engineering manual

STEEL DECK on COLD-FORMED STEEL FRAMING

FIRST EDITION

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STEEL DECK ON COLD-FORMED STEEL FRAMING DESIGN MANUAL

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FOREWARD

This SDI Steel Deck on Cold-Formed Steel Framing Design Manual is the first design manual that specifically addresses the design of steel deck on cold-formed framing. The design of the steel deck is similar to deck on heavier rolled beams or open web steel joists, but it requires attention to some different detailing and fastening methods. This Manual will concentrate on these differences.

This Manual conforms to the ANSI/SDI RD-2017 *Standard for Steel Roof Deck*, the ANSI/SDI NC-2017 *Standard for Non-Composite Steel Floor Deck*, ANSI/SDI C-2017 *Standard for Composite Steel Floor Deck-Slabs* and the AISI S310 *North American Standard for the Design of Profiled Steel Diaphragm Panels*. The SDI recommends that the user of this Manual obtain a copy of these Standards and refer to them when using this Manual. All SDI Standards are available for free download from the SDI website. The AISI S310 Standard is available for free download from the Cold Formed Steel Engineers Institute (CFSEI) website.

Also helpful and available to the user of this Manual are the *SDI Roof Deck Design Manual* (RDDM), *SDI Floor Deck Design Manual* (FDDM) and the *SDI Diaphragm Design Manual* (DDM). Their concurrent use with this Manual is recommended.

While conforming to the requirements of the Standards, this Manual also provides recommendations of good design practices that may either not be included in the Standards, or may exceed the minimum requirements of the Standards. When recommended practice is beyond the minimum requirements of the Standards, this will be noted. In all instances, the design of steel 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 Manual.

This Manual describes the use of steel deck in common applications. Deck can be used in applications that are not covered within the scope of this Manual. The use of deck in these alternate applications may be permitted under provisions included in the applicable building code.

Where this Manual or other SDI publications refer to “Designer,” this means the entity that is responsible to the Owner for the overall structural design of the project, including the steel deck. 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 Introduction

Steel roof and floor decks are viable, cost effective options for constructing roofs and floors on cold-formed steel framing. Steel deck has several advantages over wood panel products, such as plywood and oriented strand board (OSB) in this application:

1. Steel deck is non-flammable.
2. It can span further than the typical 24 inch limit for wood panels, resulting in wider, more cost effective, framing member spacing.
3. When properly detailed, it provides stronger and stiffer diaphragms than wood panel construction.

The design and construction of steel deck on cold-formed steel framing is similar to when the deck is supported on open web steel joists or rolled beams, but there are some differences in the design and detailing. This Manual will point out those differences, and provide necessary design guidance to allow a designer to properly use steel deck in this application. This Manual assumes that the user is familiar with general application of steel deck, and this Manual also assumes that the user has access to other SDI Design Manuals, specifically the *SDI Roof Deck Design Manual* (RDDM), the *SDI Floor Deck Design Manual* (FDDM) and the *SDI Diaphragm Design Manual* (DDM).

This Manual will also refer to several design Standards:

ANSI/SDI RD-2017	Standard for Steel Roof Deck
ANSI/SDI C-2017	Standard for Composite Steel Floor Deck-Slabs
ANSI/SDI NC-2017	Standard for Non-Composite Steel Floor Deck
ANSI/AISI S100-16	North American Specification for the Design of Cold-Formed Steel Structural Members
ANSI/AISI S310-13	North American Standard for the Design of Profiled Steel Diaphragm Panels
ANSI/AISI S310-16	North American Standard for the Design of Profiled Steel Diaphragm Panels

Section 1.2 Cold-Formed Steel Framing

Cold-formed framing for floor and roof members, including trusses, typically has the base steel thickness designated in mils, or units of 1/1000 of an inch. Typical industry practice is as shown in Table 1-1.

Table 1-1 Structural Cold-Formed Framing Base Steel Thickness

Thickness Designation (mils)	Minimum Thickness (inches)	Design Thickness (inches)	Reference Gage (Not Used for Specifying)
27	0.0269	0.0283	22
33	0.0329	0.0346	20
43	0.0428	0.0451	18
54	0.0538	0.0566	16
68	0.0677	0.0713	14
97	0.0966	0.1017	12
118	0.1180	0.1242	10

Not all base steel thicknesses are available from all suppliers, and 27 mil material is rarely used for floor and roof joists, while 118 mil material is rarely used in trusses.

The most common steel material properties for cold-formed framing, including trusses, are as shown in Table 1-2:

Table 1-2 Structural Cold-Formed Framing Material Properties

Application	Yield Strength (ksi)	Ultimate Strength (ksi)
Individual framing and trusses	33	45
Individual framing and trusses	50	65
Trusses (limited suppliers)	55	70

Floor and roof framing can be constructed either of trusses or of individual members. The design of the framing is beyond the scope of this manual, and the user is referred to any of several publications that are listed in the General References section of this Manual. However, it is important that the designer be cognizant of the limitations that the shape of the supporting framing imposes on the design of the steel deck.

Individual members used as floor joists or roof rafters are usually lipped channels, as shown in Figure 1-1. There are industry standard cross sections that are available from many manufacturers.

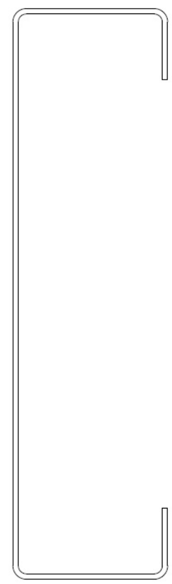

Figure 1-1 Lipped Channel

The flange width is important to consider when designing the bearing and attachment of the steel deck to the flange. Typical standard flange widths for structural cross sections are 1-5/8 inch (S162), 2 inch (S200) and 2-1/2 inch (S250). Also available, but not as commonly used in this application are 3 inch (S300) and 3-1/2 inch (S350).

Cold-formed steel trusses are commonly used for both floor and roof applications. Trusses are usually designed by the manufacturer as a specialty engineered item. Trusses can use lipped channels for top chords (referred to as C-Section Trusses), however there are proprietary sections used by some truss manufacturers, examples of which are shown in Figure 1-2.

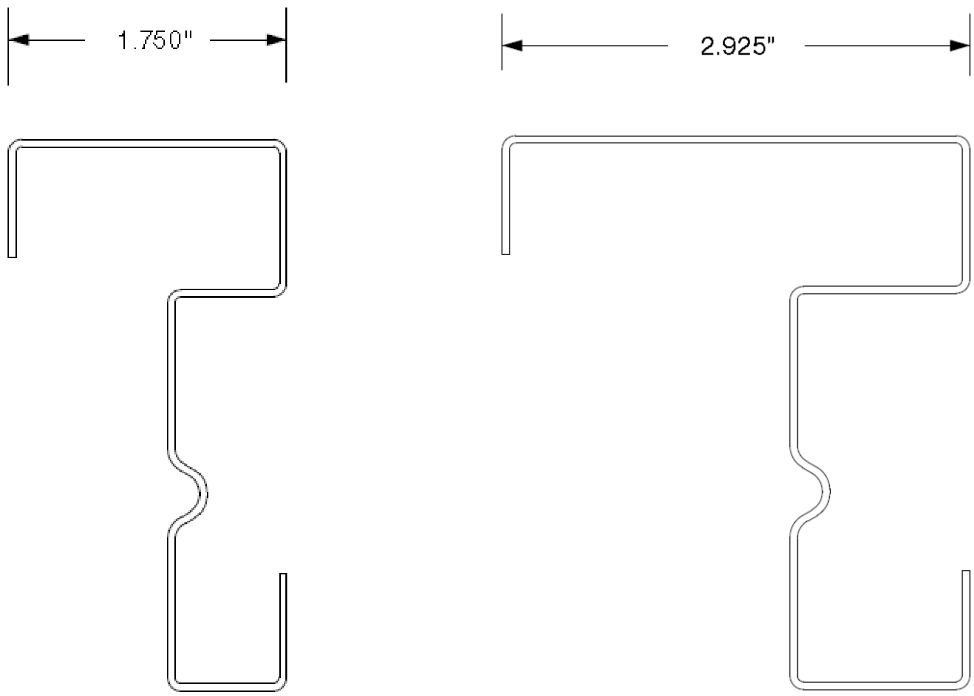

Figure 1-2A Proprietary Truss Chords (Courtesy of Aegis)

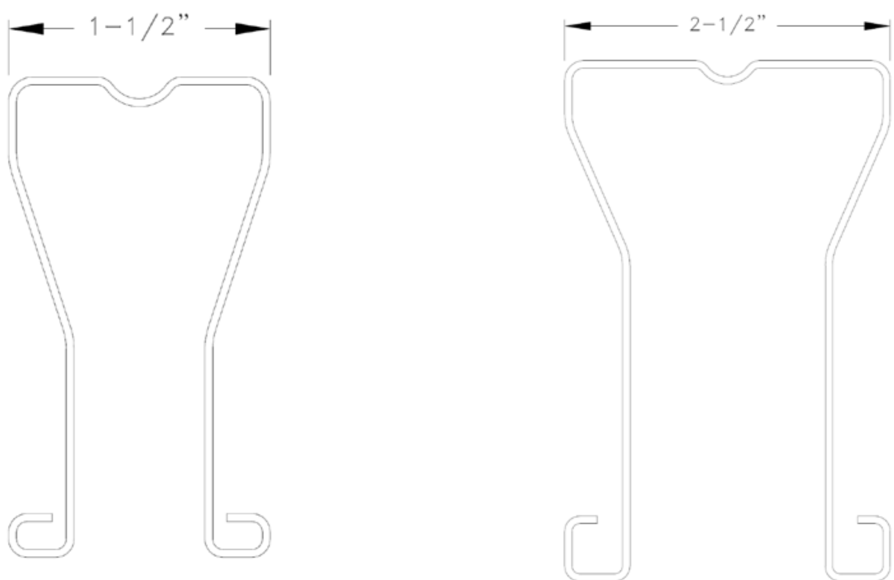

Figure 1-2B Proprietary Truss Chords (Courtesy of Alpine Trussteel)

Both individual members and trusses can be connected into multiple plies when needed for load carrying capacity.

Perhaps the most important consideration is the flat width of the framing member or truss top chord, which is measured as the overall flange width minus the outside corner radii. This is shown in Figure 1-3. The corner radii can significantly reduce the available bearing width for the steel deck.

Figure 1-3 Effect of Corner Radii on Available Bearing Width

Section 1.3 Deck Profiles and Materials

Steel deck profiles for roof and floor applications will be covered in the appropriate chapters. Additional information on deck profiles, materials, and finishes can be found in the SDI RDDM and SDI FDDM.

Section 1.4 Design Responsibilities

The specific responsibilities for design are laid out in two Code of Standard Practice publications; the AISI Code of Standard Practice for Cold-Formed Steel Structural Framing (AISI S202) and the SDI Code of Standard Practice (SDI-COSP).

Section 1.4.1 AISI Code of Standard Practice for Cold-Formed Steel Structural Framing

The following is excerpted from AISI S202:

J1 Steel Floor and Roof Deck

J1.1 Responsibility of Building Designer

J1.1.1 Design Responsibility

The building designer shall be responsible for the design and specification of the steel deck to resist the design loads, both out-of-plane and in-plane. The design and specification shall include, as applicable, the following:

(1) Steel deck layout, including steel deck direction,

(2) Steel deck profile(s) and minimum base steel thicknesses of the steel deck,

(3) Minimum required steel deck bearing widths on support framing,

(4) Steel deck attachment type and pattern, including support, sidelap, boundary, and collector attachment requirements,

(5) Minimum support member base steel thickness, yield and tensile strengths,

(6) Required support framing at hips, valleys, ridges, eaves, penetrations, and other locations to support deck edges and resist design loads,

(7) Chords, collectors, shear transfer framing, blocking, and other items as required to develop a complete diaphragm system, and

(8) Integration of steel deck with overframing.

J1.1.2 Required Information in Construction Documents

The building designer, through the construction documents, shall provide information for the supply and installation of the steel deck and other structural elements and shall provide the applicable information listed in Section J.1.1.1 within the construction documents.

Commentary:

The building designer is responsible for design of the steel deck as part of a complete system for gravity, uplift, and diaphragm loads in accordance with the applicable standards ANSI/SDI RD, ANSI/SDI NC, and ANSI /SDI C.

In complying with these responsibilities, the building designer should consider the following:

(1) Minimum required bearing widths for the steel deck, including locations where the steel deck may need to be butted rather than continuous or lapped;

(2) Minimum CFS structural framing or truss top chord thickness for development of diaphragm forces and fastener pullout;

(3) Limits on the number of fasteners that can be installed within a single steel deck rib;

(4) The design for deck support at locations where the steel deck changes direction and at unsupported steel deck edges, such as at valleys, hips, eaves, ridges, penetrations and openings; and

(5) The design of a complete diaphragm system, including shear blocking and other shear transfer devices.

When the design of the steel deck is to be performed by a specialty designer, the building designer must specify the following:

(1) Extent of steel deck and direction of span;

(2) Magnitude, type, and location of all loads to be supported by the steel deck;

(3) Magnitude and type of lateral load to be transferred;

(4) Load path (i.e., where loads originate and where they are to be transferred);

(5) Bearing material and conditions; and

(6) Any special requirements for the design of the gravity, uplift, or lateral load transferring elements.

J1.1.3 Review Steel Deck Submittal Packages

The building designer shall review the steel deck submittal package. All such submittals shall provide for a notation indicating review status.

J1.2 Responsibility of Steel Deck Support Framing Designers

J1.2.1 The trusses shall be designed in accordance with Section I1.

J1.2.2 The CFS structural framing shall be designed in accordance with Chapters A through H.

J1.3 Responsibility of Steel Deck Supplier

The steel deck supplier shall supply steel deck in accordance with the requirements of the construction documents, and a steel deck submittal package when required by the construction documents.

Commentary: The requirements of the steel deck submittal package are covered in SDI COSP, Code of Standard Practice.

J1.4 Responsibility of Contractor

The contractor shall review proposed construction operations to determine if construction loads will exceed those specified in SDI-RD, SDI-NC, and SDI-C, as applicable, and notify the building designer prior to submission of the steel deck submittal if those loads are to be exceeded.

Section 1.4.2 SDI Code of Standard Practice

The following is excerpted from the SDI Code of Standard Practice. Underlining is for emphasis.

Section 4 Estimating and Bidding

4.2 Base Bids: Unless otherwise specified, base bids shall be based on the following scopes:

4.2.1 Roof Deck: Base bids for roof deck shall include roof deck as shown in plan on the structural drawings. Base bid shall also include ridge, hip and valley plates which are not part of the vertical load resisting system, and sump pans per architectural drawings and specifications. No other deck or accessories shall be included unless specified.

4.2.2 Composite Floor Deck and Non-Composite Floor Deck: Base bids shall include deck as shown in plan on the structural drawings and only those accessories specifically designated on the structural drawings and called for in the appropriate division of the specifications. No other deck or accessories shall be included unless specified.

4.2.3 Special Details: Any material required to support the steel deck shall not be included. The design of deck supports shall be the responsibility of the designer. Deck shall be furnished in sheet lengths of 6 feet (2.0 m) or greater. Any deck sheets requiring lengths less than 6 feet (2.0 m) shall be field cut by others, unless specifically noted within the scope.

4.3 Excluded Materials: Unless otherwise specified, the following materials are excluded from the bid.

4.3.5 Support Materials: Deck supporting members, including but not limited to ridges, hips, valleys, span direction changes and floor deck shoring.

One key point to remember is that the deck supplier is NOT RESPONSIBLE for providing any support framing.

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ROOF DECK

SECTION 2

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Section 2.1 Roof Deck Profiles and Finishes

Any of the roof deck profiles shown in the SDI Roof Deck Design Manual (RDDM) can be used on cold-formed steel roof trusses or rafters, with 1-1/2 inch Wide Rib (WR) deck (Figure 2.1) being the most common for spans of up to 8 feet. Engineering information, including section properties and span tables, can be found in the RDDM and in manufacturers literature.

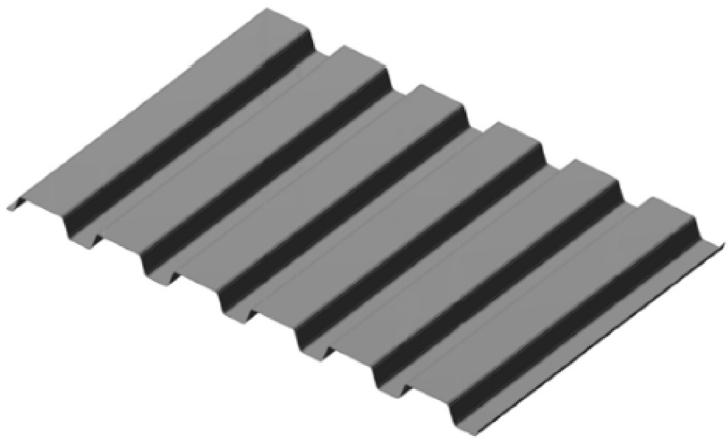

Figure 2.1 - 1-1/2 Inch Wide Rib (WR) Deck

For closely spaced trusses or rafters, form deck of 9/16 inch to 1 inch depth (Figure 2.2) can also be used. Engineering information for these deck profiles can be found in manufacturer's literature.

Figure 2.2 - Form Deck for Roof Applications

Because of the preponderance of WR deck for this application, this Manual will concentrate on this deck profile.

Section 2.2 Sloped Roof Diaphragms

Buildings with sloped roofs are much more common than flat, or almost flat roofs, when supported by cold-formed steel framing. This section will address the differences between flat roof diaphragms and sloped roof diaphragms. Standard diaphragm assumptions will be made; that the diaphragm carries shear only and that the diaphragm chords resist axial forces due to bending of the diaphragm as a deep beam.

