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ROOF DECK DESIGN

second edition

STEEL DECK INSTITUTE

sdi

STEEL DECK INSTITUTE

ROOF DECK DESIGN MANUAL

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AUTHORED BY

Thomas Sputo, Ph.D., P.E., S.E.
SDI Technical Director

STEEL DECK INSTITUTE
P.O. Box 426
Glenshaw, PA 15116
Phone: (412) 487-3325
Fax: (412) 487-3326
www.sdi.org

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Since hazards may be associated with the handling, installation, or use of steel and its accessories, prudent construction practices should always be followed. The Steel Deck Institute recommends that parties involved in the handling, installation or use of steel and its accessories review all applicable manufacturers' safety data sheets, applicable rules and regulations of the Occupational Safety and Health Administration and other government agencies having jurisdiction over such handling, installation or use, and other relevant construction practice publications.

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FOREWARD

This *SDI Roof Deck Design Manual, 2nd Edition* updates, expands upon, and replaces the First Edition of this Manual (2012), as the main steel deck industry manual on steel roof deck design. This Handbook was developed to conform to the requirements of the ANSI/SDI RD-2017 "Standard for Steel Roof Deck." The SDI recommends that the user of this handbook obtain a copy of this Standard and refer to this Standard when using this handbook. The ANSI/SDI RD-2017 Standard is available for free download from the SDI website.

While conforming to the requirements of the Standard, this Handbook also provides recommendations of good design practices that may either not be included in the Standard, or may exceed the minimum requirements of the Standard. When recommended practice is beyond the minimum requirements of the Standard, this will be noted. In all instances, the design of steel roof deck as a component of a building or other structure is within the scope of practice of a licensed professional engineer or architect, and all liability for compliance with building code requirements is the responsibility of that designer. Professional judgment must be exercised when the user applies the data or recommendations contained in this Handbook.

Where this Handbook or other SDI publications refer to "Designer," this means "The licensed professional responsible for the content of the drawings and specifications from which the steel deck is to be constructed." This is usually the structural engineer of record, however it may be the architect or other licensed professional acting within the scope of their license.

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INTRODUCTION

SECTION 1

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Section 1.1 General Description of Roof Deck

Roof deck is a structural component of a building system. It performs several functions in that system:

1. It encloses the building from the exterior environment and for fire resistance.
2. It serves as the base for constructing the roof covering.
3. It resists gravity (roof dead, roof live, snow, and rain), seismic and wind loads.
4. It provides bracing to beams and joists.
5. Through diaphragm action, it transfers lateral loads (wind and seismic) to the vertical components of the lateral force resisting systems.

The roof deck is not intended to be a watertight component of the building system. Roof covering construction may include insulation board and nail-base that is attached to the steel roof deck either by mechanical fasteners or adhesives, along with a water resistant membrane. The design of roof covering is outside the scope of this Manual, and the reader is referred to other resources for this information, including the *NRCA Roofing Manual: Membrane Roof Systems*.

Section 1.2 Roof Deck Production

Deck manufacturers obtain steel in coil form either directly from steel producing companies or from intermediary suppliers. The steel is ordered to an ASTM specification, SDI specification, or other acceptable criteria. When the coil is received at the deck plant it is logged into the inventory and each subsequent job produced from the coil is recorded.

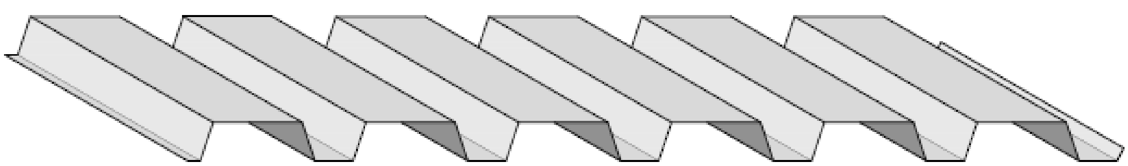
All of the generic profiles are now produced on roll forming equipment. A roll forming line consists of an uncoiler and a series of graduated tools that bend the flat sheet into the required shape. The sheet passes through the tooling in a continuous strip and at the correct instant the deck is cut to length (with a shear blade) with a square cut. The resulting profile is nominally consistent in cross section.

Some manufacturers have a coil coating line set up to paint the uncoiled steel immediately before the sheet is formed. Some manufacturers pre-paint coils or order pre-painted coils from coil coating companies for later forming into deck. In either case, the painting process is the same. As the steel unreels from the coil it is first cleaned and dried. It is sent through paint rollers and then baked, so that when it emerges it is coated with a provisional and flexible primer coat of paint. The steel can be recoiled or proceed directly into the roll former. The primer paint is designed to be compatible with a finish top coat. The primer paint can be applied over uncoated or galvanized steel.

Section 1.3 Commonly Specified Roof Deck Profiles

Wide Rib Deck (WR)

Wide rib deck (commonly referred to as Type B deck) is the most commonly used roof deck. This profile is better optimized for structural performance than narrow rib (NR) or intermediate rib (IR) profiles, and should be specified when a 1-1/2" profile is desired. The wider rib opening makes attachment to the supporting structure easier than with NR or IR profiles. Currently used insulation thicknesses can span the rib opening. Most manufacturers of steel roof deck can supply this profile, and load tables for this profile are included in this Manual.



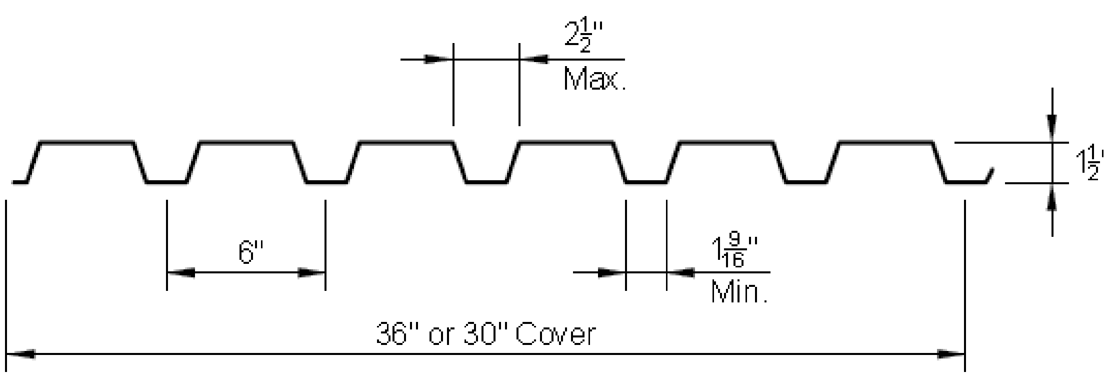


Figure 1.1 – Wide Rib (WR) Deck

Deep Rib Deck (DR)

Deep rib deck (commonly referred to as Type N deck) is a 3 inch deep profile that is designed to span greater distances than 1-1/2 inch deck. Due to the greater span length and the rib spacing, particular attention needs to be paid to uplift anchorage of this deck in high wind zones. Many manufacturers of steel roof deck can supply this profile, and load tables for this profile are included in this Manual.

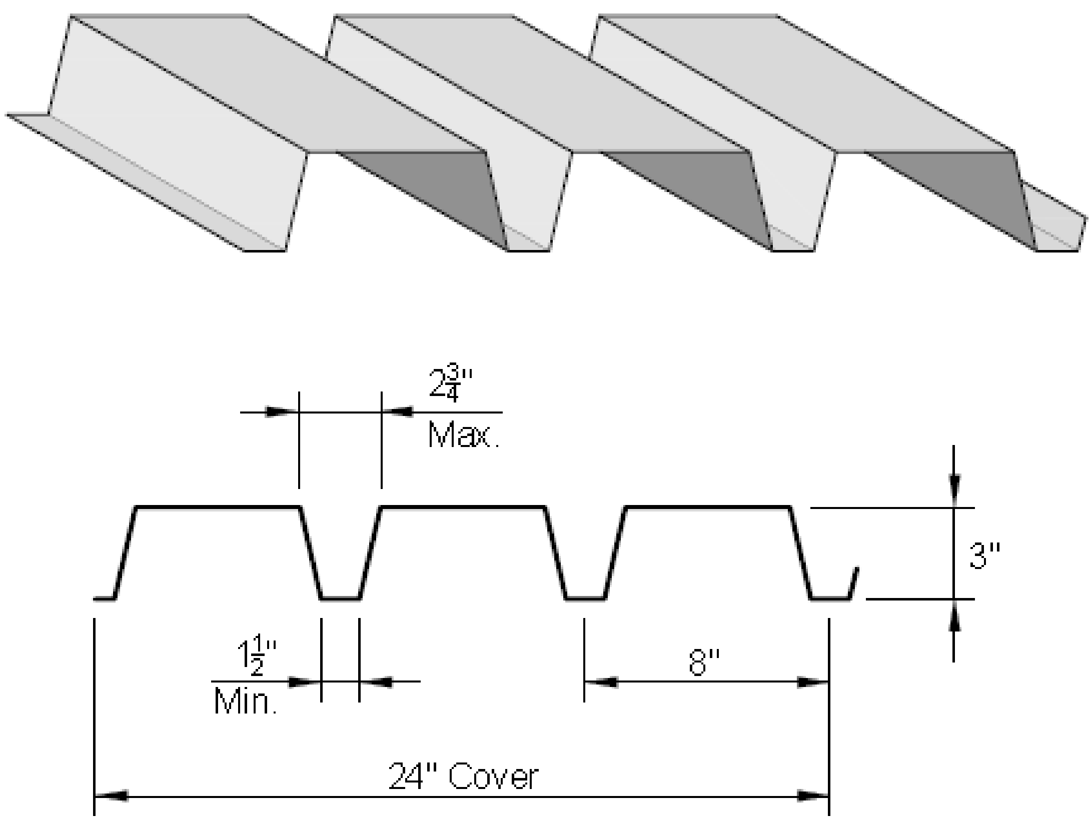


Figure 1.2 – Deep Rib (DR) Deck

Some manufacturers can supply a similar 3 inch deep x 8 inch pitch panel that is 32 inches wide with slightly different dimensions and section properties. These manufacturers should be contacted directly for information on their particular product.

Section 1.4 Other Roof Deck Profiles

Narrow Rib Deck (NR)

Narrow rib deck (commonly referred to as Type A deck) was originally designed to accommodate thin insulation boards. The narrow rib opening makes attachment to the supporting framing more difficult than the other profiles because of the restricted space in which to put a mechanical fastener or a weld. In the narrow rib profile the flanges are not balanced and the centroid is near the top of the section. That makes this profile relatively inefficient compared to the wide rib (WR) profile. Currently used insulation board thicknesses no longer require narrow rib openings. As a result, narrow rib deck is seldom used in new construction, but it does appear in some older buildings, and may be required when replacing damaged deck in existing structures that used this deck. Because of the difficulty in fastening this deck, unless its use is required by the type and thickness of the insulation, it is not recommended for new construction.

Load tables for this profile are not included in this Manual, however they were contained in the first edition of this manual (RDDM01, 2012) or are available from manufacturers who supply this profile.

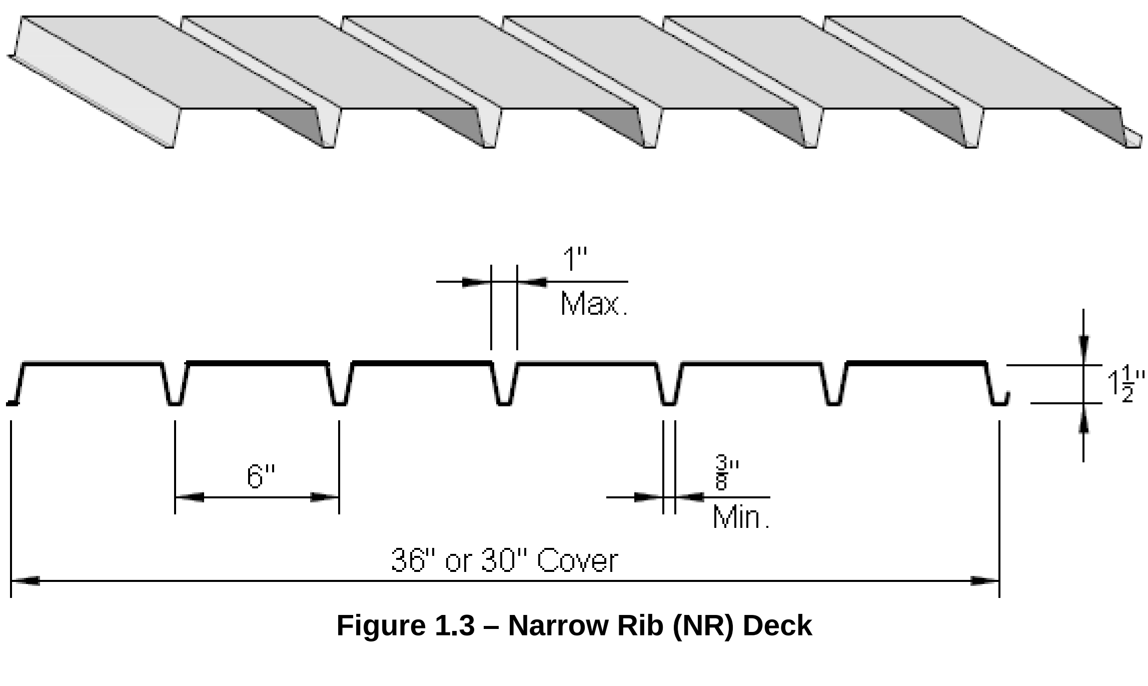


Figure 1.3 – Narrow Rib (NR) Deck

Intermediate Rib Deck (IR)

Intermediate rib deck (commonly referred to as Type F deck) is the transition deck between narrow rib and wide rib. It also can accept fairly thin insulation boards and still be easily attached; however, it is not as structurally efficient as the wide rib profile.

Load tables for this profile are not included in this Manual, however they were contained in the first edition of this manual (RDDM01, 2012) or are available from manufacturers who supply this profile.

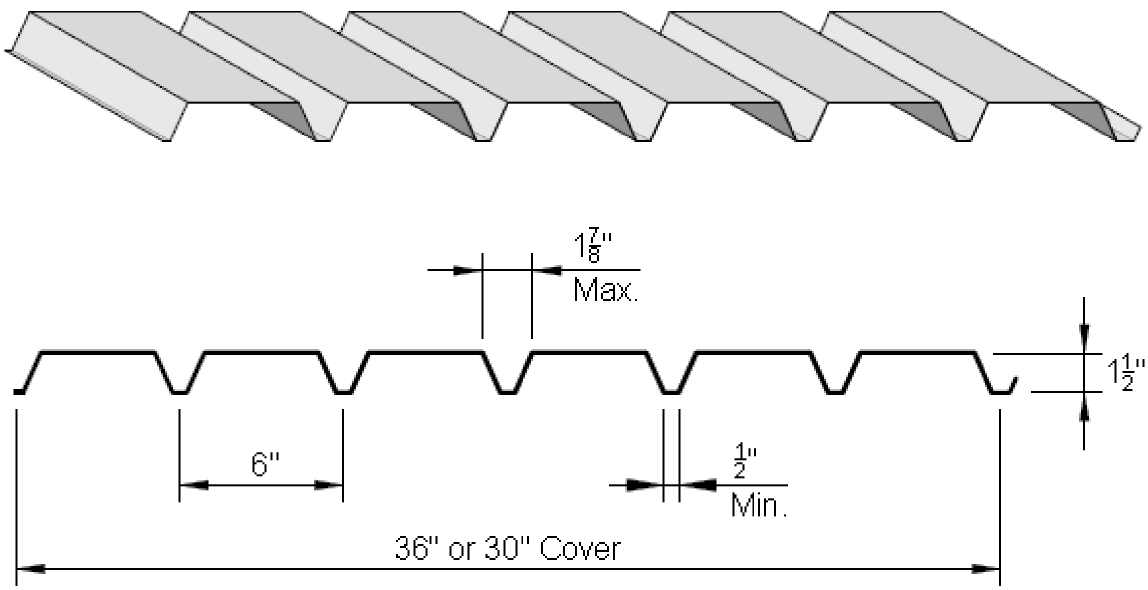


Figure 1.4 – Intermediate Rib (IR) Deck

Long Span Roof Deck

Long span roof decks (4.5DR, 6DR, and 7.5DR) are produced by some manufacturers. However, there is less uniformity among these products than with 1-1/2 inch and 3 inch deck profiles. A general profile is shown in Figure 1.5. These decks are provided in depths of 4.5, 6 and 7.5 inch. Long span profiles are designed to carry roof loads over spans of up to 35 feet.

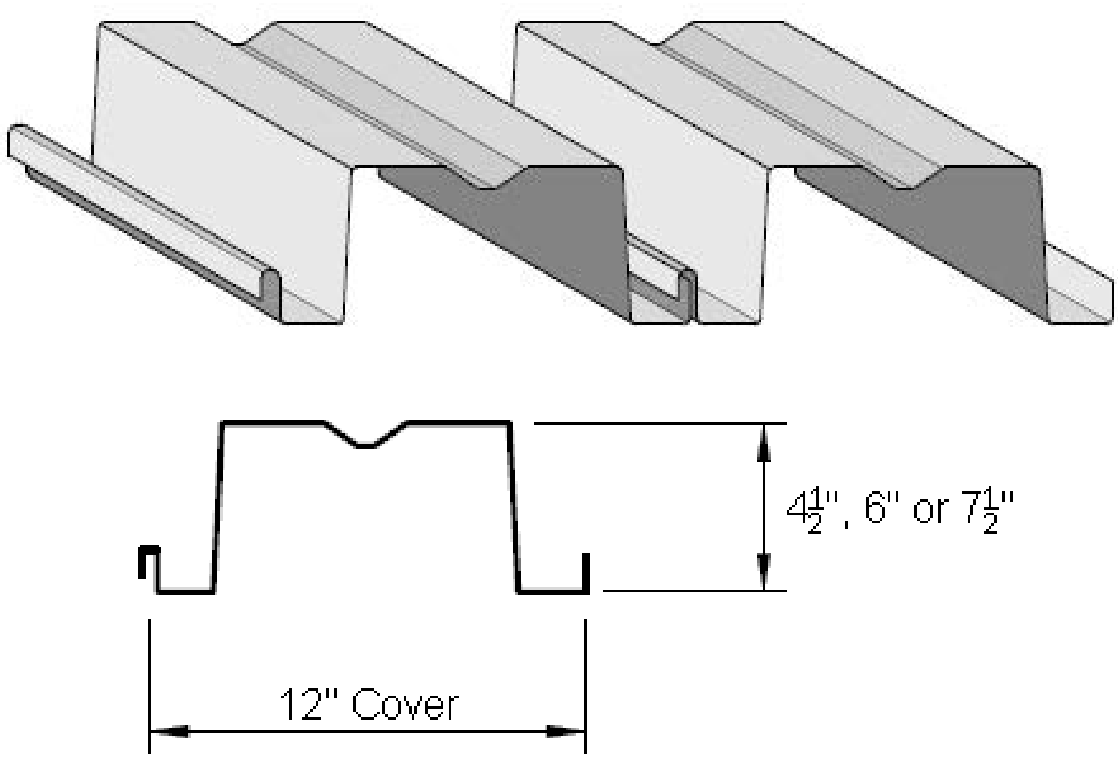


Figure 1.5 – Long Span Deck

Short Span Roof Deck

Roof deck shallower than 1-1/2" is available from many manufacturers, however there is more variability in the available profiles between manufacturers. When the required deck spans are short (less than 4 foot), a shallow profile, as shown in Figure 1.6 may be used. These profiles are also commonly used as a non-composite form deck for lightweight insulating concrete (LWIC) roofs and for floor construction. Because the profile varies between manufacturers, load tables provided by the specific manufacturer should be relied upon.

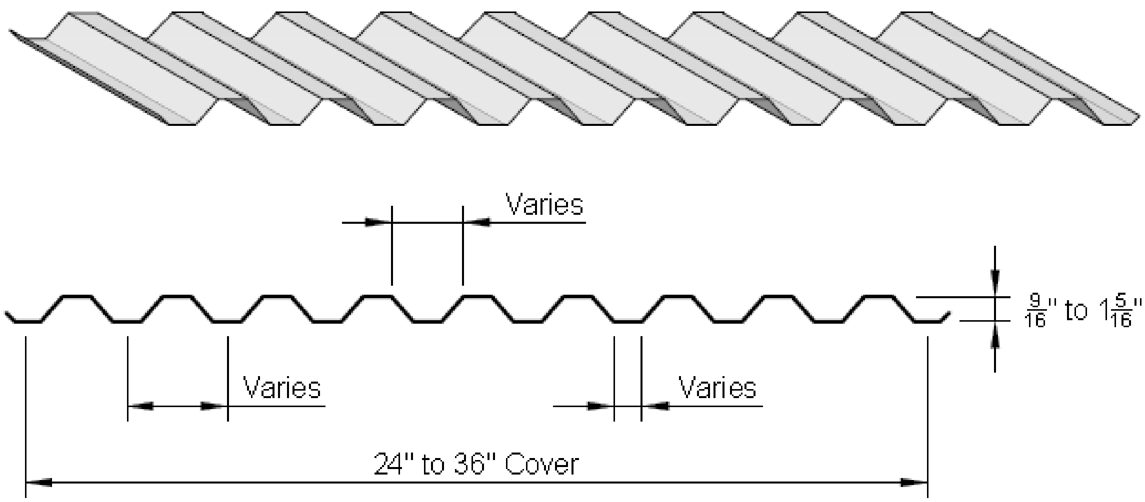


Figure 1.6 - Shallow Roof Deck.

Dovetail Roof Deck

Most of the common roof deck profiles are of an open trapezoid profile. Some manufacturers do supply a dovetail profile with re-entrant corners. These profiles vary between manufacturers, but typically are 2 inches or deeper in depth, and have a 6 or 8 inch nominal rib pitch. A typical profile is shown in Figure 1.7. This profile is also commonly used as a composite deck for floor construction. Because the profile varies between manufacturers, load tables provided by the specific manufacturer should be relied upon.

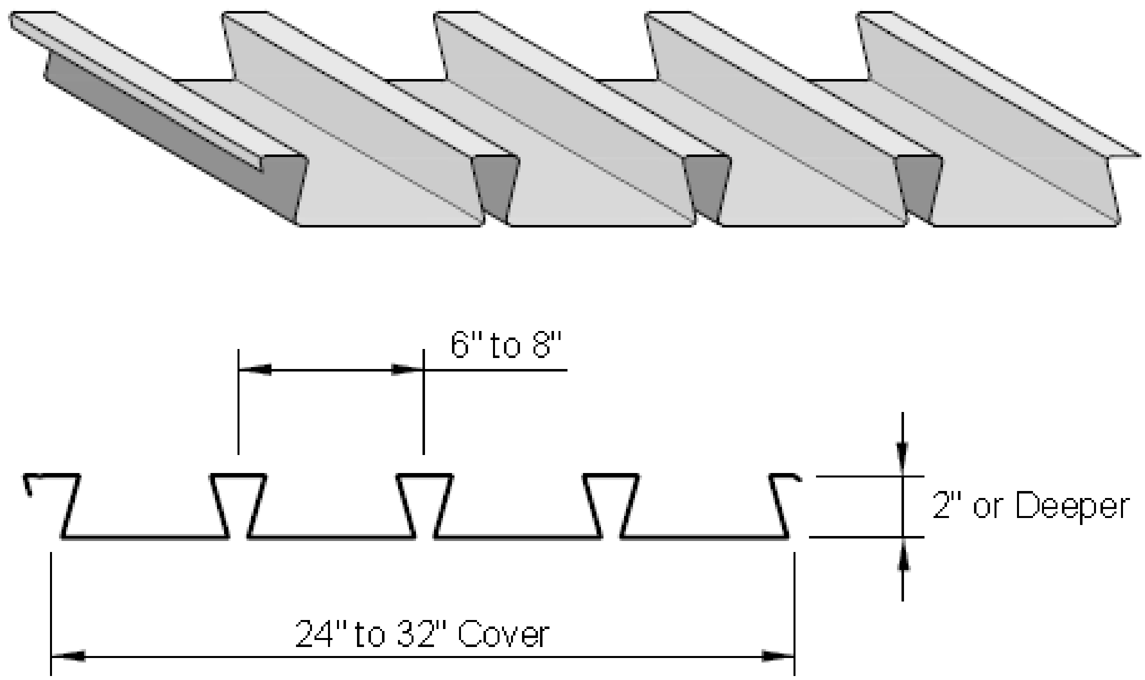


Figure 1.7 - Dovetail Roof Deck

Other Roof Deck Profiles

Other roof deck profiles are occasionally used, some of which are also used as floor form deck. Available profiles vary and individual manufacturers should be contacted for availability.

Cellular Roof Deck

Some manufacturers produce cellular roof deck profiles. Cellular profiles are made by attaching a flat sheet or another deck section to the bottom of the corrugated roof deck profiles. Due to the increased structural properties of the composite cross section, these profiles can span larger distances or support larger live loads with the same profile depth. The bottom sheet is frequently used to create an exposed finished ceiling. The bottom sheet may also include perforations to make it acoustic deck.

Cellular roof decks are manufactured by either resistance welding or mechanically fastening the upper and lower elements. If aesthetics are a concern in the final installed condition, consideration can be given by the Designer to specifying finished post-weld surface treatment and side-lap attachments. Special field applied paints or embossed and textured metal surfaces may also be specified. The manufacturer should be contacted for specific recommendations when specifying such surface treatment.

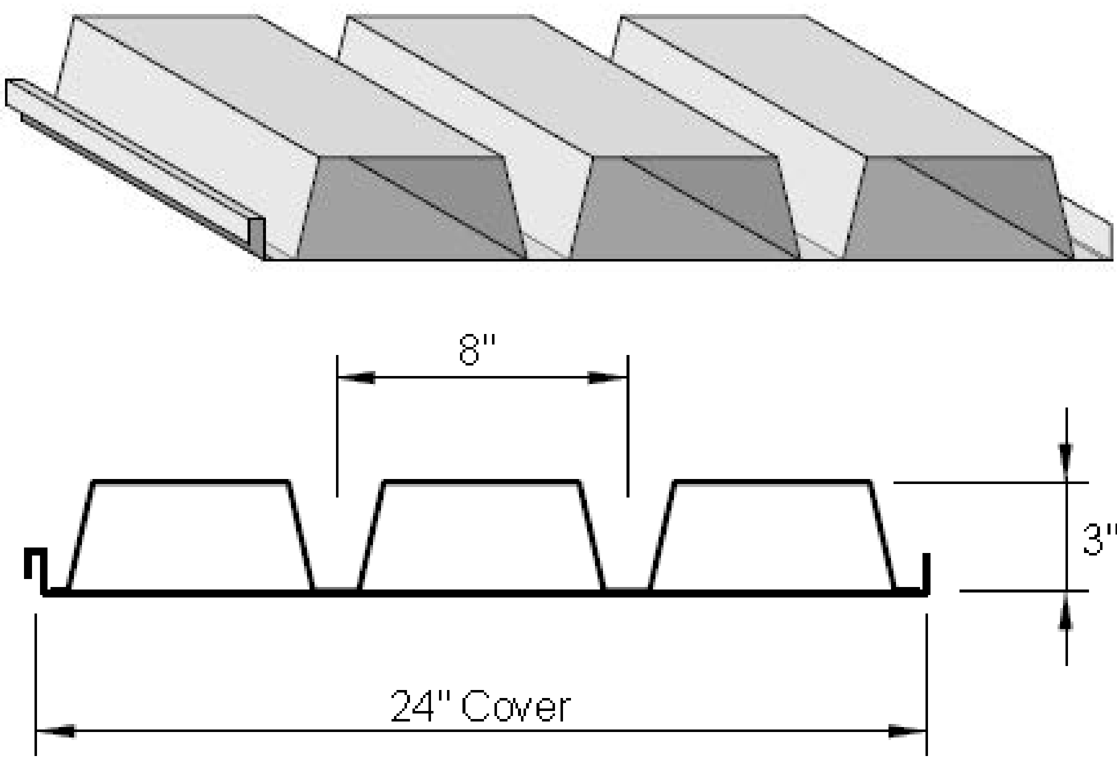
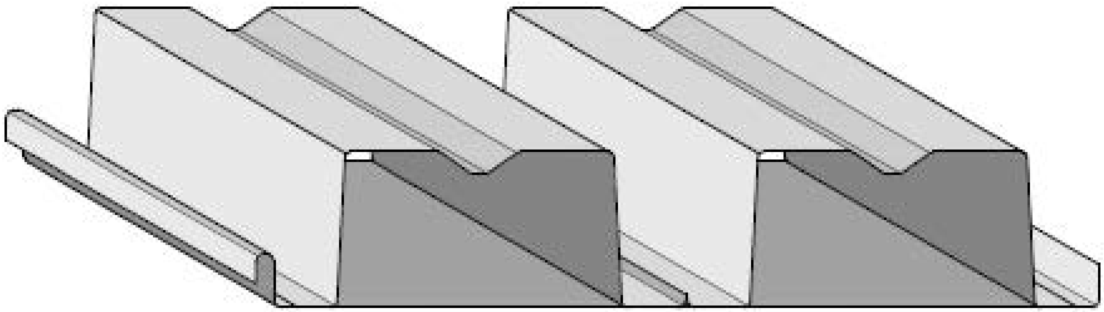


Figure 1.8 – DR Cellular Deck



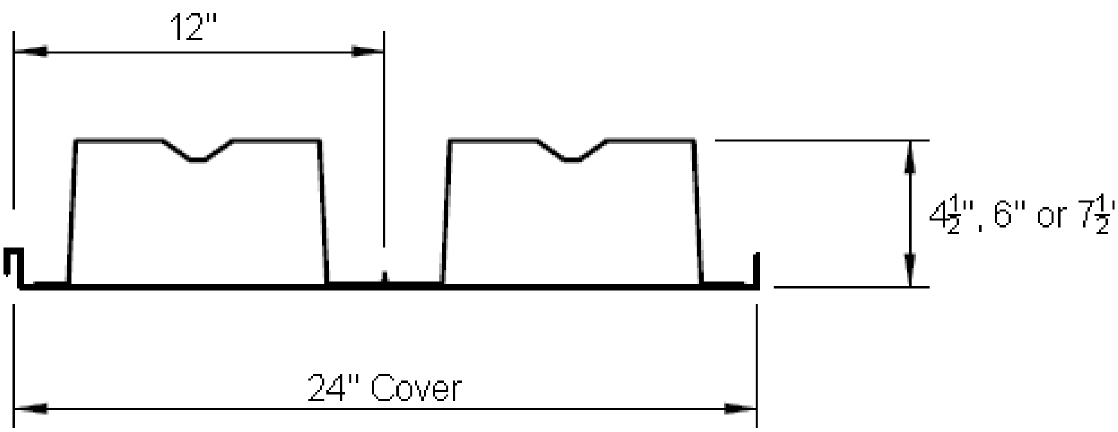


Figure 1.9 – Long Span Cellular Deck

Cellular profiles should be butted (not lapped) and side-lap fastener options are limited. Please refer to individual manufactured for connection limitations.

Acoustic Roof Deck

Sound absorbing decks are available in the following profiles; 1.5 inch Wide Rib Type WR, 3 inch Deep Rib Type DR, Dovetail, and Long Span Decks. All of these sections can be furnished as either plated cellular deck or perforated corrugated panels.

Acoustical decks are manufactured by perforating the webs in non-cellular deck, or the bottom sheet in cellular deck. Sound absorbing material is installed within the ribs of the deck. In open rib decks, the sound absorbing elements are field installed by the roofing contractor. For cellular decks, contact the manufacturer regarding installation procedures for the sound absorbing elements. The load carrying capacity of the deck may be affected slightly (approximately 5%) due to the perforations. Acoustical dovetail and other roof deck profiles are perforated in the bottom flanges. Individual manufacturers should be consulted for specific design information.

Figures 1.10 and 1.12 show tables of sound absorption data at various frequencies for typical acoustically treated roof deck. This data is obtained by conducting the ASTM C423 test with mounting conforming to E795 at an accredited acoustical laboratory. The sound absorption coefficients represent the percentage of noise that the tested surface converts to other energy forms which does not reflect as sound. The usual tested frequencies are 125, 250, 500, 1000, 2000, and 4000 Hertz and the Noise Reduction Coefficient (NRC) is the average of the middle four frequencies (250, 500, 1000, and 2000 Hertz). The average is rounded to the nearest 0.05. Because of measurement methods the sound absorption number for a particular frequency can be greater than 1; but for any specific use at that frequency the value should be taken as 1.

Noise Reduction Coefficients (NRC) and Sound Absorption Coefficients (SAC) are listed for typical products. The NRC value for wide rib type WR is 0.60 and deep rib type 3DR is 0.70 when tested with 1.50 pcf and 1.65 pcf sound absorbing fiberglass batts respectively and 2 inch thick polyisocyanurate foam insulation board above the deck. These values are based upon testing that was performed by the SDI. Individual deck manufacturers should be consulted for the NRC values that can be obtained by the use of a specific product.

The finish for exposed acoustical decks should be either galvanized or galvanized and painted. Finish coats (if applied correctly in the field) do not adversely impact the acoustical properties of the deck.

