+/Ω | Mn-/Ω | Vn/Ω |
|------|-----|------|----------------------|----------------------|----------------------|----------------------|-----------|-----------|--------|
| Gage | ksi | psf | in. ³ /ft | in. ³ /ft | in. ⁴ /ft | in. ⁴ /ft | lbs-ft/ft | lbs-ft/ft | lbs/ft |
| 22 | 50 | 1.90 | 0.176 | 0.188 | 0.178 | 0.192 | 439 | 469 | 2688 |

Web Crippling V1.0 in Accordance with AISI S100-16 and IAPMO ER-2018

Figure 4.8 22 gage PLB-36 Web Crippling

Design of steel roof deck for single, double, and triple span conditions for a concentrated roof live load is shown in Figures 4.9, 4.10, and 4.11 respectively. For this example, the location of the concentrated roof live load is placed at the mid-span of a single span or a mid-span of the first span for multi-span conditions. This is not the location of the maximum moment. The maximum moment, except for the single span condition, will occur somewhere between 0.4l and 0.5l from the end for double and triple span conditions. This small error is unconservative by about 3% which is insignificant and offsets the slightly conservative line load compared to the concentrated distributed load over 2½ ft length. For critical situations a more exact analysis may be performed.

Single Span:

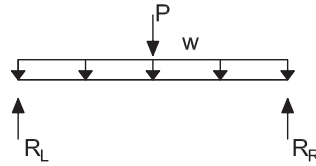


Figure 4.9 Concentrated Roof Load on Single Span

$$+M = Pl/4 + wl^2/8 = 120 \text{ plf} (8 \text{ ft})/4 + 8 \text{ psf} (8 \text{ ft})^2/8 = 304 \text{ lb-ft} < 439 \text{ lb-ft}$$

$$R_L = R_R = V_L = V_R = P/2 + wl/2 = 120 \text{ plf} / 2 + 8 \text{ psf} (8 \text{ ft}) / 2 = 92 \text{ plf} < 934 \text{ plf}$$

Double Span:

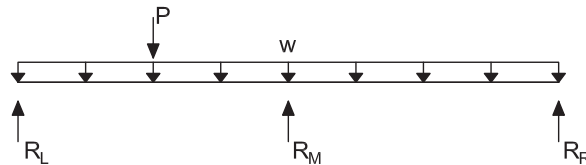


Figure 4.10 Concentrated Roof Load on Double Span

$$+M = 0.203Pl + wl^2/16 = 0.230 (120 \text{ plf})(8 \text{ ft}) + 8 \text{ psf} (8 \text{ ft})^2/16 = 252 \text{ lb-ft} < 439 \text{ lb-ft}$$

$$-M = 0.09375Pl + wl^2/8 = 0.09375 (120 \text{ plf}) (8 \text{ ft}) + 8 \text{ psf} (8 \text{ ft})^2/8 = 154 \text{ lb-ft} < 442 \text{ lb-ft}$$

$$R_L = V_L = 0.406P + 0.375wl = 0.406 (120 \text{ plf}) + 0.375 (8 \text{ psf})(8 \text{ ft}) = 73 \text{ plf} < 934 \text{ plf}$$

$$R_M = V_{RM} + V_{RL} = (0.594P + 0.625wl) + (0.094P + 0.625wl)$$

$$= (0.594(120 \text{ plf}) + 0.625(8 \text{ psf})(8 \text{ ft})) + (0.094(120 \text{ plf}) + 0.625(8 \text{ psf})(8 \text{ ft}))$$

$$= 111 \text{ plf} + 51 \text{ plf} = 162 \text{ plf} < 1670 \text{ plf}$$

Bending and Shear Interaction is checked in accordance with AISI S100 Section H2. Combined bending and web crippling is not required to be checked in accordance with the exception in AISI S100 Section H3.

$$\sqrt{\left(\frac{M}{M_a}\right)^2 + \left(\frac{V}{V_a}\right)^2} \leq 1.0$$

AISI eq. H2-1

Positive Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.252 \text{ k-ft/ft}}{0.439 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.12 \text{ kip/ft}}{2.7 \text{ kip/ft}}\right)^2} = 0.58 \leq 1.0$$

Negative Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.154 \text{ k-ft/ft}}{0.442 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.162 \text{ kip/ft}}{2.7 \text{ kip/ft}}\right)^2} = 0.64 \leq 1.0$$

Triple Span:

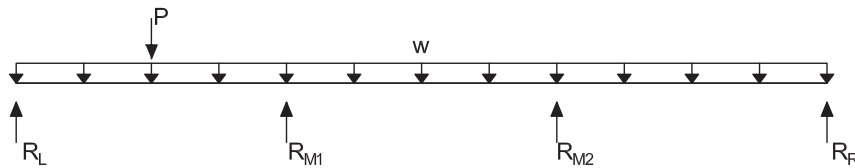


Figure 4.11: Concentrated Roof Load on Triple Span

$$+M = 0.20PI + 0.075wl^2 = 0.2(120 \text{ plf})(8 \text{ ft}) + 0.075(8 \text{ psf})(8 \text{ ft})^2 = 230 \text{ lb-ft} < 439 \text{ lb-ft}$$

$$-M = 0.10PI + 0.10wl^2 = 0.1(120 \text{ plf})(8 \text{ ft}) + 0.1(8 \text{ psf})(8 \text{ ft})^2 = 301 \text{ lb-ft} < 442 \text{ lb-ft}$$

$$R_L = V_L = 0.4P + 0.4wl = 0.4(120 \text{ plf}) + 0.4(8 \text{ psf})(8 \text{ ft}) = 74 \text{ plf} < 934 \text{ plf}$$

$$\begin{aligned} R_{M1} &= V_{RM1L} + V_{RML2} = (0.6P + 0.6wl) + (0.125P + 0.5wl) \\ &= (0.6(120\text{plf}) + 0.6(8 \text{ psf})(8 \text{ ft})) + (0.125(120 \text{ plf}) + 0.5(8 \text{ psf})(8 \text{ ft})) \\ &= 111 \text{ plf} + 47 \text{ plf} = 157 \text{ plf} < 1670 \text{ plf} \end{aligned}$$

Bending and Shear Interaction:

Positive Bending and Shear:

$$\sqrt{\left(\frac{0.230 \text{ k-ft/ft}}{0.439 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.12 \text{ kip/ft}}{2.7 \text{ kip/ft}}\right)^2} = 0.53 \leq 1.0$$

Negative Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.301 \text{ k-ft/ft}}{0.442 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.157 \text{ kip/ft}}{2.7 \text{ kip/ft}}\right)^2} = 0.68 \leq 1.0$$

Based on both web-crippling strength at supports and combined bending and web shear, the 22 ga PLB-36 will support the concentrated roof live load for maintenance workers.

4.1.5.2 Steel Roof Deck Concentrated Load Design for Deflection

The deflection of the steel deck due to maintenance workers is more complex than simply checking the mid-span deflection due to the equivalent line load as was done for strength. Light gauge steel roof deck feels flexible at relatively long spans prior to the roof system being attached. Although the roof system is not a structural element of the building, the insulation boards act to redistribute the concentrated load over a wider area of steel deck than the footprint of the load, as shown in Figures 4.12a and 4.12b. This significantly reduces the deflection of the steel deck by redistributing the concentrated load over a larger area of steel roof deck. This is a difficult analytical model to solve, however load-deflection testing provides an easy alternative to this complex calculation.

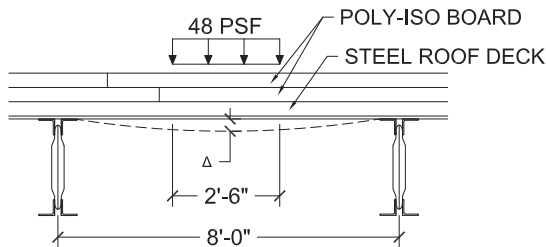


Figure 4.12a Concentrated Roof Load Across Steel Roof Deck Span

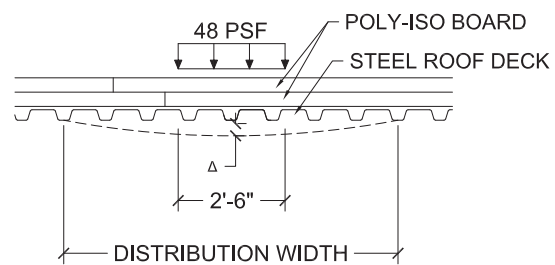


Figure 4.12b Concentrated Roof Load Redistribution of Poly-iso Board

To assist the designer, IAPMO ER-2018 includes a table, PLB-36 Roof Deck Spans for Concentrated Loads, with the maximum IBC concentrated roof load spans for PLB-36 roof deck. A partial extract of the table is shown in Figure 4.13.

| PLB-36 AND HSB-36 ROOF DECK SPANS FOR CONCENTRATED LOADS ¹⁻⁴ | | | | |
|-------------------------------------------------------------------------|-----------------|--------------------------------------------|---------|---------|
| Deck Gage | Number of Spans | Maximum Span based on Live Load Deflection | | |
| | | L / 360 | L / 240 | L / 180 |
| 22 | 1 | 7'-5" | 9'-5" | 11'-5" |
| | 2 | 7'-10" | 10'-11" | 13'-0" |
| | 3 | 7'-10" | 10'-11" | 12'-11" |

Figure 4.13 Roof Deck Span for Roof Concentrated Load

The maximum permissible clear span of the steel deck based on strength and L/180 deflection may be taken directly from the table for 22 gage PLB-36 steel deck with a minimum of 2 layers 1½" poly-iso board insulation as part of the roof assembly as follows:

Single Span: 11'-5" maximum span > 8'-0" span

Double Span: 13'-0" maximum span > 8'-0" span

Triple Span: 12'-11" maximum span > 8'-0" span

The vertical load analysis including dead load, live load and wind load demonstrates that 22 gage PLB-36 is acceptable for open web steel joist spacing at 8 ft on center.

4.1.6 Roof Membrane Wind Uplift Considerations for Steel Deck Design

The roof system has a significant impact on the wind uplift load path to the steel roof deck. Most flat roof warehouse type structures have a membrane roof system that is either fully adhered or mechanically attached. Fully adhered roofs transfer a uniform uplift load to the steel roof deck. Mechanically attached roof membranes apply a concentrated line load to the roof deck at each seam in the roof membrane.

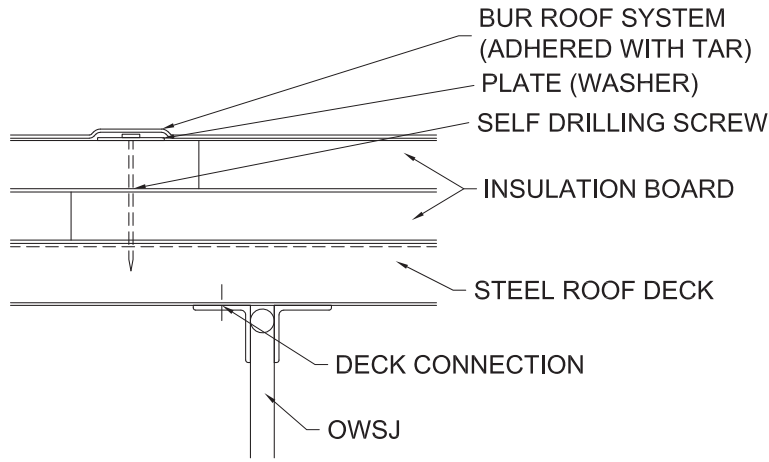


Figure 4.14a Fully Adhered Roof Assembly

Fully adhered roof systems are comprised of a roof membrane that is bonded to the top insulation board or cover board, typically with hot tar or an adhesive. The insulation board, and coverboard when used, are mechanically attached to the steel deck with a uniform series of self-drilling screws and insulation plates as shown in Figure 4.14a. The wind uplift pressure applied to the roof membrane is transmitted to the top insulation board or cover board that is retained by the plates and self-drilling screws to the steel roof deck. The screws fall in a regular array, with a wider spaced pattern around 2 ft apart for low wind uplift force, to a heavy pattern as close as 6 inches o.c. for high wind uplift forces as shown in Figures 4.14b and 4.14c. This array of self-drilling screws applies an overall uniform load to the steel roof deck as shown in Figure 4.15. Designing the steel deck to resist the uniform components and cladding uplift load for a fully adhered roof system is easy because it creates a uniform uplift load on a slender beam that is easy to analyze.

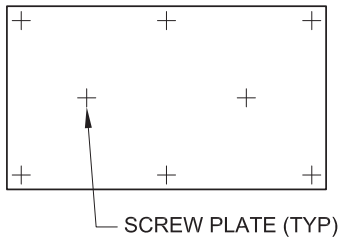


Figure 4.14b Light Attachment Pattern

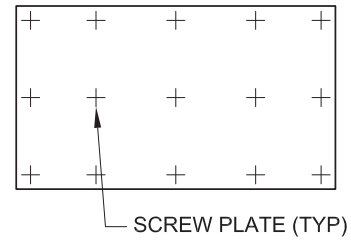


Figure 4.14c Heavy Attachment Pattern

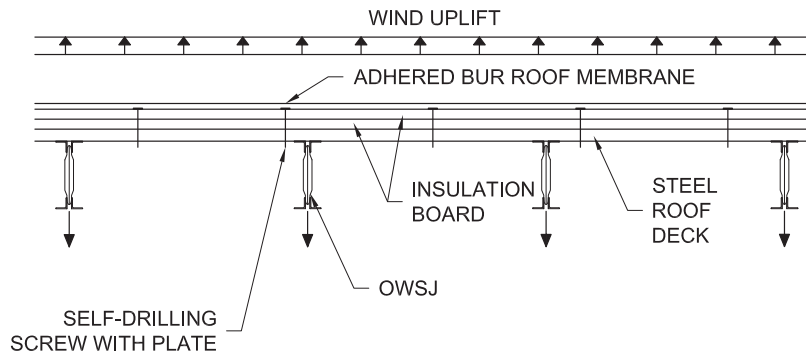


Figure 4.15 Fully Adhered Wind Uplift

Mechanically attached membrane roofs are a far more complicated system to design. At first glance the assembly does not look significantly different than a fully adhered roof. The roof membrane is supported by insulation, and cover board when used, that is attached to the steel roof deck with a series of screws to hold the boards in place as shown in Figure 4.16. The critical difference is that the roof membrane is directly attached to the steel roof deck only at the seams of the single ply membrane. Under wind uplift loads the single ply roof is only restrained by the plates and screws to the steel deck at the seams of the membrane. This creates a series of relatively high concentrated line loads on the steel deck rather than a modest uniform uplift load. Figure 4.17 depicts the wind uplift pulling the single ply away from the surface of the roof boards as a catenary with the plates and screws then putting high line loads on the steel roof deck.

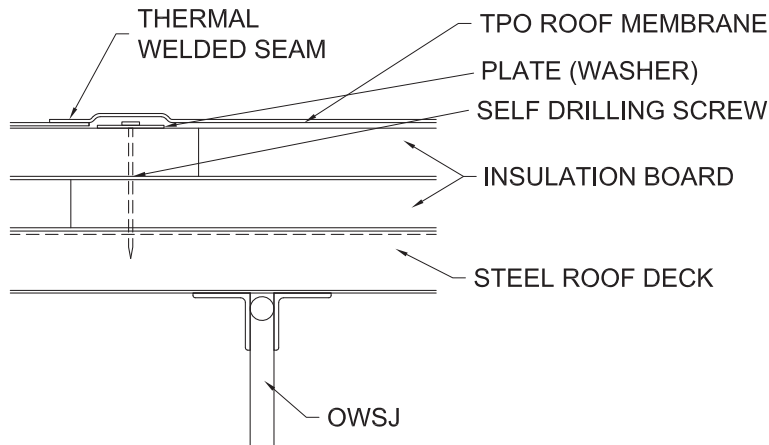


Figure 4.16 Mechanically Attached Roof System

Single ply roof membrane come in widths from under 4 ft to as wide as 12 ft. A 12 ft roof membrane would have an 11.5 ft tributary width considering a 6 in lap. If the steel deck was designed with an 8 ft span and the row of attachment happened at mid-span of a single span sheet, the concentrated load due to 11.5 ft of tributary width would generate a moment 2.9 times greater than a fully adhered roof. A steel roof deck that is designed efficiently for the uniform uplift load of a fully adhered roof may fall well short for the same load applied to a mechanically attached roof membrane.

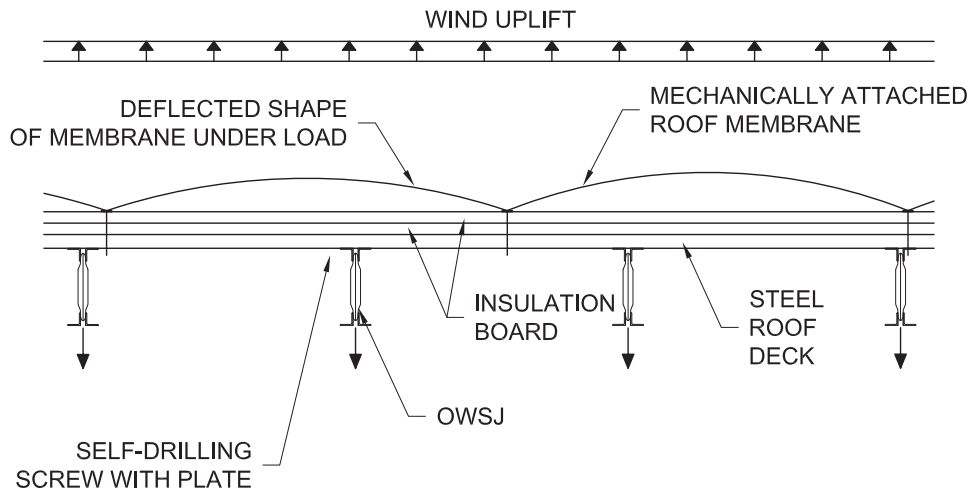


Figure 4.17 Uplift on Mechanically Attached Roof Membrane

Mechanically attached roof membranes create a much more complicated design process for the engineer designing the steel roof deck. There are essentially two options. Work with the owner and architect to specify a fully adhered roof, or work with the architect to determine the maximum acceptable width of mechanically attached roof membrane and then design the deck for the required line loads. One complication is that the location of the line loads will likely not be known before the roof system is installed.

This problem extends to the open web steel joists (OWSJ). If the design is for 6 ft OWSJ spacing and 12 ft wide mechanically attached roofing, then it is likely that some part of the structure will end up with loading as shown in Figure 4.18. This loading essentially skips every other OWSJ, applying double the uplift to every other OWSJ.

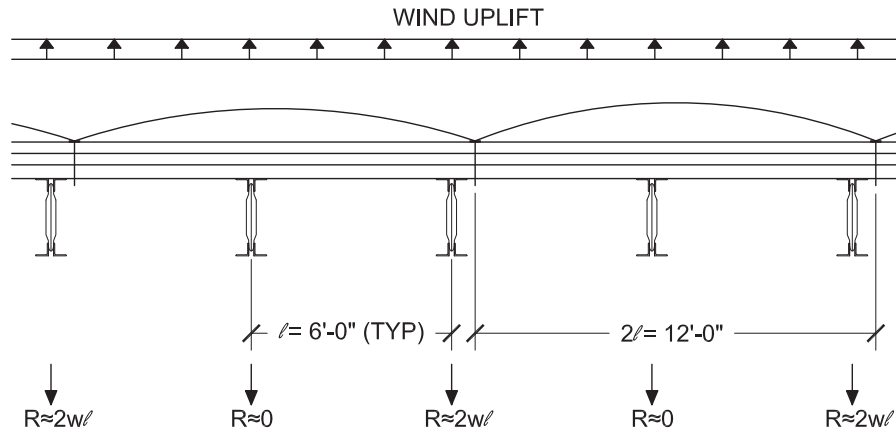


Figure 4.18 Mechanically Attached Roof Skip Loading on OWSJ

Based on these considerations, the simple recommendation is to work with the architect and owner to specify a fully adhered roof system in lieu of a mechanically attached roof system. This avoids the complexities of working through the concentrated loads layout of the roof membrane compared to the spacing of the OWSJ to determine the appropriate steel roof deck gage, connection pattern and the appropriate worst-case uplift for the OWSJ specification.

4.1.7 Steel Roof Deck Outward Wind Uplift Uniform Load Design

The design of the steel deck for outward wind uplift loading will be based on a fully adhered roof system for this example. The strength of the steel roof deck to resist the net uplift loads from the web-based uniform load design tool shown in Figure 4.6 is compared to the net wind uplift developed in Section 4.1.3 as follows.

Outward Loading Strength Check:

Load Combination 7 for Zones 2 and 3 controls outward loading.

Max. Uniform Load > Required Uniform Load
Single Span:-64 psf > -24.7 psf, therefore acceptable
Double Span:-59 psf > -24.7 psf, therefore acceptable
Triple Span: -74 psf > -24.7 psf, therefore acceptable

Outward Loading Deflection Check:

Wind load for Zones 2 and 3 controls outward loading.

Max. Uniform Load > Required Uniform Load
Single Span:-37 psf > -19.4 psf, therefore acceptable
Double Span:-59 psf > -19.4 psf, therefore acceptable
Triple Span: -74 psf > -19.4 psf, therefore acceptable

The minimum 22 gage PLB-36 steel deck is adequate for strength and deflection for both the inward and outward (uplift) vertical loads. For projects in higher wind areas than this example, it may be necessary to use a different gage deck in Zones 2 and 3 than in Zones 1' and 1, to keep the building design as efficient as possible. The next step is to verify that the connections of the steel deck to supports are adequate to resist the net wind uplift load.

4.1.8 Steel Roof Deck Outward Wind Uplift Fastening to Supports Design

The attachment of the steel deck to the open web steel joists must be checked in addition to checking wind uplift strength of the steel roof deck. This needs to be checked separately because the effective wind area for the fasteners is less than that for the steel roof deck resulting in higher wind uplift pressures. This check only addresses tension on the connections to establish the minimum required attachment pattern for wind uplift. Section 5 will address the effects of combined wind uplift and diaphragm shear stresses on the fasteners. As noted in Section 4.1.4, Hilti X-HSN 24 power actuated fasteners (PAFs), commonly referred to as Hilti pins or high shear nails will be used. Arc spot welds, screws, and other manufacturers of PAFs are also suitable for steel roof deck attachment.

The deck fastener's effective wind area is smaller than the effective wind area for the steel roof deck sheet leading to higher external pressure coefficients, GC_p . This is because the width used for the effective wind area for a fastener is the fastener spacing. For most common deck spans, with any deck attachment pattern less than 12 in o.c., the area will be less than 10 sf, leading to the highest-pressure coefficients and corresponding wind uplift pressures because the pressure coefficient is capped for effective areas less than 10 sf.

Effective Wind Area of fasteners at 12 in o.c. (36/4 pattern for PLB-36 deck as shown in Figure 4.19).

$$A_e = 1 \text{ ft}(8 \text{ ft}) = 8 \text{ sf}$$

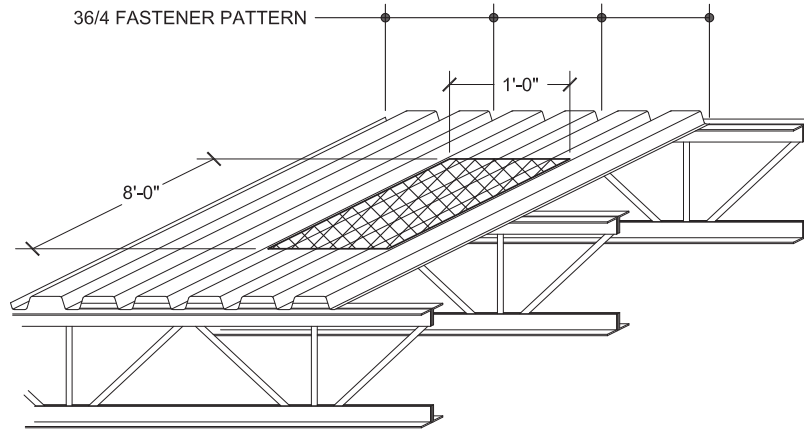


Figure 4.19 Effective Wind Area of Fastener

The fastener outward external pressure coefficients for areas less than 10 sf from ASCE 7 Figure 30.3-2A are summarized in Table 4.3. This summary takes into account the effect of parapets on the corner Zone 3 in accordance with Note 5 in ASCE 7 Figure 30.3-2A.

| Zone | 1' | 1 | 2 | 3 |
|--------|-------|-------|------|------|
| GC_p | -0.90 | -1.70 | -2.3 | -2.3 |

Table 4.3 Roof Deck Fastener C&C External Pressure Coefficients, GC_p

The wind design pressure, p , for the steel deck fasteners based on ASCE 7 Section 30.3.2:

$$p = q_h [(GC_p) - (GC_{pi})] \quad \text{ASCE 7 eq 30.4-1}$$

For Zone 1:

$$-p_1 = 20.3 [(-1.70) - (0.18)] = -38.1 \text{ psf (outward)}$$

Zone 1', 2, and 3 outward wind pressures for the fasteners are determined in a similar manner and are summarized in Table 4.4.

| Zone | 1' | 1 | 2 | 3 |
|-----------|-------|-------|-------|-------|
| p (psf) | -21.9 | -38.1 | -50.3 | -50.3 |

Table 4.4 Roof Deck Fastener C&C External Pressures

Net wind uplift load on fasteners based on governing ASD load combination.

$$7. \quad 0.6D + 0.6W =$$

$$0.6(5) + 0.6(-21.9) = -10.1 \text{ psf (for wind Zone 1')}$$

$$0.6(5) + 0.6(-38.1) = -19.9 \text{ psf (for wind Zone 1)}$$

$$0.6(5) + 0.6(-50.3) = -27.2 \text{ psf (for wind Zones 2 and 3)}$$

$$0.6(5) + 0.6(-68.5) = -38.1 \text{ psf (for wind Zone 3)}$$

The maximum uniform load the Hilti X-HSN 24 fasteners can resist in tension is compared to the governing load above. The capacity of the Hilti X-HSN 24 fasteners (R_n/Ω) is given in the Verco design tool summary output, Figure 4.6.

Fastener Tension Check:

Single Span: -73 psf > -38.1 psf, therefore acceptable

Double Span: -58 psf > -38.1 psf, therefore acceptable

Triple Span: -66 psf > -38.1 psf, therefore acceptable

The minimum 22 gage PLB-36 steel deck with Hilti X-HSN 24 PAF with 36/4 pattern is adequate for the vertical loading requirements of this project.

4.2 Open Web Steel Joist Vertical Load Design

The next roof members that need to be designed are the open web steel joists (OWSJ). The OWSJ provide vertical support for the loads applied by the steel roof deck. In addition, the OWSJ support the dead loads from miscellaneous mechanical, electrical, and the fire sprinkler system including both branch and main lines.

The OWSJ being used on this example building are Steel Joist Institute (SJI) style trusses. The OWSJ are designed in accordance with SJI-100 Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders referenced in Section 2207 of the IBC. This code section also delineates the responsibilities of the Engineer of Record (EOR) and Vulcraft for the OWSJ design. The EOR provides the design criteria including the loading and deflection limits. Vulcraft designs the physical truss including the angle sizes, welds, and seats to meet the design criteria specified by the EOR.

4.2.1 Open Web Steel Joist Dead Loads:

Following the same convention as the steel roof deck, the inward dead load will be different than the dead load used in combination with outward wind uplift outward loading. The following are the applicable dead loads summarized for the OWSJ as developed in Section 2.

| Dead Load | Inward Loading | Uplift Loading |
|----------------------------------------|-----------------|-----------------|
| Roof System | 3.0 psf | 3.0 psf |
| Re-Roof | 2.0 psf | 0 psf |
| Steel Deck | 3.0 psf | 2.0 psf |
| Automatic Sprinklers | 2.0 psf | 2.0 psf |
| Miscellaneous | 1.5 psf | 1.5 psf |
| Open Web Steel Joists | 2.0 psf | 2.0 psf |
| Total for Open Web Steel Joists | 13.5 psf | 10.5 psf |

Table 4.5 OWSJ Dead Loads

The uniform dead load allowance for the automatic sprinklers is generally adequate to cover the branch lines and small sprinkler mains up to 3 in diameter. Larger sprinkler mains, 4 in diameter and larger, generally need to be addressed as a concentrated load or as an Add-Load discussed in Section 4.2.7.

4.2.2 Open Web Steel Joist Live Loads:

The live load for the OWSJ is based on the tributary area and applicable live load reduction. This reduction is based on the OWSJ tributary area which is the product of the effective width of one half the deck span on either side of the joist and the span length of the joist as shown in Figure 4.20.

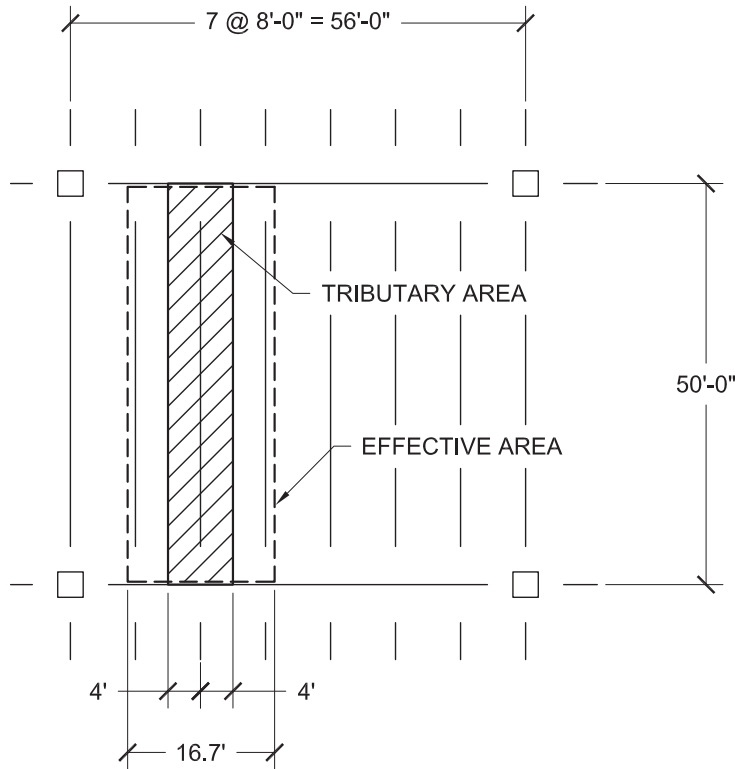


Figure 4.20 OWSJ Tributary Area

$$L_j = 50 \text{ ft OWSJ Length}$$

$$w = (8 \text{ ft}/2 + 8 \text{ ft}/2) = 8 \text{ ft tributary width}$$

$$A_t = L_j w = 50 \text{ ft}(8 \text{ ft}) = 400 \text{ sf}$$

IBC Section 1607.13.2.1

IBC Section 1607.13.2.1 allows the roof live load for ordinary roofs to be reduced for areas greater than 200 sf but place a lower limit of not less than 12 psf.

OWSJ Reduced Roof Live Load:

$$L_r = L_o R_1 R_2$$

IBC eq 16-26

$$\text{Where } 12 \text{ psf} \leq L_r \leq 20 \text{ psf}$$

$$R_1 = 1 \text{ for } A_t \leq 200 \text{ sf}$$

IBC eq 16-27

$$R_1 = 1.2 - 0.001 A_t \text{ for } 200 \text{ sf} < A_t < 600 \text{ sf}$$

IBC eq 16-28

$$R_1 = 0.6 \text{ for } A_t \geq 600 \text{ sf}$$

IBC eq 16-29

Therefore:

$$R_1 = 1.2 - 0.001(400 \text{ sf}) = 0.8$$

IBC eq 16-28

$$R_2 = 1 \text{ for roof slopes } \leq 4:12$$

IBC eq 16-30

$$L_r = 20 \text{ psf}(0.8)(1.0) = 16 \text{ psf for the OWSJ}$$

IBC eq 16-26

$$L_r = 16 \text{ psf} > 12 \text{ psf minimum, therefore 16 psf controls.}$$

4.2.3 Open Web Steel Joists C&C Wind Pressure

The development of the wind design pressure for the OWSJ is the same as the steel roof deck for the first 5 Steps of the design process. The OWSJ wind design is picked up at Step 6 determining the effective wind area.

Step 6 (continued): The effective wind area of the OWSJ needs to be calculated to determine the external pressure coefficients, GC_p , for the different zones of the roof.

As the tributary area of a member gets larger it is important to check to determine if C&C wind loads will be used or if it is permitted to use the lower MWFRS wind loads. ASCE 7 Section 30.2.3 permits a member to be designed for the lower MWFRS loads if the tributary area is greater than 700 sf. To make this determination it is critical that the tributary area is used, not the effective area to determine the wind pressure coefficients. The tributary area of a member and the effective wind area are not necessarily the same. The tributary area is span length multiplied by half the spacing of members on each side, where the effective wind area defined in ASCE 7 Section 26.2 is the span length multiplied by an effective width that need not be less than one-third the span length. For the OWSJ the tributary area and effective wind area are determined as follows.

Tributary Area:

$$a_t = L (s/2 + s/2) = L (s) = 50 \text{ ft}(8 \text{ ft}) = 400 \text{ sf}$$

IBC Section 1607.13.2.1

Where:

L = span length of member

s = tributary width relative to span

The tributary area is less than 700 sf, therefore the OWSJ will be designed using C&C wind loading.

Effective Wind Area:

ASCE 7 Section 26.2

w = 8 ft width based on joist spacing

w = $L/3 = 50 \text{ ft}/3 = 16.7 \text{ ft}$ based on span length (governs)

$$a_e = L (w) = 50 \text{ ft}(16.7 \text{ ft}) = 835 \text{ sf}$$

Where:

L = span length of member

w = effective width relative to member that need not be less than $L/3$

The external pressure coefficients are determined from ASCE 7 Figure 30.3-2A based on the 835 sf effective wind area. The corner Zone 3 is permitted to be designed using the lower Zone 2 pressure coefficients for structures with parapets 3 ft or greater as developed in Section 4.1.2.3 for the steel roof deck wind loading.

| Zone | 1' | 1 | 2 | 3 | 1' & 1, | 2 & 3 |
|--------|-------|-------|-------|-------|---------|-------|
| GC_p | -0.44 | -1.00 | -1.40 | -1.40 | +0.20 | +0.70 |

Table 4.6 OWSJ C&C External Pressure Coefficients, GC_p

Step 7: The wind design pressure, p , for the OWSJ is then determined based on the velocity pressure, internal pressure coefficients, and external pressure coefficients in accordance with ASCE 7 Section 30.3.2

$$p = q_h [(GC_p) - (GC_{pi})] \quad \text{ASCE 7 eq 30.3-1}$$

For Zone 1:

$$-p_1 = 20.3[(-1.00) - (0.18)] = -23.9 \text{ psf (outward)}$$

$$+p_1 = 20.3[(+0.20) - (-0.18)] = +7.7 \text{ psf (inward)} \leq 16 \text{ psf, therefore 16 psf governs}$$

Joist design pressures for the roof surfaces are determined for all Zones:

| Zone | 1' | 1 | 2 | 3 | 1' & 1 | 2 & 3 |
|-----------|-------|-------|-------|-------|--------|-------|
| p (psf) | -16.0 | -23.9 | -32.0 | -32.0 | +16.0 | +17.8 |

Table 4.7 OWSJ C&C External Pressures

4.2.4 Open Web Steel Joists Load Combinations

The ASD load combinations will be used to determine the applicable loads for the open web steel joists (OWSJ). When using OWSJ, the design professional only needs to specify the loading, freeing them from the analysis of the trusses themselves. Vulcraft then checks all load combinations based on the specified loads.

The OWSJ for this building will be specified using load per foot designations rather than standard series SJI designations. This is easier for the design professional because they can stop once they have determined the load per foot that will be used to design the OWSJ. This eliminates the step of having to look for an SJI standard designation joist in the Vulcraft load tables. This is also the most efficient way to specify OWSJ because the design is based on the exact specified load rather than the next strongest standard series joist that exceeds the required loads.

The basic load per foot designation for an OWSJ includes the total load and the live load applied to the top chord. In addition to the basic load per foot, the outward uplift and inward wind loads need to be specified. Section 4.2.7 will address specifying concentrated loads.

Inward Load Combinations:

Load combination 3 will be used to develop the total gravity load for the open web steel joist specification.

$$3. \quad D + (L_r \text{ or } S \text{ or } R)$$

$$13.5 \text{ psf} + 16 \text{ psf} = 29.5 \text{ psf}$$

Load combination 6a is presented here to demonstrate that this load combination governs for wind design. Vulcraft will back out the dead and live load per foot from the designation and combine with wind to check this load combination. The EOR therefore only needs to consider load combination 3 for the OWSJ specification.

6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$

$13.5 + 0.75(0) + 0.75((0.6(16)) + 0.75(16) = 32.7 \text{ psf, governs (for wind Zones 1', or 1)}$

$13.5 + 0.75(0) + 0.75((0.6(17.8)) + 0.75(16) = 33.5 \text{ psf, governs (for wind Zones 2, or 3)}$

Inward wind loads per Table 4.7 would be called out on the structural plans. See how the wind loads are presented on the sample structural plans in the Appendix. ASD Load Combinations 3 and 6 will be run by Vulcraft when the joist is designed.

Outward Loading (Net wind uplift):

7. $0.6D + 0.6W$

$0.6(10.5) + 0.6(-16.0) = -2.7 \text{ psf (wind Zone 1')}$

$0.6(10.5) + 0.6(-23.9) = -7.5 \text{ psf (wind Zone 1)}$

$0.6(10.5) + 0.6(-32.0) = -12.3 \text{ psf (wind Zones 2 and 3)}$

The deflection check for wind will use the 0.42 load factor for components and cladding wind pressure following Note f of IBC Table 1604.3. The weight of the OWSJ is taken as zero in accordance with Note g of Table 1604.3.

Load Combinations for Deflection:

$W = 0.42W$

$W = 0.42(16.0) = 6.4 \text{ psf (inward, Zones 1' and 1)}$

$W = 0.42(17.8) = 7.5 \text{ psf (inward, Zones 2 and 3)}$

$W = 0.42(-16.0) = -6.4 \text{ psf (outward, Zone 1')}$

$W = 0.42(-23.9) = -10.0 \text{ psf (outward, Zone 1)}$

$W = 0.42(-32.0) = -13.4 \text{ psf (outward, Zones 2 and 3)}$

4.2.5 Open Web Steel Joists Specification

The most economical joists are specified as load per foot. This method allows the designer to specify the exact design loads, allowing the joist manufacturer to optimize the joist to meet those loads. Load per foot joist are specified based on the joist depth, series, total load, live load and net wind uplift load. It is critical that the designer of record clearly state that the specified loads are ASD or LRFD design basis for the OWSJ. Figure 4.21 illustrates the SJI Load/Load designation.

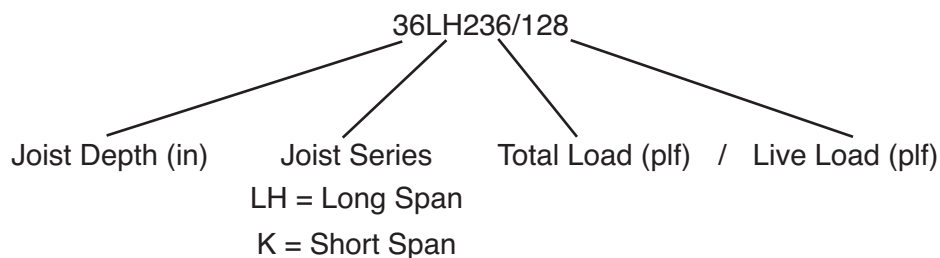


Figure 4.21 SJI Load/Load Designation Format

To specify an OWSJ one of the first considerations is determining the most economical depth. A good “Rule of Thumb” to estimate the most economical depth for roofs with axial loads is as follows:

20 psf reducible roof live load: Joist depth (in) = Joist Span (ft) /2 + 6 to 8 inches

25 psf non-reducible roof snow load: Joist depth (in) = Joist Span (ft) /2 + 8 to 10 inches

This depth may be a bit deeper than those recommended in other sources. The additional depth accounts for the effect of axial load for seismic wall anchorage developed in Section 6.0 of this example.

For this project:

Joist Depth (in)=Joist Span in feet/2 + 6 to 8 inches

=50/2 + 6 = 31 in

=50/2 + 8 = 33 in

The next consideration will be determining the appropriate series of OWSJ to specify. There are three common series of SJI OWSJ; short span, long span and deep long span joists. It is not critical that the designer of record understand all the intricacies of the design of each series. It is important to understand when to specify each joist series. The primary driving feature is the depth range for the OWSJ series as shown in Table 4.8. 1/2

| | Series | Depth | Seat Depth |
|----------------|--------|------------------|------------|
| Short Span | K | 10 to 30 in | 2½ in |
| Long Span | LH | 18 to 48 in | 5 in |
| Deep Long Span | DLH | 48 in and deeper | 5 in |

Table 4.8 OWSJ Series

Based on the economical depth, a 32 in deep LH Series long span OWSJ is selected for this project, falling within the long span depth range of 18 to 48 inches.

With the OWSJ depth and series selected, the load per foot and wind loading is developed. The loads listed in the designation are in pounds per linear foot (plf). The uniform distributed loads from the load combinations in Section 4.2.3 are multiplied by the 8 foot tributary width for the joist spacing.

For the gravity loading:

Load per foot = (Uniform Load in psf)(Tributary width in feet)

Total Load = (29.5 psf)(8 ft) = 236 plf

Live Load = (16.0 psf)(8 ft) = 128 plf

Joist Designation:

32LH236/128

For warehouse projects without a ceiling, the same L/180 deflection limit of the steel deck may be applied to the OWSJ. This could create a significant shortcoming if some portion of the warehouse was converted to office space with a ceiling in the future. For roof members supporting a non-plaster ceiling, the live load and wind load deflection limit is a more restrictive L/240 in accordance with IBC Table 1604.3. The OWSJ live load deflection will be set to L/240 for the potential of future office use.

Live Load Deflection ≤ L/240

For wind loading, the best way to convey the required wind loading to Vulcraft is to provide a wind Zone diagram in the shape of the actual building, and a table with the load for each Zone. Table 4.9 provides the OWSJ net wind loads corresponding to each Zone in Figure 4.3.

| Open Web Steel Joist Wind Load Schedule ¹ | | | | | |
|------------------------------------------------------|------------------------------------|--------|-------------|---------------------------------------|-------------|
| Member | Net Wind Uplift (psf) ² | | | Downward Wind 1.0W (psf) ³ | |
| | Zone 1' | Zone 1 | Zones 2 & 3 | Zone 1' & 1 | Zones 2 & 3 |
| OWSJ | 2.7 | 7.5 | 12.3 | 16.0 | 17.8 |

(1)Vulcraft to use these loads in the appropriate load combinations.

(2)Net wind uplift is the result of the 0.6D+0.6W load combination.

(3)Downward wind loads are nominal loads (strength level).

Table 4.9 OWSJ Wind Load Schedule

Open Web Steel Joist Load per Foot vs. SJI Standard Series Designation

The SJI load per foot designation is the most efficient way to specify an OWSJ. Vulcraft designs OWSJ for the specific loads on the project. The load per foot designation provides the basic required total load per foot and live load per foot to be applied to the OWSJ. The OWSJ on this project will also be used as part of the wall anchorage system for the building as discussed in Sections 6 and 8. This will result in the OWSJ needing to be designed for axial loads. That requires the breakdown between dead load and live to run all the load combinations. The traditional SJI Standard designations, such as 32LH06, are based on a total uniform load and a service load that will produce an L/360 deflection. Standard designation OWSJ do not provide the breakdown between dead load and live load, therefore Vulcraft cannot run complete load combinations for wind and seismic without first sending a request for information to the designer to determine the dead load per foot and live load per foot.

When a joist is specified using the SJI Standard designation, the design professional is responsible for checking the joist for all the vertical inward load cases and deflection when selecting the standard joist designation. This requires that load cases 3, 4, and 6 as well as all the deflection requirements must be computed and compared against the SJI load tables to determine the designation. This requires additional steps and time compared to simply providing Vulcraft the total load and live load per foot as illustrated in this example.

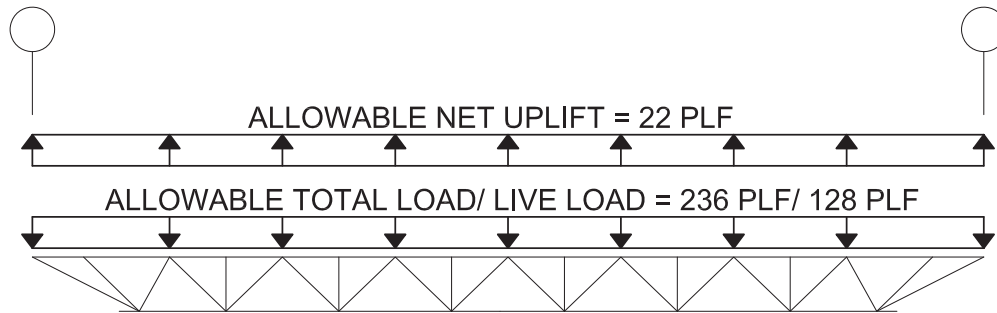
The SJI Standard designations are tied to the SJI load tables. If a design professional specifies a 32LH06 for a 50 ft span, the smallest SJI standard designation for 32 in deep, Vulcraft must design the joist to be able to support 338 plf total load as listed in the SJI load tables. In addition, Vulcraft must make the joist stiff enough that the deflection from a 211 plf load will only be L/360. The total load on this example joist was only 236 plf, for D + L. The uniform load from Load Combination 6 with Zone 2 downward is only 268 plf. This is significantly less than 338 plf. This example joist only requires L/240 deflection for 128 plf, which is significantly less than the deflection requirements for a 32LH06. As a result, the 32LH06 would be heavier than the load per foot joist to support the required loads. This would translate to a 32LH06 joist that is therefore more expensive, and requires more work by the design professional to specify than a load per foot specified joist.

4.2.6 Open Web Steel Joist Free Body Diagrams

The free body diagrams in Figures 4.22a, 4.22b, and 4.22c provide a visualization that depicts how Vulcraft would apply the specified design loads on the OWSJ. These free body diagrams are not necessary to provide to Vulcraft to convey the design loads.

The simplest joist is the typical joist in the interior field, wind Zone 1', as shown in Figure 4.22a. This joist has one uniform load designation per foot for gravity load of 236 plf total load and 128 plf roof live load and does not span across wind Zones therefore one uplift load of 22 plf resulting from the net uplift of 2.7 psf and an 8 ft tributary width (joist spacing).

$$\text{Allowable Net Uplift} = -2.7 \text{ psf (8 ft)} = -22 \text{ plf, Zone 1'}$$

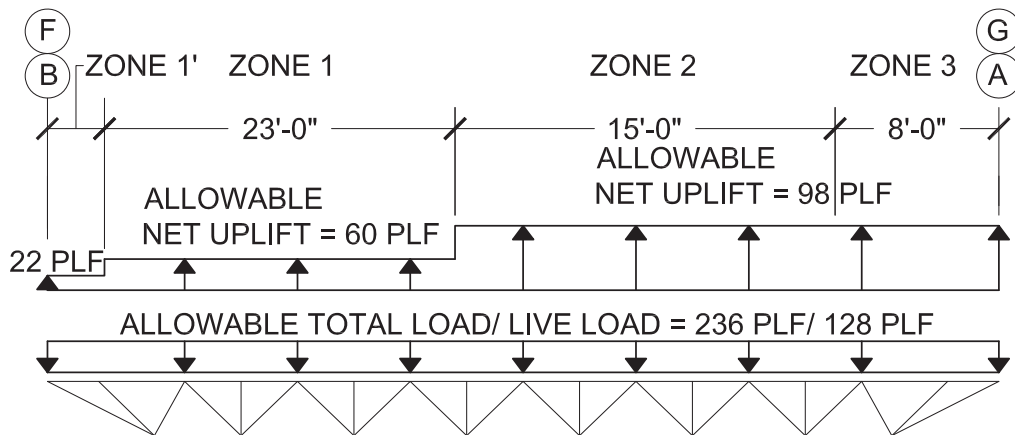


32LH236/128 with 22 plf allowable net uplift

Figure 4.22a Typical Zone 1' OWSJ

The joists near the walls span between different roof wind uplift zones. Figure 4.22b shows the loading applied to the typical joist away from the corners, along the wall on lines A or G. This OWSJ has the same gravity loading as the typical joist in the interior, but has roof net uplift from Zones 1', 1, 2, and 3.

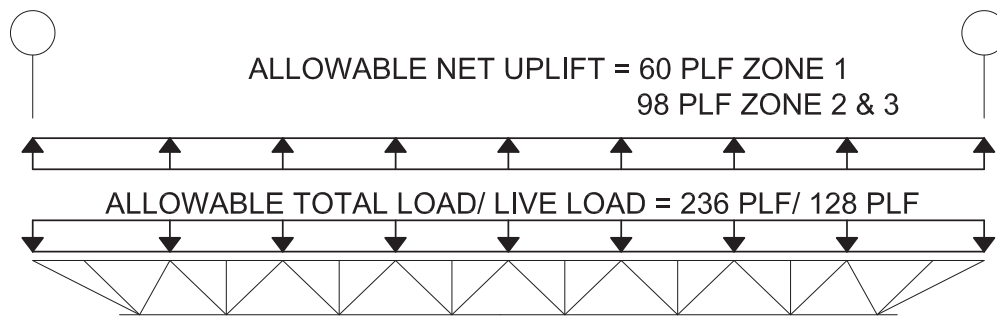
$$\begin{aligned} \text{Allowable Net Uplift} &= -7.5 \text{ psf (8 ft)} = -60 \text{ plf, Zone 1} \\ &= -12.3 \text{ psf (8 ft)} = -98 \text{ plf, Zones 2 \& 3} \end{aligned}$$



32LH236/128 with Allowable Net Uplift of 22 plf Zone 1', 60 plf Zone 1, & 98 plf Zones 2 & 3

Figure 4.22b End Wall Typical Zones 1', 1, 2 and 3 OWSJ

Figure 4.22c shows the loading applied to the typical joists near the end walls near Lines 1 and 10. These OWSJ has the same gravity loading as the typical joist in the interior but have net roof uplift from either Zone 1, or Zones 2 and 3. Keeping in mind that for the OWSJ, the net uplift in the corner Zone 3 is the same as the edge Zone 2.



32LH236/128 with allowable net uplift of 60 plf Zone 1 or 98 plf Zones 2 & 3

Figure 4.22c Typical Side Wall Zones 1 or 2 & 3 OWSJ

4.2.7 Concentrated Loads on Open Web Steel Joists

Concentrated loads on open web steel joist fall into two categories, the minimum concentrated load requirements specified in the building code and any loads from other building components or systems. These loads are commonly associated with mechanical, electrical, plumbing, and fire sprinkler systems, but may also be from other structural components such as roof screen posts, braces or fall protection anchorages.

There are two approaches to addressing concentrated loads on OWSJ. The first is to attempt to locate all the concentrated loads before the OWSJ are designed and fabricated and to have Vulcraft account for each individual load and condition. This method is often a good solution for roof top mechanical units and heavy hanging loads such as conveyor systems or fire sprinkle mains lines that are known when the building is designed and erected. The second is to have Vulcraft design all the OWSJ for a maximum concentrated load that can go at any panel point on the joist. This is called an Add-Load. Treating a concentrated load as an Add-Load may increase the weight of the joist slightly. The advantage is that the Add-Load is the easiest way to apply concentrated loads because there is no coordination required during the design of the OWSJ to account for loads at specified locations.

SJI Definition:

Add-Load: A single vertical concentrated load that occurs at any one panel point along the joist or joist girder (top chord or bottom chord). This is in addition to loads in the joist or joist girder designation.

The best approach is often to incorporate both specific concentrated loads from heavy mechanical equipment that can be easily located at the time the joist are designed and fabricated by Vulcraft, and to include an Add-Load to account for future loads that may be added after the structure is erected. Mechanical equipment loads and support frames are discussed further in Section 11.

The basic concentrated load for roofs in the building code is a 300 lb roof live load spread over a 2½ ft x 2½ ft area, which by inspection, will not govern the overall design of the OWSJ compared to the uniform roof live load. The question then is what would be an appropriate future Add-Load for a warehouse roof structure? There is not a specific building code requirement for this. An Add-Load for future loads gives the building flexibility for future tenant improvements and reduces the need for retrofit of the joists for those improvements. A 500 lb Add-Load is a common addition to the uniform load requirements of OWSJ on warehouse roofs. This size Add-Load would be adequate to support light roof top mechanical units. For warehouses that may have future office space within the warehouse, the Add-Load may be increased to 1000 lb in those areas to account for heavier mechanical equipment that may be necessary over the office space.

For this example warehouse, the OWSJ will be specified with the basic 500 lb Add-Load.

Joist Add-Load = 500 lb

4.3 Open Web Steel Joist Girder Vertical Load Design

The open web steel joist girders (OWSJG) support the OWSJ in the roof structure. OWSJG support dead load applied by the OWSJ and may support additional loads from other elements of the structure including fire sprinkler mains and miscellaneous mechanical loads.

4.3.1 Open Web Steel Joist Girder Dead Loads

The following are the applicable dead loads summarized for the OWSJG as developed in Section 2.

| | Inward Loading | Uplift Loading |
|----------------------------------------------|----------------|----------------|
| Roof System | 3.0 psf | 3.0 psf |
| Re-Roof | 2.0 psf | 0 psf |
| Steel Deck | 3.0 psf | 2.0 psf |
| Automatic Sprinklers | 2.0 psf | 2.0 psf |
| Miscellaneous | 1.5 psf | 1.5 psf |
| Open Web Steel Joists | 2.0 psf | 2.0 psf |
| Open Web Steel Joist Girders | 1.5 psf | 1.5 psf |
| <hr/> Total for Open Web Steel Joist Girders | <hr/> 15.0 psf | <hr/> 12.0 psf |

Table 4.8 OWSJG Dead Loads

The weight of 4 in and larger sprinkler mains are not covered by the pounds per square foot uniform loads in the breakdown above, as discussed in Section 4.2.1 for OWSJ. Concentrated loads will need to be provided to Vulcraft in the OWSJG notes to account for the 4 in and larger mains.

4.3.2 Open Web Steel Joist Girder Live Loads

Roof live load for this project is reducible based on the tributary area of the OWSJG. The OWSJG tributary area is one half the OWSJ span on either side of the OWSJG multiplied by the span of the OWSJG, as shown in Figure 4.23.

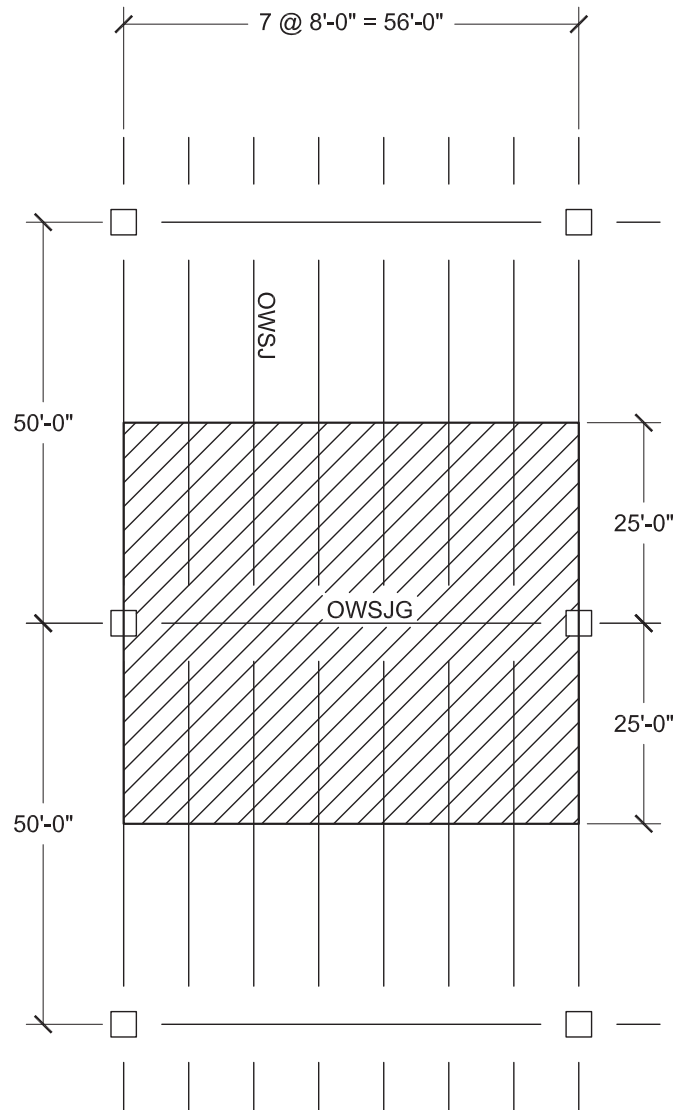


Figure 4.23 OWSJG Tributary Area

Tributary Area for OWSJG:

$$L = 56 \text{ ft OWSJG Length}$$

$$w = s/2 + s/2 = 50 \text{ ft}/2 + 50 \text{ ft}/2 = 50 \text{ ft tributary width}$$

$$A_t = Lw = 56 \text{ ft}(50 \text{ ft}) = 2800 \text{ sf}$$

IBC Section 1607.13.2.1

OWSJG Roof Live Load:

$$R_1 = 0.6 \text{ for } A_t \geq 600 \text{ sf}$$

IBC eq 16-29

$$R_2 = 1 \text{ for roof slopes } \leq 4:12$$

IBC eq 16-30

$$L_r = 20 \text{ psf}(0.6)(1.0) = 12 \text{ psf for the OWSJG}$$

IBC eq 16-26

4.3.3 Open Web Steel Joist Girder Wind Pressure

The development of the wind design pressure for the open web steel joist girders (OWSJG) is the same as the steel roof deck and the OWSJ for the first 5 Steps of the design process. The OWSJG wind design begins to deviate with determining the effective wind area in Step 6.

Step 6 (continued): The effective wind area of the OWSJG needs to be calculated to determine the External Pressure Coefficients, GC_p , for the different Zones of the roof.

As the tributary area of a member gets larger it is important to check to determine if C&C wind loads will be used or if the MWFRS is permitted to be used. ASCE 7 Section 30.2.3 permits the member to be designed for the lower MWFRS loads if the tributary area is greater than 700 sf. The OWSJG have a large tributary area and therefore may fall within the provision to use MWFRS wind loads.

The 2800 sf Tributary Area for the OWSJG is significantly larger than 700 sf, therefore the MWFRS provisions are permitted to be used for the wind loading on the Joist Girders.

The envelope procedure for low rise buildings will be used in this example to determine the MWFRS loads. This building qualifies as a low-rise building with the parapet height of 37 ft which is well under the 60 ft maximum height for low-rise buildings and the parapet height does not exceed either the 300 ft or 504 ft width and length of the building. It is acceptable to use the envelope procedure in ASCE 7 Chapter 28 because the building is regular shaped and does not have characteristics that make it subject to across-wind loading, vortex shedding, galloping, or flutter. The site must also not have channeling effects or buffeting of upwind obstructions, which are not present for this example building. Use of ASCE 7 Chapter 28 Part 1: Enclosed and Partially Enclosed Low-Rise Buildings is therefore acceptable for this example.

Step 6 (continued): The external pressure coefficients GC_{pf} are determined from ASCE 7 Figure 28.3-1. This table lists the GC_{pf} values and provides diagrams for each of the different wind Zones. The wind Zone numbers, and layout are different than those for C&C loading. In addition, the Zones are different based on the wind direction as shown in Figure 4.24a and 4.24b.

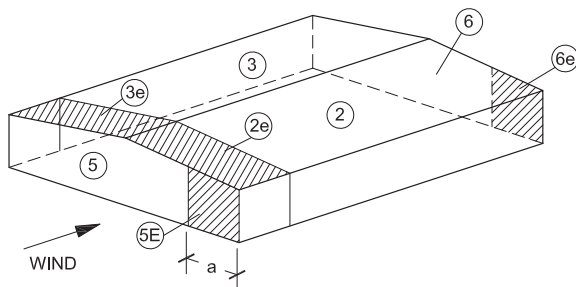


Figure 4.24a Wall Load Case A Surfaces

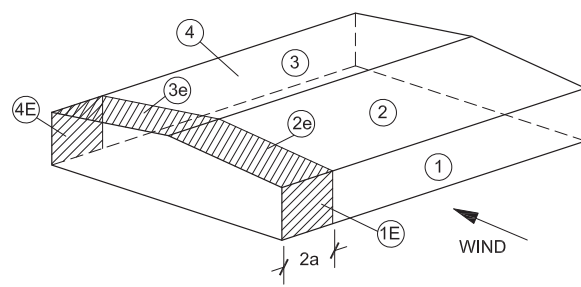


Figure 4.24b Wall Load Case B Surfaces

The width of the roof Zone, a , is 60% of the height in accordance with ASCE Figure 28.3-1.

The mean roof height above grade:

$$h = (36 \text{ ft} + 32 \text{ ft})/2 + 4 \text{ ft} = 38 \text{ ft}$$

Eave height including parapets above grade:

$$h_{\text{eave}} = 37 \text{ ft} + 4 \text{ ft} = 41 \text{ ft}$$

Zones occur at a distance, a or $2a$, from the walls where a is 10% of the shortest plan dimension, or 40% of h , (whichever is smaller), but not less than 4% of the shortest plan dimension or 3 ft in accordance with ASCE 7 Figure 28.3-1.

Edge Distance, a, shall be the minimum of:

$$0.1L = 0.1(300 \text{ ft}) = 30 \text{ ft}$$

$$0.1W = 0.1(504 \text{ ft}) = 50 \text{ ft}$$

$$0.4h = 0.4(41 \text{ ft}) = 16.4 \text{ ft (N-S)}$$

$$0.4h = 0.4(38 \text{ ft}) = 15.2 \text{ ft (E-W) - controls}$$

But no less than the lessor of:

$$0.04L = 0.04(300 \text{ ft}) = 12 \text{ ft} \geq 3 \text{ ft}$$

$$0.04W = 0.04(504 \text{ ft}) = 20 \text{ ft} \geq 3 \text{ ft}$$

Governing:

$$a = 15 \text{ ft}$$

$$2a = 30 \text{ ft}$$

The external pressure coefficients for this roof with 0.32 in per foot, or 1.5° slope, are taken from ASCE 7 Figure 28.3-1 for the 0°-5° angle category as listed in Table 4.9. There is no positive pressure coefficient for Zones 2E, 2, or 3 for a roof with a slope between 0° – 5°, therefore there is no external positive pressure wind loads.

OWSJG External Pressure Coefficients:

ASCE 7 Figure 28.3-1

| Zone | 2E | 2 | 3 |
|-----------|-------|-------|-------|
| GC_{pf} | -1.07 | -0.69 | -0.37 |

Table 4.9 OWSJG External Pressure Coefficients, GC_{pf}

There is an important allowance to reduce the wind uplift in the field of the roof by following Note 8 of ASCE 7 Figure 28.3-1. If the GC_{pf} values for Zones 2 and 2E are both negative, then the lower pressure of Zone 3 and 3E may be applied in the field of the roof.

ASCE 7 Figure 28.3-1 Note 8:

The roof pressure coefficient (GC_{pf}), when negative in Zone 2 and 2E, shall be applied in Zone 2/2E for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient (GC_{pf}) for Zone 3/3E.

The distance to the point at which it is permitted to use the lower wind pressure of Zone 3 in Zone 2 is the minimum of 0.5 x (Building Horizontal Dimension) and 2.5 x (Eave Height).

East-West:

$$b = 0.5L = 0.5(504 \text{ ft}) = 252 \text{ ft}$$

$$b = 2.5h_{\text{eave}} = 2.5(41 \text{ ft}) = 102.5 \text{ ft - controls}$$

North-South:

$$b = 0.5W = 0.5(300 \text{ ft}) = 150 \text{ ft}$$

$$b = 2.5h_{\text{eave}} = 2.5(41 \text{ ft}) = 102.5 \text{ ft} - \text{controls}$$

The application of the lower pressure over distance, b , for one wind direction for Load Case A and Load Case B is shown in Figures 4.25a and 4.25b.

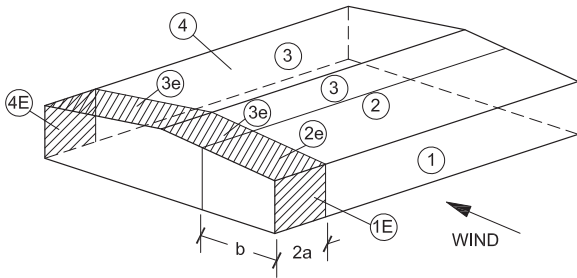


Figure 4.25a Wall Load Case A Surfaces

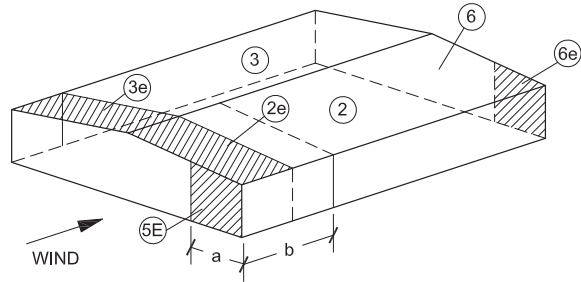


Figure 4.25b Wall Load Case B Surfaces

To develop a complete wind Zone diagram for the OWSJG in the roof structure, all 4 wind directions of Load Case A and Load Case B are overlaid. The result is the wind Zones for the OWSJG shown in Figure 4.26.

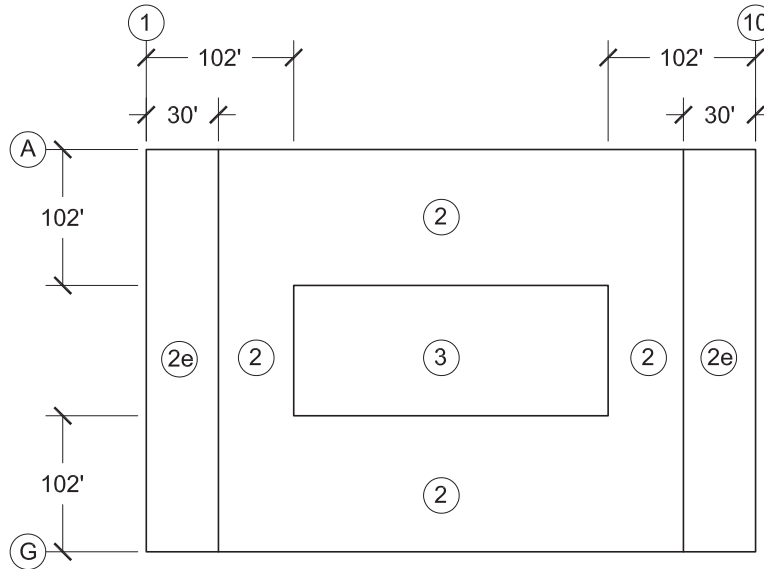


Figure 4.26 OWSJG MWFRS Wind Zones

Step 7: The wind design pressure, p , for the OWSJG is then determined based on the velocity pressure, internal pressure coefficients, and external pressure coefficients in accordance with ASCE 7 Section 28.3.1.

$$p = q_h [(GC_{pi}) - (GC_{pe})] \quad \text{ASCE 7 eq 28.3-1}$$

The minimum wind load for the MWFRS is specified in ASCE 7 Sections 28.3.4. The minimum roof wind load for enclosed or partially enclosed buildings for the design of the MWFRS shall not be less than 16 psf for walls and 8 psf for roofs projected on a vertical plane in the assumed wind direction. For this roof with a very low slope of 1.5° , this is very small when the vertical component is determined.

$$\text{Roof} = \tan(1.5^\circ)(8 \text{ psf}) = +0.2 \text{ psf minimum}$$

For Zone 2E:

$$-p_{2E} = 20.3[(-1.07)-(0.18)] = -25.3 \text{ psf (outward)}$$

$$+p_{2E} = 20.3[(0)-(-0.18)] = +3.6 \text{ psf (inward)}$$

Joist Girder design pressures for the roof are determined for all Zones.

| Zone | 2E | 2 | 3 | 2E, 2, & 3 |
|---------|-------|-------|-------|------------|
| p (psf) | -25.3 | -17.6 | -11.1 | +3.6 |

Table 4.10 OWSJG MWFRS External Pressures

4.3.4 Open Web Steel Joist Girder Load Combinations

The OWSJG vertical load is based on the ASD load combinations considering the full weight of the roof including re-roof in combination with inward wind load. For the uplift deflection, full weight of the roof structure less the re-roof is considered in combination with wind uplift, including the additional wind reduction of 0.42 is applied to the components and cladding wind pressure, following Note f of IBC Table 1604.3.

Load Combinations for Allowable Strength Design:

Inward Loading:

3. $D + (L_r \text{ or } S \text{ or } R)$

$$15.0 \text{ psf} + 12 \text{ psf} = 27.0 \text{ psf governs}$$

Load combination 6a is presented here to demonstrate that this load combination governs for wind design. Vulcraft will back the dead and live load out of the load per foot designations and combine with wind to check this load combination. The EOR therefore only needs to consider load combination 3 for the OWSJG specification.

6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$

$$15 + 0.75(0) + 0.75(0.6(3.6)) + 0.75(12) = 25.6 \text{ psf}$$

Outward Loading (net wind uplift):

7. $0.6D + 0.6W$ (Net ASD Wind Uplift)

$$0.6(12) + 0.6(-25.3) = -8.0 \text{ psf (wind Zone 2E)}$$

$$0.6(12) + 0.6(-17.6) = -3.4 \text{ psf (wind Zone 2)}$$

$$0.6(12) + 0.6(-11.1) = +0.5 \text{ psf (wind Zone 3)}$$

When net uplift is positive it means the dead load is greater than the uplift load for this Zone, therefore uplift is 0.0 psf (wind Zone 3).

For the inward wind load to impact the OWSJG design, the 0.6W from load combination 4 or the 0.75L_r + 0.75(0.6W) from load combination 7 would have to be greater than L_r.

$$0.6W = 0.6(+3.6\text{psf}) < L_r$$

$$0.75L_r + 0.75(0.6W) = 0.75(12 \text{ psf}) + 0.75(0.6)(3.6 \text{ psf}) = 10.6 \text{ psf} < 12 \text{ psf} = L_r$$

Since neither of these loads is greater than L_r , they will not control the Joist Girder design. As a result, the inward wind load will not be called out for the Joist Girder design.

The deflection check for the OWSJG follows the limits in IBC Table 1604.3. The wind loads for the Joist Girders are based on the MWFRS loads, so Footnote f would not apply, therefore it is appropriate to use the allowable wind load, $0.6W$. The OWSJG falls under steel structural members, therefore the dead load, D , is taken as zero for the total load deflection check.

Load Combinations for Deflection:

Wind deflection:

$$0.6W$$

$$0.6(3.6) = 2.2 \text{ psf (inward, Zone 2E, 2, and 3)}$$

$$0.6(-25.3) = -15.2 \text{ psf (outward, Zone 2E) - controls outward}$$

$$0.6(-17.6) = -10.6 \text{ psf (outward, Zone 2)}$$

$$0.6(-11.1) = -6.7 \text{ psf (outward, Zone 3)}$$

Total load deflection:

$$D + L_r = 0 + 12 = 12 \text{ psf}$$

4.3.5 Open Web Steel Joist Girder Specification

The OWSJG will be specified using the SJI Joist Girder designation with one addition. The standard SJI OWSJG designation only specifies the total load at the panel points. For this building, the live load portion of the panel point load will also be provided. This should be done for any project where the design professional wants Vulcraft to run a deflection check. It must be provided when the OWSJG have axial load in order to run all the load combinations. Figure 4.27 illustrates the Joist Girder designation.

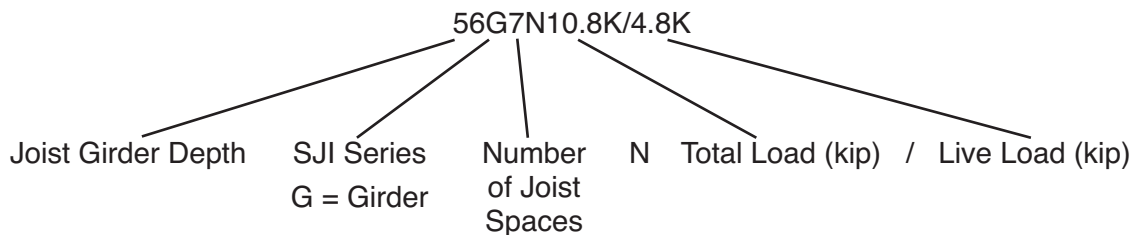


Figure 4.27 OWSJG Designation Format

The first step in building the SJI Joist Girder designation is to determine the Joist Girder depth.