Section 2.2.1 Monoslope (Shed) Roof Diaphragms - Wind

Section 2.2.1.1 Transversely Loaded Monoslope Roof Diaphragms

Consider a shed roof diaphragm, loaded transversely by wind:

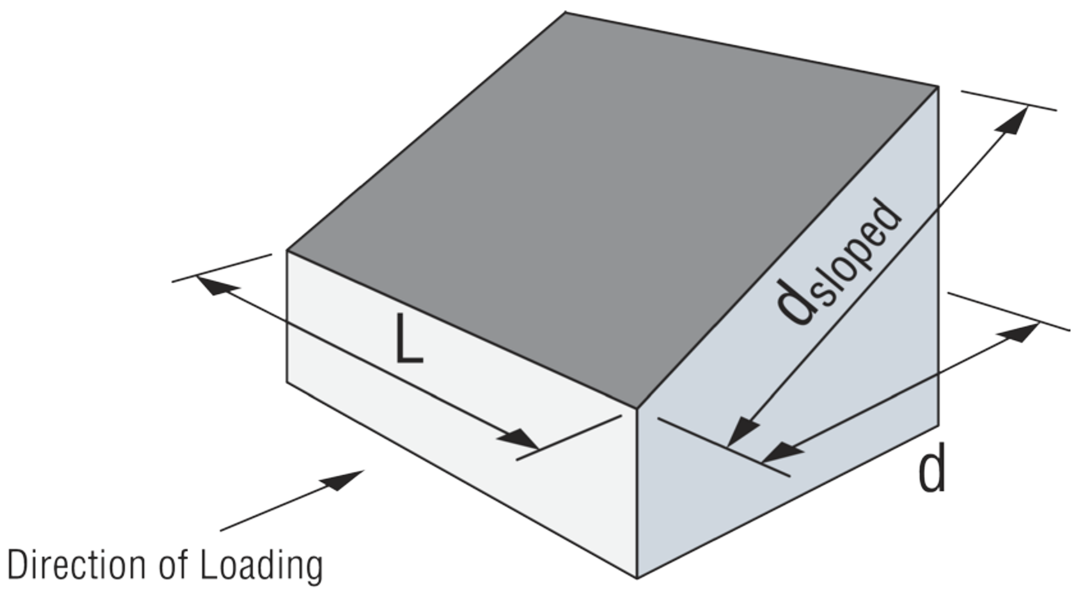

Figure 2.3 - Monoslope Roof

Where:

d	=	Diaphragm depth (parallel to force)
L	=	Diaphragm length(perpendicular to force)

Using the geometry of the roof, a "slope factor", α can be derived:

$$\frac{d_{sloped}}{d} = \frac{d_{sloped}}{d_{sloped}(\cos\phi)} = \frac{1}{\cos\phi} = \alpha$$

The relationship of the force in the sloped diaphragm and the length of the sloped diaphragm can be related to the horizontal force and dimension as a factor of the roof slope angle.

$$(F_{sloped})\cos\phi = F_x$$

$$(d_{sloped})\cos\phi = d$$

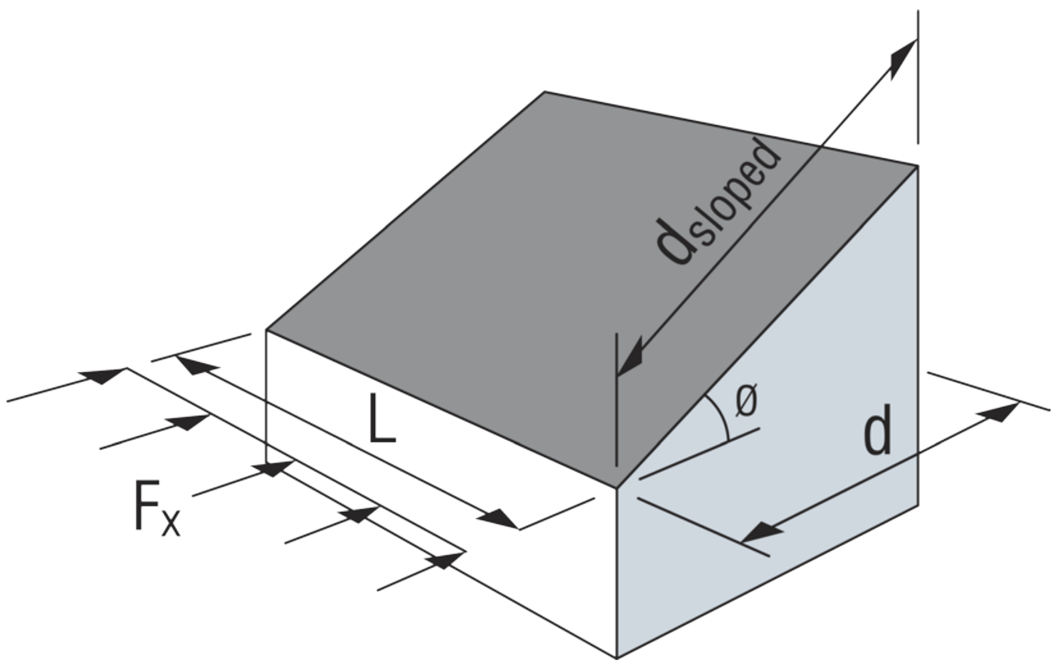

Figure 2.4 - Monoslope Diaphragm Transversely Loaded

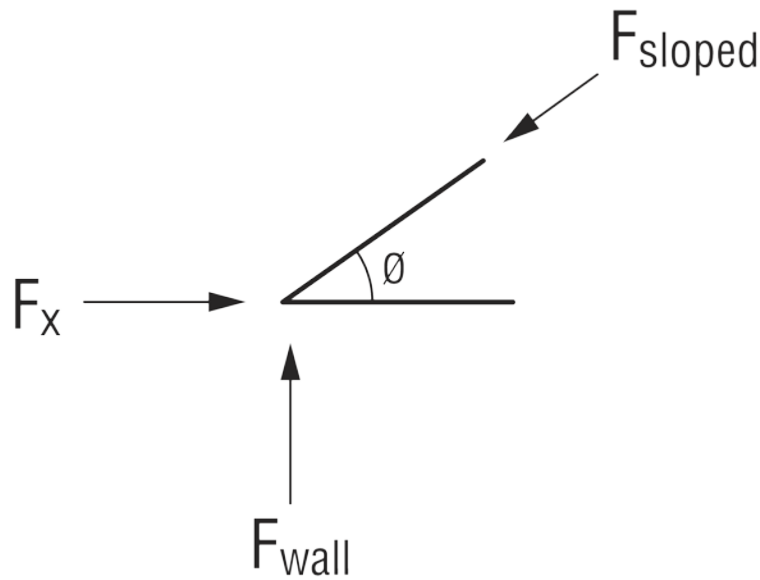

Figure 2.5 - Resolution of Forces

Using this geometrical relationship it can be seen that the force in the sloped diaphragm per unit length (pounds per linear foot) is the same as if the roof were flat.

$$v_{diaphragm-sloped} = \frac{F_{sloped}}{d_{sloped}} = \frac{\frac{F_x}{\cos\phi}}{\frac{d}{\cos\phi}} = \frac{F_x(\alpha)}{d(\alpha)} = \frac{F_x}{d} = v_{diaphragm-flat}$$

This understanding greatly simplifies the analysis of sloped roof diaphragms.

Section 2.2.1.1 Longitudinally Loaded Monoslope Roof Diaphragms

Next consider a shed roofed building loaded by wind in the longitudinal direction.

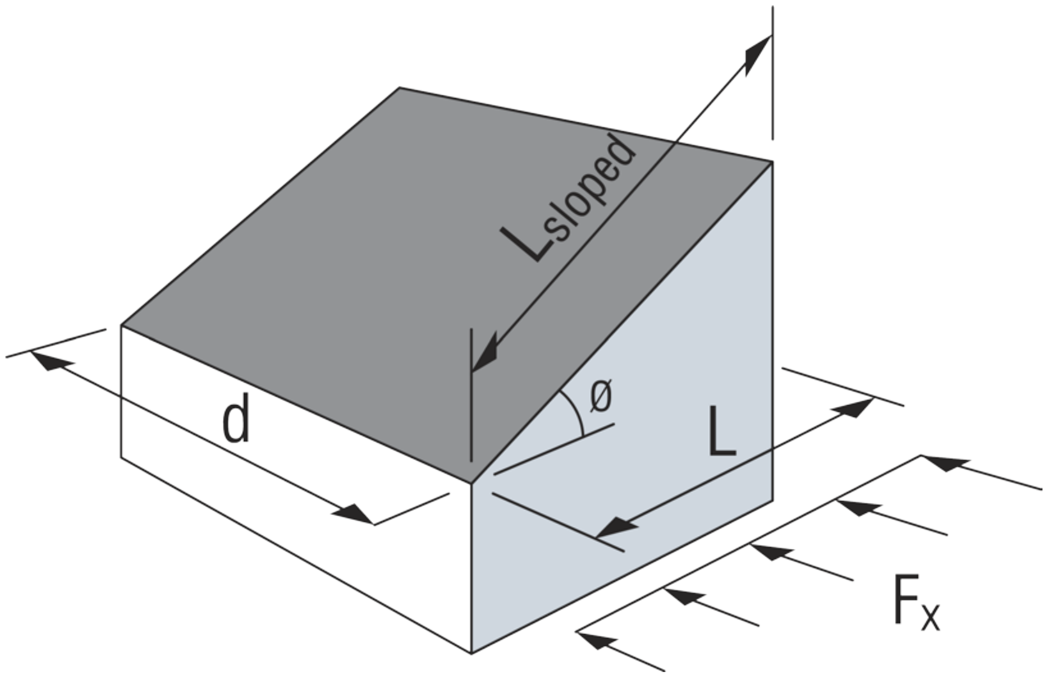

Figure 2.6 - Monoslope Diaphragm Transversely Loaded

Again, looking at the geometry:

$$L_{sloped}(\cos\phi) = L$$

$$F_{sloped} = F_x(\cos\phi)$$

$$v_{flat} = \frac{L(F_x/ft)}{d}$$

$$v_{sloped} = \frac{L_{sloped}(f_{sloped}/ft)}{d} = \frac{F_x(\cos\phi/ft)(\frac{L}{\cos\phi})}{d} = \frac{F_x/ft\,(L)}{d} = v_{flat}$$

It can once again be seen that the force in the sloped diaphragm per unit length (pounds per linear foot) is the same as if the roof were flat.

So, we can see that the diaphragm shear for a sloped roof is equal to the diaphragm shear based on the plan dimension (or flat roof dimension).

Section 2.2.1.3 Lateral Loads on Roof Plane

Wind forces act normal to the roof plane, and can be resolved into forces in the primary axes.

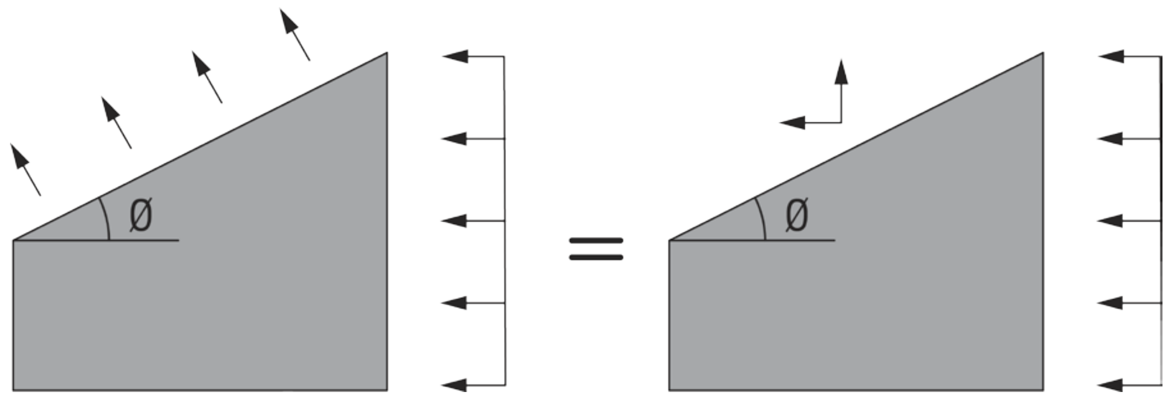

Figure 2.7 - Lateral Loads on Roof Plane

sloped roof: $q_{normal}(A) = q_{normal}(L)(d_{sloped}) = \dfrac{q_{normal}(L)(d)}{\cos\phi} = \alpha q_{normal}(L)(d)$

flat roof: $q_{normal}(A) = q_{normal}(L)(d)$

Where:

A = Roof Area

We can see that the relationship of wind uplift load on the sloped diaphragm can be compared to the load on a flat roof by the "slope factor", α.

Section 2.2.2 Lateral Loads - Seismic

Looking at a shed roof diaphragm resisting seismic loads transversely:

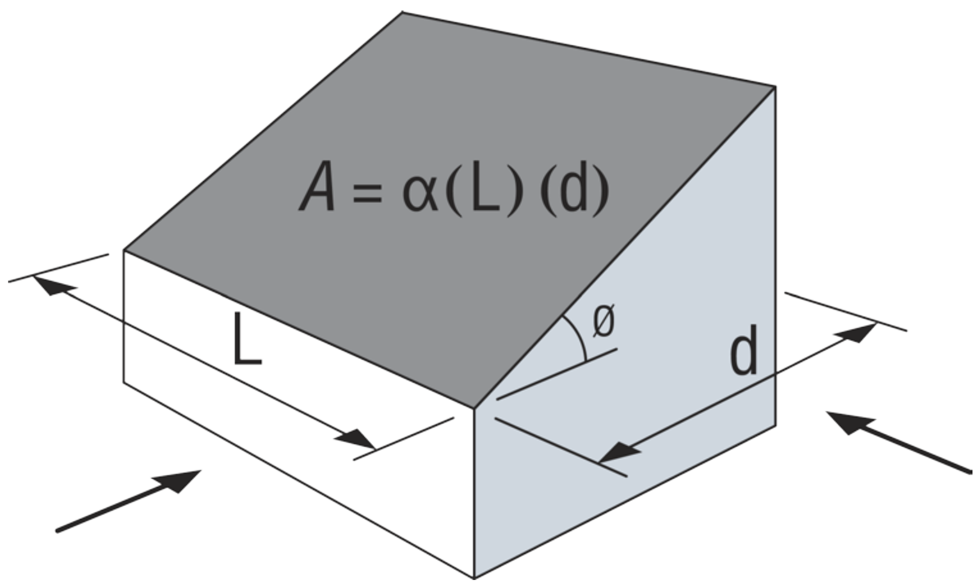

Figure 2.8 - Seismically Loaded Diaphragm

Total Diaphragm Weight =

sloped diaphragm: $w_{diaphragm}(A) = w_{diaphragm}(L)(d_{sloped}) = \alpha w_{diaphragm}(L)(d)$

flat diaphragm: $w_{diaphragm}(L)(d)$

Where:

$w_{diaphragm}$ = weight / unit area

Therefore, lateral loads associated with the sloped diaphragm are increased by the slope factor of:

$$\frac{1}{\cos\phi} = \alpha$$

Section 2.2.3 Chords

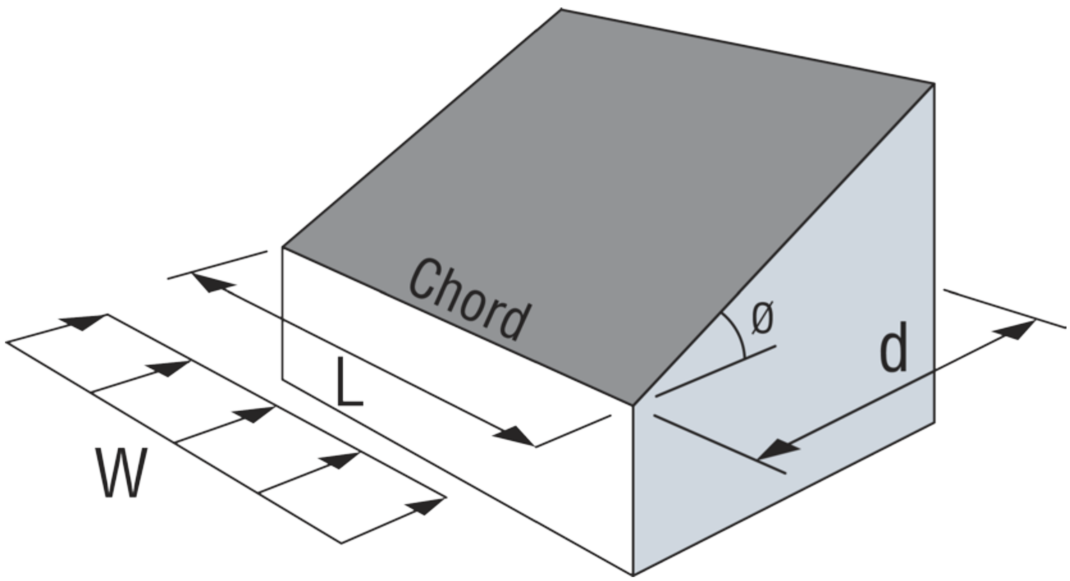

Figure 2.9 - Diaphragm Chord

For a transversely loaded roof, the forces in the diaphragm chords can be shown to be equal to the chord forces in a flat roof.

$$w_{sloped}(\cos\phi) = w$$

$$d_{sloped}(\cos\phi) = d$$

$$P_{chord} = \frac{wL^2}{8d}$$ for a flat roof

$$P_{chord-sloped} = \frac{w_{sloped}L^2}{8d_{sloped}} = \frac{\alpha wL^2}{8\alpha d} = \frac{wL^2}{8d}$$ for a sloped roof

Section 2.2.4 Collectors

For a roof diaphragm loaded transversely, and the collectors on the transverse walls, the following relationship for the collector forces is found.

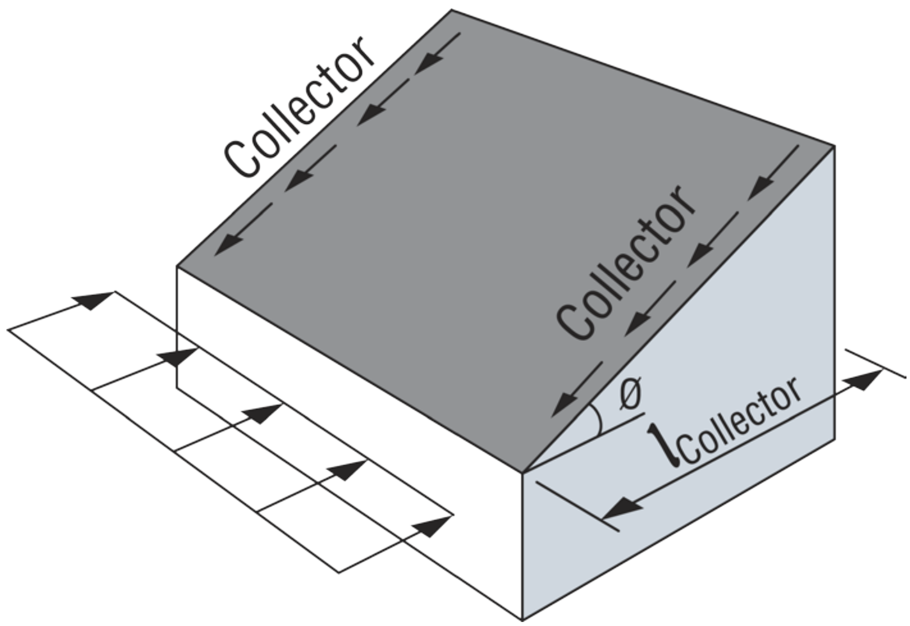

Figure 2.10 - Diaphragm Transverse Collector

$$P_{collector} = v(l_{collector})$$

$$P_{slopedcollector} = v(l_{collectorsloped}) = \frac{v(l_{collector})}{\cos\theta} = \alpha v(l_{collector})$$

For a roof diaphragm loaded longitudinally, and the collectors on the longitudinal walls, the following relationship for the collector forces is found.

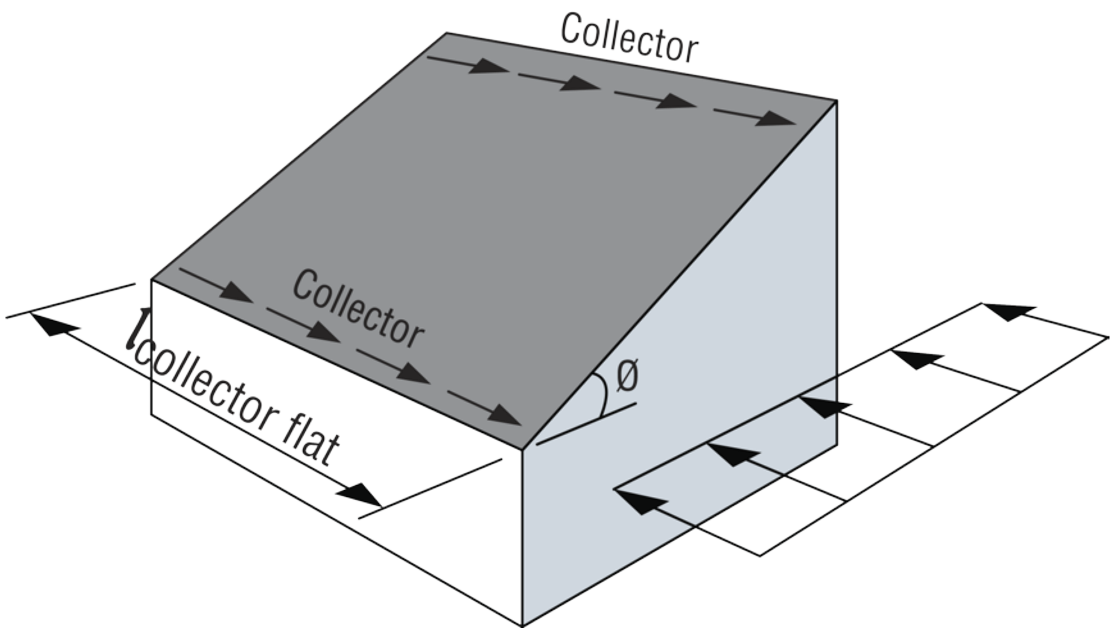

Figure 2.11 - Diaphragm Longitudinal Collector

$$v_{flat} = v_{sloped}$$

$$P_{collectorflat} = v_{flat}(l_{sloped}) = v_{sloped}(l_{sloped}) = P_{collectorsloped}$$

Section 2.3 Gable End Wall Loading of Diaphragms

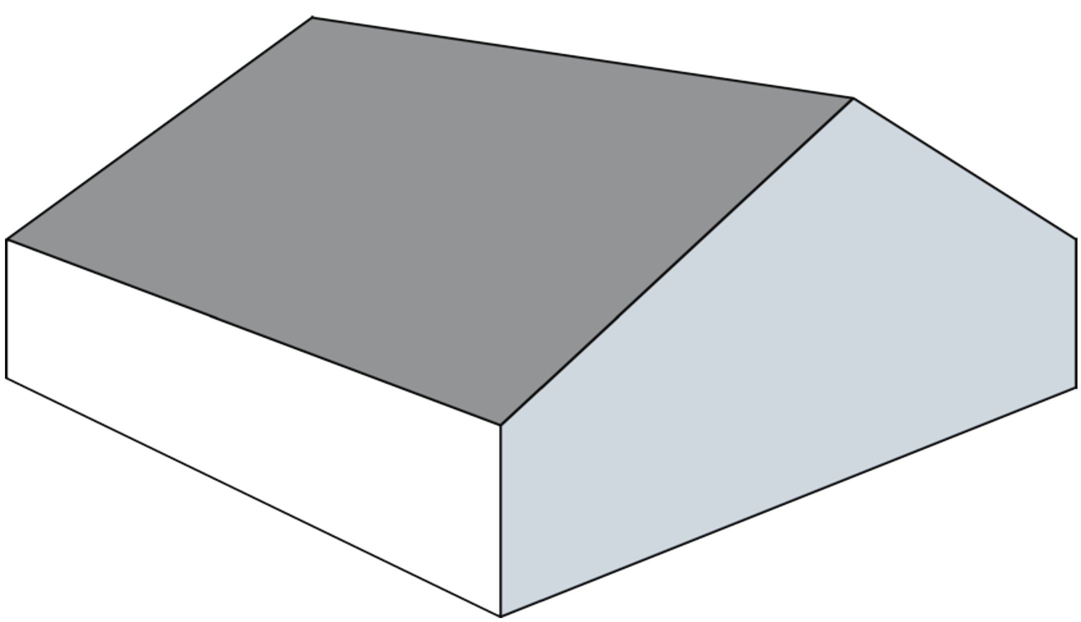

Figure 2.12 - Gable End Wall

Refer to Example 7 for different methods of framing the endwall and how the forces are transferred into the roof diaphragm.