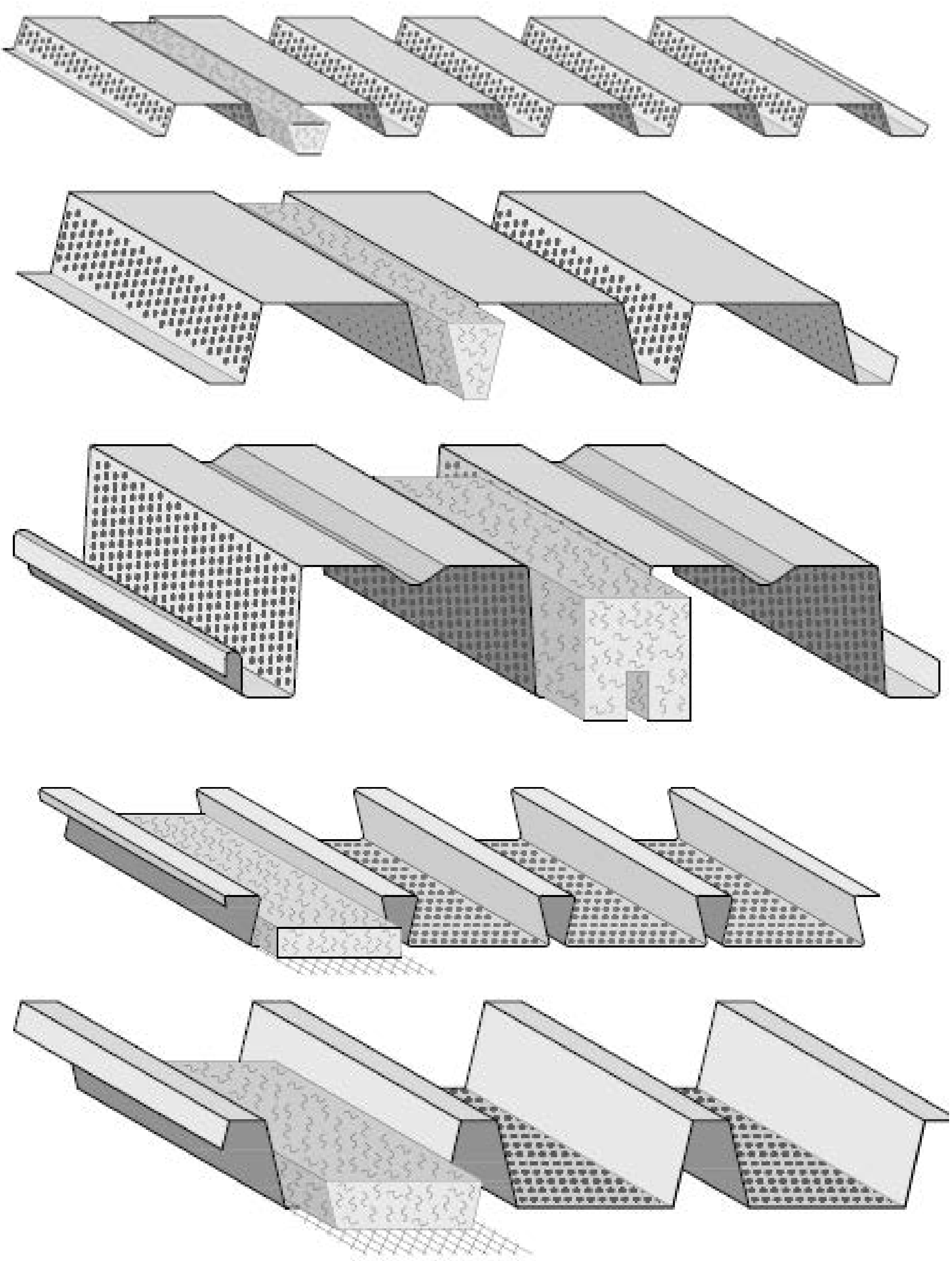
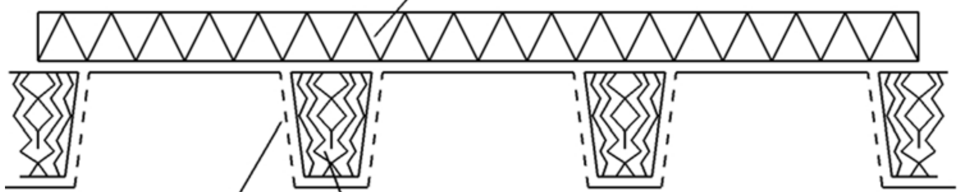


Figure 1.9 – Non-Cellular Acoustical Decks



		Sound Absorption Coefficients (Frequency)						**NRC**
		125	**250**	**500**	**1000**	**2000**	**4000**	
1.5 inch	Acoustic Deck	0.11	0.18	0.66	1.02	0.61	0.33	0.60
3 inch	Acoustic Deck	0.18	0.39	0.88	0.93	0.58	0.39	0.70

Figure 1.10 – Non-Cellular Acoustical Deck SAC and NRC

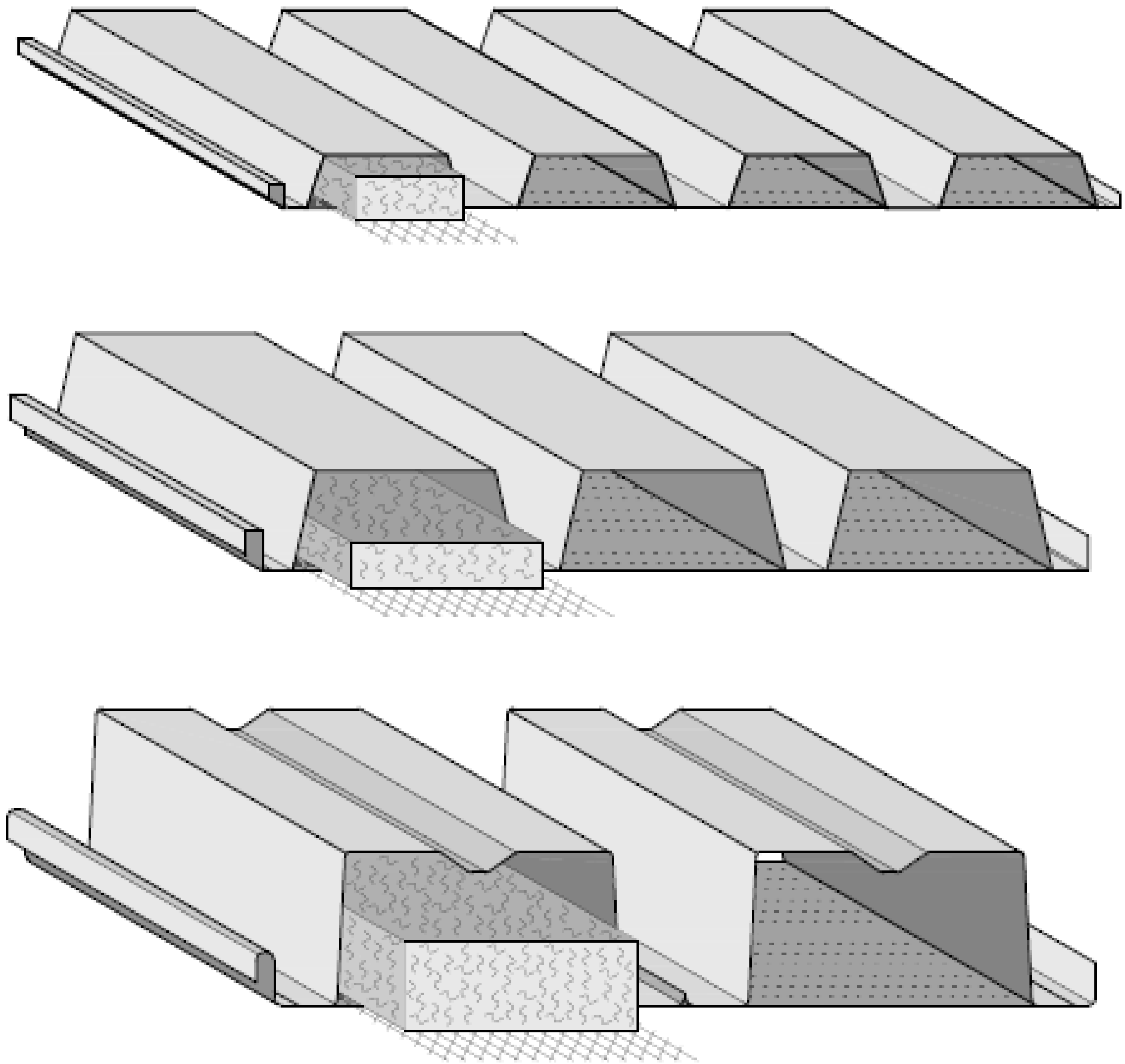
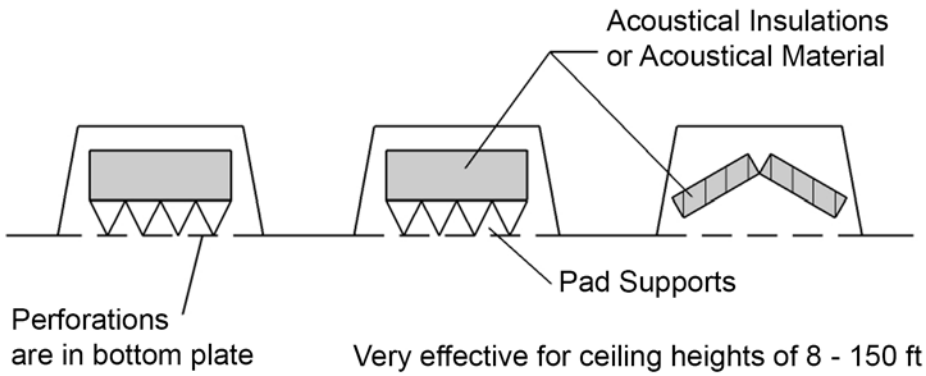


Figure 1.11 – Cellular Acoustical Decks



Panel	Sound Absorption Coefficients (Frequency)						NRC
	125	250	500	1000	2000	4000	
3 inch Deep Cell	0.47	0.57	0.95	0.98	0.82	0.69	0.85

Figure 1.12 – Cellular Acoustical Deck SAC and NRC

The noise absorption at any particular frequency and the NRC is a function of the total construction. Higher NRC values can be obtained by using fiberglass insulation board for the insulation material on top of the deck system in lieu of the commonly used foam board insulation. Consult individual SDI member companies for their recommendations and be aware that insulation board elected for its thermal characteristics will not have the same NRC as fiberglass board. Substitution of specified roofing components will affect acoustical performance.

Section 1.5 Side-lap Configurations

Deck can be manufactured with either a nested or interlocking side lap, depending upon the manufacturer. Some manufacturers can provide either profile. A nested side lap can be connected by screws or arc fillet or arc spot welds. An interlocking side lap can be connected using screws, top arc seam welds, button punches, or proprietary clenching systems, depending upon the exact interlocking side-lap configuration. Some interlocking side-laps cannot accept screws and the manufacturer's details should be consulted.

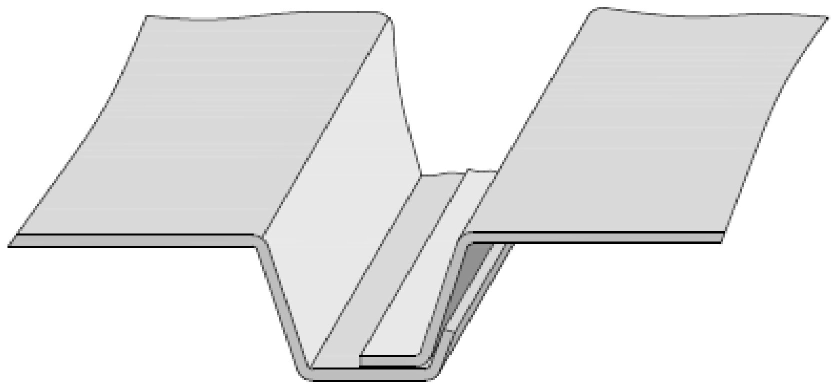


Figure 1.13 – Nested Side-lap

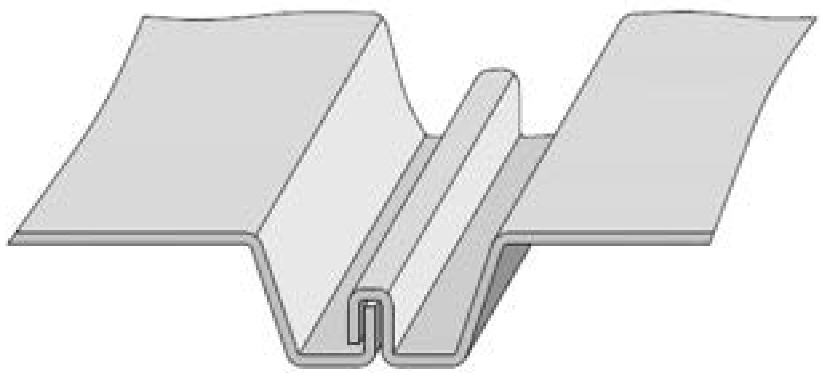


Figure 1.14 – Interlocking Side-lap

Section 1.6 Materials

Any steel that is listed in Section A of the AISI S100-16 Standard is permitted to be used for steel roof deck. Steel roof deck is most commonly produced from carbon steel meeting ASTM A1008, galvanized or galvanealled steel meeting ASTM A653, or metallic coated steel meeting ASTM A924. Some manufacturers may have the ability to produce steel deck from other materials, such as stainless steel or aluminum; however, deck produced from materials other than carbon or alloy steel is outside the scope of the ANSI/SDI RD-2017 Standard and this Manual. The availability of alternate materials must be confirmed with the individual manufacturer.

The load tables in this Manual are based on steel with a yield strength of 40 ksi. Many manufacturers' product specific load tables are also based on the same assumption, but most manufacturers have the ability to use steels with specified yield strengths exceeding 60 ksi. The literature of an individual manufacturer should be consulted to determine the specific steel grades used for their products.

Section 1.7 Deck Finishes

Roof deck is available in several finishes. The availability of any specific finish should be verified with the specific deck manufacturer. No deck finish is completely resistant to severe environmental conditions and the Designer must consider the exposure of the deck when selecting a deck finish and in determining what, if any, additional field coatings must be applied to the deck.

Galvanized Finish

A galvanized finish consists of a layer of zinc on top of a thin layer of zinc iron alloy. A zinc coating offers corrosion resistance (relative to the base steel) due to its self-sacrificial characteristics. Zinc is anodic to steel, which causes moisture, chlorides, and sulfides to attack and oxidize the zinc before the steel. Secondarily, the zinc coating provides a barrier between the steel substrate and the environment.

Galvanized deck with zinc coating complying with ASTM A924 is available in several coating thicknesses, the most common being G-30 (0.30 oz. per sq. ft. total over both sides of the sheet), G-60 (0.60 oz. per sq. ft. total over both sides of the sheet) and G-90 (0.90 oz. per sq. ft. total over both sides of the sheet.) Other thicknesses may be available; the specific manufacturers should be consulted as to availability.

Zinc-Aluminum coating finishes complying with ASTM A924 are also available on a limited basis.

Primer Painted Finish

Primer paint may be either applied in the deck plant, or prepainted steel coils may be utilized in the deck forming process. Either method is acceptable, and should be left to the option of the deck supplier.

Approximate paint dry film thickness averages 0.3 mil for a single coat and 0.6 mil for a double coat. Manufacturers have standard colors, typically white or grey that may be varied if required for a specific project. The availability, minimum quantities and cost of special primer colors should be verified with the specific deck manufacturer.

Primer paints by nature are impermanent and provisional and should be protected as recommended in the SDI MOC. Primer paint alone is intended only to protect the steel roof deck for a short period of time under ordinary exterior atmospheric conditions. If appearance of the finished deck is important, the Designer should consider a field applied finish paint. Finish painting and its compatibility with the deck prime paint is not the responsibility of the deck supplier.

Galvanized / Primer Painted

Galvanized roof deck is available with factory primer paint applied to one or both sides of the deck. Galvanizers refer to this as a "duplex system." Primer painted galvanized deck provides somewhat more corrosion resistance and may be suitable for applications where the deck will be field painted (and eliminates the need for field prime painting).

Bare (Uncoated) Finish

Steel roof deck may be provided without any finish. This deck, sometimes referred to as “black”, is provided without paint or galvanizing. Designers should be aware of the limited resistance to corrosion provided by bare deck.

ROOF DECK DESIGN CONSIDERATIONS

SECTION 2

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Section 2.1 General

Although the decks shown are generic profiles, there are variations in dimensions from manufacturer to manufacturer. The load tables contained in this Manual are based on a comprehensive evaluation of available cross sections produced by member companies and generally reflect a conservative lower bound result. Therefore, these tables are conservative. Individual deck manufacturers may supply tables and other information that are specific to their sections and material grades. Manufacturer specific information may therefore differ from the tables provided in this Manual.

Some roofing systems (particularly mechanically attached membranes and standing seam metal roofing with clips) may not transfer the upward wind load to the steel deck as a uniform load but as a series of point loads or as a line load. Patch loading and nonuniform distributions may also be possible. Steel deck can be analyzed in the same manner as any structural member for any combination of loads. Reaction forces will transfer directly to the frame fastening and the fasteners must be sized and spaced accordingly. If the roofing system is normally constructed so that is transmits the wind load as a concentrated load or as a line load and the locations of these loads are not known beforehand, the SDI advises that conservative deck thickness and fastener patterns be specified.

Section 2.2 Cross Section Strength

Deck cross section strength is determined using the AISI S100 “North American Specification for the Design of Cold-Formed Steel Structural Members.” Because the individual elements included in the cross section are usually classified as “slender”, local buckling with resulting post-buckling strength controls the strength design of most deck sections. Local buckling occurs only in compression elements, resulting in less effective area and reduced section properties. For most cross sections, the moment of inertia and section modulus for positive and negative bending at the nominal design strength are different from the elastic values. An in-depth discussion of cold-formed steel design is beyond the scope of this Manual, and the AISI S100 Standard and Commentary should be consulted for additional information.

Section 2.3 Deflections

Using the criteria in the AISI S100 Standard, the moment of inertia of the deck section for serviceability checks (such as deflection) is usually calculated at a stress of 60% of the specified yield stress (0.60 F_y). The usual assumption is that this moment of inertia is constant along the length of the deck span. However, if the moment of inertia calculated at 0.60 F_y is less than the full elastic moment of inertia, then the effective moment of inertia will vary along the length of the deck span, due to the value of the service stress varying along the deck span. The assumption of a constant moment of inertia along the entire deck span in this instance is conservative, and further refined analysis is permitted.

The ANSI/SDI RD-2017 Standard requires deflections be limited as required by the controlling building code. For instances where there is not a controlling building code, the Standard sets the following minimum deflection requirements for roof deck in Table 2-1.

Table 2-1 Roof Deck Deflection Limits

Roof Deck Construction	Live Load	Snow or Wind[1]	Dead + Live Load
Supporting plaster ceiling	L/360	L/360	L/240
Supporting non-plaster ceiling	L/240	L/240	L/180
Not supporting ceiling	L/180	L/180	L/120

[1] Wind loads are permitted to be multiplied by 0.42 for ASCE 7-10 and ASCE 7-16, and 0.7 for ASCE 7-05 and earlier.

For cantilever members, L shall be taken as twice the length of the cantilever

The effect of deck deflections on insulation and roofing materials should also be considered by the Designer to ensure compatibility of deflections with the materials to be installed. When adhered insulation board is used, the SDI has traditionally recommended a live load deflection limit of the lesser of 1/240 of the span measured between centerlines of supports or 1 inch (25 mm) for supported spans and the lesser of 1/120 of the cantilever span or 1 inch (25 mm) for cantilever spans. These limits may not be applicable when mechanically attached insulation board is used.

For uniformly distributed loads on steel deck, the following weighted averages of the full and effective section properties provide a good estimate of an average equivalent effective section property for the estimation of deflection under service loads. These weighted averages are used in this Manual.

Simple span: $I_D = (I_x + 2I_p) / 3$

Multiple span: $I_D = (I_x + 2I_n) / 3$ if negative bending controls, or
$I_D = (I_x + 2I_p) / 3$ if positive bending controls.

Where:
I_D = Moment of inertia for deflection calculation
I_x = Full (gross) moment of inertia
I_p = Effective moment of inertia, positive bending
I_n = Effective moment of inertia, negative bending

Section 2.4 Bearing

Minimum bearing lengths for steel deck should be determined in accordance with AISI S100 for the limit states of web crippling, combined shear and flexure, and minimum fastener edge dimensions. Combined web crippling and flexure is not applicable to steel deck per the exception to AISI S100-16 Equation H3-1. AISI S100 permits a minimum bearing length of ¾ inch. For most 1-1/2 inch and 3 inch decks, experience has shown that 1-1/2 inches of bearing is sufficient. If less than 1-1/2 inches of bearing is available, or if high support reactions or concentrated loads near a support are expected, the Designer should check the deck web crippling capacity. Short bearing lengths generally require additional attention to end clearances on the fasteners.

Decks deeper than 3 inches may require longer bearing lengths. Minimum bearing lengths considering web crippling for these deeper decks should be verified with the manufacturer.

Section 2.5 Transverse Distribution of Concentrated Loads

The Steel Deck Institute sponsored research at the Missouri University of Science and Technology, formerly known as the University of Missouri - Rolla to determine a rational distribution width for a concentrated load on deck. Based on this research, the SDI makes the following recommendations for 1- 1/2" NR, IR, and WR roof deck between 16 and 22 gage thickness and used on single or multispan applications:

For $X = 0$ — Loads over a support do not create a bending moment or shear in the deck. Therefore, the number of deck webs directly loaded should be checked for two flange web crippling using the minimum of B and N.

For $0 < X \leq 0.25$ — $b_e = B + 6 \geq 12$

For $0.25 < X \leq 0.50$ — $b_e = B + 18 - \dfrac{3}{X} \geq 24 - \dfrac{3}{X}$

Where:

B = load footprint width transverse to the deck span. When the load centroid is not at the center of the footprint, let B equal twice the least dimension from the centroid to the baseplate edge; inches.

b_e = effective distribution width; inches

N = deck bearing length over supporting member; inches

X = percentage of span, measured from the nearest support to the center of the concentrated load, ≤ 0.50

For design the load is considered a point load located at the centroid of the load footprint and longitudinal distribution (parallel to the deck ribs) should be neglected.

These recommendations do not overrule the judgment of the Designer, should the stiffness of the footprint be great enough to bridge the deck in the longitudinal direction and to not load the deck; web crippling of the steel deck may still be a limit state and should be given consideration.

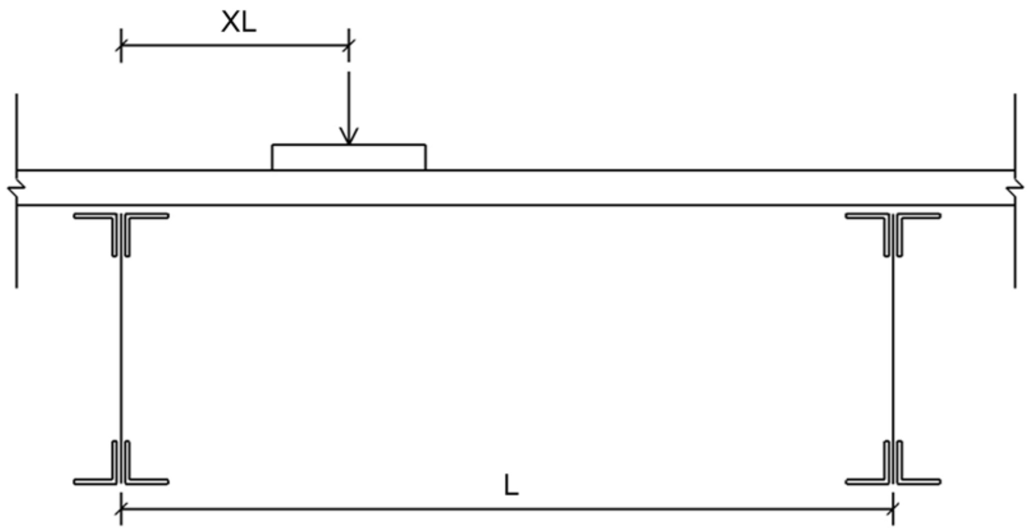

Figure 2.1 – Concentrated Load

Research to support similar recommendations are not available for 3" DR deck, and conservatively the width of bearing can be used without considering transverse distribution.

Section 2.6 Diaphragm Action

Steel roof deck and fasteners, if properly designed and constructed, can resist in-plane forces through diaphragm action. These in-plane forces in steel deck roofs typically are the result of wind or seismic forces. When diaphragm forces are combined with wind uplift forces, the design of the support fasteners needs to consider this interaction. When wind uplift is considered with in-plane diaphragm shear, both uplift and diaphragm shear should be MWFRS loads. For a discussion of this, refer to SDI Technical Note No. 1 "C&C or MWFRS Pressures for Steel Roof Deck Design: Clarification for the Engineer of Record."

For additional information on steel deck diaphragm design, the SDI Diaphragm Design Manual is recommended.

Section 2.7 Combined Flexure and Axial Force in Deck

Steel deck may be used to brace walls against wind or seismic forces in the direction where the deck ribs are perpendicular to the wall being braced. Deck where the ribs are parallel to the wall being braced is not permitted to brace the wall. Where the deck braces the wall, the axial capacity of the deck needs to be considered, particularly when combined with wind uplift.

Section 2.8 Combined Shear and Uplift on Fasteners

When shear due to in-plane diaphragm forces is combined with uplift, most often occurring due to wind, the combined effects of both forces must be considered. The AISI S100 Standard provides interaction criterion for screws and welds in Chapter J. These equations have been reformatted to use ASD and LRFD fastener design strengths found in Tables 8 and 9 in this handbook.

Screws

Combined Shear and Pull-Over using MWFRS

ASD:

$$0.71 \frac{V}{V_{ASD}} + 0.50 \frac{T}{T_{ASD}} \leq 1.0 \qquad \text{(See Table 8.4 and Table 8.3)}$$

LRFD:

$$0.71 \frac{\overline{V}}{V_{LRFD}} + 0.50 \frac{\overline{T}}{T_{LRFD}} \leq 1.0 \qquad \text{(See Table 8.4 and Table 8.3)}$$

Combined Shear and Pull-Out using MWFRS

ASD:

$$0.74 \frac{V}{V_{ASD}} + 0.74 \frac{T}{T_{ASD}} \leq 1.0 \qquad \text{(See Table 8.4 and Table 8.2)}$$

LRFD:

$$0.74 \frac{\overline{V}}{V_{LRFD}} + 0.74 \frac{\overline{T}}{T_{LRFD}} \leq 1.0 \qquad \text{(See Table 8.4 and Table 8.2)}$$

These equations for combined shear and pull-over and combined shear and pull-out are applicable within the following limits:

(1) $0.0285 \text{ in.} \leq t_1 \leq 0.0445 \text{ in.}$

(2) No. 12 and No. 14 screws

(3) $d_w \leq 0.75$ in.

(4) $F_{u1} \leq 70$ ksi

(5) $t_2/t_1 \geq 2.5$

For configurations outside of these limits, a unity interaction equation may be used to calculate combined shear and pull-out and combined shear and pull-over:

ASD:

$$\frac{V}{V_{ASD}} + \frac{T}{T_{ASD}} \leq 1.0$$

LRFD:

$$\frac{\overline{V}}{V_{LRFD}} + \frac{\overline{T}}{T_{LRFD}} \leq 1.0$$

Interaction of Shear and Tension in the Screw

ASD:

$$\frac{V}{V_{ASD}} \leq 1.0 \quad \text{and;}$$

$$0.77\frac{V}{V_{ASD}} + 0.77\frac{T}{T_{ASD}} \leq 1.0$$

LRFD:

$$\frac{\overline{V}}{V_{LRFD}} \leq 1.0 \quad \text{and;}$$

$$0.77\frac{\overline{V}}{V_{LRFD}} + 0.77\frac{\overline{T}}{T_{LRFD}} \leq 1.0$$

Power Actuated Fasteners (PAF)

AISI S100, Section J5.4 requires that combined action be considered in the design of these fasteners. Refer to manufacturer's literature for the criteria for any specific PAF.

Arc Spot Welds

ASD:

(1) If $T \leq 0.28\ T_{ASD}$, V_{ASD} is not reduced by presence of tension

(2) If $T > 0.28\ T_{ASD}$, V_{ASD} is reduced as follows

$$V_{ASD\ Reduced} = V_{ASD}\left[1-\left(\frac{T}{T_{ASD}}\right)^{1.5}\right]^{0.67} \quad \text{(See Table 7.1 and Table 7.2)}$$

LRFD:

(1) If $\overline{T} \leq 0.28\ T_{LRFD}$, V_{LRFD} is not reduced by presence of tension

(2) If $\overline{T} > 0.28\ T_{LRFD}$, V_{LRFD} is reduced as follows

$$V_{LRFD\ Reduced} = V_{LRFD}\left[1-\left(\frac{\overline{T}}{T_{LRFD}}\right)^{1.5}\right]^{0.67} \quad \text{(See Table 7.1 and Table 7.2)}$$

Where:

d_w = Nominal screw head or washer diameter

t_1 = Thickness of member in contact with screw head or washer

t_2 = Thickness of member not in contact with screw head or washer

F_{u1} = Tensile strength of member in contact with screw head or washer (≤ 62 ksi)

F_{u2} = Tensile strength of member not in contact with screw head or washer (≤ 62 ksi)

T = Required allowable tensile strength [nominal tension force] of the fastener determined by load analysis (ASD)

$\overline{T}$ = Required tensile strength [factored tension force] of the fastener determined by load analysis (LRFD)

T_{ASD} = Allowable tensile strength of the fastener (ASD)

T_{LRFD} = Design tensile strength of the fastener (LRFD)

V = Required allowable shear strength [nominal shear force] of the fastener determined by load analysis (ASD)

$\overline{V}$ = Required shear strength [factored shear force] of the fastener determined by load analysis (LRFD)

V_{ASD} = Allowable shear strength of the fastener (ASD)

V_{LRFD} = Design shear strength of the fastener (LRFD)

Section 2.9 Ponding

The primary reference for ponding design for steel framed roofs is the AISC 360-16 “Specification for Structural Steel Buildings” and Commentary, which covers this topic in Appendix A2. An alternate reference for steel joist framed roofs is the Steel Joist Institute (SJI) Technical Digest No 3, “Structural Design of Steel Joist Roofs to Resist Ponding.”

The AISC Standard states: "The roof system shall be investigated through structural analysis to ensure strength and stability under ponding conditions, unless the roof surface is configured to prevent the accumulation of water." Previous editions of the AISC Standard did not require ponding to be checked when the roof slope was ¼ inch per foot or greater. There are cases where this minimum roof slope is not enough to prevent ponding instability and AISC has changed their requirement accordingly.

AISC 360-16, Appendix 2, provides simplified and improved design methods for checking ponding. When using either method, the steel roof deck contribution to ponding can be considered as follows:

1. Deck supported upon secondary framing (beams or joists), which are supported upon primary framing (girders or joist girders).

   $I_d \geq 25\ (S^4)\ 10^{-6}$

   Where:

   I_d = Deck moment of inertia in in^4 per foot of width

   S = Deck span in feet

The following statement is contained in the AISC Commentary:

> "The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia to 0.000025 times the fourth power of its span length, as provided in Equation A-2-2."

2. When the deck is supported directly upon primary framing members (deck supported on beams which are directly supported on columns), it should be considered to be a secondary member, and the ponding design calculations in AISC 360-16 Appendix 2 should be performed with this assumption.

3. When the deck is supported directly upon walls, it should be considered to be a primary member, and the ponding design calculations in AISC 360-16 Appendix 2 should be performed with this assumption.

Checking the roof system for ponding is the responsibility of the Designer and is not checked by the deck manufacturer or deck supplier. The Designer is referred to AISC 360-16 or SJI TD3 for additional information.

Section 2.10 Reinforcement of Openings

Openings in roof deck may need to be reinforced to restore sufficient capacity that has been reduced by the opening. Cold-formed channels or zees can be inserted into the ribs of the deck to act as beams that span between supporting members.

This Manual has strength information for zee reinforcement, however other sections may be used. It is the responsibility of the Designer to select and specify the necessary reinforcing members.

In cases where the opening is large, the spans are long, or the loads are large, it will be necessary to provide a supporting frame under the deck that spans between supports. The responsibility for the design and specification of this frame lies with the Designer and is not provided by the deck supplier or deck installer.

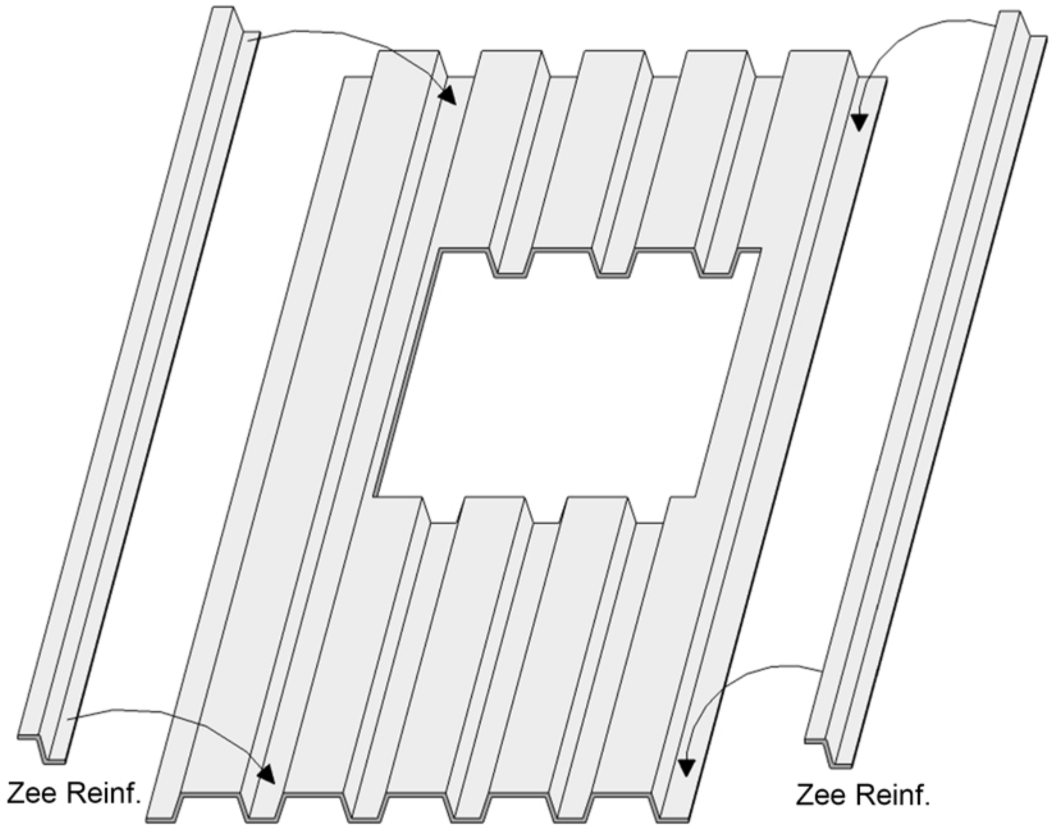

Figure 2.2 – Deck Reinforcing

Section 2.11 Lightweight Insulating Concrete

An alternative to rigid insulation boards, lightweight insulating concrete (LWIC) is cast on steel roof or form deck. It has a density of 20 to 40 pounds per cubic foot and should not be confused with lightweight structural concrete that has a density of 100--120 pounds per cubic foot. Lightweight structural concrete is frequently used on steel deck in floor construction, but rarely used in roof construction.

Lightweight insulating concrete is made using lightweight aggregate such as vermiculite or perlite or by using chemical additives to create cellular concrete. Additional information on LWIC can be found in American Concrete Institute Committee Report “ACI 523.1 Guide for Cast-in-Place Low Density Cellular Concrete”. Aggregates for such concrete are covered in ASTM Specification “C332 Standard Specification for Light-Weight Aggregates for Insulating Concrete”.

Roofs of LWIC rely on the substrate for the capacity to support dead and roof live loads. The lightweight insulating concrete is only a fill that contributes to dead load.

Some LWIC systems require venting of the steel deck, usually in the form of slots in the bottom flange and/or the webs so that water can dissipate from both the top and bottom sides of the concrete. If required, the amount of venting necessary, as a percentage of the surface area, will vary by LWIC product chosen. Generally, vermiculite or perlite concretes do require venting, while cellular concretes may not require venting. Venting requirements for a particular LWIC system should be verified with the supplier of that system.