Economical OWSJG depth “Rule of Thumb” for roofs is shown in Figure 4.28.

$$0.35 \times \text{Joist Spacing (in)} \leq \text{OWSJG Depth (in)} \leq 0.70 \times \text{Joist Spacing (in)}$$

$$0.35(96 \text{ in}) = 34 \text{ in} \leq \text{OWSJG Depth} \leq 67 \text{ in} = 0.7(96 \text{ in})$$

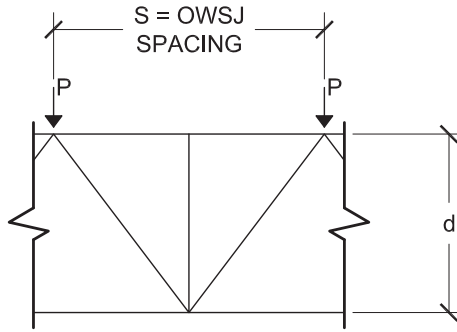


Figure 4.28 OWSJG Panel Spacing and Depth

For this project a 56 inch deep Joist Girder will be used. That depth is within the range from the rule of thumb shown above. It has also proven to be an economical choice for the 56 ft span.

The next part of the Joist Girder designation is the number of joist spaces between the ends of the girder. With the joists spaced at 8 ft o.c. and a span of 56 ft, there will be 7 joist spaces ($56 \text{ ft} / 8 \text{ ft} = 7$). The number of spaces is followed by an N to indicate it is the number of spaces. The callout for this Joist Girder would be 7N.

The last step in creating the designation is determining the panel point loading. These loads are given in kips. The uniform distributed loads from the load combinations in Section 4.3.4 are multiplied by the Joist Girder Tributary width and by the joist spacing. In Section 4.3.2, the Joist Girder Tributary Width was determined to be 50 ft.

$$\text{Panel Point Load} = (\text{Uniform Load in psf})(\text{OWSJG Tributary width})(\text{Joist Spacing})$$

$$\text{Total Load} = (27 \text{ psf})(50 \text{ ft})(8 \text{ ft}) = 10,800 \text{ lb} = 10.8 \text{ kips}$$

$$\text{Live Load} = (12 \text{ psf})(50 \text{ ft})(8 \text{ ft}) = 4800 \text{ lb} = 4.8 \text{ kips}$$

Joist Girder Designation with the L/240 deflection limit for potential future office areas as discussed in Section 4.2.4.

56G 7N 10.8K/4.8K

Live Load Deflection $\leq L/240$

The best way to convey the required wind loading to Vulcraft is to provide a wind Zone diagram, in the shape of the actual building, with the loads for each Zone. Table 4.11 provides the OWSJG wind load Zone schedule corresponding with the Zones in Figure 4.26.

| OWSJG Wind Load Schedule | | | |
|---------------------------------|--------------------------------------|--------|--------|
| Member | Net Wind Uplift (psf) ⁽¹⁾ | | |
| | Zone 2E | Zone 2 | Zone 3 |
| Joist Girder | 8.0 | 3.4 | 0.0 |

(1) Net wind uplift is the result of the $0.6D + 0.6W$ load combination.

Table 4.11 OWSJG Wind Load Diagram

4.3.6 Open Web Steel Joist Girder Free Body Diagrams

The free body diagrams in Figures 4.29a and 4.29b provide a visualization of a couple of conditions that depict how Vulcraft would apply the specified design loads to the OWSJG. These free body diagrams are not necessary to provide to Vulcraft to convey the design loads.

The simplest OWSJG is an interior joist girder in which the supported OWSJ are all within one wind zone. This OWSJG falls completely within Zone 1' and 1.

$$\text{Allowable Net Uplift} = -0 \text{ psf (50 ft)} = 0 \text{ plf, Zone 1' and 1}$$

For an OWSJG that falls completely within wind Zone 1' and 1 there is no net uplift therefore the free body diagram would only reflect the inward loads.

An OWSJ at the end walls, away from the side walls, will extend through wind Zones 2 and 2E. This girder will have progressively lower net wind uplift as it transitions toward the middle of the roof structure.

$$\text{Allowable Net Uplift} = -3.4 \text{ psf (50 ft)} = 170 \text{ plf, Zone 2}$$

$$\text{Allowable Net Uplift} = -8 \text{ psf (50 ft)} = 400 \text{ plf, Zone 2E}$$

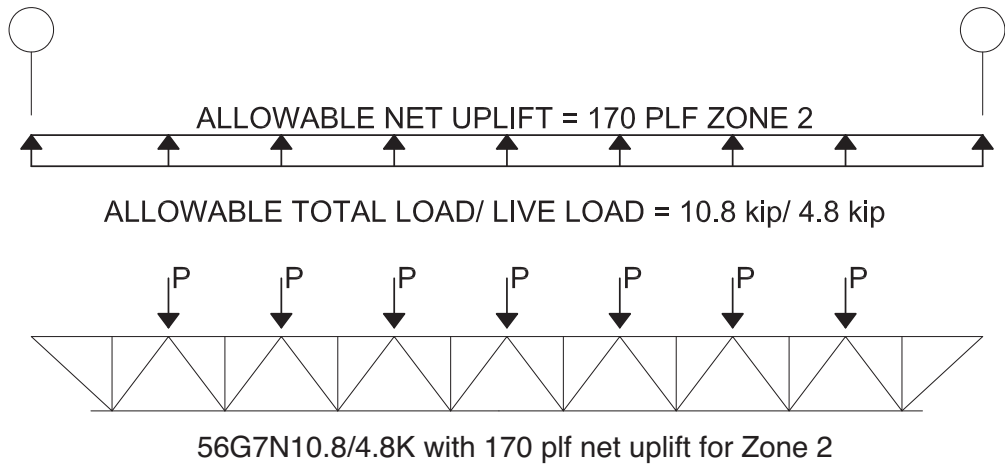
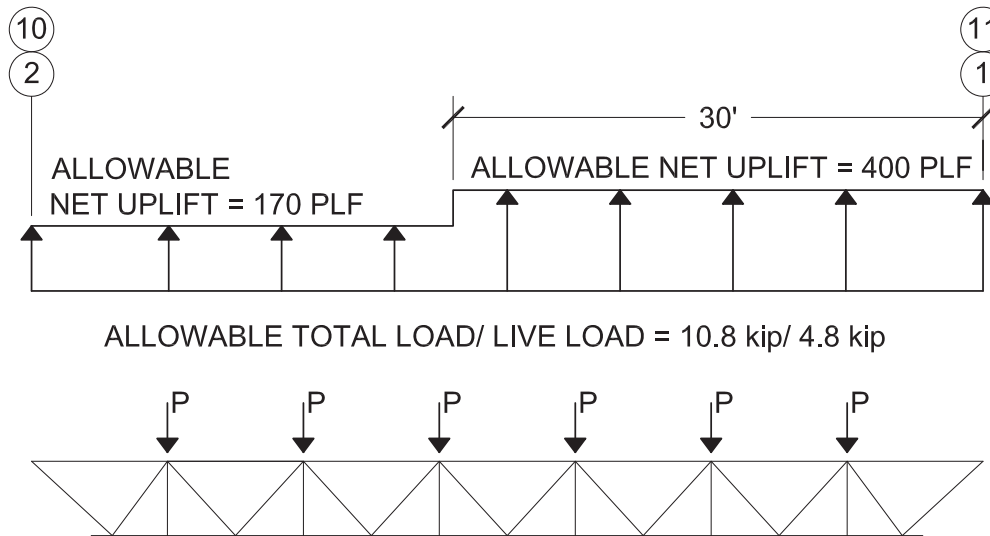


Figure 4.29a Typical Zone 2 OWSJG



56G7N12.1K/4.8K with 170 plf and 400 plf net uplift for Zones 2 and 2E Respectively

Figure 4.29b Side Wall Typical Zones 2 and 2E OWSJG

4.3.7 Concentrated Loads on Open Web Steel Joist Girders

Concentrated loads on open web steel joist girders fall into two categories, minimum concentrated loads specified by the building code and any loads from other building components. This is the same set of requirements discussed in Section 4.2.4.1 for OWSJ. The design approach is also the same as OWSJ in that either the location and magnitude of the concentrated load needs to be specified or an Add-Load provided that allows for future concentrated loads to be applied to any panel point on the OWSJG.

The basic concentrated roof live load of 300 lb over a 2½ ft x 2½ ft area is overtaken for OWSJG by the 2000 lb at “single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages” required in IBC Table 1607.1. It is not required to combine the 2000 lb point load with the uniform load applied to the OWSJG, therefore it does not govern the design because it is less than the 4.8k panel point load from the uniform roof live load.

Vulcraft will design the OWSJG for the loading provided in the designation and any concentrated loads specifically called out on the plans. The design will be as efficient as possible to support the required loading. As a result, the OWSJG may not have the strength to support any non-specified additional loads. It is common to have mechanical units or other loads added over the lifespan of the building. The best solution to accommodate future loads is with an Add-Load in the same manner as described for OWSJ in Section 4.2.4.1. For the OWSJG in a warehouse roof structure, an Add-Load of 2000 lb for future roof top units or other suspended loads has proven to be reasonable. If the design professional knows there is a high probability that the girders will get more than 2000 lb of future loads, then a larger Add-Load can be specified.

The following is the Add-Load that will be specified for this example project.

Joist Girder Add-Load = 2000 lb

For buildings that have a designated area for future office use it is common to increase the Add-Load to 4000 lb. This allows for the larger mechanical units or a greater number of small units that may be required in an office area. The example building does not have any areas with special office designations therefore the typical recommended 2000 lb Add-Load is appropriate.

5.0 Seismic Diaphragm System

The roof structure is an integral part of the seismic lateral force resisting system, acting as the diaphragm that distributes forces to the concrete shear walls. The diaphragm is analogous to a deep beam spanning between the shear walls resisting the seismic forces due to the weight of the roof structure and concrete shear walls, as shown in Figure 5.1. In diaphragm shear design, the seismic forces are applied separately to each orthogonal direction of the building. Diaphragm deflection is addressed as part of the overall building stability check in Section 5.5.

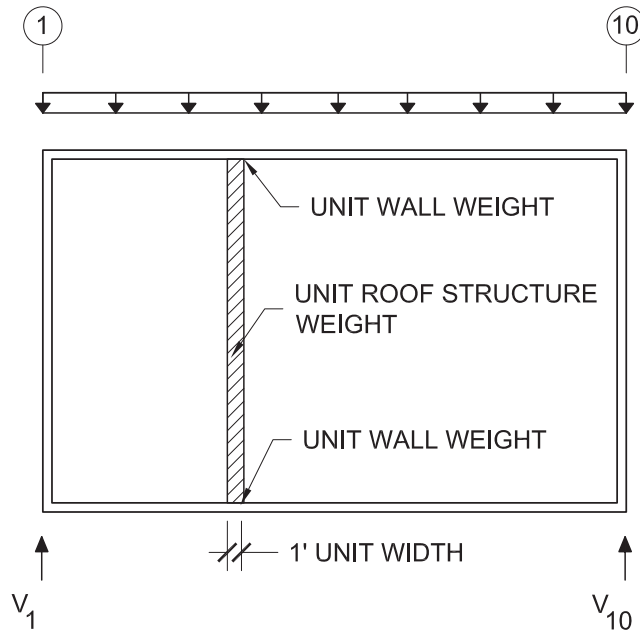


Figure 5.1 Roof Diaphragm Deep Beam Analogy

The load applied to the roof diaphragm is typically developed as a uniform load on a per foot width basis applied to the deep beam model. The most common method is to use the equivalent lateral force procedure to develop the seismic load applied to the roof diaphragm level of the building.

5.1 Seismic Force Development

Seismic design forces are determined in accordance with the provisions of ASCE 7 as specified in the IBC Section 1613. Equivalent Lateral Force Procedure is the method that is commonly used for concrete tilt-up wall warehouse structures with flexible diaphragms. Seismic design loads are defined by a number of factors, including the height of the building, vertical lateral force resisting system, site location, soil structure, occupancy of the building, and importance factor assigned to the building.

For this example, the building is a warehouse that is assigned a Risk Category of II for a structure in accordance with ASCE 7 Table 1.5-1. The warehouse occupancy does not meet the requirements to be a Category I, III, or IV occupancy for low risk, substantial risk or essential facilities for human life to be assigned a more severe risk category or seismic importance factor.

Risk Category = II

ASCE 7 Table 1.5-1

Seismic Importance Factor for a non-essential structure.

$$I_e = 1.0 \quad \text{ASCE 7 Table 1.5-2}$$

The site soil classification is assigned to Site Class D for stiff soils. This follows the requirements of ASCE 7 where soil properties are not known in sufficient detail, Site Class D shall be used unless the Authority Having Jurisdiction or geotechnical data determine the site to qualify as a different site class.

$$\text{Site Class} = D \quad \text{ASCE 7 §20.1}$$

The seismic spectral response acceleration parameters and the long-period transition period can be looked up in ASCE maps or can be obtained directly by entering the project location in web based tools such as the SEAOC / OSHPD Seismic Design Maps or the ATC Hazards by Location Map tool that provide site specific information for wind, snow, tornado and seismic conditions. The resulting Seismic Design Spectral Response for the building site in Ontario, CA are as follows:

$$S_s = 1.5 \quad \text{ASCE 7 Figure 22-1}$$

$$S_1 = 0.6 \quad \text{ASCE 7 Figure 22-2}$$

$$T_L = 8.0 \quad \text{ASCE 7 Figure 22-14}$$

Based on the building Risk Category II, Site Class D, and the seismic spectral acceleration parameters, the design spectral response accelerations are determined.

$$F_a = 1.0 \quad \text{ASCE 7 Table 11.4-1}$$

$$F_v = 1.7 \quad \text{ASCE 7 Table 11.4-2}$$

$$S_{MS} = F_a S_s = 1.0(1.5) = 1.5 \text{ (short period)} \quad \text{ASCE 7 eq 11.4-1}$$

$$S_{M1} = F_v S_1 = 1.7(0.6) = 1.02 \text{ (1-second period)} \quad \text{ASCE 7 eq 11.4-2}$$

$$S_{DS} = (2/3)S_{MS} = (2/3)(1.5) = 1.0 \text{ (short period)} \quad \text{ASCE 7 eq 11.4-3}$$

$$S_{D1} = (2/3)S_{M1} = (2/3)(1.02) = 0.68 \text{ (1-second period)} \quad \text{ASCE 7 eq 11.4-4}$$

With the building period determined, the Seismic Design Category, SDC, is determined from ASCE 7 Section 11.6 based on the seismic risk category.

$$\text{Short period SDC} = D \quad \text{ASCE 7 Table 11.6-1}$$

$$1\text{-s period SDC} = D \quad \text{ASCE 7 Table 11.6-2}$$

$$\text{Controlling SDC} = D$$

The building height for determination of the seismic lateral forces is defined in ASCE 7. The height is taken from the base to the average roof height of the structure. The base is defined as the level at which the horizontal seismic ground motions are considered to be imparted to the structure. For a warehouse building, the grade around the outside of the building is typically 4 feet below the interior floor elevation for truck loading docks as show in Figure 2.3. The tilt-up walls are typically tied to the warehouse concrete floor slab thus transmitting the seismic forces into the walls at the finished floor elevation. The seismic forces therefore enter the structure at the finished floor elevation. The building height would then be the average of the 32 ft eave height and the 36 ft ridge height for calculating the seismic lateral loads in ASCE 7.

$$h_n = (36 \text{ ft} + 32 \text{ ft})/2 = 34 \text{ ft}$$

The approximate fundamental period is allowed to be determined in accordance with ASCE 7 Section 12.8.2.1.

$$C_t = 0.020 \quad \text{ASCE 7 Table 12.8-2}$$

$$x = 0.75 \quad \text{ASCE 7 Table 12.8-2}$$

$$T = T_a = C_t h_n^x = 0.020(34)^{0.75} = 0.28 \quad \text{ASCE 7 eq 12.8-7}$$

The seismic response coefficients, C_s , used for the equivalent lateral force procedure is determined based on the design spectral response, period and response modification coefficient for the building system. This is a concrete tilt-up building designed as an intermediate precast shear wall bearing wall system.

$$R = 4 \quad \text{ASCE 7 Table 12.2-1(A5)}$$

The seismic response coefficient is determined as follows for the equivalent lateral force procedure.

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{1.0}{4.0/1.0} = 0.25 \quad \text{ASCE 7 eq 12.8-2}$$

The seismic response coefficient need not exceed the following for $T \leq T_L$.

$$T = 0.28 < 8.0 = T_L$$

$$C_{s \max} = \frac{S_{DS}}{T \frac{R}{I_e}} = \frac{0.68}{0.28 \frac{4.0}{1.0}} = 0.61 \quad \text{ASCE 7 eq 12.8-3}$$

The seismic response coefficient shall not be less than:

$$C_s = 0.044 S_{DS} I_e = 0.044(1.0)(1.0) = 0.044 \quad \text{ASCE 7 eq 12.8-5}$$

$$C_s = \frac{0.5 S_1}{R/I_e} = \frac{0.5(0.6)}{4.0/1.0} = 0.075 \quad \text{ASCE 7 eq 12.8-6}$$

$$\text{Governing } C_s = 0.25$$

For buildings that meet the requirements of ASCE Section 12.8.1.3, C_s is permitted to be calculated using a short period spectral acceleration, $S_{DS} = 1.0$, but not less than 70% of actual S_{DS} . This requires the building to be less than 5 stories tall and be a regular building, having no plan or vertical irregularities. This example warehouse building meets the requirements of this section, however for this site $S_{DS} = 1.0$ therefore S_{DS} remains unchanged.

The base shear of the example building is therefore:

$$V = C_s W = 0.25W \quad \text{ASCE 7 eq 12.8-1}$$

5.2 Diaphragm Force Coefficient

The diaphragm forces are related to the seismic base shear of the structure, but are not necessarily the same. The diaphragm force at each level of the building is determined in accordance with ASCE 7 Section 12.10 based on the story force.

$$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad \text{ASCE 7 eq. 12.10-1}$$

For a single-story building with a base shear of $V=0.25W$, developed in Section 5.1, the diaphragm design force Equation 12.10-1 simplifies to:

$$F_{px} = 0.25w_{px}$$

The diaphragm design force shall not be less than:

$$F_{px} \geq 0.2 S_{DS} I_e w_{px} = 0.2(1.0)(1.0)w_{px} = 0.2w_{px} \quad \text{ASCE 7 eq 12.10-2}$$

The diaphragm design force need not exceed:

$$F_{px} \leq 0.4 S_{DS} I_e w_{px} = 0.4(1.0)(1.0)w_{px} = 0.4w_{px} \quad \text{ASCE 7 eq 12.10-3}$$

Therefore the governing diaphragm force equation is:

$$F_{px} = 0.25w_{px}$$

5.3 Roof Diaphragm Shear Load

The steel roof deck is permitted to be idealized as a flexible diaphragm in accordance with ASCE 7 Section 12.3.1.1 because it is an untopped steel deck with concrete shear walls. As a flexible diaphragm the seismic forces for the roof are derived based on the tributary weight of the roof structure and concrete walls that are perpendicular to the direction of the seismic force.

The weight of the structure tributary to the roof diaphragm (from Section 2):

| | |
|----------------------------|---------|
| Roof Structure Dead Load = | 15 psf |
| Tilt-up Wall Dead Load = | 116 psf |

Building Geometry:

| | |
|-----------------------------------|--------|
| Top of wall = | 37 ft |
| Roof Height (North-South) = | 32 ft |
| Average Roof Height (East-West) = | 34 ft |
| Wall length (North-South) = | 300 ft |
| Wall length (East-West) = | 504 ft |

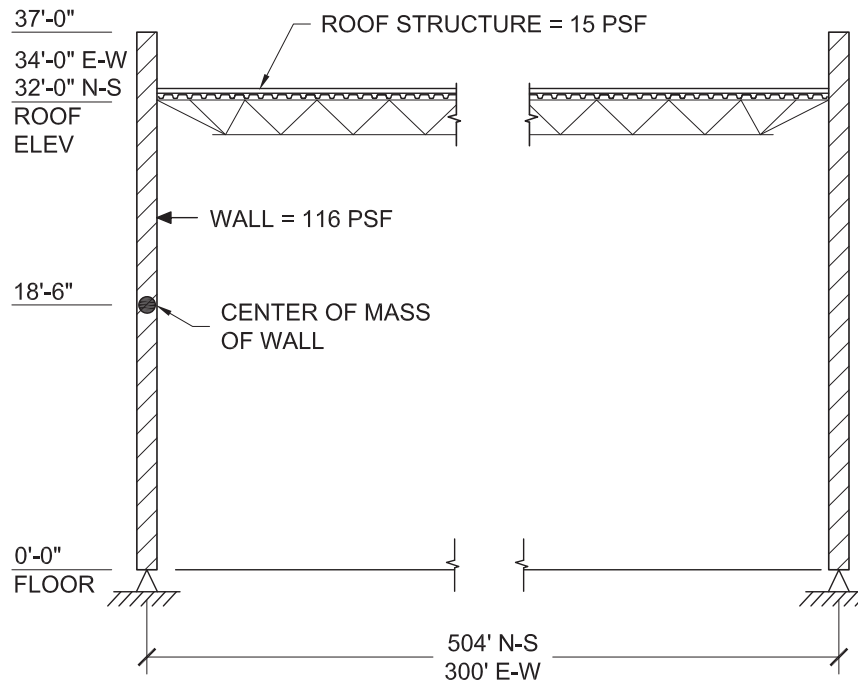


Figure 5.2 Building Cross Section

The building load tributary to the roof level is the weight of the roof structure and the tributary mass of the walls to the roof level multiplied by the seismic force coefficient. The tributary load of the walls is derived by taking the sum of the moments of the wall force relative to the floor level. For this example, the walls do not include any reduction in mass for openings. This is a reasonable assumption for warehouse structures because the most common opening is truck doors that are typically at about a 10 ft height relative to the floor level. These

doors are in the lower 1/3 of the wall height, therefore ignoring this reduction in mass has little effect on the mass tributary to the roof height.

Standard basic load combinations 6 and 7 applicable to the roof from ASCE 7 Section 2.3.6:

$$6. \quad 1.2D + E_v + E_h + L + 0.2S$$

$$7. \quad 0.9D - E_v + E_h$$

When considering the horizontal seismic diaphragm shear forces, the vertical loads are not considered; $D = 0$, $L = 0$, $H = 0$, $F = 0$, and $S = 0$. The redundancy factor, $\rho = 1.0$, following the provision of ASCE 7 Section 12.3.4.1 Note 7, for diaphragm loads determined using equation 12.10-1 for typical diaphragms. The LRFD load combinations therefore reduce to:

For LRFD:

$$F_{px} = 0.25w_{px}$$

ASCE 7 § 12.3.4.2

Diaphragm Strength Design (LRFD) vs Allowable Stress Design (ASD)

Seismic steel roof deck diaphragm design is 12% more efficient using an LRFD approach compared to an ASD approach. This stems from a disconnect between the load factors used in ASCE 7 and the method used to determine the diaphragm design safety factor from the resistance factor prescribed in AISI S100. In ASCE 7 the strength design load combinations for seismic are based on the strength level earthquake load, $1.0E$. The corresponding earthquake load for allowable stress design is 70% of the strength level earthquake load, $0.7E$. The disconnect comes from the AISI development of the safety factor based on a 1.6 load factor for live and wind load dominated combinations compared to the $1/0.7 = 1.4$ load factor that would be appropriate for seismic load effects.

LRFD

ASCE 7-16 §2.3.6 Basic Combinations with Seismic Load Effects

Horizontal Component:

$$E_h$$

Required Diaphragm Strength:

$$V_{LRFD} = E_h$$

Diaphragm Shear Design Strength:

$$\Phi V_n \geq V_{LRFD} = E_h$$

$$E_h = \Phi V_n$$

>

ASD

ASCE 7-16 §2.4.5 Basic Combinations with Seismic Load Effects

Horizontal Component:

$$0.7E_h$$

Allowable Design Strength:

$$V_{ASD} = 0.7E_h$$

Allowable Diaphragm Strength:

$$V_n / \Omega \geq V_{ASD} = 0.7E_h$$

AISI S100-16 §K2.1.2 Allowable Strength Design:

$$\Omega = 1.6 / \Phi$$

$$0.7E_h = V_n / (1.6/\Phi)$$

$$E_h = (1/0.7)(V_n / (1.6/\Phi))$$

$$E_h = 1.42\Phi V_n / 1.6$$

$$E_h = 0.89\Phi V_n$$

LRFD diaphragm design is therefore $(1-0.89)/0.89 = 0.12 = 12\%$ more efficient compared to ASD.

The factored roof diaphragm shear loads based on the tributary mass of the roof structure and walls are:

Factored Diaphragm Shear:

$$w = (\text{roof loading contribution}) + (\text{wall loading contribution})$$

$$F_{px} = 0.25w_{N-S} = 0.25[15 \text{ psf}(300 \text{ ft}) + 2[116 \text{ psf}(37 \text{ ft})(37 \text{ ft}/2)(1/32 \text{ ft})]] = 2366 \text{ plf}$$

$$F_{px} = 0.25w_{E-W} = 0.25[15 \text{ psf}(504 \text{ ft}) + 2[116 \text{ psf}(37 \text{ ft})(37 \text{ ft}/2)(1/34 \text{ ft})]] = 3058 \text{ plf}$$

The uniform seismic load applied to the roof diaphragm structure is then resolved into the maximum diaphragm forces at the perimeter shear walls. This is accomplished by considering the diaphragm as a simple beam between the end walls for a flexible diaphragm. The resolution of forces is shown in Figure 5.3.

Diaphragm Reactions at walls:

$$V_{N-S1} = V_{N-S10} = F_{px \ N-S} (L_{N-S}/2) = 2366 \text{ plf}(504 \text{ ft}/2) = 596 \text{ kips}$$

$$V_{E-WA} = V_{E-WG} = F_{px \ E-W} (L_{E-W}/2) = 3058 \text{ plf}(300 \text{ ft}/2) = 459 \text{ kips}$$

Diaphragm Shear at walls:

$$v_{N-S} = wL / (2b)$$

$$v_{N-S} = 2366 \text{ plf} (504 \text{ ft}) / (2 (300 \text{ ft})) = 1987 \text{ plf}$$

$$v_{E-W} = 3058 \text{ plf} (300 \text{ ft}) / (2 (504 \text{ ft})) = 911 \text{ plf}$$

Figure 5.3 shows the diaphragm load, diaphragm shear, and end reactions for both directions.

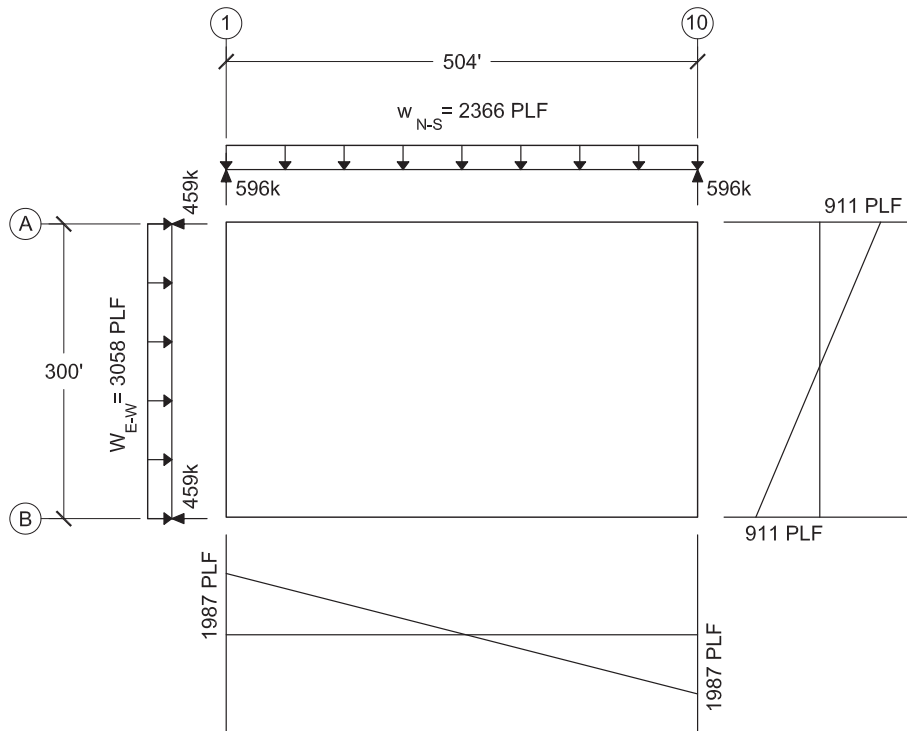


Figure 5.3 Diaphragm Forces and Shear Distribution

5.4 Skylight or Smoke Hatch Design Considerations

Most large warehouse buildings have skylights, smoke hatches or powered smoke exhaust fans. Architects typically lay out these openings in a regular pattern across the roof of the building. It is common for a warehouse project to require skylights for 2%-3% of the roof area. For this building that translates to 143 skylights. See Figure 5.4 which depicts a common skylight layout. The openings are typically aligned in rows that will reduce the effective diaphragm depth at the opening locations.

The skylights proposed for this example are 3 ft by 8 ft, which is a common skylight size. The 3 ft dimension matches the standard width of a sheet of roof deck. Skylights with a width that are not a function of 3 ft require the deck sheets to be cut in the field at every skylight, adding costs to the project. The 8 ft dimension allows the skylight frame to span between the joists. The deck ends are also supported by the joist as a result. This eliminates the need for costly below deck structural steel frames to support the skylight and the ends of the deck. Depending on the size of the warehouse, there could be several hundred to several thousand skylights/smoke hatches involved. Using an efficient skylight/smoke hatch size will minimize erection costs and minimize erection time. A 6 ft by 6 ft skylight is another example of an efficient size when joists are spaced at 6 ft on center. Further discussion of openings and reinforcement is covered in detail in Section 11.0.

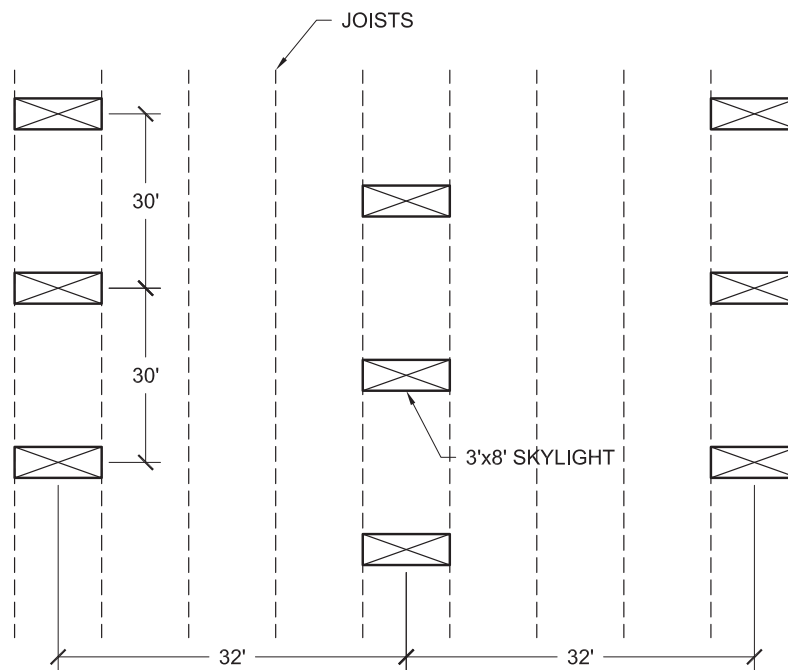


Figure 5.4 Typical Skylight Layout

This regular pattern of skylights in rows across the roof structure reduces the diaphragm depth at the opening locations. The reduction in diaphragm depth may be significant depending on the spacing of the skylights. In the North-South direction, the reduction in the depth of the diaphragm is due to 3 ft wide skylights every 30 ft across the roof structure. In the East-West direction, the reduction in the depth of the diaphragm is due to the 8 ft wide skylights every 64 ft across the roof structure (due to the stagger).

Effective Diaphragm Depth Percentage:

$$\text{Reduction}_{N-S} = [(30 \text{ ft} - 3 \text{ ft}) / 30 \text{ ft}] \times 100 = 90\%$$

$$\text{Reduction}_{E-W} = [(64 \text{ ft} - 8 \text{ ft}) / 64 \text{ ft}] \times 100 = 88\%$$

The reduction in width increases the diaphragm unit shear along the lines of skylights. The increase in diaphragm shear along the rows of skylights:

$$V_{N-S} = 1987 \text{ plf} (1/0.90) = 2208 \text{ plf LRFD}$$

$$V_{E-W} = 911 \text{ plf} (1/0.88) = 1035 \text{ plf LRFD}$$

Figure 5.5 shows the impact of the increased diaphragm shear in the North-South direction for the West side of the roof structure. The shear demand on the roof structure increases at each row of skylights relative to the adjacent diaphragm without the skylight openings.

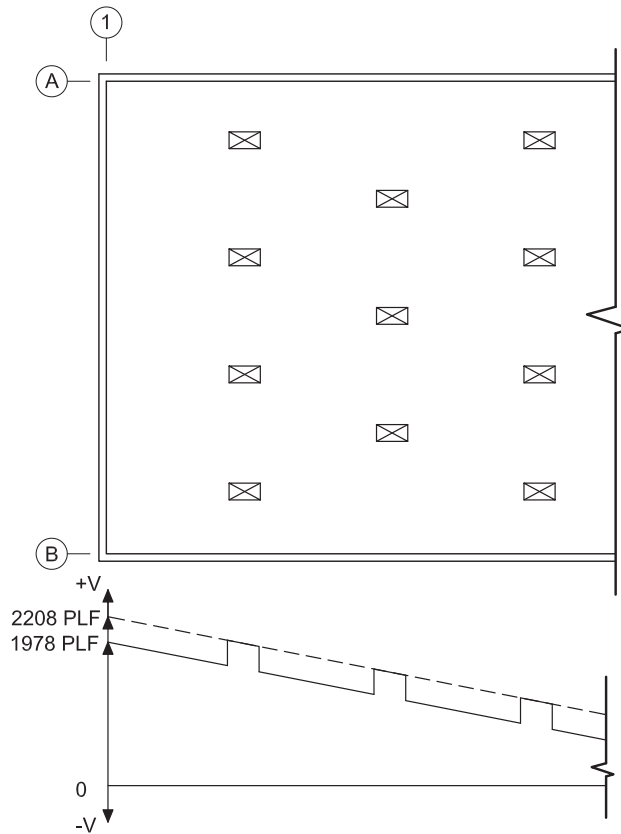


Figure 5.5 Required Diaphragm Shear with Skylights

5.5 Steel Deck Diaphragm Shear Design

Efficient diaphragm design for large structures requires a design that provides the appropriate level of shear strength for the applied shear load. In flexible diaphragms, the highest shear load is along the chords or collectors that transfer the force between the diaphragm and vertical lateral force resisting elements. For this simple rectangular shear wall warehouse, the maximum shear force is at the chords along the perimeter walls and approaches zero shear in the center of the diaphragm. For large diaphragms it is not cost effective to specify a steel deck system that can resist the maximum required shear across the entire diaphragm. The outer edges of this diaphragm have a required shear demand of up to 2208 plf. As the shear decreases, lighter gage steel roof deck along with reduced attachment patterns may be used in progressively lower shear strength zones to meet the diminishing shear demand toward the center of the diaphragm.

The attachment of steel roof deck has a large influence on the labor hours that are required. In high seismic areas, the traditional arc spot welds (puddle welds) to the open web steel joists and top seam welds for sidelaps have been replaced by high performance mechanical connectors to the open web steel joists and the deck sidelaps. For this example, the high shear performance PunchLok II system will be specified for the sidelap connections and Hilti X-HSN 24 Power Actuated Fasteners (PAFs) will be specified for the connection to the joists. This is a highly cost-effective combination that delivers high diaphragm shear performance. Pneutek power driven fasteners are another good option for high performance power actuated fasteners that are equally effective in combination with the Verco PunchLok II system. Arc spot welds have fallen out of

favor for large warehouse roof structures due to the high labor cost and low production rate of arc spot welds combined with associated quality and inspection costs.

Traditional design of steel roof deck diaphragms was based on tabular diaphragm shear strength and flexibility tables published by manufacturers and included in product evaluation reports. The evaluation reports from ICC-ES and IAPMO-UES provided an independent verification of diaphragm shear strength prior to the International Building Code (IBC) providing analytical methods to determine diaphragm strength and stiffness. The 2018 IBC changes this paradigm by providing methodology to calculate the strength of steel deck diaphragms following the provisions of AISI S310 North American Standard for the Design of Profiled Steel Diaphragm Panels. AISI S310 is the method specified in Section I2 of AISI S100 North American Specification for the Design of Cold-Formed Steel Structural Members as specified in Section 2210 of the IBC for all cold-formed steel design. As with most cold-formed steel design, the analytical equations are complex and burdensome. To overcome this complexity, Verco provides the design community with the efficient web-based Steel Roof Deck Diaphragm design tool to determine diaphragm shear strength and shear stiffness following the methods of AISI S310. This interactive web-based tool replaces hundreds of catalog pages of diaphragm shear tables, allowing the designer to quickly generate a project specific diaphragm shear table.

5.5.1 North-South Diaphragm Shear Design

The efficient design of a steel deck diaphragm requires the highest diaphragm shear strength at the ends of the diaphragm with shear dissipating to zero near the center of the diaphragm. In the N-S direction, the maximum required shear strength is 2208 plf. The first step in the design of the diaphragm is to determine an efficient deck gage and attachment pattern for the highest shear required in the North-South direction near Lines 1 and 10. This will then be followed by creating progressively lower shear deck attachment zones that correspond with the reduced shear toward the middle of the diaphragm.

Lighter gage deck is more economical than heavier gage deck for common attachment patterns when not exceeding one PAF connection per rib to the supports and PunchLok system VSC2 sidelap connections not less than 6 to 8 inches on center. The first deck that will be checked is the minimum 22 gage deck that was able to support the vertical loads developed in Section 4.1.7. The support attachment will be with the Hilti X-HSN24 PAF in a 36/7/4 pattern as shown in Figure 5.6. Experience has shown that the minimum 36/4 pattern generates low diaphragm shear capacity, and that increasing the connection pattern at the sheet ends to 36/7 greatly increases the diaphragm shear capacity with the minimum number of increased PAF connections. This essentially provides virtually the same high shear as a 36/7 pattern with the installed cost of the equivalent attachment of a 36/5 pattern for a 4 span deck sheet.

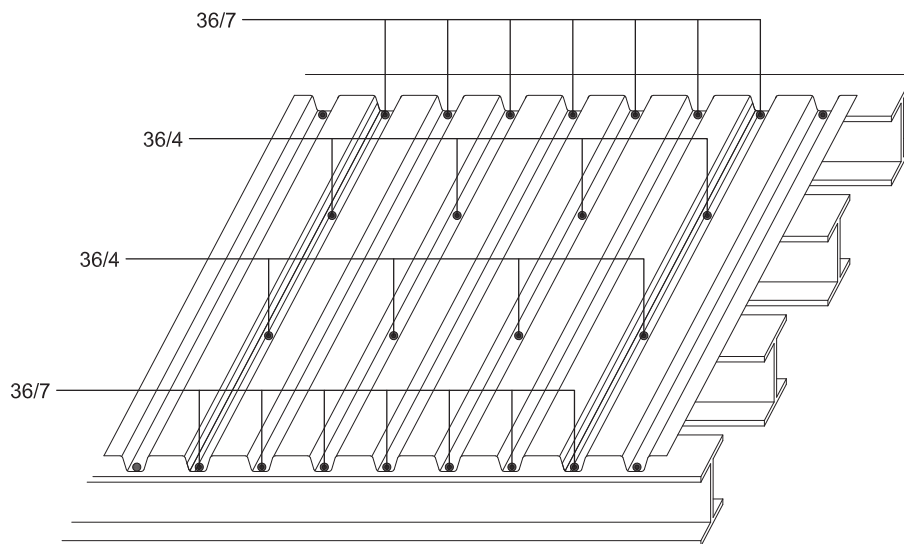


Figure 5.6 36/7/4 Attachment Pattern

The diaphragm shear strength for the PLB-36 steel roof deck will be determined using the Verco web-based Steel Deck Diaphragm tool. The following summarizes the inputs that will be used to generate the diaphragm shear table for this attachment pattern.

For Diaphragm Shear capacity, LRFD design method is being used.

Choose the 2018 Diaphragm Design web-based tool at: <https://vercodeck.com/design-tools/>

Deck Selection:

| | |
|----------------|-----------|
| Design Method: | LRFD |
| Deck Option: | Roof Deck |
| Deck Type: | PLB-36 |
| Deck Gage: | 22 |
| Deck Grade: | Grade 50 |

At this point an assumption needs to be made about how many spans the deck will be relative to the support framing. The deck layout for fabrication and erection is generally not known when the diaphragm is designed. The Steel Deck Institute recommends that a 3 span deck sheet be used for general design. This is reasonable because the shear is a bit higher for 1 and 2 span conditions and is not significantly lower for more than 3-span conditions.

| | |
|------------------|---|
| Number of Spans: | 3 |
|------------------|---|

For the connections to supports the Hilti X-HSN 24 power actuated fastener is selected. This is a fastener that is suitable for open web steel joists with a top chord thickness from 1/8 inch to 3/8 inch thick. For this project a top chord of 0.125 inch is selected. This is a reasonable minimum thickness for open web joists spanning 50 ft at 8 ft o.c. The thickness of the joist does not influence the diaphragm shear strength for seismic however it does have an impact on combined shear and wind uplift that is covered in Section 7. The grade of steel is left at A572 Grade 50. This can be changed to A529 Grade 50, the most common grade for open web steel joists, however for Hilti fasteners the grade does not influence the seismic diaphragm shear for 22 gage because the mode of failure is slotting of the deck around the shank of the fastener. The end bearing length of the deck on the support, either at the chord at the perimeter, or at an interior sheet end does potentially influence the diaphragm shear strength when exterior support local web buckling mode of failure governs. In this case a shorter deck end bearing yields a lower diaphragm shear than a longer bearing. A 2 inch end bearing is chosen assuming that the open web steel joist top chord angle will be at least a 2 inch angle size and that the bearing on the perimeter chords, typically a ledger angle, will be 2 inch or greater. For high shear diaphragms, a 36/7/4 attachment pattern representing Hilti fasteners in every low flute at the sheet ends, and every other flute at interior supports of the deck sheet, provides the highest diaphragm performance for the fewest fasteners. A perpendicular attachment pattern of 36/7 is selected for the ends of the deck sheet and a 36/4 pattern is selected for the interior supports of the deck sheet.

Support Connections:

| | |
|----------------------------------------------|--------------------|
| Deck Sheet End Condition: | Both Ends Lapped |
| Deck to Support Connection Type: | Hilti X-HSN 24 PAF |
| Support Member Thickness: | 0.125 in |
| Support Member Type: | A572 Grade 50 |
| Deck End Bearing Length: | 2 in |
| Perpendicular Attachment Pattern - End: | 36/7 |
| Perpendicular Attachment Pattern – Interior: | 36/4 |

PLB deck (as well as other “PL_” decks) utilizes the PunchLok II system, with VSC2 sidelap connections. This high strength connection has a relatively low installed cost compared to a top arc seam welded sidelap, while developing diaphragm shear strengths similar to welded sidelap connections.

Sidelap Connections:

| | |
|---------------------------|---------------------------------|
| Sidelap Connection Type: | PunchLok II Connection (VSC2) |
| Table Generated Based on: | Sidelap Connection Spacing (in) |

For the determination of diaphragm shear strength for seismic, the net wind uplift of the MWFRS can be left at 0 psf. Wind uplift is not combined with seismic forces in the load combinations in ASCE 7, therefore the combined diaphragm shear and wind uplift does not apply to seismic design. Section 7 will address the design of the diaphragm for wind forces including a check for combined shear and wind uplift. The Verco design tool generates the tables for both seismic and wind at the same time therefore the MWFRS net wind uplift from Section 7 will be entered at this time as well in this example.

Wind Uplift:

| | | |
|--------------------------------------|----------|--------------------------|
| MWFRS Design Net Wind Uplift (LRFD): | 20.9 psf | developed in Section 7.2 |
|--------------------------------------|----------|--------------------------|

The final input step when using the Verco 2018 IBC Diaphragm Tool is to select the range for the output tables. A table starting at 5'-6" joist spacing (deck span) in 0.5 ft (6 in) increments is selected which will cover the 8 ft joist spacing for this project

Table Generator:

| | |
|------------------------------|-----|
| Start Table at Span of (ft): | 5.5 |
| Spans Increment at (ft): | 0.5 |

The resulting seismic LRFD shear table is shown in Figure 5.7. For the 8 ft open web steel joist spacing, the maximum diaphragm design shear strength for 22 gage PLB-36 with Hilt X-HSN 24 fasteners is 1902 plf. The second page of the Verco 2018 IBC Deck Design output provides the LRFD wind diaphragm shear and wind uplift interaction tables presented in Section 7.

22 ga PLB™-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck



Hilti X-HSN24 PAF Connections to Supports
 36 / 7 / 4 Perpendicular Connection Pattern to Supports
 PunchLok II Connection (VSC2) Sidelap Connections

A572 GR50 Support Member or Equivalent
 $0.125 \leq \text{Support Thickness (in.)} \leq 0.375$
 2 in. Minimum Deck End Bearing Length

LRFD Design Seismic Diaphragm Shear Strength ΦS_n (plf)

Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 2111 | 2111 | 2111 | 2111 | 2111 | 1902 | 1684 | 1503 | 1349 |
| 6 | 1966 | 1960 | 1955 | 1951 | 1947 | 1902 | 1684 | 1503 | 1349 |
| 8 | 1829 | 1756 | 1767 | 1777 | 1785 | 1729 | 1684 | 1503 | 1349 |
| 12 | 1519 | 1437 | 1484 | 1414 | 1457 | 1396 | 1436 | 1382 | 1349 |
| 18 | 1214 | 1135 | 1222 | 1154 | 1093 | 1168 | 1114 | 1064 | 1130 |
| 24 | 1026 | 954 | 1065 | 1002 | 945 | 894 | 986 | 939 | 897 |
| 36 | 813 | 752 | 890 | 834 | 785 | 740 | 700 | 664 | 768 |

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 7 | 7 | 7 | 7 | 6 | 7 | 7 | 7 | 7 |
| 6 | 7 | 7 | 7 | 7 | 7 | 7 | 8 | 9 | 10 |
| 8 | 7 | 8 | 8 | 8 | 8 | 8 | 8 | 9 | 10 |
| 12 | 9 | 10 | 10 | 9 | 9 | 10 | 9 | 10 | 10 |
| 18 | 11 | 12 | 11 | 12 | 13 | 12 | 13 | 14 | 13 |
| 24 | 13 | 14 | 13 | 14 | 15 | 16 | 15 | 15 | 16 |
| 36 | 17 | 18 | 16 | 17 | 18 | 19 | 20 | 22 | 19 |

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.)

Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 186 | 184 | 185 | 183 | 184 | 183 | 183 | 182 | 183 |
| 6 | 167 | 166 | 166 | 165 | 165 | 164 | 164 | 164 | 164 |
| 8 | 158 | 153 | 154 | 154 | 154 | 150 | 151 | 151 | 152 |
| 12 | 140 | 135 | 137 | 132 | 134 | 131 | 133 | 129 | 131 |
| 18 | 122 | 117 | 122 | 117 | 113 | 117 | 113 | 110 | 114 |
| 24 | 111 | 106 | 112 | 108 | 104 | 100 | 105 | 102 | 99 |
| 36 | 98 | 93 | 101 | 97 | 93 | 89 | 86 | 83 | 90 |

Figure 5.7 22 Gage PLB-36 Seismic Shear Table with 36/7/4 Pattern

In addition to the diaphragm shear and shear stiffness summary page, the Verco web-based steel deck diaphragm design tool provides detailed calculations supporting the tabular results in the summary page. Detailed calculations may be generated using the web-based 2018 IBC Deck Diaphragm design tool for the summary tables presented in Figures 5.7, 5.8 and 5.9.

The diaphragm shear capacity from the summary table in Figure 5.7 is compared to the required diaphragm shear of 2208 plf considering skylight effects at Lines 1 and 10. From the table, the maximum diaphragm shear that the 22 ga PLB-36 with a 36/7/4 pattern of Hilti X- HSN 24 connections is 1902 plf which is less than the required shear, therefore an increased attachment pattern or a heavier gage deck will be required. In this case it is observed that the maximum shear of 1902 plf applies to all sidelap connection spacing of 6 inches or less. This indicates a steel deck panel failure of buckling either in the field of the panel or at end supports is governing, or the attachment pattern at the sheet end is governing. The next step will be to increase to 20 gage steel roof deck while maintaining the economical 36/7/4 attachment pattern to achieve the required capacity. Figure 5.8 shows the 20 gage table with all the other inputs being the same as that for the 22 gage table.

The diaphragm shear strength from the summary table in Figure 5.8 for the 20 gage deck is compared to the required diaphragm shear of 2208 plf considering skylight effects at Lines 1 and 10. Keep in mind that the first row of skylights is several joist spaces from the wall and the required shear without skylights effects is 1987 plf. From the table, the diaphragm shear capacity of the 20 ga PLB-36 with a 36/7/4 pattern of Hilti X- HSN 24 and sidelap VSC2 connections at 8" on center is 2206 plf. This capacity exceeds the required diaphragm shear without skylight effects the walls on Lines 1 and 10, therefore is acceptable at the ends of the building where there are no skylights. The design shear strength is also adequate for the shear demand considering skylights provided that the first row of skylights is a sufficient distance from wall Line 1 or 10.

20 ga PLB™-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck



Hilti X-HSN24 PAF Connections to Supports
 36 / 7 / 4 Perpendicular Connection Pattern to Supports
 PunchLok II Connection (VSC2) Sidelap Connections

A572 GR50 Support Member or Equivalent
 $0.125 \leq \text{Support Thickness (in.)} \leq 0.375$
 2 in. Minimum Deck End Bearing Length

LRFD Design Seismic Diaphragm Shear Strength ΦS_n (plf) Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 2520 | 2520 | 2520 | 2520 | 2520 | 2496 | 2211 | 1972 | 1770 |
| 6 | 2458 | 2453 | 2448 | 2444 | 2441 | 2438 | 2211 | 1972 | 1770 |
| 8 | 2311 | 2231 | 2245 | 2257 | 2267 | 2206 | 2211 | 1972 | 1770 |
| 12 | 1957 | 1861 | 1919 | 1837 | 1891 | 1819 | 1868 | 1804 | 1770 |
| 18 | 1583 | 1486 | 1598 | 1515 | 1439 | 1537 | 1470 | 1407 | 1493 |
| 24 | 1341 | 1250 | 1398 | 1319 | 1247 | 1182 | 1305 | 1246 | 1191 |
| 36 | 1058 | 980 | 1170 | 1098 | 1034 | 977 | 925 | 879 | 1020 |

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 |
| 6 | 7 | 7 | 7 | 7 | 6 | 6 | 7 | 8 | 8 |
| 8 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 8 | 9 |
| 12 | 8 | 9 | 9 | 8 | 8 | 9 | 9 | 9 | 9 |
| 18 | 11 | 12 | 10 | 11 | 11 | 11 | 11 | 12 | 10 |
| 24 | 13 | 14 | 11 | 12 | 13 | 14 | 13 | 14 | 14 |
| 36 | 17 | 18 | 16 | 17 | 18 | 16 | 17 | 18 | 16 |

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.)

Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 231 | 229 | 230 | 229 | 229 | 228 | 229 | 228 | 228 |
| 6 | 210 | 209 | 208 | 208 | 208 | 207 | 207 | 207 | 206 |
| 8 | 199 | 193 | 194 | 194 | 195 | 191 | 191 | 192 | 193 |
| 12 | 176 | 170 | 173 | 168 | 171 | 166 | 169 | 165 | 168 |
| 18 | 154 | 148 | 154 | 149 | 143 | 149 | 145 | 141 | 146 |
| 24 | 139 | 133 | 142 | 136 | 131 | 127 | 134 | 130 | 126 |
| 36 | 122 | 115 | 127 | 122 | 117 | 113 | 108 | 105 | 115 |

Figure 5.8 20 Gage PLB-36 Seismic Shear Table with 36/7/4 Pattern

Carrying the 20 gage PLB-36 steel roof deck with a tight 8 inch o.c. sidelap spacing across the entire roof as the shear dissipates to zero at the center is not an efficient design. To create an efficient design the shear strength of the steel roof deck should reflect the required shear in that area of the roof. To accomplish this the roof is broken into zones in which the design shear strength is reduced to reflect the required shear demand as shown in Figure 5.10. In the middle of the diaphragm the shear will be at its lowest. For this area the steel deck meeting the minimum vertical load requirements is selected with the minimum attachment pattern. In this case, that is the 22 gage PLB-36 deck with the minimum 36/4 attachment pattern using Hilti X-HSN 24 fasteners from Section 4.1.7 combined with wide VSC2 sidelap spacing of 24 inches o.c. The design shear strength for this condition is 770 plf taken from the table in Figure 5.9.

22 ga PLB™-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck



Hilti X-HSN24 PAF Connections to Supports
 36 / 4 Perpendicular Connection Pattern to Supports
 PunchLok II Connection (VSC2) Sidelap Connections

A572 GR50 Support Member or Equivalent
 0.125 ≤ Support Thickness (in.) ≤ 0.375
 2 in. Minimum Deck End Bearing Length

LRFD Design Seismic Diaphragm Shear Strength ΦS_n (plf)

Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1011 |
| 6 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1011 |
| 8 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1011 |
| 12 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1056 | 1011 |
| 18 | 970 | 925 | 978 | 940 | 903 | 950 | 918 | 888 | 930 |
| 24 | 853 | 806 | 883 | 843 | 805 | 770 | 836 | 805 | 776 |
| 36 | 699 | 653 | 762 | 723 | 686 | 653 | 622 | 594 | 679 |

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 |
| 6 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 |
| 8 | 13 | 12 | 13 | 12 | 13 | 12 | 13 | 12 | 13 |
| 12 | 13 | 12 | 13 | 12 | 13 | 12 | 13 | 12 | 13 |
| 18 | 13 | 14 | 13 | 14 | 15 | 14 | 15 | 15 | 14 |
| 24 | 17 | 18 | 16 | 17 | 18 | 16 | 17 | 15 | 16 |
| 36 | 22 | 24 | 20 | 17 | 18 | 19 | 20 | 22 | 19 |

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.)

Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 185 | 183 | 184 | 182 | 183 | 182 | 183 | 182 | 182 |
| 6 | 165 | 165 | 164 | 164 | 164 | 163 | 163 | 163 | 162 |
| 8 | 156 | 151 | 152 | 152 | 152 | 149 | 149 | 150 | 150 |
| 12 | 137 | 131 | 134 | 129 | 132 | 128 | 130 | 127 | 129 |
| 18 | 118 | 113 | 118 | 113 | 109 | 114 | 110 | 107 | 111 |
| 24 | 106 | 101 | 108 | 103 | 99 | 96 | 102 | 98 | 95 |
| 36 | 92 | 87 | 96 | 92 | 88 | 85 | 81 | 78 | 86 |

Figure 5.9 22 Gage PLB-36 Seismic Shear Table with 36/4 Pattern

The diaphragm zones are established based on a transition occurring at one of the open web steel joists at 8 ft o.c. The diaphragm shear reduces at a rate of 8.76 plf per ft across the length of the diaphragm. This is the slope of the shear demand line considering skylight effects in Figures 5.5 and 5.10.

$$v_{N-S} = 2208 \text{ plf at walls at Lines 1 and 10}$$

$$L = 504 \text{ ft length of diaphragm}$$

$$r_{n-s} = (v_{N-S} + v_{N-S}) / L = (2208 \text{ plf} + 2208 \text{ plf}) / 504 \text{ ft} = 8.76 \text{ plf / ft across the diaphragm}$$

To establish the diaphragm shear zone boundaries, the distance from Lines 1 or 10 to the lowest shear Zone I with a design shear of 770 plf needs to be determined. The distance to the edge of the least shear zone is the difference between the shear at the wall on Lines 1 or 10 and the design shear strength of the zone divided by the shear dissipation across the diaphragm, resulting in a distance of 164 ft. Considering all the joists are at 8 ft o.c., this is a simple process. The practical boundary for the zone would be rounded up to an 8 ft increment or 168 ft as shown in Figure 5.9

For Zone I:

$$(2208 \text{ plf} - 770 \text{ plf}) / 8.76 \text{ plf/ft} = 164 \text{ ft or rounded up to 8 ft joist spacing} = 168 \text{ ft.}$$

To design a diaphragm that is efficient for both design and construction, more than two roof zones are required. At the walls a high shear diaphragm with a strength of 2208 plf is required and the interior low shear Zone I with a design shears of 770 plf has been established. An efficient diaphragm will include intermediate diaphragm shear zones that progressively step down in design strength as the shear demand drops across the diaphragm. This economy comes from reducing the steel roof deck gage and reducing the attachment pattern. Dropping from the 20 gage steel roof deck needed at the wall to 22 gage saves on the cost of steel. For this project the intermediate zones will be selected by varying the PunchLok II sidelap spacing with 22 gage steel deck and the 36/7/4 attachment pattern to supports. Figure 5.7 provides the diaphragm shear design strength table for this condition.

The intermediate zones are selected such that they have relatively equal lengths relative to the adjacent zones. A good starting point is to divide the difference between the required at the wall, on Line 1 or 10, and the low shear Zone I into the number of zones desired. For a project of this size we will select 2 intermediate zones as follows.

$$(2208 \text{ plf} - 770 \text{ plf}) / (2+1) = 479 \text{ plf}$$

Perfectly evenly spaced zone would therefore have a difference in design shear strength of 479 plf. From this we go to the diaphragm design shear strength table to look for attachment patterns that develop a design shear strength close to these increments. Working from the lowest shear Zone I outward, the breaks in the shear zones can be established as follows using the design shears from Figure 5.7.

For Zone II:

$$770 \text{ plf} + 479 \text{ plf} = 1249 \text{ plf}$$

Therefore, select 1168 plf with a PunchLok II (VSC2) sidelap spacing of 18" o.c.

For Zone III:

$$770 \text{ plf} + 2(479 \text{ plf}) = 1728 \text{ plf}$$

Therefore, select 1729 plf with a PunchLok II (VSC2) sidelap spacing of 8" o.c.

With the design shear for Zones II and III selected, the distance from the wall at Lines 1 or 10 to the transition in zones is determined using the same process used to establish the distance to the lowest shear Zone I:

$$l_{II} = (2208 \text{ plf} - 1168 \text{ plf}) / 8.76 \text{ plf/ft} = 119 \text{ ft or rounded up to 8 ft joist spacing} = 120 \text{ ft}$$

$$l_{III} = (2208 \text{ plf} - 1729 \text{ plf}) / 8.76 \text{ plf/ft} = 55 \text{ ft or rounded up to 8 ft joist spacing} = 56 \text{ ft}$$

The design shear for the highest shear Zone IV is 2208 plf considering the effects of skylights. A design strength of 2206 plf is selected for this zone, which is just below that design shear considering skylights, however, it is above that for the diaphragm without skylights of 1987 plf. Selecting this prohibits skylights from being placed in the first joist space from the wall.

$$l_{IV} = (2208 \text{ plf} - 2206 \text{ plf}) / 8.76 \text{ plf/ft} = 0.2 \text{ ft distance to first permissible skylight location.}$$

The resulting diaphragm zones for the North-South direction are summarized in Figure 5.10 and Table 5.1 including the Zone, deck gage, attachment type to supports, attachment pattern to supports, and sidelap attachment spacing, required diaphragm shear, and design diaphragm shear strength. See Section 5.5.4 for additional check for shear strength at skylight openings.

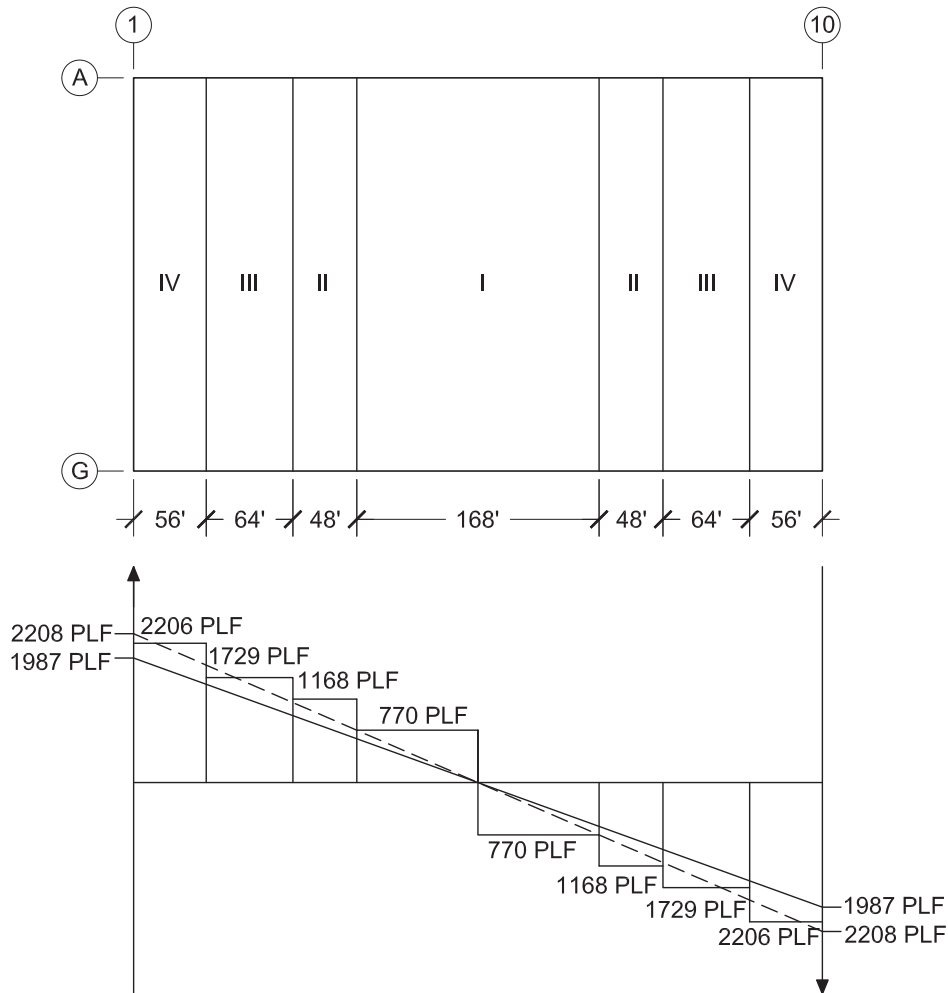


Figure 5.10 N-S Diaphragm Shear and Deck Zones

| Zone | PLB Gage | Span (ft) | Fastener to Supports | | | Sidelap Connection VSC2 (in o.c.) | Required Shear LRFD | | Design Shear Strength ϕV_n (plf) |
|------|----------|-----------|----------------------|----------|------------------------------------------|-----------------------------------|---------------------|------------------------|----------------------------------------|
| | | | Pattern | Type | Parallel chords and collectors (in o.c.) | | With Skylight (plf) | Without Skylight (plf) | |
| IV | 20 | 8 | 36/7/4 | X-HSN 24 | 7 | 8 | 2208 | 1988 | 2206 |
| III | 22 | 8 | 36/7/4 | X-HSN 24 | 8 | 8 | 1717 | 1546 | 1729 |
| II | 22 | 8 | 36/7/4 | X-HSN 24 | 12 | 12 | 1157 | 1041 | 1168 |
| I | 22 | 8 | 36/4 | X-HSN 24 | 16 | 24 | 736 | 662 | 770 |

Table 5.1 Diaphragm Shear Zones

5.5.2 East-West Diaphragm Shear Design

The diaphragm design in the East-West direction has lower required shear than in the North-South direction. The diaphragm shear attachment zones developed for the North-South direction will be used for the East-West direction. Zone II in the North-South direction exceeds the demand in the East-West direction therefore it is used for the highest shear zone in the East-West direction. See Figure 5.11 depicting the East-West diaphragm design.

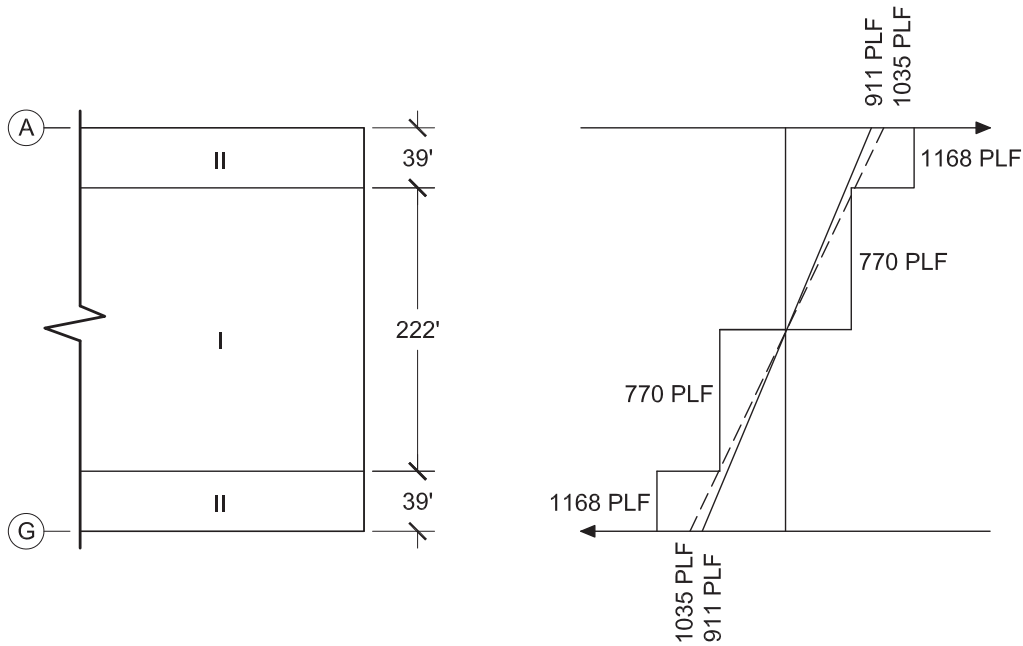


Figure 5.11 E-W Diaphragm Shear Zone Design

Following the same methods for the North-South direction, the rate of diaphragm shear dissipation toward the center of the diaphragm is determined to be 6.9 plf/ft.

$$v_{E-W} = 1035 \text{ plf at walls at Lines A and G}$$

$$L = 300 \text{ ft length of diaphragm}$$

$$r_{n-s} = (v_{E-W} + v_{E-W}) / L = (1035 \text{ plf} + 1035 \text{ plf}) / 300 \text{ ft} = 6.90 \text{ plf / ft across the diaphragm}$$

Using the shear zones established for the North-South direction, the distance to the corresponding zones in the East-West direction are determined. In the East-West direction, the transitions should be a function of the deck sheet width of 3 ft. Considering the deck sheets are at 3 ft wide the practical boundary for the zone would be rounded up to an 3 ft increment or 39 ft as shown in Figure 5.9

For Zone I:

$$(1035 \text{ plf} - 770 \text{ plf}) / 6.9 \text{ plf/ft} = 38.4 \text{ ft or rounded up to 3 ft deck width} = 39 \text{ ft}$$

5.5.3 Resulting Diaphragm Shear Zones

The resulting diaphragm shear zones are determined by overlaying the North-South and East-West diaphragm design zones as shown in Figure 5.12. These may not be the final zones for the roof structure. The wall anchorage and continuous tie requirements for concrete walls may influence the diaphragm design if sub-diaphragms are required to distribute these forces. This is fully investigated in Section 6.

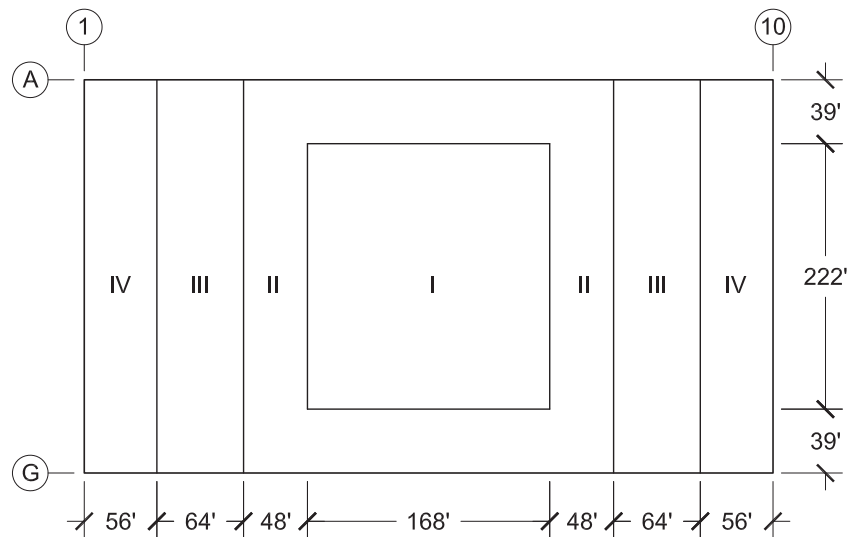


Figure 5.12 Roof Diaphragm Zone Diagram for Seismic Shear Loads

5.5.4 Shear Transfer Around Skylight or Smoke Hatch Openings

All openings in a diaphragm develop chord forces around openings. For many large diaphragms with relatively few small openings the chord forces are very small and are generally ignored. For a warehouse roof structure with a regular pattern of skylight or smoke hatch openings, consideration of the chord forces around the openings may need to be addressed. Figure 5.13 depicts the shear forces applied to the diaphragm between the skylight openings. The shear force applied to the deck between the openings on the joist lines create tension and compression chord forces in the steel deck at each opening. The costly traditional method would be to add a structural steel opening frame to resist these chord forces. A more cost-effective solution is to use the inherent axial strength of the steel deck to resist the chord forces. This is a complex analytical problem because the deck does not act as an ideally rigid body which makes it difficult to predict the width of steel roof deck that the chord forces are distributed over. Figure 5.13 shows the general area of the axial tension and compression fields that are imposed on the steel roof deck at the openings due to the chord forces of the diaphragm between the openings.

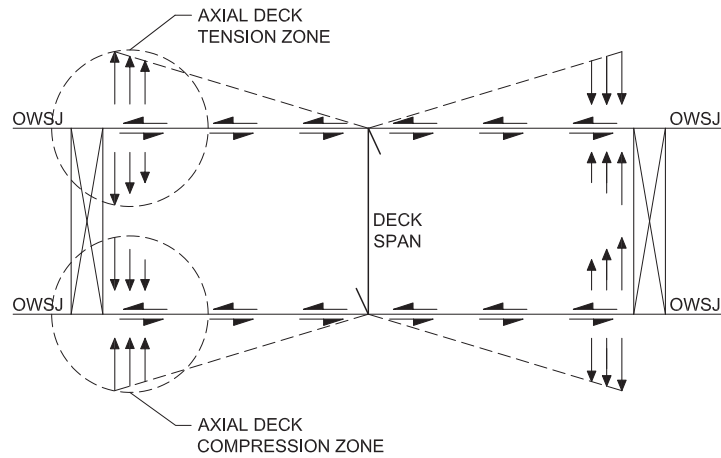


Figure 5.13 Shear Flow Between Skylights

To resolve these design complexities for common conditions, Verco undertook a full-scale diaphragm test program which lead to an innovative, low cost detail using the non-structural cold-formed steel roof curb to reinforce the diaphragm as shown in Figure 5.14. The results of this test program are presented in the Maximum Diaphragm Shear Between Openings Reinforced with Cold-Formed Steel Curbs table in IAPMO ER-2018. The IAPMO openings table presents the maximum diaphragm shear based on testing that the steel deck reinforced with a cold-formed steel roof curb can resist in an easy to use tabular form for common conditions as shown in Figure 5.15.