Section 2.4 Wind Loaded Diaphragm Strength

Roof diaphragms that are loaded by wind must resist both in-plane shear and uplift. There are two loading conditions that must be considered:

A. The roof deck is designed as the sheathing for the roof. For this case, the wind uplift on the deck and fasteners is calculated using component and cladding (C&C) wind pressures.

B. The roof deck is designed as a diaphragm. For this case, the wind uplift and in-plane diaphragm shear are both calculated using main wind force resisting system (MWFRS) wind pressures.

Case A is a straightforward check of the deck flexural capacity and the fastener tensile strength, which may be controlled by either fastener pull-out from the supporting framing or pull-over of the deck over the fastener head.

Case B requires that the interaction of tension and shear on the fastener be considered. The tension on the fastener reduces the available shear capacity of that fastener, thereby reducing the available diaphragm strength from the strength without wind uplift.

Equations for shear and uplift interaction on screws are found in Section D3.1.2 of AISI S310. Separate checks must be run for both interaction of shear and pull-over and shear and pull-out. Because the interaction equations include Ω in the ASD equation and Φ in the LRFD equation, the final resulting nominal diaphragm capacity will be slightly different for each method.

Shear and uplift interaction can be calculated by hand, however for design purposes computer assisted methods are the most practical. The SDI Diaphragm Interaction Calculator is available from the SDI website for this purpose. Additionally, software solutions are available from some SDI member and associate member companies.

A simplified method for calculating diaphragm resistance in the presence of uplift is shown in the following equation. This equation may be acceptable for preliminary design, however the user must be aware that it may produce results that may be un-conservative by 10% or more and the result of this equation must be checked against the AISI interaction equations, either by hand check or computer software.

$$S_{n\ wind} = S_{n\ no\ wind} - \left(1 - \frac{P_{nft}}{P_{nf}}\right) S_{n\ no\ side}$$

Where:

$S_{n\ wind}$	=	Diaphragm nominal shear resistance when reduced by wind uplift
$S_{n\ no\ wind}$	=	Diaphragm nominal shear resistance unreduced by wind uplift (from diaphragm table) with correct number of sidelap fasteners
$S_{n\ no\ side}$	=	Diaphragm nominal shear resistance unreduced by wind uplift (from diaphragm table) with zero sidelap fasteners
P_{nf}	=	Nominal shear resistance of support connection per fastener, without uplift
P_{nft}	=	Nominal shear resistance of support connection per fastener, in presence of wind uplift

Section 2.5 Roof Framing Details

Section 2.5.1 Step-Down Hip Roof Framing

Figure 2.13 shows a typical step-down hip framing plan. Several conditions need to be considered when designing and detailing steel deck for this situation.

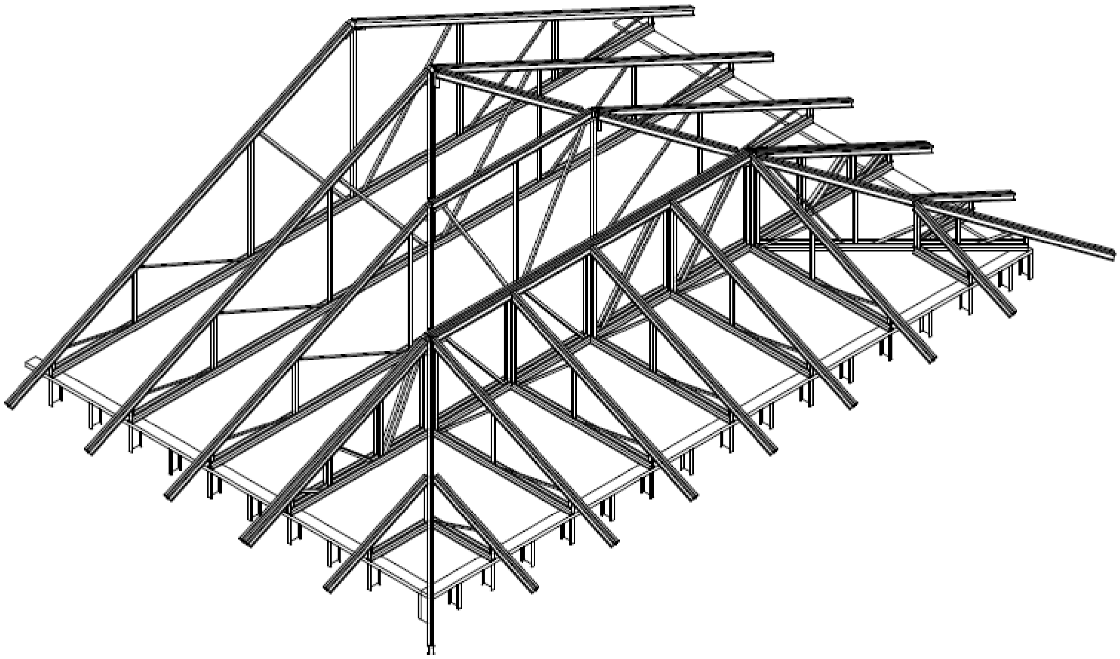

Figure 2.13 - Step-Down Hip Framing (Courtesy of Alpine Trussteel)

A. The hip lines are created by framing that has the top chord level in the line of the hip slope. Therefore, the deck is provided a "knife-edge" bearing that can cause web crippling of the deck, but in all cases does not provide a lat and level surface for the deck to be attached to using screws. The designer should design and specify additional deck support material that is field fastened to the corner hip truss top chord, as illustrated in Figure 2.14. In this case, the support material is sheet bent into a V shape with a top being flat. The deck is attached to the "legs" with screws to transfer the uplift, and the flat section is attached to the top of the framing member with screws, also to transfer the uplift. The ridge plate on top of the deck is designed to transfer the diaphragm forces across the break in the deck.

It is NOT the responsibility of the deck supplier to provide this additional bearing material unless specifically addressed in the contract.

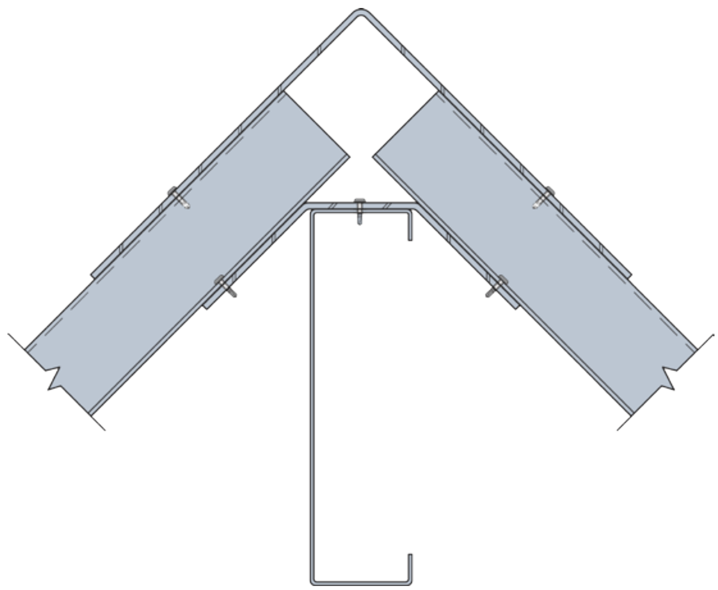

Figure 2.14 - Supplemental Deck Bearing Material on Hip or Ridge

Likewise, as shown in Figure 2.15, a similar detail can be used in a valley for the same purpose.

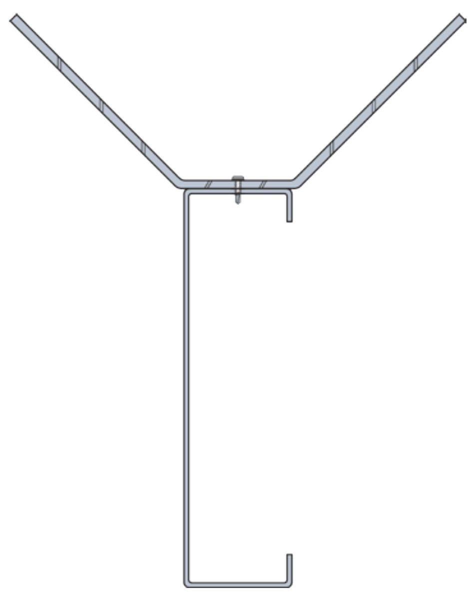

Figure 2.15 - Supplemental Deck Bearing Material on Valley
(Deck and Valley Plate Not Shown for Clarity)

B. The #1 hip truss carries the corner hip and the end jacks. Proceeding further up the roof slope are the step-down trusses. Figure 2.16 shows the hip line being framed with supplemental framing between the trusses and forming the hip line. This supplemental framing is required to support the ends of the deck that would otherwise be cantilevering from the last truss support. Supplemental deck support material must be provided on top of the supplemental framing. It is NOT the responsibility of the deck supplier to provide this additional bearing material or support framing, unless specifically addressed in the contract.

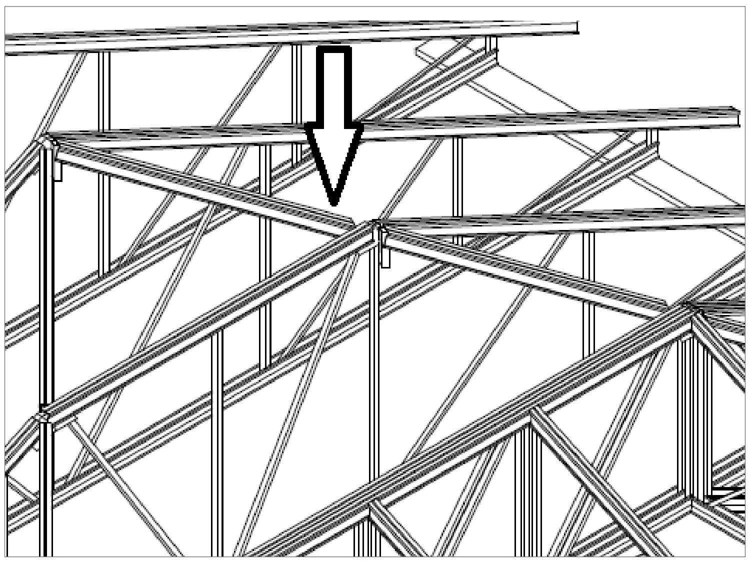

Figure 2.16 - Supplemental Framing at Hip Line

The supplemental framing can be any structural member that is capable of carrying the gravity and uplift reactions from the deck. Sometimes a truss supplier will provide a truss top chord section or a cee section that is fastened using clips to the adjacent trusses. A bent valley or ridge plate, by itself, does not possess sufficient strength to span over 2 feet, particularly when the roof slope is low and the angle of the bends in the plate are small.

Another option is to span across the open valley or hip using cold-formed zee sections, of a depth to match the deck being supported. Figure 2.17 shows 1-1/2" deep zee sections which support the deck, and a ridge plate to carry the diaphragm forces across the ridge.

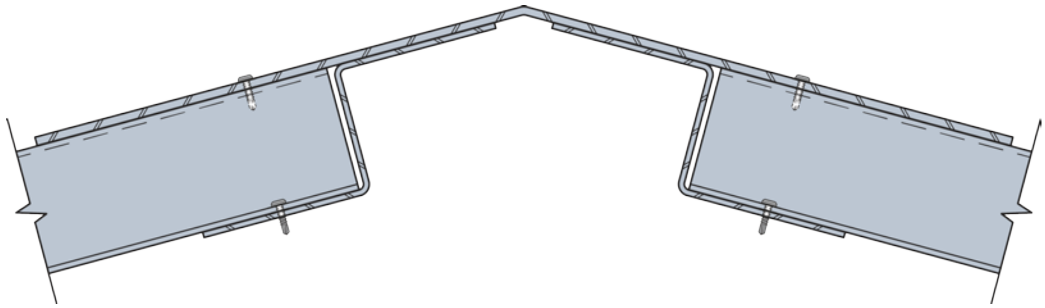

Figure 2.17 - Spanning Zees at Ridge or Hip to Support Deck

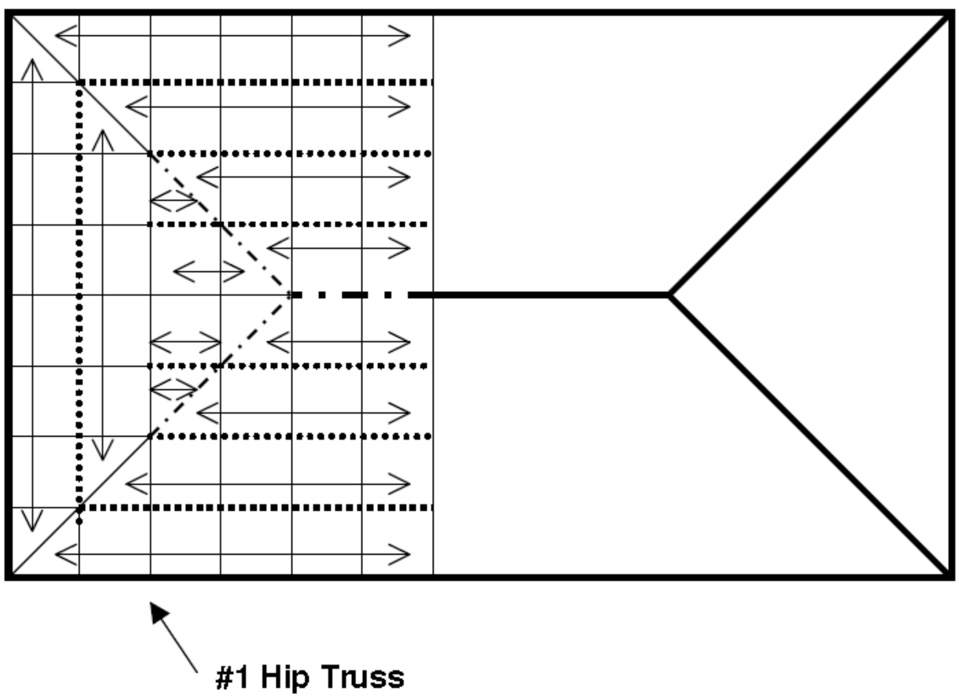

Figure 2.18 - Deck Span Change at #1 Hip Truss

Section 2.5.2 Valley Set Roof Framing

Figure 2.19 shows a typical truss valley set framing plan, where the secondary roof slope is created by truss overframing. Similar to the step-down hip, the valley (at the intersection of the gray area with the white area) is not supported by trusses. Supplemental framing and deck bearing material must be provided in the valleys. It is NOT the responsibility of the deck supplier to provide this additional bearing material or support framing, unless specifically addressed in the contract.

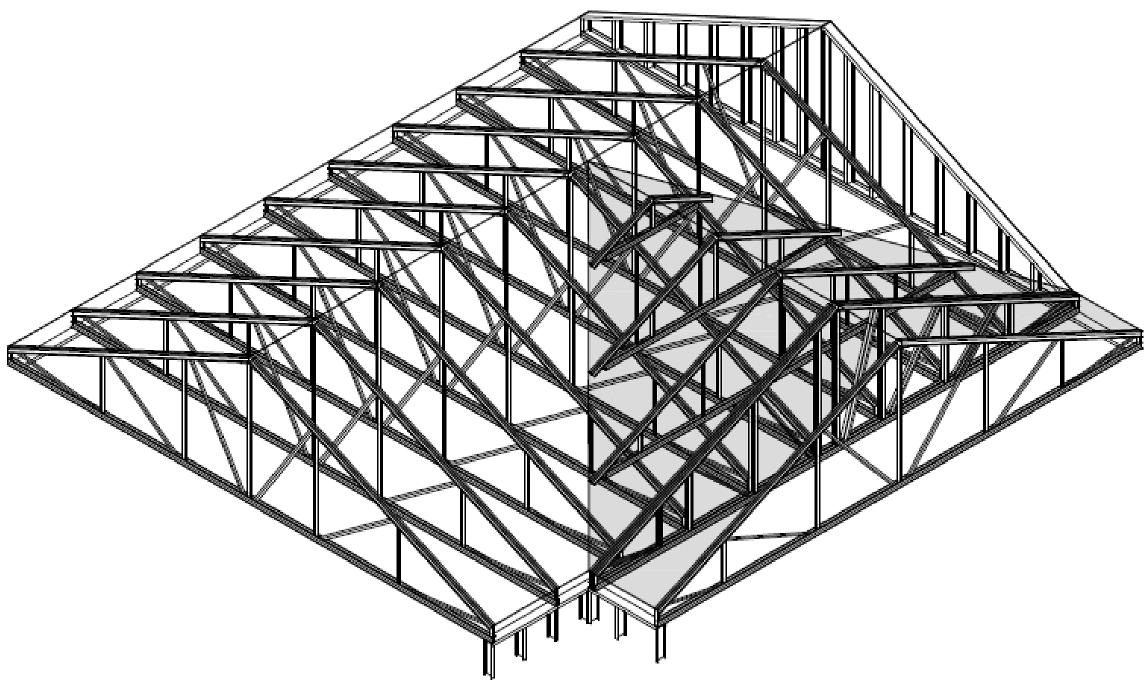

Figure 2.19 - Valley Set Truss Framing (Courtesy of Alpine Trussteel)

Valley set framing is commonly installed directly on top of the supporting trusses, as shown in Figure 2.20. This configuration allows for a direct load transfer between the supporting and valley set trusses using hardware provided by the truss manufacturer.