Additionally, it is usually required by the controlling building code or the LWIC supplier that the steel deck be galvanized to a minimum G60 coating thickness. Painted or bare steel surfaces in contact with the LWIC should be avoided.

The Steel Deck Institute has done research to establish diaphragm values for steel deck supporting lightweight insulating concrete. SDI has established two construction types for decks with insulating lightweight fills. Type 1 consists of at least 2-1/2″ of vermiculite aggregate concrete over the top of the steel deck. Type II is a built-up composite in which a board of at least two inches in thickness, made of expanded cellular polystyrene, is embedded in the LWIC. Diaphragm values for Type I construction are presented in tables in the SDI Diaphragm Design Manual. It should be noted that most LWIC is used as part of proprietary insulating systems and that manufacturers' literature should be consulted.

Section 2.12 Fire Resistance

The Designer should consider required fire resistance ratings in the design of the deck. Many fire rated assemblies that use steel roof decks are available. In the Underwriters Laboratories "Fire Resistance Directory", the deck constructions show hourly ratings for restrained and unrestrained assemblies. ASTM E119 provides information in Appendix X3 titled "Guide for Determining Conditions of Restraint for Floor and Roof Assemblies and for Individual Beams".

Specification of painted deck in areas that require spray-on fireproofing should be avoided unless specifically permitted by the applicable fire rated assembly. This must be clearly called out in the contract documents. In general, there are three types of fire resistive assemblies; those achieving the fire resistance by membrane protection, direct applied protection, or with an unprotected assembly. Of these three, only the systems that utilize direct applied protection are concerned with the finish of the steel deck. In these systems, the finish of the steel deck can be the factor that governs the fire resistance rating that is achieved. In assemblies with direct applied fire protection the finish (paint) is critical. In the Underwriters Laboratories Fire Resistance Directory, some of the individual deck manufacturing companies have steel deck units that are classified in some of the P700, P800, and P900-series roof-ceiling designs. These classified deck units are shown as having a galvanized finish or a painted/painted finish. These classified deck units have been evaluated for use in these specific designs and found acceptable.

Section 2.13 Roof Deck on Cold-Formed Steel Framing

Steel roof deck is being supported by cold-formed steel truss or rafter systems are systems increasing in use. By using steel roof deck instead of plywood or OSB sheathing, truss spacing can be increased from 2 foot to 4 foot on center or more. This often results in more cost effective roof systems that are non-combustible.

When using roof deck on cold-formed steel framing, Designers should be aware of the following points.

1. When steel deck is installed on cold-formed framing, welded support connections are usually not recommended due to difficulties in welding sheet steel to sheet steel. Screw attachment is preferred for this application.
2 When loaded in tension, screw pullout must be considered as a limit state, whereas when deck is installed on thicker supporting material, screw pullover will most likely control. With cold-formed trusses, Designers must review the top chord material thickness with regard to deck fastener pattern to provide the necessary uplift and shear resistance. In high wind zones, the gage of the deck and the top chord may have to be increased.

3. The Designer must pay particular attention to ensuring that all bearing edges of the deck are supported by adequate structural supports. Referring to Figure 2.3, the single dashed lines at the hips are points of support for the deck. The cold-formed truss manufacturer likely will not provide structural bearing at the hips, ridges and valleys of roof systems. Likewise, neither the steel deck supplier nor the installer is responsible for designing or installing this structural support. It is the responsibility of the Designer to properly design and specify this support. Typical ridge or valley plates that may be provided by the deck supplier are provided to allow the deck to transition and do not provide adequate structural support. Refer to the SDI Code of Standard Practice for additional information.
4. Particular attention must be paid to conditions that may cause the deck to bear on a “knife edge.” The SDI RD-2017 Standard, in Section 2.4.A.7 addresses minimum bearing length and Section 3.1.G addresses out of plane supports. An example is at a corner jack truss, located by the section mark in Figure 2.3 and as shown in Figure 2.4. In addition to bearing problems, the gap makes it difficult or impossible to ensure proper screw installation. It is not the responsibility of the deck supplier or deck installer to provide supplemental framing material to ensure proper bearing of the deck in this or similar conditions. The Designer must provide sufficient information in the project plans and specifications to provide adequate deck bearing.
5. Changes in deck direction within a single plane (for example, over a hip truss as shown in Figure 2.3) need additional deck bearing width to allow for proper attachment and cover plates for diaphragm continuity.
6. The Designer is responsible for the design of the diaphragm system, including all diaphragm chords and shear collectors.
7. Overbuilds created by "dummy" dormers or by two or more intersecting ridge lines over steel deck is not recommended. This condition creates a valley condition with a large line load on the deck. The Designer should provide steel framing to steel framing at all overbuild areas.

The SDI Steel Deck on Cold-Formed Steel Framing Design Manual is an excellent reference which provides guidance for this application of steel deck.

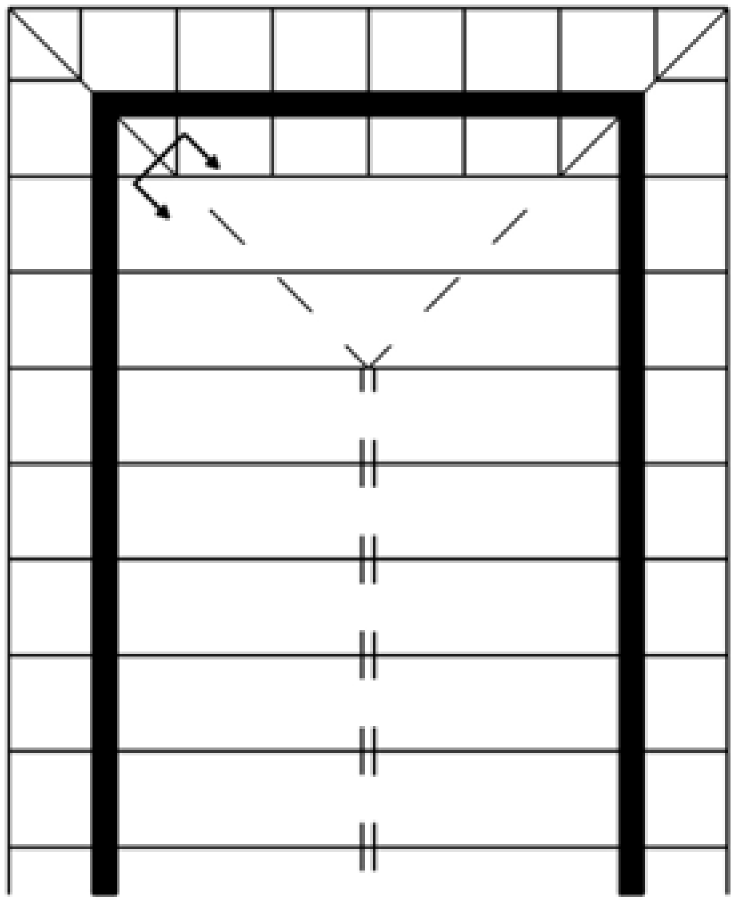

Figure 2.3 – Hip Roof Plan

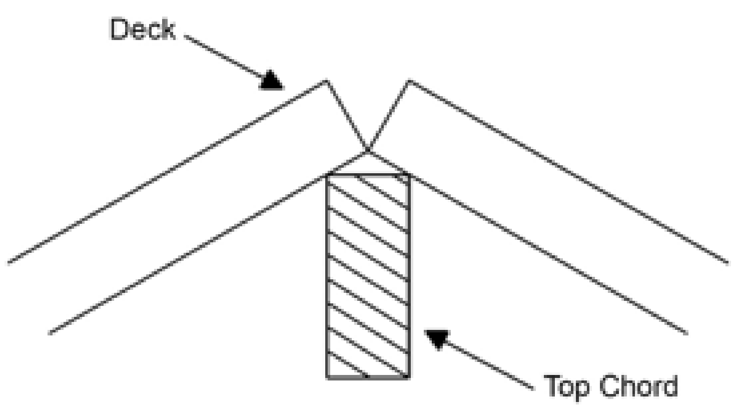

Figure 2.4 – Deck Support at Corner Jack Truss

Section 2.14 Mechanically Attached Single-Ply Roof Membranes

For both new construction and reroofing or recovering of existing roofs, mechanically attached single-ply roofing membranes are often used. As opposed to adhered membranes which are continuously adhered to the insulation board, and uniformly load the deck when loaded by wind uplift, mechanically attached membranes are attached using lines of mechanical fasteners at the membrane seams. These seams can be spaced from 4 foot to over 20 foot on center in some applications. Then loaded by wind uplift, these membranes load the steel deck with line loads that can increase the bending moment by several hundred percent over a uniformly applied uplift.

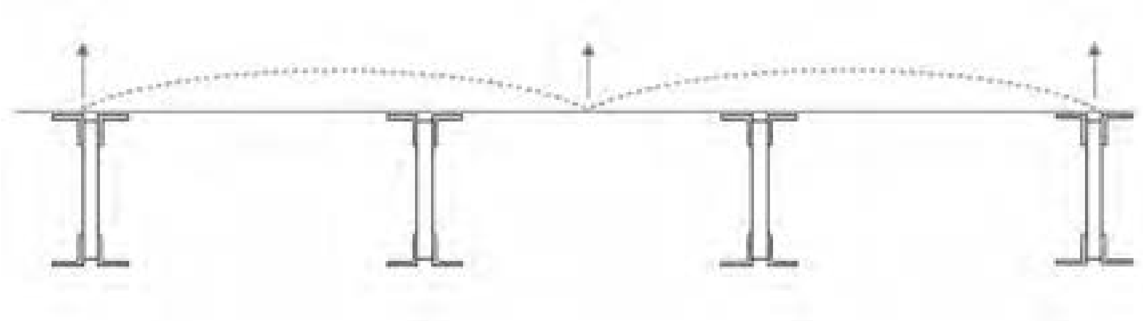

Figure 2.4 – Membrane Loading Deck

Some key points that the Designer should keep in mind.

1. The lines of fasteners must not be permitted to be placed parallel to the roof deck ribs. In this case, the deck resistance in reduced to that of a single rib, and failure of the steel deck is inevitable.
2. The Designer must consider the effect of the line loads on the steel roof deck and underlying framing, and design the deck and framing accordingly.
3. When reroofing or recovering an existing roof, strict attention must be given to the capacity of an existing steel deck that was originally designed for a uniform uplift load. A mechanically attached membrane may not be feasible.

SDI Technical Note 7, "Mechanical Attachment of Single-ply Roofing Membranes to Steel Roof Deck: Implications for Steel Deck Design" should be referred to for additional information.

FASTENERS

SECTION 3

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Section 3.1 Fasteners

The importance of quality fastening cannot be overstated. The type and number of fasteners primarily depends on horizontal (diaphragm) shear loads and (or) uplift tension loads. Horizontal loads result from wind or earthquakes, uplift loads from wind suction. The magnitude of these loads must be determined by the Designer as part of the overall building design process, and are not known by the deck supplier or detailer. Fastening is therefore a design function and the type and number of fasteners cannot be selected by the deck supplier or by the deck detailer, but should clearly be shown on the drawings by the Designer. The information in this Manual is presented to aid the Designer in the choice of fasteners. In no case should the deck installer be allowed to substitute fasteners for those shown on the plans without first consulting the Designer.

Deck fasteners can be mechanical fasteners (screws, power-actuated fasteners, or mechanically formed connections) or welds.

Fasteners are described by function as follows:

Support Fastener: A fastener connecting one or more sheets to the support framing. Also referred to as a structural fastener.

Side-lap Fastener: A fastener connecting adjacent panels to each other, but not connecting to the support framing. Also referred to as a stitch fastener.

Section 3.2 Welding

Sheet steel welding methods and operator qualifications are described in the American Welding Society Structural Welding Code - Sheet Steel, AWS D1.3. Arc welding can be used for both support and side-lap connections. The essential issues in forming a good weld rest in bringing the surfaces to be welded to fusion temperature at the same time, avoiding “burn-out” in the sheet, achieving adequate penetration into the beam or joist, and obtaining proper weld engagement on the weld perimeter.

The most common filler metal used for welding steel deck is an E6022 electrode, due to the ability of that electrode to produce welds with good penetration. AWS D1.3 does permit the use of "undermatched" electrodes, provided that the strength of the welded connection is calculated. Therefore, an E60xx electrode can be used with any strength of sheet steel. Strength of welds can be calculated using the provisions of AISI S100. Additionally, the use of low-hydrogen electrodes is seldom necessary and may actually reduce the weld quality due to the higher amperages usually required for these electrodes.

Refer to SDI Technical Note No. 2 "SDI Commentary on Application of AWS D1.3-2018 to Welding of Steel Deck" for further discussion on electrode selection.

<u>Section 3.2.1 Arc Spot Welds</u>

Arc spot welds may be used for support fasteners, however, they can also be used for side-lap fasteners for deck that is 20 gage or thicker. The arc spot or “puddle” weld is started by striking an arc on the deck surface causing a hole to form in the deck. The weld operation then continues by depositing electrode material on the beam or joist and allowing the molten “puddle” to engage the penetrated deck. It is essential that the finished weld penetrates into the supporting beam or joist and that the puddle engages the deck on the weld perimeter. The complete welding process usually requires an average arc time in excess of 8 seconds for a 5/8" diameter weld, and an average in excess of 13 seconds for a 3/4" diameter weld. Additional time may be required to weld multiple deck thicknesses. Weld quality also depends heavily on the properties of the parts being connected, welding machine settings and experience / technique of the welder. This process requires a welder who is AWS qualified to make these specific welds on the roof deck project. Arc spot welds can be made through multiple thicknesses of steel deck, as long as the total base metal (bare steel) thickness of the deck does not exceed 0.15 in.

Figure 3.1 – Arc Spot Weld

Weld washers are small elements of sheet steel with a punched hole at their center and may be curved to fit into the valleys of deck panels. Washers may be of differing thickness and have different hole diameters or hole shapes. The most common type is approximately 0.06” thick with holes of 3/8” in diameter and a minimum ultimate strength of 45 ksi, and may be designated as 3/8” x 16 gage washers. Weld washers are laid in position on the deck units, an arc is struck on the sheet inside the hole, and the operation continues usually until the hole is filled. The weld washer acts as a heat sink and retards burn-away of the sheet. The washer permits welds in thin deck that might otherwise burn away from the welding operation faster than weld material can be deposited.

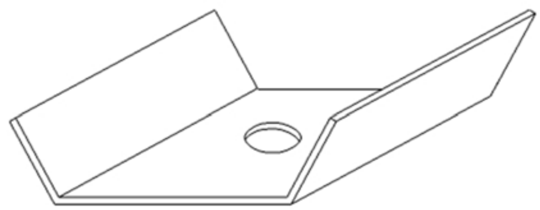

Figure 3.2 – Weld Washer

Weld washers are not recommended with deck design thicknesses equal to or greater than 0.028 inches because their use may actually reduce the weld penetration. Weld washers are required for welding in deck panels thinner than 0.028 inches. 22 gage deck is 0.0295 inches thick and does not require weld washers.

Diaphragm tables using arc spot welds may be based on "visible" or "effective" diameters; therefore, Designers should clarify weld diameters when referencing these tables.

Welding machine power settings required usually are well below those needed for welding in hot-rolled steels. The settings should be such that electrode burn-off rates are between 0.15 inches and 0.25 inches of rod per second in typical E6022 or E7014, 5/32 inch diameter rods. Heavier support steel requires more welding time, but increased power settings may burn out the deck faster than electrode material can be deposited.

A preliminary field quality check can be made by placing a pair of welds in adjacent valleys at one end of a panel. An inspection should show the weld material in fused contact over most of the weld perimeter. Spotty contact may indicate power settings that are excessive. The opposite end of the panel can be rotated, within the panel plane, placing the welds in shear, and continued rotation can lead to separation. Separation, leaving no apparent external perimeter distress, but occurring at the sheet-to structure plane, may indicate insufficient welding time and poor fusion with the support steel. Failure around the external weld perimeter, showing distress within the panel, but the weld still attached to the support steel, indicates a higher quality weld. The ending of the welding operation may not permit complete fusion on the whole perimeter. Good fusion should be visible over no less than 90 percent of the weld perimeter (Clause 6.1.1.4 of AWS D1.3 permits undercut on 12% of the weld perimeter).

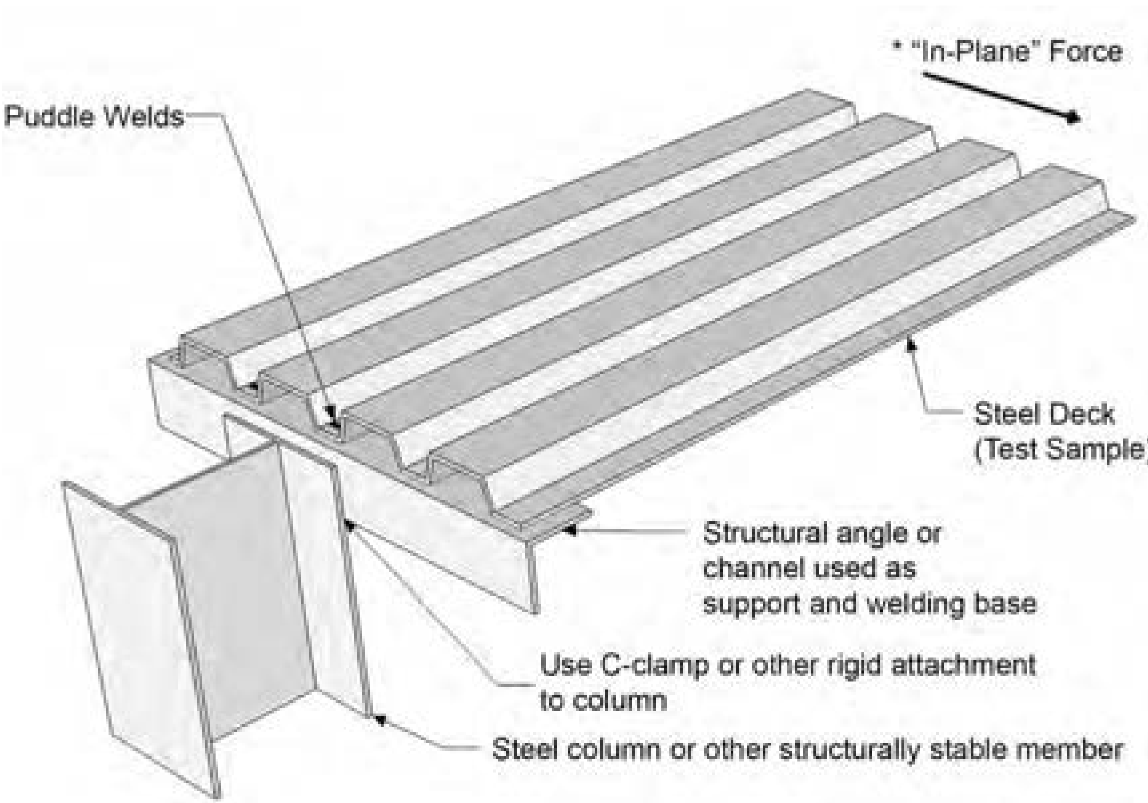

Figure 3.3 – Field Arc Spot Weld Quality Control Test Procedure

<u>Section 3.2.2 Arc Top Seam and Arc Fillet Welds</u>

The top arc seam side-lap weld has long tenure in roof deck diaphragm side-laps. As with all deck welds, field quality control by the erector is required. Greater deck depth and narrow flute gaps increase the difficulty of making this connection. As shown in Figures 3.4 and 3.5, both vertical-to-vertical and hem-to-vertical connections are possible. In either case, firm contact is required for fusion and shear transfer. The hem lap is pinched or button punched to clamp the vertical leg and to establish contact between the three vertical legs. The hem lap must be burned through and fusion established at the top of at least the two adjacent vertical legs with one being in each of the respective panels. At a hem lap, that leg must be closest to the center of the panel. With proper clamping, fusion at all three legs is common and preferred. Fusion must exist at both vertical legs. Blowholes at top seam side-lap weld ends are to be expected and are not detrimental to the strength, which is based on the fused length.

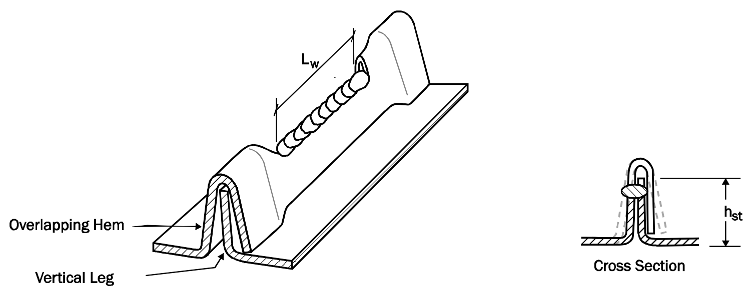

Figure 3.4 – Arc top seam weld - Vertical Leg and Overlapping Hem Joint

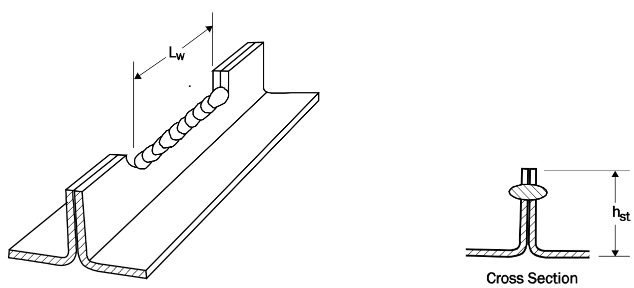

Figure 3.5 – Arc top seam weld - Back-to-Back Vertical Leg Joint

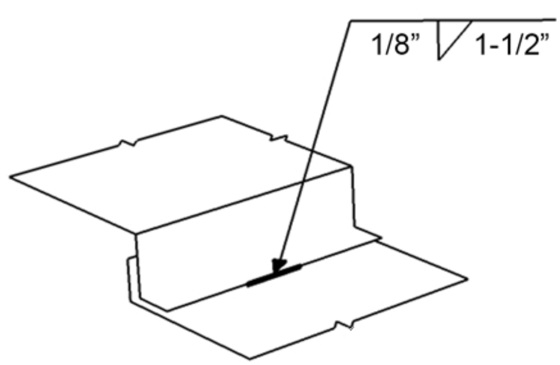

Figure 3.6 – Side-lap Fillet Weld

Figure 3.6 shows a fillet weld at a nested side-lap. The design weld leg size does not exceed the thickness of the thinner sheet joined per AWS D1.3, Clause 2.3.3.2. However, it is common (but not required) to specify a minimum 1/8 inch weld to ensure that an adequate weld is made.

Section 3.3 Screw Fasteners

Screw fasteners are commonly used for structural and side-lap connections of steel decks and work well for attachment of most nestable profiles and types. These mechanical fastening systems are alternates to arc spot welding or power-actuated fasteners.

Common side-lap screw sizes include #10 and #12 self-drilling screws. #12, #14 or ¼" screws are generally used for steel deck roof attachment to structural supports. These structural screws resist both uplift and shear.

Self-drilling screw points drill through the steel deck panels and support steel prior to threads tapping. Typical screw drill points range from No. 3 – 5. See Figure 3.7 below for some typical self-drilling tapping screw profiles.

Figure 3.7 – Typical Screws

Standard screws generally have hex washer head configurations. Screws may also be configured with pre-mounted steel washers that help in resisting pullover due to wind uplift forces.

AISI S100 Section J4 provides design provisions for screw connections and performance in shear limited by tilting and bearing failures, tension pullout, tension pullover and combined shear and tension limit states. The values presented in this Manual are consistent with the AISI S100 design provisions for screws.

Proprietary screws are also available for structural and side-lap screw applications. The capacities of these screws are developed through testing in accordance with AISI S904 and S905 test methods. The SDI Diaphragm Design Manual, the AISI S100 Standard, and specific product evaluation reports provide design equations or capacities of these screw fasteners in shear and tension.

Ergonomic, stand-up screw fastening systems exist on the market today and offer high production rates for speedy installation. Additionally some manufacturers have collated screws available reducing the re-load time and increasing the deck installer productivity. Screw fastening tools can access standard and deep deck profiles and offer a reliable connection method for steel roof deck attachment. Electric or battery operated screw fastening tools with maximum 2500 rpm should be used to avoid burning up of the screw drill tip. Screw fastening tools should have torque clutches and/or depth gauges to avoid over-driving the screw and fracturing of screw heads. Screw fasteners should be installed with a minimum of three threads protruding through the base steel component.

Side-lap screw connections must positively engage both steel deck roof panel sheets. See Figure 3.9 for illustrations of screw side-lap connections.

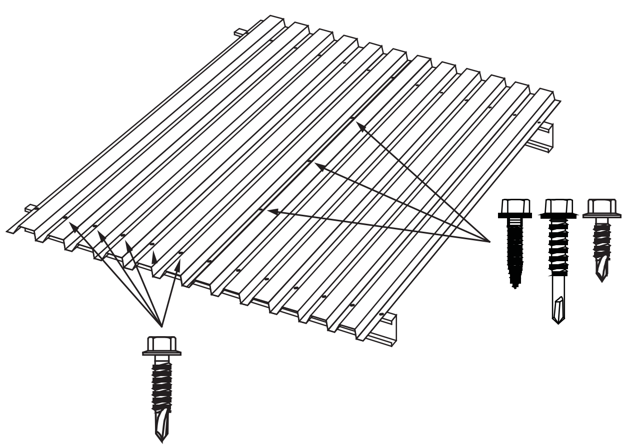

Figure 3.8 – Support and Side-lap Screws

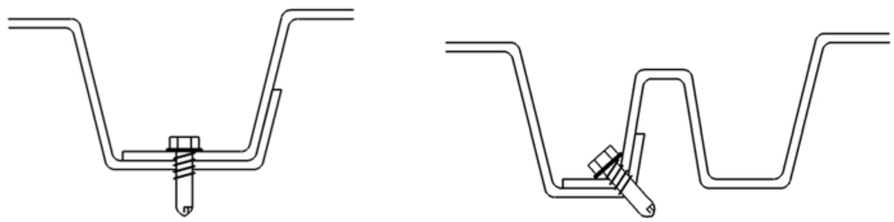

Figure 3.9 – Screw Side-lap Connections

Section 3.4 Power-Actuated Fasteners

Power-actuated fasteners are commonly used for steel roof deck attachment, resisting both uplift and diaphragm loading. These mechanical fastening systems are alternatives to arc spot welding or screw fastening of the steel roof panels to the supporting steel members (open web steel joists or steel beams). Power-actuated fasteners are not used for steel deck side-lap connections. Power-actuated fasteners are acceptable with all steel roof deck profiles and thicknesses, provided that the fastener head or washer diameter is not too large for the lower deck flute, and that the installation tool can fit in the profile, for attachment to base steels of thickness 1/8 inch or greater. Attachment to support thicknesses less than 1/8” may be permitted for specific fasteners.

Each system is proprietary in nature, with different sources of driving energy, either powder-actuated cartridges or compressed air. Today’s power-actuated fastening tool systems are advanced in design with ergonomic, stand-up systems available that can improve deck installer productivity. Together with large magazines for holding many fasteners, this is a very efficient and reliable method of installing roof deck.

Types of Power-Actuated Fasteners

Power-actuated fasteners are available with pre-mounted steel washers to assist in clamping the steel deck panels to the supporting steel members. Other fastener types have large heads without steel washers. The washers or large heads also serve to resist tension pull-over due to wind uplift forces. The SDI Diaphragm Design Manual provides more information on power-actuated fasteners used for steel deck attachment.

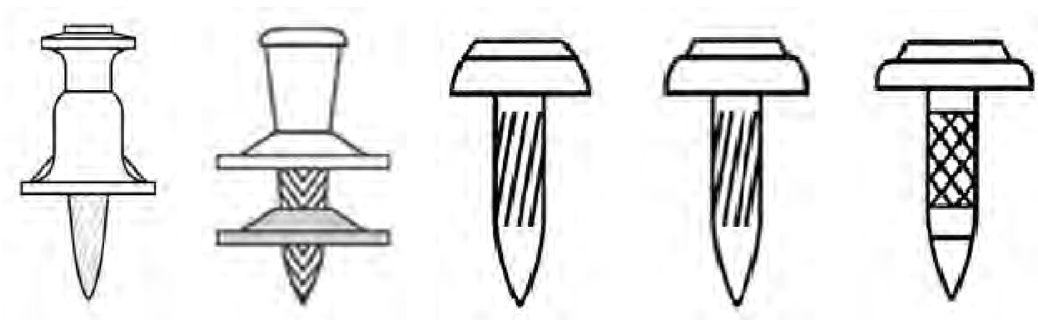

Figure 3.10 – Examples of Power-actuated Fasteners

Technical Data

Design equations for tension, shear and combined loading of power-actuated fasteners are provided in the SDI Diaphragm Design Manual. These equations are based on extensive research done by manufacturers and supported by the SDI. Performance data are developed for power-actuated fasteners through testing programs conducted in accordance with the AISI S905, AISI S907 and ASTM E1190 test standards.

The SDI Diaphragm Design Manual provides information on power-actuated fasteners for resisting tension uplift and combined shear and tension forces due to combined horizontal diaphragm shear loads and wind uplift. These design equations can be used alternatively to the design equations provided in the RDDM for screw fasteners and welds.

Alternatively, AISI S310, Section B1.1, allows for nominal strength of the connection to be determined by testing and by calibration in accordance with Section E1 or E2 of AISI S310. These nominal strengths, along with calibrated safety factors, may be used in lieu of the equations provided in the DDM.

Other sources for technical data for International Building Code (IBC) compliance are ICC-ES Evaluation Reports based on AC43 Acceptance Criteria for Steel Deck Roof and Floor Systems or IAPMO-ES (International Association of Plumbing and Mechanical Officials) Evaluation Reports based upon AC43 and IAPMO EC 007 Acceptance Criteria.

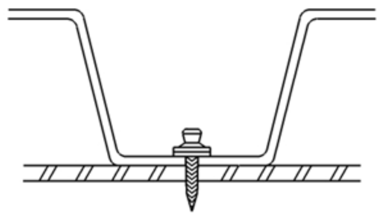

Figure 3.11 – Power-Actuated Fastener Attachment of Steel Deck to Framing

Section 3.5 Mechanically Formed Side-lap Connections

Side-lap connections can be formed by crimping the upstanding edge of interlocking side-lap deck. Crimps can only be made with deck that is designed with the upstanding edge to receive them and not all deck has upstanding edges that will accept crimps. Crimping can be categorized as either generic "button punching" or one of several proprietary mechanically formed connection systems.

Generic button punches serve only to align the deck side-laps and provide little resistance to shear at the panel edge. Proprietary mechanically formed connection systems are tested connections formed using specific tools for a specific deck. These proprietary systems have defined shear strength and stiffness values that are contained within research and evaluation reports. Information on these proprietary systems can be obtained from specific manufacturers.

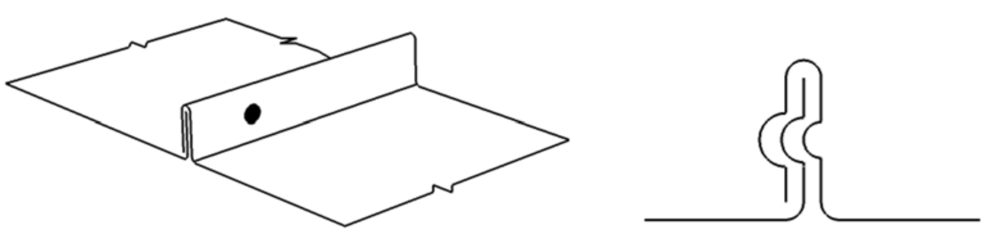

Figure 3.12 – Side-lap Button Punch

CONSTRUCTION PRACTICES

SECTION 4

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Section 4.1 Common Industry Practices

The Designer should be aware of common industry practices related to deck design and installation. Excellent sources of this information include the SDI Code of Standard Practice, the SDI Manual of Construction with Steel Deck, and the SDI Standard Practice Details. The most efficient and economical roof deck construction results when the design adheres to these industry practices.

Section 4.2 Welding Requirements

The deck installation contractor is responsible for following AWS D1.3 and Welding Procedure Specification (WPS) documents. A WPS is a detailed document providing required variables for a specific welding application to assure repeatability by properly trained welders and welding operators. WPS documents must be written for all welds permitted as prequalified and all welds qualified in conformance with Clause 6 of AWS D1.3.

Each deck installation contractor must be responsible for inspection and testing of WPS qualification tests and welder performance testing as described in AWS D1.3. Arc spot weld WPS are not described in Clause 5 of AWS D1.3 and, therefore, must be qualified by testing and recorded on a Procedure Qualification Record (PQR.)

Visual inspection is required to determine if a weld meets the acceptance criteria of AWS D1.3. It is the deck installation contractor's responsibility to ensure that all WPSs and welders are qualified. The EOR may accept previously qualified or prequalified WPS. However, if the EOR does not accept such evidence, the deck installation contractor must successfully complete the required tests prior to welding.

Section 4.3 Mechanical Fastening Requirements

The deck installation contractor is responsible for complying with the installation instructions provided by the fastener supplier for the specific fastener. Training provided by the fastener supplier in the installation of the fasteners and operation of fastening equipment may be required.

Some general criteria for proper fastener installation include:

Proper fastener driving (not over- or under-driven)

Proper fastener selection for the substrate and total fastened metal thickness

Complying with minimum edge distance limits on both the deck and substrate

Generally, visual inspection of the installed fasteners is required. Some fastener suppliers may provide "feeler gages" to check for proper fastener driving.

Section 4.4 Adhesion of Spray Fireproofing and Field Paint

If spray fireproofing is to be applied to the deck, the Designer must verify that the prime paint being provided is compatible with the fireproofing that is specified. Galvanized and bare deck can also receive spray fireproofing. The adherence of fireproofing materials is dependent on many variables and neither the seller, the deck manufacturer, nor the deck installer can be responsible for the adhesion or adhesive ability of the fireproofing.

If field painting is intended, it is recommended that the steel surface, whether galvanized or prime painted, be checked for compatibility by the painting contractor, following the recommendations of the field coating manufacturer, particularly with regard to ambient application temperatures and humidity, cleanliness, surface moisture and surface preparation if required.

Section 4.5 Repair of Deck Finish at Welds

In most cases, deck welds are removed from a corrosive environment when the roof covering is installed and the building is dried-in, and no weld touch up paint or cold galvanizing is necessary. In those instances where the welds are left exposed to a corrosive atmosphere, the weld should be wire brushed and coated with an approved coating material, specified by the Designer.

Section 4.6 Quality Control and Quality Assurance

The SDI has developed the ANSI/SDI QA/QC-2017 “Standard for Quality Control and Quality Assurance for Installation of Steel Deck” which provides requirements for steel deck installation quality in a mandatory format that can be used for inspection purposes. This Standard is available for download from the SDI website and it is recommended that Designers require compliance with the quality procedures in the Standard through incorporation of the Standard in project specifications to promote quality in deck installation.