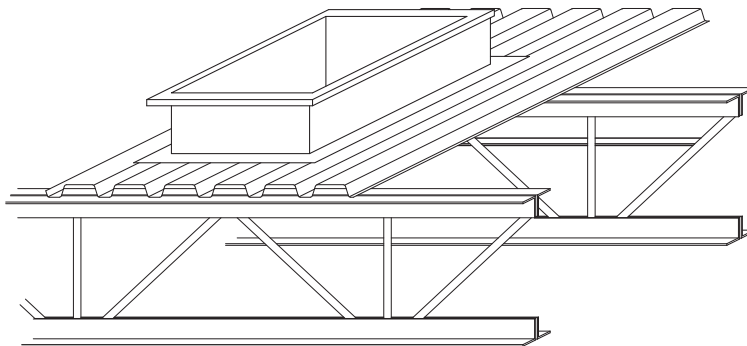


Figure 5.14 Openings Reinforced with Cold-Formed Steel Roof Curbs

| MAXIMUM DIAPHRAGM SHEAR BETWEEN OPENINGS REINFORCED WITH COLD-FORMED STEEL CURBS ^{1,2a-f} | | | | | | | |
|----------------------------------------------------------------------------------------------------|-----------|-----------------------------------------------------|-------|--------|-------------------------------------------------|-------|--------|
| Deck Profile | Deck Gage | ASD - Allowable Diaphragm Shear, S_n/Ω (plf) | | | LRFD - Design Diaphragm Shear, ϕS_n (plf) | | |
| | | Span Length (ft-in) | | | Span Length (ft-in) | | |
| | | 6'-0" | 8'-0" | 10'-0" | 6'-0" | 8'-0" | 10'-0" |
| PLB-36 HSB-36 | 22 | 1127 | 1116 | - | 1831 | 1814 | - |
| | 20 | 1408 | 1398 | 1313 | 2288 | 2272 | 2134 |
| | 18 | 1929 | 1920 | 1914 | 3135 | 3120 | 3110 |

Figure 5.15 Maximum Diaphragm Shear Between Openings Reinforced with Cold-Formed Steel Curbs (IAPMO ER-2018)

To confirm that Cold-Formed Steel Curbs can be used to reinforce the skylight openings, the worst-case diaphragm shear for each deck gage is checked against the capacities in the IAPMO report.

20 GA with 8'-0" span: $\Phi S_n = 2272 \text{ plf} > v = 2208 \text{ plf}$

22 GA with 8'-0" span: $\Phi S_n = 1814 \text{ plf} > v = 1728 \text{ plf}$

The diaphragm shear is less than the strength in the IAPMO report, therefore Cold-Formed Steel Curbs may be used to reinforce the openings, eliminating the need for costly below deck structural steel support frames. Section 11.1 provides additional discussion and details regarding reinforcing of openings in steel roof diaphragms.

5.6 Diaphragm Chords

Flexible diaphragm design is idealized with the assumption that the steel roof deck panels resist the in-plane diaphragm shear and that diaphragm chords resist the in-plane bending. An analogy may be drawn to a steel wide flange beam in which the steel deck acts as the web and the chords act as the flanges.

The most efficient location of the chords to resist bending is as far apart as possible at the diaphragm edges. For concrete tilt-up, precast concrete and unit masonry buildings, the walls are often designed using heavy steel channels, steel bent plate channels, or angles as combined chords and ledgers to support the steel deck. The maximum chord forces are a result of the tension-compression couple of the diaphragm chords. Figure 5.16 depicts the traditional loading and resistance of the diaphragm chords in the North-South direction. The East-West direction is similar.

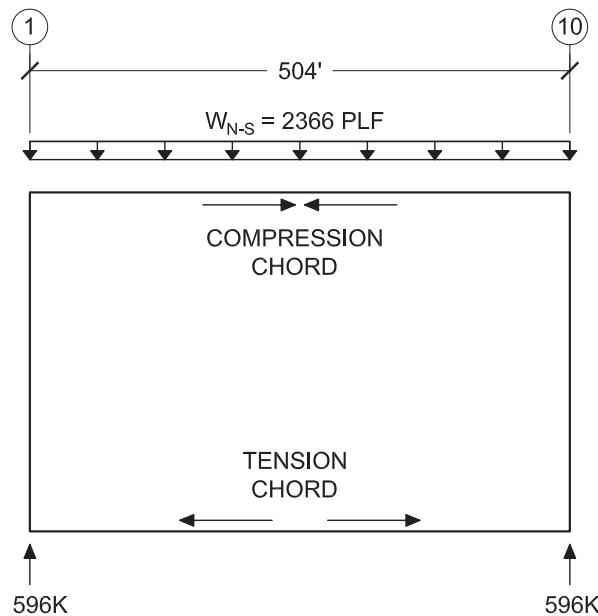


Figure 5.16 Traditional Seismic Diaphragm Chord Forces

The maximum moment of the diaphragm (LRFD):

$$M_{\max} = wL^2/8$$

The maximum chord force of the diaphragm:

$$T_{\max} = C_{\max} = M_{\max}/b = wL^2/(8b)$$

$$\text{For N-S: } T = C = (2.366 \text{ kip/ft})(504 \text{ ft})^2/(8(300 \text{ ft})) = 250 \text{ kips}$$

$$\text{For E-W: } T = C = (3.058 \text{ kip/ft})(300 \text{ ft})^2/(8(504 \text{ ft})) = 68 \text{ kips}$$

Figure 5.17 shows bent plate channel and angle ledgers.

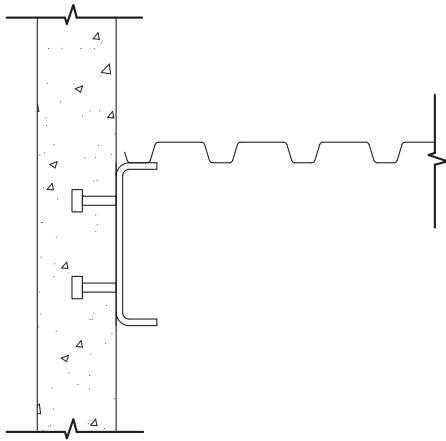


Figure 5.17a Bent Plate Channel Chord

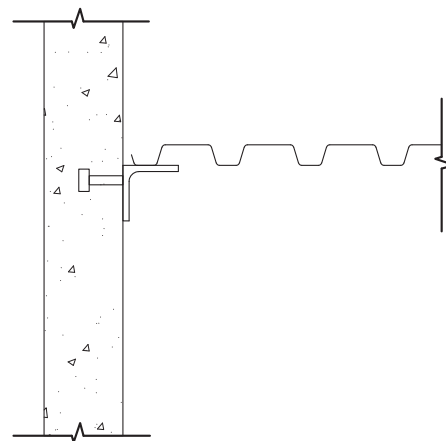


Figure 5.17b Angle Chord

The required chord steel using ASTM A36 hot rolled channels or angles in the North-South direction in accordance with AISC 360 is as follows.

$$\text{For N-S: } A_s = T/\phi F_y = (250 \text{ kips})/(0.9 (36 \text{ ksi})) = 7.73 \text{ in}^2$$

Use hot rolled standard channel C 12x 30 = 8.81 in²

A popular alternate to the hot rolled channel is a bent plate that provides several advantages over the hot rolled channel. One advantage to bent plate is it can be fabricated out of Grade 50 steel, reducing the weight of steel in the chords. The bent plate can also be sized to meet the specific area of steel needed rather than the next largest size standard channel to optimize the steel usage.

For N-S: 5/16" thick bent plate option

$$A_s = T/\phi F_y = (250 \text{ kips})/(0.9 (50 \text{ ksi})) = 5.56 \text{ in}^2$$

Required plate width:

$$\text{Width} = A_s / (5/16 \text{ in}) = 5.56 \text{ in}^2 / (5/16 \text{ in}) = 17.8 \text{ in}$$

Use 5/16" x 18" wide plate to form the bent plate ledger.

A 3 in top flange is common for a bent plate channel ledger. This provides plenty of room for the deck and its attachment. To ensure the correct area of the bent plate is provided, the overall width of the plate should be shown in the structural details. The reason for this is there is a minor amount of stretching when the plate is bent. With 3 in flanges, the height of the bent plate channel will be approximately 12 in, similar to the rolled channel option. Figure 5.18 illustrates how this bent plate channel should be called out in the structural details.

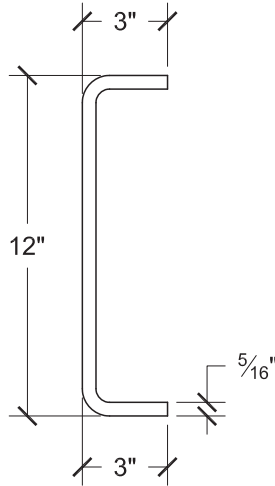


Figure 5.18 Bent Plate Detail

The third advantage of the 5/16" thick bent plate channel is it is within the allowable range for the X-HSN24 fastener. The Hilti X-HSN24 fastener selected for the diaphragm is suitable for 1/8 in thick to 3/8 in thick support steel. The C12 x 30 has a minimum flange thickness of 1/2 in, which exceeds the range for the Hilti X-HSN24 fastener. The channel requires a different fastener to be used at the ledger than the rest of the building. The Hilti X-ENP-19 fastener is suitable for 1/4 inch and thicker support steel and can be substituted for the X-HSN24 fastener for the channel ledger option. If a hot rolled channel ledger is used, it is important that this substitution be noted in the steel deck attachment schedule, so the proper fasteners are installed in the field. See Steel Deck Schedule in the Plans and Details Appendix for an example of this call out.

For the E-W direction, where the deck frames into the wall, it is common to use an angle for the ledger.

$$\text{For E-W: } A_s = T/\phi F_y = (68 \text{ kips})/(0.9 (36 \text{ ksi})) = 2.11 \text{ in}^2$$

$$\text{Use L 5 x 3 x 5/16 (Long Leg Vertical) = 2.41 in}^2$$

Similar to the North-South ledger, the angle ledger should be limited to 5/16 in thick, if possible, otherwise the X-ENP-19 pin would have to be specified. For buildings with large chord forces, the required area of steel may exceed those available with 5/16 in thick angles. In cases like this, one option would be to use a 5/16 in bent plate ledger. A second option would be to use the ledger and the joists in combination as a collective chord member, with forces distributed based on strain.

5.7 North-South Diaphragm Deflection and P-Delta Check

The in-plane deflection of the diaphragm needs to be considered to ensure the stability of the structure. ASCE 7 provides little more than a broad statement of how to address diaphragm deflection.

12.12.2 Diaphragm Deflection: The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

The intent of this broad statement is to ensure that structural components remain stable, avoiding large displacements that could lead to excessive P-delta effects on the vertical systems. For the non-structural components, the deflection limit ensures that the connections are not compromised, which could lead to

localized collapse, and falling hazards of non-structural elements of the building. The story drift limitations of ASCE 7 Table 12.12-1 apply to the vertical Seismic Force Resisting System elements, not the diaphragm. Neither the IBC nor ASCE 7 provide specific requirements for the diaphragm, leaving deformation compatibility up to the engineering judgment of the design professional.

For steel roof deck diaphragms, the primary stiffness that governs the diaphragm deflection is a result of the steel roof deck and the connection of the steel deck to the structure. Less significant deflection may be attributed to the in-plane flexure of the diaphragm. The deflection of the diaphragm may be slightly offset by stiffness contributions from partial fixity of columns and walls that are assumed to be pin-based but have some fixity due to being tied to both the footings and the floor slab. These small stiffnesses that may reduce the diaphragm deflection are ignored when compared to the large deflections attributed to the steel roof deck diaphragm panels from the original design of the building.

The determination of the deflection of the roof structure is based on the deep beam analogy similar to that used to determine the shear strength for the roof diaphragm system. The deflection is the result of the in-plane shear deflection of the steel roof deck and connections acting as a stressed skin.

Deflection for a uniformly loaded simple span diaphragm is shown in Figure 5.19:

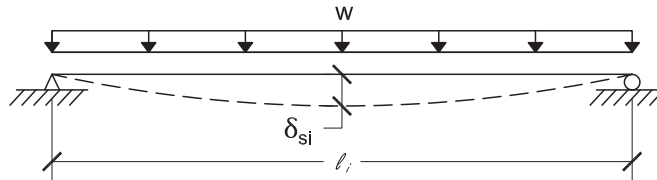


Figure 5.19 Shear Deflection of a Simple Beam

$$\delta_{si} = \frac{v_{i\text{ ave}} L_i}{1000 G'}$$

$v_{i\text{ ave}}$ = Average shear on cantilever segment, lb/ft

L_i = Length of cantilever, ft

G' = Shear Stiffness, kip/in

1000 in the formula above is for converting G' to lb/in

To determine the strength level shear deflection for a multi-zone diaphragm, the diaphragm is idealized as a series of guided cantilevers. The cantilever on each side of the diaphragm for a given zone represents one half of the simple beam. The maximum deflection will occur where the shear is zero. The maximum deflection will be the sum of the deflections of the cantilever segments between the edge of the diaphragm and the point of zero shear. Since this example building has a constant uniform load, the shear will be zero at the middle of the diaphragm.

The predictive deflection equation is one half a simple span with a uniform shear. This is appropriate for use in this example because the building is symmetric with one half of each span represented by the corresponding shear zone on each half of the diaphragm.

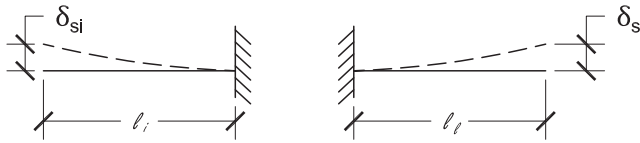


Figure 5.20 Cantilever Shear Deflection

$$\delta_{si} = \frac{0.5v_{i\text{ ave}}L_i}{1000G'}$$

$v_{i\text{ ave}}$ = Average shear on cantilever segment, lb/ft

L_i = Length of cantilever, ft

G' = Shear Stiffness, kip/in

The Shear Stiffness factor, G' , is a function of the steel roof deck profile, gage, span, connections and connection pattern. This shear stiffness is calculated following provisions of AISI S310. This is an even more tedious calculation than the shear strength. To aid the designer, the Verco web-based steel deck diaphragm design tool performs the calculation for the shear stiffness and presents the results in the same summary tables as the diaphragm shear strengths shown in Figures 5.7, 5.8, and 5.9.

The longer length of the diaphragm of 504 ft is in the East-West direction, therefore the North-South diaphragm deflection will be the worst-case deflection. Figure 5.21 and Table 5.2 summarize the determination of the diaphragm shear deflection.

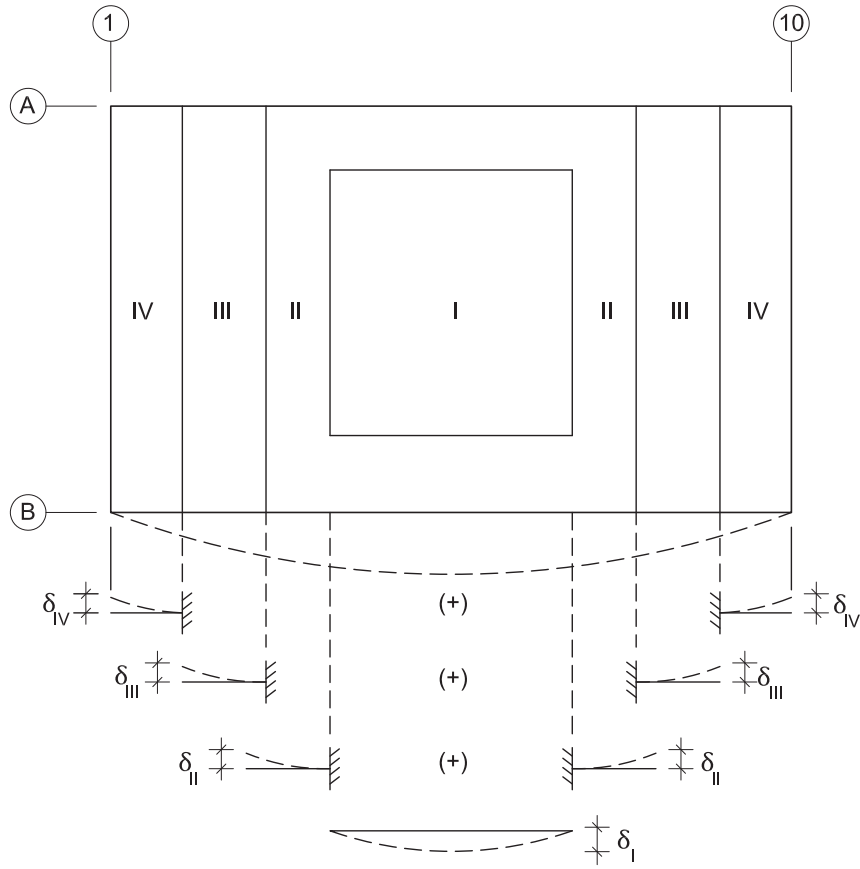


Figure 5.21 Deflection Zones as a Series of Cantilevers

The diaphragm shear for deflection does not include skylight effects. This is a judgment based on there being more field of the diaphragm without skylights than with skylights. This follows common design practice for flexible diaphragms.

| Zone | v_{left} plf | v_{right} plf | $v_{i\ ave}$ plf | L_i ft | G' kip/in | $0.5(v_{i\ ave} L_i)/(1000G')$ in |
|-----------------------------------------------------------|-------------------|--------------------|---------------------|-------------|----------------|--------------------------------------|
| IV | 1987 | 1545 | 1766 | 56 | 191 | 0.26 |
| III | 1545 | 1041 | 1293 | 64 | 150 | 0.28 |
| II | 1041 | 662 | 852 | 48 | 117 | 0.17 |
| I | 662 | 0 | 331 | 84 | 96 | 0.14 |
| I | 0 | 662 | 331 | 84 | 96 | 0.14 |
| II | 662 | 1041 | 852 | 48 | 117 | 0.17 |
| III | 1041 | 1545 | 1293 | 64 | 150 | 0.28 |
| IV | 1545 | 1987 | 1766 | 56 | 191 | 0.26 |
| Total Elastic Strength Level Shear Deflection, δ_v | | | | | | 1.71 |

Table 5.2 Steel Roof Deck In-Plane Shear Deflection

Addition of deflection due to flexure using slender beam mechanics is not appropriate. The basic assumptions of slender beam mechanics are not achieved, as noted below. Figure 5.22 shows the roof diaphragm as a deep beam with a width (depth) to length ratio of 1 : 1.7. When the diaphragm deflects, the end walls cannot rotate to stay normal to the deflected centerline of the diaphragm because they are restrained by the longitudinal walls. Further the chords of the diaphragm cannot change substantially in length because they are restrained by the shear stiffness of the concrete shear walls.

Slender beam theory assumptions and how they are not met:

1. For slender beams, the beam is several times longer than the depth, typically a minimum of 3 to 5 times the depth, which is not the case for this building.
2. Beam theory requires an isotropic (or orthotropic) and homogeneous material. The in-plane steel deck diaphragm stiffness is an order of magnitude greater than the chords members. Even though both elements are steel, the difference in stiffness prohibits the diaphragm from activating the chords in flexure as a simple beam.
3. The beam is assumed to be in pure bending, plane sections remain plane, which is not the case because the diaphragm cannot deflect and stay normal to the centerline of the deflected shape.

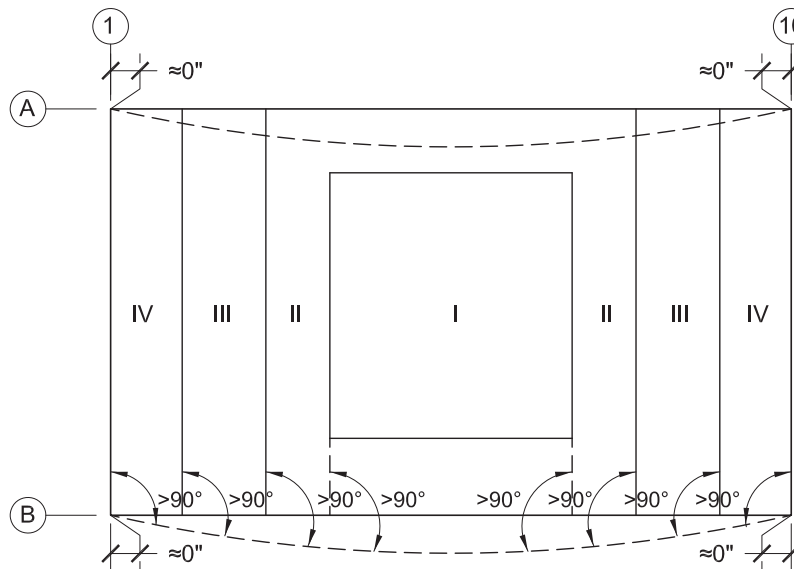


Figure 5.22 Shear Deflection of Diaphragm

Therefore, the appropriate diaphragm deflection is based on shear deflection of a deep beam.

The inelastic diaphragm deflection is determined from the unfactored strength-based diaphragm load (1.0E). The 1.71 inches of deflection calculated above is the strength-based, elastic deflection. The amplified maximum diaphragm deflection is based on the ASCE 7 Section 12.8.6 Story Drift Determination using Equation 12.8-15.

$$\delta_x = C_d \delta_{xe} / I_e$$

ASCE 7 eq 12.8-15

$$C_d = 4$$

ASCE 7 Table 12.2-1 (A5)

$$I_e = 1.0$$

see Section 4

$$\delta_{xe} = \delta_s = 1.71 \text{ in}$$

$$\delta_x = 4.0(1.71 \text{ in})/1.0 = 6.84 \text{ in}$$

In addition to checking to ensure that the diaphragm deflection does not compromise other structural and non-structural attached elements, the deflection of the diaphragm should be investigated to ensure building stability. As the diaphragm deflects, the weight tributary to the roof level diaphragm moves from the center of the building. This shift in the center of mass adds additional lateral load (thrust) to the structure that may lead to further deflection. The increased deflection then may contribute more lateral load and thus further deflection. The deflection must be investigated to ensure structural instability from the P-delta effect does not lead to progressive collapse.

ASCE 7 Section 12.8.7 was developed to investigate the P-delta effect of drift on the vertical lateral force resisting systems for multi-story buildings and was not intended for use to analyze large diaphragms in single story buildings. In the absence of prescriptive requirements, it is reasonable to apply this method to rigid wall flexible diaphragm structures as a guide to predict this effect.

For this structure, the concrete walls are assumed to be rigid, therefore their in-plane shear displacement is assumed to be small and contribute little compared to the flexible diaphragm. Figure 5.23 shows the deflected shape of the roof diaphragm relative to the walls of the structure.

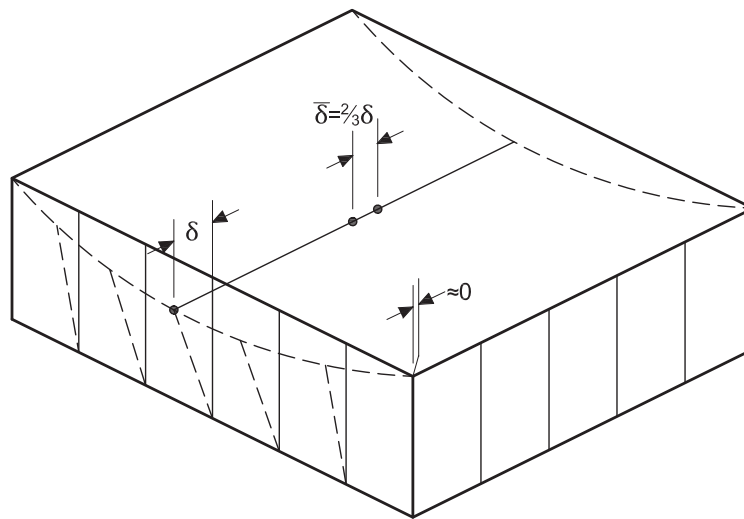


Figure 5.23: Deflection of Diaphragm Relative to the Rigid Walls

The P-delta provisions of the ASCE 7 Section 12.8.7 may be applied to this system as a guide to investigate structural stability as follows.

Stability coefficient equation:

$$\theta = P_x \Delta_e / (V_x h_{sx} C_d) \leq 0.10 \quad \text{ASCE 7 eq 12.8-16}$$

Overstrength coefficient:

$$C_d = 4.0 \quad \text{ASCE 7 Table 12.2-1 (A5)}$$

Roof level height (conservatively use low point on the roof)

$$h_{sx} = 32 \text{ ft (12 in/ft)} = 384 \text{ in}$$

Importance Factor:

$$I_e = 1.0 \quad \text{ASCE 7 Section 11.5.1}$$

The total unfactored vertical design load tributary to the roof Level:

$$P_{x \text{ roof}} = 15 \text{ psf}(504 \text{ ft})(300 \text{ ft})(1 \text{ kip}/1000 \text{ lb}) = 2268 \text{ kips}$$

$$P_{x\text{ wall}} = 116 \text{ psf}(32 \text{ ft}/2 + 5 \text{ ft})(504 \text{ ft})(2 \text{ walls}) = 2256 \text{ kips}$$

$$P_x = 2268 \text{ kips} + 2502 \text{ kips} = 4770 \text{ kips}$$

The unfactored seismic shear force acting at the roof level:

$$V_x = 596 \text{ kips} + 596 \text{ kips} = 1192 \text{ kips}$$

The average movement of the center of mass of the roof structure based on the estimated maximum strength level roof diaphragm deflection is estimated based on a parabolic deflected shape. For a parabolic curve, the center of mass is at 2/3 of the height, or in this case the in-plane deflection.

$$\Delta = (2/3)\delta_x = (2/3)(6.84 \text{ in}) = 4.56 \text{ in}$$

$$\theta = P_x \Delta l_e / (V_x h_{sx} C_d) = 4770 \text{ kip}(4.56 \text{ in})(1.0) / [1192 \text{ kips}(384 \text{ in})(4.0)] = 0.011 \leq 0.10$$

The predicted stability coefficient is much less than the 10% limit therefore no further investigation of the stability $P\Delta$ effects is required for this structure.

Comparison with Hybrid Panelized Wood Roof Structures

Hybrid panelized wood roof structures are a common roof diaphragm system for warehouse structures in the Western U.S. The 1987 plf (1390 plf ASD) required diaphragm shear, without skylight consideration, along the East and West walls is well within the maximum design strength for a blocked high shear diaphragm strength of 2232 plf (1395 plf ASD) for 15/32 Structural I sheathing.

The shear deflection, G_a , for blocked high shear 15/32" Structural I sheathing ranges from 24 kip/in to 32 kip/in, which is 5 times less stiff than a comparable Verco steel roof deck diaphragm using the PunchLok II system. This translates to a design level shear deflection of 6 inches for this structure compared to the 1.7 inches for the steel roof deck solution. For wood diaphragms, the American Wood Council Special Design Provisions for Wind & Seismic recommends that the shear deflection of the wood diaphragm be added to the flexural deflections and chord slip of the diaphragm.

$$\delta = \text{Flexure} + \text{Shear} + \text{Chord Slip}$$

$$\delta = 4.6 \text{ in} + 6.0 \text{ in} + 0 \text{ in} = 10.6 \text{ in}$$

When this is increased to the deflection for the design level event for wood, the deflection increases to over 42 inches compared to the steel roof deck of only 6.9 inches.

$$\delta_M = C_d \delta / l_e = 4 (10.6 \text{ in}) / 1.0 = 42.4 \text{ in}$$

Even with the large diaphragm deflection for the hybrid wood roof structure, it still meets the $P\Delta$ limit following ASCE 7 Section 12.8.7 limit with $\Phi = 0.07 < 0.10$ limit compared to the better performance of the steel deck diaphragm prediction of $\Phi = 0.01$.

$$\Delta = (2/3)\delta_M = (2/3)(42.4 \text{ in}) = 28.3 \text{ in}$$

$$\theta = P_x \Delta l_e / (V_x h_{sx} C_d) = 4770 \text{ kip}(28.3 \text{ in})(1.0) / [1192 \text{ kips}(384 \text{ in})(4.0)] = 0.07 \leq 0.10$$

5.8 East-West Diaphragm Deflection

The East–West diaphragm deflection will be much less than the North–South diaphragm deflection because the shear is lower at the walls, the width of the diaphragm is less, and the diaphragm is deeper than the North–South direction. The only reason to estimate the deflection in this direction is that the deflection is needed to estimate the prying action force when using the steel deck to resist the wall anchorage forces along Lines 1 and 10 as performed in Section 6.2.3. The method to determine the deflection is the same as that for the North–South direction. To simplify the analysis, the attachment zones based on the shear in the East–West direction are used. The additional stiffness from Zone 3 and Zone 4 at the East and West ends of the buildings is neglected. This will lead to a slightly conservative deflection.

| Zone | v_{left} (plf) | v_{right} (plf) | $v_{i \text{ ave}}$ (plf) | L_i (ft) | G' (kip/in) | $0.5(v_{i \text{ ave}} L_i)/(1000G')$ (in) |
|-----------------------------------------------------------|----------------------------|-----------------------------|------------------------------|---------------|------------------|-----------------------------------------------|
| II | 1035 | 704 | 869 | 48 | 117 | 0.18 |
| I | 704 | 0 | 352 | 102 | 96 | 0.19 |
| I | 0 | 704 | 352 | 102 | 96 | 0.19 |
| II | 704 | 1035 | 869 | 48 | 117 | 0.18 |
| Total Elastic Strength Level Shear Deflection, δ_v | | | | | | 0.74 |

Table 5.3 Steel Roof Deck In-Plane Shear Deflection

For projects where a more accurate deflection is needed, the deflection can be determined using a weighted average of the G' for each of the cantilever segments. This might be beneficial for projects with a seismic separation, or where reduced prying action is desirable for wall anchorage.

The inelastic deflection for the East–West direction is calculated the same way as the North–South direction. The C_d and I_e factors are the same for both directions in this building.

$$\delta_x = C_d \delta_{xe} / I_e$$

ASCE 7 eq 12.8-15

$$C_d = 4$$

$$I_e = 1.0$$

$$\delta_{xe} = \delta_s = 0.73 \text{ in}$$

$$\delta_x = 4.0(0.74 \text{ in})/1.0 = 2.96 \text{ in}$$

6.0 Seismic Wall Anchorage, Ties, and Sub-Diaphragms

The out-of-plane wall anchorage of concrete or masonry walls is an essential component of the design of a steel deck diaphragm roof structure. Failures of concrete or masonry shear walls have been rare in single story warehouse type structures except for the anchorage of the wall to the roof diaphragm. After the 1971 San Fernando earthquake, the building code addressed seismic wall anchorage, starting in 1973 with the inclusion of requirements for wall ties, cross ties and prohibition of cross grain bending for wood roof structures. Additional wall anchorage failures in the 1989 Loma Prieta, 1992 Landers, and 1994 Northridge earthquakes continued to plague warehouse type construction with wood diaphragms. As a result, wall anchorage forces have been increased to current force levels reflecting roof level accelerations that are 3 to 4 times the ground accelerations.

Steel elements of the wall anchorage systems for SDC C, D, E, and F are subject to a 1.4 increase in force in accordance with ASCE 7 Section 12.11.2.2.2. This material specific increase was introduced after the 1994 Northridge earthquake due to the poor performance of steel strap connector hardware used in wood panel diaphragm roof structures. Although there were no known failures of the wall anchorage of steel deck roof structures in the Northridge earthquake, all steel elements of flexible diaphragms are subject to this increase including the ledgers, joist seats, open web steel joists, and steel deck. These issues with steel connectors are well summarized in the “Implications of the January 17, 1994, Northridge Earthquake on Tilt-Up and Masonry Buildings with Wood Roofs” published in the 1994 SEAOC Convention Proceedings.

This section will develop wall anchorage force and the load path for the continuous ties in accordance with ASCE 7 Section 12.11.2.2. The continuous ties for the North and South wall anchorage forces are resisted by the open web steel joists. The East and West wall anchorage forces are resisted by a combination of the steel roof deck and OWSJG bracing the wall.

6.1 Wall Anchorage Force

The design of the wall anchorage is based on ASCE Equation 12.11-1 for wall anchorage force, $F_p = 0.4S_{DS}k_aI_eW_p$. For flexible diaphragms the amplification factor, k_a , varies from 1.0 to 2.0 for diaphragms 100 ft and longer. The effect is a wall anchorage force between 2 and 4 times that of the base shear for the building. The wall anchorage forces are determined as follows:

Wall Anchorage Forces:

$$S_{DS} = 1.0 \quad \text{see Section 4}$$

$$I_e = 1.0 \quad \text{see Section 4}$$

$$W_p = 116 \text{ psf} \quad \text{see Section 2}$$

$$k_a = 1.0 + (L_r/100) = 1.0 + (504 \text{ ft}/100 \text{ ft}) = 6.0 \geq 2.0 \text{ for E-W Wall} \quad \text{ASCE 7 eq 12.11-2}$$

$$k_a = 1.0 + (L_r/100) = 1.0 + (300 \text{ ft}/100 \text{ ft}) = 4.0 \geq 2.0 \text{ for N-S Wall} \quad \text{ASCE 7 eq 12.11-2}$$

Therefore, $k_a = 2.0$ for both N-S and E-W walls.

$$F_p = 0.4S_{DS}k_aI_eW_p = 0.4(1.0)(2.0)(1.0)(116 \text{ psf}) = 92.8 \text{ psf} \quad \text{ASCE 7 eq 12.11-1}$$

The wall anchorage forces shall not be taken less than:

$$F_{p \min} = 0.2k_aI_eW_p = 0.2(2.0)(1.0)(116 \text{ psf}) = 46.4 \text{ psf} \quad \text{ASCE 7 eq 12.11-1}$$

$$F_{p \min} = 5 \text{ psf} \quad \text{ASCE 7 §1.4.4}$$

$$F_{p \min} = 0.2W_p = 0.2(116 \text{ psf}) = 23.2 \text{ psf} \quad \text{ASCE 7 §1.4.4}$$

Therefore, $F_p = 92.8 \text{ psf}$

Evaluation of the wall anchorage force at the roof level for the wall's full height is shown in Figure 6.1.

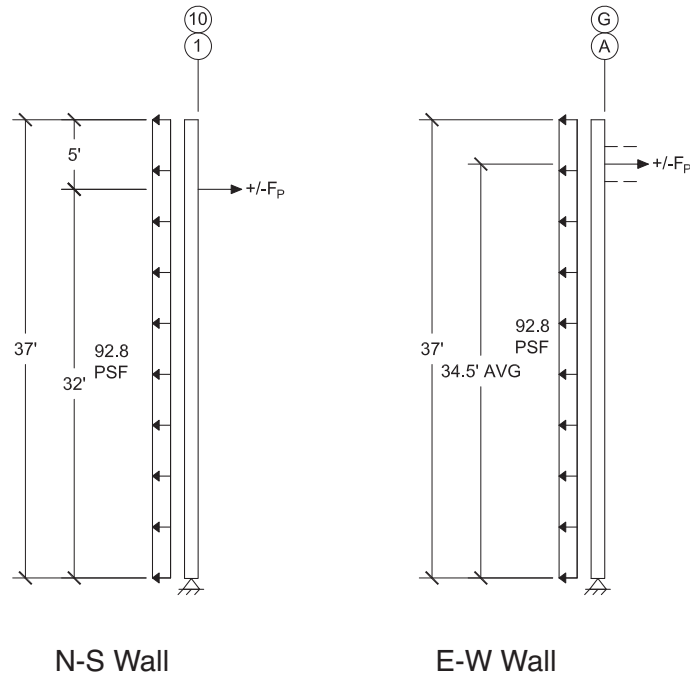


Figure 6.1 Wall Anchorage Forces

Sum of the moments about the floor level for N-S Wall:

$$F_p = 92.8 \text{ psf} (37 \text{ ft})(37 \text{ ft}/2)/32 \text{ ft} = 1985 \text{ plf}$$

The worst-case wall anchorage force is in the corners that have the maximum 37 ft parapet height and the lowest 32 ft roof height will be used for this example. For buildings that have significant changes in wall or roof height, this should be repeated for each condition to get an optimized design.

Note that ASCE 7 Section 13.4 for the attachment of non-structural concrete or masonry does not apply because the concrete or masonry shear walls are structural walls.

6.2 Steel Roof Deck Wall Anchorage Design and Continuous Ties

The East-West wall anchorage and continuous ties required by ASCE 7 Section 12.11.2.2.1 are provided by the cold-formed steel roof deck or a system using the deck for the wall anchorage in combination with a sub-diaphragm and open web steel joist girder ties across the diaphragm. For both options the deck acts as a beam column in the plane of the diaphragm resisting the wall anchorage forces.

The steel deck is not prohibited from resisting wall anchorage forces in the direction parallel with the ribs. This is often confused with the prohibition from acting as a continuous tie perpendicular to the ribs of the steel deck in ASCE 7 Section 12.11.2.2.4.

ASCE 7 12.11.2.2.4 Metal Deck Diaphragms. In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

This is common sense because the steel roof deck has virtually no tension or compression strength, similar to as an accordion, across the sheet of the steel deck. The steel roof deck has large tension and compression strength as a column in the direction parallel with the ribs that is analogous to a continuous series of hat channels resisting the axial forces.

The steel deck will also be subject to the 1.4 increase for steel elements of the wall anchorage system as required by ASCE 7 Section 12.11.2.2.2.

$$\pm 1.4F_p = \pm 1.4(1985 \text{ plf}) = \pm 2779 \text{ plf}$$

6.2.1 Out-of-Plane Wall Anchorage and Wall Ties with Steel Deck

The steel deck and connections of the steel roof deck to the ledger attached to the wall may be used to resist the out-of-plane wall anchorage force. The connections transfer the anchorage force from the ledger anchored to the wall into the steel deck parallel with the deck flutes. The steel deck acts as a compression or tension member between the steel joists resisting the wall anchorage force as shown in Figure 6.2. The steel deck must be checked as a beam column considering both the axial load from the wall anchorage force and the gravity load of the steel deck and roof system. Consideration of the eccentricity of the connection of the steel deck to the ledger relative to the neutral axis of the steel deck must also be addressed.

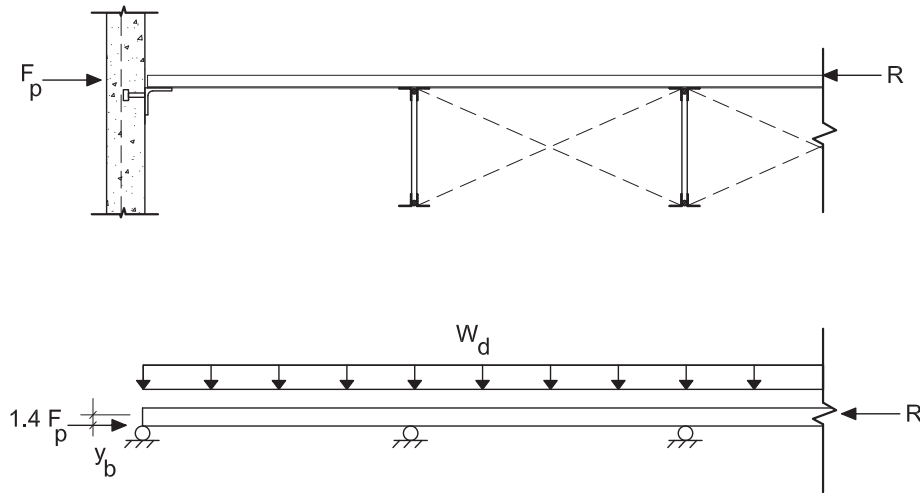
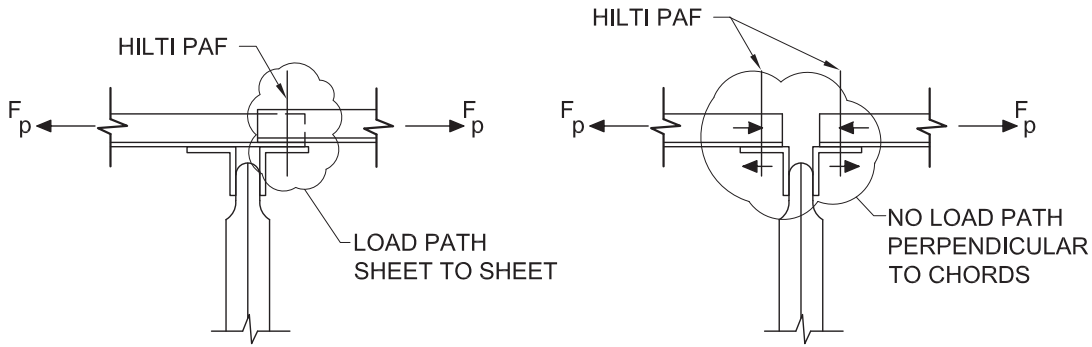


Figure 6.2 Axial Wall bracing with Steel Roof Deck

In addition to resisting the wall anchorage force at the wall, the steel deck may be used as the continuous tie across the diaphragm. The wall anchorage force must be transferred across the end laps of the deck sheets at every lap across the diaphragm as shown in Figure 6.3a. It is critical that the deck is lapped on one chord of the joist as shown to provide a direct load path from one sheet of deck to the other. When the steel deck is butted as shown in Figure 6.3b, the load path to transfer forces from one deck sheet to the next is disrupted. Butted deck should be avoided wherever possible. In locations where it is necessary to butt the steel roof deck, a tie plate or other solution must be designed to transfer the tie forces between the top chords of the OWSJ. The open web steel joists are not typically designed to transfer the axial load from one deck sheet to the other when they are attached to adjacent chords.



**Figure 6.3a
Deck Lap**

**Figure 6.3b
Butted Deck**

If conditions arise that require deck to be butted, then a field installed transfer plate and additional connection(s) must be made to transfer the axial force from sheet to sheet. This is not generally a cost-effective solution for the entire project but can be used as a remedy for an unanticipated butted steel deck condition that arises during erection of the structure.

The analysis of the steel deck to brace the walls using the steel deck connections considering combined axial and bending follows the provisions of AISI S100 for members subject to combined loading. The following Sections set up the required parameters to perform the analysis and take advantage of the Verco Steel Deck Axial and Bending web-based design tool.

6.2.2 Ledger Angle Eccentricity

The load is transferred from the concrete wall to the steel deck through the ledger angle. The ledger angle is attached to the concrete wall with either headed stud anchors or anchor bolts. The eccentricity in the load path from the headed stud or anchor bolt to the connection of the steel deck is shown in Figure 6.4. This eccentricity needs to be addressed in the design of the ledger angle. The design of the ledger angles exceeds the scope of this roof structure design guide. SEAOC Seismic Design Manual 2 Example 5 provides guidance through a similar example for an eccentrically loaded joist seat shelf angle detail.

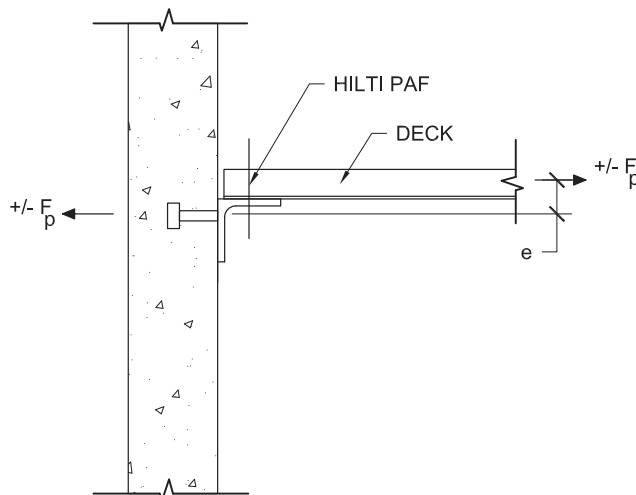


Figure 6.4 Steel Deck Wall Anchorage

6.2.3 Steel Deck Connection Prying Action of Steel Roof Deck Connections at Ledger

Prying action of the deck relative to the ledger angle needs to be considered in addition to the eccentricity of the anchor bolt or shear stud relative to the ledger angle deck connection. As the wall leans inward or outward in a seismic event the ledger rotates with the wall relative to the plane of the steel deck. Assuming that the ledger and the deck are relatively rigid compared to the local deformation around the connection leads to tension on the connection due to the prying effect of the displacement of the wall as shown in Figure 6.5. Neither the provisions of AISI S100 or the SDI DDM provide methods to predict the stiffness of a connection in tension for screws, welds, or PAFs. Testing of the connections in steel deck is required to determine the tension force on the connection due to prying action of the wall rotating inward or outward during a seismic event. Verco presents the results of testing following the AISI S905 test method in IAPMO ER-2018 for a variety of common steel deck connections.

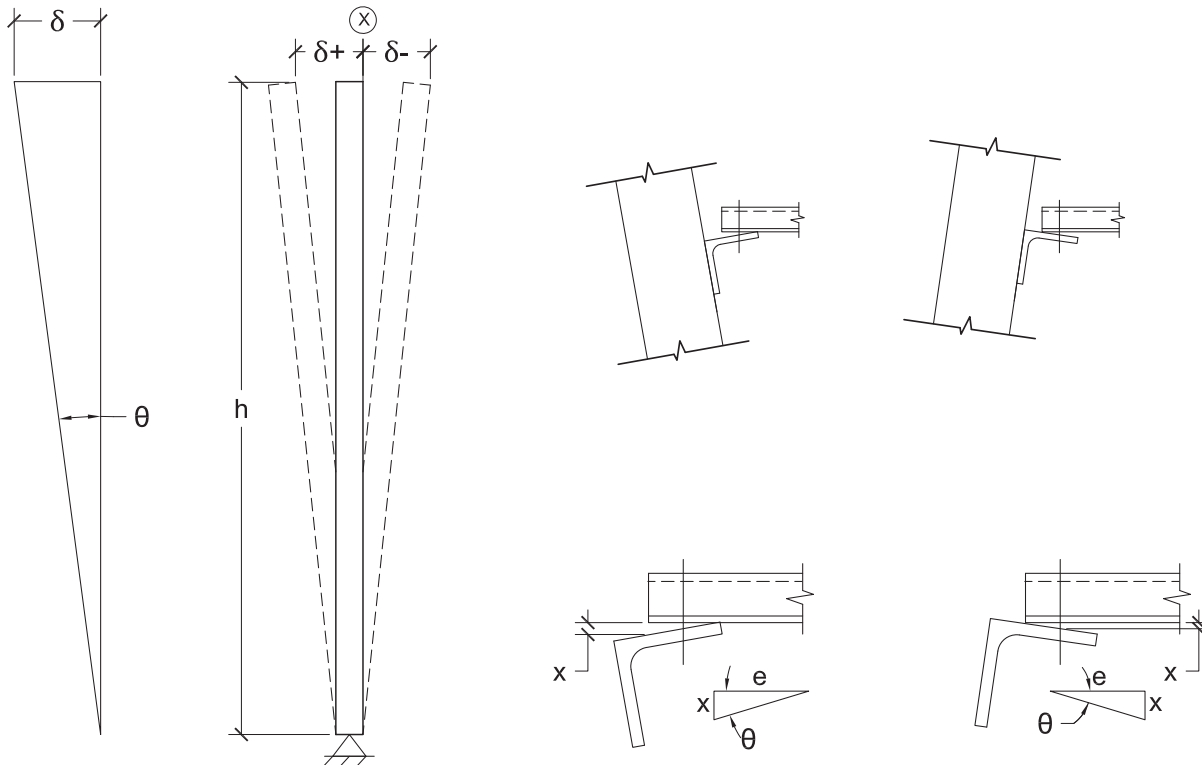


Figure 6.5 Wall Deflection

For this project:

- | | | |
|------------|------------------------------------------|-----------------|
| h_r | = 34 ft, average end wall roof height | see Section 2 |
| δ_x | = 2.9 in, amplified diaphragm deflection | see Section 5.8 |
| e | = 1.5 in, leverage length of connection | |

The rotation of the ledger angle relative to the roof structure is equal to the rotation of the wall relative to vertical.

$$\theta = \tan^{-1}((\delta x)/h) = \tan^{-1}(2.9 \text{ in} / (34 \text{ ft} (12 \text{ in/ft})) = 0.41^\circ$$

$$x = e \tan(\theta) = (1.5 \text{ in}) \tan(0.41^\circ) = 0.011 \text{ in}$$

The tension load resulting from the prying action is determined in accordance with IAPMO ER-2018 Equation T-1 as follows.

$$S_t = 1 / (1706 t^2 d_w + 2.51) \quad \text{IAPMO eq T-1}$$

$$S_t = 1 / (1706 (0.0359^2)(0.474) + 2.51) = 0.281 \text{ in/kips}$$

Where:

$t = 0.0359$ in, steel deck thickness for 20 ga PLB-36

$d_w = 0.474$ in, Hilti X-HSN 24 washer diameter

The load on the individual Hilti X-HSN 24 is a linear relationship within the design strength range of loading calculated as follows.

$$P_t = x / S_t = 0.011 / 0.281 = 0.0039 \text{ kips} = 39 \text{ lbs}$$

This tension load due to prying action at the ledger is very small and will have little influence on the connection design. The load will be carried through the calculations to demonstrate this.

6.2.4 Steel Deck Block Shear Rupture at Sheet End Connections

The wall anchorage forces are transferred to the steel deck through the connections to the supporting ledger at the wall and from sheet to sheet in the field of the diaphragm. The minimum edge distance in AISI S100 Section J5.1 for power actuated fasteners of 1/2 inch does not negate the need to check the steel deck panel for block shear rupture requirements of AISI S100 Section J6 between the connection and end of the sheet. The minimum edge distance does not ensure that block shear rupture will not govern the connection, as shown in Figure 6.6, thus reducing the strength of the connection for the PAF.

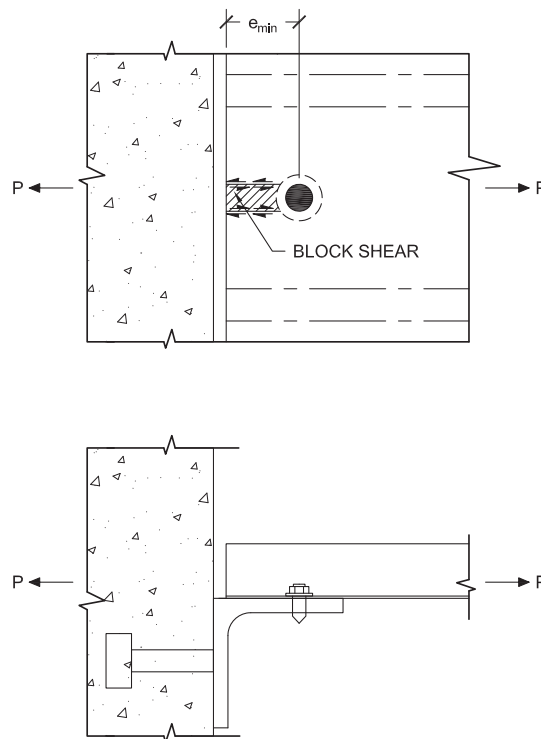


Figure 6.6 Block Shear Rupture at Steel Deck Sheet End

AISI S100 provides provisions to design connections based on testing in Section K. To help create the most efficient connections, Verco undertook an end shear rupture test program for individual connections in steel

deck. The result of this program is the support connection shear rupture strength equation, R-1, in IAPMO ER-2018. This equation provides a more optimized solution, specific to Verco steel deck profiles, than the general provisions in AISI Section J6.

The minimum end distance calculated based on shear rupture and the connection strength of the Hilti X-HSN 24 is determined as follows.

$$P_{nv} = 2 F_u t (e_{min} - d/2) \quad \text{IAPMO eq R-1}$$

$$e_{min} = 1.8 / (2 (65)(0.0359)) + 0.157 / 2 = 0.464 \text{ in}$$

Where:

$$F_u = 65 \text{ ksi, steel deck ultimate strength}$$

$$t = 0.0359 \text{ in, steel deck thickness}$$

$$d = 0.157 \text{ in, diameter of X-HSN 24 fastener}$$

$$P_{nv} \leq P_{nf} = 52 t (1 - t) = 52 (0.0364)(1-0.0329) = 1.8 \text{ kip}$$

The minimum end distance for rupture is 0.464 inches which is less than the minimum 0.5 in for PAF connections therefore 0.5 inch governs.

6.2.5 Design of Steel Roof Deck for Wall Anchorage and Ties Forces

The design of steel deck to resist the wall anchorage forces and act as the continuous ties across the diaphragm may eliminate the cost of installing costly steel blocking between the OWSJ to distribute those forces into sub-diaphragms. The basis for the combined axial and bending strength of steel deck comes directly from the provisions of AISI S100 Section H Members Under Combined Forces. This section addresses both the applicable limit states for steel deck, combined axial and bending, and combined bending and shear. These provisions for strength combined with the engineering mechanics for a multi-span beam-column are the basis for the solution.

The rigorous nature of this analysis of the steel deck and connections subject to combined axial and bending would deter most from tackling this problem without the assistance of a software-based solution. To eliminate this burden, this design example takes advantage of the Verco web-based Axial and Bending Design Tool to perform the analysis. The following summarizes the required inputs for the web-based design tool. Complete detailed calculations may be generated using the Steel Deck Axial and Bending web-based design tool.

The first check of the steel deck to resist the wall anchorage forces will be for the highest shear Zone IV along the East or West walls. The following inputs for the web-based diaphragm tool are appropriate for deck Zone IV. These inputs include all relevant seismic loading and gravity loads. The wind load inputs shown in the summary output in Figure 6.7 are developed in Section 8 of this example. The wind loading for this project is minimal and does not govern the design, as shown in Section 8. The wind loads can be omitted when using the design tool without any impact on the seismic wall anchorage design.

Deck Zone IV Inputs for the Axial and Bending Design Tool:

Vertical Loads:

The minimum dead load is applied without re-roof, because the dead load acts to offset the moment due to the eccentric load from connections at the wall.

5 psf Superimposed Dead Load

20 psf Roof Live Load

Horizontal Loads:

1985 plf, Wall anchorage force F_{px} at 1.0E level. The web-based tool applies the 1.4 increase for steel elements of flexible diaphragms.

Seismic Factors:

1.0 S_{ds} to amplify dead loads for vertical seismic

Concrete or Masonry Walls

C, D, E or F Seismic Design Category

Steel Deck Selection:

PLB-36 Roof Deck, Grade 50

20 gage at the walls for anchorage

8' – 0" foot span

End lapped deck

Steel Roof Deck Attachment at Wall:

Hilti X-HSN 24 PAF

0.31 in substrate for the 5/16 inch bent plate or hot rolled angle chord steel

36/7 Attachment pattern at deck sheet ends

0.05 in Prying action for seismic

Steel Roof Deck Attachment in the Field of the Diaphragm:

Hilti X-HSN 24 PAF

End lapped steel deck

0.125 in substrate

36/7 Attachment pattern at deck sheet ends

36/4 Attachment pattern at deck sheet intermediate supports

The combined Axial and Bending Design Tool checks the selected deck for the prescribed stress combinations in AISI S100. The primary check is for both combined axial tension or compression with combined bending and web shear. It also checks the connections for shear and tension, including combined shear and tension interaction at supports. Web crippling is also checked at supports for inward loading. This web-tool does not check service load deflection of the steel deck, or load combinations that do not consider axial loads but which may govern if there are large gravity or vertical inward or outward wind loads.

Wall Anchorage & Continuous Tie Design - Summary

20 gage PLB™-36 Grade 50 with 8' - 0" Deck Span



Project: Roof Structure Design Guide
Description:

Job No. Roof Structure Design
Engineer: Roof Structure Design

Connections to Support (Ledger/Chord) at WALL

Hilti X-HSN24 PAF
36 / 7 Perpendicular Connection Pattern
0.528 in Steel Deck Minimum End Distance

A572 GR50 Support Member or Equivalent
0.313 in \leq Support Thickness \leq 0.375 in
0.50 in Support Member Minimum Edge Distance
2.00 in Steel Deck Bearing on Support at Wall

0.01in Connection Prying Action for Seismic at Wall
0.00 in Connection Prying Action for Wind at Wall

Connections in FIELD of Diaphragm

Hilti X-HSN24 PAF
36 / 7 / 4 Perpendicular Connection Pattern
0.528 in Steel Deck Minimum End Distance

A529 GR50 Support Member or Equivalent
0.125 in \leq Support Thickness \leq 0.375 in
0.50 in Support Member Minimum Edge Distance
5.00 in Steel Deck Bearing on Supports in Field of Diaphragm
2.00 in Steel Deck Sheet End Bearing on Support in the Field
1.06 in Minimum Steel Deck End Lap Length

Summary of Calculations in accordance with ASCE 7-16, AISI S100-16, and ER-2018

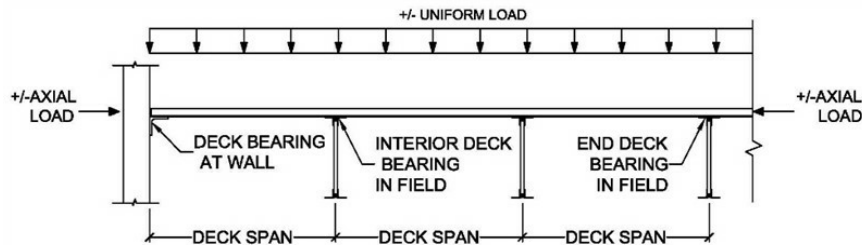
| | | | | | |
|--------|------------------------------------|------|-------------------------|------|--------------------------|
| 1-Span | Steel Deck Axial and Bending Check | 0.71 | ≤ 1.0 Pass | | |
| | Connection Strength Check | 0.99 | ≤ 1.0 Pass at wall | 0.95 | ≤ 1.0 Pass in field |
| | Web Crippling Check | 0.08 | ≤ 1.0 Pass at wall | 0.08 | ≤ 1.0 Pass in field |
| 2-Span | Steel Deck Axial and Bending Check | 0.60 | ≤ 1.0 Pass | | |
| | Connection Strength Check | 1.00 | ≤ 1.0 Pass at wall | 0.97 | ≤ 1.0 Pass in field |
| | Web Crippling Check | 0.06 | ≤ 1.0 Pass at wall | 0.12 | ≤ 1.0 Pass in field |
| 3-Span | Steel Deck Axial and Bending Check | 0.62 | ≤ 1.0 Pass | | |
| | Connection Strength Check | 0.28 | ≤ 1.0 Pass at wall | 0.96 | ≤ 1.0 Pass in field |
| | Web Crippling Check | 0.07 | ≤ 1.0 Pass at wall | 0.10 | ≤ 1.0 Pass in field |

Vertical Loads

| | |
|----------------------------------------------|------|
| Dead Load for Inward Wind Combo, D_i (psf) | 7.3 |
| Dead Load for Wind Uplift Combo, D_o (psf) | 4.3 |
| Live Load, L (psf) | 0.0 |
| Roof Live Load, L_r (psf) | 20.0 |
| Rain Load, R (psf) | 0.0 |
| Snow Load, S (psf) | 0.0 |
| Inward Wind, W_i (psf) | 0.0 |
| Wind Uplift on Steel Deck, W_o (psf) | 25.3 |
| Wind Uplift for Connections, W_o (psf) | 25.3 |

Horizontal Loads and Factors

| | |
|-----------------------------------------------------------------------------------------|---------------------|
| Wall Inward Wind Anchorage Load, W_i (plf) | 423 |
| Wall Outward Wind Anchorage Load, W_o (plf) | 342 |
| Notional Load for Wall Anchorage, $\pm N$ (plf) | 124 |
| Seismic Wall Anchorage Load, $\pm F$ (plf) | 1985 |
| Seismic Parameter, SDS | 1.00 |
| Seismic Design Category | C, D, E or |
| Wall Construction | Concrete or Masonry |
| Steel Element of Structural Wall Anchorage System Increase (ASCE 7 Section 12.11.2.2.2) | 1.4 |



Axial and Bending Web Tool Version 1.0

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Figure 6.7 Axial and Bending Web Tool Summary Page for 20 gage PLB-36 with a 36/7/4 Pattern

Figure 6.7 shows the summary page of the combined Axial and Bending strength tool including all the inputs and the stress ratios of the resulting calculations. From the selected inputs, the result for the 20 gage PLB-36 steel deck with the Hilti X-HSN 24 connections in a 36/7/4 pattern, is that the deck has adequate combined axial and bending strength to resist the loads in the field of the diaphragm, and Hilti X-HSN 24 connections in a 36/7 pattern are adequate to connect the steel deck to the wall. This is evident in the design tool output because the steel deck combined axial and bending ratios are less than 1.0 for 1, 2, and 3 span conditions, and the demand-to-capacities ratios for the connection strength and web crippling are also less than 1.0, indicating that the connections are adequate.

With the Zone IV deck verified to be adequate, the next step is to determine if the lighter, 22 gage PLB-36 deck in Zones I, II, and III is adequate to transfer the wall anchorage forces and act as continuous ties across the roof structure. For this, the web-based design tool will be used again with the following inputs.

Uniform Vertical Loading:

5 psf Superimposed Dead Load

20 psf Roof Live Load

Uniform Horizontal Load:

1,985 plf, Wall anchorage force F_{px} at 1.0E level. The web-based tool applies the 1.4 increase for steel elements of flexible diaphragms.

Seismic Parameters:

1.0 S_{ds} to amplify dead loads for vertical seismic

Concrete or Masonry Walls

C, D, E or F Seismic Design Category

Steel Deck Selection:

PLB-36 Roof Deck Grade 50

22 gage at the walls for anchorage

8' – 0" foot span

End lapped deck

Steel Deck Attachment to the Ledger/Chord at the Wall:

n/a

Steel Deck Attachment in the Field of the Diaphragm:

Hilti X-HSN 24 PAF

End lapped steel deck

0.125 in substrate

36/7 Attachment pattern at deck sheet ends

36/4 Attachment pattern at deck sheet intermediate supports

Wall Anchorage & Continuous Tie Design - Summary

22 gage HSB®-36 Grade 50 with 8' - 0" Deck Span



Project: Roof Structure Design Guide
Description:

Job No. Roof Structure Design
Engineer: Roof Structure Design

Connections to Support (Ledger/Chord) at WALL

Hilti X-HSN24 PAF
36 / 7 Perpendicular Connection Pattern
0.531 in Steel Deck Minimum End Distance

A572 GR50 Support Member or Equivalent
0.125 in ≤ Support Thickness ≤ 0.375 in
0.50 in Support Member Minimum Edge Distance
2.00 in Steel Deck Bearing on Support at Wall

0.00in Connection Prying Action for Seismic at Wall
0.00 in Connection Prying Action for Wind at Wall

Connections in FIELD of Diaphragm

Hilti X-HSN24 PAF
36 / 7 / 4 Perpendicular Connection Pattern
0.531 in Steel Deck Minimum End Distance

A529 GR50 Support Member or Equivalent
0.125 in ≤ Support Thickness ≤ 0.375 in
0.50 in Support Member Minimum Edge Distance
5.00 in Steel Deck Bearing on Supports in Field of Diaphragm
2.00 in Steel Deck Sheet End Bearing on Support in the Field
1.06 in Minimum Steel Deck End Lap Length

Summary of Calculations in accordance with ASCE 7-16, AISI S100-16, and ER-2018

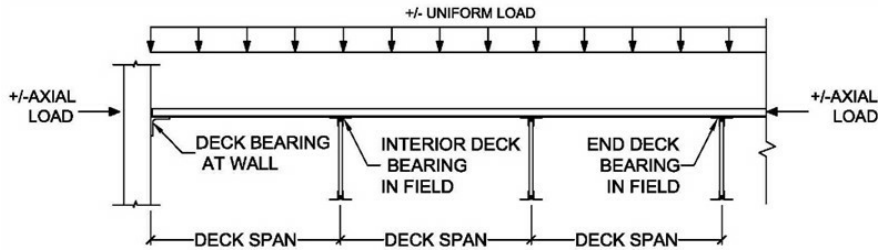
| | | | | | |
|--------|------------------------------------|------|-------------------|------|--------------------|
| 1-Span | Steel Deck Axial and Bending Check | 0.87 | ≤1.0 Pass | | |
| | Connection Strength Check | 1.13 | >1.0 FAIL at wall | 1.13 | >1.0 FAIL in field |
| | Web Crippling Check | 0.11 | ≤1.0 Pass at wall | 0.11 | ≤1.0 Pass in field |
| 2-Span | Steel Deck Axial and Bending Check | 0.73 | ≤1.0 Pass | | |
| | Connection Strength Check | 1.15 | >1.0 FAIL at wall | 1.16 | >1.0 FAIL in field |
| | Web Crippling Check | 0.09 | ≤1.0 Pass at wall | 0.16 | ≤1.0 Pass in field |
| 3-Span | Steel Deck Axial and Bending Check | 0.75 | ≤1.0 Pass | | |
| | Connection Strength Check | 0.33 | ≤1.0 Pass at wall | 1.15 | >1.0 FAIL in feild |
| | Web Crippling Check | 0.09 | ≤1.0 Pass at wall | 0.14 | ≤1.0 Pass in field |

Vertical Loads

| | |
|----------------------------------------------|------|
| Dead Load for Inward Wind Combo, D_i (psf) | 6.9 |
| Dead Load for Wind Uplift Combo, D_o (psf) | 3.9 |
| Live Load, L (psf) | 0.0 |
| Roof Live Load, L_r (psf) | 20.0 |
| Rain Load, R (psf) | 0.0 |
| Snow Load, S (psf) | 0.0 |
| Inward Wind, W_i (psf) | 0.0 |
| Wind Uplift on Steel Deck, W_o (psf) | 25.3 |
| Wind Uplift for Connections, W_o (psf) | 25.3 |

Horizontal Loads and Factors

| | |
|-----------------------------------------------------------------------------------------|---------------------|
| Wall Inward Wind Anchorage Load, W_i (plf) | 423 |
| Wall Outward Wind Anchorage Load, W_o (plf) | 342 |
| Notional Load for Wall Anchorage, $\pm N$ (plf) | 124 |
| Seismic Wall Anchorage Load, $\pm F$ (plf) | 1985 |
| Seismic Parameter, SDS | 1.00 |
| Seismic Design Category | C, D, E or |
| Wall Construction | Concrete or Masonry |
| Steel Element of Structural Wall Anchorage System Increase (ASCE 7 Section 12.11.2.2.2) | 1.4 |



Axial and Bending Web Tool Version 1.0

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Figure 6.8 Axial and Bending Web Tool Summary Page for 22 gage PLB-36 with a 36/7/4 Pattern



Figure 6.8 shows the summary page of the combined axial and bending strength tool, including all the inputs and the stress ratios of the resulting calculations for the 22 gage PLB-36 steel deck. From the selected inputs, the result for the 22 gage PLB-36 steel deck with the Hilti X-HSN 24 connections in a 36/7/4 pattern is that the steel deck is adequate, but the 36/7/4 connection pattern is overstressed, which is indicated by the connection demand-to-strength ratio greater than 1.0. Therefore, the deck attachment patterns for Zones I, II, and III are not adequate to be used as the continuous wall ties across the roof structure.

There are potential options to create the wall ties across the diaphragm in deck Zones I, II, and III. The steel deck gage can be increased from 22 to 20 gage, for it was already shown in deck Zone IV that this combination of deck gage and attachment pattern is adequate to carry the wall tie forces across the roof structure. A second option could be to keep the 22 gage deck since it works in axial and bending, and increase the attachment pattern at the sheet ends. Using the web-based design tool it can be shown that a 36/9/4 pattern is adequate to transfer the continuous wall ties forces across the diaphragm. For the first and second option, the design professional needs to consider the implications skylights and mechanical equipment openings would have on the deck as the continuous tie. The third option is to use the steel deck in Zone IV to create a series of sub-diaphragms that collect and transfer the wall anchorage forces into the OWSJG, that become axial forces in the OWSJG. In this method the OWSJG become the continuous ties across the diaphragm, utilizing steel deck sub-diaphragms at each end of the building as the transfer method to the end walls.

The two most practical methods would be to increase the 22 gage deck to 20 gage, which is a relatively modest cost increase, or to use the traditional sub-diaphragm method with axial load in the OWSJG top chords. All three options would solve the wall anchorage and continuous tie requirements, which essentially leaves this decision to be a relative cost of construction decision, requiring input from the contractor. For this design example, we will pursue the sub-diaphragm approach to demonstrate this method which is often the most economical when the wall anchorage forces are larger than those in this example.

6.2.6 Wall Anchorage and Continuous Ties Using Sub-Diaphragm Method

In the sub-diaphragm method, the wall anchorage forces are collected by the sub-diaphragm and distributed into the top chord of the OWSJG, acting as an axial load on the OWSJG, as shown in Figure 6.9. The open web steel joist girders then serve as the continuous ties across the diaphragm to satisfy the requirements of ASCE 7 Section 12.11.2.2.1 rather than the steel deck.

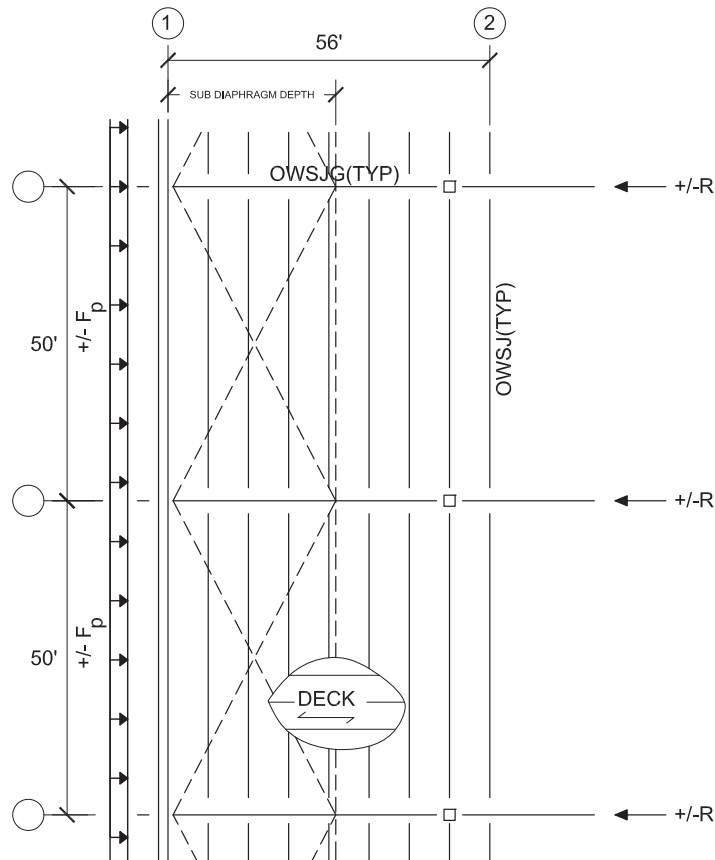


Figure 6.9 Sub-diaphragm and OWSJG collector

Sub-diaphragm reaction:

$$R = \pm 1.4F_p l/2 = \pm 1.4(1985 \text{ plf})(50 \text{ ft})(1 \text{ kip}/1000 \text{ lb})/2 = \pm 69.5 \text{ kips}$$

The width of the sub-diaphragm is determined by the in-plane shear capacity of the steel roof deck in the sub-diaphragm. The width must also satisfy the maximum length-to-width ratio of 2.5 to 1 required by ASCE 7 Section 12.11.2.2.1. This ratio is intended to provide deformation compatibility between in-plane deflection of the sub-diaphragm and the main diaphragm system.

$$w = R/\phi S_n = 69.5 \text{ kips}(1000 \text{ lb}/\text{kip})/2206 \text{ plf} = 31.5 \text{ ft}$$

$$2.5 \text{ to } 1 \text{ ratio: } w = 50 \text{ ft}/2.5 = 20 \text{ ft}$$

Use sub-diaphragm width of 4 joist spaces at 8 ft = 32 ft > 31.5

The sub-diaphragm boundary chords will be the open web steel joists used for the roof structure that are spaced at 8 ft. The sub-diaphragm width used will be 32 ft to accommodate the open web steel joist spacing.

The sub-diaphragm reactions must be transferred from the steel deck to the open web steel joist girder. Figure 6.10 shows the use of blocking on the OWSJG installed between the OWSJ seats to transfer the reaction of the sub-diaphragm from the steel roof deck to the OWSJG. The blocking is commonly specified as cold formed steel channels, bent plates or tube steel. In this example, a HSS 5x5x1/4 is selected that matches the 5 in deep long span OWSJ seat. Flare bevel welds will be used to transfer the load from the tube steel to the OWSJG, as shown in Figure 6.10. The design of the welded connections to transfer the shear to the OWSJG is not covered in this example.

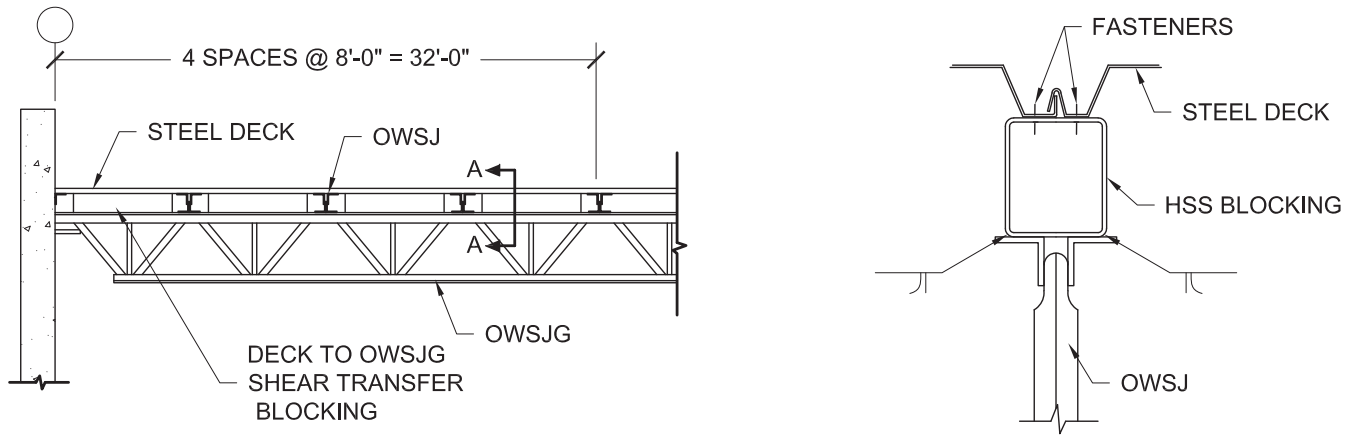


Figure 6.10 OWSJG Blocking

The steel deck parallel attachment to the HSS blocking is determined based on the reaction force and the strength of the Hilti X-HSN 24 fasteners. The number of required fasteners is determined as follows.