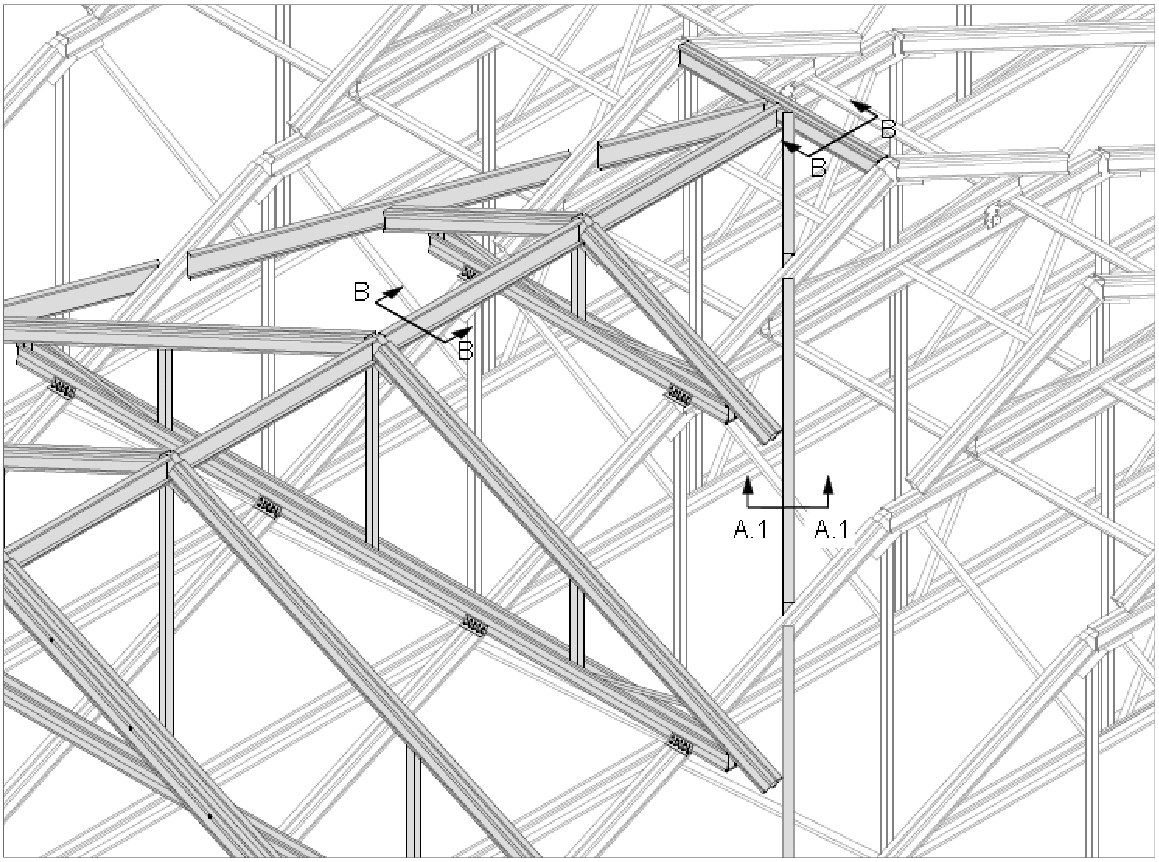

Figure 2.20 - Valley Set Truss Framing Directly Atop Supporting Trusses

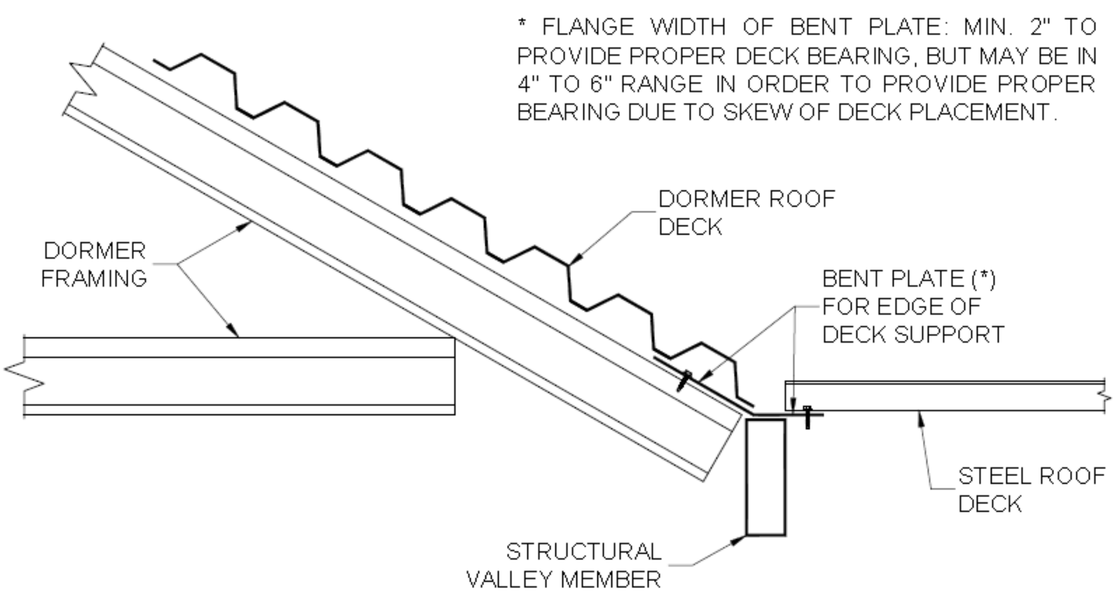

Figure 2.21 - Section A.1 - No Deck Under Dormer Framing

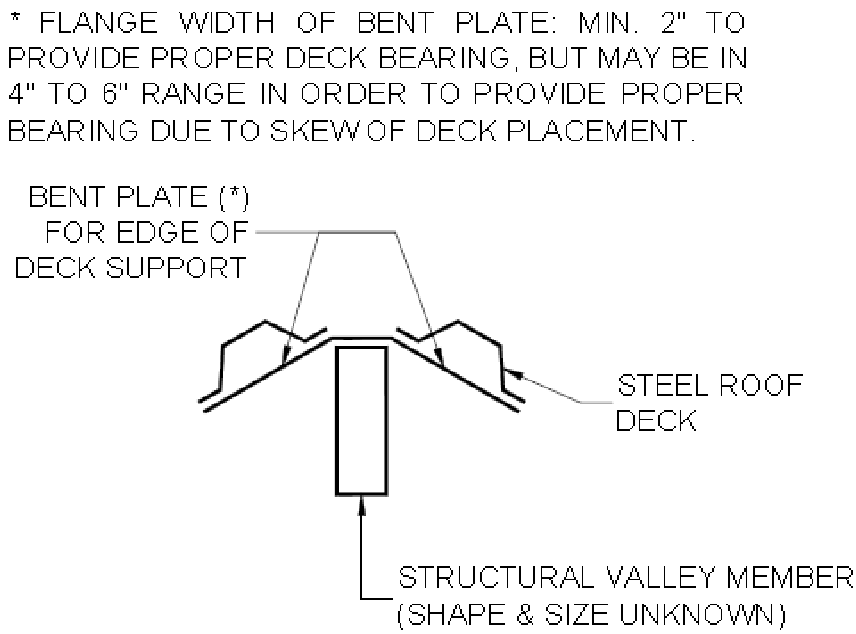

Figure 2.22 - Section B - Ridge Framing

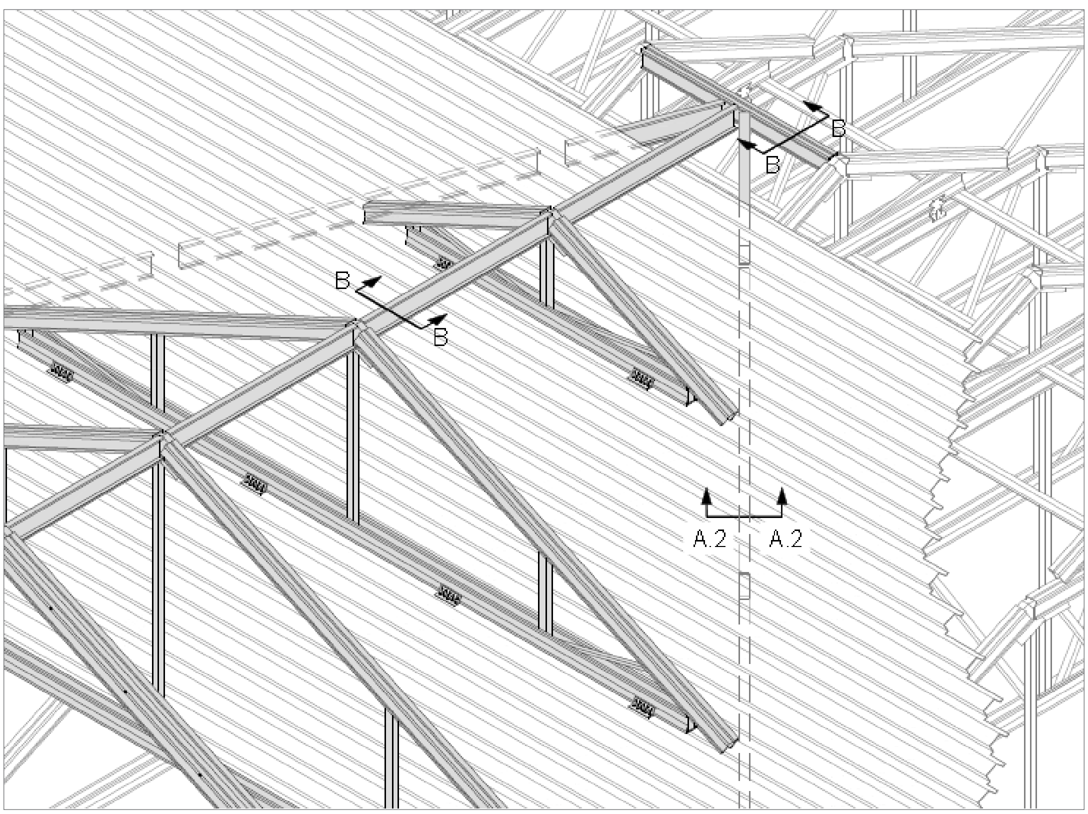

Figure 2.23 - Valley Set Truss Framing Above Roof Deck

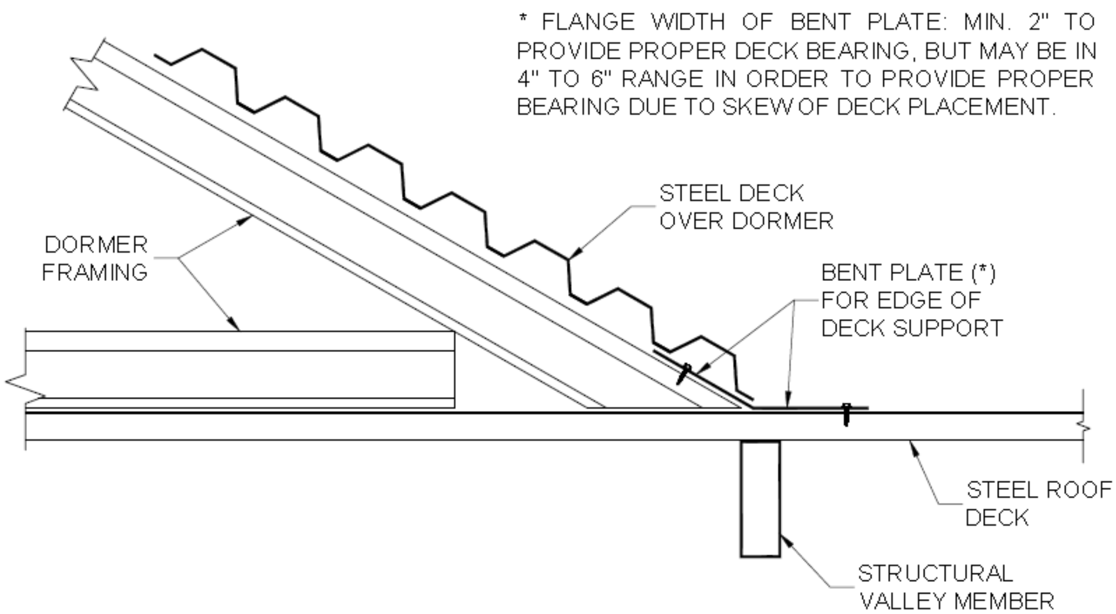

Figure 2.24 - Section A.2 - Deck Continuous Under Dormer Framing

As shown in Figure 2.23, sometimes the steel deck is run beneath the valley set trusses. There are pros and cons to this detail.

1. Pros:
   a. The diaphragm is continuous under the valley set.
2. Cons:
   a. The valley set trusses are bearing on the deck rather than directly on the primary trusses below. Therefore, the load path for both gravity and uplift must pass through the deck. The deck is loaded along a single rib, and the valley truss may not line up over a high rib. Very careful detailing by the designer is required to accomplish this and this detailing is NOT the responsibility of the deck supplier.
   b. The edges of the deck that are supported by the overframed trusses must be detailed to connect to the deck under the valley sets. A bent plate is shown in Figure 2.24 for this purpose. The detailing of this connection is not the responsibility of the deck supplier.

Section 2.6 Roof Diaphragm Details

Whether a steel deck diaphragm is supported by cold-formed trusses or rafters, rolled steel beams, or open web steel joists, the forces at the boundaries of the diaphragm must be transferred out of the diaphragm and into the boundary member.

At the heel of a truss or rafter, the diaphragm force comes out at the top chord of the truss, which is elevated above the wall top plate, which is often designed as the collector or drag strut. The detail shown in Figures 2.25 and 2.26 is one possible way of using a brake formed shape as blocking between trusses to transfer this force. This manual will not provide specific design guidance and the user is referred to one of several framing design guides.

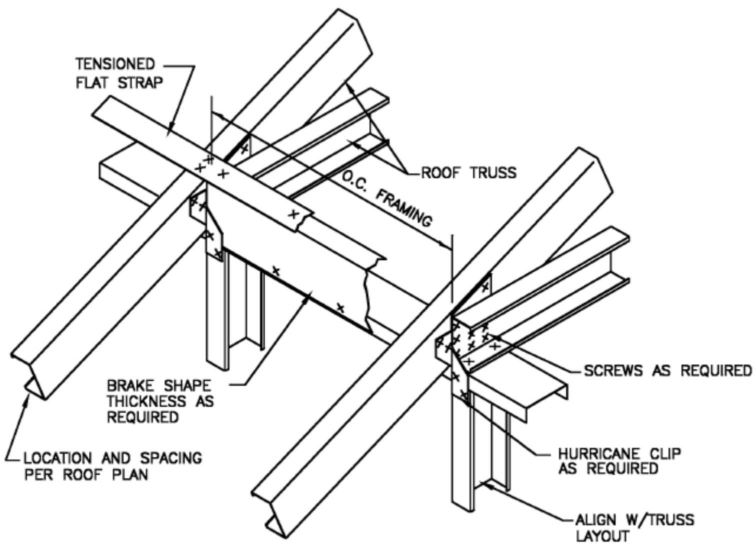

Figure 2.25 - Shear Transfer Blocking (Courtesy of CFSEI)

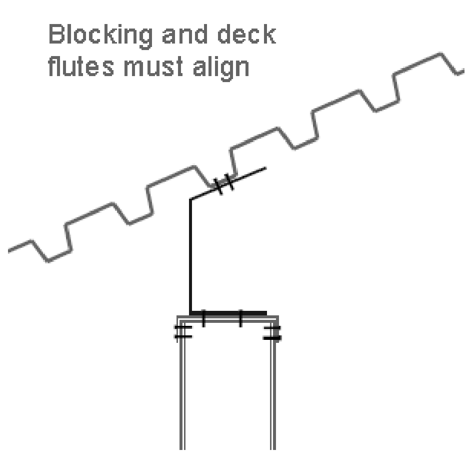

Figure 2.26 - Shear Transfer Blocking

Section 2.7 Hip Roofs

The equivalent flat roof can be used to design most buildings with sloped hip roofs. This equivalent flat roof model determines the true diaphragm shear per unit length. The field of the roof can be rapidly designed using either diaphragm load tables or the SDI Diaphragm Interaction Calculator. The latter is required where shear is caused by a wind event.

As always, special attention must be given to perimeter conditions, points of discontinuity such as butt joints, or fold lines such as hips or valleys. This concept is not new and one eave condition is addressed in AISI S310 (Appendix 2). Items requiring evaluation are:

1. Deck orientation.
2. Axial forces in the deck and the associated support connections.
   These are not the flat roof equivalent forces but are the “in plane” forces. Axial forces and diaphragm shear typically stay in the deck until a point of discontinuity is reached.
3. Spacing of fasteners.
   Skew cuts increase the flute spacing along the hip or valley line relative to the spacing at typical supports. A “rule of thumb” is to double the number of fasteners per sheet, e.g. fastener in every other flute becomes fastener in each flute. The hip member must be wide enough to allow this.
4. Design of the hip members.
   These receive and transmit all loads applied by the diaphragm and/ or during construction.
5. Static Equilibrium.
   This applies at skew cuts and perimeter conditions where the diaphragm shear per unit length along the cut or end equals that along the deck.

FLOOR DECK

SECTION 3

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Section 3.1 Floor Deck Profiles and Finishes

Any of the floor deck profiles shown in the SDI Floor Deck Design Manual (FDDM) can be used on cold-formed steel floor trusses or joists. Engineering information, including section properties and span tables, can be found in the FDDM and in manufacturers literature. For closely spaced trusses or joists (48 inches or less on center), form deck of 9/16 inch to 1 inch in depth (Figure 2.1) can also be used. For longer spans, deeper form deck or composite deck can be used.

Form deck is most often galvanized, and if galvanized, it is permitted to be considered to be a permanent form that supports the weight of the concrete slab, with the concrete slab designed to support the weight of the superimposed loads. Alternately, it is allowed to use bare or painted deck as permitted by the applicable SDI Specification (SDI C or SDI NC).

Figure 2.1 - Form Deck for Floor Applications

Section 3.2 Structural Design of Form Deck

The SDI NC Standard permits several different options for the structural design of floors using non-composite or form deck, any of which are viable design options.

A. Galvanized form deck as a permanent form: Galvanized deck is permitted (but not required) to be considered to be a permanent form that will continue to carry the weight of the deck and concrete. Therefore, the structural concrete slab placed on the deck may be designed to carry only the loads superimposed on the floor after the concrete is cured.

B. Form deck as a non-permanent form: Any form deck finish (galvanized, painted, or bare) can be considered to be a non-permanent form. Painted or bare finish deck must be considered as a non-permanent form. In this case, the structural concrete slab must be designed to carry all loads, including the slab and deck.

C. Form deck as the structural floor: A galvanized form deck can be designed to carry all dead and live loads without a structural concrete slab. Instead, a non-structural wearing surface can be placed on the deck, such as a non-structural concrete slab, a gypsum concrete topping, or wood, concrete, or gypsum board products. One potential advantage of this method is that it may be possible to continue construction and place the next level of bearing walls before the topping is placed.

Section 3.3 Structural Design of Composite Deck

Design of composite deck on cold-formed steel trusses or joists is no different than on open web steel joists or rolled beams, however, several somewhat unique conditions need to be considered.

A. Practical spacing of cold-formed floor framing is typically narrower than rolled beams or open web steel joists, therefore composite deck may not be the most economical option. Assistance in determining economical selection of deck can be obtained from deck manufacturers.

B. Many composite deck profiles cannot be end-lapped and instead must be butted. Consideration of the width of the framing and the required bearing length of the deck may not allow the use of some profiles.

Deep composite floor deck has also been used to span between bearing walls. This system is sometimes used in buildings where there are repetitive bearing walls, such as hotels, dormitories, and apartments. Consideration needs to be given to the design of the top member of the bearing wall for bending and shear between the studs. The design of the top of wall member is outside the scope of this Manual, and the user is referred to one of several cold-formed steel framing design resources.

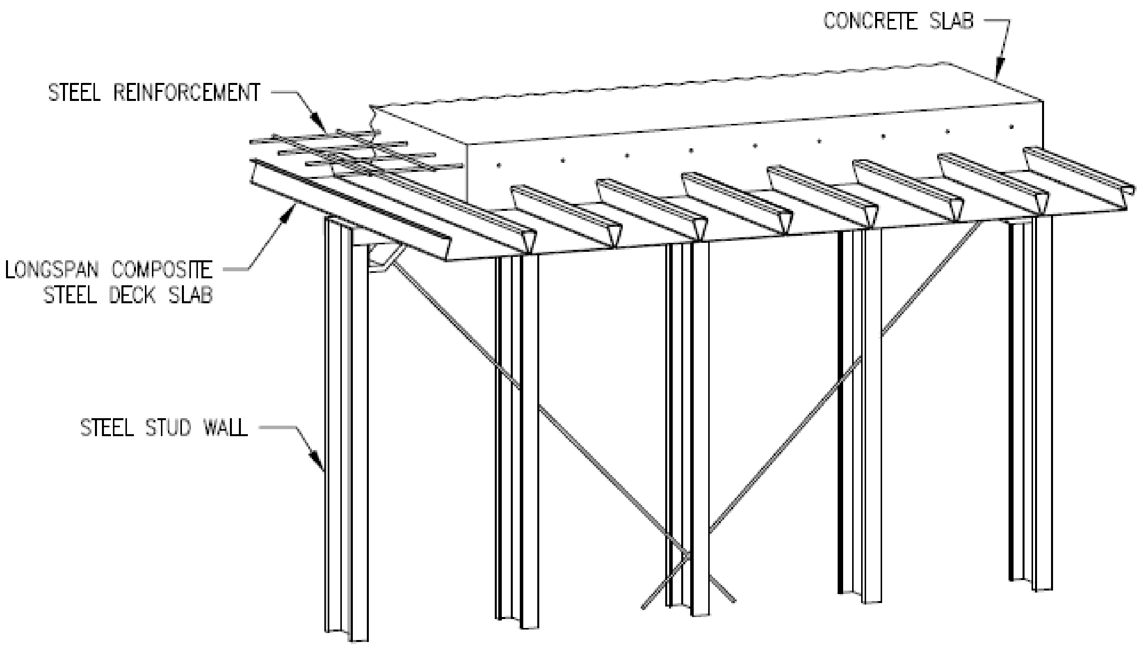

Figure 3.2 - Composite Deck on Bearing Walls

Section 3.4 Structural Design of Floor Diaphragms

The AISI S310 Standard calculates the diaphragm shear strength as the sum of the contributions of the fasteners and the concrete. For the interior areas of a diaphragm, the shear strength takes the form:

$$S_n = \frac{\beta P_{nf}}{L} + kbd_c\sqrt{f'_c} \qquad \text{(Eqn. 3.1)}$$

AISI S310 further limits the contribution of the connectors to not be greater than 25% of the total diaphragm strength.

$$\frac{\beta P_{nf}}{L} \leq 0.25S_n \qquad \text{(Eqn. 3.2)}$$

Where:

β	=	$n_s\alpha_s + \frac{1}{w^2}\left(2n_p\sum\left(x_p^2\right) + 4\sum\left(x_e^2\right)\right)$
P_{nf}	=	structural connector strength
L	=	panel length
k	=	concrete strength factor
b	=	unit width
d_c	=	concrete cover depth
f'_c	=	specified compressive strength of concrete

This formulation lends itself to several different possible methods of determining the floor diaphragm strength.