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TABLES

SECTION 5

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LIST OF TABLES

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Table 1 – Section Properties and Flexural Resistance

Profile	Gage Number	Design Thickness (inches)	Weight (psf)	$I_{x\ gross}$ (inch^4/foot)	$I_{p\ eff}$ (inch^4/foot)	$I_{n\ eff}$ (inch^4/foot)
WR	22	0.0295	1.6	0.180	0.137	0.170
WR	20	0.0358	2.0	0.217	0.177	0.213
WR	18	0.0474	2.6	0.283	0.253	0.283
WR	16	0.0598	3.3	0.353	0.337	0.353
DR	22	0.0295	2.0	0.848	0.570	0.756
DR	20	0.0358	2.4	1.024	0.725	0.946
DR	18	0.0474	3.2	1.345	1.034	1.299
DR	16	0.0598	4.0	1.675	1.400	1.650

Profile	Gage Number	$I_{D\ gravity}$ (inch^4/foot)	$I_{D\ uplift}$ (inch^4/foot)	$S_{p\ eff}$ (inch^3/foot)	$S_{n\ eff}$ (inch^3/foot)
WR	22	0.151	0.173	0.171	0.179
WR	20	0.190	0.214	0.212	0.223
WR	18	0.263	0.283	0.287	0.295
WR	16	0.342	0.353	0.366	0.374
DR	22	0.663	0.787	0.353	0.398
DR	20	0.825	0.972	0.464	0.506
DR	18	1.138	1.314	0.643	0.694
DR	16	1.492	1.658	0.847	0.893

		ASD		LRFD	
Profile	Gage Number	M_p/Ω (inch-lbs/foot)	M_n/Ω (inch-lbs/foot)	ΦM_p (inch-lbs/foot)	ΦM_n (inch-lbs/foot)
WR	22	4096	4298	6156	6460
WR	20	5078	5347	7632	8037
WR	18	6874	7061	10332	10613
WR	16	8766	8965	13176	13475
DR	22	8455	9533	12708	14328
DR	20	11114	12120	16704	18216
DR	18	15405	16619	23153	24978
DR	16	20283	21384	30485	32140

Table 1 Notes:

1. Strength and section properties are calculated assuming F_y = 40 ksi and F_u = 50 ksi. Individual deck manufacturers may supply deck using lower or higher strength steel.
2. All section properties and ASD and LRFD flexural strengths are calculated in accordance with ANSI/SDI RD-2017, Section 2.4.A.1; p = Property in positive bending; n = Property in negative bending.
3. For uniformly distributed loads on steel deck, the following weighted averages of the full and effective section properties provide a good estimate of an average equivalent effective section property for the estimation of deflection under service loads.

Simple span: $I_D = (I_x + 2I_{p\ eff}) / 3$

Multiple span: $I_D = (I_x + 2I_{n\ eff}) / 3$ if negative bending controls, or
$I_D = (I_x + 2I_{p\ eff}) / 3$ if positive bending controls.

Where:

I_D = Moment of inertia for deflection calculation under uniform load

I_x = Full (gross) moment of inertia

$I_{p\ eff}$ = Effective moment of inertia, positive bending

$I_{n\ eff}$ = Effective moment of inertia, negative bending

Table 2 – WR Deck

TABLE 2.1
1.5 WR ASD Superimposed Uniform Downward Loads (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	108	89	74	63	54	47
	20	133	110	92	78	67	58
	18	181	149	125	106	91	79
	16	230	190	159	135	116	101
Double	22	113	93	78	66	57	49
	20	141	116	97	82	71	61
	18	186	153	128	109	93	81
	16	236	194	163	138	119	103
Triple	22	142	117	98	83	71	62
	20	176	145	122	103	89	77
	18	233	192	161	137	117	102
	16	296	244	204	174	149	130

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	41	36	32	29	26	23
	20	51	45	40	36	32	29
	18	69	61	54	48	43	39
	16	88	78	69	61	55	50
Double	22	43	38	34	30	27	24
	20	54	47	42	37	34	30
	18	71	63	56	50	44	40
	16	90	79	70	63	56	51
Triple	22	54	48	43	38	34	31
	20	68	60	53	47	43	38
	18	89	79	70	63	56	51
	16	113	100	89	79	71	64

Span Cond.	Gage Number	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
Cantilever	22	317	177	113	78	57	43
	20	394	221	141	97	71	54
	18	520	292	186	128	93	71
	16	661	370	236	163	119	90

TABLE 2.2
1.5 WR ASD Superimposed Uniform Upward Loads (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	116	96	81	69	60	53
	20	145	120	101	86	75	65
	18	191	158	133	114	99	86
	16	242	201	169	145	125	110
Double	22	111	92	77	66	57	50
	20	137	114	96	82	71	62
	18	186	154	130	111	96	84
	16	237	197	166	142	123	107
Triple	22	138	114	96	82	71	62
	20	171	142	120	102	88	77
	18	232	192	162	138	120	104
	16	296	245	206	176	152	133

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	46	41	37	33	30	28
	20	58	51	46	41	38	34
	18	76	68	61	55	50	45
	16	97	86	77	70	63	58
Double	22	44	39	35	32	29	26
	20	55	49	44	40	36	33
	18	74	66	59	53	48	44
	16	95	84	75	68	62	56
Triple	22	55	49	44	39	36	33
	20	68	61	54	49	44	40
	18	92	82	73	66	60	55
	16	117	104	93	84	76	70

Span Cond.	Gage Number	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
Cantilever	22	305	172	111	77	57	44
	20	378	214	137	96	71	55
	18	512	289	186	130	96	74
	16	653	369	237	166	123	95

TABLE 2.3
1.5 WR LRFD Superimposed Uniform Downward Loads (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	162	134	112	95	82	71
	20	201	166	139	118	101	88
	18	272	225	188	160	137	119
	16	347	286	240	204	175	152
Double	22	170	140	118	100	86	75
	20	212	175	146	124	107	93
	18	280	231	193	164	141	123
	16	355	293	246	209	179	156
Triple	22	213	176	148	126	108	94
	20	266	219	184	156	134	117
	18	351	289	243	206	177	154
	16	445	367	308	262	225	196

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	62	55	49	44	39	35
	20	77	68	60	54	48	44
	18	105	92	82	73	66	59
	16	133	118	104	93	84	76
Double	22	65	58	51	46	41	37
	20	81	72	64	57	51	46
	18	107	95	84	75	68	61
	16	136	120	107	96	86	78
Triple	22	82	73	65	58	52	47
	20	102	90	80	72	65	58
	18	135	119	106	95	85	77
	16	171	151	135	120	108	98

Span Cond.	Gage Number	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
Cantilever	22	477	267	170	118	86	65
	20	593	332	212	146	107	81
	18	783	439	280	193	141	107
	16	994	557	355	246	179	136

TABLE 2.4
1.5 WR LRFD Superimposed Uniform Upward Loads (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	174	144	122	104	90	78
	20	217	180	151	129	112	98
	18	286	237	200	171	148	129
	16	363	301	253	217	187	164
Double	22	166	138	116	99	86	75
	20	206	171	144	123	106	93
	18	279	231	194	166	144	126
	16	355	294	248	212	183	160
Triple	22	207	180	144	123	107	93
	20	257	224	179	153	132	115
	18	348	295	242	207	179	156
	16	443	375	309	264	228	199

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	69	62	55	50	45	41
	20	86	77	69	62	56	51
	18	114	101	90	82	74	67
	16	144	128	115	103	94	85
Double	22	66	59	53	47	43	39
	20	82	73	65	59	53	49
	18	111	98	88	79	72	66
	16	141	126	112	101	92	84
Triple	22	82	73	65	59	53	48
	20	102	90	81	73	66	60
	18	138	122	109	99	89	81
	16	176	156	140	126	114	104

Span Cond.	Gage Number	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
Cantilever	22	458	258	166	116	86	66
	20	568	320	206	144	106	82
	18	768	434	279	194	144	111
	16	980	553	355	248	183	141

TABLE 2.5
1.5 WR Uniform Service Load that Causes L/120 Deflection (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	157	118	90	71	56	46
	20	198	148	114	89	71	57
	18	274	205	157	123	98	79
	16	356	267	205	160	128	103
Double	22	381	286	220	173	138	112
	20	479	359	276	217	173	141
	18	663	497	382	300	240	195
	16	863	647	498	391	312	253
Triple	22	298	223	172	135	108	87
	20	375	281	216	169	135	110
	18	518	389	299	234	187	152
	16	675	506	389	305	244	198

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	37	31	26	22	18	16
	20	47	39	32	27	23	20
	18	65	54	45	38	32	27
	16	85	70	58	49	42	36
Double	22	92	76	64	54	46	40
	20	115	96	80	68	58	50
	18	160	133	112	94	81	69
	16	208	173	145	123	105	90
Triple	22	72	59	50	42	36	31
	20	90	75	63	53	45	39
	18	125	103	87	73	62	54
	16	162	135	113	96	81	70

TABLE 2.6
1.5 WR Uniform Service Load that Causes L/180 Deflection (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	104	78	60	47	37	30
	20	131	98	75	59	47	37
	18	182	136	104	81	65	52
	16	237	177	135	106	84	68
Double	22	254	190	146	115	91	74
	20	319	239	184	144	115	93
	18	441	331	254	199	159	129
	16	574	431	331	260	207	168
Triple	22	198	148	114	89	71	58
	20	249	187	143	112	89	72
	18	345	258	198	155	124	100
	16	449	336	258	202	161	131

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	24	20	17	14	12	10
	20	31	25	21	17	15	12
	18	42	35	29	24	20	17
	16	55	46	38	32	27	23
Double	22	61	50	42	36	30	26
	20	76	63	53	45	38	33
	18	106	88	73	62	53	45
	16	138	114	96	81	69	59
Triple	22	47	39	33	28	23	20
	20	59	49	41	35	29	25
	18	82	68	57	48	41	35
	16	107	89	74	63	53	45

TABLE 2.7
1.5 WR Uniform Service Load that Causes L/240 Deflection (psf)

Span Cond.	Gage Number	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"
Single	22	78	58	44	35	27	22
	20	98	73	56	43	34	28
	18	136	101	77	60	48	38
	16	177	132	101	79	62	50
Double	22	190	142	109	86	68	55
	20	239	179	137	107	86	69
	18	330	247	190	149	119	96
	16	430	322	247	194	155	125
Triple	22	148	111	85	67	53	43
	20	186	139	107	84	67	54
	18	258	193	148	116	92	75
	16	336	251	193	151	120	97

Span Cond.	Gage Number	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"
Single	22	18	15	12	10	8	7
	20	22	18	15	13	10	9
	18	31	26	21	18	15	12
	16	41	33	28	23	19	16
Double	22	45	37	31	26	22	19
	20	57	47	39	33	28	24
	18	79	65	54	46	39	33
	16	102	85	71	60	51	43
Triple	22	35	29	24	20	17	15
	20	44	36	30	25	22	18
	18	61	50	42	35	30	26
	16	79	66	55	46	39	33

TABLE 2.8
1.5 WR Construction Spans
(ANSI/SDI RD-2017 Section 2.4.A.3 and 2.4.A.4)

Gage Number	Span Cond.	ASD Span	Span Cond.	ASD Span	Span Cond.	ASD Span
22	Single	6'-7"	Double or Triple	8'-1"	Cantilever	1'-8"
20		8'-1"		9'-11"		2'-1"
18		10'-8"		13'-0"		2'-8"
16		13'-2"		16'-1"		3'-4"

Gage Number	Span Cond.	LRFD Span	Span Cond.	LRFD Span	Span Cond.	LRFD Span
22	Single	7'-2"	Double or Triple	8'-9"	Cantilever	1'-9"
20		8'-9"		10'-8"		2'-2"
18		11'-8"		14'-2"		2'-10"
16		14'-5"		17'-7"		3'-6"

Table 3 – DR Deck

TABLE 3.1
3 DR ASD Superimposed Uniform Downward Loads (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	55	50	45	41	38	34
	20	72	65	59	54	49	45
	18	100	91	82	75	69	63
	16	132	119	108	99	91	83
Double	22	62	56	51	46	43	39
	20	79	71	65	59	54	50
	18	108	98	89	81	74	68
	16	139	126	115	104	96	88
Triple	22	78	70	64	58	54	49
	20	99	90	81	74	68	63
	18	136	123	112	102	94	86
	16	175	158	144	131	120	111

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	32	29	27	25	23	22
	20	42	39	36	33	31	29
	18	58	54	50	46	43	40
	16	77	71	66	61	57	53
Double	22	36	33	31	29	27	25
	20	46	42	39	36	34	32
	18	63	58	54	50	47	44
	16	81	75	69	65	60	56
Triple	22	45	42	39	36	34	31
	20	58	53	50	46	43	40
	18	79	73	68	63	59	55
	16	102	94	88	81	76	71

Span Cond.	Gage Number	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
Cantilever	22	77	62	51	43	36	31
	20	98	79	65	54	46	39
	18	134	108	89	74	63	54
	16	173	139	115	96	81	69

TABLE 3.2
3 DR ASD Superimposed Uniform Upward Loads (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	65	59	54	50	46	42
	20	83	75	69	63	58	54
	18	113	103	94	86	80	74
	16	146	133	121	111	102	95
Double	22	58	53	48	44	41	38
	20	76	69	63	58	53	49
	18	105	96	87	80	74	68
	16	139	126	115	106	97	90
Triple	22	72	66	60	55	51	47
	20	95	86	79	72	66	61
	18	131	119	109	100	92	85
	16	172	157	143	131	121	111

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	39	36	34	32	30	28
	20	50	46	43	40	38	36
	18	68	63	59	55	52	49
	16	88	82	76	71	67	63
Double	22	35	33	30	28	27	25
	20	46	43	40	37	35	33
	18	63	59	55	51	48	45
	16	83	77	72	68	63	60
Triple	22	43	40	38	35	33	31
	20	57	53	49	46	43	41
	18	79	73	68	64	60	56
	16	103	96	90	84	78	74

Span Cond.	Gage Number	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
Cantilever	22	71	58	48	41	35	30
	20	93	76	63	53	46	40
	18	129	105	87	74	63	55
	16	170	139	115	97	83	72

TABLE 3.3
3 DR LRFD Superimposed Uniform Downward Loads (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	83	75	68	62	57	52
	20	109	99	90	82	75	69
	18	151	137	124	114	104	96
	16	199	180	164	150	137	126
Double	22	94	85	77	70	64	59
	20	119	108	98	89	82	75
	18	163	148	135	123	113	103
	16	210	190	173	158	145	133
Triple	22	117	106	97	88	81	74
	20	149	135	123	112	103	95
	18	205	186	169	154	141	130
	16	264	239	217	199	182	167

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	48	45	41	38	36	33
	20	63	59	54	51	47	44
	18	88	82	76	70	65	61
	16	116	108	100	93	86	81
Double	22	55	50	47	44	41	38
	20	69	64	60	55	52	48
	18	95	88	82	76	71	66
	16	123	114	105	98	91	85
Triple	22	69	64	59	55	51	48
	20	87	81	75	70	65	61
	18	120	111	103	96	89	84
	16	155	143	133	123	115	108

Span Cond.	Gage Number	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
Cantilever	22	116	94	77	64	55	47
	20	148	119	98	82	69	60
	18	202	163	135	113	95	82
	16	261	210	173	145	123	105

TABLE 3.4
3 DR LRFD Superimposed Uniform Upward Loads (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	97	89	81	74	68	63
	20	124	113	103	94	87	80
	18	170	154	141	129	119	110
	16	218	198	181	166	153	141
Double	22	87	79	72	66	61	56
	20	114	103	94	87	80	74
	18	157	143	131	120	110	102
	16	207	188	172	158	145	134
Triple	22	108	89	89	82	75	70
	20	142	113	117	108	99	91
	18	196	154	163	149	137	127
	16	258	198	214	196	180	167

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	58	54	51	47	44	42
	20	74	69	64	60	56	53
	18	102	94	88	82	77	72
	16	131	122	113	106	99	93
Double	22	52	48	45	42	40	37
	20	68	64	59	55	52	49
	18	94	88	82	77	72	67
	16	124	115	108	101	94	89
Triple	22	65	60	56	52	49	46
	20	85	79	73	69	64	60
	18	117	109	102	95	89	83
	16	154	143	134	125	117	110

Span Cond.	Gage Number	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
Cantilever	22	107	87	72	61	52	45
	20	140	114	94	80	68	59
	18	194	157	131	110	94	82
	16	255	207	172	145	124	108

TABLE 3.5
3 DR Uniform Service Load that Causes L/120 Deflection (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	85	74	64	56	49	43
	20	106	92	79	69	61	53
	18	147	126	110	96	84	74
	16	193	166	144	125	110	97
Double	22	208	179	156	136	120	106
	20	259	223	194	169	149	131
	18	357	308	268	234	206	182
	16	468	404	351	307	270	238
Triple	22	162	140	122	106	93	82
	20	202	174	151	132	116	102
	18	279	241	209	182	160	142
	16	366	316	274	239	210	186

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	38	34	30	27	24	22
	20	47	42	37	34	30	27
	18	65	58	52	46	42	38
	16	86	76	68	61	55	49
Double	22	94	84	75	67	60	55
	20	117	104	93	84	75	68
	18	161	144	128	115	104	94
	16	211	188	169	151	136	123
Triple	22	73	65	58	52	47	42
	20	91	81	72	65	58	53
	18	126	112	100	90	81	73
	16	165	147	131	118	106	96

TABLE 3.6
3 DR Uniform Service Load that Causes L/180 Deflection (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	56	49	42	37	32	28
	20	70	60	52	45	40	35
	18	97	83	72	63	55	48
	16	127	110	95	83	72	64
Double	22	138	119	103	90	79	70
	20	172	148	129	112	99	87
	18	237	205	178	155	136	120
	16	311	268	233	203	179	158
Triple	22	108	93	81	70	62	54
	20	134	115	100	87	77	68
	18	185	160	138	121	106	93
	16	243	209	182	158	139	123

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	25	22	20	17	16	14
	20	31	27	24	22	19	17
	18	43	38	34	30	27	24
	16	56	50	44	40	35	32
Double	22	62	55	49	44	40	36
	20	77	69	61	55	49	45
	18	107	95	85	76	68	62
	16	140	124	111	100	90	81
Triple	22	48	43	38	34	31	28
	20	60	53	48	43	38	35
	18	83	74	66	59	53	48
	16	109	97	86	77	70	63

TABLE 3.7
3 DR Uniform Service Load that Causes L/240 Deflection (psf)

Span Cond.	Gage Number	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"
Single	22	42	36	31	27	24	21
	20	52	45	39	34	29	26
	18	72	62	54	47	41	36
	16	95	81	70	61	53	47
Double	22	103	89	77	67	59	52
	20	128	111	96	84	73	65
	18	177	153	133	116	101	89
	16	232	200	174	152	133	117
Triple	22	80	69	60	52	46	40
	20	100	86	75	65	57	50
	18	138	119	103	90	79	69
	16	181	156	135	118	103	91

Span Cond.	Gage Number	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
Single	22	18	16	14	13	11	10
	20	23	20	18	16	14	13
	18	31	28	25	22	20	17
	16	41	37	32	29	26	23
Double	22	46	41	37	33	29	27
	20	57	51	45	41	37	33
	18	79	70	63	56	51	46
	16	104	93	83	74	67	60
Triple	22	36	32	28	25	23	20
	20	44	39	35	31	28	25
	18	61	55	49	44	39	35
	16	81	72	64	57	51	46

TABLE 3.8
3 DR Construction Spans
(ANSI/SDI RD-2017 Section 2.4.A.3 and 2.4.A.4)

Gage Number	Span Cond.	ASD Span	Span Cond.	ASD Span	Span Cond.	ASD Span
22	Single	13'-4"	Double or Triple	16'-3"	Cantilever	3'-7"
20		17'-0"		20'-10"		4'-5"
18		22'-4"		27'-5"		5'-10"
16		27'-6"		33'-9"		7'-2"

Gage Number	Span Cond.	LRFD Span	Span Cond.	LRFD Span	Span Cond.	LRFD Span
22	Single	14'-6"	Double or Triple	17'-8"	Cantilever	3'-9"
20		18'-7"		22'-9"		4'-8"
18		24'-8"		30'-2"		6'-1"
16		30'-8"		37'-8"		7'-6"

Tables 2 and 3 Notes:

1. Load tables are calculated using Section Properties as shown in Table 1 with F_y = 40 ksi and F_u = 50 ksi. Individual deck manufacturers may supply deck using lower or higher strength steel.
2. All section properties and ASD (Ω = 1.67) and LRFD (Φ = 0.90) uniform loads are calculated in accordance with ANSI/SDI RD-2017, Section 2.4.A.1.
3. Loads shown in tables are uniformly distributed total loads in psf. Span length assumes center-to-center spacing of supports. Tabulated loads shall not be increased by assuming clear span dimensions.
4. All construction load spans are calculated using a 200 pound service load on a 1 foot width of deck, in accordance with ANSI/SDI RD-2017, Section 2.4.A.3.
5. All cantilever construction load spans are calculated using a 200 pound service load on a 1 foot width of deck and a 10 psf uniform distributed load, in accordance with ANSI/SDI RD-2017, Section 2.4.A.4.
6. Deck deflection should be checked in accordance with ANSI/SDI RD-2017, Section 2.4.A.5.
7. Bending Moment formulae used for flexural stress limitations are:

   Simple and Two Span $$M = \frac{wL^2}{8}$$

   Three Span or More $$M = \frac{wL^2}{10}$$

8. Deflection formulae for deflection limitations are:

   Simple Span $$\Delta = \frac{0.013wL^4}{EI}$$

   Two Span $$\Delta = \frac{0.0054wL^4}{EI}$$

   Three Span $$\Delta = \frac{0.0069wL^4}{EI}$$

Table 4 – Shear Strength

		Shear (lbs/ft)	
Profile	**Gage Number**	**ASD $\Omega = 1.60$**	**LRFD $\Phi=0.95$**
WR	22	1631	2479
WR	20	1954	2970
WR	18	2546	3870
WR	16	3106	4721
DR	22	2458	3736
DR	20	3619	5501
DR	18	5136	7806
DR	16	6366	9676

Table 4 Notes:

1. Strength is calculated assuming F_y = 40 ksi and F_u = 50 ksi. Individual deck manufacturers may supply deck using lower or higher strength steel.
2. All section properties and ASD and LRFD shears are calculated in accordance with ANSI/SDI RD-2017, Section 2.4.A.1.

Table 5 – Web Crippling Strength

		ASD (lbs/ft)			
Profile	**Gage Number**	**Ω = 1.70 OFE**	**Ω = 1.75 OFI**	**Ω = 1.80 TFE**	**Ω = 1.75 TFI**
WR	22	656	1039	631	1281
WR	20	937	1512	967	1887
WR	18	1592	2631	1797	3330
WR	16	2401	4027	2878	5138
DR	22	443	893	389	1042
DR	20	640	1294	611	1539
DR	18	1103	2237	1170	2724
DR	16	1679	3405	1908	4206

		LRFD (lbs/ft)			
Profile	**Gage Number**	**Φ=0.90 OFE**	**Φ=0.85 OFI**	**Φ=0.85 TFE**	**Φ=0.85 TFI**
WR	22	1004	1546	966	1906
WR	20	1434	2250	1479	2807
WR	18	2436	3914	2750	4953
WR	16	3673	5990	4403	7643
DR	22	678	1328	596	1550
DR	20	979	1925	935	2290
DR	18	1688	3327	1790	4051
DR	16	2568	5065	2920	6256

Table 5 Notes:

1. Strength is calculated assuming F_y = 40 ksi and F_u = 50 ksi. Individual deck manufacturers may supply deck using lower or higher strength steel.
2. All section properties and ASD and LRFD reactions are calculated in accordance with ANSI/SDI RD-2017, Section 2.4.A.1.
3. Web crippling designations: OFE = One Flange End Loaded; OFI = One Flange Interior Loaded; TFE = Two Flange End Loaded; TFI = Two Flange Interior Loaded.
4. Web crippling loads consider a 1-1/2” minimum end bearing and a 2-1/2” (WR) and 4” (DR) minimum interior bearing.

Table 6 – Nominal Web Crippling Strength for Varied Bearing Lengths

Profile	Gage Number		OFE	OFI	TFE	TFI
WR	22	A	401	709	569	765
		B	1.46	0.99	0.82	1.22
	20	A	608	1093	913	1199
		B	1.32	0.90	0.74	1.11
	18	A	1128	2065	1813	2313
		B	1.14	0.78	0.64	0.96
	16	A	1812	3357	3045	3813
		B	1.02	0.70	0.57	0.86
DR	22	A	271	524	351	529
		B	1.46	0.99	0.82	1.22
	20	A	415	810	577	837
		B	1.32	0.90	0.74	1.11
	18	A	781	1532	1180	1632
		B	1.14	0.78	0.64	0.96
	16	A	1267	2493	2019	2708
		B	1.02	0.70	0.57	0.86
	Ω (ASD)		1.70	1.75	1.80	1.75
	Φ (LRFD)		0.90	0.85	0.85	0.85

Table 6 Notes:

1. Strength is calculated assuming F_y = 40 ksi. Individual deck manufacturers may supply deck using lower or higher strength steel.
2. All section properties are calculated in accordance with ANSI/SDI RD-2017, Section 2.4.A.1.
3. Web crippling designations: OFE = One Flange End Loaded; OFI = One Flange Interior Loaded; TFE = Two Flange End Loaded; TFI = Two Flange Interior Loaded.
4. Nominal web crippling strength is calculated using the following equation:

$$P_n = A\left(1 + B\sqrt{N}\right) \quad \text{(lbs)}$$

Where: A and B are constants from Table 6

N is the deck bearing length in inches, not less than 0.75" and not greater than 2.90" for WR deck or 7.25" for DR deck. Longer bearing lengths are permitted but do not provide greater strength.

5. ASD strength is calculated as P_n / Ω; LRFD strength is calculated as Φ P_n .

Table 7 – Arc Spot Weld Data

TABLE 7.1
Arc Spot Weld Uplift Capacity (lbs)

			ASD (lbs) Ω = 2.50			
Case	Gage Number	Design Thickness (inches)	Visible Weld Diameter (Inches)			
			1/2	5/8	3/4	1
1	22	0.0295	347	439	531	716
	20	0.0358	415	527	639	863
	18	0.0474	536	684	833	1129
	16	0.0598	658	845	1032	1406
2	22	0.0295	650	835	1019	1388
	20	0.0358	767	991	1214	1500
	18	0.0474	814	1257	1500	1500
	16	0.0598	549	1256	1500	1500
3	22	0.0295	243	307	372	501
	20	0.0358	291	369	447	604
	18	0.0474	375	479	583	790
	16	0.0598	461	591	722	984

			LRFD (lbs) Φ = 0.60			
Case	Gage Number	Design Thickness (inches)	Visible Weld Diameter (Inches)			
			1/2	5/8	3/4	1
1	22	0.0295	520	659	797	1074
	20	0.0358	623	791	959	1294
	18	0.0474	804	1027	1249	1693
	16	0.0598	987	1267	1548	2108
2	22	0.0295	976	1252	1529	2082
	20	0.0358	1150	1486	1822	2250
	18	0.0474	1221	1885	2250	2250
	16	0.0598	823	1884	2250	2250
3	22	0.0295	364	461	558	752
	20	0.0358	436	554	671	906
	18	0.0474	563	719	874	1185
	16	0.0598	691	887	1083	1476

TABLE 7.2
Arc Spot Weld Shear Capacity (lbs)

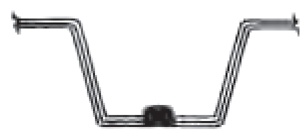
		ASD (lbs) Ω = 2.55 (2.20)			
		Visible Diameter (Inches)			
Case	**Gage Number**	**1/2**	**5/8**	**3/4**	**1**
1	22	718	909	961	1009
	20	860	1092	1324	1436
	18	(1215)	1417	1724	2337
	16	(1088)	(1898)	2137	2910
2 / 3	22	673	864	955	1003
	20	794	1026	1257	1427
	18	(694)	(1401)	1608	2221
	16	(468)	(1070)	(1919)	2725

		LRFD (lbs) Φ = 0.60 (0.70)			
		Visible Diameter (Inches)			
Case	**Gage Number**	**1/2**	**5/8**	**3/4**	**1**
1	22	1099	1391	1470	1544
	20	1316	1671	2025	2197
	18	(1871)	2168	2638	3576
	16	(1676)	(2923)	3269	4453
2 / 3	22	1030	1322	1461	1535
	20	1215	1569	1924	2184
	18	(1068)	(2157)	2460	3398
	16	(720)	(1648)	(2955)	4170

Table 7 Notes:

1. Assumes F_y = 40 ksi and F_u = 50 ksi, which provides resistances that are less than that for F_y = 33 ksi and F_u = 45 ksi steel.
2. Assumes F_{xx} = 60ksi.
3. Information in Tables 7.1 and 7.2 is developed in accordance with Chapter J of AISI S100-16.
4. Parenthesis indicate a different Ω or Φ as required by AISI S100 and as shown in the Table.
5. Arc spot welds with a 1/2 inch visible diameter, while sometimes specified, may not meet th requirement for a minimum effective diameter of 3/8 inch (AISI S310-16, Section J2.2). Refer to AISI S310-16, Equation J2.2.2.1-5.

CASE 1
Weld through single sheet thickness



CASE 2
Weld through double sheet thickness

CASE 3
Weld at edge of sheet sidelap

Table 8 – Screw Data

TABLE 8.1
Screw Dimensional, Tensile Strength, and Shear Strength

Screw Size	Major Diameter d (inches)	Head or Washer Diameter d_w (inches)	Average Tested Tensile Strength (lbs)	Tension ASD (Ω=3.00) (lbs)	Tension LRFD (Φ=0.50) (lbs)	Average Tested Shear Strength (lbs)	Shear ASD (Ω=3.00) (lbs)	Shear LRFD (Φ=0.50) (lbs)
#10	**0.190**	0.400" or 0.415"	2560	853	1280	1536	512	768
#12	**0.216**	0.400" or 0.430"	3620	1207	1810	2172	724	1086
#14	**0.240**	0.480" or 0.520"	4432	1477	2216	2659	886	1330
1/4"	**0.250**	0.480" or 0.520"	4810	1603	2405	2886	962	1443

TABLE 8.2
Screw Pull-Out Strength (lbs) (F_u = 50 ksi)

	Pull-Out - lbs (ASD) Ω = 3.00									
Base Sheet Steel Gage	**1/4"**	**3/16"**	**10**	**1/8"**	**12**	**14**	**16**	**18**	**20**	**22**
Steel Thickness (inches)	**0.2500**	**0.1875**	**0.1345**	**0.1250**	**0.1046**	**0.0747**	**0.0598**	**0.0474**	**0.0358**	**0.0295**
#10	673	505	362	336	282	201	161	128	96	79
#12	765	574	412	383	320	229	183	145	110	90
#14	850	638	457	425	356	254	203	161	122	100
1/4"	885	664	476	443	370	265	212	168	127	104
	Pull-Out - lbs (LRFD) Φ = 0.50									
#10	1009	757	543	505	422	302	241	191	145	119
#12	1148	861	617	574	480	343	274	218	164	135
#14	1275	956	686	638	533	381	305	242	183	150
1/4"	1328	996	715	664	556	397	318	252	190	157

TABLE 8.3
Screw Pull-Over Strength (lbs)

Top Sheet Steel Gage	16	18	20	22	24	26	28
Steel Thickness (inches)	**0.0598**	**0.0474**	**0.0358**	**0.0295**	**0.0238**	**0.0179**	**0.0149**
Steel F_u (ksi)	**50**	**50**	**50**	**50**	**62**	**62**	**62**
Screw Washer or Head d_w	**Pull-Over - lbs (ASD) Ω = 3.00**						
0.400”	598	474	358	295	295	222	185
0.415”	620	492	371	306	306	230	192
0.430”	643	510	385	317	317	239	199
0.480”	718	569	430	354	354	266	222
0.500”	748	593	448	369	369	277	231
Screw Washer or Head d_w	**Pull-Over - lbs (LRFD) Φ = 0.50**						
0.400”	897	711	537	443	443	333	277
0.415”	931	738	557	459	459	345	288
0.430”	964	764	577	476	476	358	298
0.480”	1076	853	644	531	531	400	333
0.500”	1121	889	671	553	553	416	346

TABLE 8.4
Screw Connection Shear Strength (lbs)

Steel Gage	10	12	14	16	18	20	22
Steel Thickness (inches)	**0.1345**	**0.1046**	**0.0747**	**0.0598**	**0.0474**	**0.0358**	**0.0295**
Steel F_u (ksi)	**50**	**50**	**50**	**50**	**50**	**50**	**50**
	Shear - lbs (ASD) Ω = 3.00						
#10	1150	894	639	511	405	306	252
#12	1307	1017	726	581	461	348	287
#14	1453	1130	807	646	512	387	319
1/4"	1513	1177	840	673	533	403	332
	Shear - lbs (LRFD) Φ = 0.50						
#10	1725	1341	958	767	608	459	378
#12	1961	1525	1089	872	691	522	430
#14	2179	1695	1210	969	768	580	478
1/4"	2270	1765	1261	1009	800	604	498

Table 8 Notes:

1. Individual screw manufacturers may have design data that differs from what is shown in Table 8. Refer to the individual manufacturers research and evaluation reports for design criteria specific to an individual screw.
2. Information in Tables 8.2, 8.3, and 8.4 is developed in accordance with Chapter J of AISI S100.
3. Table 8.2 and Table 8.4 are applicable to structural screws with support steel thickness greater than 2.5 times deck thickness.
4. Table 8.4 is based upon the calculated capacity of the screw in bearing, tilting, or combined bearing and tilting. Shaded cells in Table 8.4 indicate cases where the calculated shear capacity calculated by these sheet-based limits may exceed the shear capacity of the screw. Refer to the individual manufacturers research and evaluation reports for design criteria specific to an individual screw.