Connection strength for individual fasteners:

$$\Phi P_n = \Phi P_{nf} = \Phi 52t(1 - t) \leq \Phi P_{nvp} \quad \text{ER-2018 eq H-2}$$

$$\Phi P_n = \Phi P_{nf} = (0.70)52(0.0359)(1 - 0.0359) \leq 0.60(3.020)$$

$$\Phi P_n = 1.260 \text{ kip} \leq 1.812$$

$$\Phi P_n = 1260 \text{ lb per fastener}$$

$$N = R/(\Phi P_n) = 69.5 \text{ kips}(1000 \text{ lb/kip})/(1260 \text{ lb/fastener}) = 55 \text{ fasteners}$$

The fasteners are distributed equally to the OWSJG blocking over the 32 ft sub-diaphragm width. The 4 blocking segments fit between the OWSJ and will be assumed to be 7 ft long to allow for erection clearance for the OWSJ seats. The fastener spacing for each sub-diaphragm is then:

$$S = 7 \text{ ft } (12 \text{ in/ft}) / ((55 \text{ fasteners}/4 \text{ blocking})) = 6.1 \text{ in o.c.}, \text{ therefore use } 6 \text{ in o.c.}$$

The OWSJG act as the continuous ties that collect the tributary forces from both adjacent sub-diaphragms, therefore a line of fasteners is required for each sub diaphragm. When the steel deck has a side seam on the blocking, the fasteners should be distributed as shown in Figure 6.11a. To keep the design simple, the same 2 rows of fasteners will be called out when the steel deck panels have a low flute at the blocking as shown in Figure 6.11b.

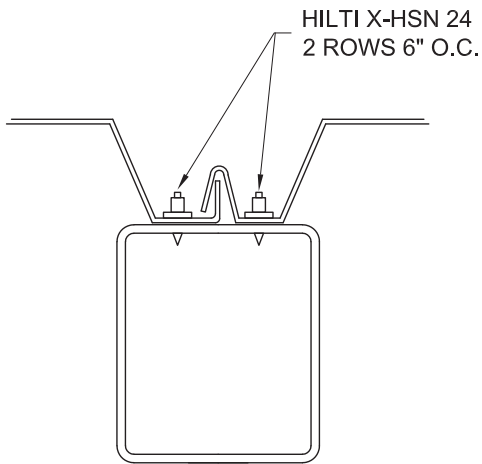


Figure 6.11a Shear Transfer at Sidelap

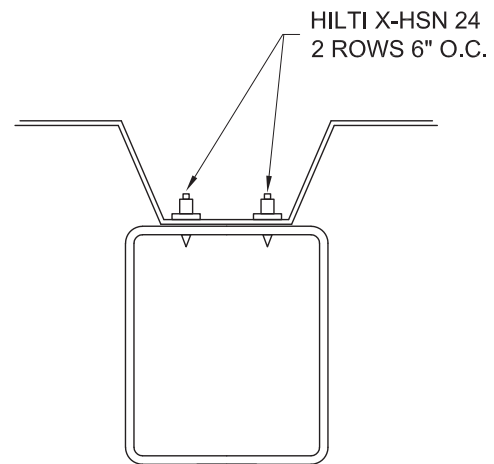


Figure 6.11b Shear Transfer at Low Flute

The open web steel joist girder top chord axial force is the result of both tributary sub-diaphragms. The axial force is the sum of the tributary force from the sub-diaphragm on each side of the OWSJG. In this example, the conservative approach of using the highest wall anchorage force from the corners is applied across the entire diaphragm.

The resulting continuous tie force including the 1.4 increase for steel elements.

$$P = R_{\text{left}} + R_{\text{right}} = 69.5 \text{ kips} + 69.5 \text{ kips} = 139 \text{ kips}$$

The axial forces transferred from the sub-diaphragm to the OWSJG must be carried across the width of the diaphragm. The design of the OWSJG connections at the columns must transfer the wall anchorage continuous tie force across the connection. A common method to accomplish this is with an axial tie plate placed vertically between the girder chords as shown in Figure 6.12, typically referred to as a knife plate. Using the Vulcraft Knife Plate Connection web-based design tool, an Grade 36, 7/8 in thick, 6 in wide, by 27 in long knife plate with 12 inches of 5/16 in fillet weld to each of the top chord angles is appropriate to resist the 139 kip axial load with a design strength of 157.0 kips, as presented in Figure 6.13.

<https://www.vulcraft.com/design-tools>

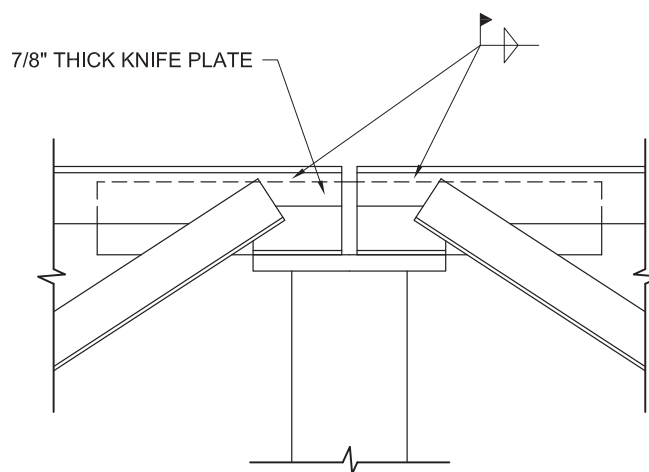


Figure 6.12 OWSJG Knife Plate

Knife Plate Connection - Top Chord

AISC 360-16 Ch. D, E, & J and SJI Specifications

Typical Joist Girder Knife Plate Connection



Knife Plate Info:

| | | |
|----------------------------|-------|-----|
| Knife Plate Thickness | 0.875 | in |
| Knife Plate Width | 6.00 | in |
| Knife Plate Overall Length | 27.00 | in |
| Knife Plate F_y (min) | 36 | ksi |
| Fillet Weld Size: t_w | 5/16 | |
| Fillet Weld Length: L_w | 12.00 | in |
| Weld Electrodes: F_{exx} | 70 | ksi |

Joist Girder Info:

| | | |
|--------------------------------|-------|-----|
| Joist Girder F_y (min) | 50 | ksi |
| Min. Top Chord Thickness | 0.310 | in |
| Min. Top Chord Vertical Leg | 4 | in |
| Max. Gap between Joist Girders | 1.50 | in |
| Top Chord Angle space | 1.0 | in |

Loading:

| | | |
|--------------------------|-------|------|
| Connection Demand: T_u | 139.0 | kips |
|--------------------------|-------|------|

Calculations:

I. Knife Plate:

| | |
|--------------------------------------------------|-----------------------|
| a) Tension: Yield: | $\Phi = 0.9$ |
| $P_n = (F_y)(\text{Width})(\text{Thickness}) =$ | 189.0 kips |
| $\Phi P_n =$ | <u>170.1 kips</u> |
| b) Tension: Rupture: | $\Phi = 0.75$ |
| Shear Lag Factor: U | $\bar{x} = 0.5$ in |
| AISC Table D3.1 Case 4: $U =$ | 0.688 |
| $A_e = AnU = (\text{width})(\text{thick.})(U) =$ | 3.609 in ² |
| $F_u =$ | 58 ksi |
| $P_n = F_u A_e =$ | 209.3 kips |
| $\Phi P_n =$ | <u>157.0 kips</u> |

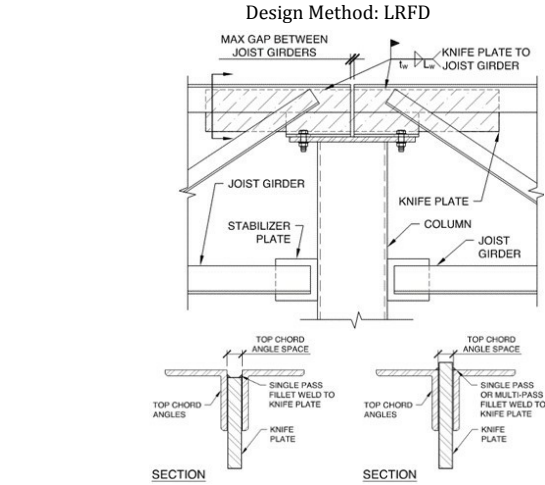
II. Weld:

| | |
|---------------------|-------------------|
| $\Phi =$ | 0.75 |
| Total Weld Length = | 24.0 in |
| $R_n =$ | 222.7 kips |
| $\Phi R_n =$ | <u>167.1 kips</u> |

III. Joist Girder:

| | |
|------------------------------------------|------------------------------|
| a) Shear: | Shear Rupture: $\Phi = 0.75$ |
| Yield: $\Phi R_n = \Phi 0.6 F_y A_1 =$ | 223.2 kips |
| Rupture: $\Phi R_n = \Phi 0.6 F_u A_1 =$ | 217.6 kips |
| $\Phi R_n =$ | <u>217.6 kips</u> |
| b) Tension: Rupture: | $\Phi = 0.75$ |
| $P_n = F_u A_e =$ | 219.8 kips |
| $\Phi P_n =$ | <u>164.8 kips</u> |

Plate Controls Connection Capacity



| | |
|------------------------------------------------------------------------------------|-------------------|
| c) Compression: | $\Phi = 0.9$ |
| radius of gyration $r = (1/\sqrt{12})$ (thickness) = | 0.2526 in |
| $L =$ Overall Length - 2(Weld Length) = | 3.00 in |
| $L_c = (1.0)(L) =$ | 3.00 in |
| $L_c/r =$ | 11.9 ≤ 25 |
| for $L_c/r \leq 25$: $P_n = (F_y)(\text{Width})(\text{Thickness})$ AISC Sec. J4.4 | 189.0 kips |
| $\Phi P_n =$ | <u>170.1 kips</u> |

$R_n = (0.6)(F_{exx})(\sqrt{2}/2)(t_w)(\text{total weld length})$

A_1 is shear area of vertical leg along length of weld

| | |
|--------------------------------------------------------------|-----------------------|
| Shear Yield: $\Phi = 1.00$ | $F_u = 65$ ksi |
| $A_1 = (2 \text{ angles})(0.31 \text{ in})(12 \text{ in}) =$ | 7.44 in ² |
| Shear Lag Factor: U | $l = 6.0$ in |
| $\bar{x} =$ | 1.115 in |
| AISC Table D3.1 Case 4: $U =$ | 0.709 |
| $An =$ | 4.768 in ² |
| $A_e = AnU =$ | 3.381 in ² |

Joist Manufacturer determines final Top Chord Angles Sizes

LRFD Connection Capacity:

Knife Plate 6in x 0.875in x 27in long with 5/16 fillet weld 12in long each side of plate to each Joist Girder

Joist Girder Requirements:

Min. Top Chord Thickness = 0.31 in
 Min. Top Chord Vertical Leg = 4 in

Capacity: $\Phi R_n = 157.0$ kips $\geq T_u$ O.K.

Calculations based on v1.0, Released 12/2020

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Figure 6.13 Vulcraft Knife Plate Connection

6.3 North-South Wall Anchorage Design and Continuous Ties

The open web steel joists (OWSJ) resist the North-South wall anchorage loads and act as continuous ties required by ASCE 7 Section 12.11.2.2.1. The anchorage force is transferred from the wall into each OWSJ. The weld from the joist seat to joist bearing angle or the embed channel ledger is sized to transfer this load. For large anchorage loads, it may be necessary to use tie plates or angles to transfer some of the load directly to the top chord of the joist from the embedded ledger. If each joist is used as a continuous tie, then the anchorage force is transferred from OWSJ to OWSJ to the diaphragm chord on the opposite side.

The OWSJ are spaced at 8 feet on center. The steel elements of the wall anchorage system are required to be designed for a 1.4 increase in strength level wall anchorage force in accordance with ASCE 7 section 12.11.2.2.2. The OWSJ top chord strength level axial wall anchorage force is determined using the wall anchorage force developed in Section 6.1.

$$1.4(1985 \text{ plf})(8 \text{ ft/joist}) = 22.2 \text{ kips/joist}$$

This axial load is applied to the design of the OWSJ and to the joist seat attached to the concrete wall as shown in Figure 6.14. It should be noted on the structural plans for Vulcraft to design the joist seat to transfer the axial load to ensure the seat is properly designed for the load transfer.

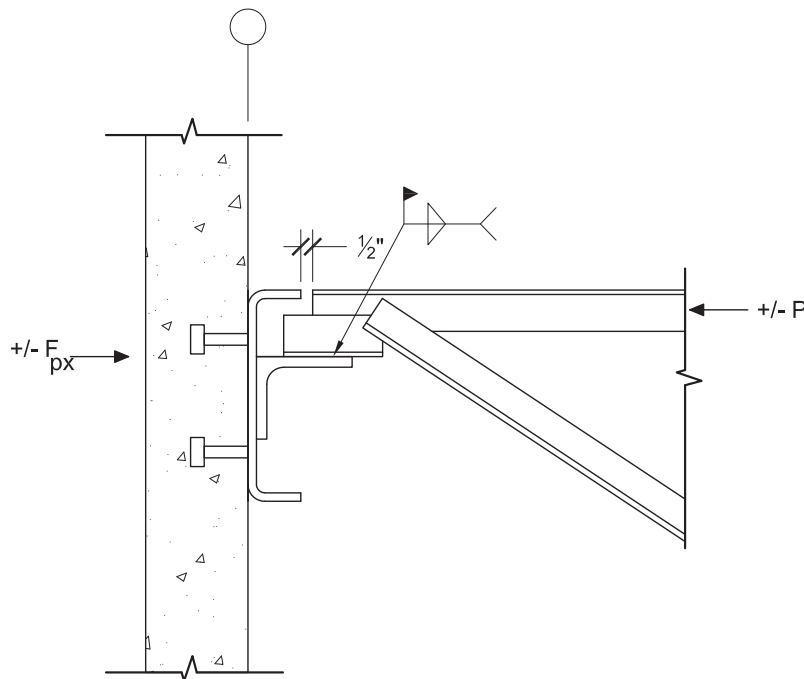


Figure 6.14 Joist to Wall Anchorage Force and Continuous Tie

The complete design of the joist bearing angle to the embedded channel and anchorage to the concrete wall is beyond the scope of this deck and joist example. The 2018 IBC edition of the SEAOC Seismic Design Manual 2, Example 5, provides a detailed design example for the concrete panel anchorage to the OWSJ seat embedded shelf angle support.

Each OWSJ will develop a continuous tie across the diaphragm. The top chords of every joist must be tied together to transfer the axial load. Steel tie plates welded to the top of the open web steel joists have been commonly used. This causes an issue with the Hilti X-HSN 24 fasteners because the increased steel thickness of the tie plate in combination with the OWSJ top chord often exceeds the 3/8" maximum support steel thickness. An improved option involves using a pair of rectangular bars under the horizontal leg of the angles, welded to the toe of the joist top chord angles. If the lap between the plate and the horizontal leg of the

top chord angle is minimized to a recommended 1/2 inch maximum lap, most of the top chord is available to install the Hilti X-HSN 24 fastener as shown in Figure 6.15. Figure 6.15 shows this detail using a pair of 1/4 in x 2 in x 10 in bars with 4 inches of 1/8 inch fillet weld to each joist chord. The tie bars in this example have been designed using the Vulcraft Joist Tie Plate web-based design tool as summarized in Figure 6.16.

<https://www.vulcraft.com/design-tools>

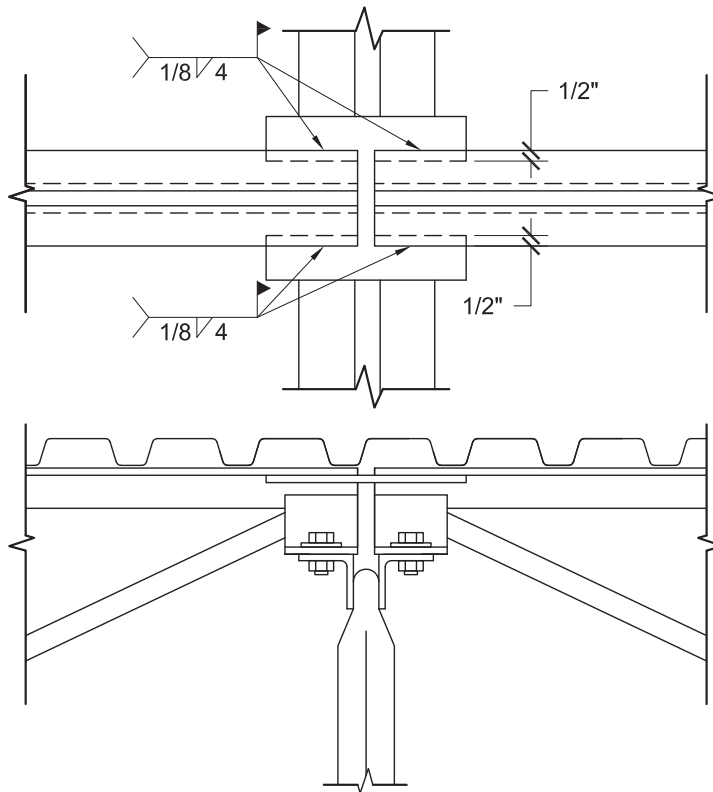


Figure 6.15 Joist-to-Joist Axial Wall Tie Force Transfer

Tie Plate Connection - Underside

AISC 360-16 Ch. D, E, & J and SJI Specifications
 Typical Joist Tie Plate Connection Under Horiz. Leg



Tie Plate Info (Both Plates):

| | | |
|----------------------------|-------|-----|
| Tie Plate Thickness | 0.25 | in |
| Tie Plate Width | 2.00 | in |
| Tie Plate Overall Length | 10.00 | in |
| Tie Plate F_y (min) | 36 | ksi |
| Fillet Weld Size: t_w | 1/8 | |
| Fillet Weld Length: L_w | 4.00 | in |
| Weld Electrodes: F_{exx} | 70 | ksi |

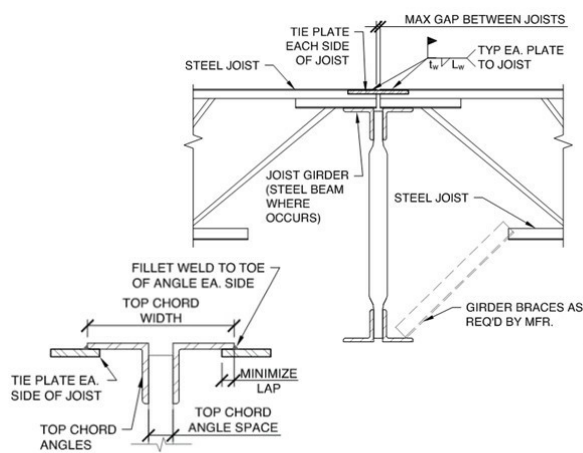
Joist Info:

| | | |
|-------------------------------|-------|-----|
| Joist F_y (min) | 50 | ksi |
| Min. Top Chord Thickness | 0.125 | in |
| Min. Top Chord Horizontal Leg | 2 | in |
| Max. Gap between joists | 1.00 | in |
| Top Chord Angle space | 1.0 | in |

Loading:

| | | |
|--------------------------|------|------|
| Connection Demand: T_u | 22.2 | kips |
|--------------------------|------|------|

Design Method: LRFD



Calculations:

I. Tie Plate:

| | | |
|-----------------------------------------------------|-------------------|-----------------|
| a) Tension: Yield: | $\Phi = 0.9$ | |
| $P_n = (F_y)(2)(\text{Width})(\text{Thick.}) =$ | 36.0 | kips |
| $\Phi P_n =$ | <u>32.4</u> | kips |
| b) Tension: Rupture: | $\Phi = 0.75$ | |
| Shear Lag Factor: U | $\bar{x} = 0.125$ | in |
| AISC Table D3.1 Case 4: $U =$ | 0.703 | |
| $A_e = AnU = (2)(\text{width})(\text{thick.})(U) =$ | 0.703 | in ² |
| $F_u =$ | 58 | ksi |
| $P_n = F_u A_e =$ | 40.8 | kips |
| $\Phi P_n =$ | <u>30.6</u> | kips |

| | | |
|-------------------------------------------------------|--------------|------|
| c) Compression: | $\Phi = 0.9$ | |
| radius of gyration $r = (1/\sqrt{12})$ (thickness) = | 0.0722 | in |
| $L = \text{Overall Length} - 2(\text{Weld Length}) =$ | 2.00 | in |
| $L_c = (1.0)(L) =$ | 2.00 | in |
| $L_c/r =$ | 27.7 | > 25 |
| AISC Sec. E3 Compression | | |
| Width/Thickness = | 8.00 | |
| $F_e =$ | 372.7 | ksi |
| AISC Eq. E3-2: | 34.6 | ksi |
| $P_n = (F_{cr})(2)(\text{Width})(\text{Thick.}) =$ | 34.6 | kips |
| $\Phi P_n =$ | <u>31.1</u> | kips |

II. Weld:

| | | |
|--------------------------------|-------------|------|
| $\Phi =$ | 0.75 | |
| Total Weld Length each Joist = | 8.0 | in |
| $R_n =$ | 29.7 | kips |
| $\Phi R_n =$ | <u>22.3</u> | kips |

$$R_n = (0.6)(F_{exx})(\sqrt{2}/2)(t_w)(\text{total weld length})$$

III. Joist:

| | | |
|------------------------------------------|------------------------------|------|
| a) Shear: | Shear Rupture: $\Phi = 0.75$ | |
| Yield: $\Phi R_n = \Phi 0.6 F_y A_1 =$ | 30.0 | kips |
| Rupture: $\Phi R_n = \Phi 0.6 F_u A_1 =$ | 29.3 | kips |
| $\Phi R_n =$ | <u>29.3</u> | kips |

A_1 is shear area of horizontal leg along length of weld

| | | |
|--------------------------------------------------------------|------------|-----------------|
| Shear Yield: $\Phi = 1.00$ | $F_u = 65$ | ksi |
| $A_1 = (2 \text{ angles})(0.125 \text{ in})(4 \text{ in}) =$ | 1 | in ² |

| | | |
|----------------------|---------------|------|
| b) Tension: Rupture: | $\Phi = 0.75$ | |
| $P_n = F_u A_e =$ | 35.7 | kips |
| $\Phi P_n =$ | <u>26.8</u> | kips |

| | | |
|-------------------------------|-------------------|-----------------|
| Shear Lag Factor: U | $\bar{x} = 0.546$ | in |
| AISC Table D3.1 Case 4: $U =$ | 0.568 | |
| $An =$ | 0.969 | in ² |
| $A_e = AnU =$ | 0.550 | in ² |

Joist Manufacturer determines final Top Chord Angles Sizes

LRFD Connection Capacity:

Tie Plate 2in x 0.25in x 10in long Each Side of Joist with 1/8 fillet weld 4in long each plate to each joist

Joist Requirements:

Min. Top Chord Thickness = 0.125 in
 Min. Top Chord Horizontal Leg = 2 in

Capacity: $\Phi R_n = 22.3$ kips $\geq T_u$ O.K.

Weld Controls Connection Capacity

Calculations based on v1.1, Released 7/2020

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Figure 6.16 Vulcraft Joist Tie Plate Web-Tool Summary

7.0 Wind Diaphragm Design

The wind forces act on the main diaphragm, idealized as a deep beam spanning between the end shear walls, to resist the applied wind loads in the same manner that the diaphragm resists the seismic loads presented in Section 5. The main wind force resisting system loads are applied to the walls in each orthogonal direction separately. The resulting wind pressure on the walls is resisted by the diaphragm at the ledger height and the foundation at the floor height of the building as shown in Figure 7.4, and applied as a horizontal force to the diaphragm as shown in Figures 7.3a and 7.3b.

The steel deck diaphragm design to resist wind loads requires that the combination of in-plane diaphragm shear and out-of-plane wind uplift force be considered. This is different than the seismic diaphragm design that only considers the in-plane shear induced by the seismic load. Both the diaphragm shear and the wind uplift apply loads to the connections of the steel roof deck sheets to the supporting OWSJ. The addition of the tension load on the fasteners reduces the remaining strength to resist diaphragm shear loads. This connection interaction check must be considered when designing steel roof deck diaphragms for combined shear and wind uplift.

This wind diaphragm section will address the wind check for this structure in a high seismic region to ensure the diaphragm is adequate to resist the wind loads. The wind design is for a relatively low wind speed, however the principles demonstrated are applicable to high wind regions as well.

7.1 Main Wind Force Resisting System Loads

The main wind force resisting system (MWFRS) loads are used for the combined shear and uplift diaphragm design. This is appropriate because the diaphragm is being loaded by more than one surface of the structure and is a primary structural system. The Envelope Procedures of ASCE 7 Chapter 28 Part 1, low rise buildings, will be used to determine the MWFRS loads. The MWFRS loads are applied to the building as shown in Figure 28.3-1 of ASCE 7. For this example structure with a flat roof, the wind loads on the walls drive the roof diaphragm shear in combination with the wind uplift force on the roof structure. Figures 7.1a and 7.1b illustrate the wind loads on the walls and roof for Load Cases A and B taken from ASCE 7.

The wind parameters and Steps 1 thru 5 are the same for MWFRS loads as those for C&C loads developed in Section 4 of this example.

Step 6: External pressure coefficients, GC_{pf} , are given in ASCE 7 Figure 28.3-1 for low-rise buildings. Loads in the North-South direction are based on Load Case A, since the ridge of the building runs East-West. Loads in the East-West direction are based on Load Case B, since it is the direction parallel to the ridge. The roof slope is less than 5 degrees, so the coefficients for 0-5 degrees will be used for Load Case A. In the North-South direction, wall loads will be based on surfaces 1E, 1, 4E, and 4. For the East-West direction, the wall loads will be based on surfaces 5E, 5, 6E, and 6.

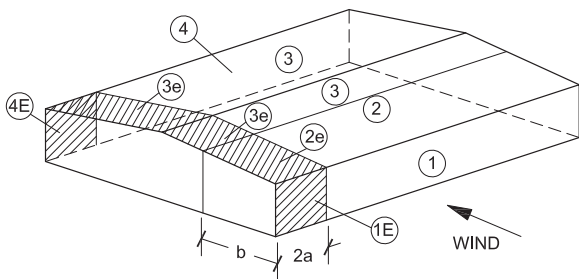


Figure 7.1a Load Case A Surfaces

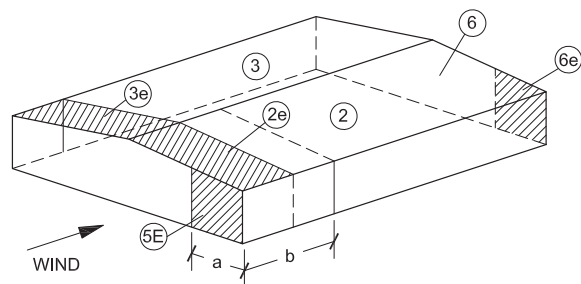


Figure 7.1b Load Case B Surfaces

GC_{pf} for Load Case A (N-S) for roof angles of 0 to 5 degrees:

ASCE 7 Figure 28.3-1

Note: + = toward the surface (inward), - = away from surface (outward)

| Surface | 1 | 2 | 3 | 4 | 1E | 2E | 3E | 4E |
|-----------|------|-------|-------|-------|------|-------|-------|-------|
| | Wall | Roof | Roof | Wall | Wall | Roof | Roof | Wall |
| GC_{pf} | 0.40 | -0.69 | -0.37 | -0.29 | 0.61 | -1.07 | -0.53 | -0.43 |

Table 7.1 N-S MWFRS External Pressure Coefficients, GC_{pf}

GC_{pf} for Load Case B (E-W) for roof angles of 0 to 90 degrees:

ASCE 7 Figure 28.3-1

| Surface | 2 | 3 | 5 | 6 | 2E | 3E | 5E | 6E |
|-----------|-------|-------|------|-------|-------|-------|------|-------|
| | Roof | Roof | Wall | Wall | Roof | Roof | Wall | Wall |
| GC_{pf} | -0.69 | -0.37 | 0.40 | -0.29 | -1.07 | -0.53 | 0.61 | -0.43 |

Table 7.2 E-W MWFRS External Pressure Coefficients, GC_{pf}

The widths of the wind surfaces for the MWFRS are developed in Section 4.3.3 and summarized as follows:

$$a = 15 \text{ ft}$$

$$2a = 30 \text{ ft}$$

The distances to the transition points at which relief is provided to use the lower wind pressure in Zone 2 and 3 were developed in Section 4.3.3.

$$b_{e-w} = 102.5 \text{ ft}$$

$$b_{n-s} = 102.5 \text{ ft}$$

Step 7: Design wind load is in accordance with ASCE 7 Section 28.3-1. The external pressure for the surfaces is applied in combination with the internal pressure, GC_{pi} . For the walls, the outward internal pressure on the opposite walls cancel each other out, therefore only the external pressures need to be considered for the determination of the lateral force on the diaphragm due to the wind load on the walls.

$$p = q_h[(GC_{pe}) - (GC_{pi})] \quad \text{ASCE 7 eq 28.3-1}$$

N-S Wall Surface 4:

$$p_4 = 20.3[(-0.29) - (0)] = -5.9 \text{ psf (outward, no } GC_{pi})$$

N-S Roof Surface 2:

$$p_2 = 20.3[(-0.69) - (0.18)] = -17.6 \text{ psf (outward)}$$

The wind loads determined for all surfaces are summarized in Tables 7.3 and 7.4.

Load Case A (N-S):

| Surface | 1 | 2 | 3 | 4 | 1E | 2E | 3E | 4E |
|--------------------------------|------|-------|-------|------|------|-------|-------|------|
| | Wall | Roof | Roof | Wall | Wall | Roof | Roof | Wall |
| p (psf) no GC _{pi} | 8.1 | | | -5.9 | 12.4 | | | -8.7 |
| p (psf) | | -17.6 | -11.2 | | | -25.4 | -14.4 | |

Table 7.3 N-S MWFRS External Pressures

Load Case B (E-W):

| Surface | 2 | 3 | 5 | 6 | 2E | 3E | 5E | 6E |
|--------------------------------|-------|-------|------|------|-------|-------|------|------|
| | Roof | Roof | Wall | Wall | Roof | Roof | Wall | Wall |
| p (psf) no GC _{pi} | | | 8.1 | -5.9 | | | 12.4 | -8.7 |
| p (psf) | -17.6 | -11.2 | | | -25.4 | -14.4 | | |

Table 7.4 E-W MWFRS External Pressures

The main wind force resisting system loads developed for the walls do not apply to parapets. Parapets have higher wind loading. The provisions of ASCE 7 Section 28.3.2 are used to determine the MWFRS loads applied to the windward and leeward parapets as shown in Figure 7.2, with wind pressure based on the height of the parapet

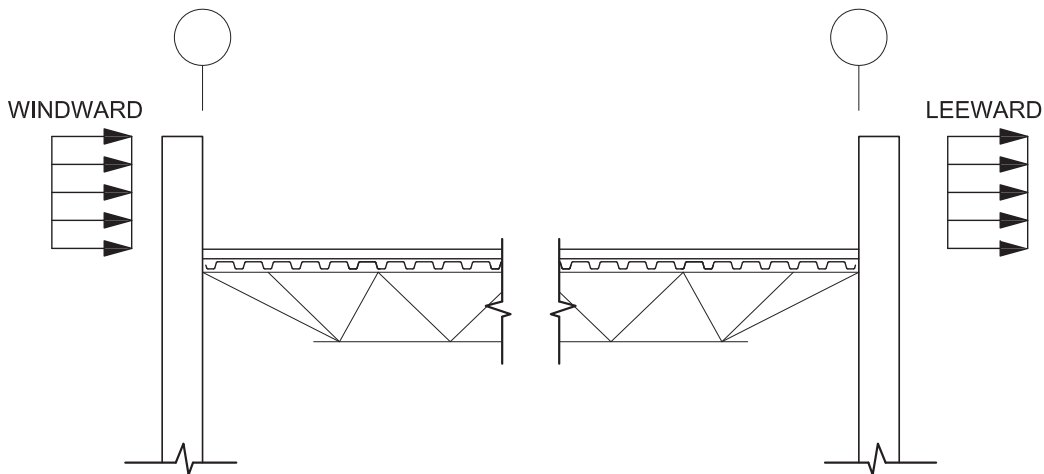


Figure 7.2 MWFRS Loads on Parapets

$$p_p = q_p GC_{pn}$$

ASCE 7 eq 28.3.2

$$h = 41 \text{ ft, top of parapet}$$

$$K_h = K_z = 2.01(z/z_g)^{(2/a)} = 2.01(41/900)^{(2/9.5)} = 1.05$$

ASCE 7 Table 26.10-1

$$q_p = 0.00256 K_z K_{zt} K_d K_E V^2 = 0.00256(1.05)(1.0)(0.85)(1.0)(95)^2 = 20.6 \text{ psf}$$

ASCE 7 eq 26.10-1

$$GC_{pn} = 1.5 \text{ for the windward parapet}$$

ASCE Section 28.3.2

$$GC_{pn} = 1.0 \text{ for the leeward parapet}$$

ASCE Section 28.3.2

North-South Direction:

$$p_{pw} = 20.6(1.5) = 30.9 \text{ psf, for the windward parapet}$$

$$p_{pl} = 20.6(1.0) = 20.6 \text{ psf, for the leeward parapet}$$

East-West Direction:

$$p_{pw} = 20.6(1.5) = 30.9 \text{ psf, for the windward parapet}$$

$$p_{pl} = 20.6(1.0) = 20.6 \text{ psf, for the leeward parapet}$$

Table 7.5 summarizes the wind pressure on the parapets.

| Parapet | North-South | | East-West | |
|-----------------|-------------|----------|-----------|----------|
| | p_{pw} | p_{pl} | p_{pw} | p_{pl} |
| p_{net} (psf) | 30.9 | 20.6 | 30.9 | 20.6 |

Table 7.5 Parapet Wind Loads

The minimum wind load for the MWFRS is specified in ASCE 7 Section 28.3.4. The minimum roof wind load for enclosed or partially enclosed buildings for the design of the MWFRS shall not be less than 16 psf for walls and 8 psf for roofs projected on a vertical plane in the assumed wind direction as shown in ASCE 7 Figure C27.1-1. This minimum wind load is to be applied as a separate load combination, not as a governing minimum pressure for each zone, as discussed in ASCE 7 Section C28.3.4.

Minimum wind loads:

$$\text{Wall} = +16 \text{ psf}$$

$$\text{Roof Uplift} = \tan(1.5^\circ)(8 \text{ psf}) = +0.2 \text{ psf}$$

7.2 Load Combinations

Load and resistance factor design is a good solution for the design of a steel deck diaphragm to resist combined in-plane diaphragm shear and out-of-plane wind uplift. The wind provisions in ASCE 7 determine the ultimate wind speed, therefore the load factor for LRFD is 1.0. For steel deck design, the governing load combination needs to be determined in ASCE 7 Section 2.3.1 Basic Combinations for LRFD. The load combinations for this roof structure include dead, roof live, and wind load. Load combinations 3 and 4 would govern for inward wind combined with dead load because of the 120% increase in dead load. To determine net wind uplift, load combination 5 would govern with outward wind reduced by 90% of the dead load.

$$3. \quad 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$$

$$4. \quad 1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$$

$$5. \quad 0.9D + 1.0W \text{ (governs for uplift)}$$

For wind forces on the exterior walls, there is no live load, snow load, rain load, or dead load, therefore the wall load combinations reduce to 1.0W. For the roof structure in this example, there is no snow load or rain load requirements, therefore the load combinations for the wind forces on the roof structure reduce to the following:

For inward wind load:

$$3. = 1.2D + 0.5W + 1.6L_r$$

$$4. = 1.2D + 1.0W + 0.5L_r$$

For wind uplift:

$$5. 0.9D + 1.0W$$

For horizontal wind loads on the roof diaphragm, the load combinations reduce to 1.0W.

These load combinations will be applied in the following Sections based on the applied loading of the structure.

Net wind uplift to be used in combination with the wind force on the diaphragm considering the minimum dead load supported by the steel deck developed in Section 4.1.1 for the Zones 2, 2E, 3, and 3E for wind loads in the North-South direction.

$$5. 0.9D + 1.0W = 0.9 (5.0 \text{ psf}) + 1.0(-17.6 \text{ psf}) = -13.1 \text{ psf for Zone 2}$$

$$5. 0.9D + 1.0W = 0.9 (5.0 \text{ psf}) + 1.0(-25.4 \text{ psf}) = -20.9 \text{ psf for Zone 2E}$$

$$5. 0.9D + 1.0W = 0.9 (5.0 \text{ psf}) + 1.0(-11.2 \text{ psf}) = -6.7 \text{ psf for Zone 3}$$

$$5. 0.9D + 1.0W = 0.9 (5.0 \text{ psf}) + 1.0(-14.4 \text{ psf}) = -9.9 \text{ psf for Zone 3E}$$

Table 7.6 summarizes the net uplift pressure on the steel roof deck to be combined with diaphragm shear.

| Surface | 2 | 3 | 2E | 3E |
|-----------------|-------|------|-------|------|
| p_{net} (psf) | -13.1 | -6.7 | -20.9 | -9.9 |

Table 7.6 N-S Net Wind Uplift

7.3 Diaphragm Shear Wind Loading

The wind pressures developed in Section 7.1 for MWFRS are applied to the walls to develop the in-plane wind load on the roof diaphragm system. For low-rise buildings using the envelope method, Figures 7.3a and 7.3b depicts the application of the wind surfaces from ASCE 7 Figure 28.3-1 to the roof diaphragm in plan view.

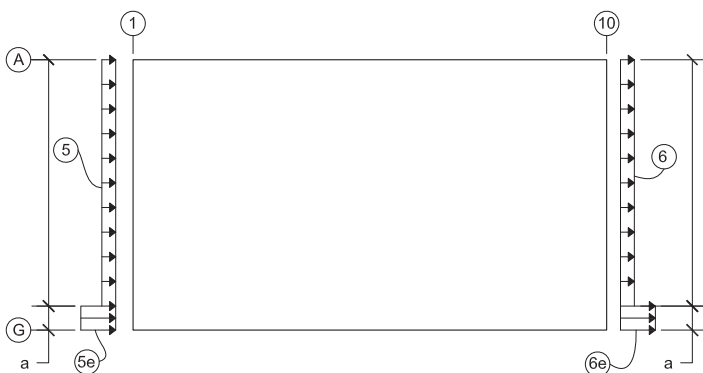


Figure 7.3a Load Case A

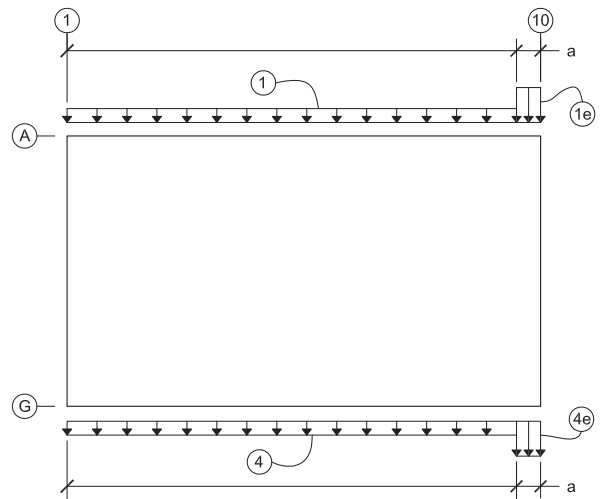


Figure 7.3b Load Case B

The windward and leeward walls act as a beam spanning from the base of the building to the roof diaphragm structure height. For the warehouse structure in this example, the wall is idealized as a simply supported beam spanning from the floor level to the roof diaphragm level of the building. The wind load on the roof can then be easily determined by summing the moments about the floor level and dividing by the height from the floor to the roof diaphragm level.

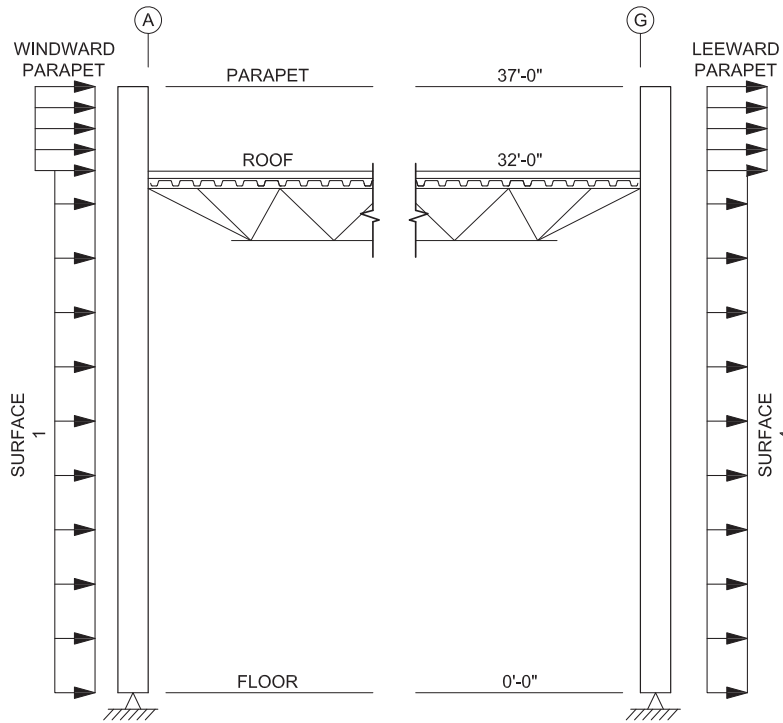


Figure 7.4 MWFRS Surfaces 1 and 4 on North-South Wall Loading

The wind loads developed in Section 7.1 are applied to the walls and summarized as follows.

Total Load Case A (N-S) without GC_{pi} :

The net wind pressure on the walls or parapets in the North-South direction for Surfaces 1 and 4, Load Case A, are summarized in Table 7.5:

$$w_w = 8.1 \text{ psf} + 5.9 \text{ psf} = 14.0 \text{ psf (walls)}$$

$$w_p = 30.9 \text{ psf} + 20.6 \text{ psf} = 51.5 \text{ psf (parapets)}$$

Total pressures for Case A (N-S) without GC_{pi} are summarized in Table 7.7:

| Surface | 1+4 | 1E+4E | Parapet |
|--------------------------------------------|------|-------|---------|
| p (psf) no GC_{pi} | 14.0 | 21.1 | 51.5 |

Table 7.7: N-S MWFRS External Pressures

Total pressures for Load Case B (E-W) without GC_{pi} are summarized in Table 7.8:

| Surface | 5+6 | 5E+6E | Parapet |
|--------------------------------------------|------|-------|---------|
| p (psf) no GC_{pi} | 14.0 | 21.1 | 51.5 |

Table 7.8 E-W MWFRS External Pressures

The wind load on the diaphragm for the main wind force resting system is the sum of the moments about floor level divided by the height from the floor to the roof diaphragm.

$$W = [p_w h_w (h_w/2) + p_p (h_p)(h_w + h_p/2)]/h_w$$

Where:

W = horizontal resultant wind force on roof diaphragm

h_w = height of wall or average height of wall from floor to roof diaphragm height

h_p = height or average height of parapet above roof diaphragm height

p_w = total horizontal pressure on walls

p_p = total horizontal pressure on parapets

For the N-S direction:

$$W_{1+4} = [14.0(32)(32/2) + 51.5(5)(32+5/2)]/32 = 501 \text{ plf}$$

$$W_{1E+4E} = [21.1(32)(32/2) + 51.5(5)(32+5/2)]/32 = 615 \text{ plf}$$

$$W_{\min} = [16.0(37.0)(37/2)]/32 = 342 \text{ plf, does not govern}$$

For the E-W direction:

$$W_{5+6} = [14.0(34)(34/2) + 51.5(3)(34+3/2)]/34 = 399 \text{ plf}$$

$$W_{5E+6E} = [21.1(34)(34/2) + 51.5(3)(34+3/2)]/34 = 520 \text{ plf}$$

$$W_{\min} = [16.0(37.0)(37/2)]/34 = 322 \text{ plf, does not govern}$$

The wind from the walls are resisted by the roof diaphragm structure. Figure 7.5 shows the MWFRS loads applied to the roof in the North-South direction. The diaphragm design will consider each orthogonal direction separately.

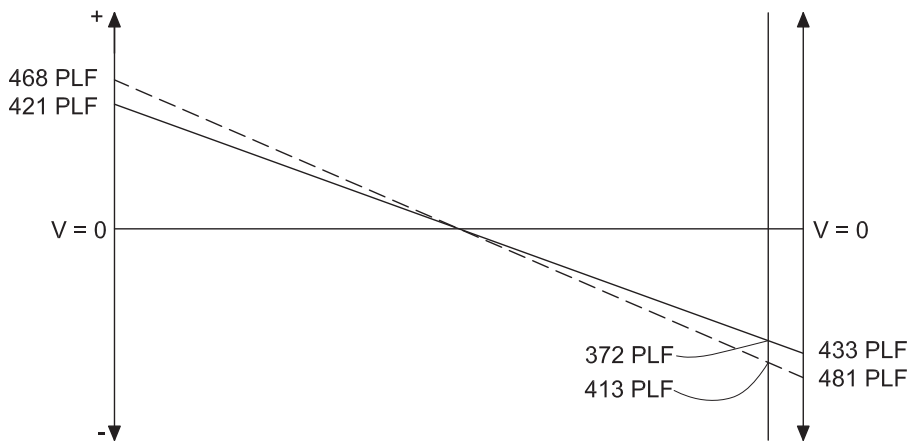
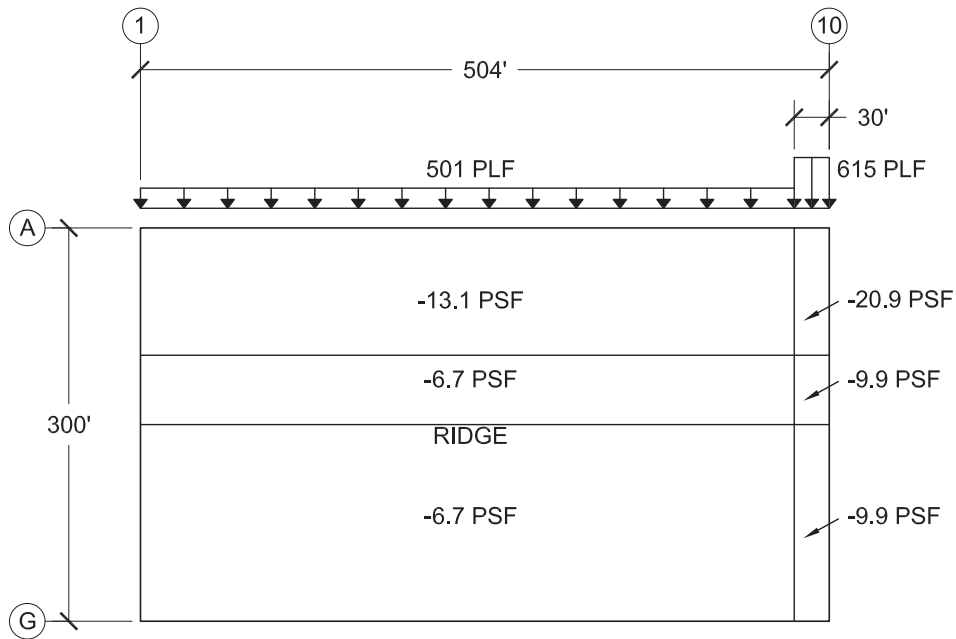


Figure 7.5 North-South Wind Diaphragm Shear and Uplift Load

The maximum diaphragm shear in the North-South direction is determined. This is accomplished by summing the moments about Line 10 to determine the highest diaphragm shear in the North-South direction on Line 10 as shown in Figure 7.5. The other condition for Load Case A is when Zones 1E and 4E are adjacent to Line 1. This will generate a mirror image result with the higher diaphragm shear along Line 1 in lieu of Line 10.

$$V_{10} = [501 \text{ plf}(474 \text{ ft})(474 \text{ ft}/2) + 615 \text{ plf}(30 \text{ ft})(474 \text{ ft} + 30 \text{ ft}/2)] / 504 \text{ ft} = 130 \text{ kip}$$

Similarly, the shear for Line 1 is determined:

$$V_1 = [501 \text{ plf}(474 \text{ ft})(474 \text{ ft}/2 + 30 \text{ ft}) + 615 \text{ plf}(30 \text{ ft})(30 \text{ ft}/2)] / 504 \text{ ft} = 126 \text{ kip}$$

The diaphragm shear per foot is then developed for Line 10 and the edge of the corner surface 1E or 4E near Line 9.5 as shown in Figure 7.5.

$$v_1 = 126 \text{ kip} / 300 \text{ ft} = 421 \text{ plf}$$

$$v_{9.5} = [130 \text{ kip} - 615 \text{ plf}(30 \text{ ft})] / 300 \text{ ft} = 372 \text{ plf}$$

$$v_{10} = 130 \text{ kip} / 300 \text{ ft} = 433 \text{ plf}$$

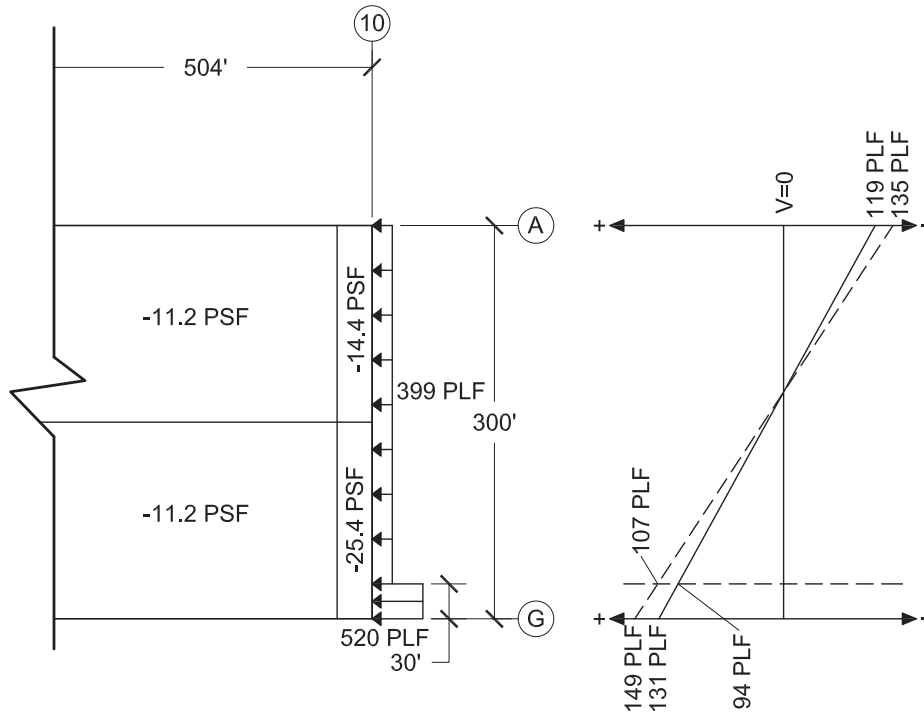


Figure 7.6 East-West Wind Diaphragm Shear and Uplift Load

The maximum diaphragm shear in the East-West direction is determined. This is accomplished by summing the moments about Line A to determine the highest diaphragm shear in the East-West direction on Line G Figure 7.6. The other condition for Load Case B is when Zones 5E and 6E are adjacent to Line A. This will generate a mirror image result with the higher diaphragm shear along Line A in lieu of Line G.

$$V_G = [399 \text{ plf}(270 \text{ ft})(270 \text{ ft}/2)+615 \text{ plf}(30 \text{ ft})(270 \text{ ft}+30 \text{ ft}/2)]/300 \text{ ft} = 66 \text{ kip}$$

Similarly, the shear for Line A is determined:

$$V_A = [399 \text{ plf}(270 \text{ ft})(270 \text{ ft}/2+30 \text{ ft})+615 \text{ plf}(30 \text{ ft})(30 \text{ ft}/2)]/300 \text{ ft} = 60 \text{ kip}$$

The diaphragm shear per foot is then developed for Line G and the edge of the corner surface 5E or 6E near Line F.5 as shown in Figure 7.6.

$$v_G = 66 \text{ kip}/504 \text{ ft} = 131 \text{ plf}$$

$$v_{F.5} = [66 \text{ kip} - 615 \text{ plf}(30 \text{ ft})]/504 \text{ ft} = 94 \text{ plf}$$

$$v_A = 60 \text{ kip}/504 \text{ ft} = 119 \text{ plf}$$

7.4 Skylight Consideration and Smoke Hatch Considerations

Repetitive skylight or smoke hatch opening in the roof diaphragm reduce the shear strength of the diaphragm along those lines of openings. A detailed discussion of this effect and the necessary shear adjustments is presented in Section 5.4 for seismic diaphragm design and is equally applicable to wind diaphragm design. The reduction in width increases the diaphragm unit shear along the lines of skylights as shown in Figures 7.5 and 7.6. The increase in diaphragm shear along the rows of skylights:

$$V_1 = 421 \text{ plf (1/0.90)} = 468 \text{ plf LRFD}$$

$$V_{9.5} = 372 \text{ plf (1/0.90)} = 413 \text{ plf LRFD}$$

$$V_{10} = 433 \text{ plf (1/0.90)} = 481 \text{ plf LRFD}$$

$$V_A = 119 \text{ plf (1/0.88)} = 135 \text{ plf LRFD}$$

$$V_{F.5} = 94 \text{ plf (1/0.88)} = 107 \text{ plf LRFD}$$

$$V_G = 131 \text{ plf (1/0.88)} = 149 \text{ plf LRFD}$$

7.5 Wind Diaphragm Design

The wind diaphragm design is a check to determine that the deck design to resist seismic diaphragm shear loads is adequate to resist the effect of the combined wind diaphragm shear and net uplift loading. This is accomplished in two steps. The wind diaphragm shear design strength in combination with the net wind uplift for the steel deck is developed using the Verco Steel Deck Diaphragm web-based design tool. This resulting strength will then be compared to the wind diaphragm shear design load for each steel roof deck attachment zone.

The steel roof deck diaphragm designed for seismic in Section 5 is analyzed to determine the diaphragm shear strength in combination with wind uplift. This is accomplished using the Verco web-based Steel Deck Diaphragm design tool. The inputs for the steel deck type and connection patterns are the same as those presented in Section 5 along with the governing net wind uplift for each steel deck zone. The governing net wind uplift for each steel roof deck attachment zone is determined from the net uplift in Figure 7.5.

| Roof Deck Zone | Net Wind Uplift |
|----------------|-----------------|
| IV | 20.9 psf |
| III | 13.1 psf |
| II | 13.1 psf |
| I | 13.1 psf |

These wind pressures apply the higher wind Zone 2 pressure across Zones 3 and 3e. For diaphragms that have high net wind uplift in combination with high wind diaphragm shear, it may be necessary to break the roof into additional zones to perform a more exact analysis.

The resulting design diaphragm shear in combination with the corresponding net wind uplift are presented in Figures 7.7, 7.8, and 7.9. These present the second pages of the summary output of the Steel Deck Diaphragm Shear design tool. The first pages for seismic design is presented in Figures 5.7, 5.8, and 5.9. The wind design shear strength for each roof deck Zone is summarized in Table 7.9.

20 ga PLB™-36 Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Lapped Deck
with MWFRS Design Net Wind Uplift (LRFD) of 20.9 psf



Hilti X-HSN24 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
PunchLok II Connection (VSC2) Sidelap Connections

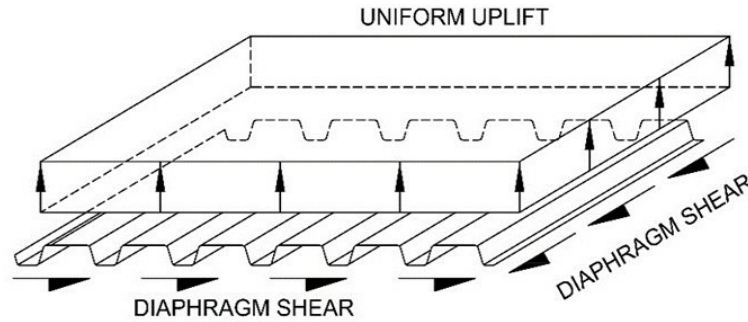
A572 GR50 Support Member or Equivalent
0.125 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

LRFD Design Combined Wind Uplift & Diaphragm Shear Strength ΦS_n (plf) Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 2441 | 2401 | 2361 | 2321 | 2281 | 2241 | 2201 | 1972 | 1770 |
| 6 | 2441 | 2401 | 2361 | 2321 | 2281 | 2241 | 2201 | 1972 | 1770 |
| 8 | 2358 | 2265 | 2251 | 2235 | 2216 | 2144 | 2127 | 1972 | 1770 |
| 12 | 2042 | 1936 | 1973 | 1883 | 1914 | 1836 | 1860 | 1791 | 1770 |
| 18 | 1679 | 1572 | 1674 | 1584 | 1501 | 1585 | 1512 | 1445 | 1514 |
| 24 | 1430 | 1329 | 1476 | 1390 | 1311 | 1240 | 1356 | 1292 | 1232 |
| 36 | 1128 | 1040 | 1240 | 1161 | 1090 | 1026 | 969 | 917 | 1061 |

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 |
| 6 | 7 | 7 | 7 | 7 | 6 | 6 | 6 | 7 | 8 |
| 8 | 7 | 7 | 7 | 7 | 6 | 6 | 6 | 7 | 8 |
| 12 | 7 | 8 | 8 | 8 | 8 | 8 | 7 | 8 | 8 |
| 18 | 9 | 10 | 9 | 9 | 10 | 9 | 9 | 10 | 9 |
| 24 | 11 | 12 | 11 | 11 | 11 | 12 | 10 | 11 | 11 |
| 36 | 17 | 14 | 13 | 14 | 15 | 14 | 15 | 15 | 13 |



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Figure 7.7 20 Gage PLB-36 Shear and Uplift Table with 36/7/4 Pattern for Zone IV

22 ga PLB™-36 Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Lapped Deck
with MWFRS Design Net Wind Uplift (LRFD) of 13.1 psf



Hilti X-HSN24 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
PunchLok II Connection (VSC2) Sidelap Connections

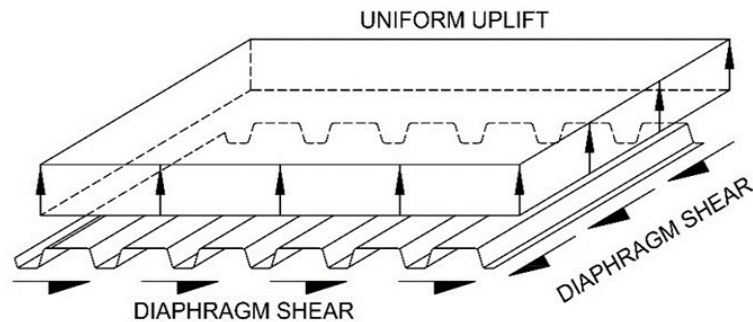
A572 GR50 Support Member or Equivalent
0.125 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

LRFD Design Combined Wind Uplift & Diaphragm Shear Strength ΦS_n (plf) Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 2177 | 2156 | 2134 | 2113 | 2092 | 1902 | 1684 | 1503 | 1349 |
| 6 | 2093 | 2074 | 2055 | 2037 | 2019 | 1902 | 1684 | 1503 | 1349 |
| 8 | 1961 | 1879 | 1879 | 1877 | 1874 | 1812 | 1684 | 1503 | 1349 |
| 12 | 1650 | 1558 | 1600 | 1523 | 1560 | 1493 | 1526 | 1467 | 1349 |
| 18 | 1329 | 1240 | 1329 | 1254 | 1186 | 1262 | 1202 | 1147 | 1212 |
| 24 | 1124 | 1043 | 1162 | 1091 | 1028 | 971 | 1068 | 1016 | 968 |
| 36 | 888 | 819 | 971 | 908 | 852 | 802 | 757 | 716 | 829 |

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 7 | 7 | 7 | 7 | 6 | 7 | 7 | 7 | 7 |
| 6 | 7 | 7 | 7 | 7 | 6 | 7 | 8 | 8 | 10 |
| 8 | 7 | 7 | 7 | 7 | 7 | 7 | 8 | 8 | 10 |
| 12 | 8 | 9 | 9 | 9 | 8 | 9 | 9 | 9 | 10 |
| 18 | 11 | 12 | 11 | 11 | 11 | 11 | 11 | 12 | 10 |
| 24 | 13 | 14 | 13 | 12 | 13 | 14 | 13 | 14 | 14 |
| 36 | 17 | 18 | 16 | 17 | 18 | 16 | 17 | 18 | 16 |



Tables generated using V1.0.3 of calculator based on AISI S310-16.

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Figure 7.8 22 Gage PLB-36 Shear and Uplift Table with 36/7/4 Pattern for Zones II and III

22 ga PLB™-36 Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Lapped Deck
with MWFRS Design Net Wind Uplift (LRFD) of 13 psf



Hilti X-HSN24 PAF Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
PunchLok II Connection (VSC2) Sidelap Connections

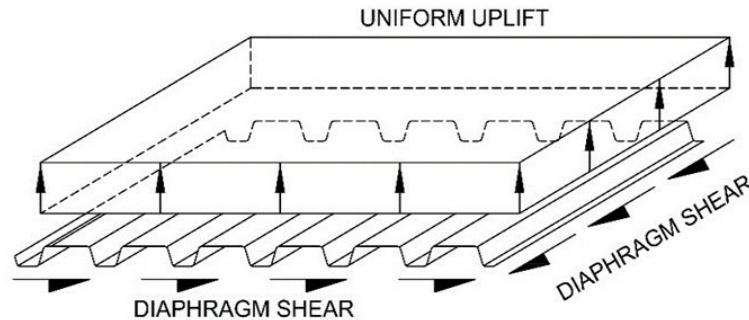
A572 GR50 Support Member or Equivalent
0.125 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

LRFD Design Combined Wind Uplift & Diaphragm Shear Strength ΦS_n (plf) Generic 3 Span Condition

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 1090 | 1079 | 1068 | 1058 | 1047 | 1036 | 1026 | 1015 | 1004 |
| 6 | 1090 | 1079 | 1068 | 1058 | 1047 | 1036 | 1026 | 1015 | 1004 |
| 8 | 1090 | 1079 | 1068 | 1058 | 1047 | 1036 | 1026 | 1015 | 1004 |
| 12 | 1090 | 1079 | 1068 | 1058 | 1047 | 1036 | 1026 | 1015 | 1004 |
| 18 | 1044 | 993 | 1040 | 998 | 958 | 998 | 963 | 929 | 963 |
| 24 | 923 | 870 | 946 | 902 | 860 | 822 | 884 | 850 | 818 |
| 36 | 759 | 708 | 822 | 778 | 737 | 700 | 666 | 635 | 721 |

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

| Sidelap Connection Spacing (in.) | Span | | | | | | | | |
|----------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" | 9'-6" |
| 4 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 |
| 6 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 |
| 8 | 13 | 12 | 13 | 12 | 13 | 12 | 13 | 12 | 13 |
| 12 | 13 | 12 | 13 | 12 | 13 | 12 | 13 | 12 | 13 |
| 18 | 13 | 14 | 13 | 14 | 13 | 14 | 13 | 14 | 13 |
| 24 | 17 | 14 | 13 | 14 | 15 | 16 | 15 | 15 | 16 |
| 36 | 17 | 18 | 16 | 17 | 18 | 19 | 20 | 22 | 16 |



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Figure 7.9 22 Gage PLB-36 Shear and Uplift Table with 36/4 Pattern for Zone I

The maximum factored wind shear demand for the seismic steel roof deck diaphragm Zones is determined from Figure 7.5 using similar triangles. The wind diaphragm shear reduces at a rate of 1.90 plf per ft across the length of the diaphragm which is used to determine the distance to the attachment zone boundaries. This example uses the higher wind pressure at Line 10 for both Lines 1 and 10 to simplify the design. Using the higher shear is slightly conservative. For conditions where this assumption may cause wind to govern, a more exact analysis may be performed.

$$v_{N-S} = 481 \text{ plf at walls at Lines 1 and 10}$$

$L = 504 \text{ ft length of diaphragm}$

$$r_{n-s} = (v_{N-S} + v_{N-S}) / L = (481 \text{ plf} + 481 \text{ plf}) / 504 \text{ ft} = 1.90 \text{ plf/ft across the diaphragm}$$

The resulting wind design diaphragm shear for each steel roof deck zone:

$$v_{IV} = 481 \text{ plf}$$

$$v_{III} = (481 \text{ plf} - 56 \text{ ft}(1.90 \text{ plf/ft})) = 373 \text{ plf}$$

$$v_{II} = 481 \text{ plf} - 120 \text{ ft}(1.90 \text{ plf/ft}) = 251 \text{ plf}$$

$$v_I = 481 \text{ plf} - 168 \text{ ft}(1.90 \text{ plf/ft}) = 160 \text{ plf}$$

The required wind diaphragm shear is summarized in Table 7.9 for comparison with the design wind shear strength in combination with wind uplift. Figure 7.10 depicts the shear strength for each steel roof deck zone compared to the wind shear demand. This clearly demonstrates that the steel roof deck has much higher diaphragm shear strength than required to resist the wind loads, therefore the steel roof deck design for seismic is more than adequate for wind in the North-South direction.

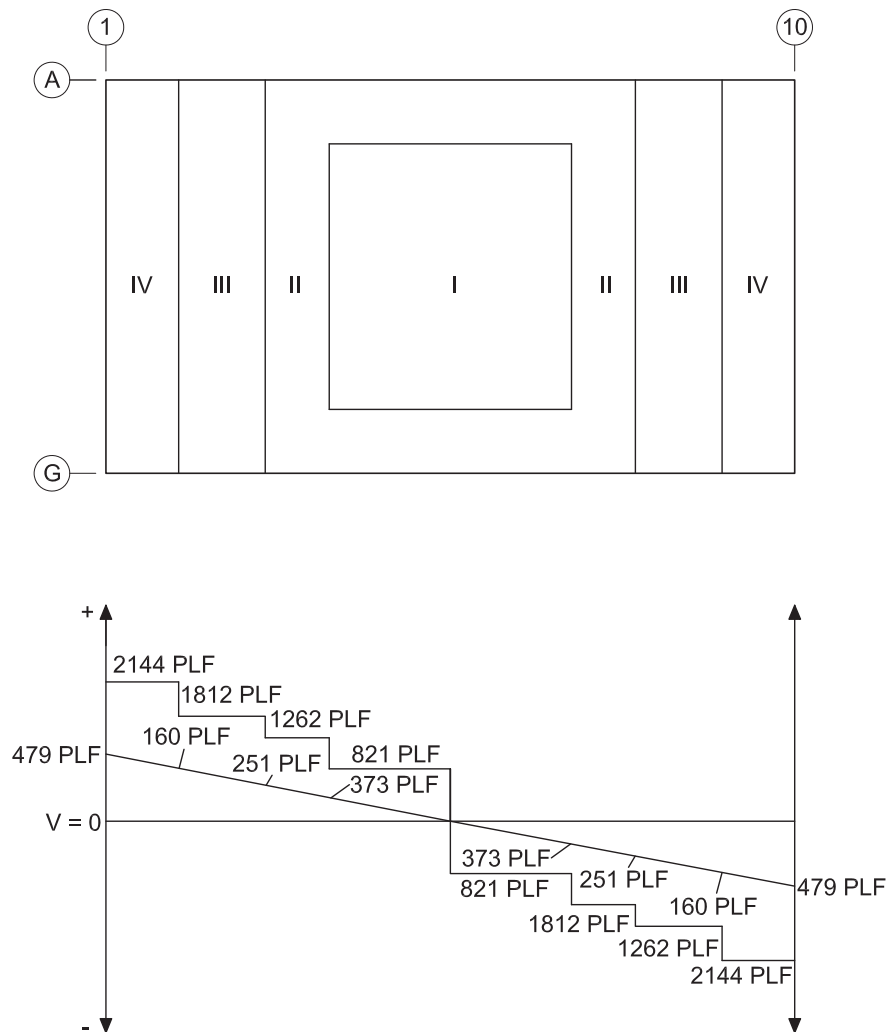


Figure 7.10 Seismic Diaphragm Shear Zones with Ultimate Wind Load

| Zone | PLB gage | Span (ft) | Fastener to Supports | | | VSC2 (in o.c.) | N-S Required | | Design Shear Strength Combined with Wind Uplift, ϕV_n (plf) |
|------|----------|-----------|----------------------|----------|------------------------------------------|----------------|-------------------------------------|---------------------------|-------------------------------------------------------------------|
| | | | Pattern | Fastener | Parallel chords and collectors (in o.c.) | | Factored Net Uplift 0.9D+1.0W (psf) | Factored Shear 1.0W (plf) | |
| IV | 20 | 8 | 36/7/4 | X-HSN 24 | 7 | 8 | 20.9 | 481 | 2144 |
| III | 22 | 8 | 36/7/4 | X-HSN 24 | 8 | 8 | 13.1 | 375 | 1812 |
| II | 22 | 8 | 36/7/4 | X-HSN 24 | 12 | 18 | 13.1 | 253 | 1262 |
| I | 22 | 8 | 36/4 | X-HSN 24 | 16 | 24 | 13.1 | 162 | 821 |

Table 7.9: Wind Shear and Uplift Interaction Check for Seismic Governed Diaphragm

In the East-West direction, the maximum required factored diaphragm shear is 131 plf with an increase to 149 plf considering the effect of skylight openings, which is less than the 1262 plf design strength for Zone II, therefore the seismic design for the steel roof deck is more than adequate for wind in the East-West direction.

7.6 Wind Diaphragm Chord Design

By inspection the wind diaphragm shear forces are approximately 30% of the seismic forces, therefore the chord design for seismic developed in Section 5.6 will be adequate for wind. If the wind loads are similar in magnitude to the seismic loads, then a chord analysis similar to that for the seismic diaphragm design is warranted.

The maximum moment of the diaphragm (LRFD):

$$M_{\max} = wL^2/8 \text{ for a uniformly loaded beam}$$

$$M_{\text{mid}} = R_2(L - x) \text{ for a uniform load at one end of beam}$$

Where:

R_2 = Reactions away from load

x = distance to desired moment

For this analysis, the Zone 1 + 4 load of 501 plf will be carried across the entire width of the diaphragm as the uniform load and the difference between the Zone 1E + 4E load and Zone 1 + 4 load will be applied as the uniform load at one end of the beam as shown in Figure 7.5.

$$R_2 = V_2 = w(2a)^2/(2L) = (615 \text{ plf} - 501 \text{ plf})(30 \text{ ft})^2 / (2(504 \text{ ft})) = 102 \text{ lbs} \approx 0.1 \text{ kip}$$

The maximum chord force of the diaphragm:

$$T_{\max} = C_{\max} = M_{\max}/b = (wL^2/8 + V_2(L - x)) / b$$

For N-S:

$$T = C = (0.501 \text{ kip/ft})(504 \text{ ft})^2/8 + (0.1 \text{ kip})(504 \text{ ft} - 504 \text{ ft} / 2) / 300 \text{ ft} = 53.8 \text{ kips}$$

The chord force of 53.8 kip for wind is much less than the 250 kip chord force for seismic, therefore the chord design for seismic is more than adequate for wind loads. A similar analysis can be performed for the East-West direction with the same end result that the chord design for seismic is more than adequate for wind loads.

7.7 Wind Diaphragm Deflection

Diaphragm deflection due to wind loading is both a structural and serviceability issue. The primary consideration is for life safety in that excessive diaphragm deflection leads to deformation compatibility issues between structure members and P-delta effects potentially leading to collapse. Neither ASCE 7 nor the IBC provides prescriptive limits to the effects of diaphragm deflection, leaving it up to the designer's judgment. The commentary to ASCE 7 Appendix C provides some guidance as to appropriate deflection limits and recommended wind speed for deflection checks.

Appropriate deflection limits are necessary to prevent damage to non-structural elements of a building. These excessive deflections can cause cracking of building cladding, interior non-structural walls and partitions. ASCE 7 Appendix Commentary Section CC.2.2 recommends that a lateral deflection limit between L/600 and L/400 of the building or story height is generally an acceptable range for most structures. For this warehouse structure we will select most liberal recommended L/400 limit.