A. Diaphragm Strength Calculated Only From the Fastener Contribution

When a floor diaphragm has a structural or non-structural concrete fill, this method is permitted and will lead to a conservative result. Only the first term in Equation 3.1 is used and the second term is set to zero. When a floor has a fill other than concrete or a board covering, unless a tested design is available, this is the only method that is permitted. Diaphragm tables using this method are included in this Manual.

B. Diaphragm Strength Calculated Only From the Concrete Contribution

When a floor diaphragm has a structural or non-structural concrete fill, this method is permitted and will lead to a slightly conservative result. Only the second term in Equation 3.1 is used and the first term is set to zero. Diaphragm tables using this method are included in this Manual.

Because the contribution of the connectors in the field is neglected, specific attention has to be paid to the load transfer of forces into and out of the diaphragm. This will require specific design of the fasteners of the deck at these locations.

C. Diaphragm Strength Calculated Using Contribution of Both Fasteners and Concrete

When a floor diaphragm has a structural or non-structural concrete fill, this method is permitted. Both terms in Equation 3.1 are used. This Manual does not have tables for direct design using this method, but the strengths from the tables for diaphragms with and without fill can be added together, limiting the contribution of the fasteners to not more than 25% of the total diaphragm strength (Equation 3.2).

Section 3.4 General Design and Detailing Considerations

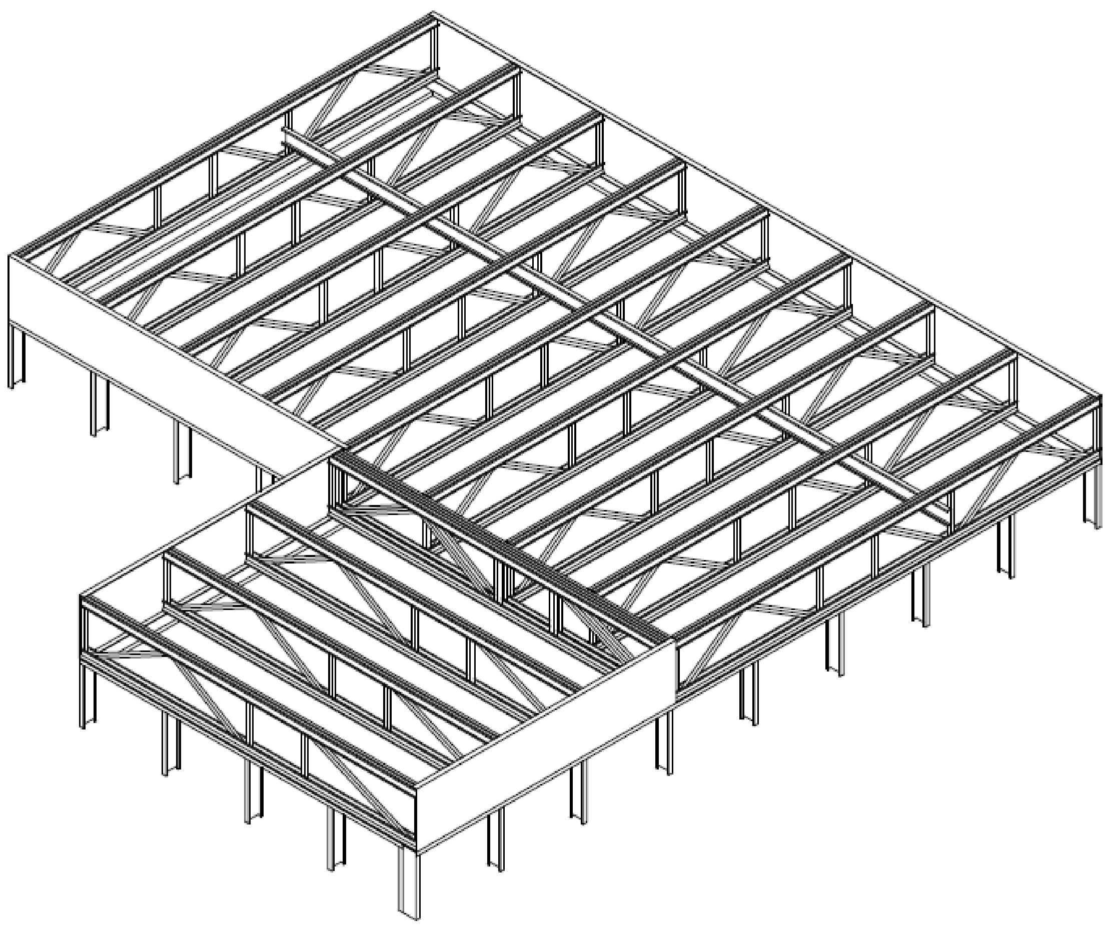

Figure 3.3 - Cold-Formed Truss Framed Floor

There are some design considerations that are common to both composite and non-composite (form) deck used for floors.

A. Available flat bearing width of the framing top flange may not permit the use of butted deck ends once the required deck bearing length (minimum of 3/4 inch per AISI S100) is taken into account. Also, minimum fastener end distance in the deck (1.5 times the screw diameter per AISI S100) may not be able to be met (for instance, minimum end distance for a #12 framing screw would be 0.324 inches or more than 5/16 inch). Deck panel underlength within SDI Standards tolerance of 1/2 inch per deck sheet and tolerance for framing placement and straightness needs to also be factored into this consideration.

B. Minimum fastener spacing (3 times the screw diameter per AISI S100) may control the number of fasteners per deck rib. For instance, the minimum center to center spacing for a #12 support screw is 0.648 or over 5/8 inches. This, combined with end distance limits, will most likely limit the number of screws to two per deck rib for the most common framing flange widths.

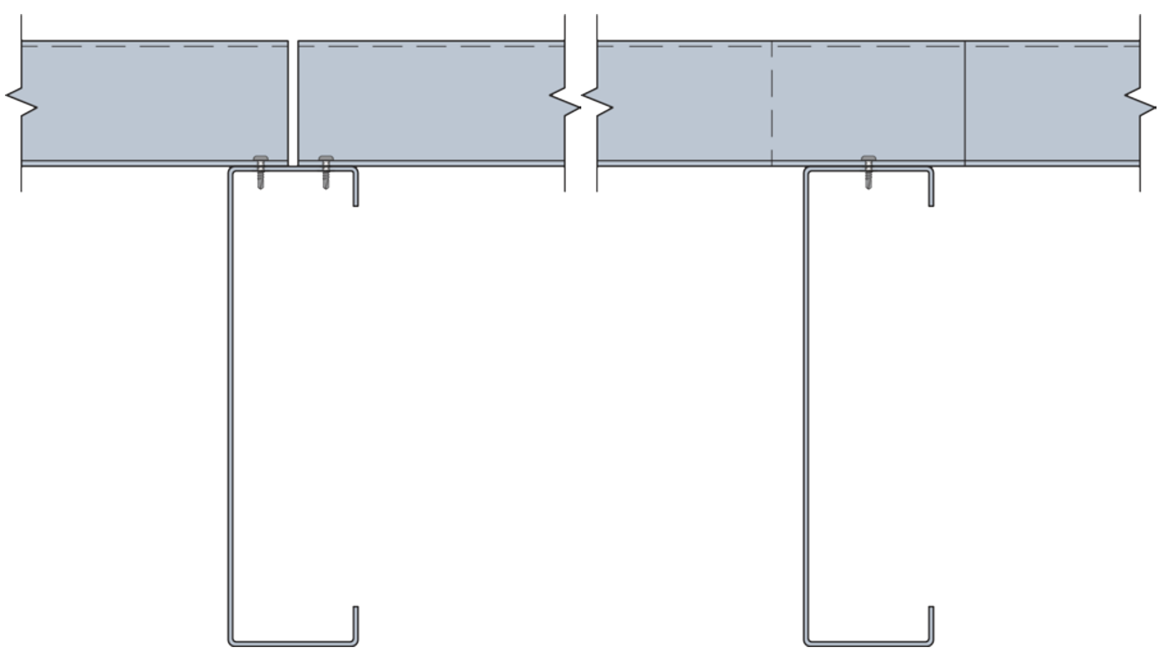

Figure 3.4 - Butted and Lapped Deck Ends

FASTENERS

SECTION 4

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Section 4.1 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 by 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 of the specifying professional and the type and number of fasteners should not be deferred to the deck supplier or 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.

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.
>
> Edge Fastener: A fastener used to transfer shear forces into collectors.

Section 4.2 Screw Fasteners

Screw fasteners are the recommended fastener for structural and side-lap connections of steel decks to cold-formed steel framing, and work well for attachment of most nestable profiles and types.

Common side-lap screw sizes include No. 10 and 12 self-drilling screws, although smaller No. 8 screws for lighter cold-formed steel support fastening applications are occasionally used. Screws with diameters of No. 12, and occasionally No. 14, or 1/4 inch are generally used for steel deck roof attachment to cold-formed steel structural supports. These structural screws can resist both tension and shear and the interaction of tension and shear.

Self-drilling screw points drill through the steel deck panels and support steel prior to threads engaging. For deck on cold-formed steel applications, typical screw drill points range from No. 1-5, but stitch, pilot point and self-piercing screws are also available. See Figure 4.1 below for some typical self-drilling tapping screw profiles.

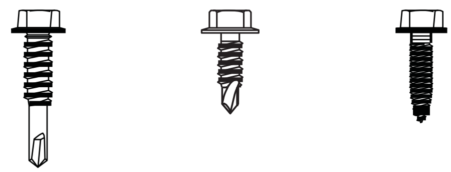

Figure 4.1 – Typical Screws

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. The screws with pre-mounted washers may also be used for exterior exposed roof deck applications, subject to local jurisdiction or code requirements.

AISI S100 provides design provisions for screw connections and performance in shear bearing, 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. AISI S310 uses the AISI S100 equations for diaphragm strength calculation, but permits the use of alternate fastener strength equations.

The SDI Diaphragm Design Manual (Section 6.5) provides alternate design equations for support and side-lap fasteners that can be used for calculating diaphragm capacities. While not used in this Manual, these equations are valid within the listed limits.

Ergonomic, stand-up screw fastening systems exist on the market today that may increase installation efficiency. Additionally, some manufacturers have collated screws available for 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 4.3 for illustrations of screw side-lap connections.

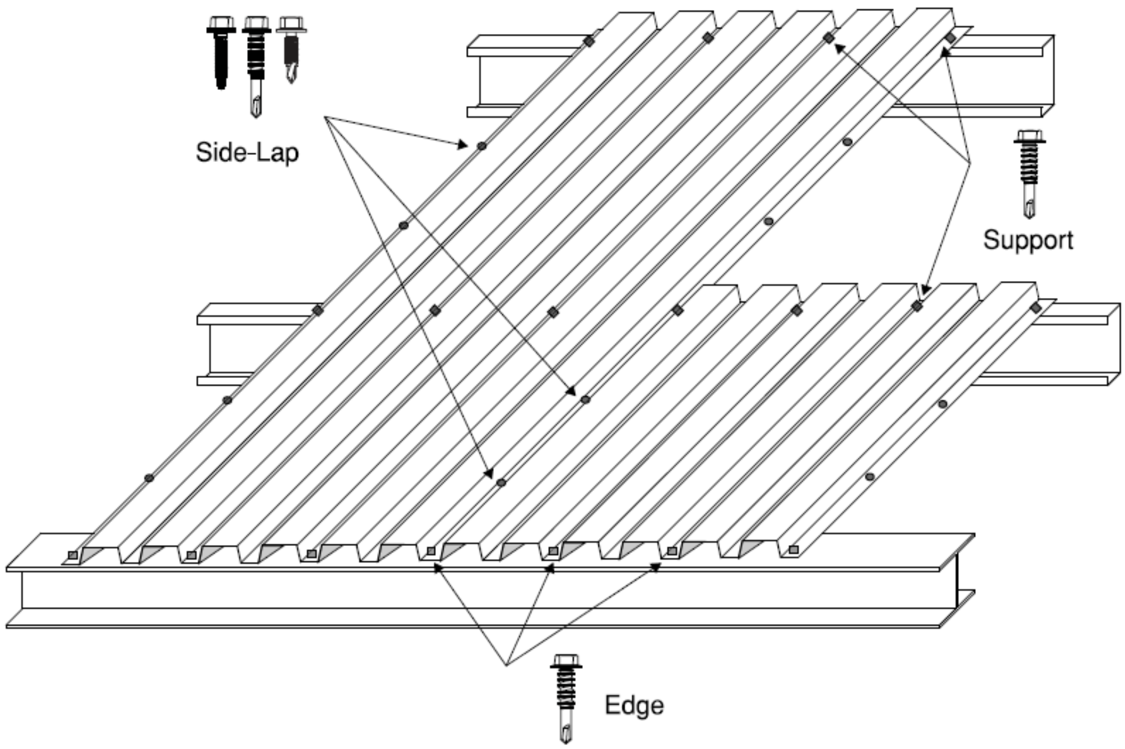

Figure 4.2 – Support, Side-lap, and Edge Screws

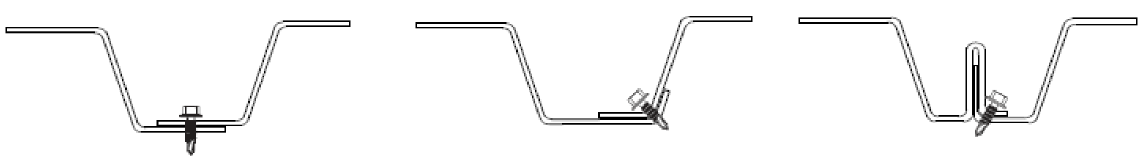

Figure 4.3 – Nested and Interlocking Side-lap Screw Connections

Screws with sealing washers should never be specified or used with steel deck that is the structural support for a finished roofing surface. These type of washers are only used for attachment of exposed metal roofing panels.

Section 4.2.1 Screw Design Considerations with Cold-Formed Framing

When deck is attached to cold-formed steel framing, consideration must be given to additional strength and stiffness limit states that do not control when attaching to thicker steel substrates.

The AISI S100 Standard lists the following strength limit states for screws loaded laterally (in shear).

A. Bearing

The bearing limit state is controlled by the bearing strength of the weaker of the deck and the support. The applicable bearing strength is a function of the thickness (t) of the ply times the tensile strength (F_u) of the ply. When deck is attached to heavier substrates, such as open web steel joists or rolled beams, the bearing strength of the deck always controls. However, on cold-formed steel supports, the bearing strength of the screw against the support will often control when the support is thinner or of lower strength steel than the deck.

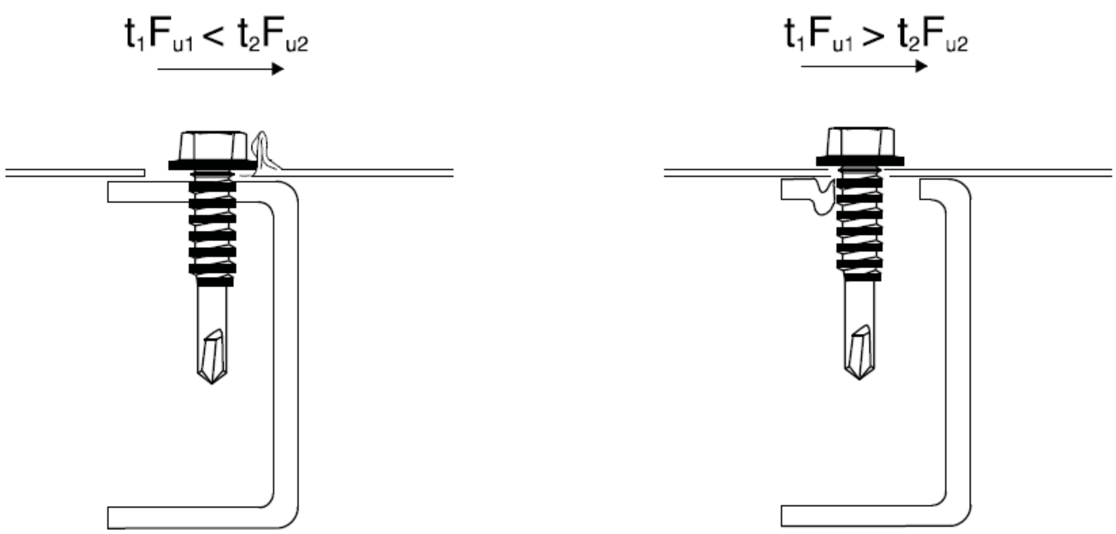

Figure 4.4 – Bearing Limit for Deck and Support

B. Tilting

Tilting may occur when the support framing is thinner than the deck. This is never the case when the supports are open web steel joists or rolled beams, but this can occur with cold-formed steel supports. This occurs when the deformation of the support framing in bearing is greater than the deck and the screw "leans over" in the support. The resulting limit state is a combined action of bearing and the screw pulling out in tension. This may control over bearing.

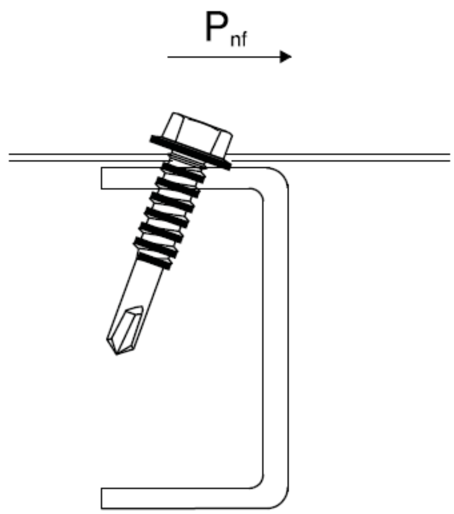

Figure 4.5 – Screw Tilting

C. Shear Strength of the Screw

This limit state is not controlled by the thickness of the deck or the support framing, and will typically only control when attaching two sheets of thicker steel.

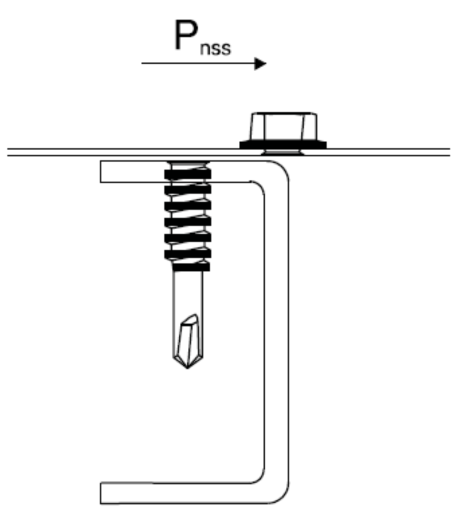

Figure 4.6 – Screw Shearing

The AISI S100 Standard lists the following strength limit states for screws loaded in tension.

D. Pull-Out

The resistance of the screw against pulling out of the support framing is the product of the thickness (t) of the support times the tensile strength (F_u) of the support. This is typically not the controlling tension limit state for deck supported on open web steel joists or rolled beams, but may control for cold-formed framing, particularly for thinner framing.