Table 9 – Fastener Patterns

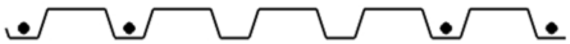
Attachment Patterns	C	Welds k	Screws k
36/3 	3	1.7	2
30/3 	2.5	1.7	2
24/3 	2	1.7	2
36/4 	3	2.7	3
30/4 	2.5	2.7	3
24/4 	2	2.7	3
36/5 	3	3.7	4
30/5 	2.5	3.7	4
24/5 	2	3.7	4
36/6 	3	4.7	5
30/6 	2.5	4.7	5
36/7 	3	5.7	6

Table 9 Notes:

1. The k value for welds takes into account the load eccentricity on the side lap. C is the cover width in feet. k is the effective number of connectors per deck cover width.
2. The uplift resistance U, in psf, can be calculated for a given fastener pattern using the following equation. P is obtained from Table 7 or 8. L is the deck span in feet.

$$U = \frac{kP}{C(\alpha L)}$$

Where: α = 1.00 for single span deck

α = 1.25 for 2-span deck

α = 1.10 for 3-span or more deck

3. The shear resistance S, in plf, can be calculated for a given fastener pattern using the following equation. This equation is applicable to a line of fasteners (such as at the perimeter of a diaphragm), not to the design of shear diaphragms. V is obtained from Table 7 or 8

$$S = \frac{kV}{C}$$

4. These values are calculated for nested side lap deck and may be conservative for interlocking side lap deck.

Table 10 – Axial Compression Capacity of Deck (kips)

TABLE 10.1
WR Deck (F_y = 40 ksi)

P_n, kips/ft / Deck Span, ft-in.							
Deck	**4'-0"**	**4'-6"**	**5'-0"**	**5'-6"**	**6'-0"**	**6'-6"**	**7'-0"**
B22	10.694	9.936	9.113	8.262	7.399	6.556	
B20	13.861	12.773	11.654	10.527	9.419	8.347	7.329
B18	19.744	18.191	16.599	15.001	13.424	11.894	10.432

P_n, kips/ft / Deck Span, ft-in.						
Deck	**7'-6"**	**8'-0"**	**8'-6"**	**9'-0"**	**9'-6"**	**10'-0"**
B22						
B20	6.487	5.787	5.198	4.695		
B18	9.219	8.152	7.221	6.441	5.781	5.218

$P_a = P_n/\Omega_c$, kips/ft / Deck Span, ft-in. Ω_c = 1.80							
Deck	**4'-0"**	**4'-6"**	**5'-0"**	**5'-6"**	**6'-0"**	**6'-6"**	**7'-0"**
B22	5.941	5.520	5.063	4.590	4.111	3.642	
B20	7.701	7.096	6.474	5.848	5.233	4.637	4.071
B18	10.969	10.106	9.222	8.334	7.458	6.608	5.796

$P_a = P_n/\Omega_c$, kips/ft / Deck Span, ft-in. Ω_c = 1.80						
Deck	**7'-6"**	**8'-0"**	**8'-6"**	**9'-0"**	**9'-6"**	**10'-0"**
B22						
B20	3.604	3.215	2.888	2.608		
B18	5.122	4.529	4.012	3.579	3.212	2.899

$\phi_c P_n$, kips/ft / Deck Span, ft-in. ϕ_c = 0.85							
Deck	**4'-0"**	**4'-6"**	**5'-0"**	**5'-6"**	**6'-0"**	**6'-6"**	**7'-0"**
B22	9.090	8.446	7.746	7.023	6.289	5.572	
B20	11.782	10.857	9.906	8.948	8.006	7.095	6.229
B18	16.783	15.462	14.109	12.751	11.410	10.110	8.867

$\phi_c P_n$, kips/ft / Deck Span, ft-in. ϕ_c = 0.85						
Deck	**7'-6"**	**8'-0"**	**8'-6"**	**9'-0"**	**9'-6"**	**10'-0"**
B22						
B20	5.514	4.919	4.418	3.991		
B18	7.836	6.929	6.138	5.475	4.914	4.435

TABLE 10.2
DR Deck (F_y = 40 ksi)

P_n, kips/ft / Deck Span, ft-in.							
Deck	**8'-0"**	**8'-6"**	**9'-0"**	**9'-6"**	**10'-0"**	**10'-6"**	**11'-0"**
N22	9.183	8.907	8.623	8.333	8.036	7.735	7.430
N20	12.782	12.392	11.977	11.544	11.104	10.660	10.208
N18	20.049	19.378	18.690	17.988	17.275	16.553	15.826

P_n, kips/ft / Deck Span, ft-in.						
Deck	**11'-6"**	**12'-0"**	**12'-0"**	**13'-0"**	**13'-6"**	**14'-0"**
N22	7.123	6.813	6.489	6.164	5.836	5.535
N20	9.751	9.294	8.840	8.390	7.933	7.517
N18	15.097	14.368	13.642	12.920	12.156	11.395

$P_a = P_n/\Omega_c$, kips/ft / Deck Span, ft-in. Ω_c = 1.80							
Deck	**8'-0"**	**8'-6"**	**9'-0"**	**9'-6"**	**10'-0"**	**10'-6"**	**11'-0"**
N22	5.102	4.948	4.791	4.629	4.464	4.297	4.128
N20	7.101	6.884	6.654	6.413	6.169	5.922	5.671
N18	11.138	10.766	10.383	9.993	9.597	9.196	8.792

$P_a = P_n/\Omega_c$, kips/ft / Deck Span, ft-in. Ω_c = 1.80						
Deck	**11'-6"**	**12'-0"**	**12'-0"**	**13'-0"**	**13'-6"**	**14'-0"**
N22	3.957	3.785	3.605	3.425	3.242	3.075
N20	5.417	5.163	4.911	4.661	4.407	4.176
N18	8.387	7.982	7.579	7.178	6.754	6.331

$\phi_c P_n$, kips/ft / Deck Span, ft-in. ϕ_c = 0.85							
Deck	**8'-0"**	**8'-6"**	**9'-0"**	**9'-6"**	**10'-0"**	**10'-6"**	**11'-0"**
N22	7.805	7.571	7.330	7.083	6.831	6.575	6.315
N20	10.865	10.533	10.180	9.813	9.439	9.061	8.677
N18	17.041	16.471	15.887	15.290	14.683	14.070	13.452

$\phi_c P_n$, kips/ft / Deck Span, ft-in. ϕ_c = 0.85						
Deck	**11'-6"**	**12'-0"**	**12'-0"**	**13'-0"**	**13'-6"**	**14'-0"**
N22	6.054	5.791	5.516	5.240	4.961	4.704
N20	8.288	7.900	7.514	7.131	6.743	6.390
N18	12.833	12.213	11.595	10.982	10.333	9.686

Table 11 – Reinforcing Zees

TABLE 11.1
Reinforcing Zee for 1.5 WR Deck

Zee Gage	20	18	16	14	12	10
ASD Allowable Moment (in-k)	0.937	1.356	1.860	2.534	3.542	4.115
Span (Ft)	ASD Allowable Uniform Load (plf)					
2	156	226	310	422	590	686
2.5	100	145	198	270	378	439
3	69	100	138	188	262	305
3.5	51	74	101	138	193	224
4	39	56	78	106	148	171
4.5	31	45	61	83	117	135
5	25	36	50	68	94	110
5.5	21	30	41	56	78	91
6	17	25	34	47	66	76
6.5	15	21	29	40	56	65
7	13	18	25	34	48	56
7.5	11	16	22	30	42	49
8	10	14	19	26	37	43
8.5	9	13	17	23	33	38
9	8	11	15	21	29	34
9.5	7	10	14	19	26	30
10	6	9	12	17	24	27

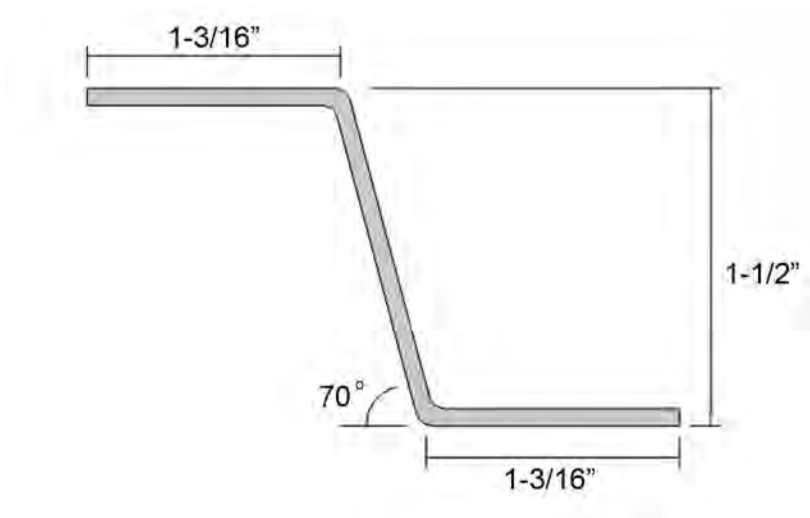


TABLE 11.2
Reinforcing Zee for 3 DR Deck

Zee Gage	20	18	16	14	12	10
ASD Allowable Moment (in-k)	**2.529**	**3.577**	**4.836**	**6.549**	**9.652**	**11.429**
Span (Ft)	**ASD Allowable Uniform Load (plf)**					
5	67	95	129	175	257	305
5.5	56	79	107	144	213	252
6	47	66	90	121	179	212
6.5	40	56	76	103	152	180
7	34	49	66	89	131	155
7.5	30	42	57	78	114	135
8	26	37	50	68	101	119
8.5	23	33	45	60	89	105
9	21	29	40	54	79	94
9.5	19	26	36	48	71	84
10	17	24	32	44	64	76
10.5	15	22	29	40	58	69
11	14	20	27	36	53	63
11.5	13	18	24	33	49	58
12	12	17	22	30	45	53
12.5	11	15	21	28	41	49
13	10	14	19	26	38	45
13.5	9	13	18	24	35	42
14	9	12	16	22	33	39
14.5	8	11	15	21	31	36
15	7	11	14	19	29	34

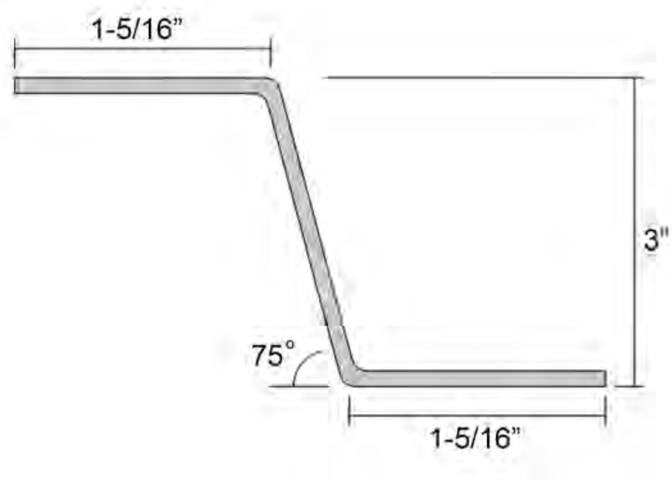
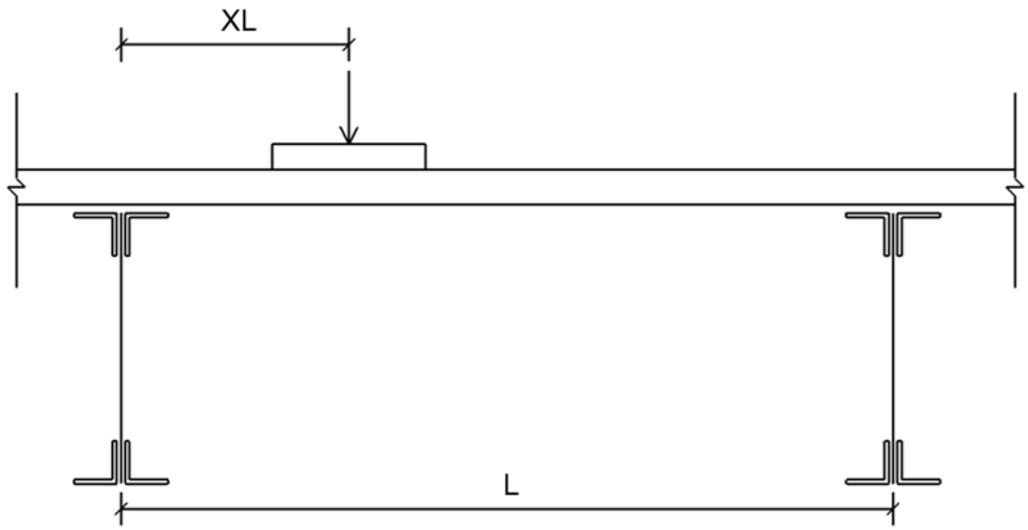


Table 11 Notes:

1. Yield strength for zee material is minimum of 33 ksi.
2. Strength is calculated in accordance with AISI S100.
3. Uniform loads shown are ASD allowable loads. To convert to LRFD loads, multiply ASD loads by 1.50.
4. Reinforcing zees should be installed in ribs adjacent to deck openings or other locations needing to be reinforced, and should be fastened to both top and bottom flanges of the deck using minimum of #10 screws at a maximum spacing of 12 inches on center.

Table 12 – Concentrated Loads on WR Deck



	Concentrated Load Width, Perpendicular to Ribs (B); b_e (Inches)					
X	**B ≤ 6"**	**B =12"**	**B =18"**	**B =24"**	**B =30"**	**B =36"**
0.00 [a]	6.0 [a]	12.0 [a]	18.0 [a]	24.0 [a]	30.0 [a]	36.0 [a]
0.00 < X ≤ 0.25	12.0	18.0	24.0	30.0	36.0	42.0
0.30	14.0	20.0	26.0	32.0	38.0	44.0
0.35	15.4	21.4	27.4	33.4	39.4	45.4
0.40	16.5	22.5	28.5	34.5	40.5	46.5
0.45	17.3	23.3	29.3	35.3	41.3	47.3
0.50	18.0	24.0	30.0	36.0	42.0	48.0

Note: [a] Loads over a support do not create a bending moment or shear in the deck. Therefore, the number of deck webs directly loaded should be checked for web crippling.

See Section 2.5 for the application of this table.

Table 13 - Maximum Allowable (ASD) Concentrated Moving Load

Moving concentrated load tables were developed from an iterative analysis of indeterminate spans. Limit states and criteria are listed below. Table 13.1 uses load distribution methodology described in Section 2.5. Table 13.2 assumes a 12 inch effective width; b_e = 12 inches fixed.

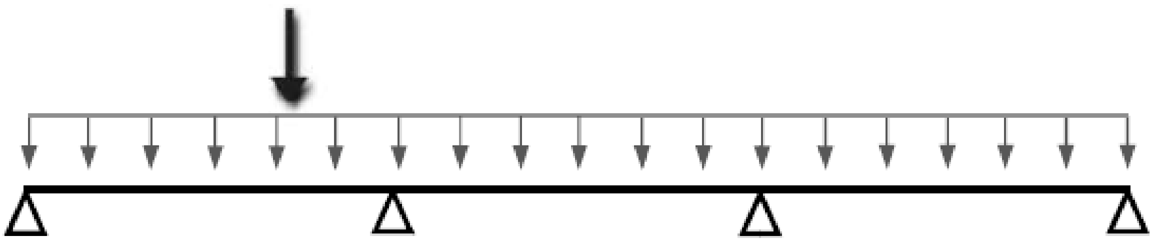


Table criteria

1.2 (deck + SDL) + 1.6 (LL) + 1.6 (P)

F_y = 40 ksi

B = 6 inches

N (OFI, TFI) = 4 inches

N (OFE, TFE) = 2 inches

Δ = L/240

2 or more equal spans

Gray shaded cells are loads that are limited by a deflection of L/240.

TABLE 13.1
Maximum Allowable (ASD) Concentrated Moving Load, lbs; b_e Varies

Deck Span, ft

10 psf SDL + 12 psf LL

	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
WR22	324	320	265	**196**	**129**	**81**					
WR20	498	428	340	**257**	**177**	**119**	**75**				
WR18	762	594	479	**377**	**270**	**194**	**135**	**90**			
WR16	980	768	624	**512**	**373**	**276**	**202**	**146**	**101**		
DR22				268	262	257	251	246	222	187	155
DR20				433	428	422	417	372	324	281	**238**
DR18				829	823	715	626	551	488	432	**370**
DR16				1282	1105	965	852	756	675	605	**520**

15 psf SDL + 20 psf LL

	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
WR22	314	308	235	**160**	**88**						
WR20	489	403	310	**222**	**135**	**72**					
WR18	743	570	449	**344**	**229**	**146**	**83**				
WR16	962	745	594	**479**	**335**	**229**	**151**				
DR22				246	237	229	220	204	152	112	71
DR20				411	403	394	378	317	263	213	**165**
DR18				807	784	670	576	496	428	367	**298**
DR16				1249	1067	921	801	701	615	540	**449**

30 psf SDL + 30 psf LL

	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
WR22	297	267	179	103							
WR20	472	358	256	172	**84**						
WR18	708	525	394	295	**177**	**88**					
WR16	928	700	540	419	**283**	**171**	**86**				
DR22				206	192	178	152	84			
DR20				372	357	343	284	206	136	70	
DR18				767	711	588	485	397	313	235	165
DR16				1187	995	840	711	601	506	420	327

TABLE 13.2
Maximum Allowable (ASD) Concentrated Moving Load, lbs; b_e = 12"

Deck Span, ft

10 psf SDL + 12 psf LL

	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
WR22	354	269	209	**141**	**93**						
WR20	446	324	270	**185**	**127**	**85**					
WR18	615	477	382	**273**	**194**	**138**	**97**	**65**			
WR16	793	619	500	**372**	**270**	**198**	**145**	**105**	**72**		
DR22				400	334	282	239	202	169	141	115
DR20				542	459	393	338	292	252	**216**	**171**
DR18				772	660	571	498	437	384	**327**	**266**
DR16				1034	889	774	680	602	536	**453**	**375**

15 psf SDL + 20 psf LL

	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
WR22	336	246	182	**114**	**63**						
WR20	428	320	243	**159**	**97**	**52**					
WR18	597	455	355	**247**	**164**	**105**	**59**				
WR16	775	596	473	**346**	**240**	**165**	**108**	**64**			
DR22				368	298	241	193	152	115	82	
DR20				511	423	352	293	242	197	158	**119**
DR18				741	624	530	453	384	330	**278**	**214**
DR16				1003	853	733	635	552	481	**404**	**323**

30 psf SDL + 30 psf LL

	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
WR20	396	279	194	**126**	**60**						
WR18	564	414	306	**214**	**127**	**63**					
WR16	742	555	424	**313**	**203**	**123**	**61**				
DR22				310	233	168	112	62			
DR20				453	358	278	211	152	99	51	
DR18				683	558	457	371	297	232	174	121
DR16				945	787	660	553	462	383	313	**234**

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EXAMPLES

SECTION 6

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LIST OF EXAMPLES

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Example 1A: Steel Deck with Uniform Spans (ASD)

Given:

WR22 deck has been selected. The deck will be installed on 6 foot spans, with all deck installed in a minimum three span condition. Using the design tables, verify the adequacy of this deck, given the following conditions. Deck F_y = 40 ksi.

(1) 3-span deck

(2) Loads, combined per ASCE 7-16.

- (a) Dead = 8 psf
- (b) Roof Live = 20 psf
- (c) Snow = 30 psf
- (d) Wind Uplift = 41.7 psf

(3) End bearing length = 1.5”, Interior bearing length = 2.5”

Solution:

Controlling Gravity Loading:	Dead + Snow Load (D + S)	=	38.0 psf
Controlling Uplift Loading:	0.6 Dead + 0.6 Wind Load (0.6D + 0.6 W)	=	-20.2 psf

Check flexure using Table 2

Gravity (Table 2.1)	98 psf allowable	OK
Uplift (Table 2.2)	96 psf allowable	OK

Check shear using Table 4

Maximum shear for a 3 span uniformly loaded beam = 0.600 wL

Design shear = 0.600 (38)(6) = 136.8 lbs

Allowable Shear = 1631 lbs OK

Note: Shear rarely controls for uniformly loaded deck and does not need to be checked when using Tables 2 or 3.

Check Web Crippling using Table 5

Maximum interior reaction for a 3 span uniformly loaded beam = 1.10 wL

Design interior reaction = 1.10 (38)(6) = 250.8 lbs

Allowable interior reaction – OFI - (2-1/2” bearing) = 1039 lbs OK

Maximum end reaction for a 3 span uniformly loaded beam = 0.400 wL

Design interior reaction = 0.400 (38)(6) = 91.2 lbs

Allowable interior reaction – OFE - (1-1/2” bearing) = 656 lbs OK

Note: Web Crippling rarely controls for uniformly loaded deck and does not need to be checked when using Tables 2 or 3.

Check Deflection

Per Table 2.7, the load that will cause a deflection of L/240 is 85 psf.

Note: Deck deflection is a serviceability limit and should be checked against ANSI/SDI RD-2017, Section 2.4.A.5 and the applicable building code. Designers should be aware that certain roofing or insulation materials may require more stringent deflection limitations.

Construction Loads

Per Table 2.8, the maximum construction span for a three span condition is 8’-01”.

Result:

WR22 deck is acceptable for this installation.

Example 1B: Steel Deck with Uniform Spans (LRFD)

Given:

WR22 deck has been selected. The deck will be installed on 6 foot spans, with all deck installed in a minimum three span condition. Use the design tables. Verify the adequacy of this deck, given the following conditions. Deck F_y = 40 ksi.

(1) 3-span deck

(2) Loads, combined per ASCE 7-16.

- (a) Dead = 8 psf
- (b) Roof Live = 20 psf
- (c) Snow = 30 psf
- (d) Wind Uplift = 41.7 psf

(3) End bearing length = 1.5", Interior bearing length = 2.5"

Solution:

Controlling Gravity Loading:	1.2 Dead + 1.6 Snow Load (1.2 D + 1.6 S)	=	57.6 psf
Controlling Uplift Loading:	0.9 Dead + 1.0 Wind Load (0.9 D + 1.0 W)	=	-34.5 psf

Check flexure using Table 2

Gravity (Table 2.3)	148 psf allowable	OK
Uplift (Table 2.4)	144 psf allowable	OK

Check Shear using Table 4

Maximum shear for a 3 span uniformly loaded beam = 0.600 wL

Design shear = 0.600 (57.6)(6) = 207.4 lbs

Allowable Shear = 2479 lbs OK

Note: Shear rarely controls for uniformly loaded deck and does not need to be checked when using Tables 2 or 3.

Check Web Crippling using Table 5

Maximum interior reaction for a 3 span uniformly loaded beam = 1.10 wL

Design interior reaction = 1.10 (57.6)(6) = 380.2 lbs

Allowable interior reaction – OFI - (2-1/2” bearing) = 1546 lbs OK

Maximum end reaction for a 3 span uniformly loaded beam = 0.400 wL

Design interior reaction = 0.400 (57.6)(6) = 138.2 lbs

Allowable interior reaction – OFE - (1-1/2” bearing) = 1004 lbs OK

Note: Web Crippling rarely controls for uniformly loaded deck and does not need to be checked when using Tables 2 or 3.

Check Deflection

Per Table 2.7, the load that will cause a deflection of L/240 is 85 psf.

Note: Deck deflection is a serviceability limit and should be checked against ANSI/SDI RD-2017, Section 2.4.A.5. and the applicable building code. Designers should be aware that certain roofing or insulation materials may require more stringent deflection limitations.

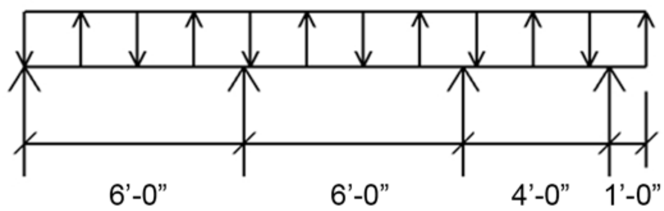
Construction Loads

Per Table 2.8, the maximum construction span for a three span condition is 8’-09”. Note that ASD and LRFD methods develop different maximum construction spans.

Result:

WR22 deck is acceptable for this installation.

Example 2A: Steel Deck with Non-Uniform Spans (ASD)



Given:

WR22 deck has been selected for this installation. Verify the adequacy of this deck, given the following conditions. Deck F_y = 40 ksi.

(1) 3-span deck with end cantilever

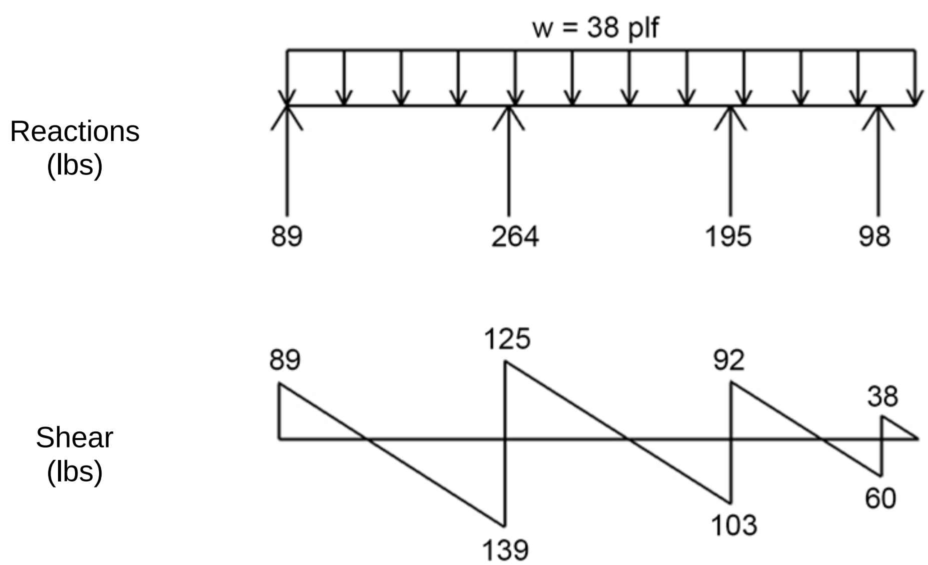
(2) Loads, combined per ASCE 7-16.

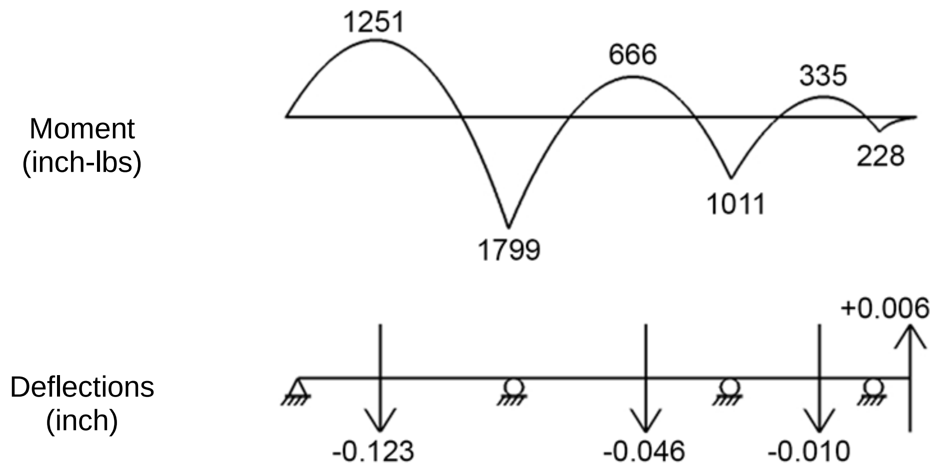
- (a) Dead = 8 psf
- (b) Roof Live = 20 psf
- (c) Snow = 30 psf
- (d) Wind Uplift = 41.7 psf

(3) End bearing length = 1.5 inch, Interior bearing length = 2.5 inch

Solution:

Controlling Gravity Loading Dead + Snow Load (D + S)





Note: If this were instead considered as three equal spans of 6 foot, (the moments used for Tables 2 and 3), the moments would be as follows:

Positive moment (end spans):	$0.08\ wL^2$	=	1313 inch-lbs
Negative moment (at interior support):	$0.10\ wL^2$	=	1642 inch-lbs

Check Flexure

Maximum positive moment = 1251 in-lbs/ft

M_P/Ω = 4096 inch-lbs (Table 1) OK

Maximum negative moment = 1799 in-lbs/ft

M_N/Ω = 4298 inch-lbs (Table 1) OK

Note: Per Table 2.1, the maximum allowable load is 98 psf for a 3-span condition with a maximum span of 6'-00".

Check Shear

Maximum shear = 139 lbs/ft

V/Ω = 1631 lbs (Table 4) OK

Note: Shear is rarely the controlling limit state.

Check Web Crippling

Maximum end reaction = 89 lbs/ft

One Flange End (OFE) R/Ω = 656 lbs (Table 5) OK

Maximum interior reaction = 264 lbs/ft

One Flange Interior (OFI) R/Ω = 1039 lbs (Table 5) OK

Note: Web crippling is rarely the controlling limit state.

Check Combined Bending and Web Crippling

Per AISI S100-16, Section H3(a), combined bending and web crippling is not an applicable limit state at interior supports of deck. At the left end support, the moment is zero, therefore the controlling limit state would be web crippling alone.

Check Combined Bending and Shear

At the critical section,

M = 1799 inch-lbs

V = 139 lbs

$$\sqrt{\left(\frac{M}{M_{allow}}\right)^2 + \left(\frac{V}{V_{allow}}\right)^2} = \sqrt{\left(\frac{1799}{4096}\right)^2 + \left(\frac{139}{1631}\right)^2} = 0.45 < 1.0 \qquad \text{OK}$$

Note: Combined bending and shear is rarely the controlling limit state.

Check Deflection

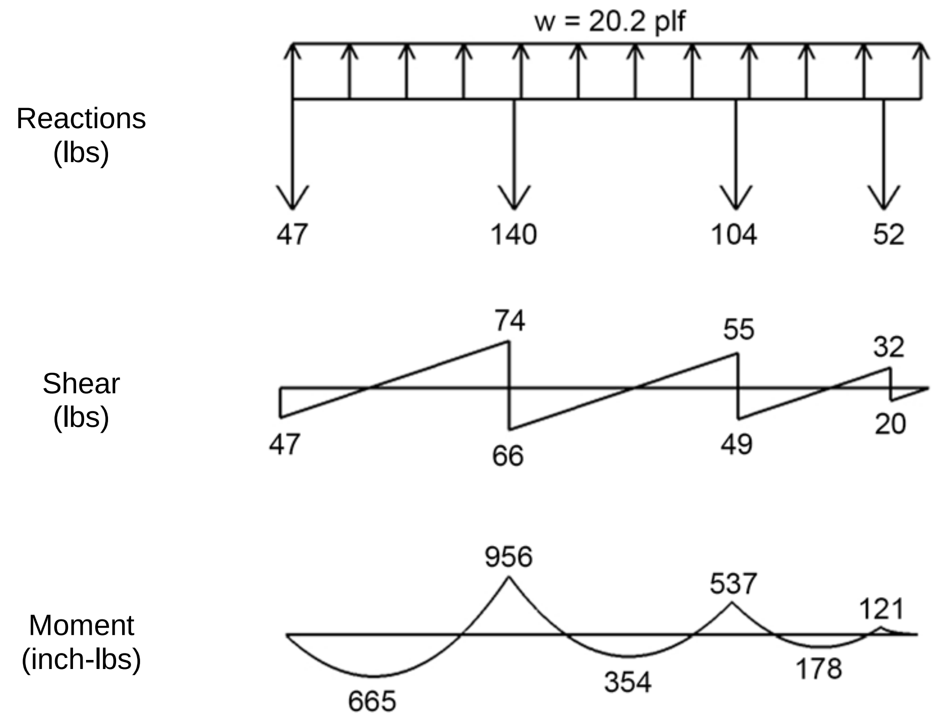
Deflection = 0.123 inch on 6 foot span with E = 29,500 ksi, $I = I_d = 0.151\ \text{inch}^4$.

$= L/585$ on total load

$= L/741$ on snow load

Note: Deck deflection is a serviceability limit and should be checked against ANSI/SDI RD-2017, Section 2.4.A.5 and the applicable building code. Designers should be aware that certain roofing or insulation materials may require more stringent deflection limitations.

Controlling Uplift Loading   0.6 Dead + 0.6 Wind Load   (0.6 D + 0.6 W)



Note: If this were instead considered as three equal spans of 6 foot, (the assumption used for Tables 2 and 3), the moments would be as follows:

Negative moment (end spans): $0.08\ wL^2 = 698$ inch-lbs

Positive moment (at support): $0.10\ wL^2 = 872$ inch-lbs

Check Flexure

Maximum positive moment = 956 inch-lbs/ft

$M_P/\Omega = 4096$ inch-lbs (Table 1) OK

Maximum negative moment = 665 inch-lbs/ft

$M_N/\Omega = 4298$ inch-lbs (Table 1) OK

Check Shear

Maximum shear = 74 lbs/ft

V/Ω = 1631 lbs (Table 4) OK

Note: Shear is rarely the controlling limit state.

Check Web Crippling

The web is not in compression during the uplift load combination; therefore, web crippling is not a limiting state.

Check Combined Bending and Shear

At the critical section,

M = 956 inch-lbs

V = 74 lbs

$$\sqrt{\left(\frac{M}{M_{allow}}\right)^2 + \left(\frac{V}{V_{allow}}\right)^2} = \sqrt{\left(\frac{956}{4096}\right)^2 + \left(\frac{74}{1631}\right)^2} = 0.23 < 1.0 \qquad \text{OK}$$

Note: Combined bending and shear is rarely the controlling limit state.

Check Deflection

Deflection was acceptable for gravity loading, and since gravity loads exceed the uplift loads, deflection is acceptable by observation.

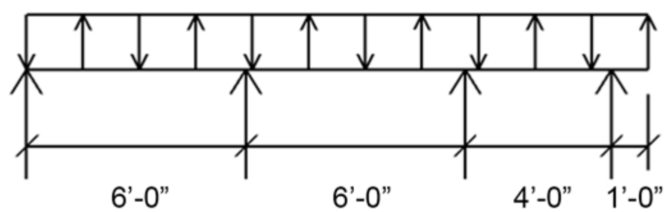
Construction Loads

Per Table 2.8, the maximum construction span for a three span condition is 8’-01” and the maximum cantilever span is 1’-8”, neither of which is exceeded.

Result:

WR22 deck is acceptable for this installation.

Example 2B: Steel Deck with Non-Uniform Spans (LRFD)



Given:

WR22 deck has been selected for this installation. Verify the adequacy of this deck, given the following conditions. Deck F_y = 40 ksi.

(1) 3-span deck with end cantilever

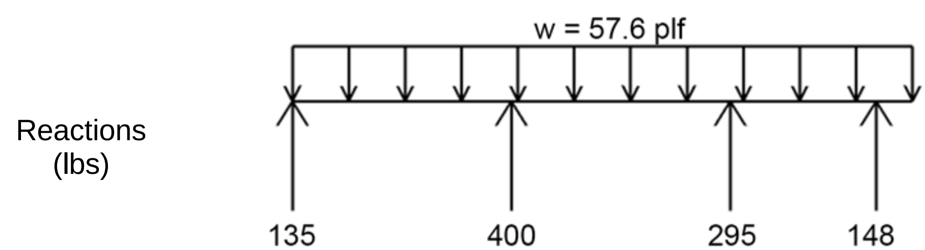
(2) Loads, combined per ASCE 7-16.