The second consideration is to determine the wind speed that produces a corresponding wind pressure that is appropriate with the deflection check. ASCE 7 provides additional guidance in Section CC.2.2 with regard to suggested wind speed for the deflection check. The use of the wind speed based on a 700-year mean re-occurrence interval (MRI), for a Category II building, that is used for strength design of the diaphragm is excessively conservative for deflection. Winds speeds based on a 10-year or 50-year MRI are appropriate for most buildings. Going up to a 100-year MRI may be appropriate for drift sensitive buildings. For this warehouse example we will use the lowest recommended 10-year MRI wind speeds. ASCE 7 Appendix CC provides wind speed maps for 10-year, 25-year, 50 year, and 100-year wind speeds to determine deflection.

The diaphragm shear for determining deflection needs to be adjusted to reflect lower wind speed of the 10-year MRI from that of the 700-year MRI used for the strength design of the diaphragm. For this single-story structure, this is a relatively straight forward conversion from the shear used for the diaphragm strength to the lower shear for the deflection check. The first step is to look up the basic wind speed from ASCE 7 Figure CC.2.1. The wind pressure is then used to determine the wind pressure, q_h .

$$V = 65 \text{ mph} \quad \text{ASCE 7 Figure CC.2.1}$$

$$q_h = 0.00256K_z K_{zt} K_d K_E V^2 \quad \text{ASCE 7 eq. 26.10-1}$$

The critical observation is that the wind pressure is related to the square of the velocity therefore it is the ratio of the 10-year MRI wind velocity squared divided by the 700-year MRI velocity squared.

$$\begin{aligned} V^2 &= (10\text{-year MRI wind velocity})^2 / (700\text{-year MRI velocity})^2 \\ &= (65 \text{ mph})^2 / (95 \text{ mph})^2 = 0.47 \end{aligned}$$

This ratio can then be used to adjust all the wind pressures that result in the diaphragm shear. By inspection of the engineering mechanics of the wind pressure on the wall and the diaphragm it is observed that the wind pressure has a linear relationship to the diaphragm shear therefore the 0.47 ratio to reduce the wind pressure can also be directly applied to the diaphragm shear. Table 7.10 summarizes the wind diaphragm shear demand developed in Section 7.3 adjusted with the 0.47 ratio of the squares of the wind speeds.

7.7.1 North-South Wind Diaphragm Deflection

The deflection will be based on the diaphragm stiffness without the consideration of skylight openings. The change in stiffness in the lines of skylights is small compared to the overall diaphragm therefore this effect will be ignored which is slightly unconservative. The design shear for deflection is as follows based on the reduction for wind speed using the diaphragm shears from Figure 7.5

$$v_1 = 421 \text{ plf} (0.47) = 198 \text{ plf}$$

$$v_{9.5} = 372 \text{ plf} (0.47) = 175 \text{ plf}$$

$$v_{10} = 433 \text{ plf} (0.47) = 203 \text{ plf}$$

The design shear for deflection at the walls on Lines 1 and 10 are within 5 plf. For simplification the higher diaphragm shear of 203 plf will be used for both walls. This is insignificantly conservative in comparison with the overall size of the building. The design shear for deflection is determined for each roof zone using similar triangles as shown in Figure 7.11.

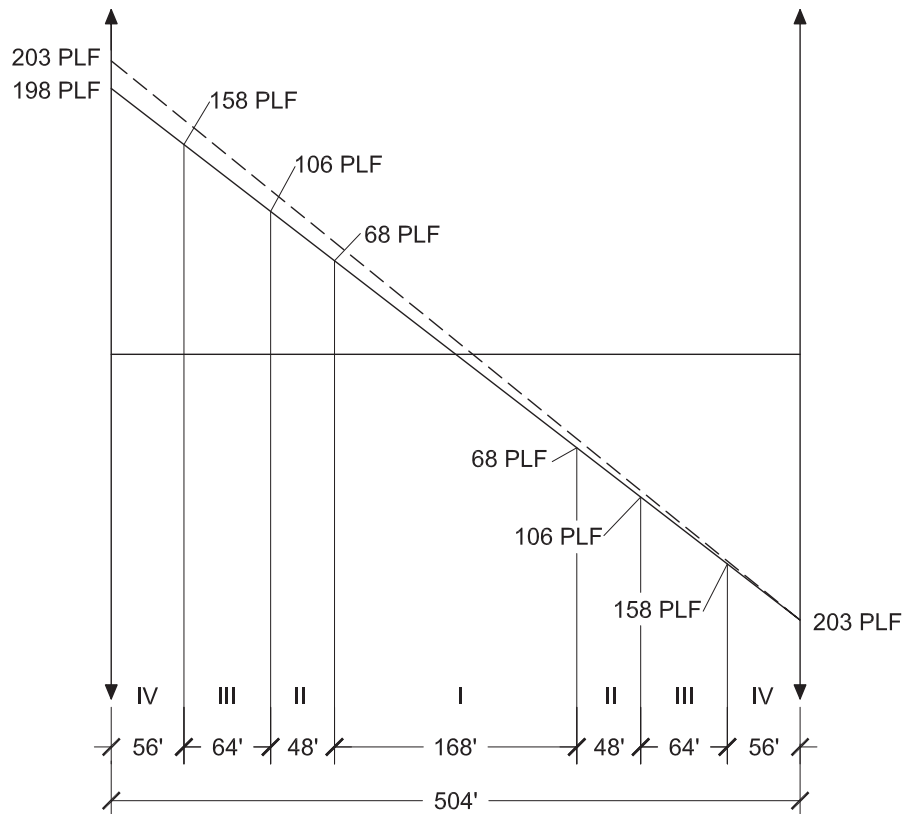


Figure 7.11 N-S Wind Diaphragm Deflection Shear Diaphragm

| Zone | v_{left} | v_{right} | $v_{i \text{ ave}}$ | L_i | G' | $0.5v_{i \text{ ave}} L_i/G'$ |
|-----------------------------------------------------------|-------------------|--------------------|---------------------|-------|----------|-------------------------------|
| | (plf) | (plf) | (plf) | (ft) | (kip/in) | (in) |
| VI | 203 | 158 | 180 | 56 | 191 | 0.03 |
| III | 158 | 106 | 132 | 64 | 150 | 0.03 |
| II | 106 | 68 | 87 | 48 | 117 | 0.02 |
| I | 68 | 0 | 34 | 84 | 96 | 0.01 |
| I | 0 | 68 | 34 | 84 | 96 | 0.01 |
| II | 68 | 106 | 87 | 48 | 117 | 0.02 |
| III | 106 | 158 | 132 | 64 | 150 | 0.03 |
| IV | 158 | 203 | 180 | 56 | 191 | 0.03 |
| Total Elastic Strength Level Shear Deflection, δ_v | | | | | | 0.17 |

Table 7.10 Steel Roof Deck In-Plane Shear Deflection

The resulting deflection based on the 10-year MRI totals 0.17 inches at mid-span. This is then compared to the recommended deflection limit of L/400.

$$L/400 = 32 \text{ ft (12 in/ft)} / 400 = 0.96 \text{ in} > 0.17 \text{ in} = \delta_v$$

Where:

$$L = 32 \text{ ft roof height above floor}$$

This demonstrates that based on the recommended limit of L/400, the diaphragm deflection is acceptable to reduce the risk of non-structural and cladding damage due to wind deflection.

7.7.2 East-West Wind Diaphragm Deflection

The East-West wind diaphragm deflection follows the same procedure as the North South direction. The primary reason to check the deflection in this direction is to check the steel deck to wall connections for out-of-plane wind anchorage of the walls to the steel deck checked in Section 8.3.1.

The maximum diaphragm shear in the East-West direction is determined. This is accomplished by summing the moments about Line A to determine the highest diaphragm shear in the North-South direction on Line G Figure 7.5.

$$V_G = [399 \text{ plf}(270 \text{ ft})(270 \text{ ft}/2)+520 \text{ plf}(30 \text{ ft})(270 \text{ ft}+30 \text{ ft}/2)]/300 \text{ ft} = 63.3 \text{ kip}$$

Similarly, the shear for Line A is determined:

$$V_A = [399 \text{ plf}(270 \text{ ft})(270 \text{ ft}/2+30 \text{ ft})+520 \text{ plf}(30 \text{ ft})(30 \text{ ft}/2)]/300 \text{ ft} = 103 \text{ kip}$$

The diaphragm shear per foot is then developed:

$$v_A = 103 \text{ kip}/504 \text{ ft} = 207 \text{ plf}$$

$$v_{F.5} = [103 \text{ kip} - 520 \text{ plf}(30 \text{ ft})]/504 \text{ ft} = 173 \text{ plf}$$

$$v_G = 63.3 \text{ kip}/504 \text{ ft} = 126 \text{ plf}$$

Reduction for deflection level wind loading:

$$v_A = 204 \text{ plf} (0.47) = 96 \text{ plf}$$

$$v_{F.5} = 173 \text{ plf} (0.47) = 81 \text{ plf}$$

$$v_G = 126 \text{ plf} (0.47) = 59 \text{ plf}$$

The design shear for deflection at the walls on Lines A and G are both very small. For simplicity the higher design shear on Line A of 96 plf will be used for both Lines A and G in this example as shown in Table 7.11.

| Zone | v_{left} (plf) | v_{right} (plf) | $v_{i \text{ ave}}$ (plf) | L_i (ft) | G' (kip/in) | $0.5(v_{i \text{ ave}} L_i)/(1000G')$ (in) |
|-----------------------------------------------------------|----------------------------|-----------------------------|------------------------------|---------------|------------------|-----------------------------------------------|
| II | 96 | 71 | 84 | 39 | 117 | 0.01 |
| I | 71 | 0 | 36 | 111 | 96 | 0.02 |
| I | 0 | 71 | 36 | 111 | 96 | 0.02 |
| II | 71 | 96 | 84 | 39 | 117 | 0.01 |
| Total Elastic Strength Level Shear Deflection, δ_v | | | | | | 0.07 |

Table 7.11 East-West Diaphragm Deflection

8.0 Wind Wall Anchorage and Ties

The roof structure resists the out-of-plane wind loads from the walls. This is the load that induces the diaphragm shear in the roof structure. It is critical that the anchorage between the wall and roof structure be designed to resist the appropriate combination of wind loads from the wall surface and roof surface simultaneously. To accomplish this, it is critical that the appropriate wind loads are determined.

The anchorage is between the concrete precast tilt-up wall panels and the roof structure. The tilt-up wall panels are the primary load bearing shear walls that act as the vertical lateral force resisting system for the building. The roof structure, including the steel deck, OWSJ, OWSJG, and chords, acts as the primary horizontal diaphragm stabilizing all the vertical elements of the building. This assembly of tilt-up walls and roof structure therefore meets the definition to qualify for MWFRS loads in ASCE 7 Section 26.2.

MAIN WIND FORCE RESISTING SYSTEM (MWFRS): An assemblage of structural elements assigned to provide support and stability for the overall building or other structure. The system generally receives wind loading from more than one surface.

The secondary question that may be raised is, should the connection between the steel roof deck or OWSJ be based on C&C or MWFRS loading? The steel roof deck or OWSJ resists the simultaneously applied wind load from bracing the wall surface and the wind load from the roof surface. The connections therefore need to be designed to resist both axial loads induced by the wall anchorage force and net wind load from the roof surface. This connection is part of the MWFRS because it receives wind load from more than one surface.

The design of the steel roof deck and OWSJ also should be done using the MWFRS loads following the same reasoning of the wall to roof structure connections. The axial load due to the wall anchorage is due to the wind on the wall surface and the bending load on the steel roof deck or OWSJ is due to the wind load on the roof surface. Both are members of the MWFRS, and resist loads from more than one surface, therefore the use of MWFRS loads is appropriate.

8.1 Wind Load on Walls

The wind pressures on the walls in combination with the wind pressure on the roof surface are required to design the roof structure to wall connections. These are used to develop the out-of-plane wall anchorage forces needed to design the roof structure to wall connections and ties that distribute the loads into the diaphragm. This is an extension of the MWFRS loads developed in Sections 4 and 7. The first 6 steps of the design of the MWFRS loads are shown in Sections 4 and 7 for the OWSJG and steel roof deck diaphragm.

The wind loads for the wall anchorage and tie force include both the external and internal wind pressures. Unlike the wall wind load for the diaphragm forces, where the internal pressure balances out on both walls, as developed in Section 7, the wall wind anchorage force on an individual wall is the net result for the external and internal pressure. The resulting wind pressures are applied as shown in Figure 8.1.

Step 7: Design wind load is in accordance with ASCE 7 Section 28.3-1 (cont.).

$$p = q_h[(GC_{pf})-(GC_{pi})] \quad \text{ASCE 7 eq 28.3-1}$$

The wind pressure for any surface shall not be less than 16 psf acting in either direction normal to the surface in accordance with ASCE 7 Section 28.3.4.

N-S Wall Surface 1:

$$p_1 = 20.3[(0.40)-(-0.18)] = 11.8 \text{ psf (inward)}$$

N-S Wall Surface 4:

$$p_4 = 20.3[(-0.29)-(0.18)] = -9.5 \text{ psf (outward)}$$

Roof surface wind pressures for all zones are presented in Tables 8.1a and 8.1b.

Load Case A (N-S) with internal pressure:

| Surface | 1 | 2 | 3 | 4 | 1E | 2E | 3E | 4E |
|----------------|------|-------|-------|------|------|-------|-------|-------|
| | Wall | Roof | Roof | Wall | Wall | Roof | Roof | Wall |
| p (psf) | 11.8 | -17.6 | -11.1 | -9.5 | 16.0 | -25.3 | -14.4 | -12.4 |

Table 8.1a N-S MWFRS Pressures

Load Case B (E-W) with internal pressure:

| Surface | 2 | 3 | 5 | 6 | 2E | 3E | 5E | 6E |
|----------------|-------|-------|------|------|-------|-------|------|-------|
| | Roof | Roof | Wall | Wall | Roof | Roof | Wall | Wall |
| p (psf) | -17.6 | -11.1 | 11.8 | -9.5 | -25.3 | -14.4 | 16.0 | -12.4 |

Table 8.1b E-W MWFRS Pressures

The wind loads on the parapets for the MWFRS loading fully developed in Section 7.1 are summarized as follows:

North-South or East-West Parapet Loads:

$p_p = 30.9$ psf, for the windward parapet

$p_p = 20.6$ psf, for the leeward parapet

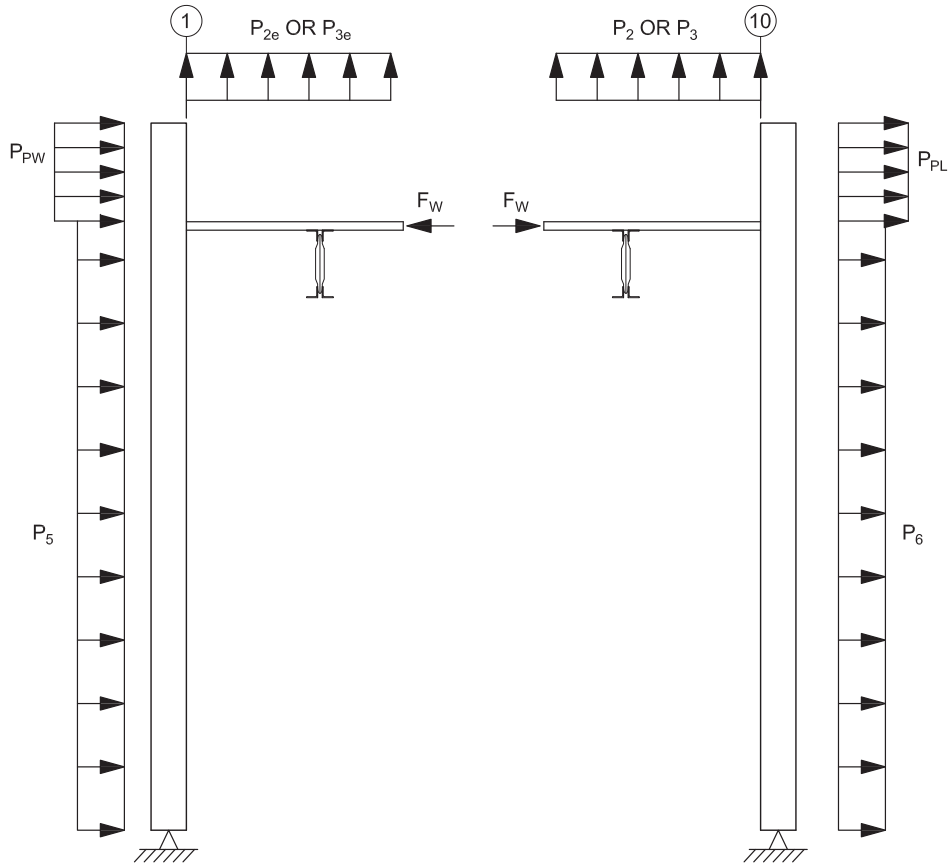


Figure 8.1 E-W Wind Wall Loading and Anchorage Force

8.2 Load Combinations

The governing load combinations are the same as those developed in Section 7.2 for the roof diaphragm with the exception that the wind loads on the walls are higher.

8.3 Wall Bracing with Steel Roof Deck

The wind loads applied to the East and West walls are resisted by the steel deck of the roof structure as shown in Figure 8.2. Resulting out-of-plane loads on windward wall load, F_w , and leeward wall load, F_L , are determined by summing the moments of the uniform pressures about the floor level and dividing by the height of the roof above the floor level for the direction being considered. This ignores any small effect of fixity of the wall panels at the floor level.

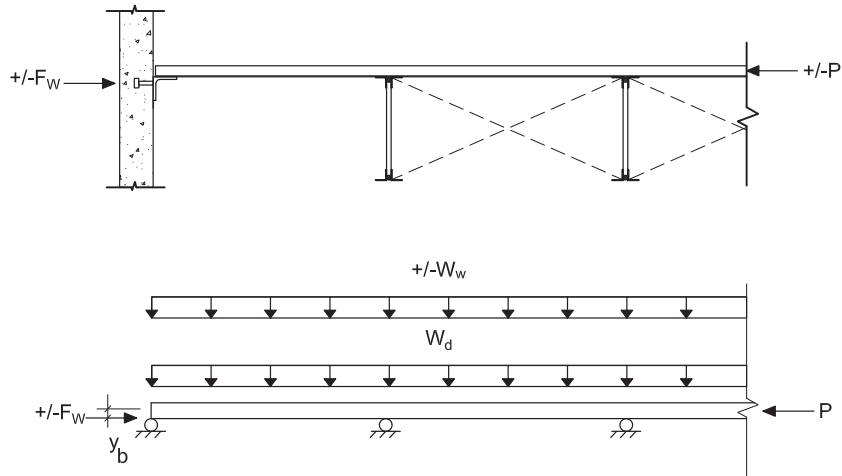


Figure 8.2 Wall Anchorage Loading for E-W wall braced by Steel Deck

For the East-West walls, the corners of the building have the tallest parapet therefore have the highest wall load into the diaphragm. The most severe wind loads occur in the corner wind zones, assuming the ground elevation is at floor level along Lines 1 and 10.

Sum moments about floor level:

$$W = [p_w h_w (h_w/2) + p_p (h_p)(h_w + h_p/2)]/h_w$$

Where:

W = horizontal resultant wind force on roof connections and members

h_w = height of wall or average height of wall from floor to roof diaphragm height between wall ties

h_p = height or average height of parapet above roof diaphragm height between wall ties

p_w = total horizontal pressure on walls

p_p = total horizontal pressure on parapets

Wall Anchorage Wind Loads:

$$W_5 = [11.8(32)(32/2) + 30.9(5)(32+5/2)]/32 = 355 \text{ plf}$$

$$W_{5E} = [16.0(32)(32/2) + 30.9(5)(32+5/2)]/32 = 423 \text{ plf (controls inward case)}$$

$$W_6 = [-9.5(32)(32/2) - 20.6(5)(32+5/2)]/32 = -263 \text{ plf}$$

$$W_{6E} = [-12.4(32)(32/2) - 20.6(5)(32+5/2)]/32 = -309 \text{ plf}$$

$$W_{\min} = [\pm 16.0(32)(32/2) \pm 16.0(5)(32+5/2)]/32 = \pm 342 \text{ plf (controls outward case)}$$

The out-of-plane wall anchorage load on the roof diaphragm is much less than seismic wall anchorage force of 2779 plf developed in Section 6.0. The lower wall anchorage force does not ensure that the deck is adequate to brace the wall for wind loads. Seismic wall anchorage forces are not combined with any net roof surface wind loads. The steel deck connections and axial-bending interaction check for seismic wall anchorage only considered the seismic load combination with dead load. For wind, even though the axial load is relatively small, there is a large out-of-plane net uplift load on the deck and connections that needs to be considered in addition to inward dead load, roof live load, and wind loads.

8.3.1 Prying Action of Steel Roof Deck at Ledger

To accurately predict the shear strength of the connections at the wall, the net uplift force on the connections due to wind must be increased to reflect the contribution of prying action due to wall deflection relative to the roof deck. The connection of the steel deck to the ledger attached to the wall will experience some prying action as the diaphragm deflects under wind loading. This is the same as that developed in Section 6.2.3 for seismic except the diaphragm deflection is much smaller leading to much less prying action on the connections due to wind loading. The prying action is determined as follows referencing Figure 6.5:

For this project:

$$\begin{aligned}h_r &= 36 \text{ ft maximum roof height on Lines 1 and 10} && \text{see Section 2.0} \\ \delta_x &= 0.07 \text{ in, wind diaphragm deflection} && \text{see Section 7.7.2} \\ e &= 1.5 \text{ in, leverage length of connection}\end{aligned}$$

The rotation of the wall relative to vertical which equals the rotation of the ledger relative to the plane of the deck:

$$\begin{aligned}\theta &= \tan^{-1} ((\delta x)/h) = \tan^{-1} (0.07 \text{ in} / (36 \text{ ft} (12 \text{ in/ft})) = 0.0093^\circ \\ x &= e \tan (\theta) = (1.5 \text{ in}) \tan (0.0093^\circ) = 0.0002 \text{ in}\end{aligned}$$

The rotation of the wall is very small leading to minimal prying action of 0.0002 inches. As shown below, this is small enough that the effects can be ignored because there is not adequate deflection of the connection to induce any tension force due to prying action.

The tension load resulting from the prying action is determined in accordance with IAPMO ER-2018 eq T-1 as follows.

$$\begin{aligned}S_t &= 1 / (1706 t^2 d_w + 2.51) && \text{IAPMO eq T-1} \\ S_t &= 1 / (1706 (0.0359^2)(0.474) + 2.51) = 0.281 \text{ in/kip}\end{aligned}$$

Where:

$$\begin{aligned}t &= 0.0359 \text{ in steel deck thickness} \\ d_w &= 0.474 \text{ in Hilti X-HSN 24 washer diameter}\end{aligned}$$

The load on the individual Hilti X-HSN 24 is a linear relationship within the design strength range of loading calculated as follows.

$$P_t = x / S_t = 0.0002 / 0.281 = 0.0007 \text{ kip} = 0.7 \text{ lb}$$

This tension load due to prying action at the ledger is extremely small and will have no influence on the connection design.

8.3.2 East-West Wall Anchorage with Steel Roof Deck

When the steel deck braces the wall for out-of-plane wind load, it is also simultaneously resisting either inward wind or wind uplift in combination with the gravity loads. The strength of the Verco steel roof deck and connections to the ledger may be determined using the web-based combined Axial and Bending design tool. The application of this tool is covered in detail in Section 6.

The wind loading for wall bracing for this example is very straight forward. This is due to the highest wall wind and roof pressures being in the corners of the buildings with the lowest roof height. At the mid-point of the walls on Lines 1 and 10 the roof elevation is higher, and the parapet height is smaller than those in the corners. By inspection, the wall will have a lower wall anchorage load because of the higher roof height and lower wind

pressures, therefore only the corner will be checked in this example. It is then easy to find the most severe conditions by looking at the highest inward and uplift wind forces to apply in combination with the wall loads. Roof wind Zone 2E for the East-West Load Case B has the highest wind uplift pressure of 25.3 psf. There is no inward wind pressure associated with this project therefore the inward wind pressure will be set to zero when using the Axial and Bending tool.

The following summarizes the additional user inputs for the wind loads in the steel deck axial and bending tool that were not addressed in Section 6.

East-West Wind Loads:

$$W = 0 \text{ psf Inward Wind Load in Zone 2E}$$

$$W = -25.3 \text{ psf Wind Uplift on Steel Deck in Zone 2E}$$

$$W = -25.3 \text{ psf Wind Uplift on Connections in Zone 2E}$$

When using MWFRS loads, the wind uplift for connections is the same as that for the steel roof deck. This would not be the case when C&C loads are used because the effective wind on the connections is less than that of the steel roof deck. This would be the case when bracing a wall that was not part of the MWFRS.

Wall Inward Wind Anchorage Load for wind Zone 5E to be combined with Uplift Zone 2E:

$$W = 423 \text{ plf Zone 5E}$$

Wall Outward Anchorage Load for all wall Zones:

$$W = 342 \text{ plf}$$

Figure 6.7 shows the summary page of the combined Axial and Bending strength tool including all of the inputs and the stress ratios of the resulting calculations. From the selected inputs, 20 gage PLB-36 steel deck with the Hilti X-HSN 24 fasteners in a 36/7/4 pattern has adequate combined axial and bending strength to resist the loads in the field of the diaphragm and to connect the steel deck to the wall. This is evident in the design tool output because the steel deck combined axial and bending ratios are less than 1.0 for 1, 2, and 3 span conditions, and the connection demand-to-strength ratios of connections are also less than 1.0, indicating that the connections adequate.

8.3.3 Wall Ties

With the check of the deck in Zone IV to brace the wall for combined wind loading there is no need to go any further. The wall anchorage force from wind is much lower than seismic therefore the sub- diaphragm and wall ties are adequate to carry the wind load across the diaphragm.

8.4 North-South Wall Bracing with Open Web Steel Joists

The wind loads on the walls applied to the North and South walls are resisted by the open web steel joists of the roof structures as shown in Figure 6.2. Resulting out-of-plane windward wall load, F_w , and leeward wall load, F_L , are determined by summing the moments of the uniform pressures about the floor level and dividing by the height of the roof above the floor level using the same methods used for the steel roof deck in Section 8.3.

For the N-S direction:

$$W_1 = [11.8(32)(32/2)+30.9(5)(32+5/2)]/32 = 355 \text{ plf}$$

$$W_{1E} = [16.0(32)(32/2)+30.9(5)(32+5/2)]/32 = 423 \text{ plf (controls inward case)}$$

$$W_4 = [-9.5(32)(32/2)-20.6(5)(32+5/2)]/32 = -263 \text{ plf}$$

$$W_{4E} = [-12.4(32)(32/2) - 20.6(5)(32+5/2)]/32 = -309 \text{ plf}$$

$$W_{\min} = [\pm 16.0(32)(32/2) \pm 16.0(5)(32+5/2)]/32 = \pm 342 \text{ plf} \quad (\text{controls outward case})$$

To determine the North-South Wall bracing axial load on each joist at 8 ft o.c., the out-of-plane wind load per foot on the diaphragm is multiplied by the joist spacing.

$$P_1 = (w_1)(s) = 355 \text{ plf}(8 \text{ ft/joist}) = 2.8 \text{ kip/joist} \quad (\text{tension in joist top chord})$$

$$P_{1E} = (w_{1E})(s) = 423 \text{ plf}(8 \text{ ft/joist}) = 3.3 \text{ kip/joist} \quad (\text{compression in joist top chord})$$

$$P_4 = (w_4)(s) = -342 \text{ plf}(8 \text{ ft/joist}) = -2.7 \text{ kip/joist} \quad (\text{tension in joist top chord})$$

$$P_{4E} = (w_{4E})(s) = -342 \text{ plf}(8 \text{ ft/joist}) = -2.7 \text{ kip/joist} \quad (\text{tension in joist top chord})$$

Where the windward wall is applying a compression load to the roof diaphragm (+) and the leeward wall is applying a tension load to the roof diaphragm (-).

The resulting maximum axial joist loads due to wind of 3.3 kip/joist are well below the seismic wall bracing joist load for this example of 22.2 kip/joist. Even though the axial forces due to wind are much lower, it is recommended that they be specified on the plans due to the interaction with wind loads on the roof surface. In high seismic regions with low design wind speeds, the wind will not likely govern, however as the wind loads get higher or the seismic load drops it may be difficult to determine which will govern.

OWSJ Wind Wall Tie axial load is governed by axial compression.

$$W = 3.3 \text{ kip/joist}$$

The connection of the joist seat to the wall embed should be checked for combined shear and tension due to wind load. The combined wind shear demand is low enough compared to the seismic shear demand on the weld connection shown in Figure 6.2 that the wind is unlikely to govern.

The axial loads for wall wind wall bracing are combined by Vulcraft with the net wind uplift. Generally, only the C&C loads, developed in Section 4, are used in combination with the axial loads. The C&C loads on the roof are higher than the MWFRS loads used to check the wind wall bracing in this section therefore this is conservative. Although using C&C wind loads on the roof surface with the wind wall bracing axial loads is conservative, it significantly simplifies the information that needs to be communicated on the structural plans to Vulcraft.

In higher wind regions, the design professional may want to use the MWFRS uplift loads in combination with the axial loads for potentially greater economy. It is recommended that this be coordinated with Vulcraft to determine whether there is any value to this level of detail, and for the best methods to communicate this in the design documents. Due to the large effective wind area that most joist will have, this level of detail and complexity will likely make very little difference to the final joist design, even in high wind areas.

9.0 Factory Mutual

Many engineers will ask, what steel roof deck is Factory Mutual approved? The answer is simple, Verco 1½ in deep B, and 3 in deep N series steel roof deck products are Factory Mutual approved, but specifying one of these decks does not lead to a Factory Mutual approved roof assembly. The key here is that the FM approval is linked to an assembly of components from different manufacturers including the roof covering (membrane), roof cover board (when used), insulation board(s), roof deck, and the attachments of the components. The assembly does not include the support framing for the roof deck.

FM Global®, commonly known as Factory Mutual, is an organization that provides risk management for their members including property loss insurance and testing of building components to reduce the risk of property loss. For a roof structure, the goal is to minimize or eliminate the risk of a roof failure due to wind or the rapid spread of a fire in a structure. This is intended to save property and goes well beyond the minimum life safety requirements of the building codes, which focus on the safety of the occupants rather than on reducing property loss due to a high wind event or fire.

Factory Mutual provides design review and assistance for projects that are being designed for a client that is a Factory Mutual member company. For these projects, the design team can take advantage of this process to help determine the best roof assembly for the client. Many clients that are not Factory Mutual member companies may specify a Factory Mutual compliant roof because they want the assurance of a high-performance roof system to minimize the risk of a roof failure or serious fire. These projects do not have the advantage of direct assistance from Factory Mutual for design review to help verify the roof assembly is Factory Mutual compliant. This leaves it up to the design team to wade through the Factory Mutual recommendations to develop the assembly themselves.

Part of the struggle for the design team is that Factory Mutual provides recommendations, not minimum design standards. An outcome of this difference is there are many recommendations that may be hard to incorporate into minimum design standards because they are recommendations of best practices rather than a minimum life safety design specification. This requires some interpretation or judgment by the design team to specify a steel roof deck that would fall within the FM recommendations for an approved roof assembly.

FM Considerations for Clients that are Not FM Member Companies

There are several important considerations for projects where the client is not a Factory Mutual member company. The client or architect may desire the confidence in the performance of a Factory Mutual roof system including the roof covering, insulation boards and attachment system to ensure weather tightness against wind and fire events. This can be delivered without imposing all of the FM requirements on the steel roof deck design. A rational performance-based steel deck design in accordance with AISI S100 and wind loads from ASCE 7, or the enhanced wind uplift of FM Data Sheet 1-28, will provide a good roof deck solution that the FM roof system can be attached to. This will generally lead to a more economical roof structure with steel deck and open web steel joists than a design following the restrictive FM methods.

9.1 Factory Mutual Data Sheets

Factory mutual approved assemblies are based on the Property Loss Prevention Data Sheets and Approval Standard for steel roof deck. When attempting to interpret these documents, it is important to remember these are recommendations, not minimum standards. Judgment must be used to follow the intent of the documents in areas where there are not clear recommendations. This example will focus on the requirements of the following FM documents.

Wind Design: FM 1-28 Wind Design, 2015 Interim Revision February 2020 (Property Loss Prevention Data Sheets)

Steel Deck Design: FM 1-29 Roof Deck Securement and Above-Deck Roof Components January, 2016 Interim Revision February 2020 (Property Loss Prevention Data Sheets)

Steel Deck Approval: Class Number 4451 Approval Standard for Profiled Steel Panels for Use as Decking in Class 1 Insulated Roof Construction, June 2012

Factory Mutual provides their property loss prevention data sheets and approval standards free of charge as a public benefit.

FM Data Sheets: <https://www.fmglobal.com/research-and-resources/fm-global-data-sheets>

9.2 Fire Rating

This guide will not address all the issues around fire performance of a roof assembly. Steel roof deck is inherently non-combustible therefore is an acceptable roof deck material for a FM assembly. Factory Mutual classifies roof assemblies based on the flame spread as Noncombustible, Class 1 or Class 2 assemblies. Examples of noncombustible decks are assemblies with concrete, fiber reinforced cement, and gypsum concrete filled steel decks. Steel roof deck in combination with the appropriate roof system meets the Class 1 requirements for a Factory Mutual approved roof assembly. The steel roof deck does not drive this approval, it is simply the support for the assembly. Approved assemblies can be selected using FM RoofNav, or from recommendations of the roof system manufacturer.

Factory Mutual RoofNav: <https://www.roofnav.com/>

RoofNav allows the user to select an assembly. These tend to be driven by the roof system and work down to the steel roof deck. This makes it difficult for an engineer to verify a particular steel roof deck design is acceptable without the architect first selecting a roof system.

9.3 Steel Roof Deck Approval

Steel roof deck used in a Factory Mutual approved roof assembly must be approved based on FM 4451 Approval Standard for Profiled Steel Panels for Use as Decking in Class 1 Insulated Roof Construction. The primary requirements for steel roof deck structural performance is the bending strength of the steel deck, deflection under concentrated load, sidelap connections, and the connections of the steel deck to supports. These are used to develop the maximum span for vertical inward and outward uplift loads.

The maximum span for the steel roof deck is the limiting of the deflection check in FM 4451 Section 4.1, and the span governed by the wind uplift rating in FM 4451 Section 4.3. for up to Class 1-90 ratings. Section 4.1 limits the span to an $L/240$ deflection limit based on a 200 plf load on the deck either at the mid-span for a single span sheet, or mid-span on one span of a 2-span condition. This is determined by engineering mechanics rather than testing. Section 4.3 limits the span of the deck, subject to the service level uniform wind load for a 1-45, 1-60 or 1-90 rating, based on engineering mechanics and the allowable strength determined in accordance with AISI S100. Verco PLB-36 roof deck spans in FM approved 1-45, 1-60, and 1-90 ratings are all governed by the inward deflection limit, as shown in Table 9.3.

These become the maximum approved steel roof deck spans for Verco PLB-36 steel roof deck when structural analysis is used for wind ratings greater than 1-90.

Comparison with Hybrid Panelized Wood Roof Structures

In the Western United States, hybrid panelized roof structures using wood structural panel decks supported by sub-purlins and open web steel joists are common. FM requires the use of FM approved 3/4 in thick fire treated plywood. The use of this plywood greatly reduces the perceived economy of the hybrid panelized roof structure compared to commonly used 15/32 in thick panels. At the time this example was written, there were no FM approved plywood manufacturers listed in RoofNav. Caution should be used if the client requires a FM approved roof structure and a wood structural panel roof deck is being considered, to ensure that the assembly will meet the FM recommendations.

9.4 FM Design Pressure

The design for wind resistance for a Factory Mutual approved design uses higher wind pressures than ASCE 7 for a Category II warehouse structure. In essence, Factory Mutual is using the enhanced wind pressure associated with critical buildings using a 1.15 importance factor. This follows more closely with the importance factor and wind speed maps from ASCE 7-05 than the current ASCE 7-16. For this building example, FM wind pressures will need to be determined following the recommendations of FM 1-28. This method will result in design wind pressures that exceed those developed in Section 4 based on the ASCE 7-16 Components and Cladding requirements. The higher wind pressures will affect the wind uplift design for the steel roof deck.

FM 1-28 Section 3.2 Wind pressure determination generally follows ASCE 7-16 methods, with modifications including the use of the wind maps and the importance factor from ASCE 7-05.

FM 1-28 Section 3.2.1 Calculating Basic Wind Pressure, q_h .

$$q_h = 0.00256K_z K_{zt} K_d K_E V^2 I \quad \text{FM eq 3.2.1a}$$

For this example, the building site is in Ontario, California, which is in a low wind speed region. The basic wind speed is taken from FM 1-28 2015 rev 2020 Figure 3.

$$V = 95 \text{ mph} \quad \text{FM 1-28 Figure 3}$$

The wind parameters related to the site will be assumed as follows from ASCE 7-16, in accordance with FM 1-28 Section 3.2.1:

Wind directionality factor, $K_d = 0.85$ ASCE 7 Table 26.6-1

Exposure Category = C ASCE 7 §26.7.3

Topographic Factor, $K_{zt} = 1.0$ ASCE 7 §26.8.2

Ground Elevation Factor, $K_e = 1.0$ ASCE 7 §26.9

Enclosure Classification = Enclosed ASCE 7 §26.12

Internal Pressure Coefficient, $GC_{pi} = \pm 0.18$ ASCE 7 Table 26.13-1

Importance Factor, $I = 1.15$ FM Section 3.2.1

The velocity pressure coefficients for the roof are determined in accordance with ASCE 7 Table 26.10.1 based on the mean roof height, and exposure category, or using the formulas in the footnotes of the table. To determine the pressure coefficients the nominal height of the atmospheric boundary layer, z_g , and the 3-sec gust-speed power law exponent, α , are taken from ASCE 7 Table 26.11-1. These factors are the same for both the MWFRS and C&C loads.

$$z_g = 900 \text{ ft} \quad \text{ASCE 7 Table 26.11-1}$$

$$\alpha = 9.5 \quad \text{ASCE 7 Table 26.11-1}$$

For $15 \text{ ft} \leq z \leq z_g$ where, $z = h = 38 \text{ ft}$

$$K_h = K_z = 2.01(z/z_g)^{(2/\alpha)} = 2.01(38/900)^{(2/9.5)} = 1.03 \quad \text{ASCE 7 Table 26.10-1}$$

The velocity pressure is then determined for $z = h = 38 \text{ ft}$, the mean roof height is used for the gable roof.

$$q_h = 0.00256K_z K_{zt} K_d K_e V^2 I = 0.00256(1.03)(1.0)(0.85)(1.0)(95)^2(1.15) = 23.3 \text{ psf}$$

For this structure, the wind pressure is the same as that determined in ASCE 7 except for the 1.15 importance factor increase in the FM method.

Use Equation 3.2.1b to determine the design pressure for the specific zone of the building.

$$p = [(GC_p) - GC_{pi}] q_h \quad \text{FM eq 3.2.1b}$$

The nominal ultimate rating of the cladding, p_u , (round up for assembly selection) considering the importance factor and safety factor.

$$p_u = p (SF)$$

Where:

GC_p = external pressure coefficient. This value will vary depending on the roof area in question and its slope, the wall area in question and the height. For values, see Section 4.1 and referenced tables and figures.

Internal pressure coefficient for enclosed buildings.

$$GC_{pi} = +/- 0.18$$

SF = safety factor of 2.0.

NOTE: The +/- signs for GC_{pi} are directional. The values are summed so as to result in the largest numerical value.

Steel Roof Deck External Pressure Coefficients:

FM Table 3.2.2a

This building does not have a 3 ft parapet around the entire perimeter, therefore the reduction of wind pressure in Zone 3 to Zone 2 levels is not permitted in accordance with Footnote 2 to FM Table 3.2.2a. This is more severe than the ASCE 7 requirements that allow for this reduction when the parapet is 3 ft or greater adjacent to Zone 3, as discussed in Section 4. The FM external pressure coefficients for the roof uplift zones are summarized in Table 9.1.

| Zone | 1' | 1 | 2 | 3 |
|--------|-------|------|------|------|
| GC_p | -0.90 | -1.7 | -2.3 | -3.2 |

Table 9.1 Roof Deck C&C External Pressure Coefficients, GC_p

This is an example of where some judgment is warranted with regard to the FM recommendations. Although there is not a 3 ft parapet around the entire roof structure, there is a 3 ft parapet adjacent to Zone 3 as discussed in Section 4.1.2.3. It may therefore be reasonable to use the reduced wind pressure of Zone 2 in the higher Zone 3 areas based on the reduced wind pressure guidance from ASCE 7-16. This would be subject to review and acceptance of FM for the specific project. For this example, we will continue with the higher wind pressure in Zone 3 following the specific FM guidance.

The wind uplift pressure is determined from the velocity pressure and the wind pressure coefficients.

$$p = [(GC_p) - GC_{pi}] q_h \quad \text{FM eq 3.2.1b}$$

$$p_{1'} = [(-0.9) - (+0.18)] (23.3) = -25.2 \text{ psf for Zone 1'}$$

$$p_1 = [(-1.7) - (+0.18)] (23.3) = -43.8 \text{ psf for Zone 1}$$

$$p_2 = [(-2.3) - (+0.18)] (23.3) = -57.8 \text{ psf for Zone 2}$$

$$p_3 = [(-3.2) - (+0.18)] (23.3) = -78.8 \text{ psf for Zone 3}$$

To determine the FM assembly design wind rating, the design wind pressure is multiplied by the safety factor and rounded up to the next design speed in 15 psf increments.

$$p_u = p \text{ (SF)}$$

$$p_{u1'} = 25.2 \text{ psf (2.0)} = 50.4 \text{ psf} \rightarrow \text{use 60 for Zone 1'}$$

$$p_{u1} = 43.8 \text{ psf (2.0)} = 87.6 \text{ psf} \rightarrow \text{use 90 for Zone 1}$$

$$p_{u2} = 57.8 \text{ psf (2.0)} = 115.6 \text{ psf} \rightarrow \text{use 120 for Zone 2}$$

$$p_{u3} = 78.8 \text{ psf (2.0)} = 157.6 \text{ psf} \rightarrow \text{use 165 for Zone 3}$$

The FM wind rating by Zone is summarized in Table 9.2.

| Zone | 1' | 1 | 2 | 3 |
|--------|----|----|-----|-----|
| Rating | 60 | 90 | 120 | 165 |

Table 9.2 Roof Deck C&C External Pressure Coefficients, GC_p

The wind zones for a low slope gable end roof is presented in FM 1-28 Figure 3.2.2a. Fortunately, the wind zones and widths are the same as those in ASCE 7, as shown in Figure 4.3.

9.5 Steel Roof Deck Design for Wind Uplift

The overall design for steel deck securement and above-deck roof components follows the recommendations of FM 1-29. This provides both prescriptive and performance-based approaches to the design of the steel roof deck. This example will follow the performance-based design for the steel roof deck and the prescriptive enhancement approach for the connections securing the steel roof deck to the open web steel joists.

9.5.1 FM Approved Steel Roof Deck

Verco PLB steel roof steel deck is FM approved. This can be verified by searching for Verco steel roof deck in FM RoofNav. RoofNav provides a summary, including the maximum span and attachment requirements, for 1-60, 1-75- and 1-90 wind ratings. All FM steel roof deck approvals provide maximum spans for the lessor of, service level uniform wind uplift ratings up to 1-90 or 200 lb concentrated gravity load deflection limited to L/240, in accordance with FM 4451. Verco maximum FM spans for PLB roof deck summarized in Figure 9.1.

Table 1. Allowable Span Limit Table, Steel Deck secured with FM Approved Fasteners

| Deck Design Thickness, MSG (in. [mm]) | Maximum Allowable, One Span, in., mm | | | | | |
|------------------------------------------|-----------------------------------------------|------|------|------|------|------|
| | Wind Rating | | | | | |
| | 1-60 | | 1-75 | | 1-90 | |
| | in. | mm | in. | mm | in. | mm |
| 22 (0.0299 [0.76]) | 72 | 1829 | 72 | 1829 | 72 | 1829 |
| 20 (0.0359 [0.91]) | 80 | 2032 | 80 | 2032 | 80 | 2032 |
| 18 (0.0478 [1.21]) | 94 | 2388 | 94 | 2388 | 94 | 2388 |
| 16 (0.0598 [1.52]) | 106 | 2692 | 106 | 2692 | 106 | 2692 |
| Deck Design Thickness, MSG (in. [mm]) | Maximum Allowable, Two or More Spans, in., mm | | | | | |
| | Wind Rating | | | | | |
| | 1-60 | | 1-75 | | 1-90 | |
| | in. | mm | in. | mm | in. | mm |
| 22 (0.0299 [0.76]) | 89 | 2261 | 89 | 2261 | 89 | 2261 |
| 20 (0.0359 [0.91]) | 97 | 2464 | 97 | 2464 | 97 | 2464 |
| 18 (0.0478 [1.21]) | 112 | 2845 | 112 | 2845 | 112 | 2845 |
| 16 (0.0598 [1.52]) | 125 | 3175 | 125 | 3175 | 125 | 3175 |

Figure 9.1 Verco PLB-36/HSB-36 FM Spans

This project has design wind ratings above 1-90 that exceed the basic approval for Verco PLB-36 steel roof deck. The design will follow the requirements of FM 1-29 for the higher wind uplift regions of the roof structure.

The maximum spans in accordance with FM 4451 are determined by engineering analysis. The maximum is limited by the strength of the steel roof deck under service level wind load and an $L/240$ deflection limit for a 200 lb concentrated load applied over a 12 in by 12 in area. The service level wind uplift is the wind rating divided by a 2.0 safety factor. For 1-165 this would be $165/2 = 82.5$ psf wind uplift. The allowable strength of the steel deck to resist this service level wind load is determined in accordance with AISI S100. Based on the service level wind load and allowable strength for the steel roof deck, the maximum span is determined using slender beam engineering mechanics for a 1-span or 2-span condition. The concentrated load of 200 lb is applied to a 1 ft width of steel deck. This essentially equates to a 200 plf line load on the steel deck. It is applied at mid-span for a 1-span condition or in the mid-span of one of the spans of a 2-span condition. The maximum span is determined using slender beam mechanics that are presented in FM 4451 based on the moment of inertia of the steel roof deck determined in accordance with AISI S100. FM 4451 does not specify the use of gross or effective moment of inertia for deflection, however by reverse engineering the Verco maximum span table it is apparent that FM uses gross moment of inertia for this calculation. This is not in accordance with AISI S100 serviceability requirements that require the use of the effective section properties under the applied load to determine deflection.

9.5.2 Adhered Roof Covering System

The design of the steel roof deck is treated as being uniformly loaded for a roof assembly in which the roof covering is adhered to the insulation board that is uniformly attached to the steel roof deck as shown in Figure 4.13. Factory Mutual 1-29 Section 2.2.3.2 also allows mechanically attached roof coverings as shown in Figure 4.14 to be treated as a uniform load for steel roof deck design when the spacing of the roof covering attachments is less than one-half the deck span length as shown in Figure 9.2.

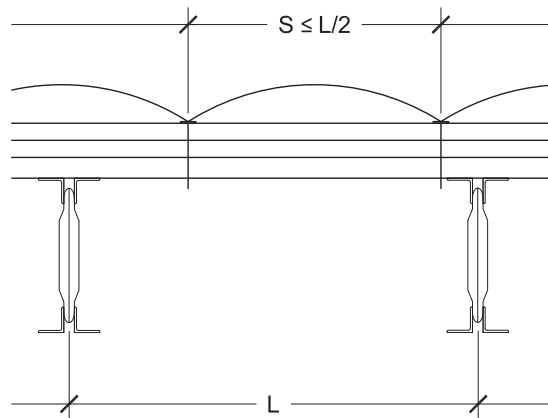


Figure 9.2 Mechanically Attached Roof Covering Spacing

In Section 4 the decision was made to use an adhered roof covering system. This design is very straight forward, based on the service level uniform wind load, strength of the steel deck and connections to supports. FM-1-29 provides routes to design the steel roof deck for strength in Section 2.2.3.2, with either the use of span tables for generic 1½ in steel roof deck, or performance-based design by a licensed engineer.

This example will first look at the tabular approach with the generic 1½ in deck. Enter FM 1-29 Table 1C, provided in Figure 9.3, and look up the 22 gage Grade 50 steel roof deck to find the maximum allowable span for the FM wind rating for Zones 1', 1, and 2 of 7.10 feet for 1-60, 1-90, and 1-120 respectively. In Zone 3, the 20 gage steel roof deck required to meet seismic diaphragm shear meets the 1-165 wind uplift rating with a maximum span of 7.78 ft, as summarized in Table 9.3.

Table 1C. Maximum Steel Deck Span (ft) for 1½ in. (38 mm) Deep, Wide Rib (Type B) Steel Deck with an Adhered Roof Cover, for Wind Ratings from 60 to 225 psf (2.9 to 10.8 kPa)

(NOTE: Use this table when the distance between rows of roof cover fasteners is one-half the deck span or less. Green font indicates that deflection governs over bending stress.)

| Yield Stress psi | Deck Gauge | Ultimate Wind Rating per RoofNav (psf) | | | | | | | | | | | |
|---------------------|------------|----------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | | Maximum Span (ft) | | | | | | | | | | | |
| | | 60 | 75 | 90 | 105 | 120 | 135 | 150 | 165 | 180 | 195 | 210 | 225 |
| 33,000 | 22 | 7.10 | 7.10 | 7.10 | 7.10 | 7.07 | 6.67 | 6.33 | 6.03 | 5.78 | 5.55 | 5.35 | 5.17 |
| | 20 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.43 | 7.05 | 6.72 | 6.44 | 6.18 | 5.96 | 5.76 |
| | 18 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 8.66 | 8.22 | 7.84 | 7.50 | 7.21 | 6.95 | 6.71 |
| | 16 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 9.89 | 9.38 | 8.94 | 8.56 | 8.23 | 7.93 | 7.66 |
| 40,000 | 22 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 6.96 | 6.64 | 6.35 | 6.10 | 5.88 | 5.68 |
| | 20 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.76 | 7.40 | 7.08 | 6.80 | 6.56 | 6.33 |
| | 18 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.04 | 8.62 | 8.25 | 7.93 | 7.64 | 7.38 |
| | 16 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.32 | 9.84 | 9.42 | 9.05 | 8.72 | 8.43 |
| 45,000 | 22 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.04 | 6.74 | 6.48 | 6.24 | 6.03 |
| | 20 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.51 | 7.22 | 6.95 | 6.72 |
| | 18 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 8.76 | 8.41 | 8.11 | 7.83 |
| | 16 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 9.99 | 9.60 | 9.25 | 8.94 |
| 50,000 | 22 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 6.93 | 6.66 | 6.42 | 6.20 |
| | 20 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.72 | 7.42 | 7.15 | 6.91 |
| | 18 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.00 | 8.65 | 8.33 | 8.05 |
| | 16 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.28 | 9.87 | 9.51 | 9.19 |
| 55,000 | 22 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 6.90 | 6.67 |
| | 20 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.69 | 7.43 |
| | 18 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 8.97 | 8.66 |
| | 16 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.24 | 9.89 |
| 60,000 + | 22 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 7.10 | 6.97 |
| | 20 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.78 | 7.77 |
| | 18 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.08 | 9.06 |
| | 16 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.36 | 10.34 |

Green font indicates that deflection governs over bending stress.

Figure 9.3 FM 1-29 Adhered Roof Cover Wind Ratings to 1½” Steel Roof Deck

FM 1-29 does not expressly state anywhere that Tables 1C, 1D or 1E for roof coverings with attachment less than ½ the deck span are based on a 1, 2 or 3-span condition. It becomes apparent that the tables are likely based on a 3-span condition by investigating the maximum span that SDI generic deck can span subject to uniform load. This is important to understand when considering a performance-based alternate to the tabular solution. In a performance-based design, limiting the deck to a 1-span condition would not be an equivalent design. A second question also goes unanswered when using the in FM 1-29 tabular solution, in that should the deck span be further limited to the manufacturer’s maximum approved deck span for 1-span conditions? It very quickly becomes clear that FM provides guidance, not complete mandatory design requirements, therefore interpretation of the intent of the FM documents is required.

| Zone | Rating | Gage | 1½" Steel Generic Roof Deck Span, F _y = 50 ksi | |
|------|--------|------|-----------------------------------------------------------|-----------|
| | | | (ft) | (ft-in) |
| 1' | 1-60 | 22 | 7.10 | = 7' - 1" |
| 1 | 1-90 | 22 | 7.10 | = 7' - 1" |
| 2 | 1-120 | 22 | 7.10 | = 7' - 1" |
| 3 | 1-165 | 20 | 7.78 | = 7' - 9" |

Table 9.3 FM Adhered Roof Cover Maximum Steel Roof Deck Span

The impact of the FM recommendations on the roof structure is that the maximum deck span is limited to 7'-1" by the more severe 200 lb concentrated load applied to a 1 ft width of the bare steel roof deck rather than the 300 lb concentrated load applied to the roof surface over a 2½ ft x 2½ ft area required by the IBC. This leads to a critical decision point. The joist spacing can be reduced from 8 ft to 7'-1" o.c., or the steel deck gage can be increased to 18 gage. This change in the joist spacing will have a relatively small impact on the joist or joist girder weight per square foot but the change from 22 gage to 18 gage steel deck will increase the weight of the steel deck by 50% which will directly impact the erected cost of the steel deck. There are also corresponding increases in erection costs due to additional joists, rows of bridging, and weight of heavier steel roof deck.

The generic steel roof deck table is slightly more conservative than the maximum approved span for the Verco PLB-36 steel roof deck for 2 spans. This is directly associated with the PLB-36 section properties as opposed to the SDI lower bound section properties of all the steel deck manufacturers used for generic tables. For this project the extra 4 inches of span going from 7'-1" generic deck to 7'-5" PLB-36 will not impact the re-spacing of the joists, therefore a solution using PLB-36 section properties is not needed to fine tune the design.

Skylight Considerations. For structures with skylights, the joist spacing can greatly impact the cost of the steel and steel roof deck erection. When the skylight curb does not span from joist-to-joist, a below deck steel support frame is required and the deck needs to be cut to fit around the opening, adding cost to the roof structure. Common skylights that work with the 3 ft PLB-36 deck width are 3 ft x 8 ft and 6 ft x 6 ft. For a warehouse structure with skylight frames, further reducing the joist spacing to 6 ft to accommodate skylight openings without costly below deck steel support frames may be an advantage.

Fire Sprinkler Considerations. Common ESFR fire sprinkler systems allow for a maximum of 100 sf per sprinkler head, with a maximum spacing of 12 ft. They also have a minimum spacing of 8 ft and minimum area of 80 sf per sprinkler head. Sprinkler heads are often located at mid-span between the joists to eliminate horizontal clearance coordination issues. An 8 ft joist spacing allows for an 8 ft x 12 ft sprinkler head layout with 96 sf per head, which is close to the 100 sf limit. A 7 ft joist spacing does not divide into any good spacing between 8 ft and 12 ft, therefore a significant amount of coordination is required for sprinkler head spacing. Reducing to a 6 ft joist spacing again allows the sprinkler head to be installed in an 8 ft x 12 ft pattern with a head at mid-span of every other joist.

9.5.3 Mechanically Attached Roof Covering System

Projects with a mechanically attached roof covering are a more complex problem. The wide spacing of the rows of roof covering screws attaching to the steel roof deck apply concentrated line loads, as opposed to a uniform load from an adhered roof. FM recommends that when the spacing of rows of screws for an attached roof covering is greater than one-half the deck span that the steel roof deck be designed for a series of concentrated line load rather than a uniform uplift load. FM 1-29 provides both a tabular solution for generic steel roof deck, and for performance-based design by an engineer based on a 3-span steel roof deck condition. The use of the tabular solution with generic deck is excessively conservative for wind uplift because the table is based on Grade 33 for 1½ in deep steel roof deck. The Verco PLB-36 steel roof deck is manufactured from Grade 50 steel which will resist significantly higher wind uplift loads.

For the engineering design of the roof structure, the width of the roof covering is often not clearly defined. The unknown element of the design is the width of the roof covering. The following spacings of rows of fasteners for mechanically attached roof covers was taken from a major roofing manufacturer’s FM assemblies. The row spacing is used to determine the allowable concentrated uplift load on the steel deck based on the service level wind load which is the FM wind rating divided by the 2.0 safety factor as presented in Table 9.4.

| Zone | PLB-36 Gage | Rating | Row Spacing | Service Level Uplift | |
|------|----------------|--------|-------------|----------------------|-------|
| | | | (in) | (psf) | (plf) |
| 1' | 22 | 1-60 | 114 | 30.0 | 285 |
| 1 | 22 | 1-90 | 114 | 45.0 | 428 |
| 2 | 22 | 1-120 | 57 | 60.0 | 285 |
| 3 | 20 | 1-165 | 38 | 82.5 | 261 |

Table 9.4 First Trial Mechanically Attached Roof Covering

For narrow width roof covering, the mechanically attached roof may be treated as a uniform load when the row spacing of fasteners is less than one-half of the deck span. For the 38 in row spacing, a 7 ft deck span would be allowed to be treated as a uniform load, however if a 6 ft deck span was selected then it would need to be treated as a concentrated load. For the wider row spacings the uplift from the roof fastening on the steel roof deck will need to be treated as a concentrated load. The concentrated loading is a more severe condition for the steel roof deck than the uniform load for the adhered roof coverings. A 6 ft deck span will be selected for the first iteration of this design process.

9.5.3.1 Steel Roof Deck Zone 3 Design

Steel roof deck design for Zone 3 will start with analyzing the steel roof deck for a roof covering with a 38 in wide attachment spacing that will meet the wind rating. The loads are applied to the steel roof deck as shown in Figure 9.4.

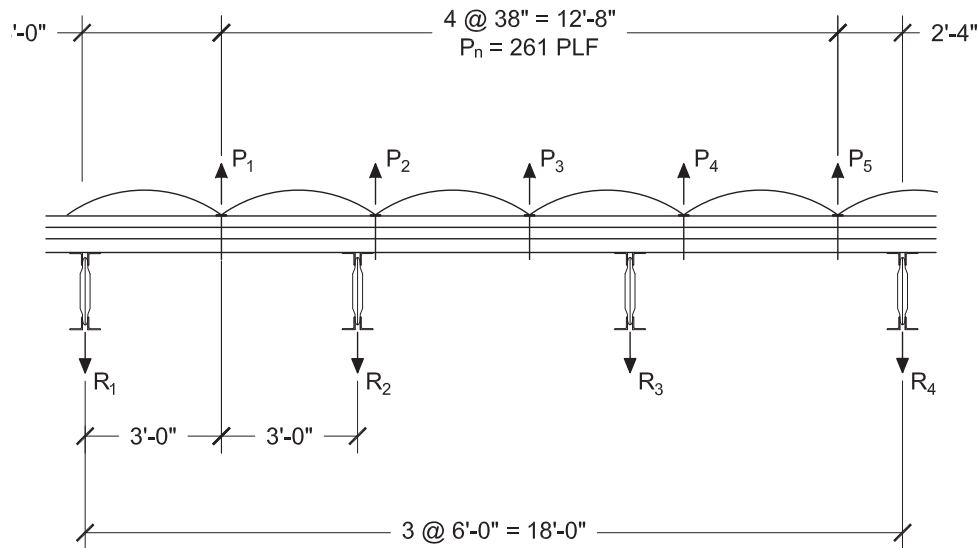


Figure 9.4 Zone 3 Free Body Diagram

Following the recommendation of FM 1-29 Section 2.2.3.2, the first concentrated uplift load is located at mid-span of the first span of the 3-span condition as shown in Figure 9.3. Using indeterminate beam software, the following maximum moment and shear are determined for positive and negative bending.

Maximum positive moment at R_3 :

$$M_{+} = 0.288 \text{ k-ft/ft}$$

$$V = 0.548 \text{ k/ft}$$

Maximum negative moment at P_5 :

$$M_{-} = 0.309 \text{ k-ft/ft}$$

$$V = 0.261 \text{ k/ft}$$

Based on the maximum allowable design moment, the bending strength of the steel roof deck is evaluated considering bending and shear interaction in accordance with AISI S100 Section H2. Combined bending and web crippling is not required to be checked in accordance with the exception in AISI S100 Section H3.

Properties for 20 gage PLB-36:

IAPMO ER-0218

$$M_{a+} = M_{n+} / \Omega = 0.958 \text{ k-ft/ft} / 1.67 = 0.573 \text{ k-ft/ft}$$

$$M_{a-} = M_{n-} / \Omega = 0.988 \text{ k-ft/ft} / 1.67 = 0.592 \text{ k-ft/ft}$$

$$V_{a-} = V_{n-} / \Omega = 5.152 \text{ k/ft} / 1.60 = 3.2 \text{ k/ft}$$

Bending and Shear Interaction:

$$\sqrt{\left(\frac{M}{M_a}\right)^2 + \left(\frac{V}{V_a}\right)^2} \leq 1.0$$

Positive Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.288 \text{ k-ft/ft}}{0.573 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.548 \text{ k/ft}}{3.2 \text{ k/ft}}\right)^2} = 0.53 \leq 1.0$$

Negative Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.309 \text{ k-ft/ft}}{0.592 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.261 \text{ k/ft}}{3.2 \text{ k/ft}}\right)^2} = 0.53 \leq 1.0$$

The bending and shear interaction check for both positive and negative bending are less than the 1.0 limit, therefore the 20 gage PLB-36 steel deck in Zone 3 at a 6 ft span is acceptable with a 38 in wide roof covering attachment spacing.

9.5.3.2 Steel Roof Deck Zone 2 Design

Steel roof deck design for Zone 2 will start with a check using a 57 in wide attachment spacing that meets the wind rating as shown in Figure 9.5.

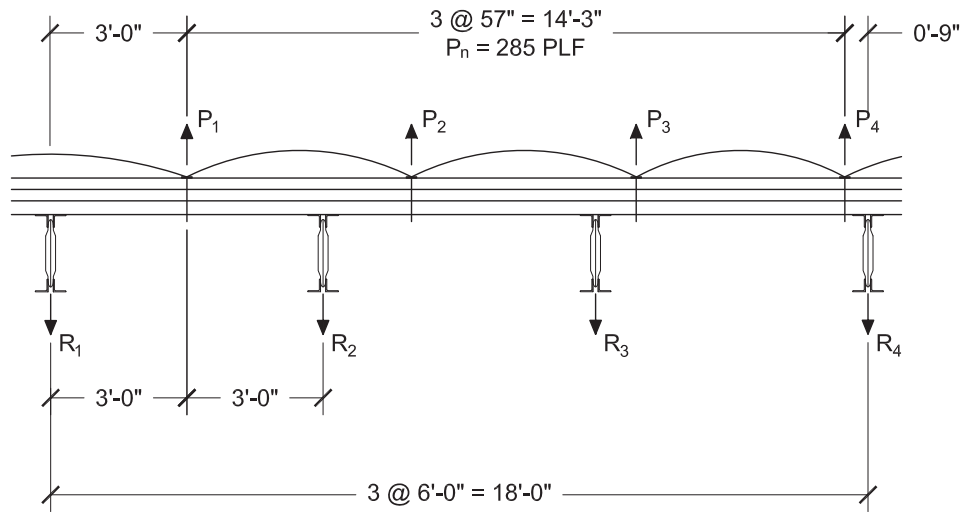


Figure 9.5 Zone 2 Free Body Diagram

Maximum positive moment at R₂:

$$M+ = 0.271 \text{ k-ft/ft}$$

$$V = 0.408 \text{ k/ft}$$

Maximum negative moment at P₁:

$$M- = 0.292 \text{ k-ft/ft}$$

$$V = 0.285 \text{ k/ft}$$

$$M_{a+} = M_{n+} / \Omega = 0.733 \text{ k-ft/ft} / 1.67 = 0.439 \text{ k-ft/ft}$$

$$M_{a-} = M_{n-} / \Omega = 0.783 \text{ k-ft/ft} / 1.67 = 0.469 \text{ k-ft/ft}$$

$$V_{a-} = V_{n-} / \Omega = 4.3 \text{ k/ft} / 1.60 = 2.7 \text{ k/ft}$$

Positive Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.271 \text{ k-ft/ft}}{0.439 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.408 \text{ k/ft}}{2.7 \text{ k/ft}}\right)^2} = 0.64 \leq 1.0$$

Negative Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.292 \text{ k-ft/ft}}{0.469 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.285 \text{ k/ft}}{2.7 \text{ k/ft}}\right)^2} = 0.63 \leq 1.0$$

The bending and shear interaction check for both positive and negative bending are less than the 1.0 limit, therefore the 22 gage PLB-36 steel deck in Zone 2 at a 6 ft span is acceptable with a 57 in wide roof covering attachment spacing.

9.5.2.3 Steel Roof Deck Zone 1 Design

Steel roof deck design for Zone 1 will start with a check using a 114 in wide attachment spacing that meets the wind rating as shown in Figure 9.6.

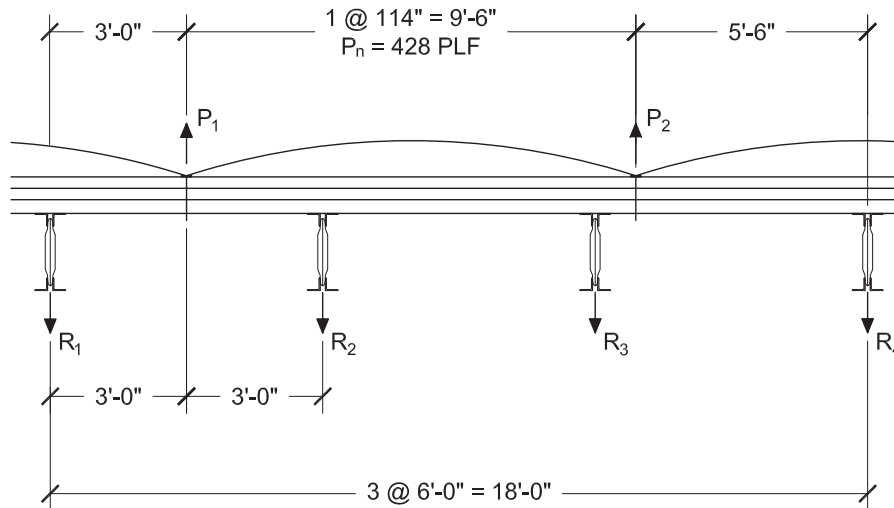


Figure 9.6 Zone 1 Free Body Diagram

Maximum positive moment at R₂:

$$M+ = 0.232 \text{ k-ft/ft}$$

$$V = 0.428 \text{ k/ft}$$

Maximum negative moment at P₁:

$$M- = 0.526 \text{ k-ft/ft}$$

$$V = 0.286 \text{ k/ft}$$

$$M_{a+} = M_{n+} / \Omega = 0.733 \text{ k-ft/ft} / 1.67 = 0.439 \text{ k-ft/ft}$$

$$M_{a-} = M_{n-} / \Omega = 0.783 \text{ k-ft/ft} / 1.67 = 0.469 \text{ k-ft/ft}$$

$$V_{a-} = V_{n-} / \Omega = 4.3 \text{ k/ft} / 1.60 = 2.7 \text{ k/ft}$$

Positive Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.232 \text{ k-ft/ft}}{0.439 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.286 \text{ k/ft}}{2.7 \text{ k/ft}}\right)^2} = 0.54 \leq 1.0$$

Negative Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.526 \text{ k-ft/ft}}{0.469 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.428 \text{ k/ft}}{2.7 \text{ k/ft}}\right)^2} = 1.13 > 1.0$$

The bending and shear interaction check for positive bending is acceptable, however the check for negative bending exceeds 1.0, therefore the 22 gage PLB-36 is not acceptable for Zone 1 with a 114 in wide fastener spacing for a mechanically attached roof covering. For Zone 1 there are two potential alternatives. A roof covering with a narrower attachment spacing, or heavier gage steel roof deck. For this example, we will choose to require that the 57 inch wide attachment spacing for Zone 2 be extended into Zone 1. This increases the distance from the wall from 23 ft to 46 ft, as shown in Figure 4.3. This essentially extends the 1-120 uplift load strength for the roof covering into the 1-90 roof zone, because the deck cannot resist the higher wind uplift concentrated loads with wider roof covering.

9.5.2.4 Steel Roof Deck Zone 1' Design

Steel roof deck design for Zone 1' will start with a check using a 114 in wide attachment spacing that meets the wind rating as shown in Figure 9.7.

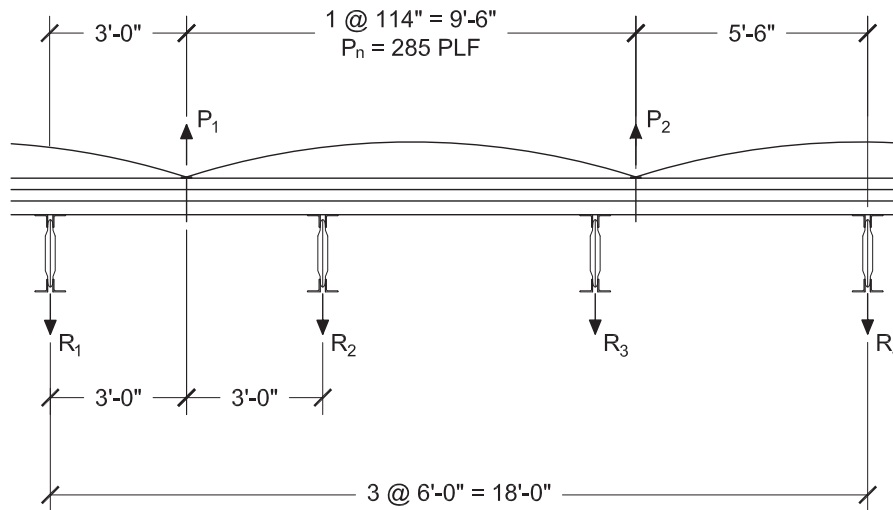


Figure 9.7 Zone 1' Free Body Diagram

Maximum positive moment at R₂:

$$M+ = 0.141 \text{ k-ft/ft}$$

$$V = 0.190 \text{ k/ft}$$

Maximum negative moment at P₁:

$$M^- = 0.321 \text{ k-ft/ft}$$

$$V = 0.261 \text{ k/ft}$$

Properties for 22 gage PLB-36:

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$$M_{a+} = M_{n+} / \Omega = 0.733 \text{ k-ft/ft} / 1.67 = 0.439 \text{ k-ft/ft}$$

$$M_{a-} = M_{n-} / \Omega = 0.783 \text{ k-ft/ft} / 1.67 = 0.469 \text{ k-ft/ft}$$

$$V_{a-} = V_{n-} / \Omega = 4.3 \text{ k/ft} / 1.60 = 2.7 \text{ k/ft}$$

Positive Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.141 \text{ k-ft/ft}}{0.439 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.190 \text{ k/ft}}{2.7 \text{ k/ft}}\right)^2} = 0.33 \leq 1.0$$

Negative Bending and Shear Interaction:

$$\sqrt{\left(\frac{0.321 \text{ k-ft/ft}}{0.469 \text{ k-ft/ft}}\right)^2 + \left(\frac{0.261 \text{ k/ft}}{2.7 \text{ k/ft}}\right)^2} = 0.73 \leq 1.0$$

The bending and shear interaction check for both positive and negative bending are less than the 1.0 limit, therefore the 22 gage PLB-36 steel deck in Zone 1' at a 6 ft span with 114 in wide roof covering is acceptable.

Based on the concentrated load analysis for the PLB-36 steel roof deck, the following summarizes the maximum row spacing for mechanically attached roof coverings that are allowed based on the steel roof deck strength. This information needs to be conveyed in the project design drawing or specifications for use by the roof contractor in selecting the roof covering widths.

| Zone | PLB-36 Gage | Rating | Row Spacing (in) | Service Level Uplift | |
|------|-------------|--------|------------------|----------------------|-------|
| | | | | (psf) | (plf) |
| 1' | 22 | 1-60 | 114 | 30.0 | 285 |
| 1 | 22 | 1-90 | 57 | 45.0 | 428 |
| 2 | 22 | 1-120 | 57 | 60.0 | 285 |
| 3 | 20 | 1-165 | 38 | 82.5 | 261 |

Table 9.5 Design Maximum Row Spacing for Mechanically Attached Roof Covering with 6 ft Deck Spans

9.6 FM Steel Deck Securement to Support Framing

The securement or connection of the steel roof deck to the support framing is addressed in FM 1-29 Section 2.2.3.4. This section provides for two methods. The first method in Section 2.2.3.4.A is the performance-based approach using RoofNav to determine the type and connection spacing for the steel roof deck to supports. This requires a specific roof membrane to be selected which may not be known at the time the structural design is being performed because the specifications likely allow for several manufacturers of roof covering to ensure a competitive bid. This leads the design team to the use of the second prescriptive method in Section 2.2.3.4.B. This method provides for a prescriptive increase in the attachment of the steel roof deck to the supports based on the attachment required in Zone 1.