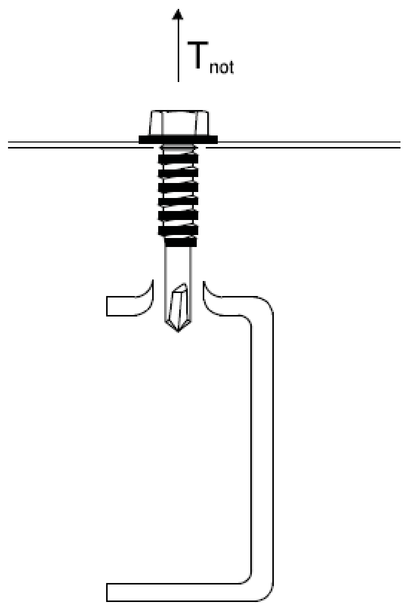

Figure 4.7 – Screw Pull-Out

E. Pull-Over

The limit state of pull-over is the deck pulling over the head of the screw. This is not dependant on the support framing.

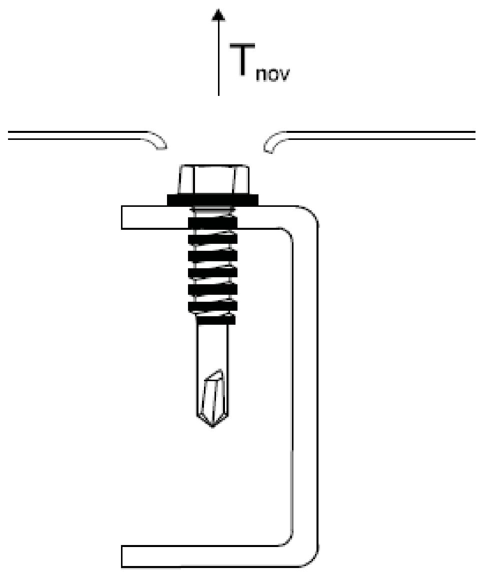

Figure 4.8 – Screw Pull-Over

F. Tensile Strength of the Screw

This limit state is not controlled by the thickness of the deck or the support framing, and will typically only control when attaching two sheets of thicker steel.

The diaphragm stiffness is also affected by the thinner support framing. The equation for diaphragm stiffness (G') in AISI S310 contains the support connection flexibility (S_f) in the equation for the slip coefficient (C) and the stiffness factor (K) for profile distortions. Diaphragms on cold-formed support framing are therefore more flexible than on thicker framing, however, this is not typically a disadvantage.

Section 4.3 Welding

Welded connections for either sidelap or support attachment is NOT RECOMMENDED due to the difficulty in making acceptable welds between two or more layers of sheet steel. Burn-through damage to truss chords and rafter and joists flanges often results and repairs to the supporting framing may not be possible. Any welding related damage to the support framing is not the responsibility of the framing supplier, deck supplier, or deck installer.

Welded side-laps are possible, but limitations on welding imposed by roof slope, burn through, and the cost effectiveness of screwed side-laps make welded side-laps undesirable.

Section 4.4 Power-Actuated Fasteners

Power-actuated fasteners are commonly used for steel roof deck attachment to support framing of 1/8 inch and thicker. At the time of publication of this Manual, no power actuated fasteners were available for connecting steel deck to cold-formed steel support framing. For additional information on these fasteners, refer to the SDI Roof Deck Design Manual and the literature of the product manufacturers.

Section 4.5 Mechanically Formed Side-Lap Connections

As an alternative to screws, side-lap connections can be formed by crimping the upstanding edge of the deck, where provided. 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 but 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 acceptance reports. Information on these proprietary systems can be obtained from specific manufacturers.

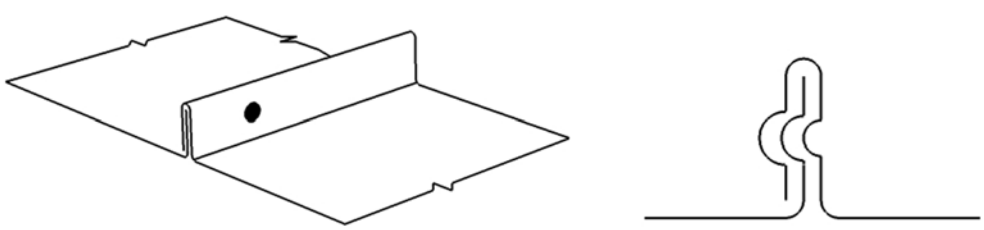

Figure 4.9 – Side-lap Button Punch

Section 4.6 Fastener Spacing Limitations

Strict consideration also needs to be paid to the number of screws that are used at a single rib at a support. Minimum spacing per AISI S100 is 3 times the screw diameter, or approximately 5/8" for a No. 12 screw. That practically limits the number of screws per rib to 2 in most practical cases. At deck ends or butt joints, the minimum end distance is 1.5 times the screw diameter, which also limits the number of screws to 2 in a rib.

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EXAMPLES

SECTION 5

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Section 5.1 List of Examples

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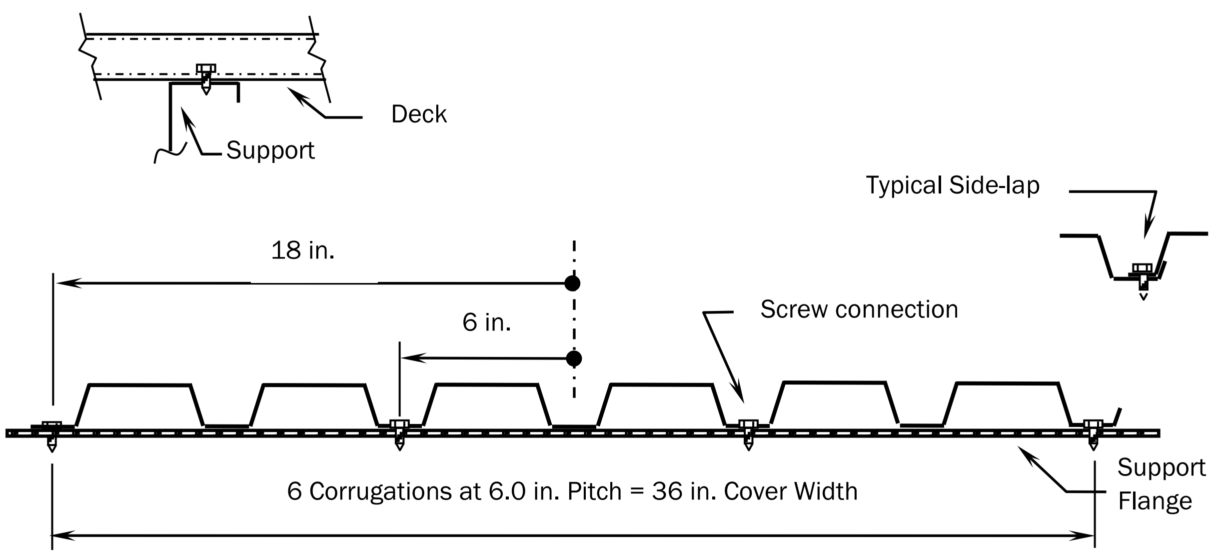
Example 1: Available Diaphragm Shear Strength in the Absence of Uplift Where the Support is Cold-Formed Steel Framing

Objective

Calculate the nominal and available diaphragm shear strength per unit length in the absence of uplift using, AISI S310 Chapter D where the support is cold-formed steel framing.

Diaphragm Configuration



4 support screws per 36 in. wide deck with one screw common - Pattern 36/4

Deck Data

See Wide Rib Deck (WR) in AISI S310, Commentary Table C-1.1a or SDI DDM04, Table 3.3-1.

See AISI S310, Figure D2.1-1 or SDI DDM04, Figure 3.2-2 for definitions of deck parameters.

Yield stress, F_y	= 80 ksi	Modulus of Elasticity, E	= 29500 ksi
Tensile strength, F_u	= 82 ksi	Panel Length, L	= 20.0 ft
Depth, D_d	= 1.47 in.	Cover width, w	= 36.0 in.
Thickness, t	= 0.036 in.	Pitch, d	= 6.00 in.
Top flat width, f	= 3.56 in.	Web flat width, w	= 1.53 in.
Bottom flat width, 2e	= 1.56 in.		
Moment of Inertia, I_{xg}	= 0.190 in.^{4}/ft	This is I_x value from manufacturer and conservatively used for I_{xg}.	

WR deck will be end-lapped.

Note: AISI S310 Chapter D limits (a) through (d) are satisfied. Therefore, Section D1 is used to determine the diaphragm strength.

Specified material is ASTM A653 SS Grade 80. AISI S100 Section A2.3.3 design requirements apply. **Use**: F_y = 60 ksi F_u = 62 ksi

Edge dimensions must be checked using AISI S100 Sections E4.1 and E4.2 as applicable. This example assumes that the "as produced" deck provides adequate edge dimensions. Consult deck manufacturer for dimensions and determine whether screws are permitted at side-laps, and review deck end dimensions at end-laps.

Steel Support Data

Yield stress, F_y	= 40 ksi	Tensile strength, F_u	= 55 ksi
Thickness, t	= 0.048 in.	Support Spacing, L_v (shear span)	= 4.00 ft

Material is based on ASTM A653 SS Grade 40.

Connection Schedule

Support connection: Pattern = 36/4 – See configuration above.

#12 screw	d = 0.216 in.
P_{nss} = 2.0 kips	P_{nts} = 2.7 kips

Same support connection type and spacing will be used at interior and exterior supports.

Note: P_{nss} = Breaking nominal shear strength of screw.

P_{nts} = Breaking nominal tensile strength of screw.

Side-lap connection:

#12 screw	d = 0.216 in.
P_{nss} = 2.0 kips	Screw nominal shear breaking strength
Spacing = 24 in. o.c.	(Between supports)

Determine Available Strength per AISI S310 Eqs. D-1 and D-2

Safety and resistance factors are in AISI S310 Section B1.

$$\frac{S_n}{\Omega} = \min\left(\frac{S_{nf}}{\Omega_{df}}, \frac{S_{nb}}{\Omega_{db}}\right) \quad \text{for ASD} \qquad \phi S_n = \min(\phi_{df} S_{nf}, \phi_{db} S_{nb}) \quad \text{for LRFD and LSD}$$

$$S_{nf} = \min(S_{ni}, S_{nc}, S_{ne})$$

Note: S_{nf} is in Section D1, S_{nb} is in AISI 310 Section D2, and S_{ne} is in Section D1.

Calculate nominal diaphragm shear strength per unit length controlled by screws, S_{nf}, using Section D1:

$$S_{ni} = [2A(\lambda - 1) + \beta]\frac{P_{nf}}{L} \qquad \text{AISI S310 Eq. D1-1}$$

$$S_{nc} = \left(\frac{N^2\beta^2}{L^2N^2 + \beta^2}\right)^{0.5} P_{nf} \qquad \text{AISI S310 Eq. D1-2}$$

Note: S_{nf} is minimum of S_{ni} and S_{nc}. This example requires that sufficient edge connections will be provided at the edge panels so S_{ne} will not control S_{nf}. Details are developed in this example. Same P_{nf} will be used at all supports.

Calculate support connection strength, P_{nf} (AISI S310 Section D1.1.2):

$$\frac{t_2}{t_1} = \frac{0.048\text{ in.}}{0.036\text{ in.}} = 1.33 \qquad \text{Therefore, } 1.0 < \frac{t_2}{t_1} \le 2.5$$

$$P_{nf} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2} \qquad \text{AISI S100 Eq. E4.3.1-1}$$

$$= 4.2\left((0.048\text{ in.})^3(0.216\text{ in.})\right)^{1/2}(55\frac{\text{k}}{\text{in.}^2}) = 1.13\text{ k}$$

$$P_{nf} = 2.7 t_1 d F_{u1} \qquad \text{AISI S100 Eq. E4.3.1-4}$$

$$= 2.7(0.036\text{ in.})(0.216\text{ in.})(62\frac{\text{k}}{\text{in.}^2}) = 1.30\text{ k}$$

$$P_{nf} = 2.7 t_2 d F_{u2} \qquad \text{AISI S100 Eq. E4.3.1-5}$$

$$= 2.7(0.048\text{ in.})(0.216\text{ in.})(55\frac{\text{k}}{\text{in.}^2}) = 1.54\text{ k}$$

$P_{nss} = 2.0\text{ kips}$ See Connection Schedule

$$\frac{t_2}{t_1} = 1.0 \qquad P_{nf} = \min(1.13, 1.30, 1.54, 2.00) = 1.13\text{ k}$$

$$\frac{t_2}{t_1} = 2.5 \qquad P_{nf} = \min(1.30, 1.54, 2.00) = 1.30\text{ k}$$

$$\frac{t_2}{t_1} = 1.33 \qquad P_{nf} = 1.13 + \frac{1.33 - 1.0}{2.5 - 1}(1.30 - 1.13) = 1.17\text{ k}$$

Result: $P_{nf} = 1.17\text{ k}$ Screw tilting in support controls strength, but this value is also close to the screw bearing against deck limit (1.3 k).

Calculate side-lap connection strength, P_{ns} (Section AISI S310 Section D1.2.5 or AISI S100 Section E4.3.1):

$$\frac{t_2}{t_1} = \frac{0.036\text{ in.}}{0.036\text{ in.}} = 1.0 \qquad \text{Therefore, } \frac{t_2}{t_1} \leq 1.0 \qquad F_{u2} = F_{u1}$$

$$P_{ns} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2}$$ AISI S100 Eq. E4.3.1-1

$$= 4.2\left((0.036\text{ in.})^3(0.216\text{ in.})\right)^{1/2}\left(62\frac{\text{k}}{\text{in.}^2}\right) = 0.827\text{ k}$$

$$P_{ns} = 2.7 t_1 d F_{u1}$$ AISI S100 Eq. E4.3.1-2

$$= 2.7(0.036\text{ in.})(0.216\text{ in.})\left(62\frac{\text{k}}{\text{in.}^2}\right) = 1.30\text{ k}$$

$P_{nss} = 2.0\text{ k}$ See Connection Schedule

Result: $P_{ns} = \min(0.827, 1.30, 2.00) = 0.827\text{ kips}$ Tilting of screw in deck controls.

Calculate configuration parameters required for S_{nf}:

$A = 1.0$ Number support screws at side-lap at deck ends

$$\lambda = 1 - \frac{D_d L_v}{240\sqrt{t}} \geq 0.7$$ Required input units are defined in AISI S310 Eq. D1-4

$$= 1 - \frac{(1.47\text{ in.})(4.0\text{ ft})}{240\sqrt{0.036\text{ in.}}} = 0.871 > 0.7\text{ OK}$$ Unit-less

$$N = \frac{3\text{ screws}}{3\text{ ft}} = 1.00\frac{\text{screw}}{\text{ft}}$$ Number of screws into support per ft along deck ends

(Although four screws appear in the sketch, one is common to each deck at side-lap so N = 3/3.)

$$\beta = n_s\alpha_s + 2n_p\alpha_p^2 + 4\alpha_e^2$$ Factor defining screw interaction AISI S310 Eq. D1-5

$$n_p = \frac{L}{L_v} - 1 = \frac{20\text{ ft}}{4\text{ ft}} - 1 = 4.0$$ Number of interior supports AISI S310 Eq. D1-9

$$n_s = \left(\frac{(4\text{ ft})\left(12\frac{\text{in.}}{\text{ft}}\right)}{24\frac{\text{in.}}{\text{conn.}}} - 1\right)\frac{20\text{ ft}}{4\text{ ft}} = 5$$ Number of side-lap screws along the deck length, L

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.827\ \text{k}}{1.17\ \text{k}} = 0.707$$ Connection strength ratio AISI S310 Eq. D1-6

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{w^2}\right)\Sigma x_e^2$$ See diaphragm configuration and Figure D1-1. Eq. D1-8

$x_{e1} = x_{e3} = 6\ \text{in.}$ $x_{e2} = x_{e4} = 18\ \text{in.}$

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{(36\ \text{in.})^2}\right)\left(2(6\ \text{in.})^2 + 2(18\ \text{in.})^2\right) = 0.556$$

$$\beta = 5(0.707) + 2(4)(0.556) + 4(0.556) = 10.2$$ AISI S310 Eq. D1-5

Calculate nominal diaphragm shear strength, S_{nf}:

Note: The simplification is that the butt-joint strength (P_{nf} =1.19 k) will be applied to the end-lap strength in calculating S_{nf}.

$$S_{ni} = [2(1.0)(0.871-1) + 10.2]\frac{1.17\ \text{k}}{20\ \text{ft}} = 0.582\ \text{klf}$$ AISI S310 Eq. D1-1

$$S_{nc} = \left(\frac{(1.0\frac{1}{\text{ft}})^2(10.2)^2}{(20\text{ft})^2(1.0\frac{1}{\text{ft}})^2 + (10.2)^2}\right)^{0.5} 1.17\ \ \text{kips} = 0.532\ \text{klf}$$ AISI S310 Eq. D1-2

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.582, 0.532) = 0.532\ \text{klf}$

Controlled by corner screws at deck ends

Calculate nominal diaphragm shear strength per unit length controlled by deck buckling, S_{nb}, using AISI S310 Section D2.1:

$$S_{nb} = \frac{7890}{\alpha L_v{}^2}\left(\frac{I_{xg}^3 t^3 d}{s}\right)^{0.25}$$ AISI S310 Eq. D2.1-1

Note: See Deck Data for parameters. Required units are defined in AISI S310 Section D2.1.

Coefficient, 7890, includes necessary adjustments – See Commentary Section D2.1.

$\alpha = 1$ Conversion factor for U.S. customary units

$s = 2e + 2w + f = 1.56\text{ in.} + 2(1.53\text{ in.}) + 3.56\text{ in.} = 8.18\text{ in.}$ AISI S310 Eq. D2.1-2

$$S_{nb} = \frac{7890}{(1)(4\text{ ft})^2}\left(\frac{(0.190\text{ in.}^4/\text{ft})^3\ (0.036\text{ in.})^3(6.0\text{ in.})}{8.18\text{ in.}}\right)^{0.25} = 10.9\text{ klf}$$

$S_{nf} = 0.532\text{ klf} << S_{nb} = 10.9\text{ klf}$

Result Nominal Diaphragm Shear Strength per Unit Length, S_n

$S_n = 0.532$ klf Based on one support screw connection strength, P_{nf} = 1.17k.

Note: $S_n = \min(S_{ni}, S_{nc}, S_{nb})$ controls this strength. However, a designer must develop an edge detail requiring S_{ne} to be greater than 0.532 klf so S_n can flow to the lateral force resisting system, e.g. a shear wall or an interior frame where shear flowing from two sides of the diaphragm can be additive, which can require: $S_{ne} >> S_n$.

Calculate the nominal diaphragm shear strength at building edge, S_{ne}:

$$S_{ne} = \frac{(2\alpha_1 + n_p\alpha_2)P_{nf} + n_eP_{nfs}}{L}$$ AISI S310 Eq. D1-3

Consider two edge detail configurations:

1. Starting condition allows standard installation of full width deck, w_e = 36 in., and starting bottom flat lands on edge support. See Figure on next page.
2. Ending condition requires a partial width deck, w_e = 22 in., and bottom flat does not land on an edge support. See Figure on next page.

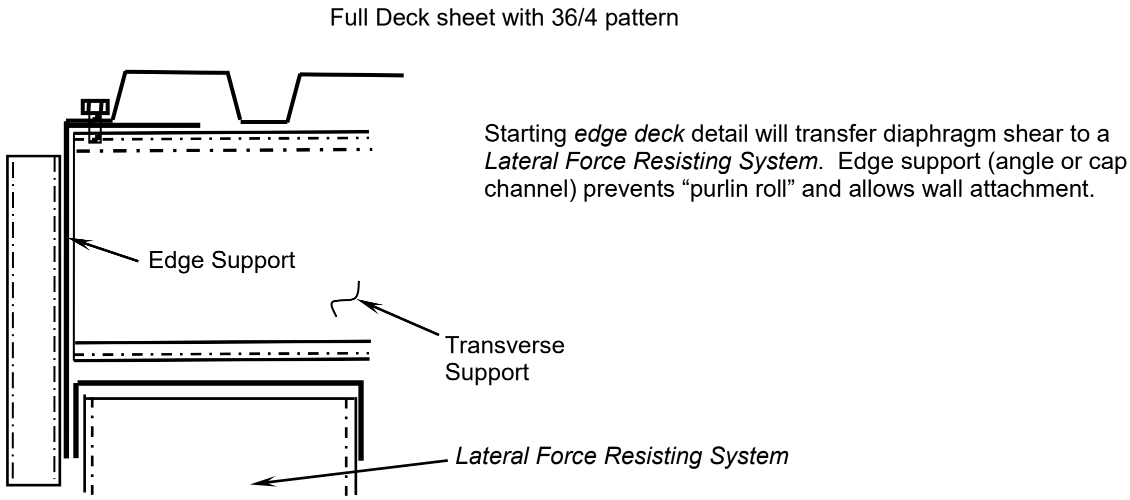
Try to match side-lap screw spacing = 24 in. o.c. along the edge support so $n_e = n_s$.