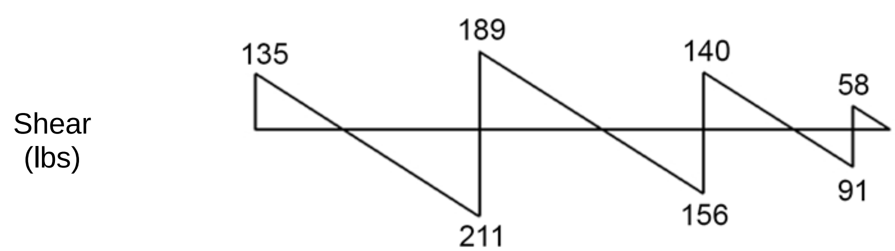
- (a) Dead = 8 psf
- (b) Roof Live = 20 psf
- (c) Snow = 30 psf
- (d) Wind Uplift = 42 psf

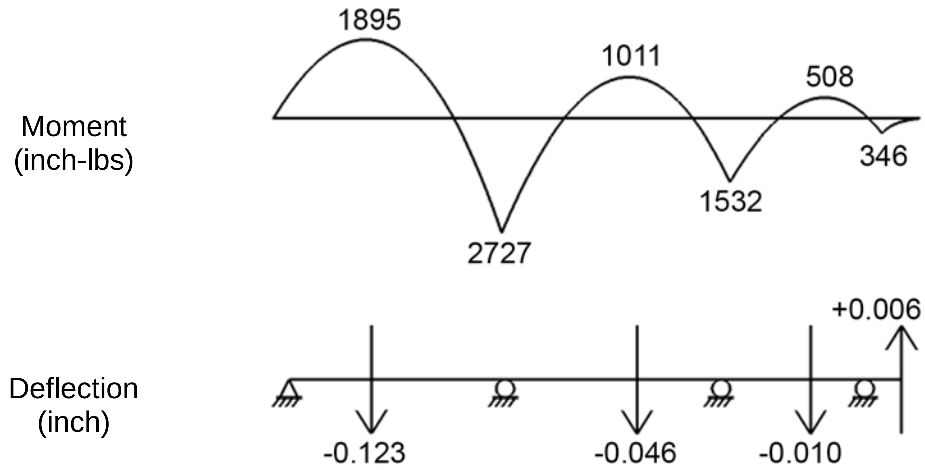
(3) End bearing length = 1.5 inch, Interior bearing length = 2.5 inch

Solution:

Controlling Gravity Loading 1.2 Dead + 1.6 Snow Load (1.2 D + 1.6 S)







Note: If this were instead considered as three equal spans of 6 foot, (the assumption used for Tables 2-5), the moments would be as follows:

Positive moment (end spans):	$0.08\ wL^2$	=	1991 inch-lbs
Negative moment (at support):	$0.10\ wL^2$	=	2488 inch-lbs

Check Flexure

Maximum positive moment = 1895 inch-lbs/ft

ϕM_P = 6156 inch-lbs (Table 1) OK

Maximum negative moment = 2728 inch-lbs/ft

ϕM_N = 6460 inch-lbs (Table 1) OK

Note: Per Table 2.4, the maximum factored load is 148 psf for a 3-span condition with a maximum span of 6'-00".

Check Shear

Maximum shear = 211 lbs/ft

ϕV = 2479 lbs (Table 4) OK

Note: Shear is rarely the controlling limit state.

Check Web Crippling

Maximum end reaction = 135 lbs/ft

One Flange End (OFE) $\phi R = 1004$ lbs (Table 5) OK

Maximum interior reaction = 400 lbs/ft

One Flange Interior (OFI) $\phi R = 1546$ lbs (Table 5) OK

Note: Web crippling is rarely the controlling limit state.

Check Combined Bending and Web Crippling

Per AISI S100-16, Section H3(a), combined bending and web crippling is not an applicable limit state at interior supports of deck. At the left end support, the moment is zero, therefore the controlling limit state would be web crippling alone.

Check Combined Bending and Shear

At the critical section,

M = 2728 inch-lbs

V = 211 lbs

$$\sqrt{\left(\frac{M}{M_{allow}}\right)^2 + \left(\frac{V}{V_{allow}}\right)^2} = \sqrt{\left(\frac{2728}{6460}\right)^2 + \left(\frac{211}{2479}\right)^2} = 0.43 < 1.0 \qquad \text{OK}$$

Note: Combined bending and shear is rarely the controlling limit state.

Check Deflection

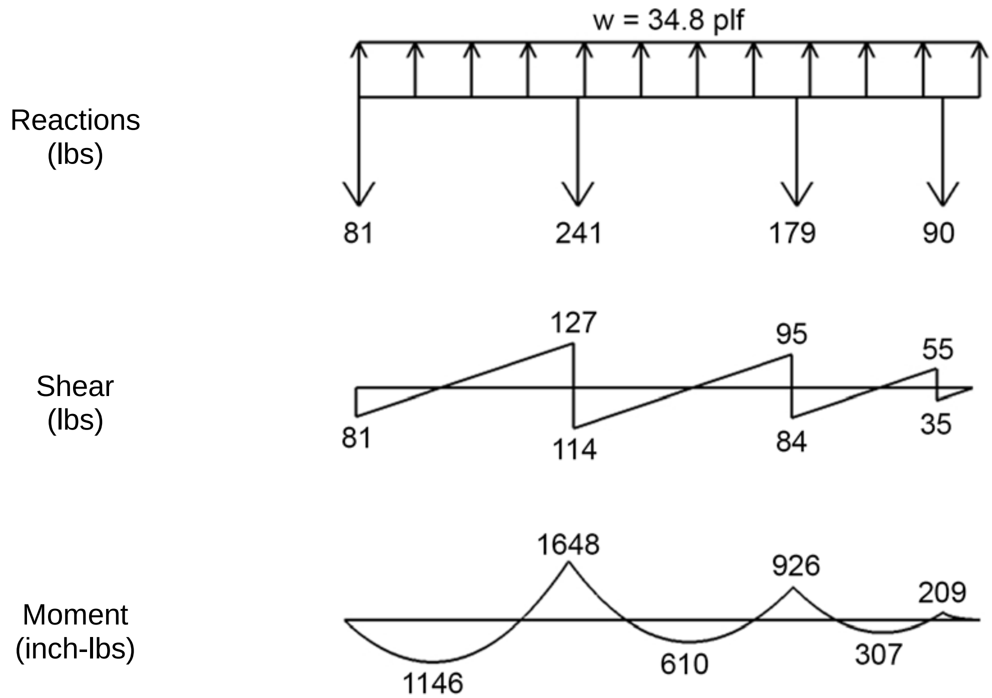
Deflection = 0.123 inches on 6 foot span

$= L/585$ on total load

$= L/741$ on snow load

Note: Deck deflection is a serviceability limit and should be checked against ANSI/SDI RD-2017, Section 2.4.A.5 and the applicable building code. Designers should be aware that certain roofing or insulation materials may require more stringent deflection limitations.

Controlling Uplift Loading 0.9 Dead + 1.0 Wind Load (0.9 D + 1.0 W)



Note: If this were instead considered as three equal spans of 6 foot, (the assumption used for Tables 2 and 3), the moments would be as follows:

Negative moment (end spans):	$0.08\ wL^2$	=	1203 inch-lbs
Positive moment (at support):	$0.10\ wL^2$	=	1503 inch-lbs

Check Flexure

Maximum positive moment = 1648 inch-lbs/ft

ϕM_P = 6156 inch-lbs (Table 1) OK

Maximum negative moment = 1146 inch-lbs/ft

ϕM_N = 6460 inch-lbs (Table 1) OK

Check Shear

Maximum shear = 127 lbs/ft

ϕV = 2479 lbs (Table 4) OK

Note: Shear is rarely the controlling limit state.

Check Web Crippling

The web is not in compression during the uplift load combination; therefore web crippling is not a limiting state.

Check Combined Bending and Shear

At the critical section,

M = 1648 inch-lbs

V = 127 lbs

$$\sqrt{\left(\frac{M}{M_{factored}}\right)^2 + \left(\frac{V}{V_{factored}}\right)^2} = \sqrt{\left(\frac{1648}{6460}\right)^2 + \left(\frac{127}{2479}\right)^2} = 0.26 < 1.0 \quad \text{OK}$$

Note: Combined bending and shear is rarely the controlling limit state.

Check Deflection

Deflection was acceptable for gravity loading, and since gravity loads exceed the uplift loads, deflection is acceptable by observation.

Construction Loads

Per Table 2.8, the maximum construction span for a three span condition is 8’-09” and the maximum cantilever span is 1’-09”, neither of which is exceeded.

<u>Result:</u>

WR22 deck is acceptable for this installation.

Example 3: Uplift Resistance of Welds and Screws

Given:

A 22 gage deck is fastened with a 36/5 pattern. The deck span is 6 foot in a 3-span condition. Calculate the resisting uplift for 5/8 inch (0.625) visible diameter welds and for #12 screws with a head diameter of 0.430 inches. Purlins are open web steel joists; therefore, pull-out is not a concern. Deck F_y = 40 ksi, F_u = 50 ksi.

Solution:

Use the equation: $U = \dfrac{Pk}{c(\alpha L)}$

For a 3-span configuration, α = 1.10

Welds (ASD)

P = 439 lbs (Table 7.1)

c = 3 ft, k = 3.7 (Table 9)

$$U = \frac{3.7 \cdot (439)}{3 \cdot (1.10 \cdot 6)} = 82 \text{ psf}$$

Screws (ASD)

P = 317 lbs (Table 8.3)

c = 3 ft, k = 4.0 (Table 9)

$$U = \frac{4.0 \cdot (317)}{3 \cdot (1.10 \cdot 6)} = 64 \text{ psf}$$

Welds (LRFD)

P = 659 lbs (Table 7.1)

c = 3 ft, k = 3.7 (Table 9)

$$U = \frac{3.7 \cdot (659)}{3 \cdot (1.10 \cdot 6)} = 123 \text{ psf}$$

Screws (LRFD)

P = 476 lbs (Table 8.3)

c = 3 ft, k = 4.0 (Table 9)

$$U = \frac{4.0 \cdot (476)}{3 \cdot (1.10 \cdot 6)} = 96 \text{ psf}$$

Note: This example uses continuous span beam reactions in lieu of tributary area method. Methodology is at the engineers discretion.

Example 4: Line Loads on Deck

Given:

A Standing Seam Roofing (SSR) is installed over roof deck on joists. Joists are spaced at 6 foot on center. On this project, drainage requires that the standing seam span the same direction as the roof deck. The SSR system requires that the clips be spaced at a 5'-0" o.c. maximum. The insulation chosen is 3 inch fiberglass that is not so rigid that snow load will be transmitted through uniform bearing to the deck. Consider the influence of the SSR construction on the structural ability of the system given the following criteria, using an ASD solution.

(1) Field of roof (slope 1/12, 30 foot high building)

(2) Bearing on joist = 3.5 inches

(3) Deck Properties

- (a) F_y = 40 ksi
- (b) F_u = 50 ksi

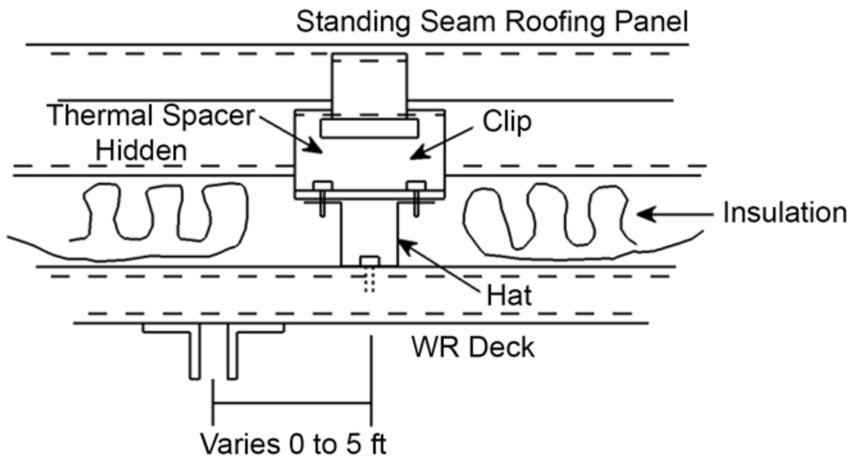
(4) Loads combined per ASCE 7-16

- (a) Dead = 5 psf
- (b) Snow = 30 psf
- (c) Wind including ASD load combination factor of 0.6
  - (i) Ext. Pressure = $-$26, +8 psf

    Note: 8 psf is positive pressure for this slope
  - (ii) Int. Pressure = $\pm$5 psf

Note: The Perimeter Zone will have different loading. WR22 deck on a 6 foot span has been selected using standard load tables (See Table 2):

Uplift Deflection	85 psf	(Δ = L/240)	>	($-$26$-$5) psf	OK
Down Vertical	98 psf		>	(30+5+8) psf	OK
Uplift Vertical	96 psf		>	($-$26+5$-$5) psf	OK



Note: Hat chosen because SSR requires 2 screws per clip. This eliminates prying on screw into deck.

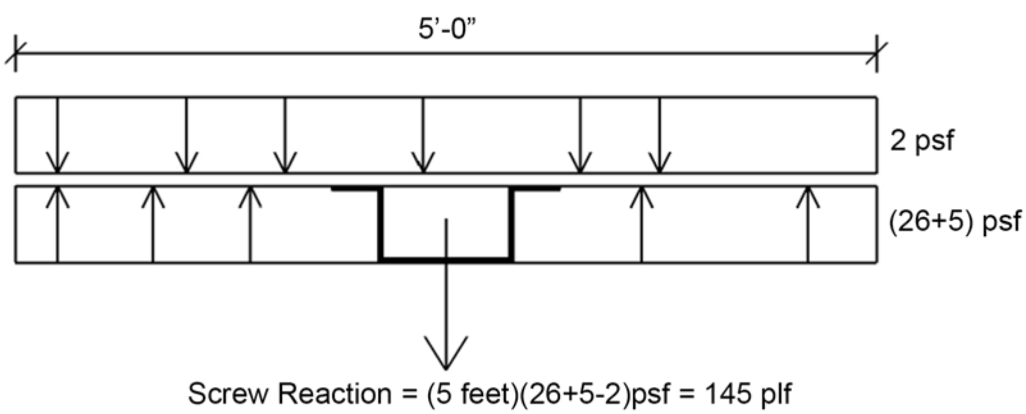
Solution:

Check Practicality of Screws into Deck

Assume that all of the internal pressure bleeds through deck and loads underside of the SSR.

Assume 0.6 DL of SSR = 2 psf

Case = 0.6 (DL + Wind)



Published screw pullout

Using #12 screw in 22 gage,

F_u = 50 ksi

P_{all} = 90 lbs/screw (Table 8.2)

Required transverse spacing = $\left(\frac{90}{145}\right)(12)$ = 7.4 inch < 18 inch

Use #12 at 6 inches o.c. into deck if hat is at 5'-0" o.c.

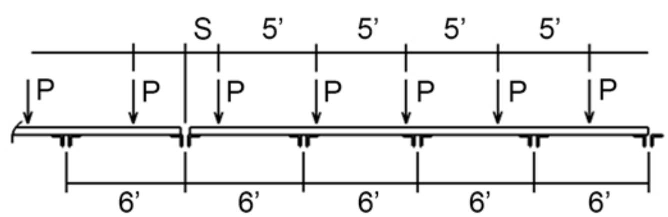
(Note: Deck ribs are 6" on center)

Note: 18 inch screw spacing limit is chosen to assure that all of deck width acts to resist load.

Therefore, the proposed system is possible, so continue with the design. (Some project specifications may not allow fastening to the deck underlayment. If so, space joists at 5'-0" o.c. and locate the hat directly over the joists. The deck then becomes a working platform, provides diaphragm action, and must be checked for two flange web crippling only. Hat must be checked separately and is beyond the scope of this Manual.)

Analyze Deck in Field of Roof

S varies from 0 inch to 5'-0"

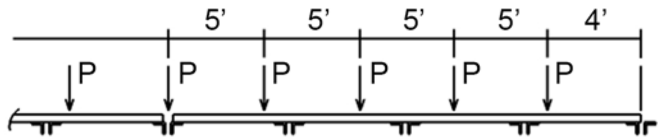


P (Dead + Snow) = (5)(35) = 175 plf (At 5 foot on center, this provides a maximum of 2 line loads per 6 foot deck span)

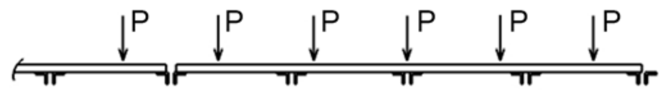
P (Dead + Wind) = -145 plf

Condition (Results per Foot)

R_{ext} = 185 lbs

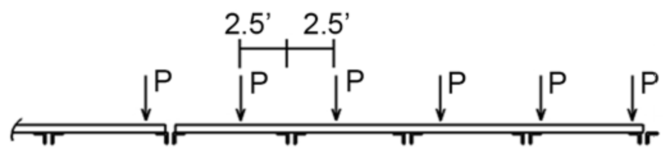


R_{int} = 250 lbs

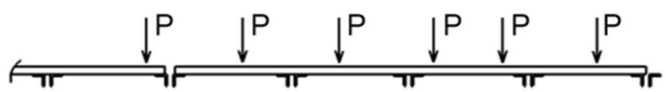


$-M$ = 2100 inch-lbs

V = 130 lbs

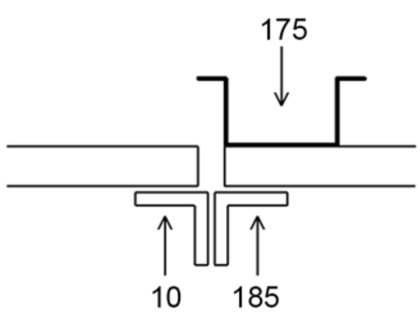


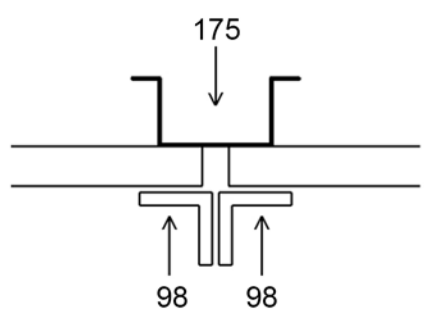
$+M$ = 2244 inch-lbs



$\Delta = L/450$

Δ = 0.16 inch   OK





Deck Data and Section Properties – WR 22 Deck

I_D (inch^4/ ft)	**M_P/Ω** (inch-lbs)	**M_N/Ω** (inch-lbs)	**V_n/Ω** (lbs/ft)	**1 Flange Load** (lbs/ft)		**2 Flange Load** (lbs/ft)	
				OFE (1.5 inch)	OFI (2.5 inch)	TFE (1.5 inch)	TFI (2.5 inch)
0.151	4096	4298	1631	656	1039	631	1281

Check Deck Web Crippling

Hat flange is approximately 2.5 inches wide.

Use lesser widths shown in Table 6 to conservatively check web crippling.

R_{ext} = 185 < 631 OK

R_{ext} = 98 < 631 OK

R_{int} = 250 < 1039 OK

Note: Since the deck web crippling is OK, a more exact analysis of bearing width is not required.

Check Bending

2244 inch-lbs $\leq$ 4096 inch-lbs OK

Check Maximum Shear

V = 185 lbs << 1631 lbs OK

Check Bending and Shear Interaction

$$\sqrt{\left(\frac{2100}{4096}\right)^2 + \left(\frac{130}{1631}\right)^2} = 0.52 < 1.0 \quad \text{OK}$$

For comparison purposes, a uniform load is considered in lieu of the more exact analysis as follows:

$+M = 0.08 \times 35 \times 6^2$ = 1210 inch-lbs vs. 2244 inch-lbs

$-M = 0.1 \times 35 \times 6^2$ = 1512 inch-lbs vs. 2100 inch-lbs

$V = 0.6 \times 35 \times 6$ = 126 lbs vs. 100 lbs

Check Wind Uplift

w = $-$145 plf < 175 plf

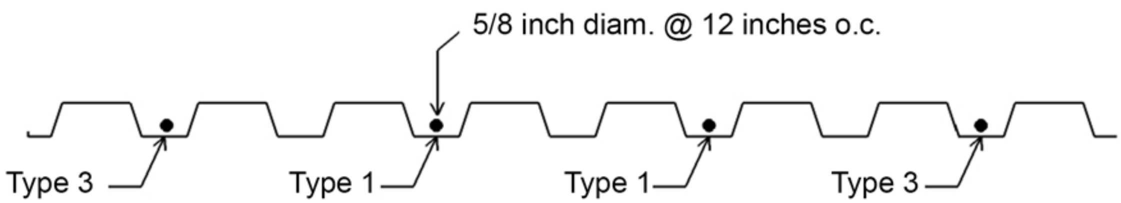
By inspection, uplift will not control flexure or shear

Check Welds for Deck to Support

R_{ext} = (145/175) 185 = 155 plf

R_{int} = (145/175) 250 = 208 plf

Minimum Weld Allowed per ANSI/SDI RD-2017



Allowable Uplift per Weld (Table 7.1)

Type 3 = 307 lbs/weld

Type 2 = 835 lbs/weld

Type 1 = 439 lbs/weld

Note: Type 3 welds are located at lap locations. (Type 3 = 70% of Type 1) AISI S100 J2.2.3

Results:

Allowable uplift per 3 foot wide sheet =

(1) Type 3 weld plus (2) Type 1 weld

307 + 2 (439) = 1185 lbs/sheet = 395 plf > 208 plf OK

Note: Type 3 welds are shared with adjacent sheets; 1/2 of Type 3 + 1/2 of Type 3

Consider Welds at end of sheet

For prying, multiply by 0.5

395×0.5 = 197 lbs

197 > 155 OK, weld pattern works

Note: 0.5 is a prying factor per AISI S100 Section J2.2.3. Check of Type 3 neglects Type 1 sharing.

Example 4 Notes:

(1) Perimeter and corner can be designed in similar fashion and probably will require welds at 6 inches o.c. or less spacing.

(2) Generally, under uniform roof live loading the capacity of the roof deck is so great that decking selected to resist construction loads will provide more than enough capacity to resist normal service loads. Snow drifting, and other non-uniform loading should be checked.

(3) Fastening often is the primary design consideration.

Example 5: Web Crippling Strength

Given:

For a WR22 deck, calculate the web crippling strength for the end bearing length of 3/4 inches and interior bearing length of 4 inches. Deck F_y = 40 ksi, F_u = 50 ksi.

Solution:

Refer to Table 6 in Section 5.

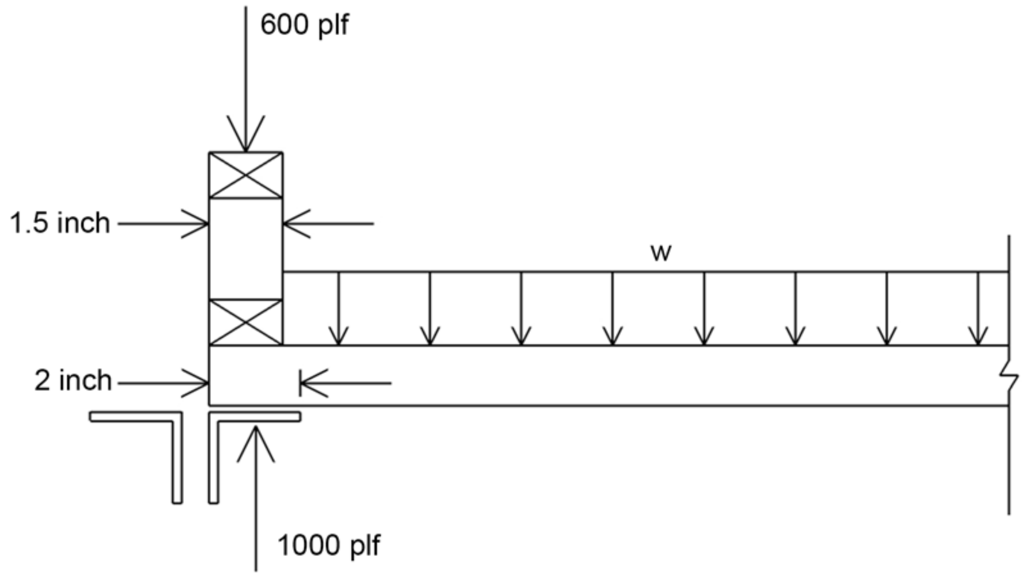
The end bearing of 0.75 inches meets the minimum bearing length required.

The interior bearing length of 4 inches exceeds the maximum of 2.90 inches, so calculate web crippling using the bearing length of 2.90 inches.

$P_N = A\,(1+B\sqrt{N})$ (From Table 6)

	OFE	OFI	TFE	TFI
A	401	709	569	765
B	1.46	0.99	0.82	1.22
N (inches)	0.75	2.90	0.75	2.90
Ω	1.70	1.75	1.80	1.75
ϕ	0.90	0.85	0.85	0.85
P_n (lbs)	908	1904	973	2354
P_n / Ω (lbs)	534	1088	540	1345
ϕP_n (lbs)	817	1619	827	2001

Example 6: Web Crippling with Different Bearing Lengths



Given:

Select a WR deck that will meet web crippling requirements, using an ASD solution. The loads shown are combined dead plus roof live. Deck F_y = 40 ksi, F_u = 50 ksi.

(1) Bearing length (N) for roof curb is 1.5 inches

(2) Bearing length (N) for deck on joist is 2 inches

Solution:

The AISI S100 Standard has criteria for two-flange end (TFE) web crippling, but those provisions deal with the simplified situation of two equal loads with two equal bearing lengths. Therefore judgment and rational engineering must be applied.

Alternate rational models other than what is presented in this example are possible, and exclusion of other solutions does not prevent the use of those other rational models.

Option 1: Perform two TFE checks and ensure that both conditions are satisfied.

(1) 600 lbs on a 1.5 inch bearing length; and

(2) 1000 lbs on a 2 inch bearing length

Try WR20

Per Table 5, P_n/Ω = 967 lbs on a 1.5" bearing length (TFE) OK

Per Table 6, P_n/Ω = 1038 lbs on a 2" bearing length (TFE) OK

(A = 913; B = 0.74; N = 2"; Ω = 1.80; P_n = 1868 lbs.)

Option 2: Consider the 1000 plf load resisted on a 1.5 inch bearing length. Both OFE and TFE should be investigated.

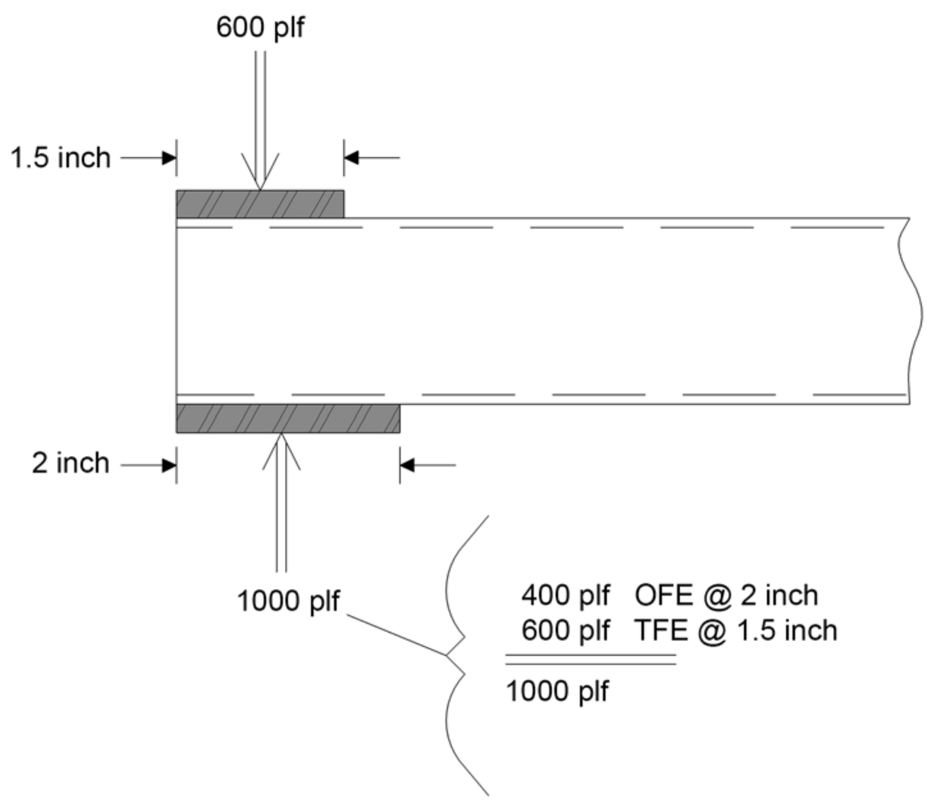
Try WR20

Per Table 5, P_n/Ω = 937 lbs on a 1.5" bearing length (OFE) No Good

Per Table 5, P_n/Ω = 967 lbs on a 1.5" bearing length (TFE) OK

Option 3: Explicitly consider the interaction of one flange and two flange loading.

Model



Try WR20

Check One Flange Exterior (OFE)

N = 2 inch

From Table 6,

$A = 608 \quad B = 1.32 \quad \Omega = 1.70$

$$P_n = A(1 + B\sqrt{N}) = 608(1 + 1.32\sqrt{2}) = 1743 \text{ lbs}$$

$$\frac{P_n}{\Omega} = \frac{1743}{1.70} = 1025 \text{ lbs}$$

Check Two Flange Exterior (TFE)

N = 1.5 inch

From Table 6,

$A = 913 \quad B = 0.74 \quad \Omega = 1.80$

$$P_n = A(1 + B\sqrt{N}) = 913(1 + 0.74\sqrt{1.5}) = 1740 \text{ lbs}$$

$$\frac{P_n}{\Omega} = \frac{1740}{1.80} = 967 \text{ lbs}$$

Assuming a linear interaction of OFE and TFE web crippling, with 60% being TFE, the interaction diagram gives an allowable capacity of 990 lbs, which is less than the 1000 lbs required. Therefore, WR20 is not adequate.

Try WR18

Check One Flange Exterior (OFE)

From Table 6,

$A = 1128 \quad B = 1.14 \quad \Omega = 1.70$

$$P_n = 1128(1 + 1.14\sqrt{2}) = 2946 \text{ lbs}$$

$$\frac{P_n}{\Omega} = \frac{2446}{1.70} = 1733 \text{ lbs}$$

Check Two Flange Exterior (TFE)

From Table 6,

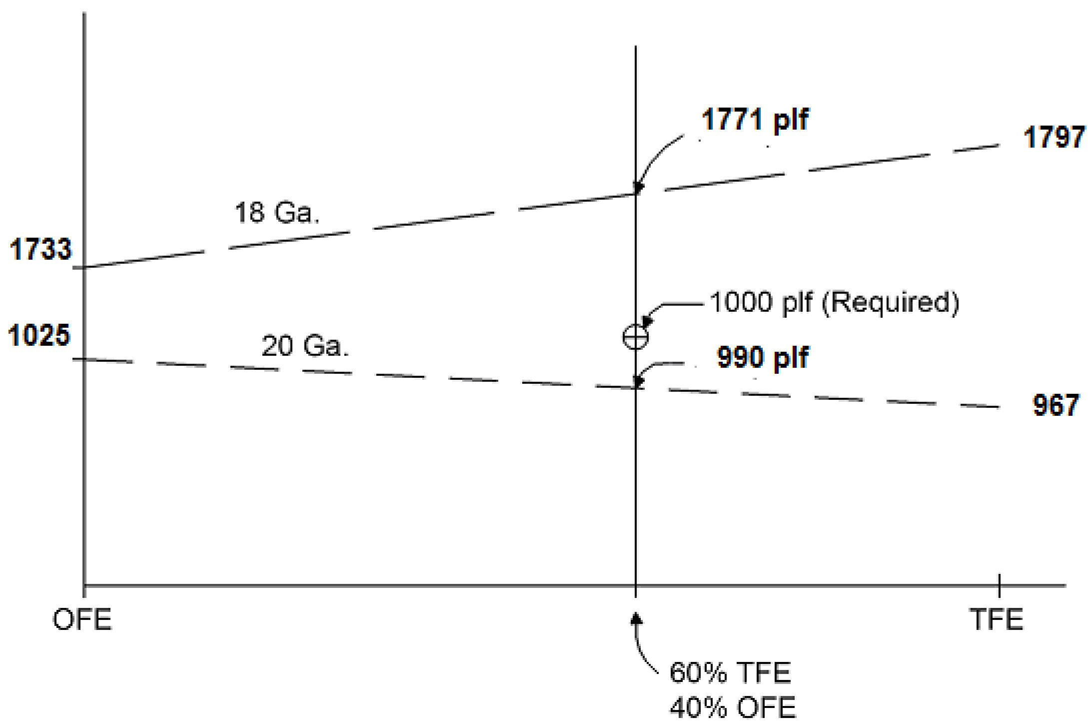
$A = 1813 \quad B = 0.64 \quad \Omega = 1.80$

$P_n = 1813\,(1 + 0.64\sqrt{1.5}) = 3234 \text{ lbs}$

$$\frac{P_n}{\Omega} = \frac{3234}{1.80} = 1797 \text{ lbs}$$

Assuming a linear interaction of OFE and TFE web crippling, the interaction diagram gives an allowable capacity of 1771 lbs, which is greater than the 1000 lbs required. WR18 deck meets web crippling limit state.

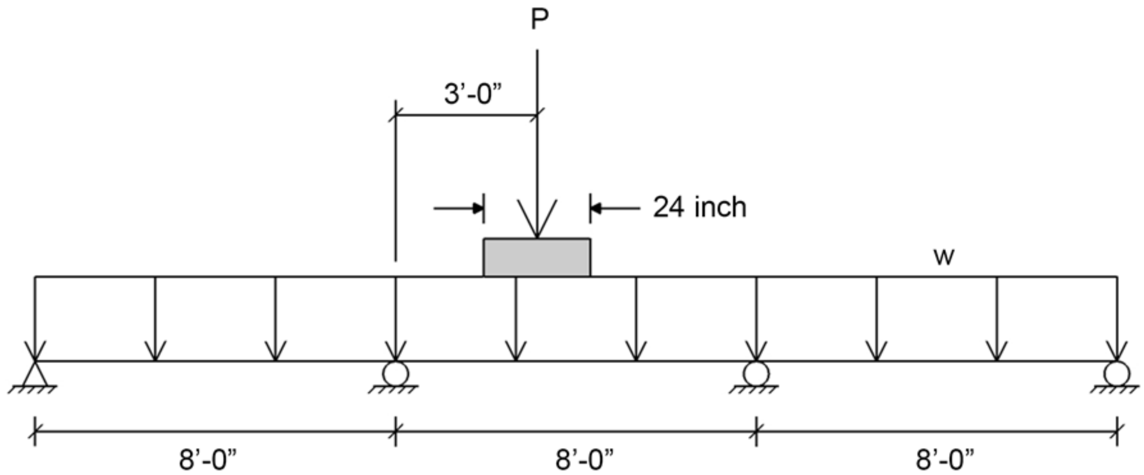
Flexure and shear must also be satisfied.