The first step is to determine the connection spacing for the steel roof deck to supports for Zone 1. This project uses Hilti X-HSN 24 power actuated fasteners. The attachment spacing for Hilti X-HSN 24 fasteners for 1-60, 1-75, and 1-90 ratings can be found by searching for Hilti in RoofNav and selecting the X-HSN 24 from the results. Figure 9.8 provides an extract from this document for the X-HSN 24 in 1/8" and thicker support steel.

The Approved maximum allowable steel deck spans used with X-HSN 24 steel deck fasteners with 33 ksi (227 MPa) or 80 ksi (552 MPa) steel deck when attached to structural steel supporting members in the thickness range of greater than or equal to 0.125 in. (3.2 mm) but less than 0.375 in. (9.5 mm):

| X-HSN 24 | Fastener O.C. Spacing | Rating | MAX SPANS [INCHES] | | | | | | | |
|----------|-----------------------|--------|--------------------|--------|--------|--------|--------|--------|--------|--------|
| | | | 22 ga | | 20 ga | | 18 ga | | 16 ga | |
| | | | 33 ksi | 80 ksi | 33 ksi | 80 ksi | 33 ksi | 80 ksi | 33 ksi | 80 ksi |
| 6 | 60 | 60 | 262 | 478 | 322 | 800 | 437 | 800 | 800 | 800 |
| | | 75 | 209 | 382 | 258 | 640 | 350 | 640 | 640 | 640 |
| | | 90 | 174 | 318 | 215 | 533 | 291 | 533 | 533 | 533 |
| 8 | 60 | 60 | 196 | 358 | 241 | 600 | 328 | 600 | 600 | 600 |
| | | 75 | 157 | 286 | 193 | 480 | 262 | 480 | 480 | 480 |
| | | 90 | 131 | 239 | 161 | 400 | 218 | 400 | 400 | 400 |
| 12 | 60 | 60 | 131 | 239 | 161 | 400 | 218 | 400 | 400 | 400 |
| | | 75 | 104 | 191 | 129 | 320 | 175 | 320 | 320 | 320 |
| | | 90 | 87 | 159 | 107 | 266 | 145 | 266 | 266 | 266 |

Figure 9.8 FM Approved Maximum Span for Hilti X-HSN 24 PAFs

Deck sidelaps need to be secured using FM approved fasteners. The maximum spacing for Verco PunchLok sidelap connections from RoofNav are shown in Figure 9.9. FM 1-29 Table 4, shown in Figure 9.10, provides the maximum side lap securement spacing based on roof Zone and wind uplift ratings.

Steel Deck Side Lap Requirements

Type PLB-36 steel deck side laps are secured using the Verco PunchLok tool spaced at maximum 36 in. (914 mm) o.c.

Figure 9.9 PLB-36 Steel Deck Maximum Sidelap VSC2 Attachment with PunchLok II Tool

Table 4. Maximum Side Lap Securement Spacing

| <i>Zone 1 RoofNav Rating (psf)</i> | <i>Zone 1, in. (mm)</i> | <i>Zone 2, in. (mm)</i> | <i>Zone 3, in. (mm)</i> |
|------------------------------------|-------------------------|-------------------------|-------------------------|
| < = 90 | 36 (914) | 30 (762) | 30 (762) |
| 105 - 120 | 30 (762) | 15 (381) | 15 (381) |
| > = 135 | 24 (610) | 15 (381) | 15 (381) |

Figure 9.10 FM 1-29 Sidelap Spacing

The resulting steel roof deck schedule based on the roof covering width, strength of the steel deck and securement of the steel deck is presented in Table 9.6.

| Zone | PLB-36 Gage | Rating | Hilti X-HSN 24 | | Maximum VSC2 Spacing (in) |
|------|-------------|--------|----------------|------------------------|---------------------------|
| | | | Spacing (in) | Maximum Deck Span (in) | |
| 1' | 22 | 1-60 | 12 | 131 | 36 |
| 1 | 22 | 1-90 | 12 | 87 | 36 |
| 2 | 22 | 1-120 | 6 | n/a | 15 |
| 3 | 20 | 1-165 | 2 per rib at 6 | n/a | 15 |

Table 9.6 PLB-36 Securement with Hilti X-HSN 24 Fasteners

FM 1-29 provides guidance on interpreting the FM fastener spacing with SDI standard attachment patterns in Section 2.2.3.8. A 36/4 pattern meets the 12 in spacing requirement, and the 36/7 pattern meets the 6 in spacing requirement. FM does not address the 36/14 pattern which provides 2 fastener per rib meeting the Zone 3 requirements for this project. Based on the increased attachment pattern to meet FM 1-29 requirements, additional roof attachment zones need to be added to the steel roof deck attachment schedule for seismic diaphragm strength.

9.7 FM Steel Roof Deck Schedule

The steel roof deck schedule for this example in a high seismic area will require greater attachment of the steel roof deck around the perimeter to meet FM wind uplift requirements. This is not needed to meet the components and cladding wind uplift requirements of ASCE 7 developed in Section 4. To achieve this, the FM wind zone roof deck schedule is overlaid on the seismic roof deck schedule diagram. The result is the need to add 3 additional zones to the steel roof deck schedule as shown in Figure 9.11. These account for the empirical increase in the minimum Hilti X-HSN 24 fasteners to supports and increased PunchLok II VSC2 connections around the perimeter. The steel roof deck schedule in Table 9.7 reflects the additional 3 zones.

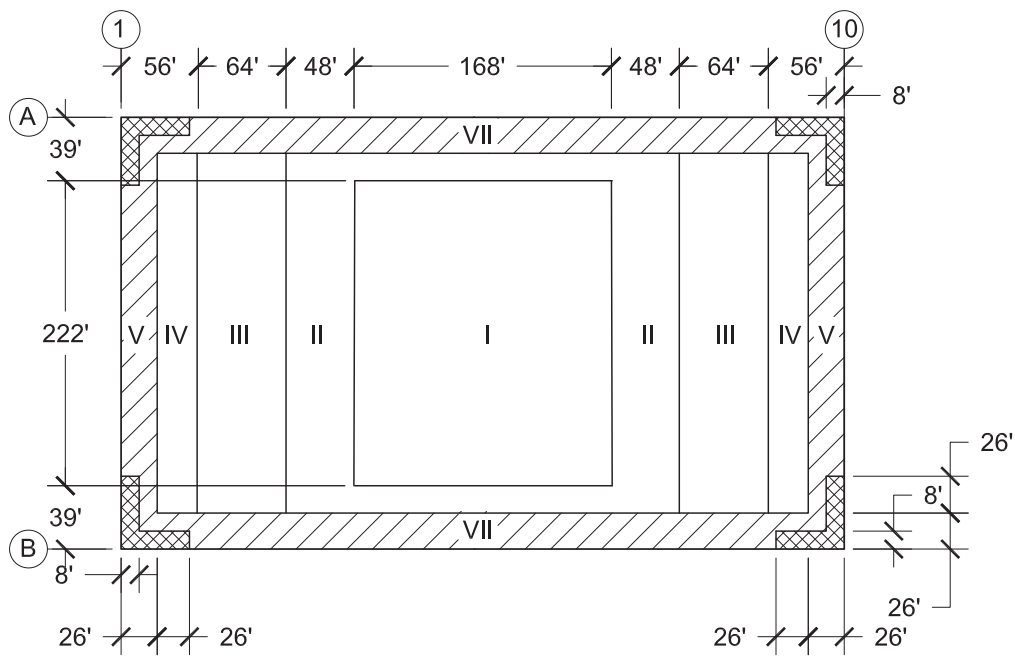


Figure 9.11 FM Wind Zone Attachment Overlay on Diaphragm Shear Zones

| Zone | PLB-36 Gage | Span (ft) | Fastener to Supports | | | Sidelap connection VSC2 (in o.c.) | Wind Uplift Zone(s) |
|------|-------------|-----------|----------------------|----------|------------------------------------------|-----------------------------------|---------------------|
| | | | Pattern | Type | Parallel chords and collectors (in o.c.) | | |
| VII | 22 | 6 | 36/7 | X-HSN 24 | 8 | 15 | 2 |
| VI | 20 | 6 | 36/14 | X-HSN 24 | 7 | 15 | 3 |
| V | 20 | 6 | 36/7 | X-HSN 24 | 7 | 15 | 2 |
| IV | 20 | 6 | 36/7/4 | X-HSN 24 | 7 | 8 | 1 & 2 |
| III | 22 | 6 | 36/7/4 | X-HSN 24 | 8 | 8 | 1 & 1' |
| II | 22 | 6 | 36/7/4 | X-HSN 24 | 12 | 18 | 1 & 1' |
| I | 22 | 6 | 36/4 | X-HSN 24 | 16 | 24 | 1' |

Table 9.7 Deck Attachment Schedule with FM Wind Requirements

10.0 Very Large Warehouse Structures

Steel roof deck is an ideal solution for very large warehouse structures with proven scalability from a modest 100,000 square foot warehouse to over 2,000,000 square foot distribution centers. These very large warehouse or manufacturing structures come with their own unique set of challenges. Large roof structures must resist large forces. The gravity and vertical wind loads see little if any change, however the seismic and wind lateral loads grow quickly as the length of the diaphragm grows. In addition to the gravity, seismic, and wind loads that govern small to modest warehouse structures, very large structures require that self-straining forces due to thermal expansions and contraction need to be considered.

A roof structure utilizing Verco steel roof deck combined with Vulcraft joists and joist girders can meet all these design challenges. The most efficient roof structure for a very large warehouse is a single diaphragm as shown in Figure 10.1. One large diaphragm generates very high diaphragm shear forces. The Verco PunchLok II system has the high diaphragm shear strength to meet the shear demands of large structures with a single large diaphragm. This system also has high diaphragm shear stiffness to limit the maximum in-plane diaphragm deflection to ensure structural stability and serviceability of the structure.

The high strength single diaphragm must also be able to mitigate thermal expansion of the very large steel roof structure. The inherent ductility of the Verco PunchLok II system in combination with Hilti power actuated connections to Vulcraft steel joists, relieves the stress build up due to thermal expansion or contraction. This unique system developed to dissipate seismic energy, also has the benefit of relieving the accumulating thermal stress through the ductility of the steel deck, connections, and supporting joists. Reverse cyclic testing provides the basis for the development of this system utilizing the Verco PunchLok II system in conjunction with Hilti power actuated fasteners to Vulcraft joists, details of which are presented in IAPMO ER-2018. Figure 10.6 provides an extract of these provisions.

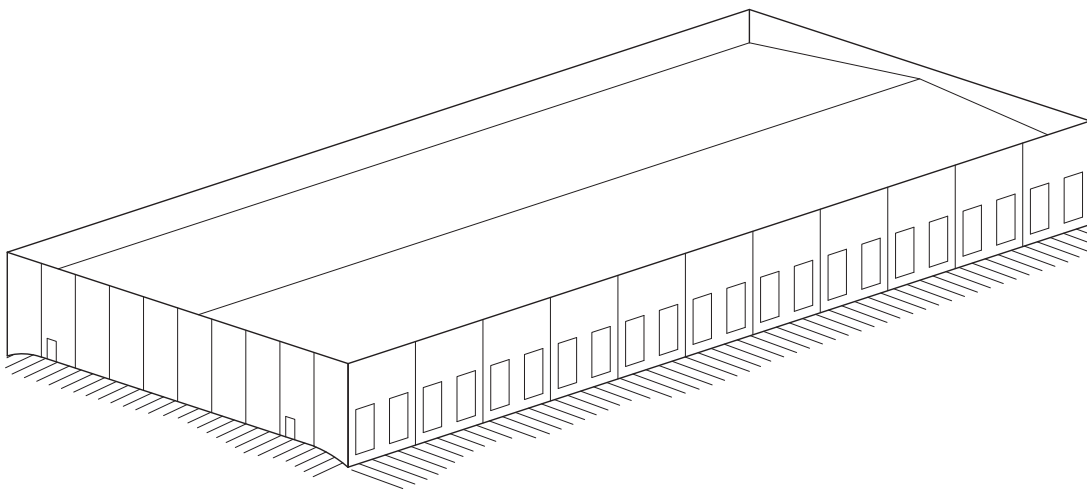


Figure 10.1 Single Diaphragm Large Warehouse Structure

The design methods presented for the 300 ft by 504 ft modest sized warehouse in the preceding Sections of this example provide the basic methods to explore the design for a large 770,000 sf warehouse. This structure is 1288 ft long by 600 ft deep with the same 56 ft by 50 ft bay size, as shown in Figure 10.2 and cross-section in Figure 10.3.

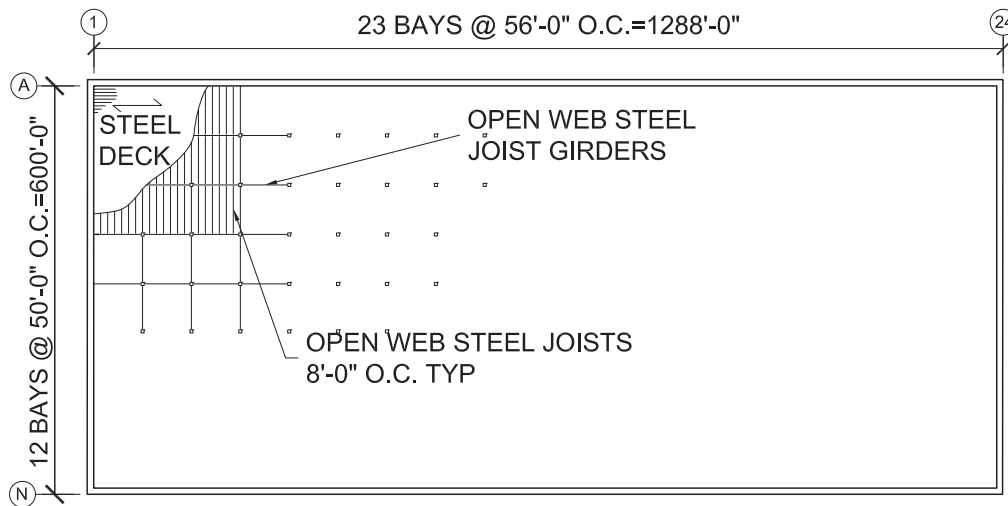


Figure 10.2 Large Warehouse Plan View

The roof elevation at the ridge increases from 34 ft for the small warehouse describe in Section 2 to 40 ft due to the increased width and maintaining a roof slope of approximately 1/4 in per foot for roof drainage as shown in Figure 10.3.

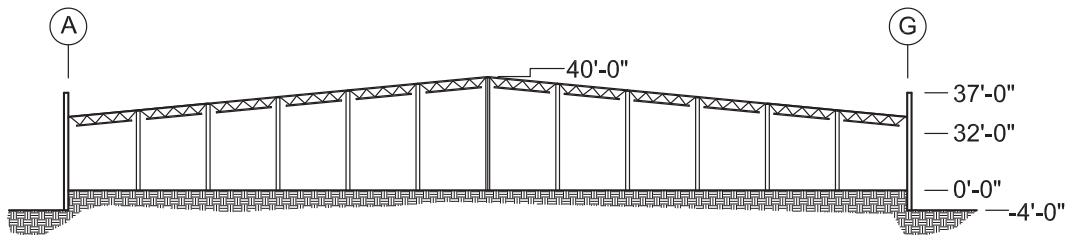


Figure 10.3 Large Warehouse Cross Section

The design parameters are the same for this larger warehouse structure in the same location as the smaller structure developed in Sections 2 through 5. Key to this is that the increased height of the structure is still below the limits for a warehouse of unlimited square footage described in Section 2. The vertical loads for the roof structure with a 56 ft by 50 ft bay size is also the same as those for the smaller structure. The development of the large diaphragm will focus on higher seismic force related to the increased size of the roof diaphragm structure and the considerations for thermal expansion.

10.1 Thermal Expansion

A primary consideration for large steel roof structures is thermal expansion. Industry guidance from NAS Report No. 65 Expansion Joints in Buildings¹ leads to guidance for a maximum recommended steel roof structure length between 300 ft to 450 ft. This report investigated one concrete moment frame building system and extrapolated results to other building types. The primary outcome of this report was that more research is needed.

These recommended maximum building lengths in the Expansion Joint in Buildings Report are based on construction practices in place in 1974. Steel roof deck structures at this time would be constructed with hot rolled steel members or open web steel joists. The steel deck would be attached to the supporting member with low ductility arc spot welds. The stiffness of the arc spot weld would not relieve any thermal movement, therefore stresses and movement due to thermal expansion would quickly grow to unsustainable levels. This recommended maximum length is considerably less than the 600 ft width or the 1288 ft length of the large warehouse single diaphragm roof structure under consideration in this example.

The traditional approach to mitigate thermal expansion is to divide the steel roof structure into a series of smaller diaphragms that do not exceed the recommended length, as shown in Figure 10.4. This is a reasonable solution in that the steel roof structure is within the recommended limits and the tilt-up walls with panel joints every 30 to 35 ft do not accumulate significant thermal movement.

Footnote: (1) 1974 the Building Research Advisory Board of the National Academy of Sciences (NAS) published Federal Construction Council Technical Report No. 65 "Expansion Joints in Buildings" (NAS, 1974)

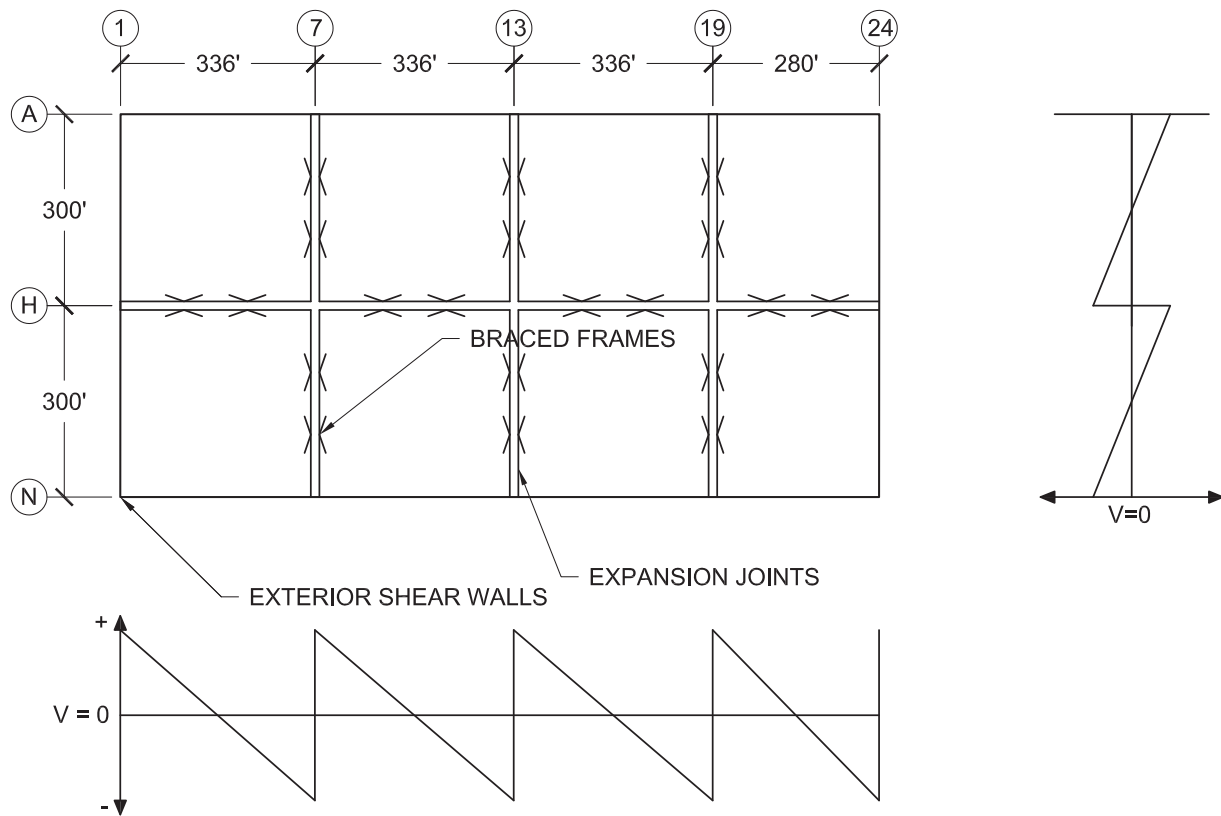


Figure 10.4 Large Roof Structure with Thermal Expansion Joints

A large roof structure separated into 8 separate diaphragms for thermal expansion creates a series of design issues that need to be solved that are not necessary for a single large diaphragm. For a structure in seismic areas, the expansion joints not only have to account for the thermal expansion but also need to be wide enough to account for seismic separation between the buildings as shown in Figure 10.5. This is necessary because when the building is separated into 8 individual diaphragms, each diaphragm, and associated vertical lateral force resisting system becomes a separate building, even though the roof membrane and exterior walls will be continuous. Some type of lateral force resisting system is needed at each of the diaphragm sides to provide lateral support. In this example, braced frames are provided for the lateral force resisting system in the interior of the structure and the concrete tilt-up wall panels provide the lateral force resisting system around the building perimeter.

At the expansion joints, a double row of columns and braced frames are a good solution. In a high seismic location it is necessary to design a slip joint which only requires a few inches of thermal expansion, to

accommodate a foot or more of slip needed for the seismic separation across the expansion joint. Figure 10.5b shows a representative thermal expansion joint suitable for wind governed roof structures, that generally does not have an adequate range of motion to provide the building separation required for seismic events.

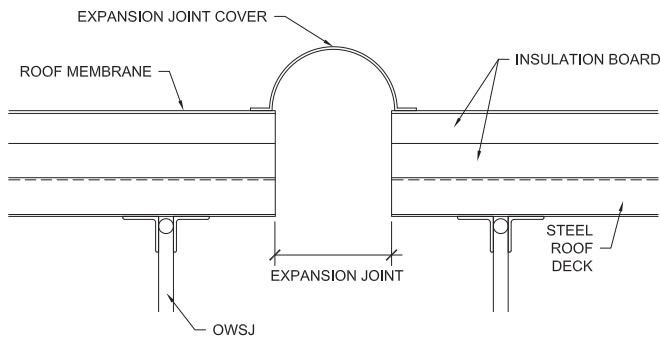


Figure 10.5a Expansion Joint

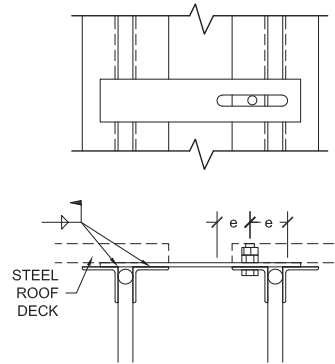


Figure 10.5b Slip Expansion Joint

Expansion joints and the associated braced frames add to the cost of the structure. The increased complexity drives the design cost up for both the engineer and the architect due to the need to generate the appropriate details to provide for the structural design and weather tightness of the roof system. Costs extend beyond the design into the increased construction costs to provide the double braced frame lines and associated footings, in lieu of the single row of gravity columns and OWSJ, along the expansion joint lines. The client that will occupy the structure also must plan for usage of the space around the interior braced frames rather than having a wide-open space without obstructions. There may also be future cost due to maintenance of the roof membrane expansion joint covers that may fail prior to the field of the roof. This can all be eliminated with a ductile single diaphragm structure that can self-relieve the accumulations of thermal expansion forces.

10.2 Large Single Diaphragm Structure Design

The key to a successful large single diaphragm roof structure is to mitigate the build up of thermal expansion stresses, meet the design loads, and satisfy building stability requirements. Verco steel roof deck using the PunchLok II system in combination with Hilti power actuated fasteners and Vulcraft open web steel joists meets each of these challenging design considerations. Each of these considerations will be addressed in this example.

10.2.1 Thermal Expansion

The 1288 ft by 600 ft single diaphragm is much larger than the industry recommended length for thermal expansion of non-ductile systems. The Verco PunchLok II system in combination with Hilti power actuated fasteners and Vulcraft open web steel joists provide the ductility needed as verified in IAPMO ER-2018. Figure 10.6 provides an extract of the requirements for unlimited length diaphragms from IAPMO ER-2018.

Reviewing the requirements for unlimited length diaphragms for thermal expansion:

The length, L , and width, b , of the Steel Roof Deck Diaphragm support members, chords and collectors are permitted to be unlimited between joints for differential thermal expansion or contraction (thermal expansion joints) provided the following conditions exist:

- a. Vertical Load system, including the vertical lateral force resisting system, does not have Building Separation Joints, Seismic Joints, Expansion joints or similar joints with respect to the area of the diaphragm under consideration.

There are no expansions joints in the vertical lateral force resisting system of this large diaphragm example therefore the diaphragm is not interrupted, and this requirement is met.

b. Wall systems, both perimeter and interior, to be constructed as continuous walls, individual wall panels, or wall segments. Individual wall panels or wall segments are permitted to be precast concrete walls, site cast concrete walls, tilt-up concrete walls, masonry walls with or without crack control joints, stud wall system with or without crack control joints, or wind girt framed wall systems with or without crack control joints. A combination of these systems may be used. See Figures A and B (in Figure 10.6).

This example is an on-site precast concrete tilt up structure that meets the requirements for the vertical lateral force resisting system.

c. Diaphragm has continuous chord members and a positive load path capable of transferring diaphragm forces between the diaphragm and the Vertical Lateral Force Resisting System.

The diaphragm will have continuous chords around the entire perimeter that connect the diaphragm to the shear walls, meeting this requirement. This is comprised of bent plate channel ledgers on Lines A and N, and an angle ledger on Lines 1 and 24. The chords will need to be attached to the wall panels with studs or anchor bolts for shear transfer and wall anchorage.

d. Steel Roof Deck Diaphragm is covered by a roofing membrane system; deck is not directly exposed to sun or elements in final occupied condition.

This warehouse will have a roof membrane and insulation board meeting this requirement.

e. Steel roof deck support members consists of one or a combination of the following.

1. Vulcraft Open Web Steel Joists and Joist Girders in accordance with SJI-100.
2. Structural Steel members in accordance with AISC 360.
3. Cold-Formed Steel members in accordance with AISI S100.
4. Bearing walls.

To meet this requirement the steel roof deck will be supported by Vulcraft open web steel joists and joist girders. The joist chords will need to be either steel in accordance with AISC 360 or cold-formed steel in accordance designed in accordance with AISI S100. Vulcraft OWSJ and OWSJG meet this requirement.

f. Steel Roof Deck Diaphragm Attachment:

1. Sidelap Connection: PunchLok II system with VSC2 sidelap connections.
2. Connection to Supports: Hilti X-HSN 24 or X-ENP19 Power Actuated Fasteners.

Verco steel roof deck with the PunchLok II system VSC2 sidelap connections and Hilti X-HSN 24 or X-ENP-19 power actuated fasteners must be specified to meet this requirement. This is the most important requirement to ensure the ductility of the diaphragm to relieve stresses due to thermal expansion.

STEEL ROOF DECK DIAPHRAGM LENGTH FOR DIFFERENTIAL THERMAL EFFECTS

The length, L , and width, b , of the roof diaphragm, in accordance with the definition sections of the IBC or ASCE 7, including the steel roof deck, support members (framing), chords and collectors shall be permitted to be of unlimited length between joints for differential thermal expansion or contraction (thermal expansion joints) provided the following conditions exist:

- a. Vertical Load system, including the vertical lateral force resisting system, does not have Building Separation Joints, Seismic Joints, Expansion joints or similar joints that interrupt the diaphragm chord with respect to the area of the diaphragm under consideration.
- b. Wall systems, both perimeter and interior, to be constructed as continuous walls, individual wall panels, or wall segments. Individual wall panels or wall segments are permitted to be precast concrete walls, site cast concrete walls, tilt-up concrete walls, masonry walls with or without crack control joints, stud wall system with or without crack control joints, or wind girt framed wall systems with or without crack control joints. A combination of these systems may be used. See Figures A and B.
- c. Diaphragm has continuous chord members and a positive load path capable of transferring diaphragm forces between the diaphragm and the Vertical Lateral Force Resisting System.
- d. Steel Roof Deck Diaphragm is covered by a roofing membrane system; deck is not directly exposed to sun or elements in final occupied condition.
- e. Steel roof deck support members consists of one or a combination of the following.
 1. Vulcraft Open Web Steel Joists and Joist Girders in accordance with SJI-100.
 2. Structural Steel members in accordance with AISC 360.
 3. Cold-Formed Steel members in accordance with AISI S100.
 4. Bearing walls.
- f. Steel Roof Deck Diaphragm Attachment:
 1. Sidelap Connection: PunchLok II System with VSC2 sidelap connections.
 2. Connection to Supports: Hilti X-HSN 24 or X-ENP-19 Power Actuated Fasteners.

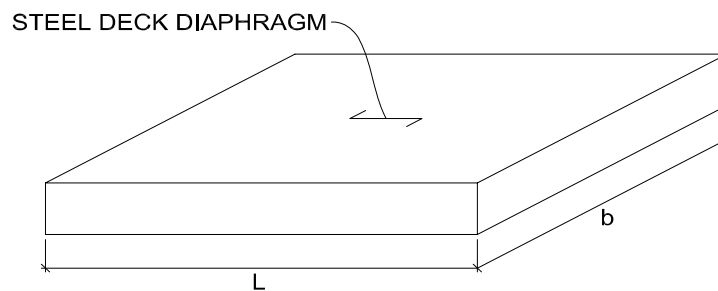


Figure A: Structure with Continuous Walls

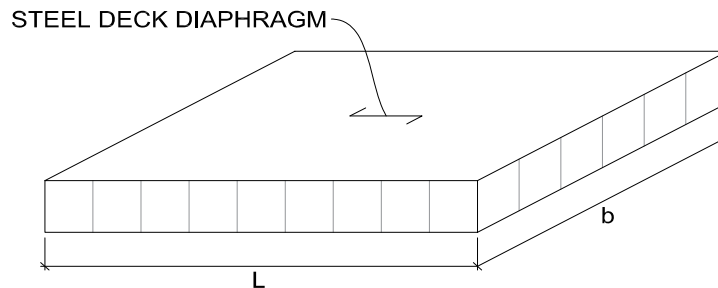


Figure B: Structure with Individual Wall Panels or Wall Segments

Figure 10.6 Extract of Unlimited Length Diaphragms from IAPMO ER-0218

10.2.2 Large Diaphragm Shear Strength

For this large roof structure, the weight of the roof structure, in addition to the North and South walls, is the same as that developed in Section 5.3. The East and West walls are higher for this large roof structure than those developed in Section 5, and the higher shear forces may lead to thicker walls. In this example we will only address the more severe North-South loading on the diaphragm. The East-West analysis will be similar to that in Section 5.

The factored roof diaphragm shear loads based on the tributary mass of the roof structure and walls in the North-South Direction:

$$F_{px} = w_{N-S} = 0.25[15 \text{ psf}(600 \text{ ft}) + 2[116 \text{ psf}(37 \text{ ft})(37 \text{ ft}/2)(1/32 \text{ ft})]] = 3487 \text{ plf}$$

Diaphragm Reactions at East and West walls:

$$V_{N-S1} = V_{N-S24} = w_{N-S}(L_{N-S}/2) = 3487 \text{ plf}(1288 \text{ ft}/2) = 2246 \text{ kips}$$

Diaphragm Shear at East-West walls:

$$v_{N-S} = V / b$$

$$v_{N-S} = 2246 \text{ kips} / 600 \text{ ft} = 3742 \text{ plf}$$

Figure 10.7 shows the diaphragm load, diaphragm shear, and end reactions for both directions. Based on this shear distribution, the PLB-36 steel roof deck gage and attachment pattern are selected as presented in Table 10.1. This follows the same methods in Section 5.

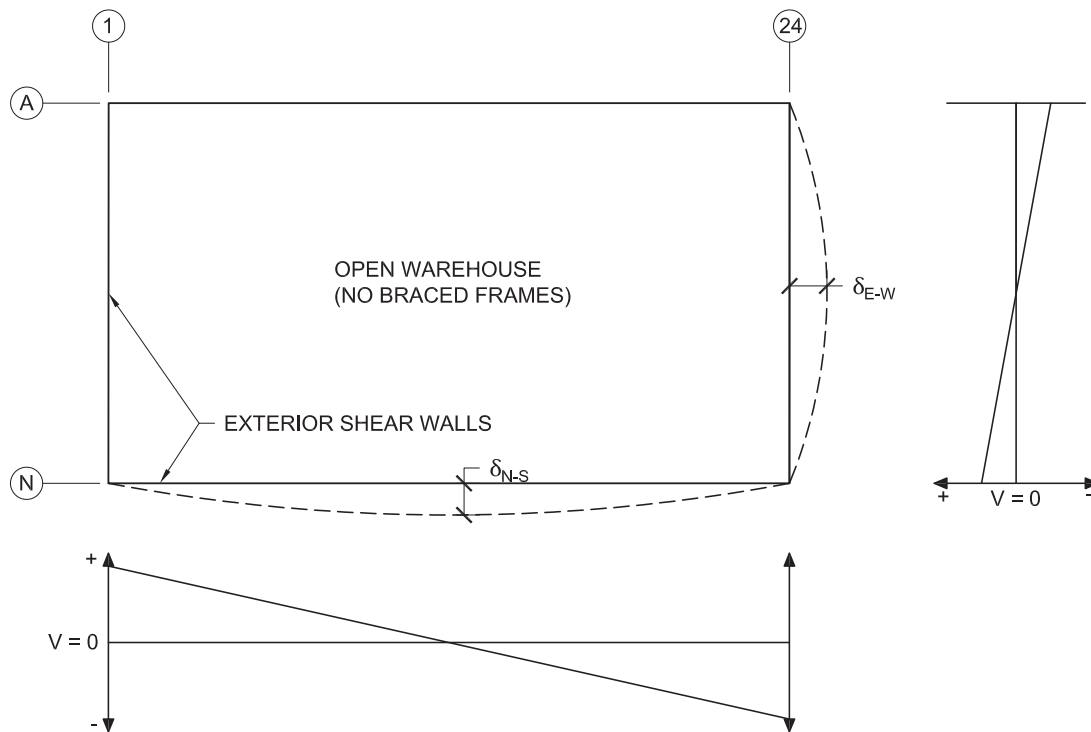


Figure 10.7 Unobstructed Large Roof Structure with No Interior Braced Frames

The high strength of the Vercor PunchLok II system in combination with the Hilti X-HSN 24 power actuated fasteners exceeds the maximum required diaphragm design strength of 3742 plf for this large single diaphragm structure with 18 gage deck along the East and West walls as shown in Table 10.1. For an efficient design, the attachment pattern reduces from a heavy 36/14/4 pattern at the walls to a light 36/4 pattern in the center. The steel roof deck gage is also reduced from a heavy 18 gage deck along the walls to a light 22 gage deck in the center of the diaphragm where the shear requirement is low.

| Zone | PLB Gage | Span (ft) | Fastener to Supports | | | Sidelap connection VSC2 (in o.c.) | Required Shear LRFD (plf) | Design Shear Strength ϕV_n (plf) |
|------|----------|-----------|----------------------|----------|------------------------------------------|-----------------------------------|---------------------------|----------------------------------------|
| | | | Pattern | Pin | Parallel chords and collectors (in o.c.) | | | |
| VI | 18 | 8 | 36/14/4 | X-HSN 24 | 6 | 8 | 3742 | 3820 |
| V | 18 | 8 | 36/7/4 | X-HSN 24 | 7 | 8 | 3045 | 3090 |
| IV | 20 | 8 | 36/7/4 | X-HSN 24 | 7 | 8 | 2162 | 2206 |
| III | 22 | 8 | 36/7/4 | X-HSN 24 | 8 | 8 | 1697 | 1729 |
| II | 22 | 36/7/4 | 36/7/4 | X-HSN 24 | 12 | 18 | 1139 | 1168 |
| I | 22 | 36/4 | 36/4 | X-HSN 24 | 16 | 24 | 767 | 770 |

Table 10.1 Large Diaphragm Shear Zones

Single Large Diaphragm Strength Comparison with Hybrid Panelized Wood Roof Structures

Hybrid panelized wood roof structures are a common roof diaphragm system for large warehouse structures in the Western U.S. The maximum design shear strength for a blocked high shear diaphragm for 23/32 Structural I sheathing is 2880 plf (1800 plf ASD) and is a much less stiff diaphragm, $G_a = 34$ kip/in. This is a much more costly design than the typical hybrid design for economy that limits the shear to 2232 plf (1395 plf ASD) for 15/32 Structural I sheathing. It is not possible to provide clients with an open interior without interior braced frames or shear walls that will divide the single diaphragm into two or more diaphragms with lower design shear requirements. The use of steel deck roof diaphragms provides your clients a larger open warehouse with fewer interior obstructions from braced frames or shear walls when planning their interior space.

10.2.3 Large Diaphragm Shear Deflection and P-Delta Check

Having adequate design strength to resist the seismic load does not ensure that the roof structure will adequately brace the warehouse building. A large diaphragm with high shear loads may potentially have large in-plane diaphragm deflections. Excessive deflection can lead to building stability and deformation compatibility issues. The deflection will be determined and then used for a P-Delta check to ensure building stability and deformation compatibility.

The shear deflection for the very larger single diaphragm roof structure is presented in Table 10.2.

| Zone | V_{left} (lb/ft) | V_{right} (lb/ft) | $V_{i\ ave}$ (lb/ft) | L_i (ft) | G' (kip/in) | $0.5(v_{i\ ave} L_i)/(1000G')$ (in) |
|-----------------------------------------------------------|-----------------------|------------------------|-------------------------|---------------|------------------|----------------------------------------|
| VI | 3742 | 3045 | 3393 | 120 | 279 | 0.73 |
| V | 3045 | 2162 | 2603 | 152 | 274 | 0.72 |
| IV | 2162 | 1697 | 1929 | 80 | 191 | 0.40 |
| III | 1697 | 1139 | 1418 | 96 | 150 | 0.45 |
| II | 1139 | 767 | 953 | 64 | 117 | 0.26 |
| I | 767 | 0 | 383 | 132 | 96 | 0.26 |
| I | 0 | 767 | 383 | 132 | 96 | 0.26 |
| II | 767 | 1139 | 953 | 64 | 117 | 0.26 |
| III | 1139 | 1697 | 1418 | 96 | 150 | 0.45 |
| IV | 1697 | 2162 | 1929 | 80 | 191 | 0.40 |
| V | 2162 | 3045 | 2603 | 152 | 274 | 0.72 |
| VI | 3045 | 3742 | 3393 | 120 | 279 | 0.73 |
| Total Elastic Strength Level Shear Deflection, δ_v | | | | | | 5.67 |

Table 10.2 Large Diaphragm Deflection

Using the same method presented in Section 5.7, the inelastic diaphragm deflection is determined from the unfactored strength-based diaphragm load (1.0E). The 5.67 inches of deflection calculated above is the strength-based elastic deflection. The maximum diaphragm deflection is based on the ASCE 7 Section 12.8.6 Story Drift Determination using Equation 12.8-15.

$$\delta_x = C_d \delta_{xe} / I_e$$

ASCE 7 eq 12.8-15

$$C_d = 4$$

ASCE 7 Table 12.2-1 (A5)

$$I_e = 1.0$$

see Section 4

$$\delta_{xe} = \delta_s = 5.67 \text{ in}$$

$$\delta_x = 4.0(5.67 \text{ in})/1.0 = 22.7 \text{ in}$$

The application of the $P\Delta$ provisions of the ASCE 7 Section 12.8.7 may be applied to this system as a guide to investigate structural stability as follows.

Stability coefficient equation:

$$\theta = P_x \Delta I_e / (V_x h_{sx} C_d) \leq 0.10$$

ASCE 7 eq 12.8-16

Overstrength coefficient:

$$C_d = 4.0$$

ASCE 7 Table 12.2-1 (A5)

Roof level height (conservatively use low point on the roof):

$$h_{sx} = 32 \text{ ft (12 in/ft)} = 384 \text{ in}$$

Importance Factor:

$$I_e = 1.0$$

see Section 4

The total unfactored vertical design load tributary to the roof Level:

$$P_{x \text{ roof}} = 15 \text{ psf}(1288 \text{ ft})(600 \text{ ft})(1 \text{ kip}/1000 \text{ lb}) = 11592 \text{ kip}$$

$$P_{x \text{ wall}} = 116 \text{ psf}(32 \text{ ft}/2 + 5 \text{ ft})(1288 \text{ ft})(2 \text{ walls}) = 6275 \text{ kip}$$

$$P_x = 11592 \text{ kip} + 6275 \text{ kip} = 17867 \text{ kip}$$

The unfactored seismic shear force acting at the roof level:

$$V_x = 3742 \text{ plf} (600 \text{ ft}) + 3742 \text{ plf} (600 \text{ ft}) = 2246 \text{ kips} + 2246 \text{ kips} = 4492 \text{ kips}$$

The average movement of the center of mass of the roof structure based on the estimated maximum strength level roof diaphragm deflection is estimated based on a parabolic deflected shape. For a parabolic curve, the center of mass is at 2/3 of the height or in this case, 2/3 of the in-plane deflection.

$$\Delta = (2/3)\delta_x = (2/3)(22.7 \text{ in}) = 15.1 \text{ in}$$

$$\theta = P_x \Delta / (V_x h_{sx} C_d) = 17867 \text{ kip}(15.1 \text{ in})(1.0) / [4492 \text{ kip}(384 \text{ in})(4.0)] = 0.04 \leq 0.10$$

The predicted stability coefficient is much less than the 10% limit, therefore no further investigation of the stability $P\Delta$ effects is required for this structure.

Single Large Diaphragm Deflection Comparison with Hybrid Panelized Wood Roof Structures

The required diaphragm shear for this large single diaphragm roof structure exceeds the strength of any plywood option for a hybrid panelized wood roof structure. The amplified maximum deflection for this steel roof deck diaphragm is 22.7 inches, which is half of the deflection of the hybrid panelized roof structure of 42.4 inches for the small 500 ft by 300 ft discussed at the end of Section 5.7. Designers that are comfortable with large amplified maximum deflection of small to mid-sized hybrid wood roof structures should have very little concern over the modest deflection of this large single diaphragm steel roof deck structure.

10.3 High Shear Large Diaphragms with Interior Brace Frames

The elimination of thermal expansion joints due to the ductility of the Verco PunchLok II system and Hilti power actuated fasteners does not preclude the use of interior braced frames when desirable. There are many common situations that will require vertical lateral force resisting systems in locations in addition to the end walls. These include longer length diaphragms in which the diaphragm shear exceeds the strength of available systems, or when diaphragm deflection needs to be limited for stability or the serviceability of the structure. In this section we will split the large single diaphragm investigated in Section 10.2 in half to demonstrate the difference in the design as shown in Figure 10.8. This concept can be used to divide diaphragm into as many segments as required without the need for expansion joints.

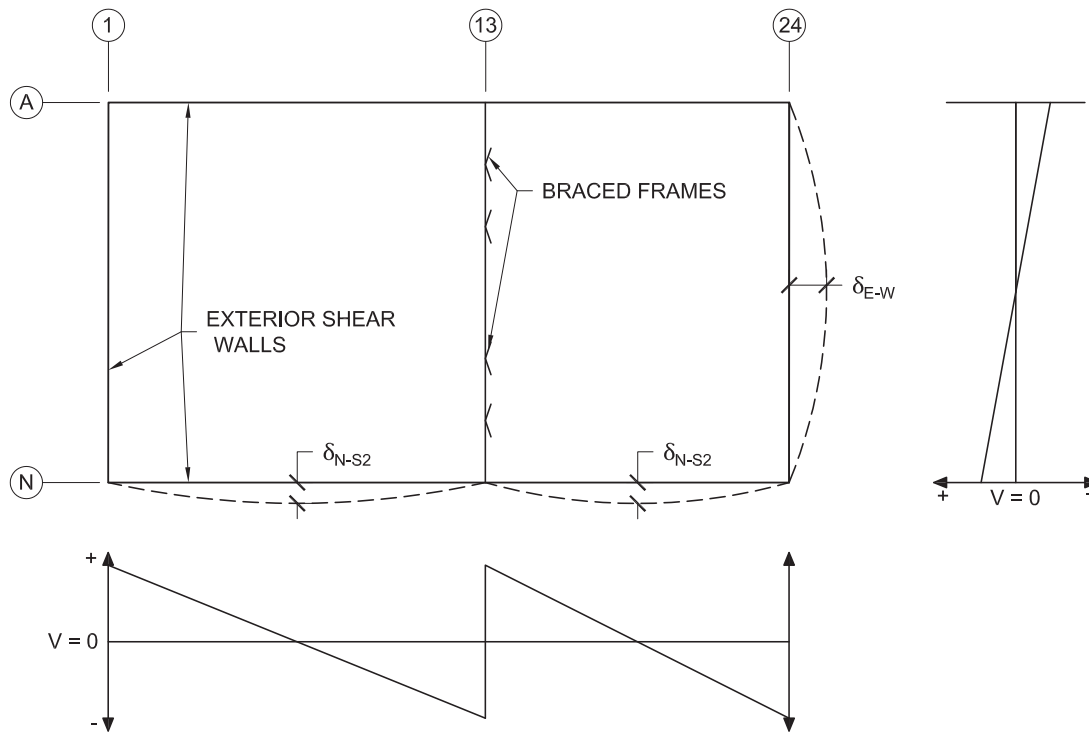


Figure 10.8 Large Roof Structure Diaphragm with Interior Braced Frame

10.3.1 Large Diaphragm Design with Interior Braced Frame

For this large roof structure with a single interior braced frame at Line 13, the diaphragm is divided into a West half and East half. The weight of the roof structure and the North and South walls are the same as that developed in Sections 5.2 and 5.3. In the following example, the West side from Lines 1 to 13 will be investigated because it will have higher diaphragm forces than the East half of the structure which is one bay shorter in length.

The factored roof diaphragm shear loads based on the tributary mass of the roof structure and walls in the North-South direction:

$$F_{px} = w_{N-S} = 0.25[15 \text{ psf}(600 \text{ ft}) + 2[116 \text{ psf}(37 \text{ ft})(37 \text{ ft}/2)(1/32 \text{ ft})]] = 3490 \text{ plf}$$

Diaphragm Reactions at walls East and West walls:

$$V_{N-S1} = V_{N-S24} = w_{N-S}(L_{N-S}/2) = 3490 \text{ plf}(672 \text{ ft}/2) = 1172 \text{ kips}$$

Diaphragm Shear at East and West walls:

$$v_{N-S} = V / b$$

$$v_{N-S} = 1172 \text{ kips} / 600 \text{ ft} = 1952 \text{ plf}$$

Figure 10.8 shows the diaphragm load, diaphragm shear, and end reactions for both directions. Based on this shear distribution, the PLB-36 steel roof deck gage and attachment pattern are selected as presented in Table 10.3. This follows the same methods in Section 5.

| Zone | PLB gage | Span (ft) | Fastener to Supports | | | Sidelap connection VSC2 (in o.c.) | Required Shear LRFD (plf) | Design Shear Strength ϕV_n (plf) |
|------|----------|-----------|----------------------|----------|------------------------------------------|-----------------------------------|---------------------------|----------------------------------------|
| | | | Pattern | Pin | Parallel chords and collectors (in o.c.) | | | |
| IV | 20 | 8 | 36/7/4 | X-HSN 24 | 7 | 8 | 1952 | 2206 |
| III | 22 | 8 | 36/7/4 | X-HSN 24 | 8 | 8 | 1720 | 1729 |
| II | 22 | 8 | 36/7/4 | X-HSN 24 | 12 | 18 | 1162 | 1162 |
| I | 22 | 8 | 36/4 | X-HSN 24 | 16 | 24 | 744 | 770 |

Table 10.3 Large Diaphragm with Braced Frame Line Shear Zones

The strength of the Verco PunchLok II system in combination with the Hilti X-HSN 24 power actuated fasteners easily meets the required diaphragm design strength of 1952 plf for the West half of this large diaphragm structure as shown in Table 10.3. This design reduces the cost of the steel roof deck by eliminating the heavier 18 gage steel deck and the cost of the 36/14/4 pattern. The trade-off is the addition of a single braced frame line and associated footings on Line 13 that are far more expensive than the gravity only columns, joists, and footing displaced. This design is optimized for economy by zoning the deck from 20 gage to 22 gage in the center and reducing the attachment pattern from a 36/7/4 to a 36/4 in the center.

10.3.2 Large Diaphragm Shear Deflection and P-Delta Check with Braced Frame Line

Similar to the single large diaphragm, adequate design strength to resist the seismic load does not ensure that the roof structure will adequately brace the warehouse building. A large diaphragm with high shear loads may potentially have large in-plane diaphragm deflections. In addition to the diaphragm deflection, the deflection of the braced frame needs to be considered. The deflection of the tilt-up shear walls is very small and was ignored in the single diaphragm check. Buckling restrained braced frames (BRB's) are commonly used in warehouse structures. They can be used in all high seismic categories C through F with a seismic force resisting system factor, $R = 8.0$, that is greater than the concrete shear walls, $R = 4.0$, which govern the design. The combination of the diaphragm and the vertical lateral force resisting system needs to be accounted for in the $P\Delta$ check to ensure building stability and deformation compatibility.

The shear deflection for the very large diaphragm roof structure with braced frame is presented in Table 10.4.

| Zone | V_{left} (lb/ft) | V_{right} (lb/ft) | $V_{i\ ave}$ (lb/ft) | L_i (ft) | G' (kip/in) | $0.5(v_{i\ ave} L_i)/(1000G')$ (in) |
|-----------------------------------------------------------|--------------------|---------------------|----------------------|------------|---------------|-------------------------------------|
| IV | 1952 | 1720 | 1836 | 40 | 191 | 0.19 |
| III | 1720 | 1162 | 1441 | 96 | 150 | 0.46 |
| II | 1162 | 744 | 953 | 72 | 117 | 0.29 |
| I | 744 | 0 | 372 | 128 | 96 | 0.25 |
| I | 0 | 744 | 372 | 128 | 96 | 0.25 |
| II | 744 | 1162 | 953 | 72 | 117 | 0.29 |
| III | 1162 | 1720 | 1441 | 96 | 150 | 0.46 |
| IV | 1720 | 1952 | 1836 | 40 | 191 | 0.19 |
| Total Elastic Strength Level Shear Deflection, δ_v | | | | | | 2.39 |

Table 10.4 Large Diaphragm with Braced Frame Line Deflection

Using the same method presented in Section 5.7, the inelastic diaphragm deflection is determined from the unfactored strength-based diaphragm load (1.0E). The 2.39 in of deflection calculated above is the strength-based elastic deflection. The maximum diaphragm deflection is based on the ASCE 7 Section 12.8.6 Story Drift Determination using equation 12.8-15.

$$\delta_x = C_d \delta_{xe} / I_e \quad \text{ASCE 7 eq 12.8-15}$$

$$C_d = 4 \quad \text{ASCE 7 Table 12.2-1 (A5)}$$

$$I_e = 1.0 \quad \text{see Section 4}$$

$$\delta_{xe} = \delta_s = 2.39 \text{ in}$$

$$\delta_x = 4.0(2.39 \text{ in})/1.0 = 9.56 \text{ in}$$

The application of the $P\Delta$ provisions of the ASCE 7 Section 12.8.7 may be applied to this system as a guide to investigate structural stability as follows.

Stability coefficient equation:

$$\theta = P_x \Delta I_e / (V_x h_{sx} C_d) \leq 0.10 \quad \text{ASCE 7 eq 12.8-16}$$

Overstrength coefficient:

$$C_d = 4.0 \quad \text{ASCE 7 Table 12.2-1 (A5)}$$

Roof level height (conservatively use low point on the roof):

$$h_{sx} = 32 \text{ ft} (12 \text{ in/ft}) = 384 \text{ in}$$

Importance Factor:

$$I_e = 1.0 \quad \text{see Section 4}$$

The total unfactored vertical design load tributary to the roof Level:

$$P_{x \text{ roof}} = 15 \text{ psf}(672 \text{ ft})(600 \text{ ft})(1 \text{ kip}/1000 \text{ lb}) = 6048 \text{ kips}$$

$$P_{x \text{ wall}} = 116 \text{ psf}(32 \text{ ft}/2 + 5 \text{ ft})(672 \text{ ft})(2 \text{ walls}) = 3274 \text{ kips}$$

$$P_x = 6048 \text{ kips} + 3274 \text{ kips} = 9322 \text{ kips}$$

The unfactored seismic shear force acting at the roof level:

$$V_x = 1952 \text{ plf} (600 \text{ ft}) + 1952 \text{ plf} (600 \text{ ft}) = 1172 \text{ kips} + 1172 \text{ kips} = 2344 \text{ kips}$$

The average movement of the center of mass of the roof structure based on the estimated maximum strength level roof diaphragm deflection is estimated based on a parabolic deflected shape. For a parabolic curve, the center of mass is at 2/3 of the height, or in this case 2/3 of the in-plane deflection.

$$\Delta = (2/3)\delta_x = (2/3)(9.56 \text{ in}) = 6.37 \text{ in}$$

$$\theta = P_x \Delta I_e / (V_x h_{sx} C_d) = 9322 \text{ kip}(6.37 \text{ in})(1.0) / [2344 \text{ kip}(384 \text{ in})(4.0)] = 0.02 \leq 0.10$$

The predicted stability coefficient is much less than the 10% limit therefore no further investigation of the stability $P\Delta$ effects is required for this structure.

Comparison of Diaphragm with Braced Frame to Hybrid Panelized Wood Roof Structures

The shear deflection for the divided diaphragm with a hybrid panelized wood roof structure is much greater than the comparable steel roof deck design. The shear deflection, G_a , for blocked high shear 1/2" OSB Structural I sheathing ranges 15 kip/in to 70 kip/in which is 5 times less stiff than a comparable Verco steel roof deck diaphragm using the PunchLok II system. This translates to a design level shear deflection of 12.1 in for this structure compared to the 2.4 in for the steel roof deck solution. For wood diaphragms, the American Wood Council *Special Design Provisions for Wind & Seismic* specifies that the shear deflection of the wood diaphragm should be added to the flexural deflection and chord slip of the diaphragm.

$$\delta = \text{Flexure} + \text{Shear} + \text{Chord Slip}$$

$$\delta = 2.8 \text{ in} + 9.3 \text{ in} + 0 \text{ in} = 12.1$$

When this is increased to the deflection for the design level event for wood, the deflection goes up to over 48 in compared to the steel roof deck deflection of only 6.9 in.

$$\delta_M = C_d \delta / I_e = 4 (12.1 \text{ in}) / 1.0 = 48.3 \text{ in}$$

Even with the large diaphragm deflection for the hybrid wood roof structure, it still meets the $P\Delta$ limit following ASCE 7 Section 12.8.7, with $\Phi = 0.05 < 0.10$, compared to the better performance of the steel deck diaphragm prediction of $\Phi = 0.02$.

$$\Delta = (2/3)\delta_M = (2/3)(48.3 \text{ in}) = 32.2 \text{ in}$$

$$\Theta = P_x \Delta I_e / (V_x h_{sx} C_d) = 9322 \text{ kip}(32.2 \text{ in})(1.0) / [3904 \text{ kip}(384 \text{ in})(4.0)] = 0.05 \leq 0.10$$

10.4 Example Large Project Successes

Many successful projects have been constructed over the past several decades with larger steel deck diaphragms. These projects have deck running in the strong direction up for 1250 ft in length. The following projects highlight diaphragms constructed with the most current best practices using Verco PLB-36 deck with Hilti power actuated fasteners to supports and the Verco PunchLok II sidelap system.

Central Avenue - 2017



Figure 10.9 Warehouse 850 E. Central Avenue, Fresno, CA

This warehouse constructed in 2017 in Fresno, CA is an excellent example of a large steel deck diaphragm. This 670,000 square foot concrete tilt-up wall building is 1242 ft long and 540 ft wide. The 22 gage PLB steel deck with PunchLok II sidelap connections runs the 1242 ft length of the building. There are no thermal expansion joints and one interior transverse braced frame line. The inherent ductility of the PunchLok II steel deck attached to the open web steel joists and steel ledger with Hilti X-HSN 24 power actuated fasteners (PAFs) allows the steel deck roof system to relieve the thermal expansion.

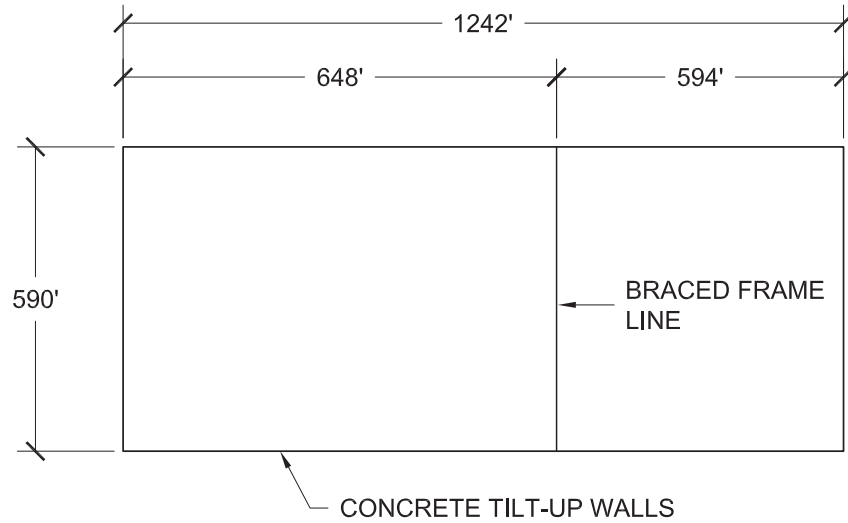


Figure 10.10 Plan View Fresno Warehouse

Harlan Road South - 2016



Figure 10.11 Warehouse at 18290 S. Harlan Road, Lathrop, CA

This warehouse constructed in 2016 in Lathrop, CA is another excellent example of a long steel deck diaphragm. This 518,000 square foot building is 1250 ft long and 430 ft wide with a skewed corner. The 20 gage PLB steel deck runs the 1250 ft length of the building. There are no thermal expansion joints and two interior shear wall lines. The steel roof deck using the Verco PunchLok II system attached to the open web steel joists and steel ledger with Hilti power actuated fasteners (PAF) provides the necessary ductility to relieve thermal stresses along the length of the diaphragm.

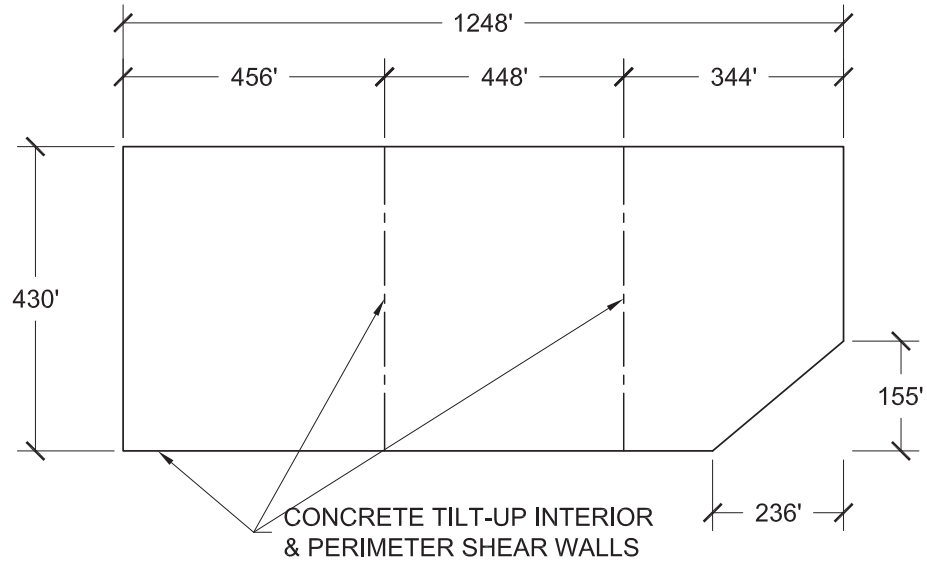


Figure 10.12 Plan View Lathrop Warehouse

11.0 Skylights, Smoke Hatches, Roof Top Mechanical Units, and Miscellaneous Roof Penetrations

Roof structures incorporate a variety of accessories including skylight openings, MEP penetrations, fire suppression system penetrations, and supports for roof top mechanical equipment. These range from small pipe penetrations that may not require any special details, up to large roof top mechanical units that may require structural support frames. Many intermediate penetrations for lightweight skylights, smoke hatches, and exhaust fans require forethought and appropriate detailing. Good detailing will minimize the cost impact on the roof structure while ensuring that the openings do not compromise the integrity of the roof structure.

11.1 Skylights and Smoke Hatches

Skylights are an important part of providing daylight through the roof structure. Traditionally in roof structures with steel roof deck, the skylight openings have been reinforced with costly structural steel opening frames that are better suited for supporting roof top mechanical units as shown in Figure 11.1a. A better solution for the skylights in this example building is to take advantage of the 8 ft joist spacing and eliminate the need for structural steel support frames as shown in Figure 11.1b. The choice of the consistent 8 ft joist spacing allows for this economy.

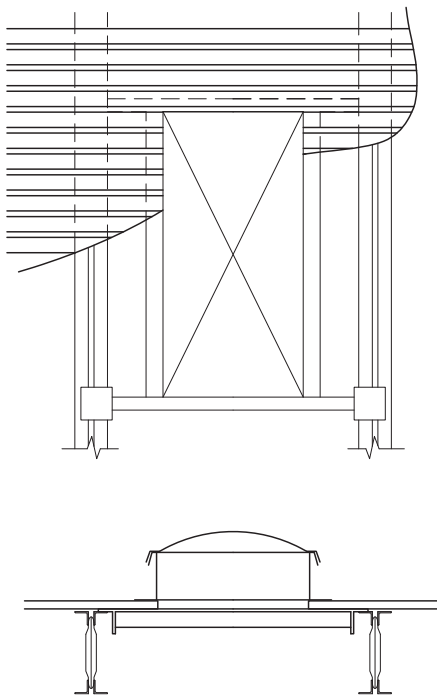


Figure 11.1a Skylight with Frame

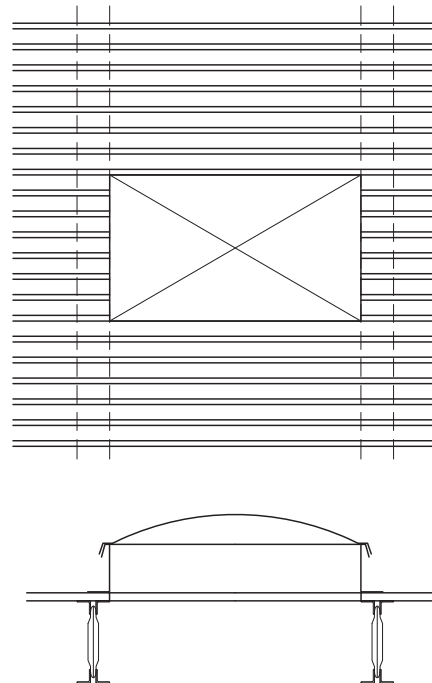


Figure 11.1b Skylight without Frame

The skylight curb running between joists in Figure 11.1b is supported by the steel deck that spans between joists. The deck is not cut between the joists therefore no opening framing is required to support the steel deck around the opening. The choice of a 3 ft x 8 ft nominal skylight is the most economical size for 1½ in deep PLB-36 steel deck with a 3 ft sheet width. By choosing the 3 ft width, it is possible to detail and fabricate the steel deck to provide for the 3 ft x 8 ft openings without any field cutting of the steel deck as shown in Figure 11.2. The deck sheets in Figure 11.2 are detailed and fabricated as typical 4-span sheets for the field with 1-span and 2-span sheets to accommodate the skylight openings without the need to field cut the steel roof deck. The traditional 4 ft x 8 ft skylights, which are meant to replace a sheet of plywood, are not ideally suited for 3 ft wide steel roof deck because the deck must be field cut to match the 4 ft skylight width.

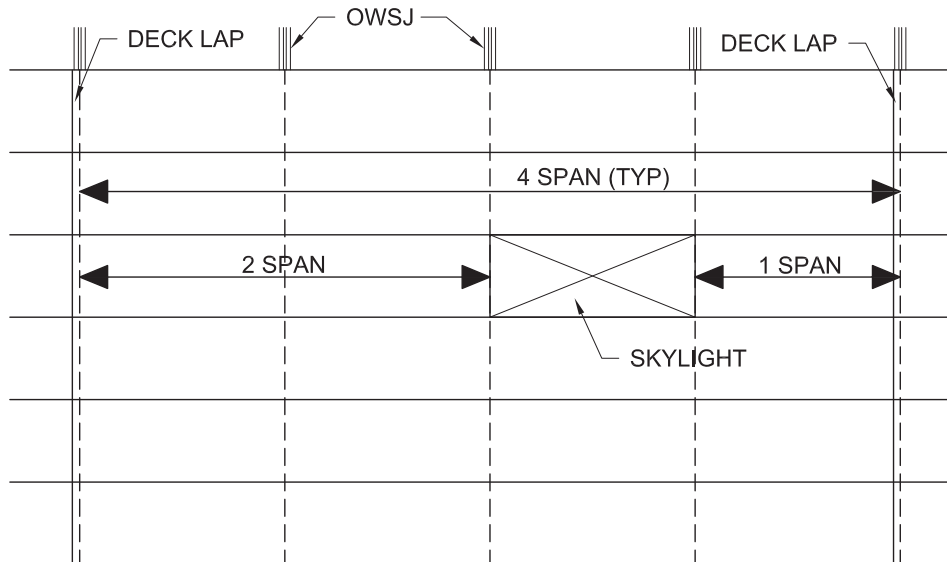


Figure 11.2 Detailing Deck Around Skylight Openings

Factory Mutual standards discussed in Section 9 may limit the maximum joist spacing for 22 gage PLB-36 deck to 6 ft or increase the deck gage to remain at the 8 ft joist spacing. If the decision is made to maintain the economy of the 22 gage PLB-36 steel roof deck, then the joist spacing will be reduced to 6 ft o.c. For projects with a 6 ft joist spacing similar economy can be achieved with either 3 ft x 6 ft nominal skylights or larger 6 ft x 6 ft nominal skylights. The larger 6 ft x 6 ft nominal skylights provide 36 sq ft of daylight, which is greater than the 32 square feet of the traditional 4 ft x 8 ft skylights, without the need to field cut the steel roof deck.

Smoke hatches are similar to skylights and many models double as skylights. UL listed smoke hatches come in more limited sizes, however common sizes include both 6 ft and 8 ft lengths in varying widths from 4 ft to 5 ft wide. These hatches can also be oriented to eliminate the need for deck support frames and minimize the field cutting of deck.

11.2 Roof Top Mechanical Unit Support Frames

Roof top mechanical units (RTU) are a feature of large roof structures. The weight and lateral loads of the mechanical units must be resisted by the roof structure. It is rare that the width of a RTU will match the joist spacing and thus eliminate the need for a frame to support the unit between joists. The RTU frame serves the dual role of supporting the weight of the unit and reinforcing any opening(s) in the steel deck at the unit. The steel deck may support the weight of the unit transferred from the RTU curb through the webs of the steel deck to the support frame below.

Traditional mechanical unit support frames are fabricated using hot rolled angle, channel or wide flange sections. The frames are typically a double H or single H configuration as shown in Figures 11.3a and 11.3b respectively. These are generally shop welded with hanger angles that sit on the top chords of the steel joists. This type of shop fabricated frame is a reasonably efficient solution for new construction when the frames can be shop welded and set in place on the joists before the steel deck is installed.

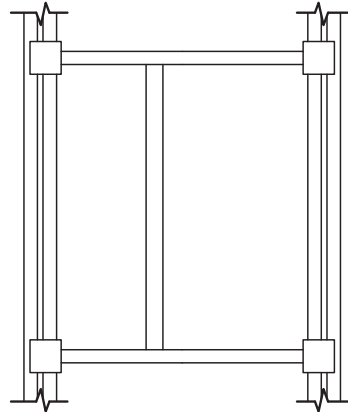
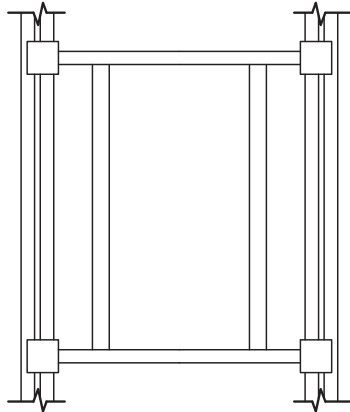


Figure 11.3a Double H Opening Frame Figure 11.3b Single H Opening Frame

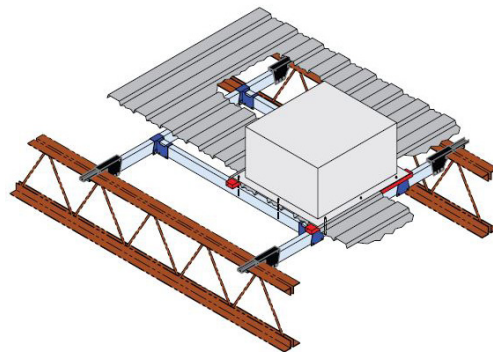
For retrofit situations, frames that are field welded below the steel roof deck are costly and the welding leads to fire protection issues when there are combustible finishes or contents in the building. Several companies fabricate bolt-together pre-engineered RTU support frames. These are good options for retrofit situations and also work well for new construction. Figures 11.4a and 11.4b depicts two such manufacturers of bolt-together pre-engineered RTU frames. These pre-engineered frames often provide greater economy than custom shop welded steel angle frames.

Verco Decking Inc. neither endorses or discourages the use of pre-engineered deck support frames manufactured by QuickFrame or Chicago Clamp. These products are shown as an example of an alternate to the traditional welded frame that are suitable for use in roof structures with steel deck diaphragms. Determining suitability for the use of these products is the responsibility of the project design professional.



<http://www.quickframes.us/>

Figure 11.4a QuickFrame®



<http://chicagoclampcompany.com/>

Figure 11.4b Chicago Clamp®

11.3 Roof Top Mechanical Unit Example

Most warehouse distribution buildings have a heated and air-conditioned office in a portion of the building. The HVAC units for the office are typically located on the roof structure above the office. For this structure, a typical 25-ton capacity air-conditioner/furnace gas/electric package unit will be located on the roof deck, supported by a structural frame below the deck. This example focuses on the strength of the deck to support the RTU above the structural support frame. The unit shown in Figure 11.5 is representative of the size and weight of a mid-size RTU that will be used in this example.

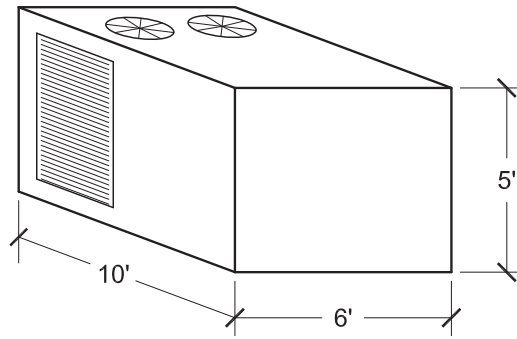


Figure 11.5 Typical HVAC Roof Top Unit

Roof top unit operating weight from manufacturer's literature.

$$W_p = 2400 \text{ lb} \qquad \text{ASCE 7 §13.3.1}$$

The design loads on an RTU include gravity, seismic, and wind. In this example we will address both seismic and wind load design of the RTU support from the steel roof deck.

11.3.1 Seismic Design for RTU

The seismic load on the RTU falls under the provisions of ASCE 7 Chapter 13 Seismic Design Requirements for Nonstructural Components.

To determine if seismic design of the RTU unit is required, the seismic design category (SDC) of the building must be considered. The SDC for the nonstructural components shall be assigned the same seismic design category as the structure that they occupy or to which they are attached.

$$\text{Seismic Design Category D} \qquad \text{ASCE 7 §13.1.2}$$

All nonstructural components must be designed to resist seismic loads unless they are specifically exempted from the requirement. This warehouse example falls into Seismic Design Category D therefore Exemptions 4 and 5 of ASCE 7-10 Section 13.1.4 that exempt mechanical and electrical components in Seismic Design Category B and with 1.0 in Seismic Design Category C respectively do not apply. For this warehouse in Seismic Category D, the RTU shall be designed to resist seismic loads.

To determine the seismic load on the RTU unit, the usage of the component must be considered. The HVAC RTU for this warehouse does not meet the requirement to have an importance factor of 1.5 as defined in ASCE 7 Section 13.1.3, therefore the component importance factor is set to 1.0.