Try “rule of thumb” and install a support connection in each flute of the partial width deck.

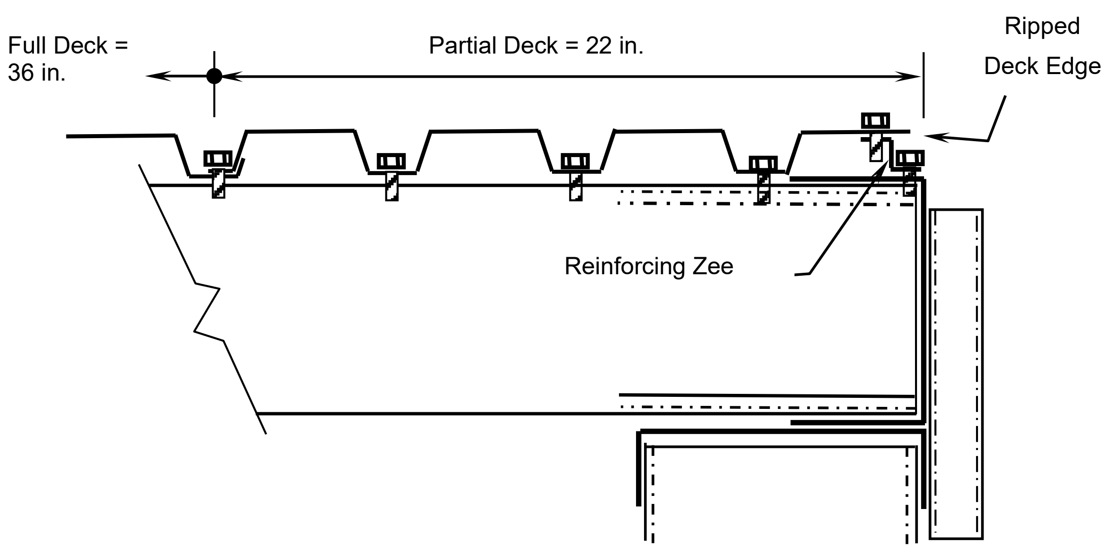
Ripped partial width deck requires a reinforced edge detail to transfer shear to the lateral force resisting system.

The same support connection type (#12 screw) will be used along the edge deck over the edge support parallel with the deck span.

Edge supports are the same material and thickness as the transverse support. Try Reinforcing Zee = 0.059 in. thick.



Starting Condition



Ending Condition

Starting Condition at edge

Calculate support connection strengths along starting edge, P_{nf} and P_{nfs}.

Result: $P_{nf} = P_{nfs} = 1.17\text{ k}$ Edge support material and deck overlay are the same as at transverse supports.

Calculate parameters necessary for S_{ne}

36/4 pattern at interior and exterior supports in the starting edge deck

$$\alpha_1 = \alpha_2 = \left(\frac{\sum x_{ee}}{w_e}\right)$$ AISI S310 Eq. D1-10, 11

$x_{ee1} = x_{ee3} = 6\text{ in.}$ $x_{ee2} = x_{ee4} = 18\text{ in.}$ AISI S310 Figure D1-1

$w_e = 36\text{ in.}$

$$\alpha_1 = \alpha_2 = \left(\frac{6+18+6+18}{36}\right) = 1.33$$

$n_e = n_s = 5$ matches side-lap screw spacing at typical deck

$n_p = 4$ matches transverse support spacing at typical deck

$$S_{ne} = \frac{(2(1.33)+4(1.33))1.17+5(1.17)}{20} = 0.759\text{ klf}$$

Note: S_{ne} is based on the butt joint simplification and uses P_{nf} = 1.17 k as the support connection strength.

Quick design just counts the fasteners along the edge support.

$$S_{ne} = \frac{(2+4)1.17+5(1.17)}{20} = 0.644\text{ klf} > 0.532\text{ klf.}$$

Quick design is adequate in this case.

The capacity of the lateral force resisting system and its members including the edge support must be checked but is assumed to be adequate to develop S_n = 0.532 klf.

Result: S_{ne} = 0.759 klf > 0.532 klf

Use typical deck connections in the edge deck at the starting edge condition.

Use #12 screws at 24 in. o.c. between transverse supports along the edge support.

Ending Condition at edge

The reinforcing detail shown in the Figure is not the only detail that is possible. The common feature is that a web is provided to transfer shear from the deck's ripped top flat to the edge support. Sometimes finish strips are used but these are typically the same thickness as deck and can be weaker than deck. All details require that the designer:

1. Check the stability of the new web and the deck's ripped edge, and
2. Determine the required number of screws at two connection planes.

Erectors might adjust deck cover at several sheets with nestable side-laps, e.g. slide the male lip over 1/2 in. in the side-laps near the end of the run. This allows the last deck to be ripped in the bottom flat or back lapped to avoid ripping and repeats the same detail as the starting condition. In this example the last deck's cover is changed from 22 in. to 24 in. by using this process. Partial width deck sheets still must be designed.

Partial Width Deck Sheet at Ending Condition with Ripped Deck

S_n = 0.532 klf must be developed. Try 22/5 pattern or #12 screw at 6 in. o. c. over supports, and #12 screw side-lap connections at 24 in. o.c. at side-lap between partial and last full-width deck sheet.

Calculate configuration parameters required for S_{nf}:

$A = 1.0$ Most parameters same as full width sheet

$\lambda = 0.871 > 0.7 \text{ OK}$ AISI S310 Eq. D1-4

$N = \dfrac{4 \text{ screws}}{1.83 \text{ ft}} = 2.19 \dfrac{\text{screw}}{\text{ft}}$ (Theoretically could use 4.5 screws)

$\beta = n_s \alpha_s + 2 n_p \alpha_p^2 + 4 \alpha_e^2$ AISI S310 Eq. D1-5

$n_p = 4.0$

$n_s = 5$

$\alpha_s = 0.707$ AISI S310 Eq. D1-6

$\alpha_p^2 = \alpha_e^2 = \left(\dfrac{1}{w^2}\right) \sum x_e^2$ See diaphragm configuration and AISI S310 Figure D1-1.

$x_{e1} = x_{e3} = 6 \text{ in.} \qquad x_{e2} = 12 \text{ in.} \qquad x_{e4} = 9.5 \text{ in.}$

$\alpha_p^2 = \alpha_e^2 = \left(\dfrac{1}{(22 \text{ in.})^2}\right)\left(2(6 \text{ in.})^2 + (12 \text{ in.})^2 + (9.5 \text{ in.})^2\right) = 0.633$ AISI S310 Eq. D1-8

$\beta = 5(0.707) + 2(4)(0.633) + 4(0.633) = 11.1$ AISI S310 Eq. D1-5

Calculate nominal diaphragm shear strength, S_{nf}:

Note: The simplification is that the butt-joint support connection strength (P_{nf} =1.17 k) will be applied to the end-lap strength in calculating S_{nf}.

$$S_{ni} = [2(1.0)(0.871-1)+11.1]\frac{1.17\text{ k}}{20\text{ ft}} = 0.634\text{ klf}$$ AISI S310 Eq. D1-1

$$S_{nc} = \left(\frac{(2.19\frac{1}{\text{ft}})^2(11.1)^2}{(20\text{ft})^2(2.19\frac{1}{\text{ft}})^2+(11.1)^2}\right)^{0.5} 1.17\ \ \text{kips} = 0.629\text{ klf}$$ AISI S310 Eq. D1-2

$S_{nb} = 10.9\text{ klf}$ From full width deck AISI S310 Eq. D2.1-1

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.634, 0.629) = 0.629\text{ klf}$

Controlled by corner screws at deck ends but almost balanced.

Deck buckling is not a factor.

Partial width deck S_{nf} > 0.532 klf and rule of thumb works.

Use # 12 screws at 22/5 support pattern (6 in. o.c. typical).

Use # 12 screws at 24 in. o.c. at side-lap between last full deck and partial deck sheet.

Note: Different side-lap conditions exist along the sides of the ending deck. The typical deck condition is used in the calculation and the edge condition will be designed to contribute greater strength.

Calculate the nominal diaphragm shear strength at ending edge, S_{ne}:

$$S_{ne} = \frac{(2\alpha_1 + n_p\alpha_2)P_{nf} + n_e P_{nfs}}{L}$$ AISI S310 Eq. D1-3

Note: AISI S310 Eq. D1-3 applies at plane between reinforcing Zee and edge support

Calculate support connection strength, P_{nf}, at Zee (AISI S310 Section D1.1.2):

$\frac{t_2}{t_1} = \frac{0.048\text{ in.}}{0.059\text{ in.}} = 0.814$ Therefore, $\frac{t_2}{t_1} \leq 1.0$

$P_{nf} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2}$ AISI S100 Eq. E4.3.1-1

$= 4.2\left((0.048\ \text{in.})^3(0.216\ \text{in.})\right)^{1/2}(55\frac{k}{\text{in.}^2}) = 1.13\ k$

$P_{nf} = 2.7 t_1 d F_{u1}$ AISI S100 Eq. E4.3.1-2

$= 2.7(0.059\ \text{in.})(0.216\ \text{in.})(55\frac{k}{\text{in.}^2}) = 1.89\ k$

$P_{nf} = 2.7 t_2 d F_{u2}$ AISI S100 Eq. E4.3.1-3

$= 2.7(0.048\ \text{in.})(0.216\ \text{in.})(55\frac{k}{\text{in.}^2}) = 1.54\ k$

$P_{nss} = 2.0\text{ kips}$ See Connection Schedule

$P_{nf} = \min(1.13, 1.89, 1.54, 2.00) = 1.13\text{ k}$

$P_{nf} = 1.13\text{ k}$ Screw tilting in edge support controls strength.

Note: Top ply = 0.059 in. has slightly less strength than 0.036 in., P_{nf} = 1.13 vs. 1.17 k because of tilting and high strength steel, F_u = 82 ksi at t= 0.036 in.

Result: $P_{nfs} = P_{nf} = 1.13\text{ k}$ Edge support material same as at transverse supports.

Calculate parameters necessary for S_{ne} at ending edge

22/5 pattern at interior and exterior supports in the ending edge deck

$\alpha_1 = \alpha_2 = \left(\frac{\sum x_{ee}}{w_e}\right)$ AISI S310 Eq. D1-10, 11

$x_{e1} = x_{e3} = 6\text{ in.}$ $x_{e2} = 12\text{ in.}$ $x_{e4} = 9.5\text{ in.}$ AISI S310 Figure D1-1

$w_e = 22\text{ in.}$

$\alpha_1 = \alpha_2 = \left(\frac{0 + 6 + 12 + 6 + 9.5}{22}\right) = 1.52$

$n_e = n_s = 5$ matches side-lap screw spacing at typical deck

$n_p = 4$ matches transverse support spacing at typical deck

$$S_{ne} = \frac{(2(1.52) + 4(1.52))1.13 + 5(1.13)}{20} = 0.798\,\text{klf}$$ AISI S310 Eq. D1-3

Result: S_{ne} = 0.798 klf > 0.532 klf

Note: Reinforcing Zees often are press broken in 10 ft lengths.

Schedule: Zee bottom flat to edge support:

Typical screw spacing: 2 in., 22 in., 24 in., 24 in., 24 in., 22 in., 2 in.

Use #12 screws over the transverse supports and at 24 in. o.c. maximum spacing between supports. Locate a screw either side of the butt end.

Calculate shear stress in 1-1/2 in. Zee (ASD):

Let: $\Omega = \Omega_d = 2.35$

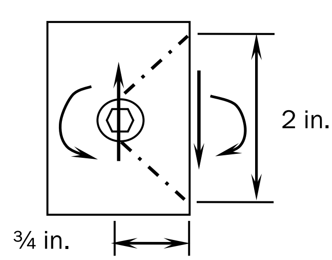
ASD Available diaphragm shear strength at edge $= \dfrac{0.532\,\text{klf}}{2.35} = 0.226\,\text{klf}$

Note: For illustration, 2.35 is the smallest safety factor(AISI S310-2013) and creates the largest ASD shear stress in 1-1/2 in. Zee for a nominal diaphragm shear per unit length.

$$\tau_v = \frac{0.226\,\frac{\text{k}}{\text{ft}}}{12(0.059)\,\frac{\text{in.}^2}{\text{ft}}} = 0.319\,\text{ksi}$$ Based on entire Zee length resisting shear equally

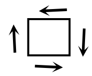
Since shear has to get out at connections, consider that not all zones of Zee work equally in the bottom flat. Top and Bottom flat dimensions are 1 1/4 in. Use 2 in. of bottom flat to resist shear transfer. Maximum Zee tributary length is 2.0 ft per screw.

$$\tau_v \le \frac{2.0\,\text{ft}(0.226)\frac{\text{k}}{\text{ft}}}{2(0.059)\,\text{in.}^2} = 3.83\,\text{ksi}$$



Screw in Zee bottom flat

Determine nominal shear strength, $F_{v,}$ of bottom flat at connection:

$$\frac{h}{t} = \frac{0.75\,\text{in.}}{0.059\,\text{in.}} = 12.7$$  AISI S100 Section C3.2.1

Note: Screw line should approximate a web over a 2 in. length, k_v = 5.34, but conservatively rationalize k_v = 0.57 because plate has free edge in the adjacent area.

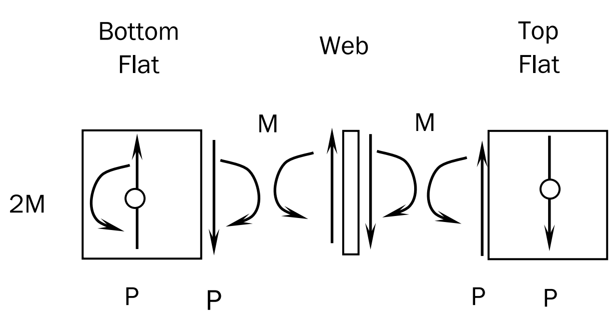
$$\sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500\,\text{ksi}(0.57)}{40\,\text{ksi}}} = 20.5 > 12.7$$

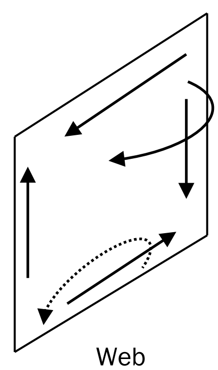
F_v = 0.6(40 ksi) = 24 ksi AISI S100 Eq. C3.2.1-2

$$\text{Allowable shear stress} = \frac{F_v}{1.6} = \frac{24}{1.6} = 15.0\,\text{ksi} >> 3.83\,\text{ksi} \quad \text{OK}$$

Calculate St. Venant shear in Zee web due to twisting (screw eccentricity):

Note: Twisting due to screw eccentricity (2M) will create minimal and secondary (x axis) reactions on bottom flat screws – lever arms = 2, 6 and 10 ft vs. 1 ½ in.
Refer to Advanced Mechanics of Materials by Seely and Smith or other textbooks for torsional resistance of rectangular cross-sections:





$$\textit{ASD Available Diaphragm Strength} = 0.226\text{klf}$$

P = 0.226 klf Shear transfer to Zee

M = (0.75 in.)(0.226 klf) = 0.170 in. k/ft

$$\iota_{SVv} = \frac{3M}{bt^2} = \frac{3(0.170)\,\frac{\text{in.k}}{\text{ft}}}{12(0.059)^2\,\frac{\text{in.}^3}{\text{ft}}} = 12.2\,\text{ksi}$$

This neglects twisting resistance of deck that is attached to top flat.

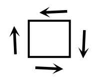
Calculate resultant shear stress at outer edges of web:

$f_v \le \tau_v + \tau_{SVv} = 3.83 + 12.2 = 16.0 \text{ ksi}$ At extreme fiber across thickness

Ave. = 3.83 ksi across thickness

Determine nominal shear strength, F_v, of web: AISI S100 Section C3.2.1

$$\frac{h}{t} = \frac{1.5\text{ in.}}{0.059\text{ in.}} = 25.4$$



AISI S100 Eq. C3.2.1-2

$$\text{For } \frac{h}{t} \le \sqrt{\frac{Ek_v}{F_y}} \quad F_v = 0.6F_y \qquad \sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500\text{ ksi}(5.34)}{40\text{ ksi}}} = 62.8 > 25.4$$

$F_v = 0.6(40\text{ ksi}) = 24\text{ ksi}$ Allowable shear stress = $\frac{F_v}{1.6} = \frac{24}{1.6} = 15.0\text{ ksi}$

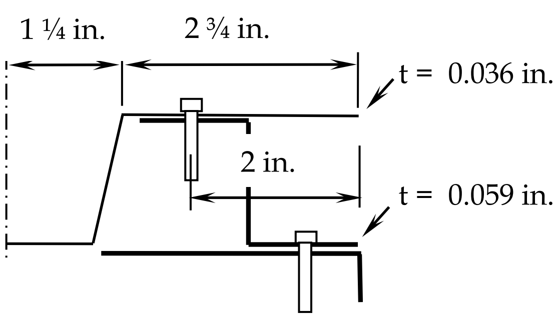
$16.0\text{ ksi} \approx 15\text{ ksi}$ Assumptions are very conservative and neglect:

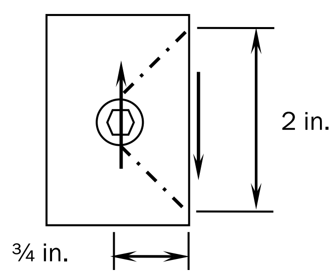
1. Ability to distribute stress along Zee, e.g. 3.83 ksi vs. 0.319 ksi,
2. Ability to distribute stress across thickness, e.g. 12.2 ksi vs. 0 ksi, and
3. Deck top flat contribution resisting twisting.

Result: t = 0.059 in. and F_y = 40 ksi OK at Zee.

Calculate support connection strength at ripped deck

Try #12 screws at 16 in. o.c. (over supports and between supports)





Screw in Zee top flat

Calculate support connection strength, P_{nf}, at edge (AISI S310 Section D1.1.2):

$$\frac{t_2}{t_1} = \frac{0.059\text{ in.}}{0.036\text{ in.}} = 1.64 \quad \text{Therefore,} \quad 1.0 \le \frac{t_2}{t_1} \le 2.5$$

$$P_{nf} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2}$$ AISI S100 Eq. E4.3.1-1

$$= 4.2\left((0.059\text{ in.})^3 (0.216\text{ in.})\right)^{1/2} (55\frac{\text{k}}{\text{in.}^2}) = 1.54\text{ k}$$

$$P_{nf} = 2.7 t_1 d F_{u1}$$ AISI S100 Eq. E4.3.1-2

$$= 2.7(0.036\text{ in.})(0.216\text{ in.})(62\frac{\text{k}}{\text{in.}^2}) = 1.30\text{ k}$$

$$P_{nf} = 2.7 t_2 d F_{u2}$$ AISI S100 Eq. E4.3.1-3

$$= 2.7(0.059\text{ in.})(0.216\text{ in.})(55\frac{\text{k}}{\text{in.}^2}) = 1.89\text{ k}$$

$$\frac{t_2}{t_1} = 1.0 \qquad P_{nf} = \min(1.54, 1.30, 1.89, 2.00) = 1.30\text{ k}$$

$$\frac{t_2}{t_1} = 2.5 \qquad P_{nf} = \min(1.30, 1.89, 2.00) = 1.30\text{ k}$$

$$\frac{t_2}{t_1} = 1.64 \qquad P_{nf} = 1.30 + \frac{1.64 - 1.0}{2.5 - 1}(1.30 - 1.30) = 1.30\text{ k}$$

Result: $P_{nf} = 1.30\text{k}$ Deck bearing controls at deck to Zee connection

Calculate S_{ne} at ending edge between deck and Zee

Note: Use quick design by counting screws along Zee

$n_p = 4.0$ Same as typical deck

$n_s = 5(2) = 10$ Spacing = 16 in. o.c. in 4 ft span

$$S_{ne} = \frac{(2+4)1.30 + 10(1.30)}{20} = 1.04\text{ klf} > 0.532\text{ klf}$$ AISI S310 Eq. D1-3

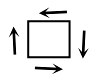
Calculate shear stress in deck (ASD)

Note: Distance from screw to deck web = 0.75 in. (same as Zee so develop 2 in.)

$$\tau_v \le \frac{1.33\,\text{ft}(0.226)\frac{\text{k}}{\text{ft}}}{2(0.036)\,\text{in.}^2} = 4.17\,\text{ksi}$$

Determine nominal shear strength, F_v, of panel's top flat at connection:

$$\frac{h}{t} = \frac{0.75\,\text{in.}}{0.036\,\text{in.}} = 20.8$$



AISI S100 Section C3.2.1

Note: Screw line should approximate a web over a 2 in. length, k_v = 5.34, but conservatively rationalize k_v = 0.57 because deck has free edge in the adjacent area.

$$20.8 > \sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500\,\text{ksi}(0.57)}{60\,\text{ksi}}} = 16.7 \qquad 1.51\sqrt{\frac{Ek_v}{F_y}} = 25.3 > 20.8$$

$$F_v = \frac{0.6\sqrt{Ek_vF_y}}{(h/t)} = \frac{0.60\sqrt{29500\,\text{ksi}(0.57)(60\,\text{ksi})}}{(20.8)} = 29.0\,\text{ksi}$$

AISI S100 Eq. C3.2.1-3

Allowable shear stress = $\frac{F_v}{1.6} = \frac{29}{1.6} = 18.1\,\text{ksi} >> 4.17\,\text{ksi}$ OK

Result: Use #12 screws at 16 in. o.c. (over supports and between supports)

Note: Could justify 24 in. o.c. to match typical deck but judgment was to stiffen ripped edge to better approximate a deck flute.