20 Ga. $\quad \dfrac{1000}{990} = 1.010 \quad$ No Good

18 Ga. $\quad \dfrac{1000}{1771} = 0.564 \quad$ OK

Example 7: Concentrated Load with Transverse Distribution



Given:

Select a WR deck to support the roof load condition below. Use an ASD solution. Combine loads using ASCE 7-16. Deck F_y = 40 ksi, F_u = 50 ksi.

(1) Uniform Dead Load = 10 psf

(2) Uniform Live Load = 20 psf

(3) Concentrated Dead Load = 700 lbs on baseplate

- (a) Baseplate size is 24 inches parallel to deck span and 30 inches perpendicular to deck span
- (b) Deck End Bearing Length = 1.5 inch
- (c) Deck Interior Bearing Length = 3 inch

Solution:

Calculate the transverse distribution of the concentrated load using the procedure found in Section 2.5.

L = 8 ft XL = 3 ft X = 0.375

$$b_e = B + 18 - \frac{3}{X} \geq 24 - \frac{3}{X}$$

$$= 30 + 18 - \frac{3}{0.375} \geq 24 - \frac{3}{0.375}$$

$$= 40 \text{ inch} \geq 16 \text{ inch}$$

Also see Table 12, where b_e = 40 inches by interpolation.

Therefore the 40 inch dimension controls the transverse distribution.

Concentrated Load is converted to a line load as 700 lbs $\times$ 12 / 40 = 210 plf.

From a structural analysis using w = 30 plf and P = 210 lbs, the maximum moments and shears are found in the middle span:

M_n = 3918 inch-lbs at the left support

M_p = 3632 inch-lbs under the concentrated load

V = 255 lbs at the left support

$R_{INTERIOR}$ = 416 lbs at the left of the second support from the left (OFI)

$R_{EXTERIOR}$ = 83 lbs at the right support of the 3rd span (OFE)

<u>Try WR22</u>

For this condition,

$\frac{M_n}{\Omega}$ = 4298 inch-lbs (Table 1) > 3918 inch-lbs OK

$\frac{M_p}{\Omega}$ = 4096 inch-lbs (Table 1) > 3632 inch-lbs OK

V_{ALLOW} = 1631 lbs (Table 4) > 255 lbs OK

Allowable Web Crippling, (Table 5)

OFE = 656 lbs (1.5 inch min.) > 83 lbs OK

OFI = 1039 lbs (2.5 inch min.) > 416 lbs OK

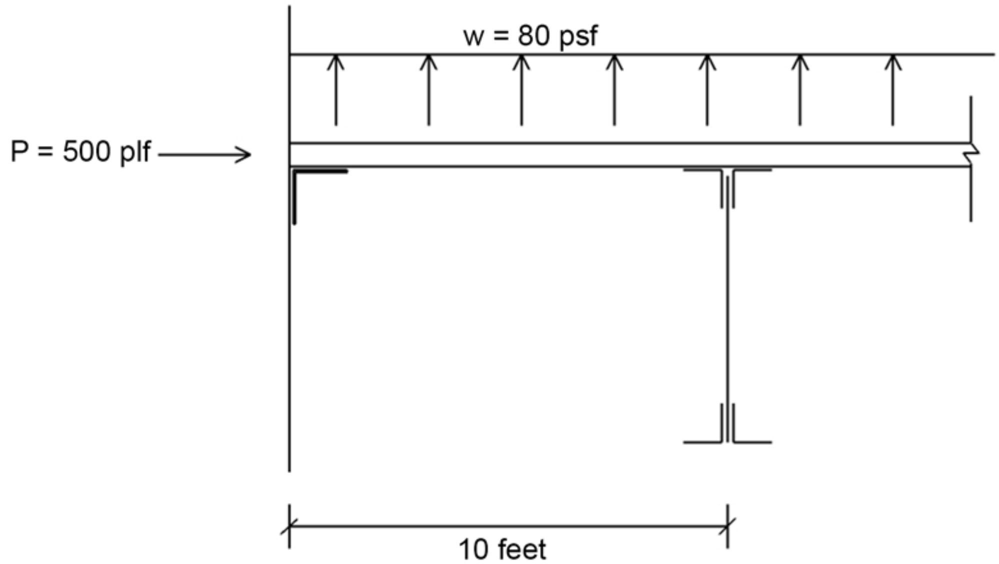
Therefore,

$$\sqrt{\left(\frac{V}{V_a}\right)^2 + \left(\frac{M}{M_a}\right)^2} = \sqrt{\left(\frac{255}{1631}\right)^2 + \left(\frac{3918}{4298}\right)^2} = 0.925 \leq 1.0 \text{ OK}$$

<u>Result:</u>

WR22 deck is acceptable for this condition.

Example 8: Steel Deck Bracing Wall - Combined Compression and Bending



Given:

The deck selected is DR22. Check the deck for combined axial compression and bending using an LRFD solution. Deck F_y = 40 ksi, F_u = 50 ksi.

Solution:

Controlling Loading

(1) Loads are combined using ASCE 7-16.

(2) The controlling load combination is 0.6 Dead + 0.6 Wind (0.6 D + 0.6 W). The loads shown in the figure are factored by this load combination.

(3) Assuming D = 0 is conservative.

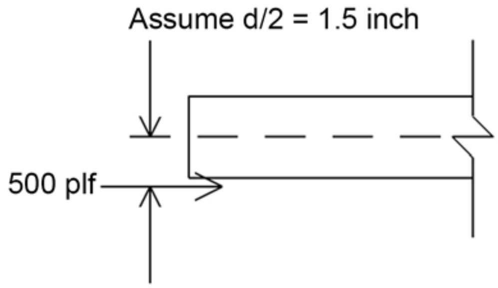
Flexure

LRFD flexural capacity (W_{LRFD}) for 10 foot, 3-span condition is 108 psf. (Table 3.4)

Axial

LRFD axial compression capacity (P_{LRFD}) for 10 foot span is 6831 lbs per one foot of width. (Table 10.2)

Combined



Assume the eccentricity of the load is d/2.

Moment due to eccentric loading is

500 (1.5) = 750 inch-lbs/ft

From Table 1, use the lesser of ϕM_P and ϕM_n ,

$\phi M_P = 12{,}708$ inch-lbs

Use AISI Equation H1.2-1 (modified),

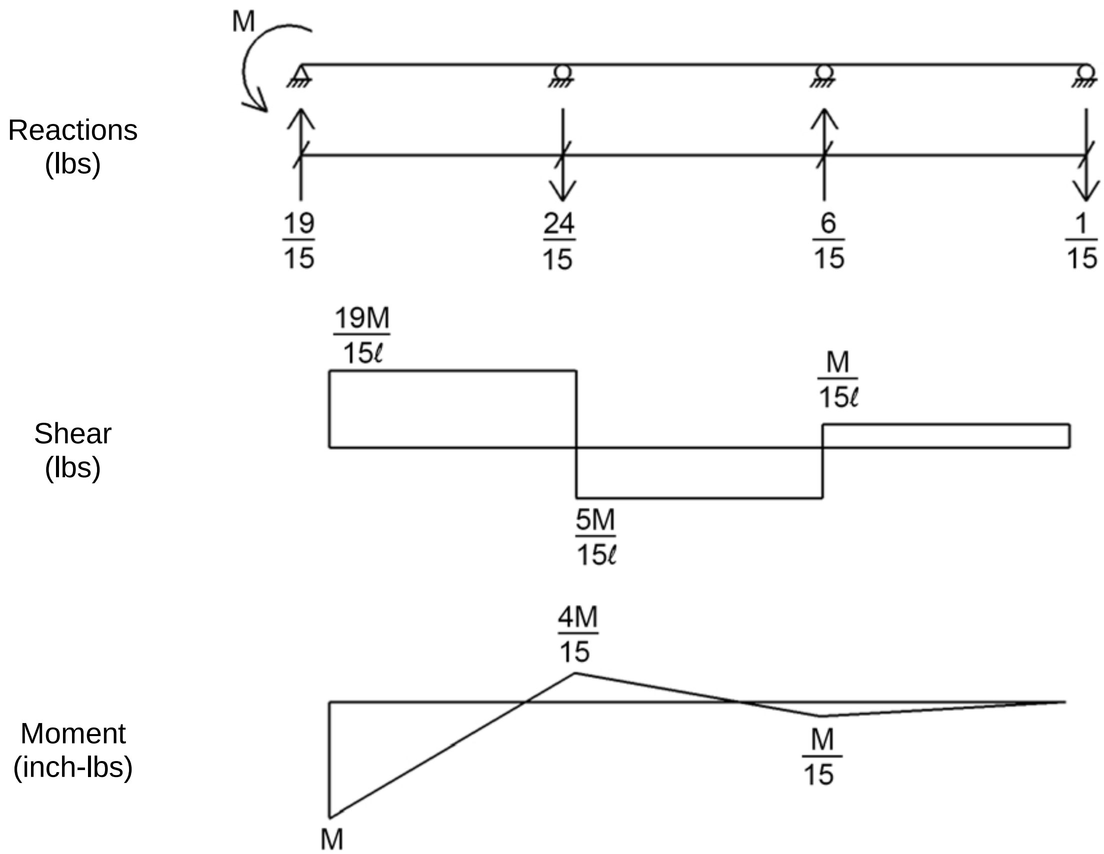
$$\frac{M}{\phi M_n} + \frac{W}{W_{LRFD}} + \frac{P}{P_{LRFD}} \leq 1.0$$

$$\frac{750}{12708} + \frac{80}{108} + \frac{500}{6831} = 0.87 < 1.0 \qquad \text{OK}$$

Note: The greatest contribution to the loading is the wind uplift and the calculation is not very sensitive to the assumed eccentricity of the loading.

<u>A more rigorous solution can be found by applying all the loads to a three span deck system</u>

<u>For a 3-span member</u>

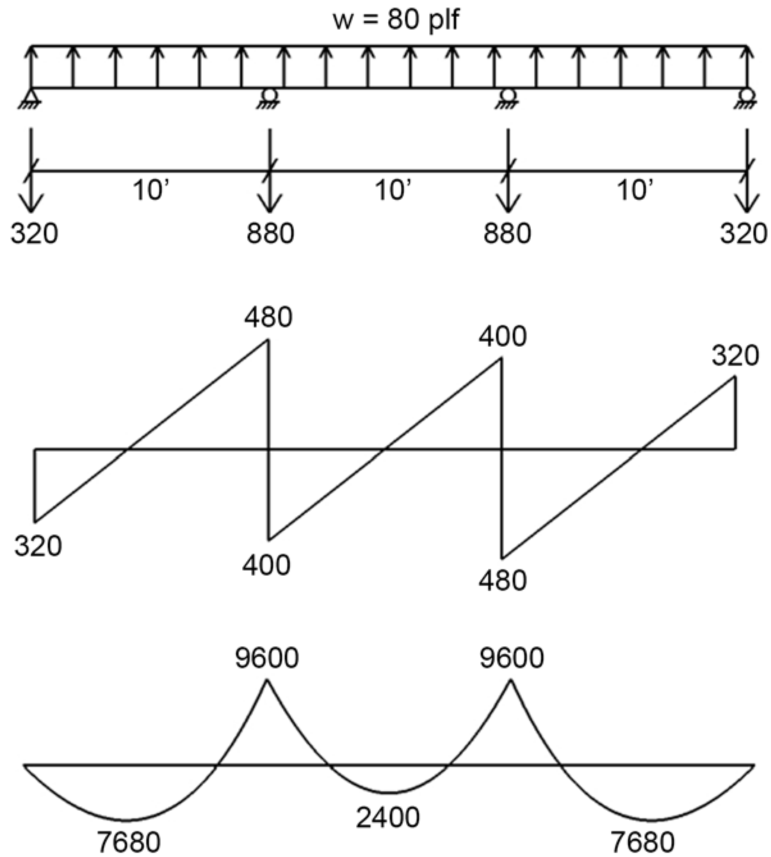


Uplift Loading

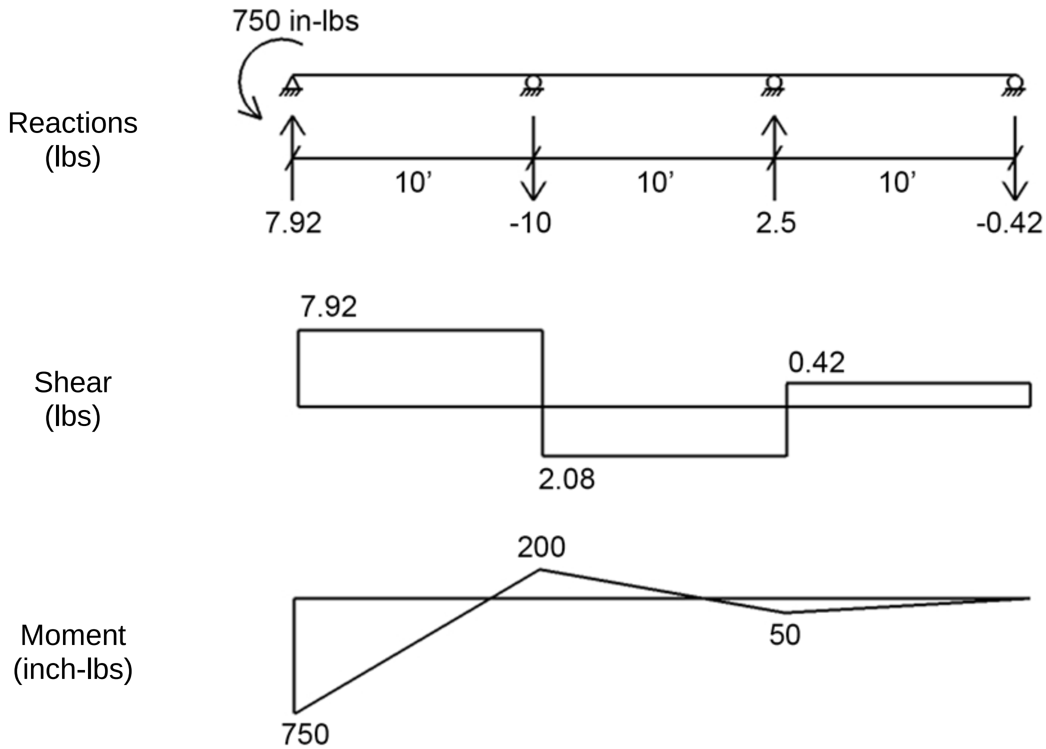
Reactions
(lbs)

Shear
(lbs)

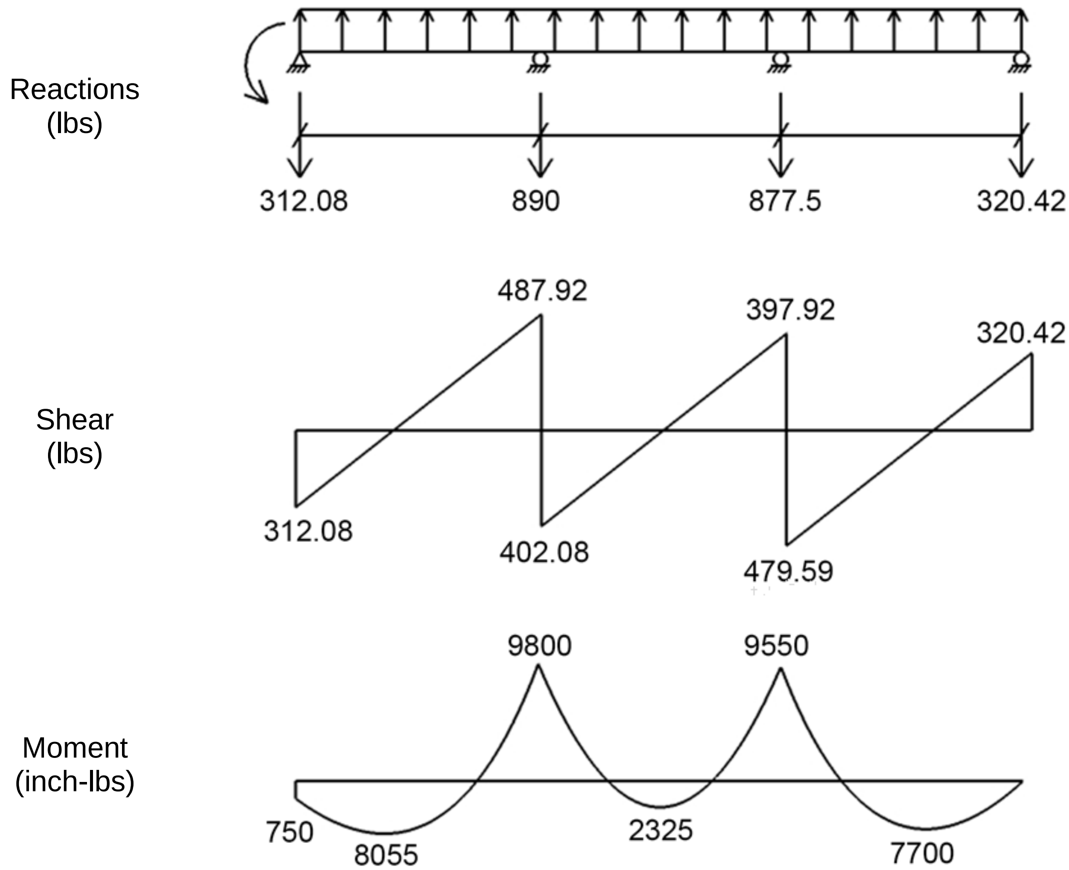
Moment
(inch-lbs)



Moment Loading



Combined Uplift and Moment Loadings



use AISI Equation H1.2-1 (Modified)

$$\frac{P}{P_{LRFD}} + \frac{M}{M_{LRFD}}$$

At M = 8055 inch-lbs/ft of width (compression on the bottom of the deck using M_{neg})

$$\phi_b M_{neg} = \phi_b F_y S_{neg} = 0.9(40)(0.398) = 14{,}328 \text{ inch-lbs/ft of width}$$

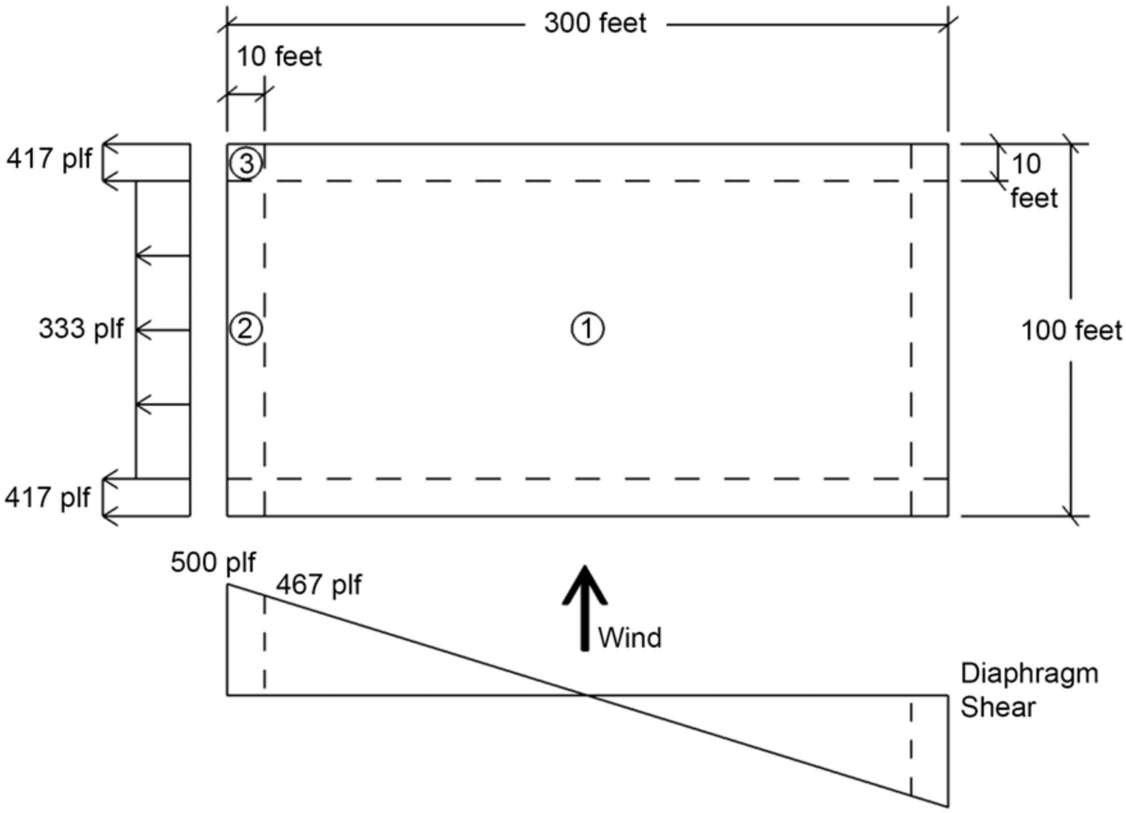
$$\frac{500}{6831} + \frac{8058}{14328} = 0.63 < 1.0 \qquad \text{OK}$$

At M = 9800 inch-lbs/ft of width (compression on the top of the deck using M_{pos})

$$\phi_b M_{pos} = \phi_b F_y S_{pos} = 0.9(40)(0.353) = 12{,}708 \text{ inch-lbs/ft of width}$$

$$\frac{500}{6831} + \frac{9800}{12708} = 0.84 < 1.0 \qquad \text{OK}$$

Example 9A: Fasteners Under Combined Uplift and Shear – Joist Supports



Given:

The WR18 steel deck is supported on open web steel joists at 5'-0" o.c. The deck is installed in a 3-span condition. The deck is fastened using #12 screws with 0.430 inch diameter head in a 36/7 pattern. Check the adequacy of the fasteners at the wall for combined wind uplift, in-plane shear, and sidewall anchorage given the following wind uplift pressures. Use an ASD solution. Combine loads using ASCE 7-16. Deck F_y = 40 ksi, F_u = 50 ksi.

Zone 1 = −40.5 psf
Zone 2 = −67.9 psf
Zone 3 = −102.2 psf

Solution:

Calculate Loads

Using an ASD load combination of 0.6 Dead + 0.6 Wind (0.6 D + 0.6 W), and conservatively assuming the dead load to be equal to 0.

Zone 1 = 24.3 psf

Zone 2 = 40.7 psf

Zone 3 = 61.3 psf

Uplift

c = 3.0, k = 6, α = 1.10 (Table 9)

Allowable pull-over strength = 510 lbs/screw (Table 8.3)

$$U = \frac{kP}{c(\alpha L)} = \frac{6.0(510)}{3.0(1.10)(5)} = 183.6 \text{ psf} > 61.3 \text{ psf} \qquad \text{OK}$$

Shear

Allowable shear strength is 461 lbs/screw (Table 9.4)

$$S = \frac{kV}{c} = \frac{6.0(461)}{3.0} = 922 \text{ plf} > 391 \text{ plf} \qquad \text{OK}$$

Combined endwall suction, diaphragm shear and pull-over

The effect of these combined forces on the fasteners must be considered by the Designer. This is typically performed using an analysis that reduces the available capacity of the diaphragm that is outside the scope of this Manual. The reader is referred to the SDI Diaphragm Design Manual for additional information.

Notes:

1. Zones 1 and 2 can be checked using a similar approach.

Example 9B:

Fasteners Under Combined Uplift and Shear – Cold-Formed Support

Given:

The same roof as in Example 9A is framed using trusses with cold –formed steel top chords at 4 foot on center. What is the minimum top chord thickness that will be adequate? Refer to 9A for loadings.

Solution:

Try 16 gage top chord (F_u = 50 ksi)

Uplift

c = 3.0, k = 6, α = 1.10 (Table 9)

Allowable pull-out strength = 183 lbs/screw (Table 8.2)

Allowable pull-over strength = 510 lbs/screw (Table 8.3)

$$U = \frac{kP}{c(\alpha L)} = \frac{6.0(165)}{3.0(1.10)(4)} = 82.4 \text{ psf} > 61.3 \text{ psf} \quad \text{OK}$$

Shear for screw bearing on truss chord

Allowable shear strength is 581 lbs/screw (Table 8.4)

$$S = \frac{kV}{c} = \frac{6.0(581)}{3.0} = 1162 \text{ plf} > 391 \text{ plf} \quad \text{OK}$$

Shear for screw bearing on deck (18 gage, F_u = 50 ksi)

Allowable shear strength is 461 lbs/screw (Table 8.4)

$$S = \frac{kV}{c} = \frac{6.0(461)}{3.0} = 922 \text{ plf} > 391 \text{ plf} \quad \text{OK}$$

Combined endwall suction, diaphragm shear and pull-over

The effect of these combined forces on the fasteners must be considered by the Designer. This is typically performed using an analysis that reduces the available capacity of the diaphragm that is outside the scope of this Manual. The reader is referred to the SDI Diaphragm Design Manual for additional information.

Notes:

1. Zones 1 and 2 can be checked using a similar approach.

Example 10: Ponding Check for Deck Supported on Joist and Girder

Given:

A typical roof bay is framed using W21x44 girders which in turn support 24K7 joists at 5 foot on center. The joists support WR22 deck. Using AISC 360-16, Appendix 2, determine if the deck meets ponding criteria.

Solution:

$I_{gravity}$ = 0.151 inch^4/ft (Table 1)

Use $I_d \geq 25(S^4)10^{-6}$ (AISC Eq. A-2-2)

$= 25(5^4)10^{-6}$

= 0.0156 inch^4/ft

$I_{gravity}$ > 0.0156 Deck is acceptable

Note: The girders and joists must still be checked using AISC 360-16 or SJI TD3.

Example 11: Ponding Check for Deck Supported on Girder Only

Given:

DR20 deck that spans 15 feet is directly supported by W14x30 beams that span 30 feet between columns. Check this roof system for ponding instability using AISC 360-10, Appendix 2. The roof dead load is 6 psf and rain load is 12 psf.

Solution:

Simplified Method (AISC Appendix 2.1)

Primary: W14x30

Secondary: DR20

L_p = 30 ft

L_s = 15 ft

S = 1 ft

I_p = 291 inch^4

I_s = 0.825 inch^4/ft (deck is measured by inch^4/ft) (Table 1)

Check that AISC Eq. A-2-1 conditions are met

$$C_p = \frac{32(L_s)(L_p)^4}{10^7(I_p)} \qquad \text{(AISC Eq. A-2-3)}$$

$$= \frac{32(15)(30)^4}{10^7(291)} = 0.134$$

$$C_s = \frac{32(S)(L_s)^4}{10^7(I_s)} \qquad \text{(AISC Eq. A-2-4)}$$

$$= \frac{32(1)(15)^4}{10^7(0.825)} = 0.196$$

$$C_p + 0.9(C_s) \le 0.25 \qquad \text{(AISC Eq. A-2-1)}$$

$$= 0.134 + 0.9(0.196) = 0.311 \qquad \text{No Good}$$

In this instance, changing the framing to W18x40 beams (I_p = 612 inch^4) would provide an acceptable solution.

C_p = 0.063

C_s = 0.196

$C_p + 0.9\,(C_s) = 0.24$ OK

Note: Other combinations of deck and framing could also prove acceptable.

Solution:

Improved Method (AISC Appendix 2.2)

For DR20

S_n = 0.506 inch^3/ft (Table 1)

S_p = 0.464 inch^3/ft (Table 1)

F_y = 40 ksi

w = Dead + Rain Loads = 6 + 12 = 18 psf

The design moment (M) for a 3-span condition is $\frac{w\ell^2}{10}$ at support and $\frac{w\ell^2}{12.5}$ at midspan.

For this method, f_o is calculated using a load combination of Dead load + Rain load (D + R).

$$M_n = \frac{w\ell^2}{10} = \frac{(18)(15)^2}{10} = 405 \text{ ft-lbs} = 4860 \text{ inch-lbs/ft}$$

$$M_p = \frac{w\ell^2}{12.5} = \frac{(18)(15)^2}{12.5} = 324 \text{ ft-lbs} = 3888 \text{ inch-lbs/ft}$$

$$f_{o_n} = \frac{M_n}{S_n} = \frac{4860}{0.506} = 9605 \text{ psi} \qquad \text{controls}$$

$$f_{o_p} = \frac{M_p}{S_p} = \frac{3888}{0.464} = 8379 \text{ psi}$$

For W14x30

$S = 42 \text{ inch}^3$

$F_y = 50$ ksi

w = (Dead + Rain Loads)(Tributary Width) + Self Weight = (6 + 12)(15) + 30 = 300 lbs/ft

Assuming simple support for the beam,

$$M = \frac{w\ell^2}{8} = \frac{300(30)^2}{8} = 33{,}750 \text{ ft-lbs} = 33.75 \text{ ft-kips}$$

$$f_o = \frac{33.75(12)}{42.0} = 9.64 \text{ ksi}$$

Calculate the stress index

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \qquad \text{(AISC Eq. A-2-3)}$$

$$= \left(\frac{0.8(50) - 9.64}{9.64}\right) = 3.15$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \qquad \text{(AISC Eq. A-2-4)}$$

$$= \left(\frac{0.8(40) - 9.60}{9.60}\right) = 2.33$$

Using AISC Figure A-2-1, enter with U_p = 3.15 and move across to C_s = 0.207 (previously calculated) and read upper limit of C_p = 0.58. Since the upper limit of 0.58 is greater than the previously calculated value of C_p = 0.134, the W14x30 is acceptable.

Using AISC Figure A-2-2, enter with U_s = 2.33 and move across to C_p = 0.134 (previously calculated) and read upper limit of C_s = 0.58. Since the upper limit of 0.58 is greater than the previously calculated value of C_s = 0.196, the DR20 deck is acceptable.

Note: This is a typical result where the improved method, by virtue of being more accurate, will provide more economical ponding checks.

Example 12: Deck Penetration Reinforcement

Given:

WR22 deck is installed on a 6’-0” span, in a 3-span condition. A 24 inch wide opening is cut through the deck for a duct penetration. The roof curb installed over this opening is attached to and capable of transferring the load at the edges of the opening transversely to the adjacent uncut deck edges. Design reinforcing zees to carry the design service loads across the penetration. Deck F_y = 40 ksi, F_u = 50 ksi.

(1) Dead Load = 10 psf

(2) Snow Load = 30 psf

Solution:

Provide reinforcing zees on each edge. Tributary width to each zee is 12 inches (1 foot).

w = (10 + 30) psf $\times$ 1 ft = 40 plf

1.5Z14 on 6 foot span = 47 plf allowable (Table 11.1)

Provide (2) 6 foot long 1.5Z14, one on each edge of opening. Attach to deck with #10 screw at 12 inches on center on both flanges.

Example 13: Sump Pan Installation

Given:

A special case of roof penetration is the sump pan. When properly attached, the sump pan will carry the load of the deck it replaces. It also acts as a small header to transfer loads into adjacent uncut sheets. Approximate per foot (of width) section properties of a standard (0.075 inch) sump pan are: I = 0.36 $inch^4$, S_p = 0.20 $inch^3$. Approximate sump pan analysis methods are shown and a reinforcing technique is shown for cases where the sump pan is located in an end span. Solve using the ASD method and the following information.