Component Importance Factor:

$$I_p = 1.0 \qquad \text{ASCE 7 §13.1.3}$$

Short period spectral acceleration for the structure determined in Section 5.0 of this example.

$$S_{DS} = 1.0 \qquad \text{ASCE 7 §11.4.5}$$

The seismic coefficients are taken from ASCE 7 Table 13.6-1 for this gas/electric air-side mechanical unit.

$$a_p = 2.5 \qquad \text{ASCE 7 Table 13.6-1}$$

$$R_p = 6.0 \qquad \text{ASCE 7 Table 13.6-1}$$

Average roof height of structure as shown in Figure 11.6.

$$h = 34 \text{ ft}$$

ASCE 7 §13.3.1.1

Determine the height of the structure to the point of attachment of the component with respect to the base. For this structure the height of the base of the RTU is the same as the roof elevation, therefore it is appropriate to use the average roof elevation as shown in Figure 11.6.

$$z = 34 \text{ ft}$$

ASCE 7 §13.3.1.1

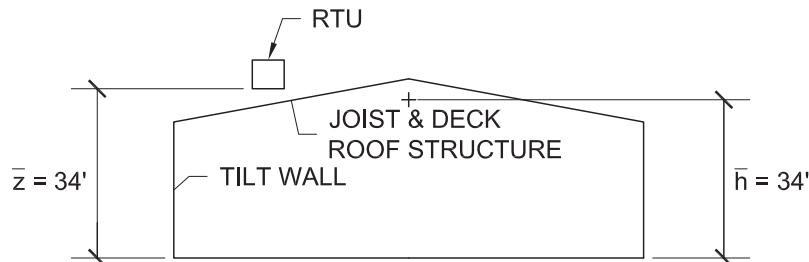


Figure 11.6 Average Roof and RTU Height

The seismic design force for nonstructural components is determined in accordance with ASCE 7 Section 13.3.1.

Seismic horizontal loads:

$$F_p = \frac{0.4a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2 \frac{z}{h}\right) = 0.5W_p$$

ASCE 7 eq 13.3-1

$$F_{p \max} = 1.6S_{DS} I_p W_p = 1.6(1.0)(1.0)W_p = 1.6W_p$$

ASCE 7 eq 13.3-2

$$F_{p \min} = 0.3S_{DS} I_p W_p = 0.3(1.0)(1.0)W_p = 0.3W_p$$

ASCE 7 eq 13.3-3

$$\text{Governing } F_p = 0.5W_p$$

Seismic horizontal load:

$$E_h = \rho Q_E F_p = 0.5W_p = 0.5(2400 \text{ lb}) = 1200 \text{ lb}$$

ASCE 7 eq 12.4-3

$$\text{Where } \rho = 1.0$$

ASCE 7 Section 12.3.4.1

Seismic vertical load:

$$E_v = \pm 0.2S_{DS} W_p = \pm 0.2(1.0)(2400 \text{ lb}) = \pm 480 \text{ lb}$$

ASCE 7 §13.3.1.2

Standard basic load combinations 1, 2, 6, and 7 applicable to the RTU unit from ASCE 7 Section 2.3.6:

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S or R)
6. 1.2D + E_v + E_h + L + 0.2S
7. 0.9D – E_v + E_h

Where L, L_r, S, and R, are not present, the combinations simplify for the RTU unit.

1. 1.4D
2. 1.2D
6. 1.2D + E_v + E_h
7. 0.9D – E_v + E_h

Where D = W_p

The forces are applied to the RTU as shown in the free body diagram in Figure 11.7.

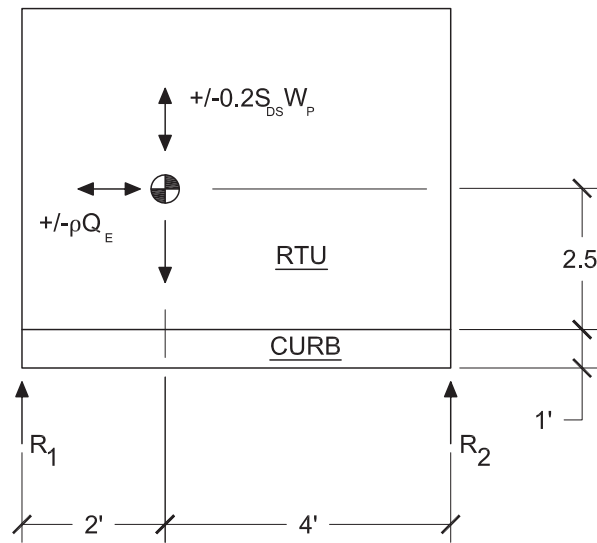


Figure 11.7 Seismic RTU Free Body Diagram

Base shear of RTU frame due to seismic:

$$V_h = E_h = 1.0(1200 \text{ lb}) = 1200 \text{ lb}$$

Force of RTU support frame:

Load Combination 1 (Gravity only):

$$\sum M_{R2} = 0 = 1.4(2400 \text{ lb})(4 \text{ ft}) - R_1(6 \text{ ft})$$

$$R_1 = 2240 \text{ lb}$$

$$\sum M_{R1} = 0 = 1.4(2400 \text{ lb})(2 \text{ ft}) - R_2(6 \text{ ft})$$

$$R_2 = 1120 \text{ lb}$$

Load Combination 6 (Seismic):

$$\sum M_{R2} = 0 = 1.2(2400 \text{ lb})(4 \text{ ft}) \pm 480 \text{ lb}(3.5 \text{ ft}) + 1200 \text{ lb}(4 \text{ ft}) - R_1(6 \text{ ft})$$

$$R_1 = \frac{11520 \text{ lb-ft} \pm 1640 \text{ lb-ft} + 4800 \text{ lb-ft}}{6 \text{ ft}} = \{+3000 \text{ lb}, +2440 \text{ lb}\}$$

$$\sum M_{R1} = 0 = 1.2(2400 \text{ lb})(2 \text{ ft}) \pm 480 \text{ lb}(3.5 \text{ ft}) + 1200 \text{ lb}(2 \text{ ft}) - R_2(6 \text{ ft})$$

$$R_2 = \frac{5760 \text{ lb-ft} \pm 1680 \text{ lb-ft} + 2400 \text{ lb-ft}}{6 \text{ ft}} = \{+1640 \text{ lb}, +1080 \text{ lb}\}$$

Load Combination 7 (Seismic):

$$\sum M_{R2} = 0 = 0.9(2400 \text{ lb})(4 \text{ ft}) \pm 480 \text{ lb} (3.5 \text{ ft}) - 1200 \text{ lb} (4 \text{ ft}) - R_1(6 \text{ ft})$$

$$R_1 = \frac{8640 \text{ lb-ft} \pm 1680 \text{ lb-ft} (-4800 \text{ lb-ft})}{6 \text{ ft}} = \{+920 \text{ lb}, +360 \text{ lb}\}$$

$$\sum M_{R1} = 0 = 0.9(2400 \text{ lb})(2 \text{ ft}) \pm 480 \text{ lb}(3.5 \text{ ft}) - 1200 \text{ lb} (2 \text{ ft}) - R_2(6 \text{ ft})$$

$$R_2 = \frac{4320 \text{ lb-ft} \pm 1680 \text{ lb-ft} (-2400 \text{ lb-ft})}{6 \text{ ft}} = \{+3600 \text{ lb}, +40 \text{ lb}\}$$

All reactions for seismic are positive (+) indicating that there is no tension at either reaction. The reaction forces are for compression, bearing on the steel roof deck and below deck RTU support frame.

Governing Reaction for Seismic:

$$R_1 = +3600 \text{ lb}$$

There are only positive reactions for the seismic load combinations, therefore the RTU is not subject to overturning. The reactions will create bearing on the steel deck requiring a web crippling check to verify the steel deck is adequate to support the RTU curb.

11.3.2 Wind Design for RTU

The wind load design is similar to the main building structure, except that for RTU units the combined pressure coefficient and gust Factor, GC_r , is used. The wind design for Steps 1 through 5 are the same as discussed in Section 4 except for the combined factor.

Step 1: The warehouse is assigned a Risk Category of II for structures in accordance with ASCE 7 Table 1.5-1 because the occupancy does not meet the requirements to be Category I, III, or IV occupancies for low risk, substantial risk or essential facilities for human life.

Risk Category = II

ASCE 7 Table 1.5-1

Step 2: For this example, the building site is in Ontario, California, which is in a low wind speed region. The basic wind speed is taken from ASCE 7 Figure 26.5-1B.

$V = 95 \text{ mph}$

ASCE 7 Figure 26.5-1B

Step 3: For this example, the wind parameters related to the site will be assumed as follows:

Wind directionality factor, $K_d = 0.85$

ASCE 7 Table 26.6-1

Exposure Category = C

ASCE 7 §26.7

Topographic Factor, $K_{zt} = 1.0$

ASCE 7 §26.8

Ground Elevation Factor, $K_e = 1.0$

ASCE 7 §26.9

To determine the combined pressure coefficient and gust factor, the vertical projected area of the rooftop unit is compared to 10% of the projected area of the building in accordance with ASCE 7 Section 29.4.1.

Projected area of building based on lesser width of building to be conservative in determining GC_r .

Width of building, $B = 300 \text{ ft}$

Mean roof height, $h = 34.5 \text{ ft}$

The lower mean roof height for the wall on Line 1 or 10 assuming that grade is at floor level to develop the minimum projected area.

$$0.1Bh = 0.1 (300 \text{ ft})(34.5 \text{ ft}) = 1035 \text{ sf}$$

Projected area of RTU:

$$A_r = h_u(w_u) = 6\text{ft} (10 \text{ ft}) = 60 \text{ sf} < 1035 \text{ sf} = 0.1Bh$$

Combined Pressure Coefficient and Gust Factor: ASCE 7 §29.4.1

$$GC_r = 1.9 \text{ horizontal load}$$

$$GC_r = 1.5 \text{ vertical load}$$

Step 4: The velocity pressure coefficients for the roof are determined in accordance with ASCE 7 Table 26.10.1 based on the mean roof height, and exposure category, or using the formulas in the footnotes of the table. To determine the pressure coefficients, the nominal height of the atmospheric boundary layer, z_g , and the 3-sec gust-speed power law exponent, α , are taken from ASCE 7 Table 26.11-1. These factors are the same for both the MWFRS and C&C loads.

$$z_g = 900 \text{ ft} \quad \text{ASCE 7 Table 26.11-1}$$

$$\alpha = 9.5 \quad \text{ASCE 7 Table 26.11-1}$$

For $15 \text{ ft} \leq z \leq z_g$ where, $z = h = 38 \text{ ft}$

$$K_h = K_z = 2.01(z/z_g)^{(2/\alpha)} = 2.01(38/900)^{(2/9.5)} = 1.03 \quad \text{ASCE 7 Table 26.10-1}$$

Step 5: The velocity pressure is then determined for $z = h = 38 \text{ ft}$, the mean roof height used for the gable roof.

$$q_h = 0.00256K_zK_{zt}K_dV^2 = 0.00256(1.03)(1.0)(0.85)(95)^2 = 20.3 \text{ psf} \quad \text{ASCE 7 eq 26.10-1}$$

Step 6: The force coefficient, C_r , for the RTU is part of the combined GC_r determined in Step 4.

Step 7: The wind force on the RTU is determined following ASCE 7 Section 29.4.2.

$$F_h = q_h(GC_r)A_r = (20.3 \text{ psf})(1.9)(A_r) \quad \text{ASCE 7 eq 29.4-2}$$

$$F_h = 38.6 \text{ psf}(A_r) = 38.6 \text{ psf}(60 \text{ sf}) = 2314 \text{ lb}$$

$$F_v = q_h(GC_r)A_r \quad \text{ASCE 7 eq 29.4-3}$$

$$F_v = (20.3 \text{ psf})(1.5)(10 \text{ ft})(6 \text{ ft}) = 30.45 \text{ psf} (60 \text{ sf}) = 1827 \text{ lb}$$

Load combinations for strength design:

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L + 0.5(L_r or S or R)
5. 0.9D + 1.0W

Where L, L_r, S, and R, are not present the combinations simplify for the RTU unit.

1. 1.4D
2. 1.2D
3. 1.2D + 0.5W
4. 1.2D + 1.0W
5. 0.9D + 1.0W

By inspection, load combination 5 will govern for overturning because the vertical wind load is always uplift. Load combination 4 will govern for inward loading because there is no component of wind acting against the dead load. The reactions are determined from the forces shown in the free body diagram in Figure 11.8.

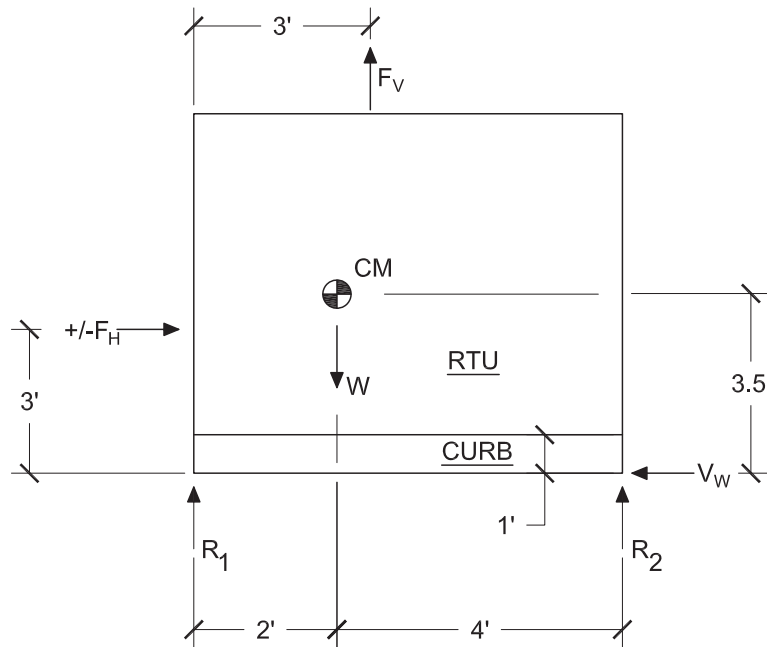


Figure 11.8 Wind RTU Free Body Diagram

Base shear of RTU frame due to wind:

$$V_h = 1.0W = 1.0Fh = 2314 \text{ lb}$$

Overturning Force of RTU support frame:

Load Combination 4 (Wind):

$$1.0W = 1.0(2314 \text{ lb}) = 2314 \text{ lb (horizontal)}$$

$$\sum M_{R2} = 0 = 1.2(2400 \text{ lb})(4 \text{ ft}) \pm 1.0(2314 \text{ lb})(3 \text{ ft}) - 1.0(1827 \text{ lb})(3 \text{ ft}) - R_1(6 \text{ ft})$$

$$R_1 = \frac{11520 \text{ lb-ft} \pm 6942 \text{ lb-ft} - 5481 \text{ lb-ft}}{6 \text{ ft}} = \{+2164 \text{ lb}, -150 \text{ lb}\}$$

$$\sum M_{R1} = 0 = 1.2(2400 \text{ lb})(2 \text{ ft}) \pm 1.0(2314 \text{ lb})(3 \text{ ft}) - 1.0(1827 \text{ lb})(3 \text{ ft}) - R_2(6 \text{ ft})$$

$$R_2 = \frac{5760 \text{ lb-ft} \pm 6942 \text{ lb-ft} - 5481 \text{ lb-ft}}{6 \text{ ft}} = \{+1204 \text{ lb}, -1110 \text{ lb}\}$$

Load Combination 5 (Wind):

$$\sum M_{R2} = 0 = 0.9(2400 \text{ lb})(4 \text{ ft}) \pm 1.0(2314 \text{ lb})(3 \text{ ft}) - 1.0(1827 \text{ lb})(3 \text{ ft}) - R_1(6 \text{ ft})$$

$$R_1 = \frac{8640 \text{ lb-ft} \pm 6942 \text{ lb-ft} - 5481 \text{ lb-ft}}{6 \text{ ft}} = \{+1684 \text{ lb} - 631 \text{ lb}\}$$

$$\sum M_{R1} = 0 = 0.9(2400 \text{ lb})(2 \text{ ft}) \pm 1.0(2314 \text{ lb})(3 \text{ ft}) - 1.0(1827 \text{ lb})(3 \text{ ft}) - R_2(6 \text{ ft})$$

$$R_2 = \frac{4320 \text{ lb-ft} \pm 6942 \text{ lb-ft} - 5481 \text{ lb-ft}}{6 \text{ ft}} = \{+963 \text{ lb}, -1351 \text{ lb}\}$$

Governing Reactions:

$$R_1 = +2164 \text{ lb (compression)}$$

$$R_2 = -1351 \text{ lb (tension)}$$

For wind load combinations, there are both the uplift creating tension on the connections at R_2 and compression creating bearing on the steel deck at R_1 . This will require both a tension check on the connections of the RTU curb to the steel deck, and a web-crippling check of the compression on the steel deck.

11.3.3 RTU Curb Bearing Load on Steel Deck

The steel deck bears on the below deck mechanical support frame that supports the RTU curb. The average distributed load along the 10 ft length of the RTU curb is determined for the highest compressive load. The distributed load shown in Figure 11.9a is resisted by the web crippling strength of the steel deck in 2-flange loading as shown in Figure 11.9b.

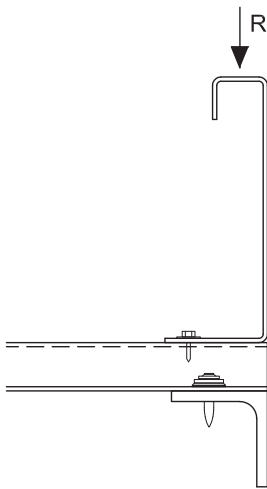


Figure 11.9a RTU Curb on Deck & Frame

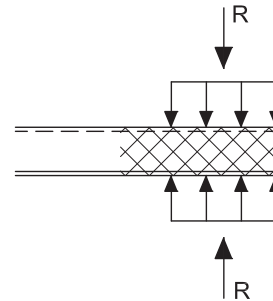


Figure 11.9b Two Flange Loading

Web crippling strength is determined in accordance with AISI S100 Section G5. To assist with this detailed calculation, the Verco web-based design tool for Web Crippling is used to determine the strength. The summary output is shown in Figure 11.10.

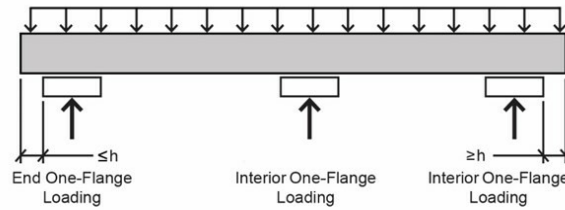
Governing Reaction from Gravity, Seismic and Wind load combinations is from seismic combination 7.

$$R_1 = +3600 \text{ lb}$$

The average reaction per foot over the 10 ft RTU length of 360 plf is much less than the 2-flange loading design strength for a 3 in bearing width of 2410 plf from Figure 11.10

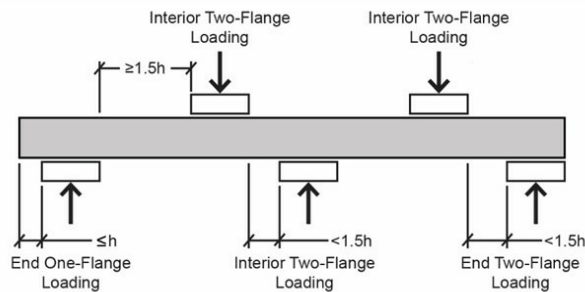
$$R_{\max} / L = 3600 \text{ lb}/10 \text{ ft} = 360 \text{ plf} < 2410 \text{ plf} = \Phi P_n$$

One Flange Loading for Uniform Load



| Reaction Design Strength at Supports Based on Web Crippling for One Flange Loading, LRFD (plf) | | | | | | | | | |
|------------------------------------------------------------------------------------------------|---------|------|------|------|------|------|------|------|------|
| | Bearing | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 3.50 | 4.00 | 4.50 |
| End | ΦRn | 1611 | 1817 | 1991 | 2144 | 2282 | 2410 | 2461 | 2461 |
| Interior | ΦRn | 2419 | 2676 | 2893 | 3084 | 3256 | 3415 | 3479 | 3479 |

One or Two Flange Loading for Concentrated Loads



| Reaction Design Strength at Supports Based on Web Crippling for Two Flange Loading, LRFD (plf) | | | | | | | | | |
|------------------------------------------------------------------------------------------------|---------|------|------|------|------|------|------|------|------|
| | Bearing | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 3.50 | 4.00 | 4.50 |
| End | ΦRn | 1838 | 2014 | 2162 | 2292 | 2410 | 2518 | 2562 | 2562 |
| Interior | ΦRn | 2947 | 3295 | 3588 | 3847 | 4081 | 4296 | 4383 | 4383 |

| Steel Deck Properties | | | | | | | | | |
|-----------------------|-----|------|----------------------|----------------------|----------------------|----------------------|-----------|-----------|--------|
| | Fy | wdd | Se+ | Se- | Id+ | Id- | ΦMn+ | ΦMn- | ΦVn |
| Gage | ksi | psf | in. ³ /ft | in. ³ /ft | in. ⁴ /ft | in. ⁴ /ft | lbs-ft/ft | lbs-ft/ft | lbs/ft |
| 20 | 50 | 2.30 | 0.230 | 0.237 | 0.219 | 0.231 | 862 | 889 | 4894 |

Web Crippling V1.0 in Accordance with AISI S100-16 and IAPMO ER-2018

Figure 11.10 Web Crippling Design Strength for 22 gage PLB-36

The factored web crippling strength is 7 times greater than the maximum distributed load, providing more than adequate capacity to resist the seismic or wind loads of the RTU.

11.3.4 RTU Curb Shear and Tension Connection Design to Steel Roof Deck

The RTU curb is attached to the steel roof deck to prevent sliding and overturning with self-drilling screws. The design of this connection is checked for shear, tension and combined shear and tension following the provisions of AISI S100 Section J.4

The horizontal shear strength of the connections between the RTU curb and the steel deck are designed to resist the seismic shear load effect. For this curb, the shear will be transferred with No. 12 self-drilling screws as shown in Figure 11.11.

Governing Reaction from Gravity, Seismic and Wind load combinations is for wind load combinations.

$$V = 1.0W = 2314 \text{ lb}$$

$$R_2 = -1351 \text{ lb (tension)}$$

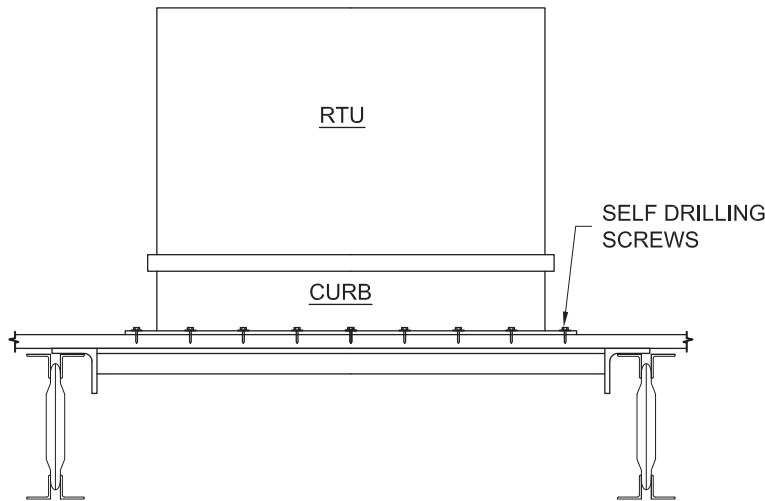


Figure 11.11 RTU Unit and Curb on Steel Deck

The connection of the RTU curb flange bearing on the steel deck is shown in Figure 11.12. This connection is designed to resist both shear and tension due to the wind or seismic loading on the RTU.

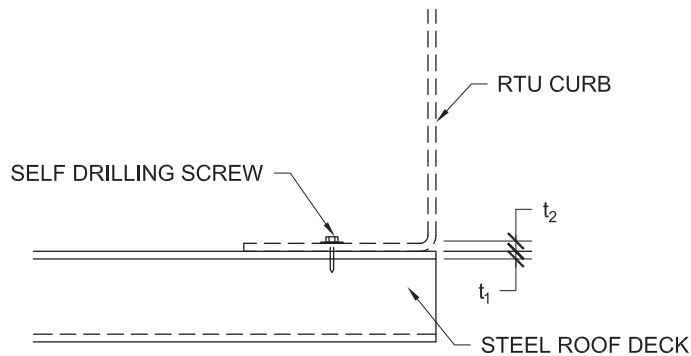


Figure 11.12 Curb to Steel Deck Screw Connection

The shear capacity of No. 12-14 hex washer head self-drilling screws is determined in accordance with AISI S100 Section J4.3.

$$t_1 = 0.060 \text{ in}$$

Typical RTU Curb

$$F_{u2} = 45 \text{ ksi}$$

Typical RTU Curb

$$t_2 = 0.0299 \text{ in}$$

PLB-36

$$F_{u1} = 65 \text{ ksi}$$

PLB-36

$$d = 0.216 \text{ in} \quad \text{AISI S100 Table C-J4-1}$$

$$t_2 / t_1 = 0.0299 / 0.060 = 0.50$$

$$P_{nvs} = 2000 \text{ lb} \quad \text{AISI S100 §J4.3.2}$$

Where P_{nvs} is based on the reported shear strength of major screw manufacturers.

For $t_2 / t_1 \leq 1.0$, P_{ns} shall be the minimum of:

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} = 4.2((0.0299 \text{ in})^3 0.216 \text{ in})^{1/2} 65 \text{ ksi} = 0.656 \text{ kip} \quad \text{AISI S100 eq J4.3.1-1}$$

$$P_{ns} = 2.7t_1 d F_{u1} = 2.7(0.060 \text{ in})(0.216 \text{ in})(45 \text{ ksi}) = 1.57 \text{ kip} \quad \text{AISI S100 eq J 4.3.1-3}$$

$$P_{ns} = 2.7t_2 d F_{u2} = 2.7(0.0299 \text{ in})(0.216 \text{ in})(65 \text{ ksi}) = 1.13 \text{ kip} \quad \text{AISI S100 eq J4.3.1-2}$$

$$\text{Governing } P_{ns} = 0.656 \text{ kip}$$

$$\phi = 0.50 \text{ (LRFD)} \quad \text{AISI S100 §J4}$$

$$P_{as} = \phi P_{ns} = 0.50(0.656 \text{ kip}) = 0.328 \text{ kip/screw} \quad \text{AISI S100 §B.3.2.2}$$

$$\nabla / P_{as} = 2314 \text{ lb} / 328 \text{ lb/screw} = 7 \text{ screws}$$

The tension strength of the screw:

Pullout for Screw:

$$P_{not} = 0.85t_c d F_{u2} \quad \text{AISI S100 eq J4.4.1-1}$$

$$P_{not} = 0.85(0.0299 \text{ in})(0.216 \text{ in})(65 \text{ ksi}) = 0.357 \text{ kip}$$

Pullover for Screw:

$$P_{nov} = 1.5t_1 d'_w F_{u1} \quad \text{AISI S100 eq J4.4.2-1}$$

$$P_{nov} = 1.5(0.060 \text{ in})(0.398 \text{ in})(45 \text{ ksi}) = 1.612 \text{ kip}$$

Where $d'_w = 0.398 \text{ in}$ for a No. 12 HWH screw

Tensile Strength of Screw:

$$P_{nts} = 2778 \text{ lb} \quad \text{AISI S100 §J4.4.3}$$

Where P_{nts} is based on the reported tensile strength of major screw manufacturers.

Design Tension Strength:

$$P_n = P_{nts} = 357 \text{ lb governing pull-out of screw}$$

$$P_{at} = \phi P_n = 0.5(357 \text{ lb}) = 179 \text{ lb}$$

$$\text{Where } \phi = 0.50 \quad \text{AISI S100 §J4}$$

The minimum required number of screws to resist the tension of the RTU curb.

$$R_2 / P_{at} = 1351 \text{ lb} / 179 \text{ lb/screw} = 7.5, \text{ therefore use 8 screws}$$

The screws attaching the RTU curb at reaction, R_2 , will be subject to combined shear and tension. This requires that screws be checked for this condition. To do this a trial number of screws needs to be selected. For this example, screws at 12 in o.c. around the entire RTU curb will be checked which works out to 32 screws for the 6 ft x 10 ft curb as shown in Figure 11.13.

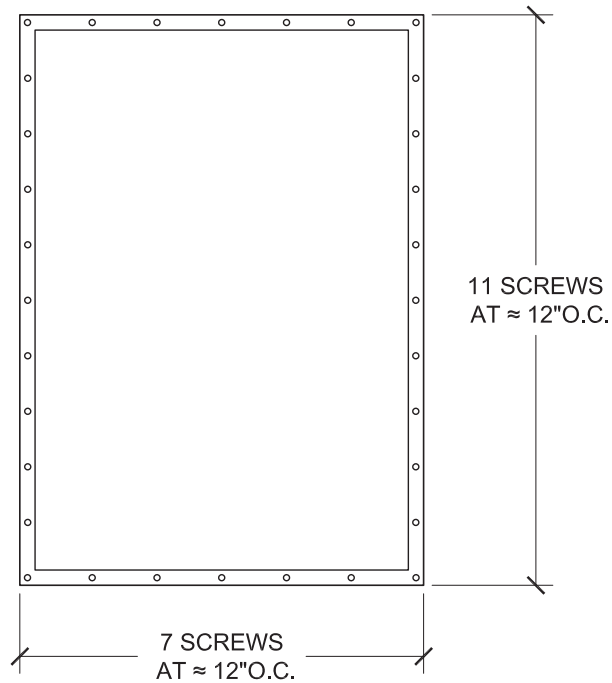


Figure 11.13 RTU Curb to Deck Screw Layout

Average shear per screw:

$$V / N = 2314 \text{ lb} / 32 \text{ screws} = 72 \text{ lb/screw}$$

Tension per screw on the 10 ft side at R_2 :

$$R_2 / N = 1351 \text{ lb} / 11 \text{ screws} = 123 \text{ lb/screw}$$

Combined Shear and Pull-Over:

$$\frac{V}{P_{nv}} + 0.71 \frac{\bar{T}}{P_{nov}} \leq 1.10\phi \quad \text{AISI S100 eq J4.5.1-1b}$$

Where:

$$P_{nv} = 2.7t_1dF_{u1} = 1.570 \text{ kip} = 1575 \text{ lb} \quad \text{AISI S100 eq J4.5.1-2}$$

$$P_{nov} = 1.5t_1d_wF_{u1} = 1.611 \text{ kip} = 1612 \text{ lb} \quad \text{AISI S100 eq J4.5.1-3}$$

$$\frac{72 \text{ lb}}{1575 \text{ lb}} + 0.71 \frac{123 \text{ lb}}{1612 \text{ lb}} = 0.100 \leq 0.715 = 1.10(0.65)$$

Combined Shear and Pull-Out:

$$\frac{V}{P_{nv}} + \frac{\bar{T}}{P_{not}} \leq 1.15\phi \quad \text{AISI S100 eq J4.5.2-1b}$$

Where:

$$P_{nv} = 4.2(t_2^3 d)^{1/2} F_{u2} = 0.656 \text{ kip} = 656 \text{ lb} \quad \text{AISI S100 eq J4.5.2-2}$$

$$P_{not} = 0.85 t_c d F_{u2} = 0.357 \text{ kip} = 357 \text{ lbs} \quad \text{AISI S100 eq J4.5.2-3}$$

$$\frac{72 \text{ lb}}{656 \text{ lb}} + \frac{123 \text{ lb}}{357 \text{ lb}} = 0.454 \leq 0.690 = 1.15(0.60)$$

Combined Shear and Tension:

$$\frac{V}{P_{nsv}} + \frac{\bar{T}}{P_{nts}} \leq 1.3\phi \quad \text{AISI S100 eq J4.5.3-1b}$$

Where:

$$P_{nsv} = 2000 \text{ lb}$$

$$P_{nts} = 2778 \text{ lb}$$

$$\frac{72 \text{ lb}}{2000 \text{ lb}} + \frac{123 \text{ lb}}{2778 \text{ lb}} = 0.090 \leq 0.650 = 1.3(0.5)$$

The interaction checks for the screws at 12 in o.c for shear combined with pull-over, pull-out and tension all meet the interaction requirements.

11.3.5 RTU Tension Connection Design of Steel Roof Deck to Supports

The steel roof deck has adequate strength to support the RTU for both inward loading through web crippling and the tension strength of the screw connections between the RTU curb and steel roof deck. Following the load path, the steel roof deck must distribute the shear and vertical loads to the structure. The shear force of 260 plf across the 6 ft ends of the RTU is relatively low compared to the minimum diaphragm shear strength of the steel roof deck of 770 plf.

$$2314 \text{ lb} / (2 (6 \text{ ft})) = 193 \text{ plf} < 770 \text{ plf}$$

This demonstrates that the steel roof deck diaphragm has more than adequate strength to resist the localized shear induced by the RTU. The second issue that needs to be addressed is the tension load applied to the steel roof deck connections to the support frame following the provisions of AISI S100 and IAPMO ER-2018. Figure 11.14 shows the Hilti X-HSN 24 power actuated fastener connection of the steel roof deck to the support frame.

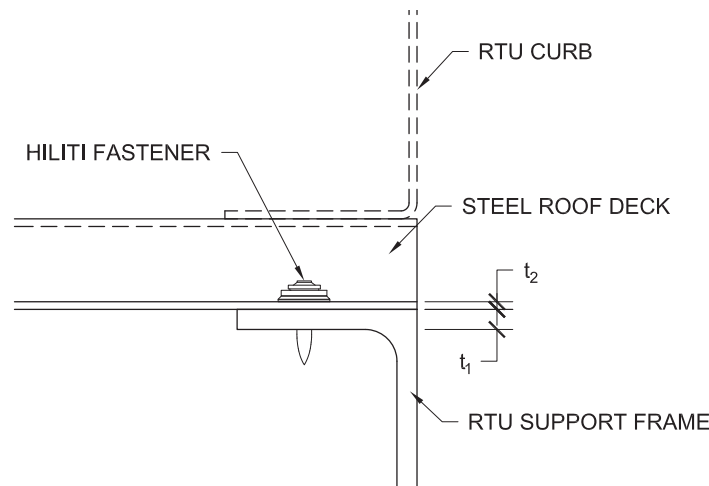


Figure 11.14 Steel Roof Deck to Support Frame PAF Connection

The tension strength of the Hilti X-HSN 24 PAF:

Pullout for PAF:

$$P_{\text{not}} = 8t_{\text{support}} + 0.0888 \leq 1.875 \text{ kip} \quad \text{IAPMO ER-2018 eq H-4}$$

$$P_{\text{not}} = 8(0.125 \text{ in}) + 0.0888 = 1.088 \text{ kip}$$

$$\Phi P_{\text{not}} = 0.65(1.088 \text{ kip}) = 0.708 \text{ kip}$$

Pullover for PAF:

$$P_{\text{not}} = \alpha_w t_w d'_w F_{u1} \quad \text{AISI S100 eq J5.2.3-1}$$

Where

$$\alpha_w = 1.5 \text{ for PAF with washer}$$

$$d'_w = 0.474 \text{ in for a X-HSN 24} \quad \text{IAPMO ER-2018}$$

$$P_{\text{not}} = 1.5(0.0299 \text{ in})(0.474 \text{ in})(65 \text{ ksi}) = 1.382 \text{ kip}$$

$$\Phi P_{\text{not}} = 0.5(1.382 \text{ kips}) = 0.691 \text{ kip}$$

Tensile Strength of PAF:

$$P_{\text{nts}} = 5.033 \text{ kip} \quad \text{IAPMO ER-2018}$$

$$\Phi P_{\text{nts}} = 0.60(5.033 \text{ kip}) = 3.02 \text{ kip}$$

Governing Design Tension Strength:

$$P_n = P_{\text{nts}} = 0.691 \text{ kip}$$

The minimum required number of PAFs to resist the tension of the RTU curb.

$$R_2 / P_{\text{at}} = 1351 \text{ lb} / 691 \text{ lb/PAF} = 2 \text{ PAFs}$$

The minimum of 2 Hilti X-HSN 24 power actuated fasteners along the 10 ft length of the RTU curb is much less than 13 fasteners along the 10 ft sheet length provided by the minimum 36/4 steel roof deck attachment pattern.

$$(10 \text{ ft} / (3 \text{ ft} / 36 \text{ in deck sheet}))(4 \text{ PAF}/36 \text{ in sheet}) = 13 \text{ PAFs} > 3 \text{ PAFs required}$$

The tensile strength of the PAF with the minimum 36/4 attachment pattern is more than adequate to resist the overturning of the RTU unit due to wind or seismic loads.

In this example, combined shear and tension interaction for the Hilti X-HSN 24 fasteners is not performed. The assumed load path for the shear of the RTU unit through the curb to the structure is collected by the steel deck diaphragm. The RTU support frame does not transfer the load to the vertical lateral force resisting system. This support frame redistributes the vertical load of the RTU to the adjacent OWSJ. The shear force is resisted by the steel roof deck, therefore there is no significant shear transfer between the steel deck and the support frame.

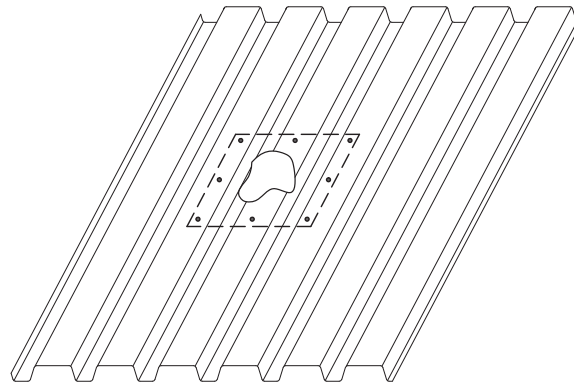
11.4 Miscellaneous Steel Roof Deck Penetrations

Miscellaneous small and mid-size penetrations for mechanical, electrical and plumbing are generally not located on any of the construction documents. These penetrations are generally not located in the design drawings and are cut by the trade requiring the opening during construction. If the designer of record does not provide guidance for reinforcing these penetrations, it is possible that no reinforcement will be installed.

To prevent this, it is important to provide details addressing reinforcing for small miscellaneous penetrations through the steel roof deck.

11.4.1 Small Penetrations

The Steel Deck Institute (SDI) Manual of Construction, 3rd Edition (MOC3) provides guidance for reinforcing small penetrations. The SDI MOC3 recommends for 1½ in deep PLB-36 roof deck that reinforcement is not required when the penetration does not affect more than one rib of the deck. With careful positioning, this equates to a penetration of up to 6 in in diameter. For larger penetrations, the SDI recommends the use of an above deck stiffening plate to limit local displacement of the deck around the hole as shown in Figure 11.15.



| | |
|-----------------------------------------|-----------------------------------------------------------------|
| Up to 6 in Diameter, One Rib Removed | No reinforcing plate required or 0.045 in Thick Plate (min.) |
| 6 in to 8 in Diameter, Two Ribs Removed | 0.045 in Thick Plate (min.) |
| 8 in to 13 in Diameter | 0.057 in Thick Plate (min.) |
| Over 13 in Diameter | Design for reinforcement required |

Figure 11.15 SDI Recommended Reinforcing for Small Penetrations

The SDI does not provide specific recommendations for 3 in deep PLN-24 or PLN3-32 roof decks. These profiles have an 8 in pitch and are often used for longer spans. For these deeper decks it would be reasonable to limit the size of the penetration to 6 inches in diameter and not cut through more than one web of the steel deck. The stiffener plates for penetrations following the schedule for PLB-36 deck is reasonable for larger penetrations up to 13 in.

11.4.2 Mid-Size Penetrations

Mid-size openings are those larger than 13 in or clusters of small holes that structurally constitute a single penetration when considered as a group of holes. Examples of these may be small HVAC duct penetrations, light exhaust fan penetrations, and sump pans. The strength of the roof deck may be significantly compromised at the penetration, therefore requiring reinforcing.

Steel deck can support loads when cantilevered from the supports. This may be an adequate solution for penetrations that support light loads such as skylights, and vent fans. The design of the deck may be treated as a beam on a per foot width basis. It is critical that the steel roof deck have a back span to resist rotation over the joist for the cantilever as shown in Figure 11.16. It is critical that the skylight curb or mechanical unit curb be sufficient to stiffen the edge of the deck to limit local deflection. This can be accomplished by screwing the flange of the curb to each top flute of the steel deck.

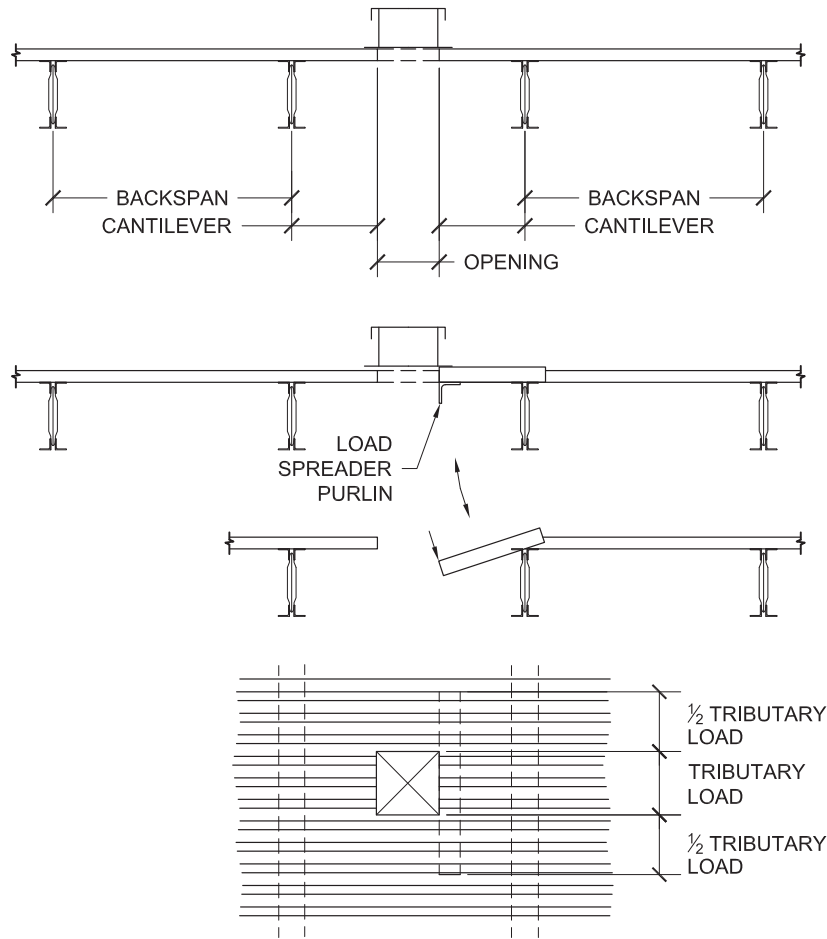


Figure 11.16 Cantilever Support of Curb

It is possible in some conditions to support an opening in the deck when the deck does not have a back span. This may be accomplished using a load distribution purlin that carries the load tributary to the deck requiring support to the adjacent sheets of deck. This is applicable when the deck's bending strength has enough reserve capacity beyond the design loads to carry the additional load from the distribution purlin. The primary disadvantage to this solution is that it typically requires erection labor below and above the opening at the same time to install the load distribution purlin.

For conditions in which the steel deck does not have adequate reserve capacity for distribution purlins to be effective, the deck may be reinforced with C or Z purlins in the deck flutes adjacent to the opening as shown in Figure 11.17. These C or Z purlins are typically made to match the depth of the deck and provide additional bending strength to support the opening.

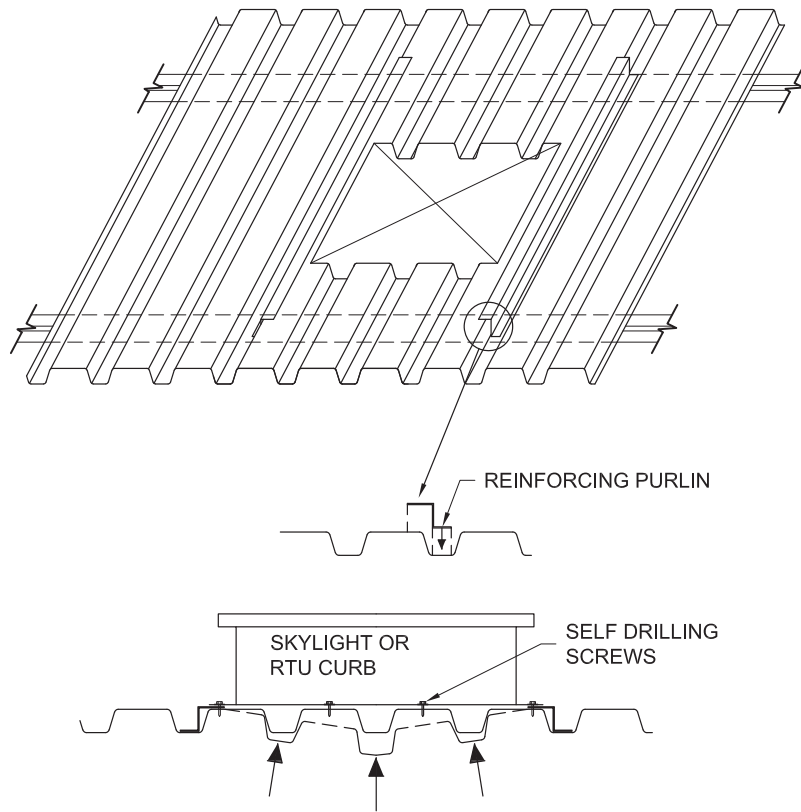


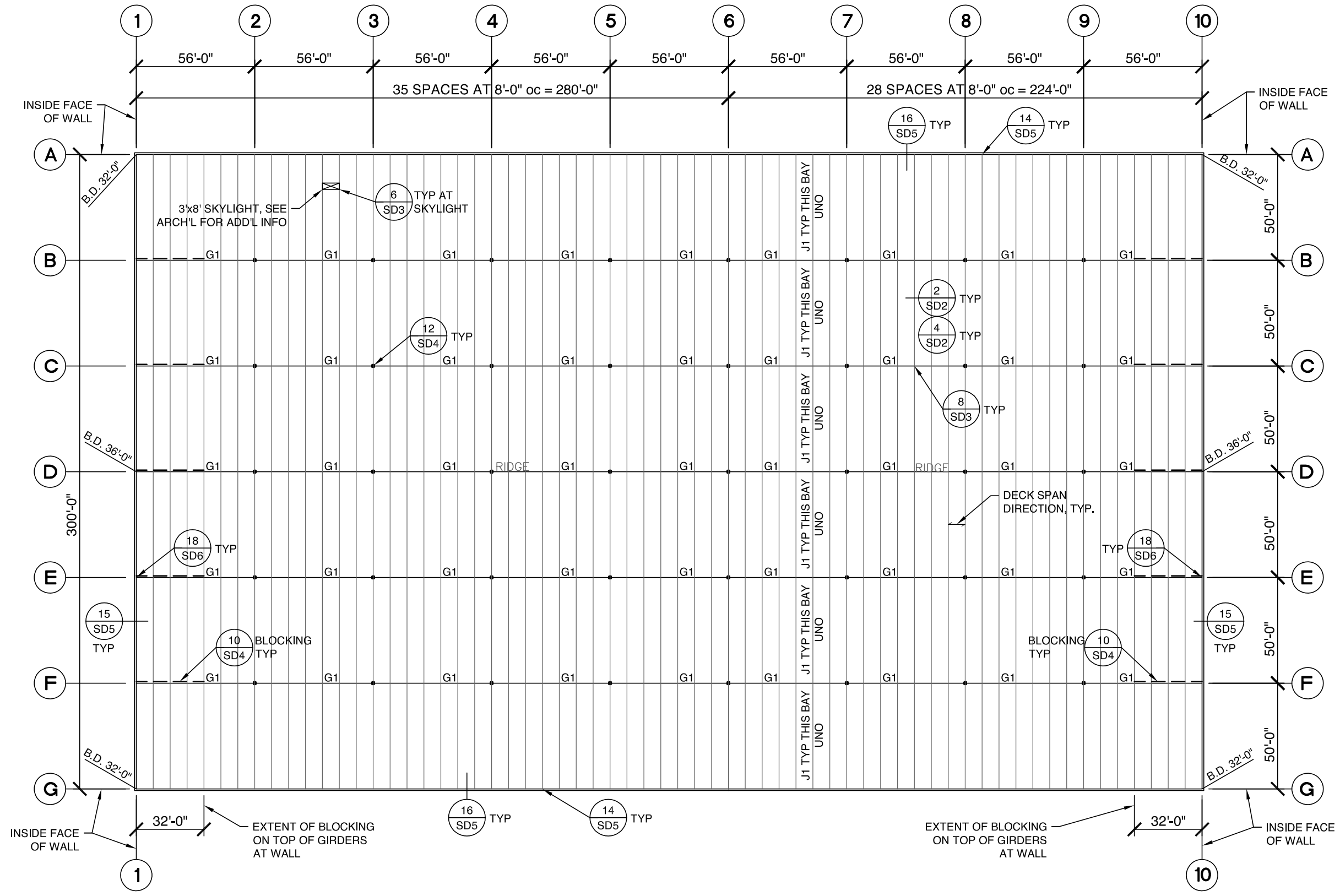
Figure 11.17 Above Deck Reinforcement

Appendix Plans, Notes, Schedules, and Details

The following example plans, notes, schedules, and details reflect the roof structure design developed in Sections 2 to 8 of the design example for the 300 ft by 504 ft warehouse. Common member sizes, weldments, fastening, and bolts shown are representative of common sizes. Appropriate sizes must be determined for any specific project.

Contents of Appendix

| | |
|------------|-------------------------|
| S2.0 | Roof Framing Plan |
| S2.1 | Roof Notes and Schedule |
| S2.2 | Joist Wind Loads |
| S2.3 | Girder Wind Loads |
| S2.4 | Deck Diaphragm Schedule |
| SD1 | General Notes |
| SD2 to SD6 | Details |



ROOF FRAMING PLAN
SCALE 1" = 50'-0"

- ROOF NOTES:**
- SEE SD1 FOR GENERAL NOTES.
 - SEE S2.1 FOR JOIST SCHEDULE, JOIST GIRDER SCHEDULE, AND ROOF JOIST & JOIST GIRDER NOTES.
 - SEE S2.2 & S2.3 FOR WIND LOADING ON JOISTS AND JOIST GIRDERS.
 - SEE S2.4 FOR STEEL ROOF DECK GAGE AND ATTACHMENT SCHEDULE
 - B.D. = BOTTOM OF DECK ELEVATION TO FLOOR SLAB DIRECTLY BELOW

ROOF JOIST & JOIST GIRDER NOTES:

- SEE OPEN WEB STEEL JOIST & JOIST GIRDER GENERAL NOTES ON SHEET SD1 FOR ADDITIONAL REQUIREMENTS.
- DESIGN LOADS:

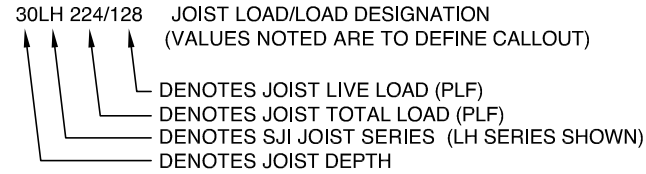
| | JOISTS | GIRDERS |
|------------------------|----------|----------|
| DEAD LOAD: WAREHOUSE | 13.5 PSF | 15.0 PSF |
| LIVE LOAD: (REDUCIBLE) | 20.0 PSF | 20.0 PSF |

WIND LOADS: SEE WIND LOAD DIAGRAMS AND SCHEDULES

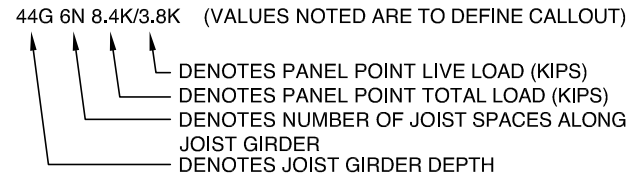
- JOIST AND JOIST GIRDER DEFLECTION CRITERIA:
AT WAREHOUSE: L/240 FOR LIVE LOAD
- ROOF JOISTS AND ROOF JOIST GIRDERS TO BE FABRICATED WITH STANDARD SJI CAMBER, U.N.O.
- SEE JOIST SCHEDULE AND JOIST GIRDER SCHEDULE FOR DESIGNATIONS. DESIGNATIONS INCLUDE JOIST & JOIST GIRDER SELF-WEIGHT.

A. DESIGNATION DEFINITIONS:

1. JOISTS:



2. JOIST GIRDERS:



SEE ROOF FRAMING PLAN FOR ADDITIONAL POINT LOADS TO JOIST GIRDERS WHERE A GIVEN JOIST SPACE IS WIDER THAN TYPICAL JOIST SPACE.

- AXIAL LOADS: SEE JOIST & JOIST GIRDER SCHEDULES FOR SEISMIC AXIAL LOADS AND WIND AXIAL LOADS, WHERE NOTED. AXIAL LOADS GIVEN IN SCHEDULE ARE NOMINAL CODE LEVEL LOADS (1.0E & 1.0W) FOR REQUIRED LOAD COMBINATIONS.
- ADD-LOADS TO BE INCLUDED IN DESIGN:
 - JOIST ADD-LOAD:
 - WAREHOUSE JOIST: DEAD LOAD ADD-LOAD = 500LB
 - JOIST GIRDER ADD-LOAD:
 - WAREHOUSE JOIST GIRDER: DEAD LOAD ADD-LOAD = 2000LB
- SEE JOIST SCHEDULE FOR BEND-CHECK LOAD REQUIREMENTS TO TOP AND BOTTOM CHORD WHERE NOTED.
- CONTRACTOR TO PROVIDE FIELD INSTALLED WEBS AT CONCENTRATED LOADS BETWEEN PANEL POINTS GREATER THAN THE FOLLOWING:
 - TOP CHORD LOAD: GREATER OF 100LB OR TOP CHORD BEND-CHECK LOAD IN SCHEDULE.
 - BOTTOM CHORD LOAD: GREATER OF 100LB OR BOTTOM CHORD BEND-CHECK LOAD IN SCHEDULE.

FOR ADDITIONAL INFO SEE DETAIL 3/SD2.
- MECHANICAL EQUIPMENT SUPPORTED BY ROOF (A.C. UNITS, EXHAUST FANS, SUPPLY FANS, HVLS FANS AND SIMILAR):
 - JOISTS IMPACTED BY MECHANICAL EQUIPMENT SHOWN ON STRUCTURAL ROOF FRAMING PLAN SHALL BE DESIGNED FOR 2/3 THE TOTAL SPECIFIED WEIGHT AT EACH JOIST SUPPORTING THE UNIT. LOAD FROM MECHANICAL UNIT SHALL BE TREATED AS AN ADD-LOAD AND IS IN ADDITION TO ADD-LOADS IN JOIST SCHEDULE AND OTHER JOIST NOTES.
 - JOIST GIRDERS IMPACTED BY MECHANICAL EQUIPMENT SHOWN ON STRUCTURAL ROOF FRAMING PLAN SHALL BE DESIGNED FOR THE TOTAL SPECIFIED WEIGHT OF THE UNIT AND SHALL ASSUME THE MECHANICAL UNITS MAY BE LOCATED ANYWHERE ALONG THE LENGTH OF THE SUPPORTING JOIST FOR WORST CASE DESIGN CONDITION. TOTAL SPECIFIED UNIT WEIGHT MAY BE TREATED AS AN ADD-LOAD AT THE MANUFACTURER'S DISCRETION IN LIEU OF POINT LOADS AT PANEL POINTS.
- SEE ROOF FRAMING PLAN FOR ADDITIONAL POINT LOADS AND HANGING LOADS TO BE INCLUDED IN THE JOIST AND JOIST GIRDER DESIGN (WHERE NOTED).

- SPRINKLER MAINS AND SWAY BRACES:
 - VERIFY LOCATION AND SIZE OF ALL SPRINKLER MAINS AND SWAY BRACE LOADS WITH FIRE PROTECTION DRAWINGS.
 - JOIST MANUFACTURER TO DESIGN AND DETAIL JOISTS AND JOIST GIRDERS FOR THE FOLLOWING SPRINKLER MAINS AND SWAY BRACE LOADS:
 - 4" AND LARGER SPRINKLER MAINS AND THEIR SWAY BRACE LOADS.
 - THE WEIGHT OF THE PIPE WITH WATER, AS NOTED ON FIRE PROTECTION PLANS, TO BE USED. IF THE LOADS ARE NOT CLEARLY SHOWN ON THE FIRE PROTECTION PLANS, USE THE FOLLOWING LOADS:
 - 4" S.M. = 18 PLF
 - 6" S.M. = 32 PLF
 - 8" S.M. = 50PLF
 - SPRINKLER RISERS:
 - WHERE BRACKETS ARE PROVIDED TO SUPPORT VERTICAL WEIGHT OF THE SPRINKLER RISER, NO ADDITIONAL LOAD IS REQUIRED.
 - WHERE BRACKETS ARE NOT PROVIDED TO SUPPORT THE SPRINKLER RISER, DESIGN TRUSS FOR THE WEIGHT OF THE RISER SHOWN ON FIRE PROTECTION PLANS (2250LB IF NOT SHOWN) AT RISER LOCATION.

| ROOF JOIST SCHEDULE ⁽¹⁾⁽²⁾ | | | | |
|---------------------------------------|--------------|-----------------------------------------------|--------------------------------------------|----------|
| JOIST MARK | DESIGNATION | SEISMIC AXIAL LOAD 1.0E (KIPS) ⁽³⁾ | WIND AXIAL LOAD 1.0W (KIPS) ⁽³⁾ | COMMENTS |
| J1 | 32LH 236/128 | 22.5 | 3.5 | |

- SEE ROOF JOIST & JOIST GIRDER NOTES FOR ADDITIONAL REQUIREMENTS.
- SEE WIND LOAD DIAGRAM FOR NET WIND UPLIFT LOADS AND DOWNWARD WIND LOADS TO BE INCLUDED IN JOIST DESIGN.
- TOP CHORD AXIAL LOAD. TENSION OR COMPRESSION LOAD.

| ROOF JOIST GIRDER SCHEDULE ⁽¹⁾⁽²⁾ | | | | |
|----------------------------------------------|-------------------|-----------------------------------------------|--------------------------------------------|----------|
| JOIST GIRDER MARK | DESIGNATION | SEISMIC AXIAL LOAD 1.0E (KIPS) ⁽³⁾ | WIND AXIAL LOAD 1.0W (KIPS) ⁽³⁾ | COMMENTS |
| G1 | 56G 7N 10.8K/4.8K | 140 | 20 | |

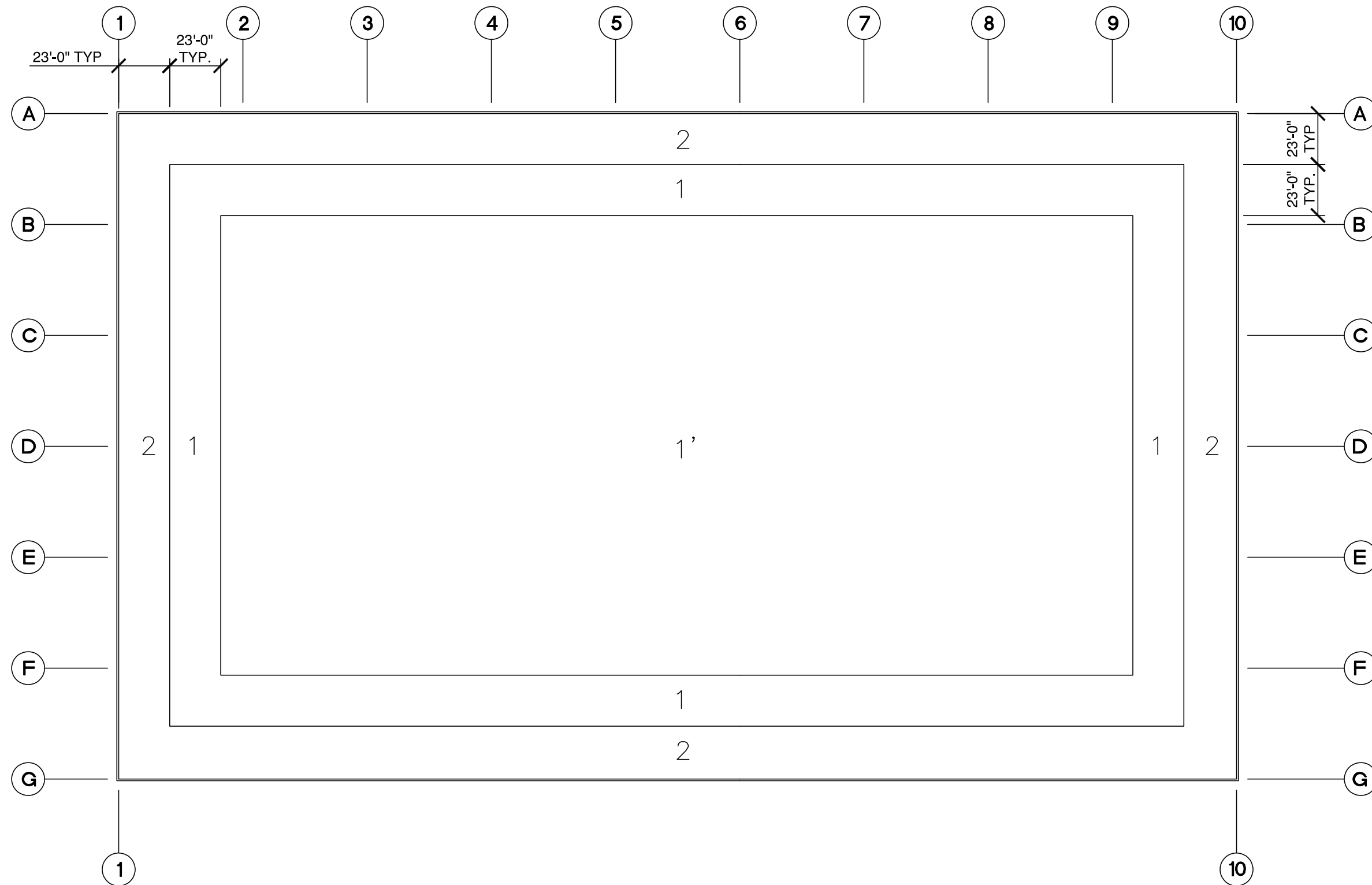
- SEE ROOF JOIST & JOIST GIRDER NOTES FOR ADDITIONAL REQUIREMENTS.
- SEE WIND LOAD DIAGRAM FOR NET WIND UPLIFT LOADS TO BE INCLUDED IN JOIST GIRDER DESIGN.
- TOP CHORD AXIAL LOAD. TENSION OR COMPRESSION LOAD.



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TITLE: ROOF NOTES & SCHEDULE
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S2.1

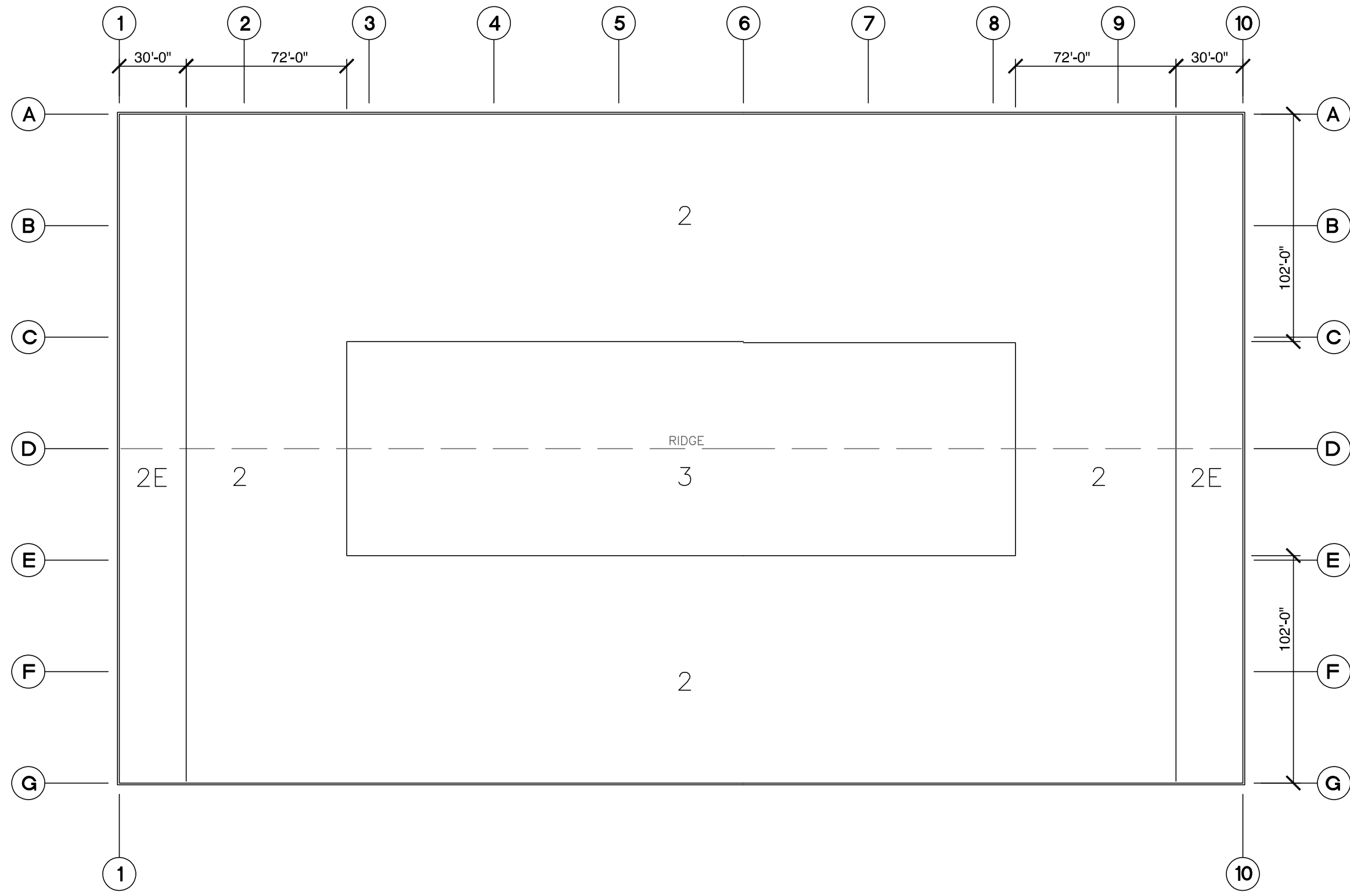


JOIST WIND LOAD DIAGRAM
 NO SCALE

| JOIST WIND LOAD SCHEDULE | | | | | |
|--------------------------|--------------------------------------|--------|--------|-----------------------------------------|--------|
| MEMBER | NET WIND UPLIFT (PSF) ⁽¹⁾ | | | DOWNWARD WIND 1.0W (PSF) ⁽²⁾ | |
| | ZONE 1' | ZONE 1 | ZONE 2 | ZONE 1' & 1 | ZONE 2 |
| JOIST | 2.7 | 7.5 | 12.3 | 16.0 | 17.8 |

(1) NET WIND UPLIFT IS THE RESULT OF THE 0.6D+0.6W LOAD COMBINATION.
 (2) DOWNWARD WIND LOADS ARE NOMINAL LOADS (STRENGTH LEVEL). MANUFACTURER TO USE THESE LOADS IN THE APPROPRIATE LOAD COMBINATIONS.





JOIST GIRDER WIND LOAD DIAGRAM
 NO SCALE

| JOIST GIRDER WIND LOAD SCHEDULE ⁽²⁾ | | | |
|------------------------------------------------|--------------------------------------|--------|--------|
| MEMBER | NET WIND UPLIFT (PSF) ⁽¹⁾ | | |
| | ZONE 2E | ZONE 2 | ZONE 3 |
| JOIST GIRDER | 8.0 | 3.4 | 0.0 |

(1) NET WIND UPLIFT IS THE RESULT OF THE 0.6D+0.6W LOAD COMBINATION.
 (2) WIND LOADS TO JOIST GIRDERS ARE BASED ON MWFRS LOADS AS PERMITTED BY ASCE 7 SEC. 30.2.3, BASED ON JOIST GIRDER TRIBUTARY AREA BEING GREATER THAN 700 ft².



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ONTARIO, CA

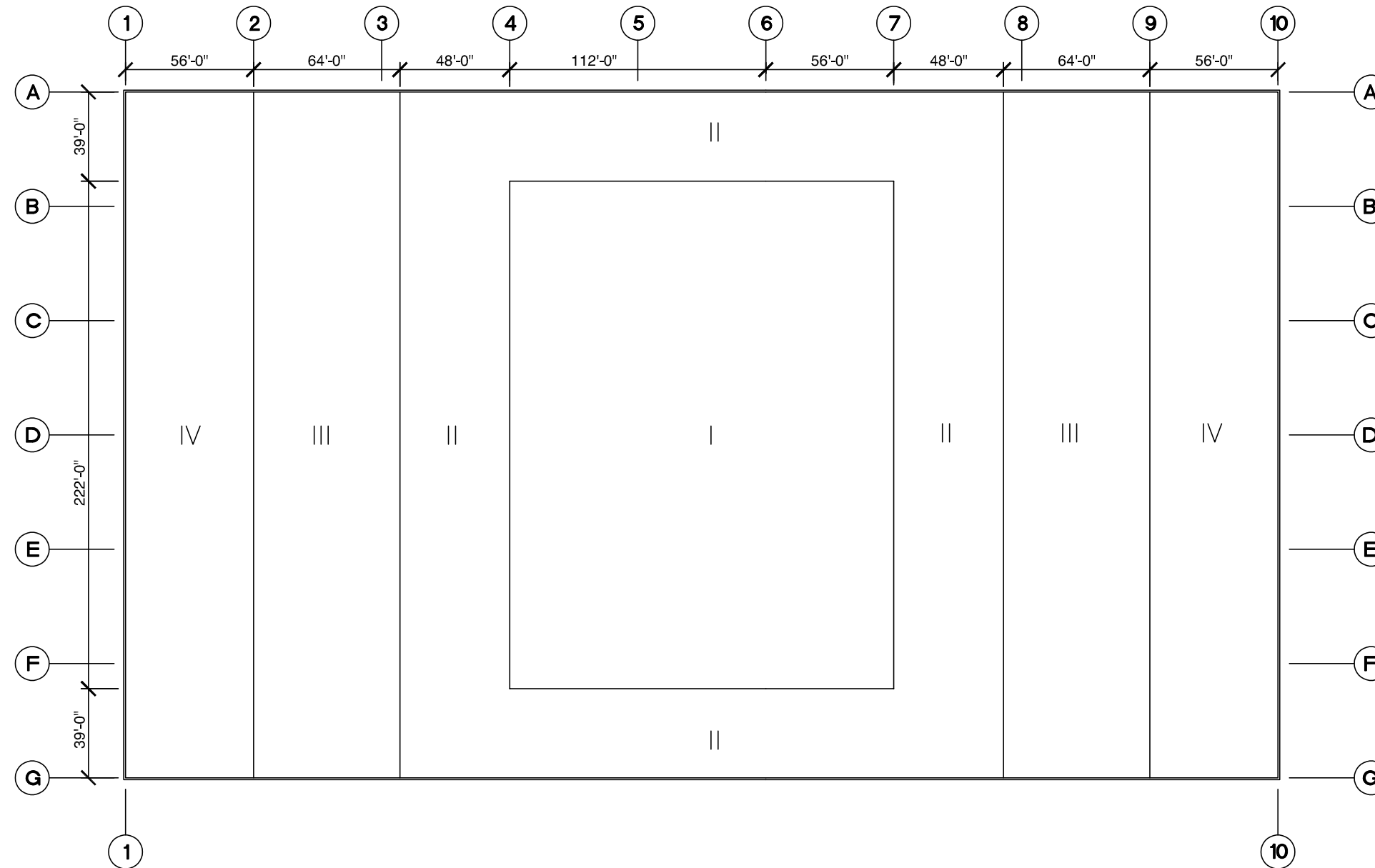
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GIRDER WIND LOADS

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S2.3

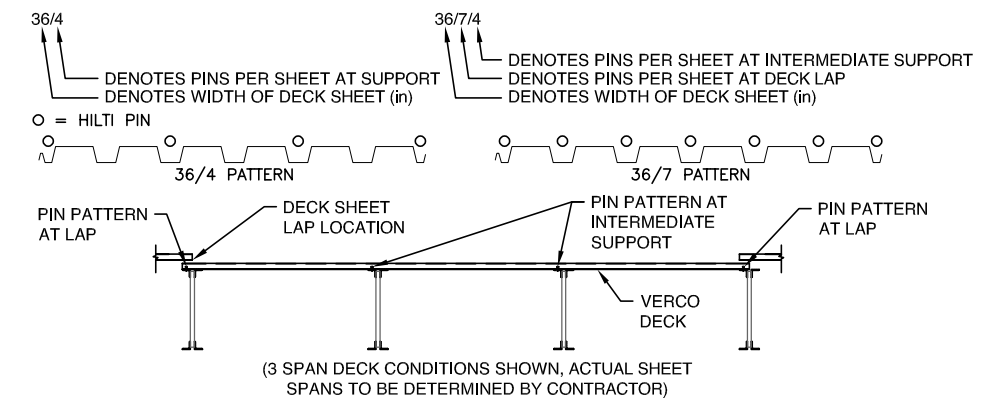


STEEL ROOF DECK ATTACHMENT DIAGRAM
NO SCALE

| STEEL ROOF DECK GAGE AND ATTACHMENT SCHEDULE | | | | |
|----------------------------------------------|----------------------------|--------------------------------------------------------|-----------------------------------------------------------------|-----------------------------------------------------|
| ZONE | VERCO PLB-36 DECK GAGE (1) | PIN ATTACHMENT PATTERN AT PERPENDICULAR SUPPORT (2)(4) | BOUNDARY PIN SPACING WHERE DECK IS PARALLEL TO WALL/SUPPORT (2) | DECK SIDE LAP CONNECTION AND SPACING EACH SHEET (3) |
| I | 22 | 36/4 | 16" o.c. | VSC2 @ 24" o.c. |
| II | 22 | 36/7/4 | 12" o.c. | VSC2 @ 18" o.c. |
| III | 22 | 36/7/4 | 8" o.c. | VSC2 @ 8" o.c. |
| IV | 20 | 36/7/4 | 7" o.c. | VSC2 @ 8" o.c. |

- (1) DECK TO BE VERCO DECK GRADE 50 PER IAPMO ER 2018. SUBSTITUTION OF ANOTHER MANUFACTURER'S DECK IS NOT ACCEPTABLE.
 (2) PINS TO BE HILTI POWDER ACTUATED FASTENERS (PAF) PER ICC ESR 2776. USE HILTI X-HSN 24 PINS FOR SUPPORTS 1/8" THICK TO 3/8" THICK. HILTI X-ENP-19 PINS MAY BE USED FOR SUPPORTS 1/4" OR THICKER. WELDING OF DECK IS NOT PERMITTED.
 (3) ATTACHMENT OF THE DECK SIDE LAPS TO BE WITH VERCO PUNCHLOK II TOOL (VSC2 CONNECTION).

(4) SEE DIAGRAMS BELOW FOR ILLUSTRATION OF FASTENER PATTERNS PER SHEET OF PLB DECK.



(3 SPAN DECK CONDITIONS SHOWN, ACTUAL SHEET SPANS TO BE DETERMINED BY CONTRACTOR)



GENERAL REQUIREMENTS:

- DESIGN CODE: 2018 INTERNATIONAL BUILDING CODE (IBC)
- RISK CATEGORY: II
- SEISMIC:
SEISMIC DESIGN CATEGORY: D
SITE CLASS: D
 $S_s = 1.5$ $S_{DS} = 1.0$
 $S_1 = 0.6$ $S_{D1} = 0.68$
- WIND:
DESIGN WIND SPEED: 95 MPH (3 SECOND GUST)
WIND EXPOSURE: C

STEEL DECK GENERAL NOTES:

- STEEL DECK SHALL BE MANUFACTURED BY VERCO DECKING, INC. IN ACCORDANCE WITH IBC SECTION 2210.1.1 AND PRODUCT EVALUATION REPORT IAPMO ER-2018
- STEEL DECK SHALL BE FORMED FROM STEEL SHEETS CONFORMING TO ASTM A653 SS GRADE 50 G60 GALVANIZED.
- SEE ARCHITECTURAL PLANS FOR AREAS WHERE BOTTOM OF DECK IS TO BE PRIMER PAINTED AND PRIMER COLOR (WHERE REQUIRED).
- SEE "STEEL ROOF DECK ATTACHMENT DIAGRAM" & "STEEL ROOF DECK GAGE AND ATTACHMENT SCHEDULE" FOR DECK TYPE, GAGE, AND ATTACHMENT.
- SUBMIT STEEL DECK SHOP DRAWINGS TO ARCHITECT AND STRUCTURAL ENGINEER FOR REVIEW AND APPROVAL. SHOP DRAWINGS SHALL SHOW TYPE OF DECK, LAYOUT OF DECK, AND ATTACHMENT. THE SIZE AND LOCATION OF ANY OPENINGS WITH A WIDTH GREATER THAN 1'-0" SHALL BE SHOWN.

COMMON STEEL DECK OPTIONS: (ALTERNATE TO NOTE 2)*

- STEEL DECK SHALL BE FORMED FROM STEEL SHEETS CONFORMING TO ASTM A1008 SS GRADE 50 GRAY PRIMER PAINTED TOP AND BOTTOM.
- STEEL DECK SHALL BE FORMED FROM STEEL SHEETS CONFORMING TO ASTM A653 SS GRADE 50 G30 GALVANIZED WITH PRIMER PAINTED BOTTOM SURFACE.
- STEEL DECK SHALL BE FORMED FROM STEEL SHEETS CONFORMING TO ASTM A653 SS GRADE 50 G90 GALVANIZED.

* WHITE OR GRAY PRIMER PAINT MAY BE SPECIFIED ON ANY GALVANIZED STEEL DECK. WHITE PRIMER PAINT IS NOT AVAILABLE ON BARE (NON-GALVANIZED) STEEL DECK.

STRUCTURAL STEEL GENERAL NOTES:

- ALL MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE LATEST EDITION OF THE AISC STEEL CONSTRUCTION MANUAL (AISC 360), INCLUDING THE "SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS" AND THE CODE OF STANDARD PRACTICE.
- MATERIAL:
ANGLES, BARS, PLATES, C, S, & M: ASTM A36 (UNLESS OTHERWISE NOTED)
RECTANGULAR AND SQUARE TUBE MEMBERS (HSS): ASTM A500 GRADE B $F_y = 46$ KSI
- SUBMIT STRUCTURAL STEEL SHOP DRAWINGS TO STRUCTURAL ENGINEER FOR REVIEW AND APPROVAL PRIOR TO FABRICATION.
- BOLTS TO BE ASTM A307 UNLESS OTHERWISE NOTED.

OPEN WEB STEEL JOIST & JOIST GIRDER GENERAL NOTES:

- JOISTS & JOIST GIRDERS TO BE ENGINEERED AND FABRICATED BY STEEL JOIST MANUFACTURER IN ACCORDANCE WITH IBC SECTION 2207. THE MANUFACTURER SHALL BE AN APPROVED SJI (STEEL JOIST INSTITUTE) FABRICATOR, VULCRAFT (OR EQUAL).
- THE MANUFACTURER SHALL DESIGN JOISTS AND JOIST GIRDERS FOR LOADING SHOWN IN THE SCHEDULES, PLUS ADDITIONAL CONCENTRATED LOADS, AXIAL LOADS, ETC. AS NOTED ON THE STRUCTURAL FRAMING PLANS AND STRUCTURAL DETAILS.
- STEEL JOIST MANUFACTURER TO SUBMIT PLACEMENT PLANS FOR REVIEW BY STRUCTURAL ENGINEER, ARCHITECT, AND GENERAL CONTRACTOR PRIOR TO FABRICATION OF THE JOISTS & JOIST GIRDERS.
- WHEN DEFERRED SUBMITTAL IS REQUIRED: CONTRACTOR TO COORDINATE WITH STEEL JOIST MANUFACTURER TO PROVIDE FINAL PLACEMENT PLANS AND CALCULATIONS (CALCULATIONS SHALL BE STAMPED/SIGNED BY A LICENSED PROFESSIONAL ENGINEER IN THE STATE WHERE THE PROJECT IS LOCATED), FOR SUBMITTAL TO THE BUILDING DEPARTMENT BY THE CONTRACTOR. THE ABOVE CALCULATIONS TO INCLUDE ALL LOADING CONDITIONS SHOWN ON DRAWINGS AND DETAILS. (STEEL JOIST PLACEMENT PLANS DO NOT REQUIRE THE SEAL AND SIGNATURE OF THE JOIST MANUFACTURER'S REGISTERED DESIGN PROFESSIONAL, IBC SECTION 2207.4).
- SJI (STEEL JOIST INSTITUTE) TERMS:
ADD-LOAD: A SINGLE VERTICAL CONCENTRATED LOAD THAT OCCURS AT ANY ONE PANEL POINT ALONG THE JOIST OR JOIST GIRDER (TOP CHORD OR BOTTOM CHORD). THIS IS IN ADDITION TO LOADS IN THE JOIST OR JOIST GIRDER DESIGNATION.
BEND-CHECK LOAD: A VERTICAL CONCENTRATED LOAD USED TO DESIGN THE JOIST OR JOIST GIRDER CHORD FOR THE ADDITIONAL BENDING STRESSES RESULTING FROM THIS LOAD BEING APPLIED AT ANY LOCATION BETWEEN THE JOIST OR JOIST GIRDER PANEL POINTS. THIS LOAD SHALL ALREADY BE ACCOUNTED FOR IN THE SPECIFIED DESIGNATION LOADS, UNIFORM LOADS, OR ADD-LOAD AND IS USED ONLY FOR THE ADDITIONAL BENDING CHECK IN THE CHORD.
- JOISTS AND JOIST GIRDERS SHALL BE COATED WITH THE MANUFACTURER'S STANDARD GRAY PRIMER, UNLESS SPECIFICALLY DIRECTED BY CONTRACTOR TO FURNISH UNPRIMED. PRIMER TO BE 1 MIL NOMINAL THICKNESS.
A. FOR COMPOSITE JOISTS, CONTRACTOR TO COORDINATE WITH JOIST MANUFACTURER IF JOISTS ARE TO BE FURNISHED UNPRIMED OR PRIMED. IF COMPOSITE JOISTS ARE TO RECEIVE PRIMER, CONTRACTOR TO COORDINATE WITH JOIST MANUFACTURER REGARDING PRIMING SCOPE AND IF TOP CHORDS ARE TO BE LEFT UNPRIMED.
- MANUFACTURER TO DESIGN AND FURNISH BRIDGING AND OR BRACES FOR JOISTS AND JOIST GIRDERS AS REQUIRED, UNLESS NOTED OTHERWISE.
- MANUFACTURER TO PROVIDE SLOPED BEARING SEATS WHERE JOIST OR JOIST GIRDER SLOPE EXCEEDS 1/4" PER FOOT (IN SPAN DIRECTION).
- ROOF JOISTS LOCATED AT A COLUMN LINE, AND ROOF JOISTS EACH SIDE OF A COLUMN LINE (WHERE JOIST DOES NOT ALIGN WITH COLUMN LINE) ARE TO BE PROVIDED WITH ERECTION BOLTS PER OSHA AND SJI REQUIREMENTS.

DATE: 2021

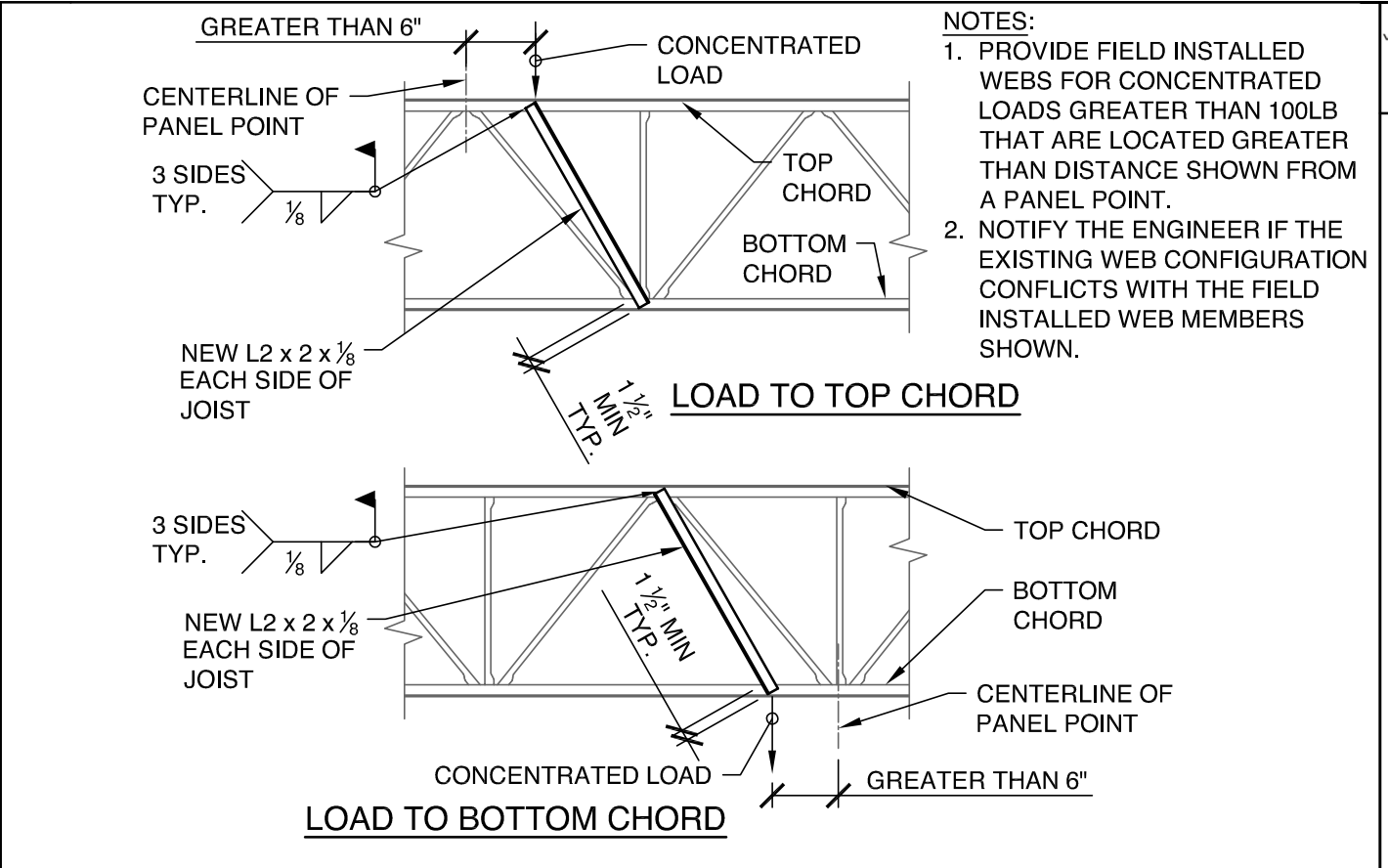
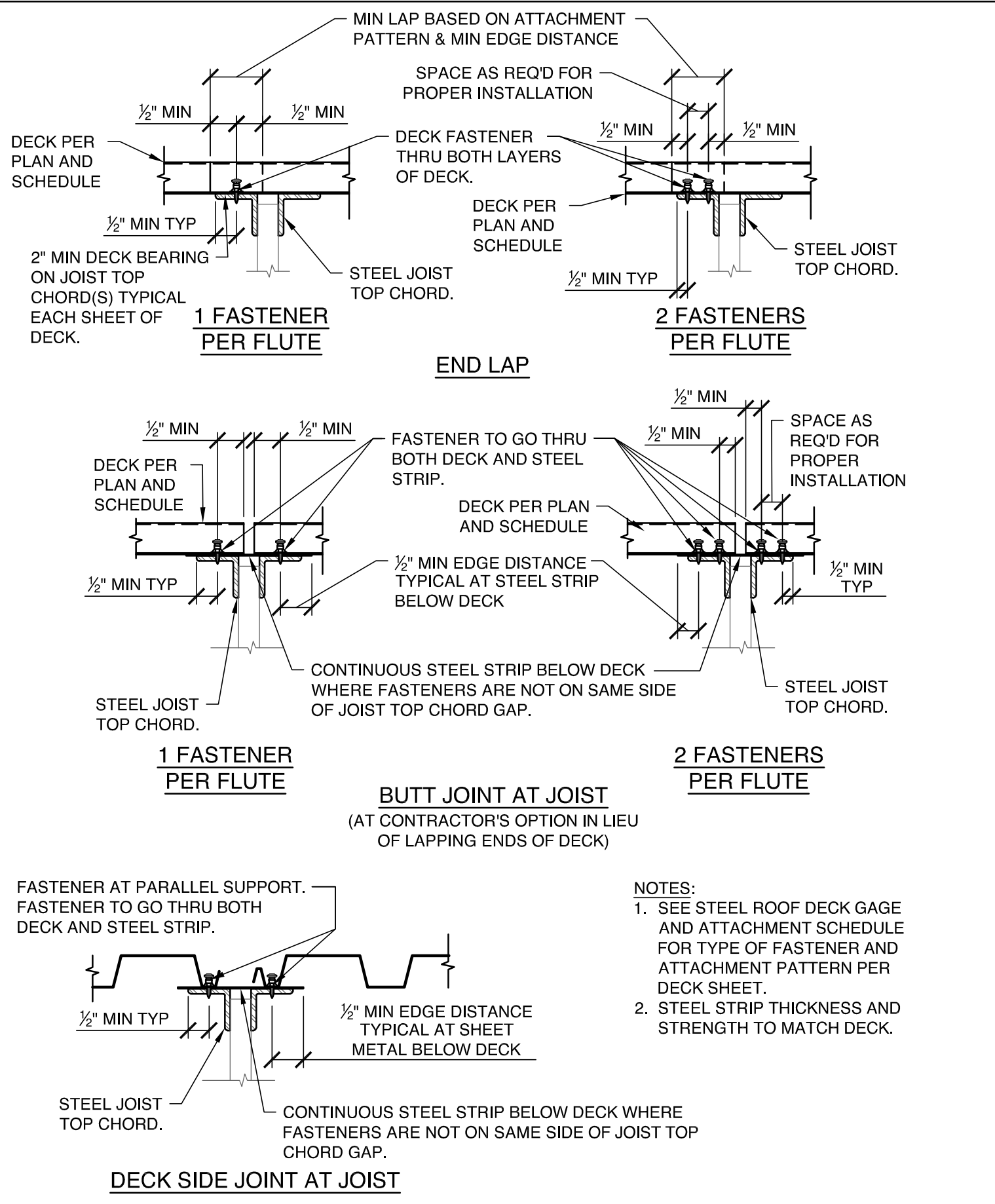
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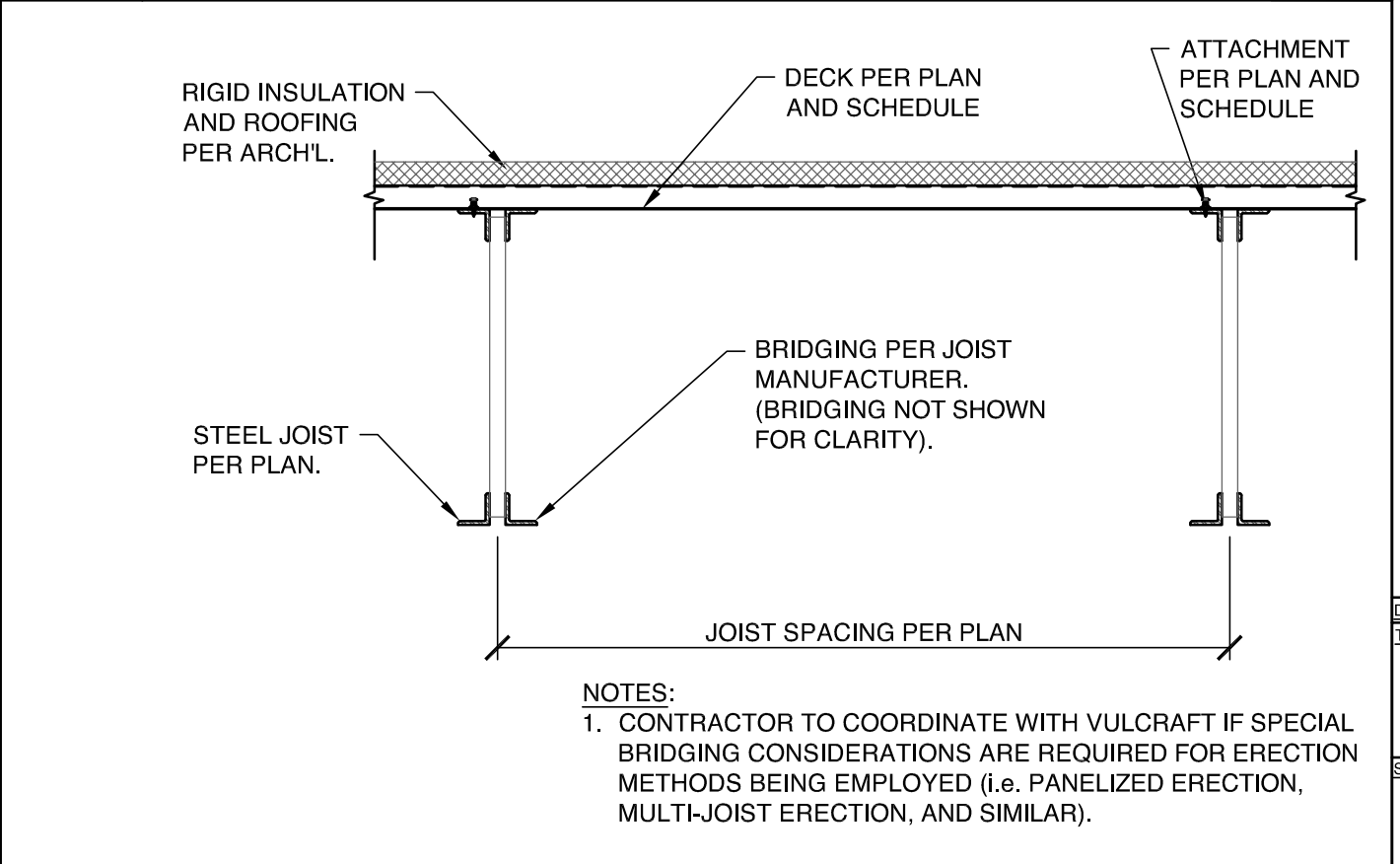
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SD1



FIELD INSTALLED WEBS
STARTING POINT DETAIL

3



SECTION AT JOIST
STARTING POINT DETAIL

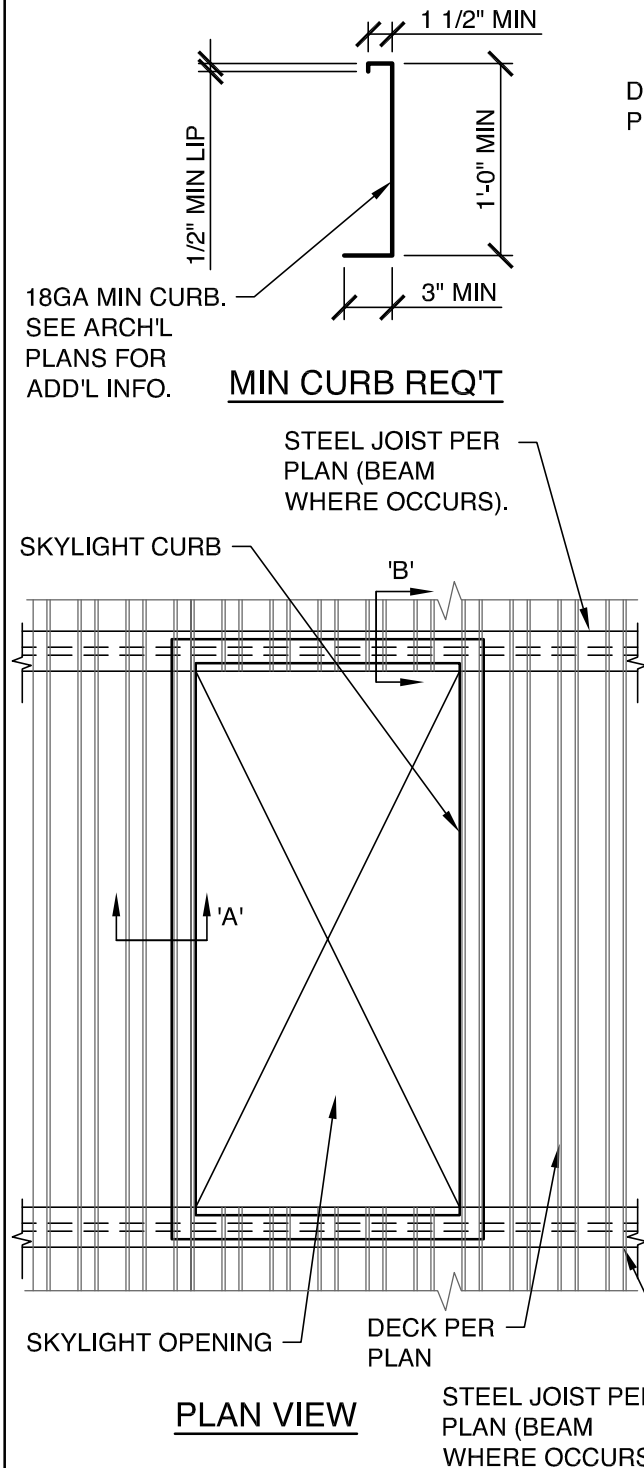
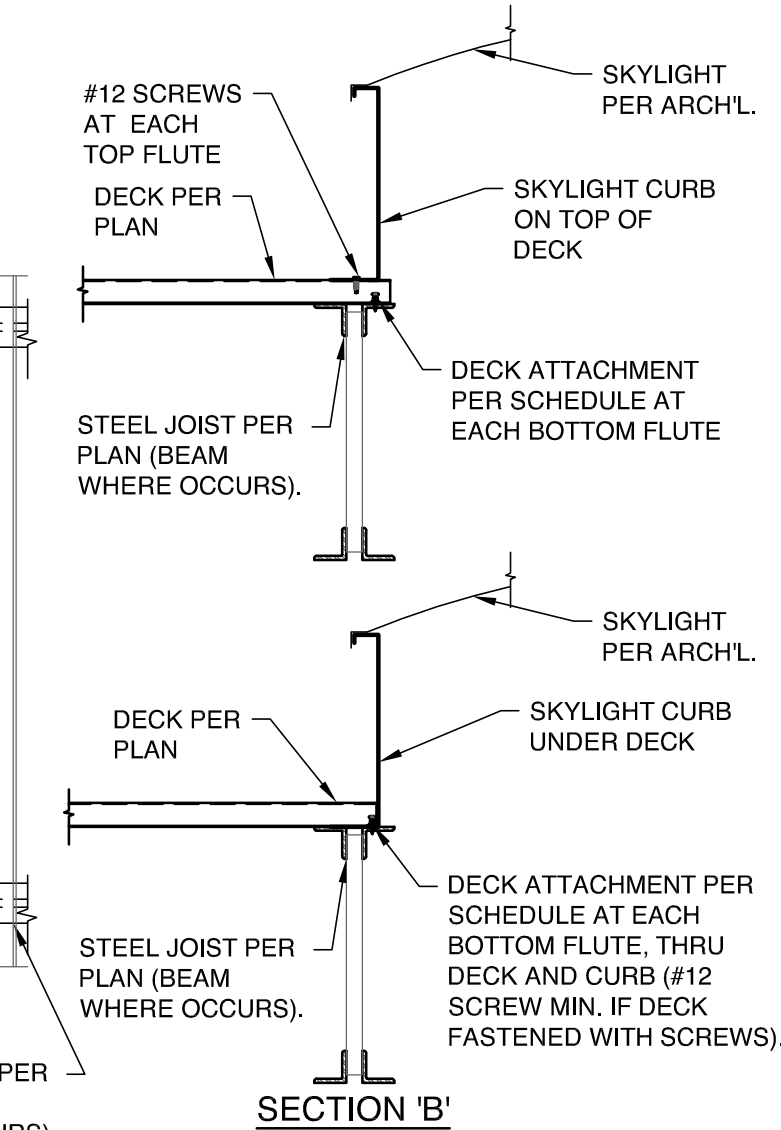
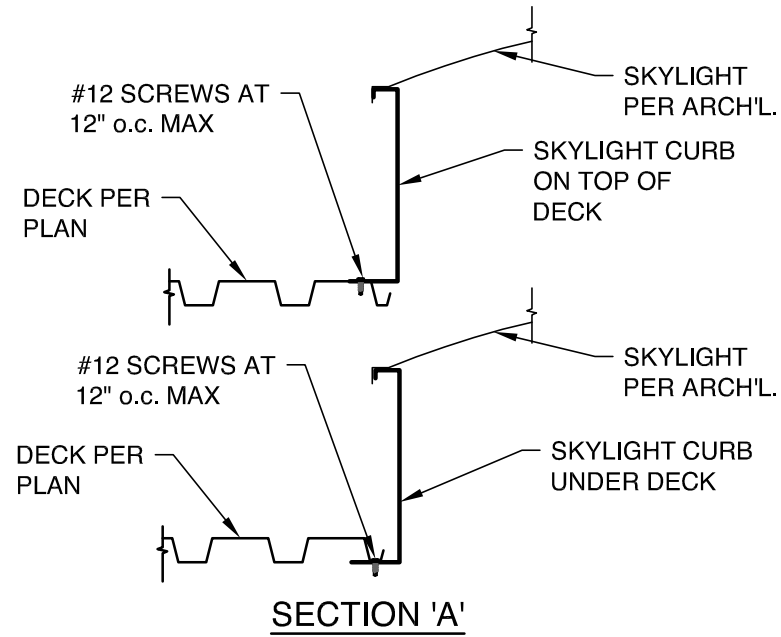
4

METAL DECK TO JOIST CONNECTION
STARTING POINT DETAIL

2

NOTES:

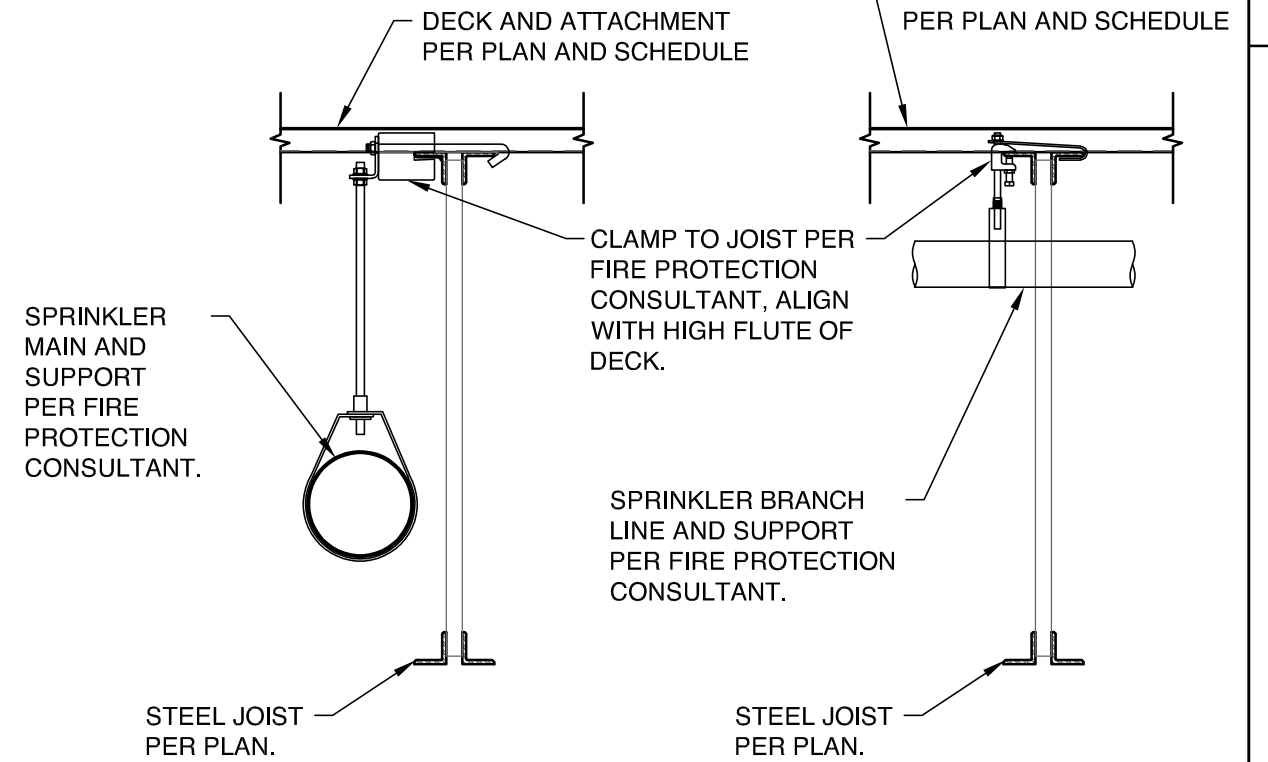
- INSULATION AND CANT STRIPS NOT SHOWN FOR CLARITY. SEE ARCH'L PLANS FOR ADD'L INFO.
- CURB SHOWN IN THIS DETAIL IS THE MINIMUM REQ'T FOR SKYLIGHT WITHOUT STEEL SUPPORT BELOW DECK. SEE ARCH'L FOR ADD'L REQ'TS.



SKYLIGHT OPENING

STARTING POINT DETAIL

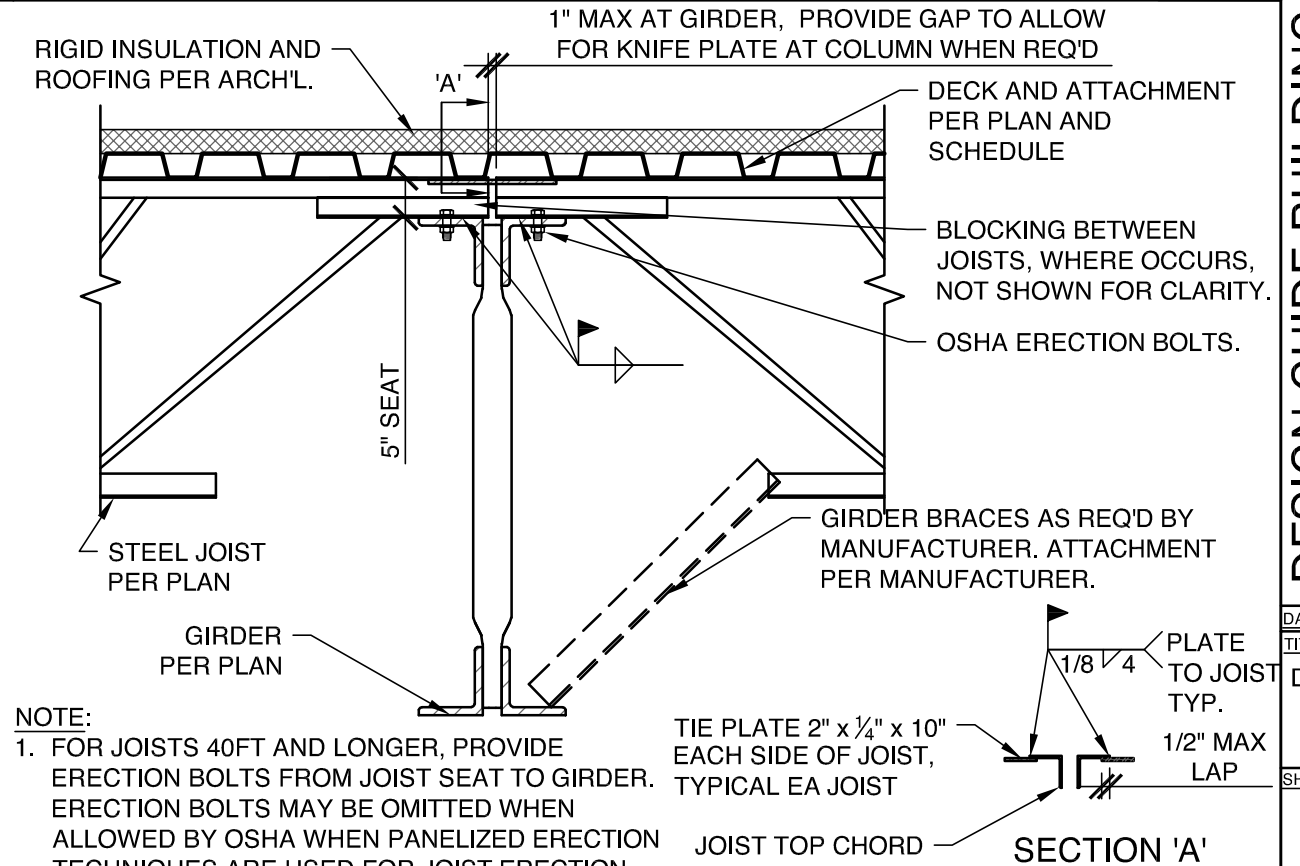
6



SPRINKLER LINES TO JOIST

STARTING POINT DETAIL

3



NOTE:

- FOR JOISTS 40FT AND LONGER, PROVIDE ERECTION BOLTS FROM JOIST SEAT TO GIRDER. ERECTION BOLTS MAY BE OMITTED WHEN ALLOWED BY OSHA WHEN PANELIZED ERECTION TECHNIQUES ARE USED FOR JOIST ERECTION.

STEEL JOIST TO JOIST GIRDER CONNECTION

STARTING POINT DETAIL

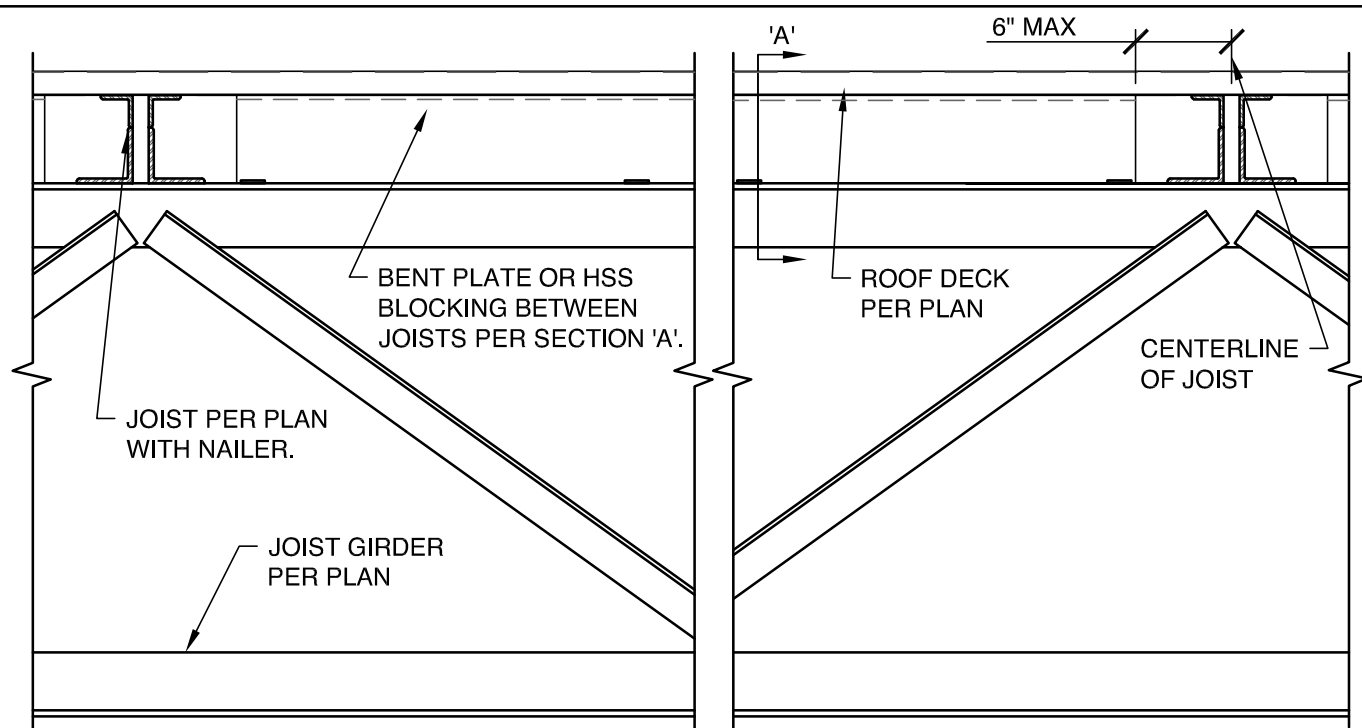
8

DESIGN GUIDE BUILDING
ONTARIO, CA

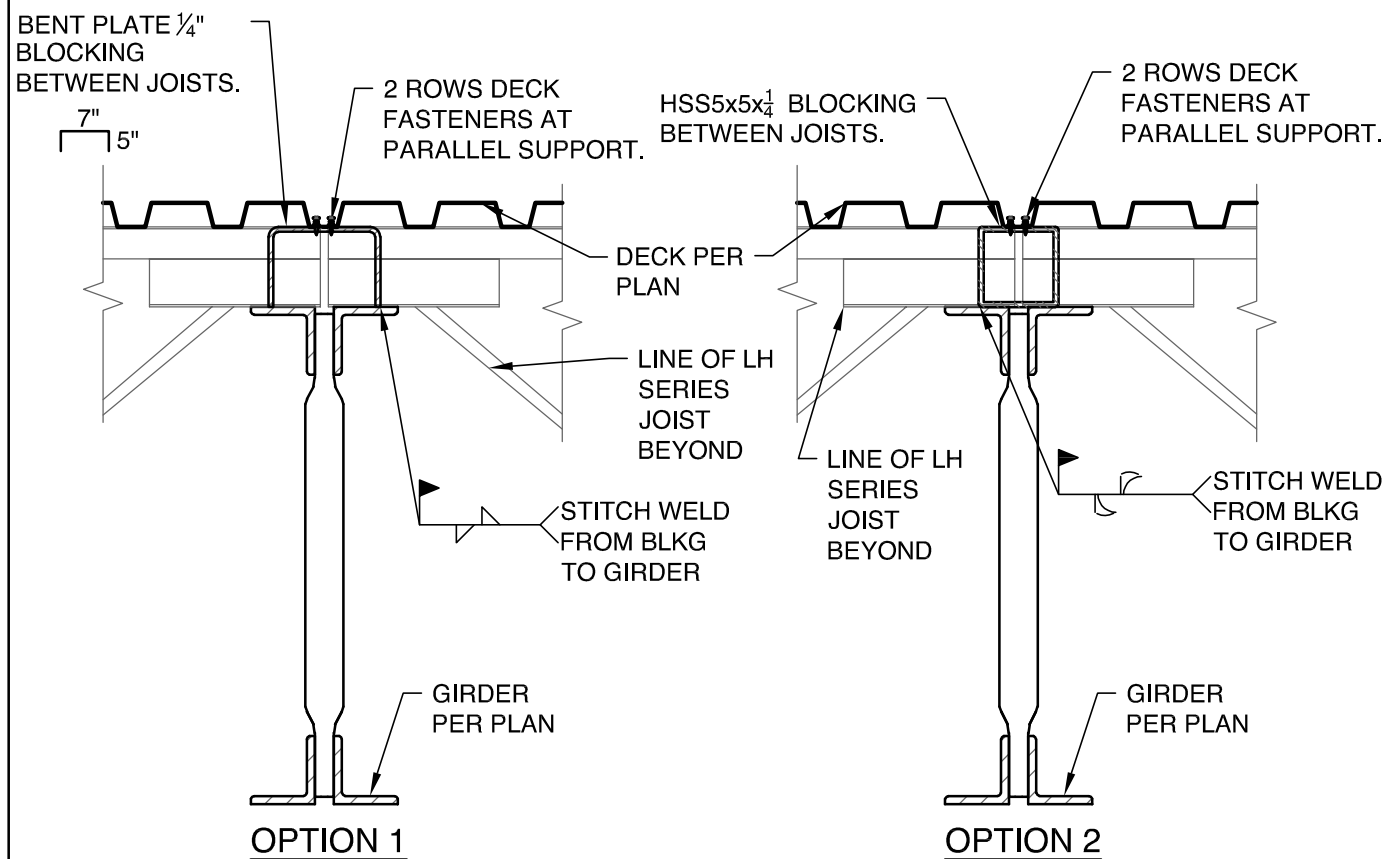
DATE: 2021
TITLE: DETAILS
NO SCALE
SHEET:

SD3





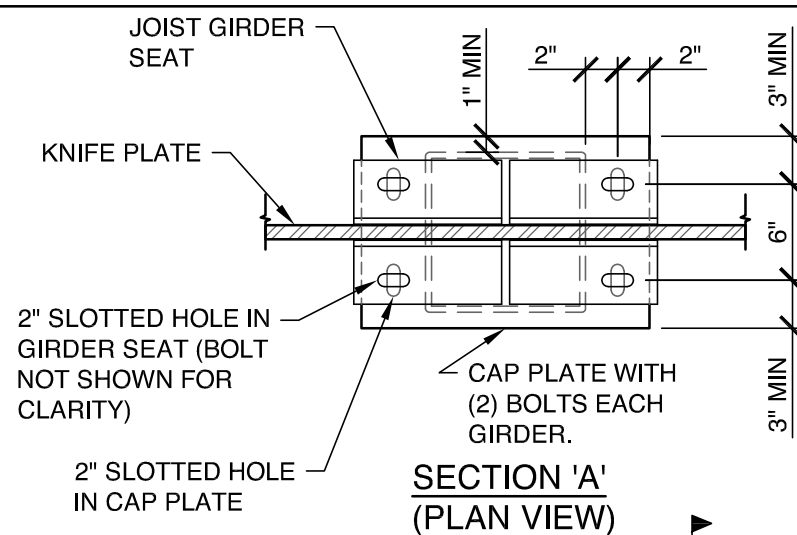
- NOTE:**
1. DECK INSTALLER TO TAKE CARE TO LOCATE LOW FLUTE OF DECK AT BLOCKING ON TOP OF JOIST GIRDER.
 2. AT CONTRACTOR'S OPTION, BLOCKING CAN BE SHOP INSTALLED. CONTRACTOR TO COORDINATE WITH VULCRAFT IF OPTION IS DESIRED.



SECTION 'A'

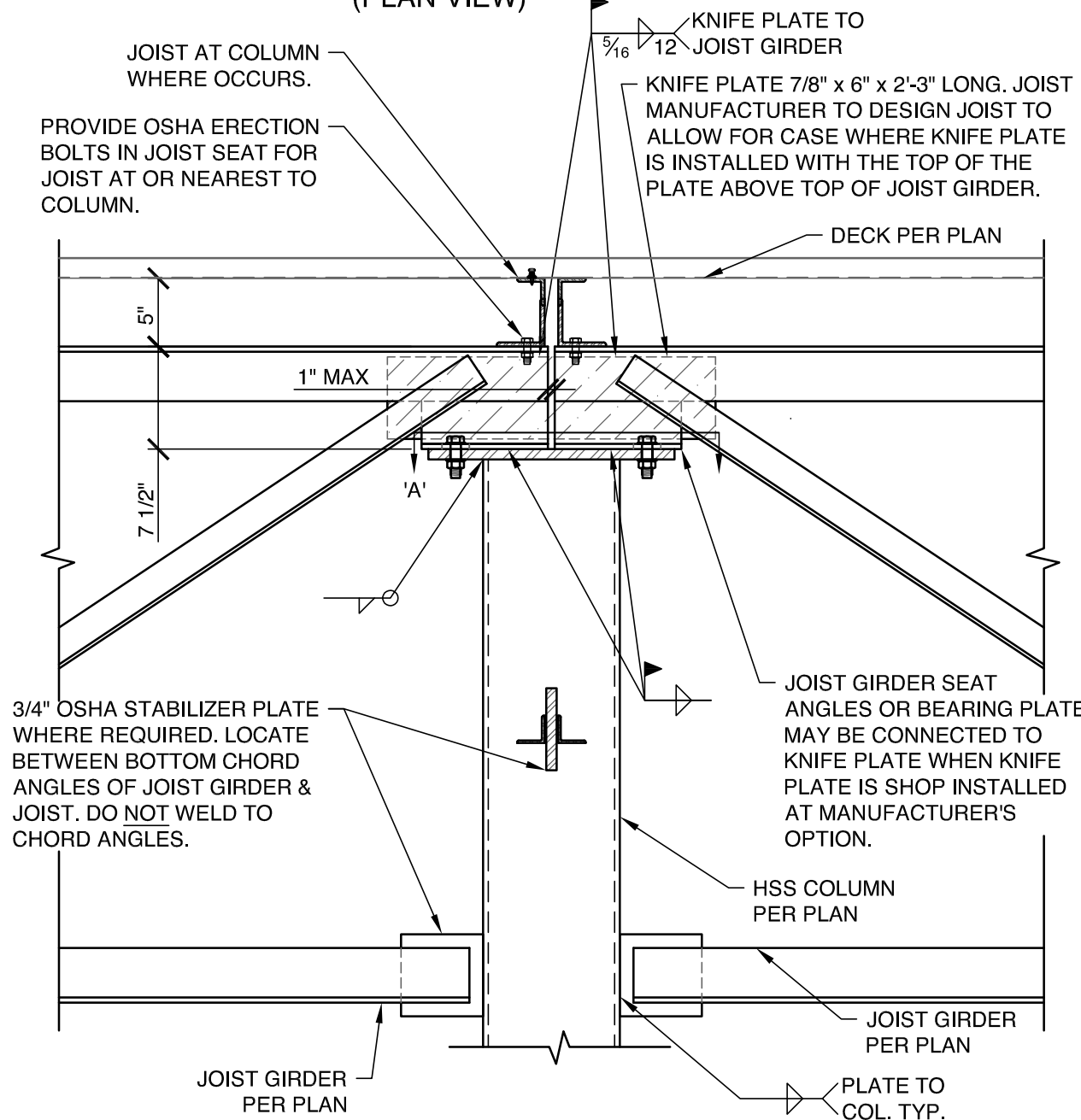
BENT PLATE BLOCKING ON JOIST GIRDER

STARTING POINT DETAIL



SECTION 'A'
(PLAN VIEW)

- NOTES:**
1. AT CONTRACTOR'S OPTION, KNIFE PLATE CAN BE SHOP INSTALLED IN 1 JOIST GIRDER. CONTRACTOR TO COORDINATE WITH VULCRAFT IF OPTION IS DESIRED.
 2. KNIFE PLATE MAY BE INTEGRATED INTO GIRDER SEAT IF SHOP INSTALLED.



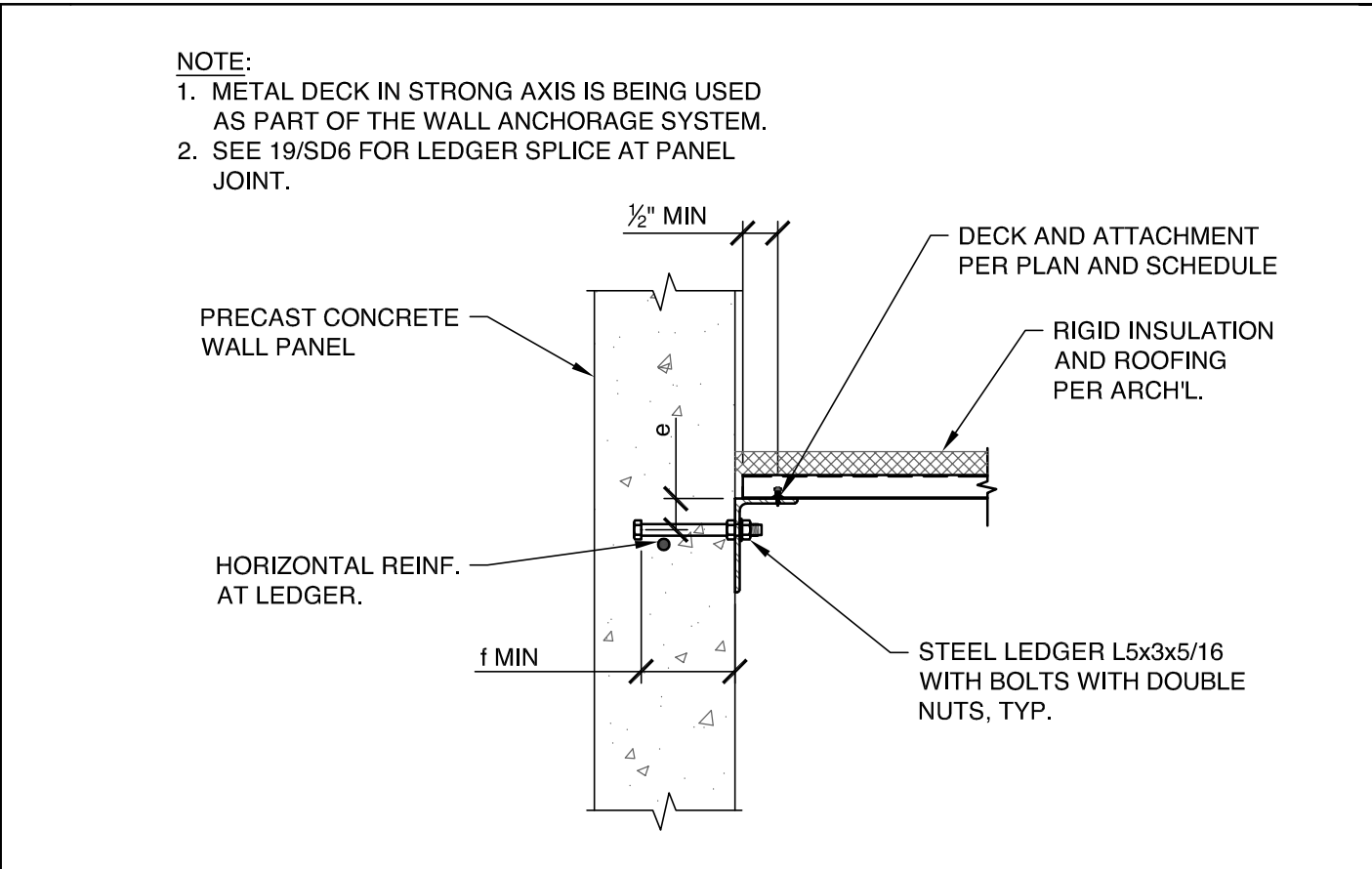
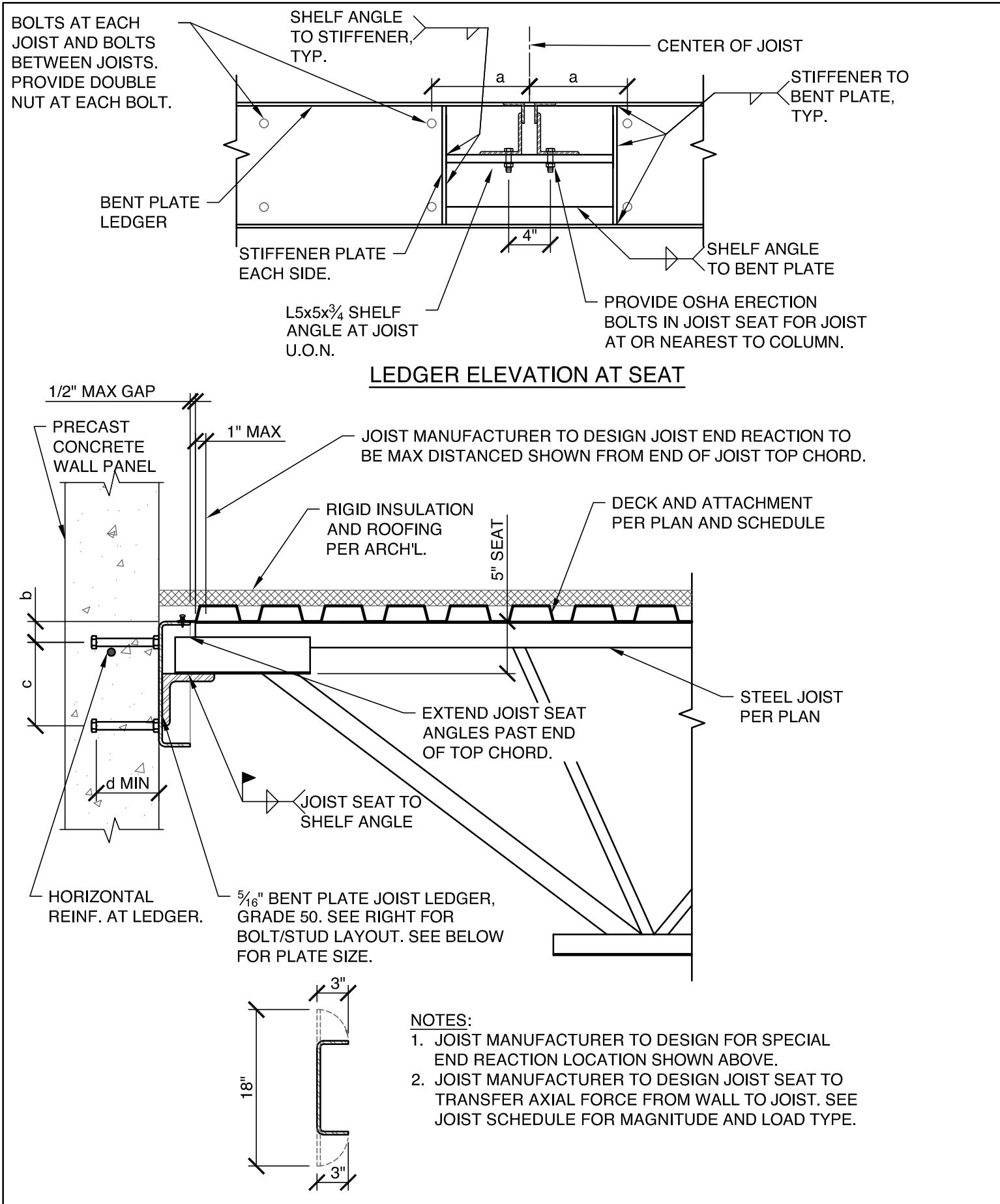
3/4" OSHA STABILIZER PLATE WHERE REQUIRED. LOCATE BETWEEN BOTTOM CHORD ANGLES OF JOIST GIRDER & JOIST. DO NOT WELD TO CHORD ANGLES.

JOIST GIRDER SEAT ANGLES OR BEARING PLATE MAY BE CONNECTED TO KNIFE PLATE WHEN KNIFE PLATE IS SHOP INSTALLED AT MANUFACTURER'S OPTION.

JOIST GIRDER TO COLUMN CONNECTION

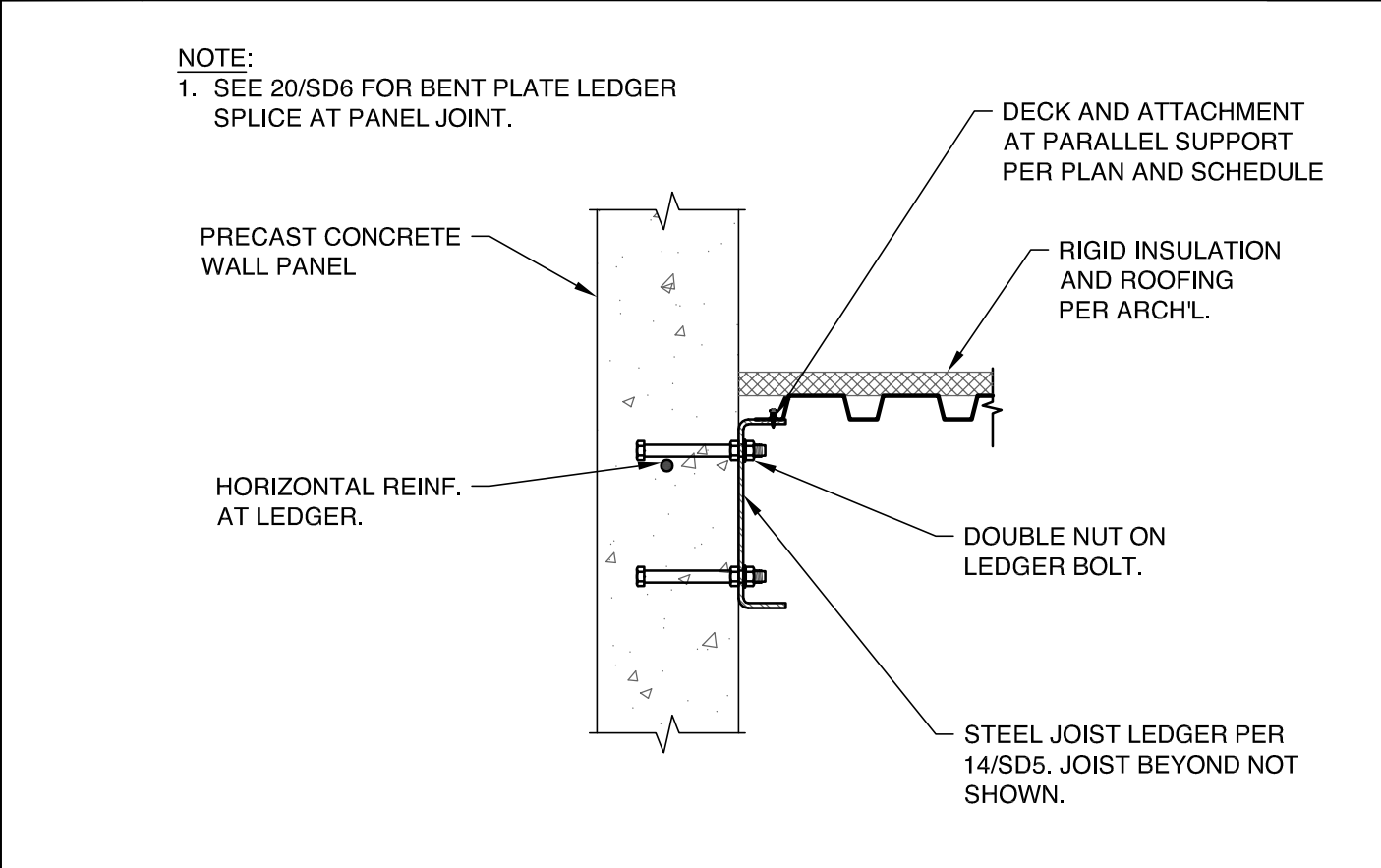
STARTING POINT DETAIL





STEEL LEDGER TO WALL PANEL
STARTING POINT DETAIL

15



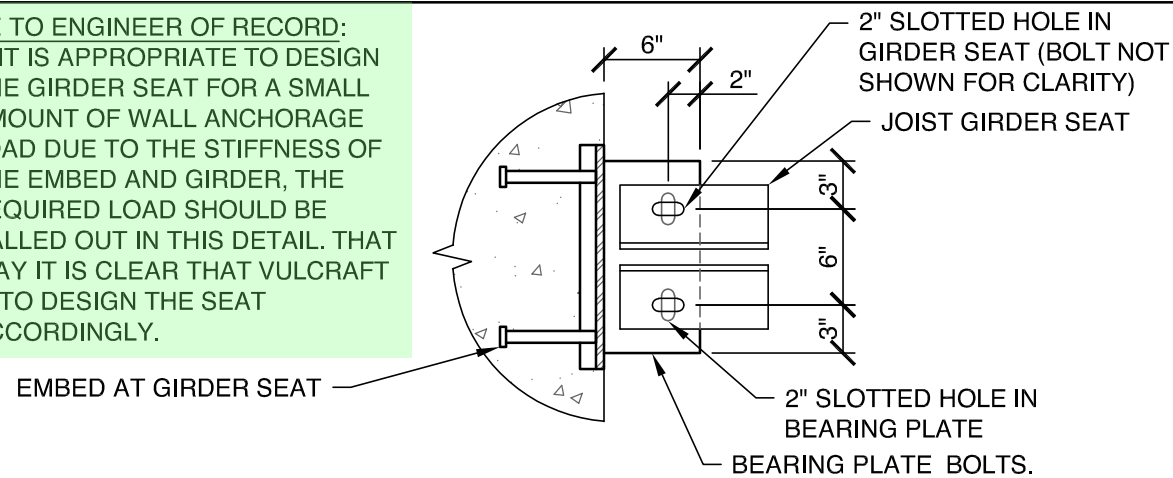
STEEL DECK TO JOIST LEDGER
STARTING POINT DETAIL

16

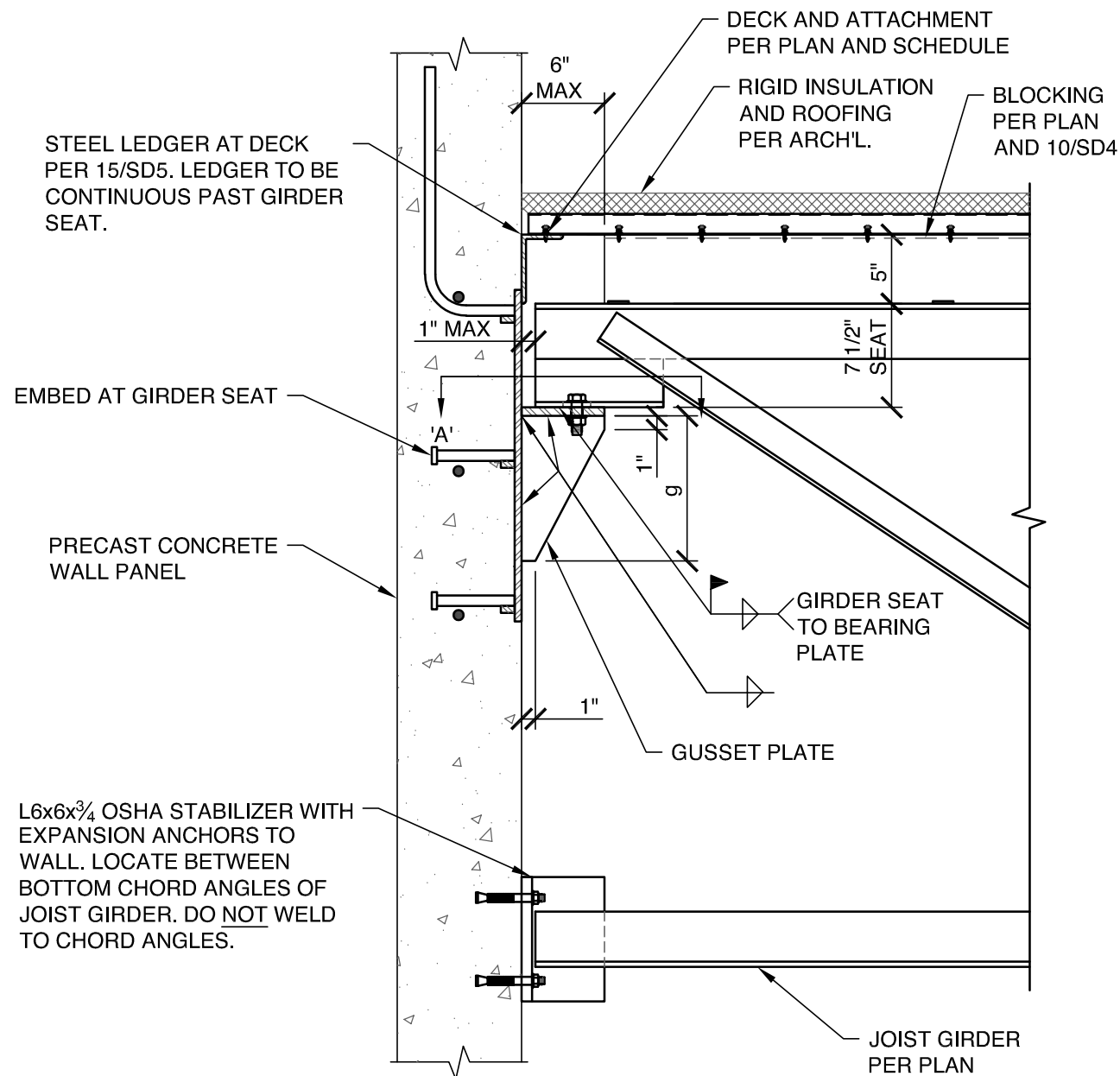
STEEL JOIST TO LEDGER
STARTING POINT DETAIL

14

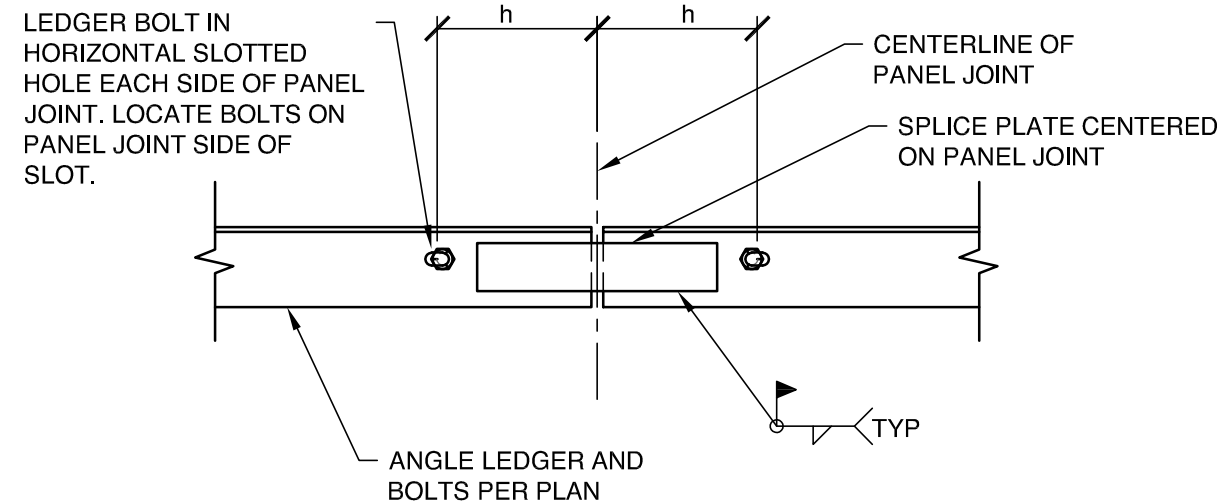
NOTE TO ENGINEER OF RECORD:
 1. IF IT IS APPROPRIATE TO DESIGN THE GIRDER SEAT FOR A SMALL AMOUNT OF WALL ANCHORAGE LOAD DUE TO THE STIFFNESS OF THE EMBED AND GIRDER, THE REQUIRED LOAD SHOULD BE CALLED OUT IN THIS DETAIL. THAT WAY IT IS CLEAR THAT VULCRAFT IS TO DESIGN THE SEAT ACCORDINGLY.



SECTION 'A'
(PLAN VIEW)

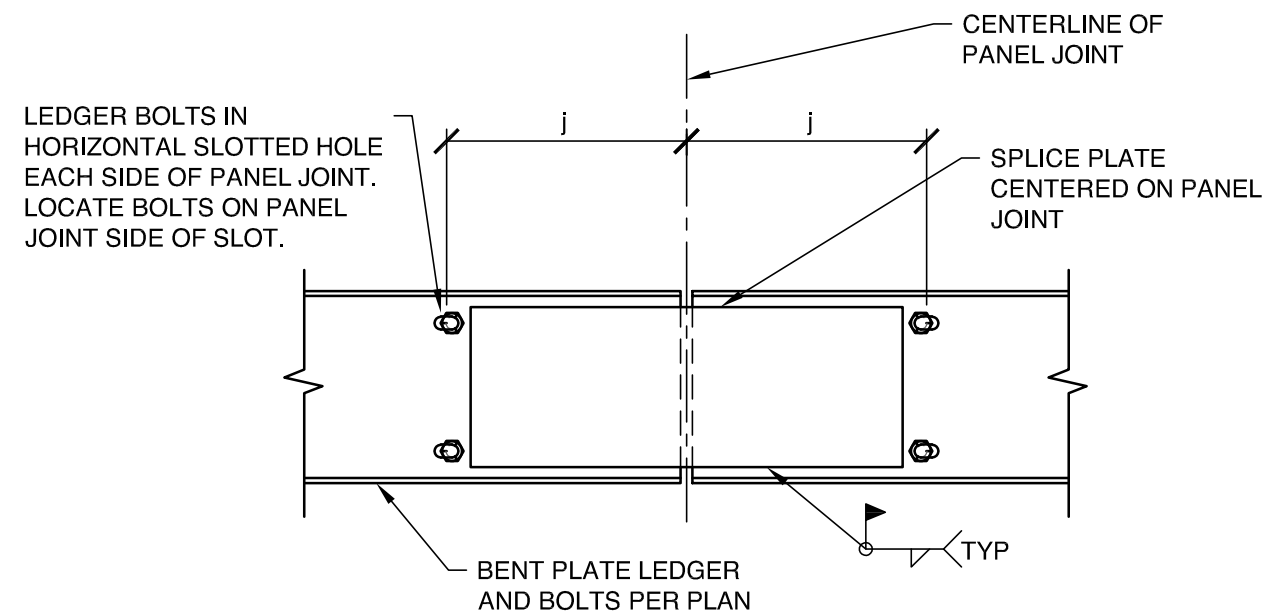


STEEL JOIST GIRDER TO WALL
STARTING POINT DETAIL



ANGLE LEDGER SPLICE AT PANEL JOINT

STARTING POINT DETAIL



BENT PLATE LEDGER SPLICE AT PANEL JOINT

STARTING POINT DETAIL