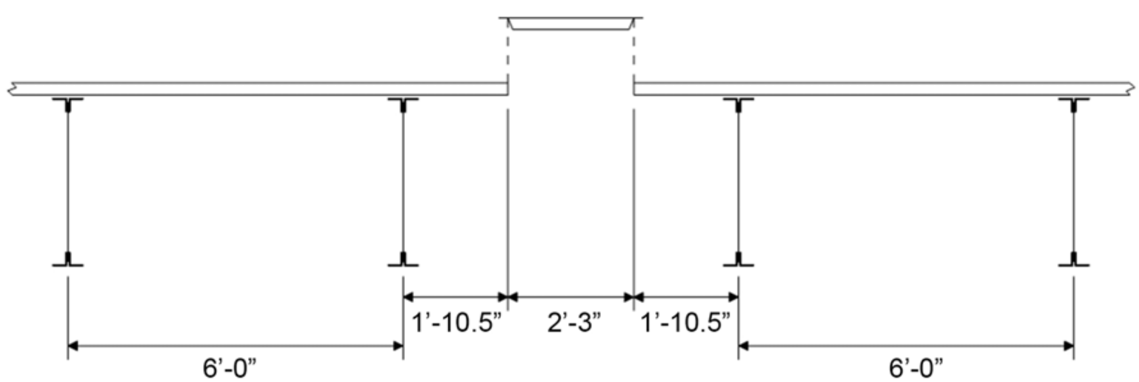
(1) WR20 Deck

(2) Snow Load = 40 psf

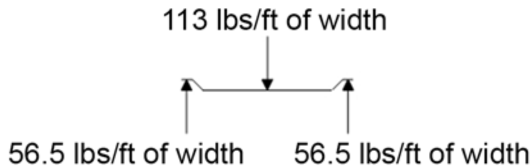
(3) Dead Load = 10 psf

Solution:

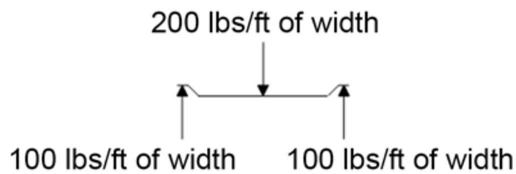
Sump Located at Center of Interior Span



$$\text{Total Load on Sump} = 50\left(\frac{27}{12}\right) = 113 \text{ lbs per foot of width}$$

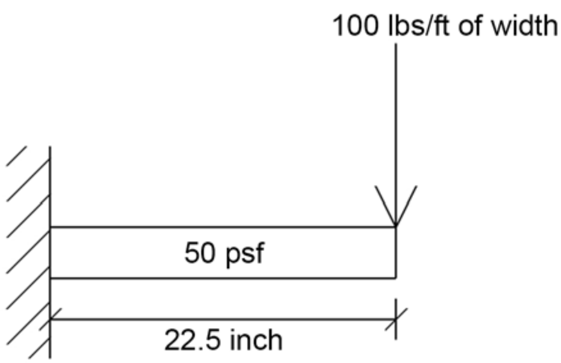


Concentrated Load from construction or maintenance = 200 lbs/ft of width



controls

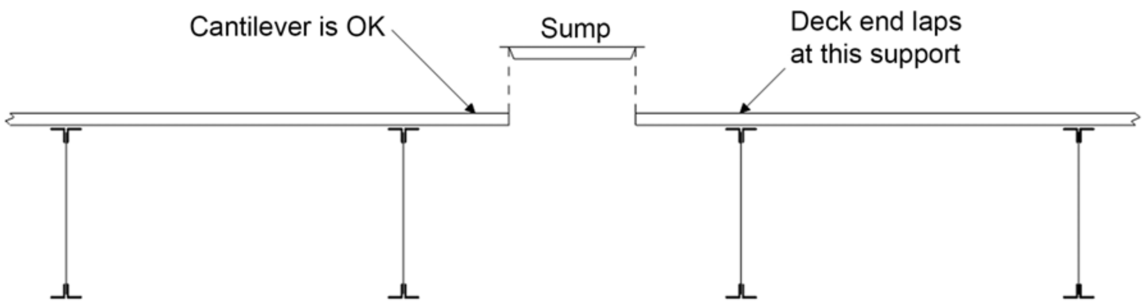
<u>Deck Cantilever Analysis</u>

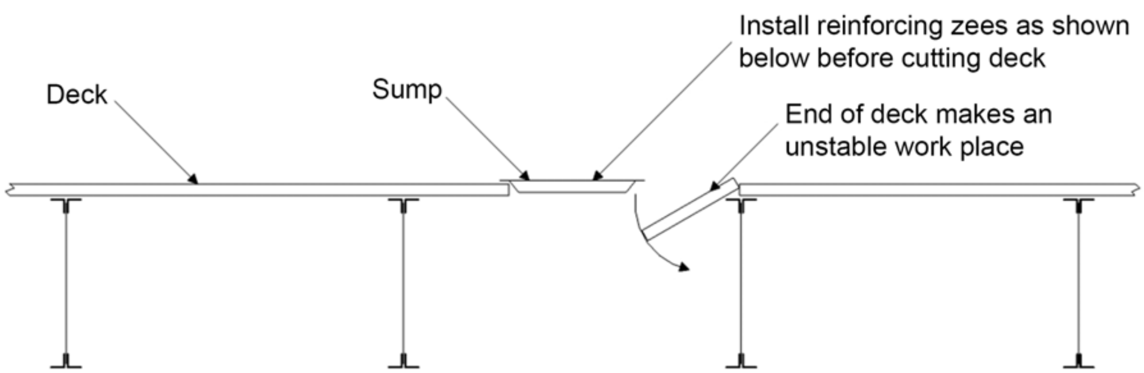


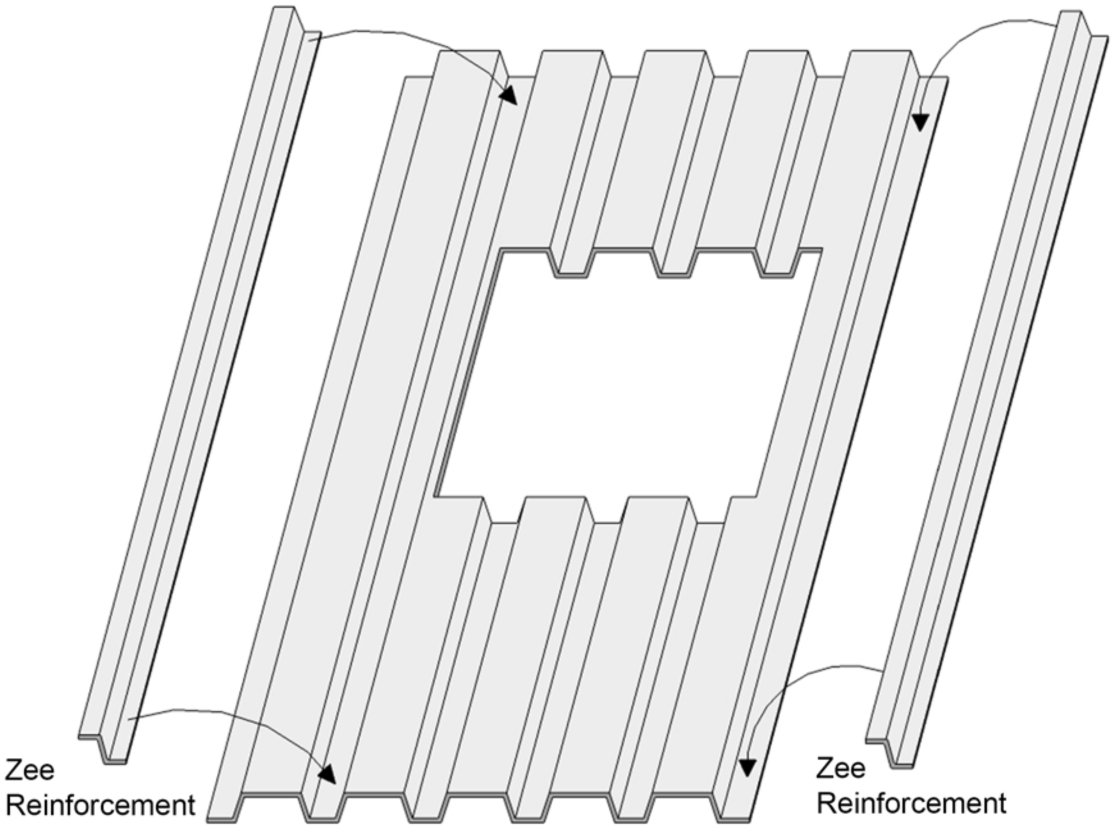
$$M = \frac{50(22.5/12)^2(12)}{2} + 100\,(22.5) = 3{,}305 \text{ inch-lbs/ft of width}$$

$$\frac{M_n}{\Omega} = 5347 \text{ inch-lbs} \quad \text{(Table 1)} \qquad \text{OK}$$

Sump Located at End Span







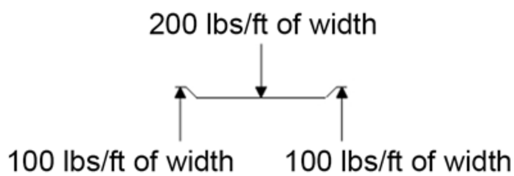
A typical sump pan requires an opening width of 28 inches. Design a 1.5 inch deep reinforcing zee in each rib at each side of opening (flush with top of deck). Zees span between joists.

Considering the uniform gravity loads:

w = 50 psf x 0.5 x (28/12) = 59 plf

Per Table 11, a 1.5Z12 has an ASD capacity of 66 plf on a 6 foot span and would be acceptable.

Considering a 200 pound construction load, and converting the concentrated load at the midspan of the zee into an equivalent uniformly distributed load:



$$M = \frac{PL}{4}$$

$$w_{eq} = \frac{8M}{L^2} = \frac{2P}{L} = \frac{2(100)}{6} = 33 \text{ plf/ft of width}$$

$$w_{eq} = 33 \text{ plf} < 66 \text{ plf}$$

Provide (2) 6 foot long 1.5Z12, one each edge of opening. Attach to deck with #10 screw at 12 inches on center on both flanges.

<u>Example 14: Mechanically Attached Single Ply Roof Membrane on Steel Deck (LRFD)</u>

Deck will be installed on a building with a flat roof (¼-inch per foot roof pitch). The building is 100 feet by 100 feet in plan. Wind pressure calculations are not shown for brevity.

The roof deck is WR22, F_y = 40 ksi. The assumed dead load of the deck and rigid insulation attached to the deck is 5 psf. The deck is supported by joists at 6'-00" on center.

The membrane is 10 foot wide in Zone 1 (Interior) and 5 foot wide in Zone 2 and 3 (Perimeter and Corner).

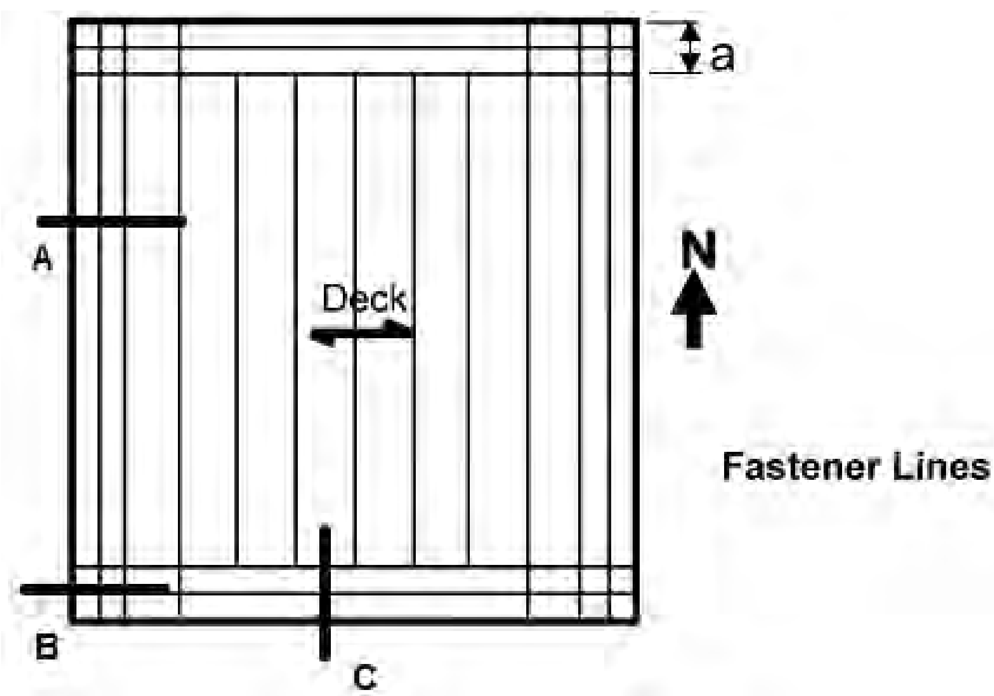


Figure 1 - Roof Plan

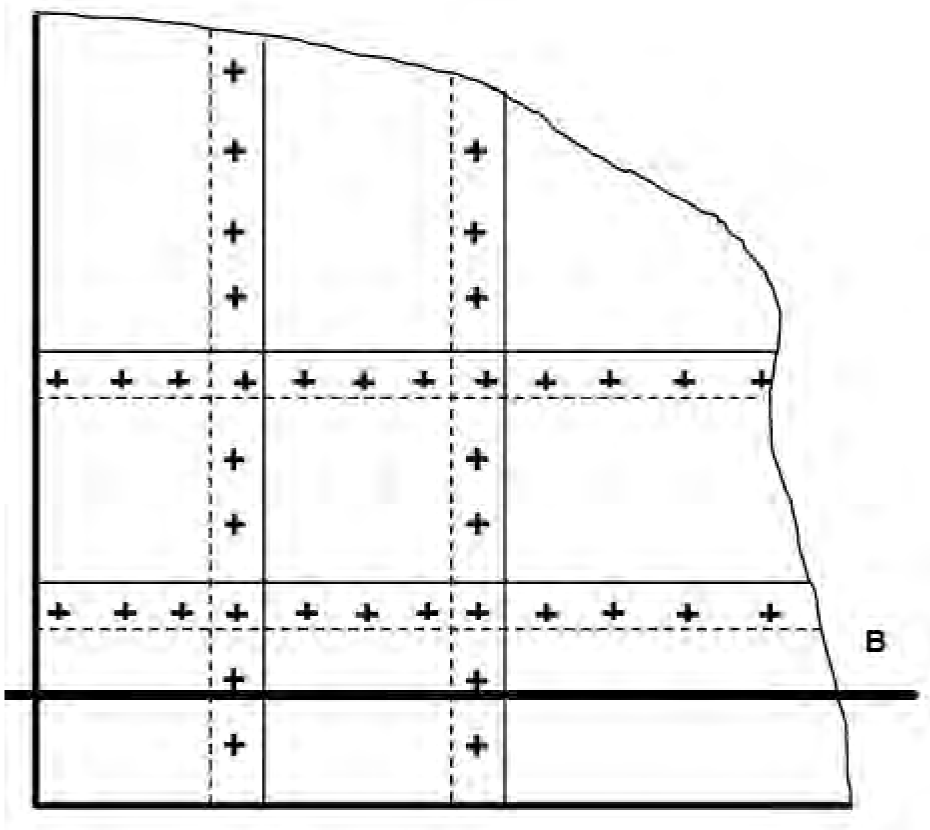


Figure 2 - Corner Zone of Roof

The controlling ASCE 7 load combination (0.9D + 1.0W), the roof dead load and the component and cladding uplift wind pressures are as follows. The edge zone is 10 foot wide.

Zone	W (psf)	D (psf)
Zone 1 (Interior)	-37.8	5.0
Zone 2 (Edge)	-60.3	5.0
Zone 3 (Corner)	-88.5	5.0

To illustrate the potential effect of the attachment pattern, determine the deck strength for the following conditions illustrated by Figures 1 and 2.

For WR22 deck, ΦM_n = 6460 in-lbs/ft; ΦM_p = 6156 in-lbs/ft (RDDM Table 1)

Case 1: Fully adhered membrane

If instead of a mechanically attached membrane, a fully adhered membrane that uniformly loads the deck in uplift was used, RDDM Table 2.4 can be used to check the deck for net uplift loading.

Zone	0.9D + 1.0W	w (psf) Table 2.4	$M_{n\ max}$	$M_{p\ max}$
Zone 1 (Interior)	-33.3 psf	-144 psf	1051 in-lb	1439 in-lb
Zone 2 (Edge)	-55.8 psf	-144 psf	1928 in-lb	2411 in-lb
Zone 3 (Corner)	-84.0 psf	-144 psf	2903 in-lb	3629 in-lb

WR 22 deck is acceptable for the case of wind uplift. By observation, combined bending and shear will not control for the deck. Connection of the deck to the supporting joists must still be checked for this load case.

Case 2: Mechanically attached membrane

For the mechanically attached membrane, the dead load can be assumed to be uniformly distributed because the insulation is attached to the steel deck using closely spaced screws, or less commonly by adhesive. The membrane is attached using a row of screws at each membrane seam.

Case 2A: Interior Zone (Field of Roof)

The membrane is spread parallel to the span of the joists, which places the seams and fastener lines perpendicular to the deck ribs. This is the preferred orientation.

The deck is uniformly loaded by the factored dead load of 4.5 psf (0.9D). With the membrane width of 10 foot, the deck is loaded by a wind uplift line load, perpendicular to the ribs, of 378 plf (37.8 psf x 10 ft.)

Interior Load Case A: With the first concentrated uplift load located at the midspan of the first deck span (3 foot) and the second concentrated load located in the 3rd span (13 foot), $M_{n\ max}$ = 460.7 ft-lbs (5528 in-lbs) and $M_{p\ max}$ = 172.1 ft-lbs (2065 in-lbs)

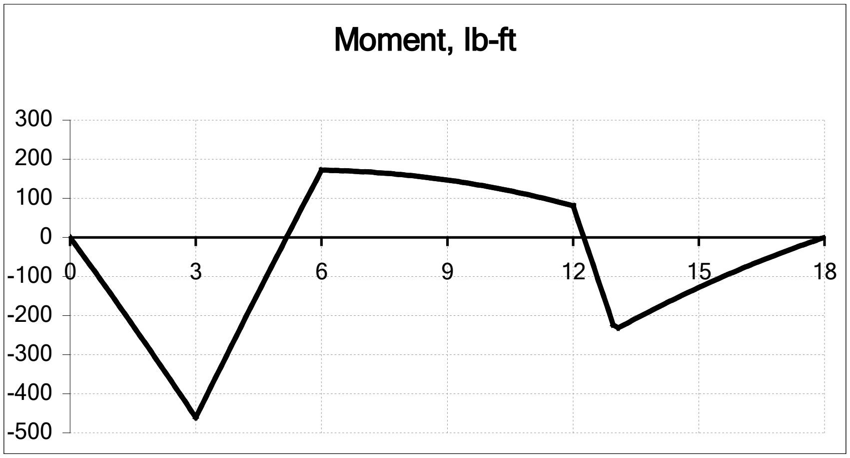


Figure 3 - Interior Load Case A

Interior Load Case B: With the first concentrated uplift load located at the midspan of the 2nd deck span (9 foot), $M_{n\ max}$ = 392.8 ft-lbs (4714 in-lbs) and $M_{p\ max}$ = 153.9 ft-lbs (1847 in-lbs)

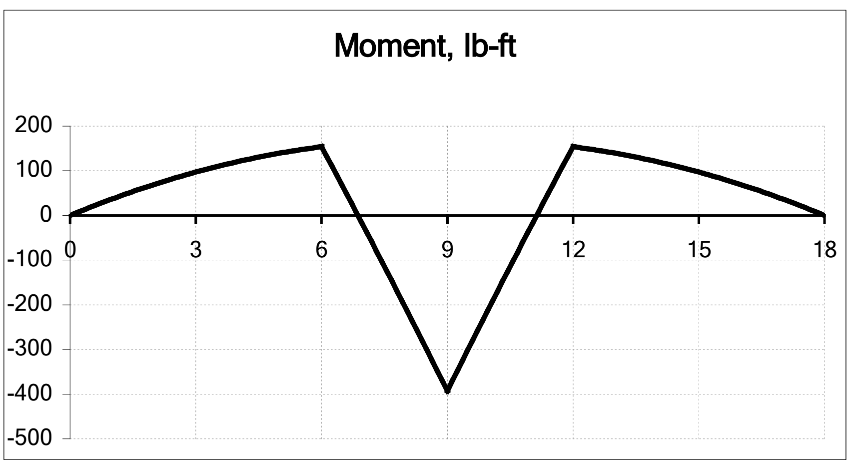


Figure 4 - Interior Load Case B

Interior Load Case A controls for both positive and negative flexure, and the WR22 deck has sufficient flexural strength.

Case 2B: North-South Perimeters in Zone 2(Attachments Perpendicular to Deck Span)

The first line load is 5 feet from the building edge and the second is 10 feet from the edge. The third is 20 feet from the edge. This is located at Section A of Figure 1.

First Line Load: The membrane area for the first line load = (5 foot) = 5.0 square feet. Line load is 5 ft x 60.3 psf (Zone 2 pressure) = 302 pounds.

Second Line Load: The membrane area for the second line load = (2.5 feet + 5 feet)(1.0 feet) = 7.5 square feet. Line load is 2.5 ft x 60.3 psf (Zone 2 pressure) plus 5 ft x 37.8 psf (Zone 1 pressure) = 340 pounds

Third Line Load: The membrane area for the third line load = (5 feet + 5 feet)(1.0 feet) = 10.0 square feet. Line load is 10 ft x 37.8 psf (Zone 1 pressure) = 378 pounds. For a 3-span deck configuration, this load occurs off the deck span and is not included in the calculation.

With the two concentrated uplift loads located as stated, $M_{n\ max}$ = 282.0 ft-lbs (3384 in-lbs) and $M_{p\ max}$ = 217.6 ft-lbs (2611.2 in-lbs)

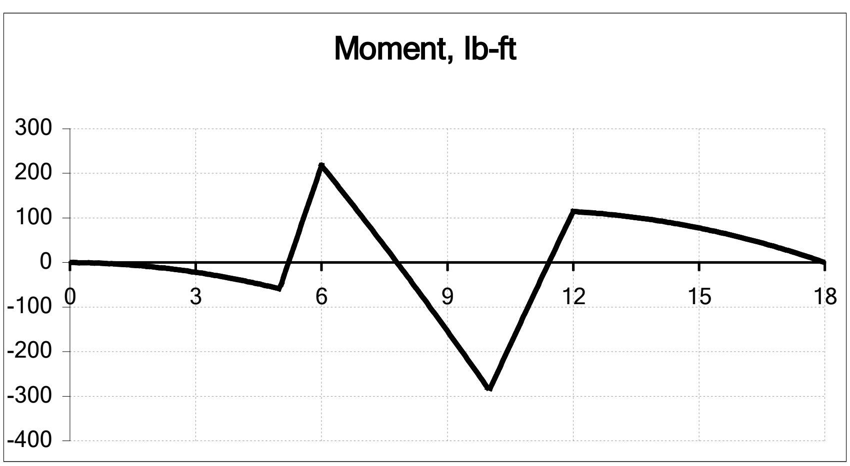


Figure 5 - North-South Perimeters in Zone 2

The WR22 deck has sufficient flexural strength.

Case 2C: North-South Perimeters in Zone 3 (Attachments Perpendicular to Deck Span)

The first line load is 5 feet from the building edge and the second is 10 feet from the edge. The third is 20 feet from the edge. This is located at Section B of Figures 1 and 2.

First Line Load: The membrane area for the first line load = (5 foot) = 5.0 square feet. Line load is 5 ft x 84.0 psf (Zone 3 pressure) = 420 pounds.

Second Line Load: The membrane area for the second line load = (2.5 feet + 5 feet)(1.0 feet) = 7.5 square feet. Line load is 2.5 ft x 84.0 psf (Zone 2 pressure) plus 5 ft x 60.3 psf (Zone 1 pressure) = 512 pounds.

The third line load occurs off the 3-span deck.

With the two concentrated uplift loads located as stated, $M_{n\ max}$ = 433.0 ft-lbs (5196 in-lbs) and $M_{p\ max}$ = 217.6 ft-lbs (3862 in-lbs)

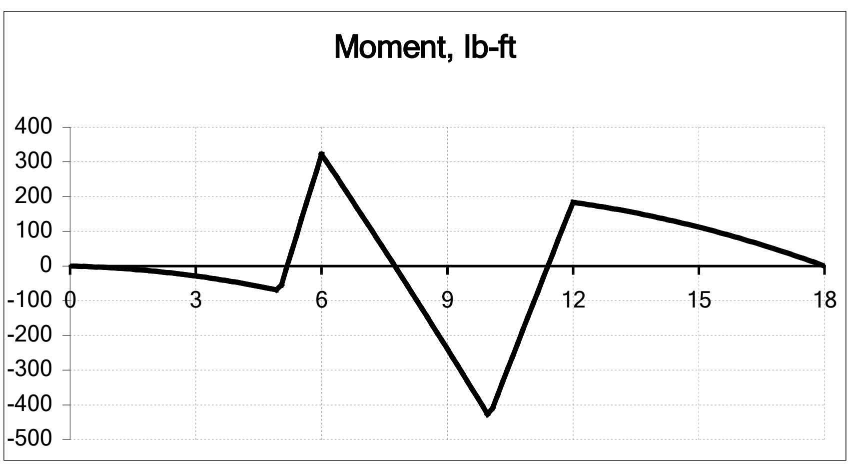


Figure 6 - North-South Perimeters in Zone 3

The WR22 deck has sufficient flexural strength.

Case 2D: East-West Perimeters (Attachments Parallel to Deck Span)

Fasteners running parallel to the deck ribs is a severe condition and not recommended. This imposes a line load parallel to the deck ribs, and loads a single deck rib. If used, the following loading conditions occur. This is located at Section C of Figure 2.

Wind uplift = 60.3 psf x 5 foot (fastener spacing) on a 6 inch width of deck.
Dead load = 4.5 psf on a 6 inch width of deck.

With the loads as stated, $M_{n\ max}$ = 855.4 ft-lbs (10265 in-lbs) per foot (20530 in-lbs on a single rib) and $M_{p\ max}$ = 1069.2 ft-lbs (12830 in-lbs) per foot (25661 in-lbs on a single rib)

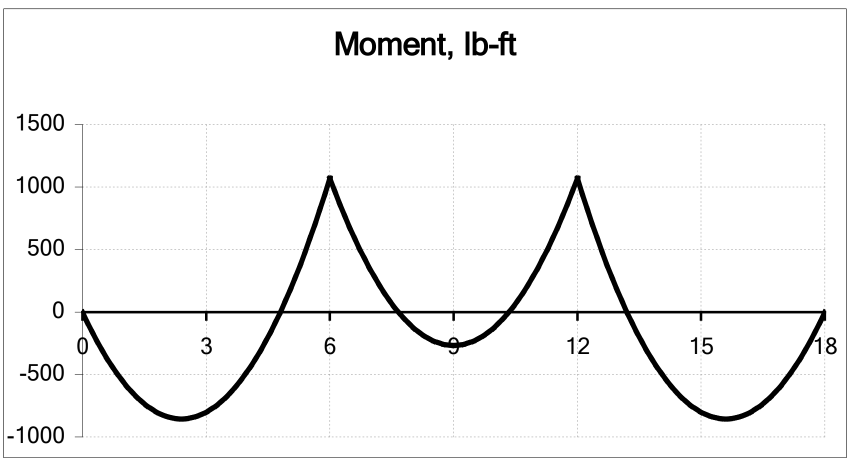


Figure 7 - North-South Perimeters in Zone 3

The WR22 deck is greatly overloaded in flexure.

Summary:

Adhered Membrane:

Adhered Membrane	$M_{n\ max}$ / ΦM_n	$M_{p\ max}$ / ΦM_p
Zone 1 (Interior)	0.16	0.23
Zone 2 (Edge)	0.30	0.39
Zone 3 (Corner)	0.45	0.59

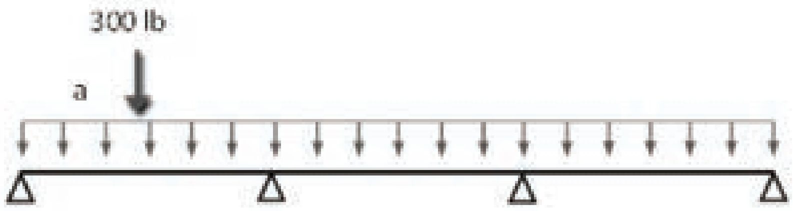
Mechanically Attached Membrane:

Mechanically Attached Membrane	$M_{n\ max}$ / ΦM_n	Moment Increase over Adhered Membrane	$M_{p\ max}$ / ΦM_p	Moment Increase over Adhered Membrane
Zone 1 (Interior)	0.86	530%	0.34	143%
N-S Perimeter in Zone 2 (Edge)	0.52	175%	0.42	109%
N-S Perimeter in Zone 3 (Corner)	0.80	179%	0.63	106%
E-W Perimeter, attached parallel to deck ribs	3.18	1065%	4.17	1065%

Observations:

1. A mechanically attached membrane greatly increases the flexural demand on the deck over an adhered membrane. Except where attached parallel to the deck ribs in the E-W perimeter, the 40 ksi WR deck has adequate flexural capacity. Lower strength 33 ksi deck may have been overloaded in flexure in Zone 1 and 3.
2. It is important that the membrane width be decreased in the edge zones as defined in ASCE 7. If the 10 foot membrane width was continued in the edge zones, the deck would have been overloaded in flexure in Zones 2 and 3.
3. Membrane width is important. A 12 foot membrane width, as opposed to the 10 foot width in this example, would have most likely overloaded the deck in flexure. Membranes much wider than 12 foot are currently in use.
4. The effect on deck securement to the support framing, and the uplift loads on the support framing, also need to be considered.
5. Deck formed from steel of higher strength than 40 ksi, or thicker gage, may be considered in new construction.
6. Careful analysis of existing structures being re-roofed or re-covered with a mechanically attached membrane, that originally had an adhered membrane, must be performed. Narrower membrane widths may be needed.

Example 15: Concentrated Load on Roof Deck (LRFD)



Given: WR deck spanning 6-0
Deck bearing length = 2 inches exterior, 4 inches interior
SDL = 15 psf, LL = 20 psf
300 lb post dead load supported on 6x6 baseplate. Post location
Is unknown.

Find: WR minimum deck gage (thickness), LRFD analysis.

Solution 1: From Table 13.1, P_{max} for WR22 = 235 lbs using a load factor = 1.6.
If load factor = 1.2, P_{max} = 235*(1.6/1.2) = 313 lbs > 300 lbs OK

Solution 2: From Table 13.2, P_{max} for WR22 = 182 lbs using a load factor = 1.6. If load factor = 1.2, P_{max} = 182*(1.6/1.2) = 243 lbs < 300 lbs NG
Use 20 gage.

Solution 3: Solution 3 locates the post at "reasonable" locations for maximum stress with load redistribution. Exact load locations for maximum stress vary with concurrent uniform load and span. Try 22 gage.

Uniform Loads: Uniform loads (psf) are converted to plf for consistency with property tables and load redistribution methodology.

From Table 1, WR22 gage = 1.6 psf (1 ft) = 1.6 plf
W = [1.2(15) + 1.2(1.6) + 1.6(20)] = 51.9 plf

Check Web Crippling (TFE):

Locate post at a = 0 inches.

From Table 12, b_e = 6 inches

From Table 6, A = 569, B = 0.82

Exterior reaction = 0.4 (51.9 plf) (6 ft) + 1.2 (300 lbs) (12 in / 6 in) = 845 lbs / foot of width

$\Phi P_{n(TFE)}$ = $\Phi A(1+B\sqrt{N})$ = 0.85 (569) (1+ 0.82 $\sqrt{2}$) = 1044 lbs > 845 lbs / foot of width OK

Check Web Crippling (OFE):

Locate post at a = 1.5 inches.

Load redistribution is unlikely. Conservatively use b_e = 6 inches

From Table 6, A = 401, B = 1.46

$\Phi P_{n(OFE)}$ = $\Phi A(1+B\sqrt{N})$ = 0.90 (401) (1+ 1.46 $\sqrt{2}$) = 1106 lbs > 845 lbs / foot of width OK

Commentary: The load is located close enough to the support that load redistribution is unlikely, but both flanges are not simultaneously loaded.

Check Web Crippling (TFI):

Locate post at a = 6 ft.

From Table 12, b_e = 6 inches

From Table 6, A = 765, B = 1.22

Interior reaction = 1.1 (51.9 plf) (6 ft) + 1.2 (300 lbs) (12 in / 6 in) = 1063 lbs / foot of width

$\Phi P_{n(TFI)}$ = $\Phi A(1+B\sqrt{N})$ = 0.85 (765) (1+ 1.22 $\sqrt{4}$) = 2237lbs > 1063 lbs / foot of width OK

Check Web Crippling (OFI):

Locate post at a = 5.875 ft.

From Table 12, b_e = 6 inches

From Table 6, A = 709, B = 0.99

$\Phi P_{n(OFI)}$ = $\Phi A(1+B\sqrt{N})$ = 0.85 (709) (1+ 0.99 $\sqrt{4}$) = 1796 lbs > 1063 lbs OK

Commentary: Table 5 assumes 1.5 inches of exterior bearing and 2.5 inches of interior bearing. Bearing lengths for this problem exceed these minimums, so Table 5 could be used to expedite web crippling checks.

Check Shear:

Locate post at a = 5.833 ft.

Load redistribution is unlikely. Conservatively use b_e = 6 inches

From Table 4, ϕV_u = 2479 lbs

Interior shear = 0.6 (51.9 plf) (6 ft) + 1.2 (300 lbs) (12 in / 6 in) = 907 lbs < 2479 lbs / foot of width OK

Commentary: Shear is unlikely to govern over web crippling.

Check Flexure: Positive Bending

Locate post at a = 0.25L = 1.5 ft.

From section 2.5, X = 0.25, b_e = 12 in.

From computer models, +M = 6000 in-lbs / foot of width

From Table 1, ϕM_p = 6156 in-lbs > 6000 in-lbs / foot of width OK

Check Flexure: Negative Bending

Locate post at a = 0.75L = 4.5 ft.

From section 2.5, X = 0.25, b_e = 12 in.

From computer models, -M = 4512 in-lbs / foot of width

From Table 1, ϕM_n =6460 in-lbs > 4512 in-lbs / foot of width OK

Check Shear Bending Interaction:

Locate post at a = 0.75L = 4.5 ft.

From Section 2.5, X = 0.25, b_e = 12 in.

From computer models, -M = 4512 in-lbs

From computer models, -V = 488 lbs

From Table 1, ϕM_n =6460 in-lbs / foot of width

From Table 4, ϕV_u = 2479 lbs / foot of width

$$\sqrt{\left(\frac{M}{\Phi M_n}\right)^2 + \left(\frac{V}{\Phi V_n}\right)^2} = \sqrt{\left(\frac{4512}{6460}\right)^2 + \left(\frac{488}{2479}\right)^2} = 0.73 < 1.0 \quad \text{OK}$$

Check Deflection:

Locate post at $a = 0.25L = 1.5$ ft.

From section 2.5, $X = 0.25$, $b_e = 12$ in.

From computer models, Δ live load + post load = 0.34 in = L / 212

Commentary: L/212 acceptability depends on ceiling types and ponding concerns.

Reference Section 2.3 for additional details.

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REFERENCES

SECTION 7

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REFERENCES

<u>American Concrete Institute (ACI)</u>
ACI 523.1 Guide for Cast-in-Place Low Density Cellular Concrete

<u>American Institute of Steel Construction (AISC)</u>
ANSI/AISC 360-16, Specification for Structural Steel Buildings

<u>American Iron and Steel Institute (AISI)</u>
AISI S100-16, North American Specification for the Design of Cold-Formed Steel Structural Members

AISI S310-16, North American Standard for the Design of Profiled Steel Diaphragm Panels

<u>American Society for Testing and Materials (ASTM)</u>
ASTM A653/A653M-15, Standard Specification for Sheet Steel, Zinc Coated (Galvanized or Zinc-Iron Alloy Coated (Galvanealed) by the Hot Dip Process.

ASTM A924 / A924M - 08a, Standard Specification for General Requirements for Steel Sheet Metallic-Coated by the Hot-Dip Process

ASTM A1008 / A1008M-15, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

ASTM C332-17, Standard Specification for Light-Weight Aggregates for Insulating Concrete

ASTM E119 - 18, Standard Test Methods for Fire Tests of Building Construction and Materials

<u>American Society of Civil Engineers (ASCE)</u>
ASCE/SEI 7-16 Minimum Design Loads for Buildings and Other Structures

<u>American Welding Society (AWS)</u>
AWS D1.3:2008, Structural Welding Code-Sheet Steel

<u>National Roofing Contractors Association (NRCA)</u>
The NRCA Roofing Manual: Membrane Roof Systems - 2019

<u>Steel Deck Institute (SDI)</u>
ANSI/SDI QA/QC-2017, Standard for Quality Control and Quality Assurance for Installation of Steel Deck

ANSI/SDI RD-2017, Standard for Steel Roof Deck

Diaphragm Design Manual, 4th Edition, 2012

SDI COSP-2017, Code of Standard Practice

SDI-MOC, Manual of Construction with Steel Deck, 3rd Edition, 2016

SDI Standard Practice Details, 2nd Edition, 2001

SDI Technical Note 7, “Mechanical Attachment of Single Ply Roofing Membranes to Steel Roof Deck: Implications for Steel Deck Design”

SDI Technical Note 8, “Quality Assurance for Welding Steel Deck”

Steel Joist Institute (SJI)
Structural Design of Steel Joist Roofs to Resist Ponding Loads (Technical Digest 3), June 2018

Underwriters Laboratories (UL)
Fire Resistance Directory

CONVERSIONS

US Customary to SI Unit Conversion Table

	TO CHANGE	MULTIPLY BY
LENGTH	in to mm ft to mm ft to m	25.4 (exact) 304.8 (exact) 0.3048 (exact)
AREA	in^2 to mm^2 ft^2 to m^2	645.16 (exact) 0.092903
MASS	lb to kg 2000 lb to 1000 kg lb/ft to kg/m lb/ft^3 to kg/m^3 lb/yd^3 to kg/m^3	0.453592 0.907185 1.48816 16.0185 0.593276
FORCE	lb to N kip to kN lb/in to N/m lb/ft to N/m kip/ft to kN/m psf to kN/m^2	4.44822 4.44822 175.127 14.5939 14.5939 47.880
PRESSURE	lb/in^2 to kPa lb/ft^2 to kPa kip/in^2 to MPa	6.89476 0.04788 6.89476
SECTION MODULUS	in^3 to mm^3 in^3/ft to mm^3/m	16387.1 53763.5
MOMENT OF INERTIA	in^4 to mm^4 in^4/ft to mm^4/m	416231 1365587

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