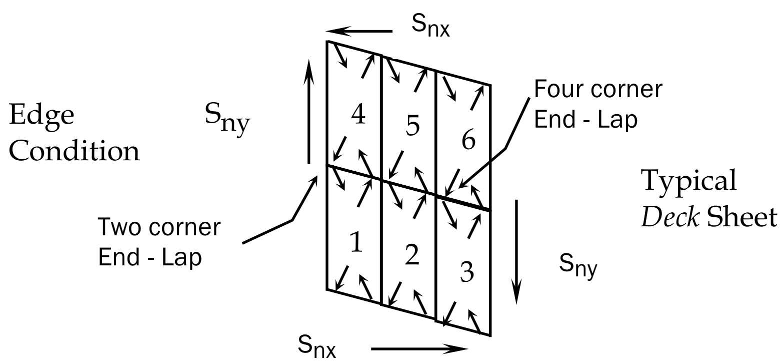
At 24 in. o.c. S_{ne} = 0.715 klf > 0.532 klf τ_v = 6.28 ksi < 18.1 ksi

Result: Proposed Edge detail at ripped deck will transfer the required S_n = 0.532 klf to lateral force resisting system.

Consider potential refinements due to end-laps in typical deck and edge deck.

Racking due to shear per unit length creates diagonal tension or compression in the connections along the deck ends with maximum forces at the corners. See Figure where deck numbering is the deck sheet installation sequence in this example.

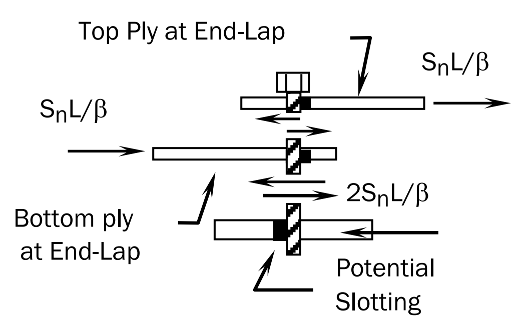
Four corner end-laps occur in typical decks and two corner end-laps occur in edge decks where nestable side-laps are used. Interlocking side-laps typically do not have a common connection at deck corners and require butt laps -- but not always.



Vectors on Deck Corner Connections due Shear Deformation

Edge condition with two corner end-lap

Deck 4 laps over deck 1 and the x axis components of the diagonal forces cancel so the only component at the faying surface between the edge support and deck 1 is along the deck's length. The y axis forces in the decks are additive. The strength at that faying surface is 1.17 k per screw and is controlled by tilting in the edge support. Only ½ (or 0.59 k) can be developed in each deck at the common screw at the end lap, and this value is much less than the bearing strength of the deck, 1.30 k. The impact of the thin edge support is significant. The impact of components on the deck strength, S_n, will be discussed later.



Use the parameters determined at the starting condition above to calculate S_{ne}.

$$S_{ne} = \frac{2(1.33)(0.59) + 4(1.33)(1.17) + 5(1.17)}{20} = 0.682 \text{ klf}$$ modified AISI S310 Eq. D1-3

Result: S_{ne} = 0.682 klf > 0.532 klf

In this case the end-lap causes a 10 percent reduction relative to the butt joint simplification but the detail is adequate – 0.759 klf vs. 0.682 klf. Longer deck, many interior supports, and thicker supports typically mitigate such a reduction. A quick design is still adequate and predicts:

$$S_{ne} = \frac{2(0.59) + 4(1.17) + 5(1.17)}{20} = 0.586 \text{ klf} > 0.532 \text{ klf}$$

Typical deck condition with four corner end-lap

All four decks lap at the common corner fastener – See decks 2, 3, 5, and 6. The x and y axes components of the diagonal forces cancel at the faying surface between the bottom deck 2 and the transverse support. There is virtually nothing to cause tilting or bearing in the support and the strength of the corner connection in each panel is controlled by the bearing strength of the panel, 1.30 k. The screw shank faying surface below:

1. Deck 6 sees the full force in the deck 6 corner screw and this is accounted for in the determination of S_{nc},
2. Deck 5 sees only the x axis component since the y axis forces cancel. That x axis component is:

   $$\frac{S_n}{N} = \frac{0.532 \text{ klf}}{1 \text{ screw per ft}} = 0.532 \text{ k per screw}$$
3. Deck 3 sees the same magnitude force as deck 6 since both x and y components are present. The vector is different.
4. Deck 2 sees no force since all balance.

However, the first interior screw adjacent to the corner screw is in a two ply end-lap and this result is the same as the two corner end-lap at the edge condition. The x components cancel but the y components are additive. This linear addition is accounted for in the determination of S_{ni}.

To simplify design use the same strength at all end-lap support connections and select the more severe case:

Support Type	P_{nf} Type	S_{nf}	
		S_{ni}	S_{nc}*
Exterior at end-lap	P_{nfe}	0.59 k	1.30 k *
Interior	P_{nfi}	1.17 k	1.17 k

* S_{nc} determines diaphragm strength controlled by the corner screw in each deck. The contribution of other deck screws is contained in the parameters, β and N, so this is included in S_{nc}.

In this case some tilting is possible at screws away from the corner at the end lap so why not use 0.59 kips when determining S_{nc}? This would be a conservative method and that reduction is accounted for in S_{ni}.

Consider shear flow through end-lap and determine S_{nf}:

Note: Use Modified AISI S310 Eqs. D1-1 and D1-2 and the above table to calculate S_{nf}.

$$S_{ni} = [2A(\lambda - 1)P_{nfe} + n_s P_{ns} + 2n_p \alpha_p^2 P_{nfi} + 4\alpha_e^2 P_{nfe}]\frac{1}{L} \qquad \text{Modified AISI S310 Eq. D1-1}$$

Calculate S_{ni} based on P_{nfi} = 1.17 k, P_{nfe} = 0.59 k:

Rewriting Modified Eq. D1-1 so closer in form to Eq. D1-1:

$$S_{ni} = [2A(\lambda - 1)\frac{P_{nfe}}{P_{nfi}} + n_s \frac{P_{ns}}{P_{nfi}} + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}}]\frac{P_{nfi}}{L}$$

See butt-joint calculation for: $A, N, L, n_p, n_s, \lambda, \alpha_e^2, \alpha_p^2, P_{ns}$

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}} \qquad \text{Modified AISI S310 Eq. D1-5}$$

$$\alpha_s = \frac{0.827\text{ k}}{1.17\text{ k}} = 0.707 \qquad \text{AISI S310 Eq. D1-6}$$

$$\beta = 5(0.707) + 2(4)(0.556) + 4(0.556)\frac{0.59\text{ k}}{1.17\text{ k}}$$

$$= 9.10$$

$$S_{ni} = [2(1.0)(0.871 - 1)\frac{0.59\text{ k}}{1.17\text{ k}} + 9.10]\frac{1.17\text{ k}}{20\text{ ft}} \qquad \text{Modified AISI S310 Eq. D1-1}$$

$$= 0.525\text{ klf}$$

Calculate S_{nc} based on P_{nfe} = 1.30 k, P_{nfi} = 1.17 k

$$S_{ni} = [2A(\lambda - 1) + n_s \frac{P_{ns}}{P_{nfe}} + 2n_p\alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2] \frac{P_{nfe}}{L}$$ Modified AISI S310 Eq. D1-1

$$\beta = n_s\alpha_s + 2n_p\alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \qquad \text{Based on } P_{nfe} \text{ to calculate } S_{nc}$$

$$\alpha_s = \frac{0.827\text{ k}}{1.30\text{ k}} = 0.636$$ AISI S310 Eq. D1-6

$$\beta = 5(0.636) + 2(4)(0.556)\frac{1.17\text{ k}}{1.30\text{ k}} + 4(0.556)$$

$$= 9.41$$

$$S_{nc} = \left(\frac{(1)^2(9.41)^2 \frac{1}{\text{ft}^2}}{(20)^2(1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (9.41)^2}\right)^{0.5} 1.30\text{ k}$$ Modified AISI S310 Eq. D1-2

$$= 0.553\ \text{klf}$$

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.525, 0.553) = 0.525\text{ klf}$

Controlled by tilting of screws at all supports

Note: The butt-joint strength (without all the refinements) is 0.532 klf vs. 0.525 klf considering shear flow across the end-laps. The refinement provides a negligible 1.3% decrease and neglects the additional stability at the four corner end-lap. The controlling limit state changed – S_{nc} to S_{ni}. The other case of one end with end-lap and one without will be between these values – say, 0.529 klf. The result would be different at three spans and with fewer side-lap screws, but many manufacturers publish tables based on the butt-joint case and this is a rational approach in this example.

The impact of thin supports can be significant when end-laps are present and S_{ne} is determined. Consider two cases:

1. If deck bearing does not control P_{nf} and either tilting, support bearing or screw breaking controls, use:

$$S_{ne} = \frac{(\alpha_1 + n_p\alpha_2)P_{nf} + n_e P_{nfs}}{L}$$ accounts for common end-lap screw.

2. If deck bearing does control P_{nf} and tilting, support bearing, and screw breaking capacity are more than twice the deck bearing capacity use:

$$S_{ne} = \frac{(2\alpha_1 + n_p\alpha_2)P_{nf} + n_e P_{nfs}}{L} \qquad \text{accounts for double shear}$$

Refinements are possible at the third case where tilting or support bearing capacity is less than twice the deck bearing capacity but simply default to case 1 for design.

Result of Example 1:

Nominal diaphragm shear strength per unit length:

S_n = 0.532 klf   Controlled by S_{nf}

Available diaphragm shear strength per unit length, S_a:

Depending on different load types, select safety and resistance factors from Section B1.

Use Ω_d and ϕ_d for connection-related diaphragm strength in AISI S100 Table D5 (for AISI 310-13) or AISI S310 Table B-1.1 (for AISI S310-16).

$\frac{S_n}{\Omega} = \frac{S_{nf}}{\Omega_{df}}$ for ASD $\phi S_n = \phi_{df} S_{nf}$ for LRFD and LSD

Example 2: Stiffness of the Configuration in Example 1

Objective

Calculate the stiffness of the configuration in Example 1.

Note: Use Sections AISI S310 D5.1.1 and D5.2. Use Appendix Section 1.4 for D_n. Stiffness considers the dominant panels in the diaphragm field.

$$G' = \left(\frac{Et}{2(1+\mu)\frac{s}{d} + \gamma_c D_n + C} \right) K \qquad \text{AISI S310 Eq. D5.1.1-1}$$

K = 1.0 for steel panels with lap-down on steel supports

$$C = \left(\frac{Et}{w} \right) \left(\frac{2L}{2\alpha_3 + n_p \alpha_4 + 2n_s \frac{S_f}{S_s}} \right) S_f \qquad \text{AISI S310 Eq. D5.1.1-2}$$

See diaphragm configuration and Figure D1-1 for x_e.

$$\alpha_3 = \alpha_4 = \frac{\sum x_e}{w} = \left(\frac{1}{36\text{ in.}} \right) \left(2(6\text{ in.}) + 2(18\text{ in.}) \right) = 1.33 \qquad \text{AISI S310 Eq. D5.1.1-3}$$

Calculate support screw flexibility, S_f, using Commentary of Section D5.2.2:

Note: Since the support is relatively thin, tilting of screw may occur at the support. Eq. D5.2.2-1 is based on thick supports where distortion is dominated by local slotting and buckling in the deck, and it will not be applicable to this configuration with a thin support. Commentary Section D5.2 should be considered.

Calculate, t_3, balance point between tilting and bearing control in support:

$$4.2\left(t_3^3 d\right)^{1/2} F_{u2} = 2.7 t_2 d F_{u2}$$

Rationale: Beyond thickness, t_3, tilting at support will not control, and flexibility is relatively independent of support thickness.

$$\left(t_3^3\right)^{1/2} = \frac{2.7(0.048\text{ in.})(0.216\text{ in.})}{4.2(0.216\text{ in.})^{0.5}} = 0.014\text{ in.}^{1.5}$$

$$t_3 = 0.058\text{ in.}$$

$$S_f = \left(3 - 1.7\left(\frac{t_2 - t_1}{t_3 - t_1}\right)\right)\left(\frac{\alpha}{1000 t_1^{0.5}}\right)$$ AISI S310 Figure C-D5.2.2-1

$$= \left(3 - 1.7\left(\frac{0.048 - 0.036}{0.058 - 0.036}\right)\right)\left(\frac{\alpha}{1000(0.036)^{0.5}}\right)$$

$\alpha = 1$ Conversion factor for U.S. customary units

$$S_f = \frac{2.07\alpha}{1000\sqrt{t}} = \frac{2.07(1)}{1000\sqrt{0.036\text{ in.}}} = 0.0109\frac{\text{in.}}{\text{k}}$$ (59% greater than S_f with thick support)

Calculate side-lap screw flexibility, S_s:

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{0.036\text{ in.}}} = 0.0158\frac{\text{in.}}{\text{k}}$$ AISI S310 Eq. D5.2.2-2

Calculate slip constant at connections, C:

$$C = \left(\frac{Et}{w}\right)\left(\frac{2L}{2\alpha_3 + n_p\alpha_4 + 2n_s\frac{S_f}{S_s}}\right)S_f$$ AISI S310 Eq. D5.1.1-2

$$\alpha_3 = \frac{\sum x_e}{w} = \frac{(2(6) + 2(18))}{36} = 1.33; \qquad \alpha_4 = \frac{\sum x_p}{w} = \alpha_3 = 1.33$$

$$C = \left(\frac{29500\frac{\text{k}}{\text{in.}^2}(0.036\text{ in.})}{36\text{ in.}}\right)\left(\frac{2(12\frac{\text{in.}}{\text{ft}})(20\text{ ft})}{2(1.33) + 4(1.33) + 2(5)\left(\frac{0.0109}{0.0158}\right)}\right)0.0109\frac{\text{in.}}{\text{k}} = 10.4$$ AISI S310 Eq. D5.1.1-2

Calculate D_n using S310 Appendix 1:

$$D_n = \frac{D}{L}$$ AISI S310 Eq. 1.4-1

$$D = \frac{U_1D_1 + U_2D_2 + U_3D_3 + U_4D_4}{U_1 + U_2 + U_3 + U_4}$$ AISI S310 Eq. 1.4-2

Note: Only D_2 is required in this example and D_2 uses AISI S310 Eq. 1.4-4.
AISI S310 Commentary Table C-1.2 lists D_2 =7726 in. at t = 0.0358 in. for WR deck and this value can be used in design. Adjust D_2 based on t.

$$D_2 = 7726\left(\frac{.0358}{.036}\right)^{1.5} = 7662 \text{ in.}$$ AISI S310 Eq. 1.4-4

$U_2 = 6 \qquad U_1 = U_3 = U_4 = 0$

$$D = \frac{0 + 6(7662) + 0 + 0}{0 + 6 + 0 + 0} = 7662 \text{ in.}$$ AISI S310 Eq. 1.4-2

$$D_n = \frac{7662 \text{ in.}}{20 \text{ ft} \times 12 \frac{\text{in.}}{\text{ft}}} = 31.9 \quad \text{(Unit-less)}$$ AISI S310 Eq. 1.4-1

$\gamma_c = 0.71$ AISI S310 Table 1.3-1

Calculate G'

From Deck Data

s = f + 2e + 2w = 3.56 in. + 1.56 in.+2(1.53 in.) = 8.18 in. AISI S310 Eq. 2.1-2

d = 6 in.

μ = 0.3 (Poisson's ratio for steel)

$$G' = \left(\frac{29500 \frac{\text{k}}{\text{in.}^2}(0.036)\text{in.}}{2(1+0.3)\frac{8.18 \text{ in.}}{6 \text{ in.}} + 0.71(31.9) + 10.4}\right)1 = 29.0 \frac{\text{k}}{\text{in.}}$$ AISI S310 Eq. D5.1.1-1

Note: If the support is much thicker:

$$S_f = \frac{1.3\alpha}{1000\sqrt{t}} = \frac{1.3(1)}{1000\sqrt{0.036\text{ in.}}} = 0.00685\,\frac{\text{in.}}{\text{k}}$$ AISI S310 Eq. D5.2.2-1

$C = 7.88$ 24% less slippage and distortion at connections

$G' = 31.2\text{ k/in.}$ 7% less deflection because warping dominates deflection

Result Example 2 - Stiffness

$$G' = 29.0\,\frac{\text{k}}{\text{in.}}$$

Example 3: Available Diaphragm Shear Strength With Wind Uplift Where the Support is Cold-Formed Steel Framing

Objective

For the diaphragm configuration in Example 1, calculate the nominal and available diaphragm shear strength per unit length in the presence of uplift using AISI S310 Chapter D where the support is cold-formed steel framing. Assume an ultimate design wind pressure of 50 psf, calculated using a Component and Cladding (C&C) method from either ASCE 7-10 or ASCE 7-16.

Calculation of Fastener Interaction

AISI S310, Section D3.1.2.1 provides separate interaction equations for ASD and LRFD methods. Both interaction equations provide a reduced nominal screw shear strength (resistance), P_{nft}, in the presence of tension, for both screw pull-over and screw pull-out.

The #12 screw support fastener has a diameter (d) of 0.216 inches, and a head diameter (d_w) of 0.500 inches.

ASD Solution using AISI S310-2016 and AISI S100 - 2016

The ASD uplift pressure is 0.6 times the ultimate design pressure, or 0.6(50) = 30 psf.

For the interaction of shear and pull-over:

$$P_{nft} = P_{nf} \qquad \text{Eq. D3.1.2.1-1}$$

$$\left(\frac{P_{nft}}{\Omega_d P_{nf}}\right)+\left(\frac{0.71T}{P_{nov}}\right)=\frac{1.1}{\Omega} \qquad \text{Eq. D3.1.2.1-2}$$

where

Ω = 2.35

Ω_d = 2.00

T = 30 psf x 4 foot deck span / 1 screws per foot of width = 120 lbs. per screw

P_{nf} = 1.17 kips (Example 1)

P_{nov} = $1.5t_1d'_wF_{u1}$ AISI S100-10 Eq. J4.4.2-1

= 1.5(0.036)(0.500)(62) = 1.67 kips

P_{nts} = 2.7 kips (Example 1)

P_{nft} = 1.17 kips maximum with uplift

$$\left(\frac{P_{nft}}{2.00(1.17)}\right)+\left(\frac{0.71(0.120)}{1.67}\right)=\frac{1.1}{2.35} \qquad P_{nft} = 0.976 \text{ kips}$$

Result for Pullover: The value of P_{nft}, reduced by uplift is 0.976 kips.

For the interaction of shear and pull-out

$$P_{nft} = P_{nf}$$ Eq. D3.1.2.1-5

$$\left(\frac{P_{nft}}{\Omega_d P_{nf}}\right)+\left(\frac{T}{P_{not}}\right)=\frac{1.15}{\Omega}$$ Eq. D3.1.2.1-6

where

Ω = 2.55

Ω_d = 2.00

T = 120 lbs. per screw

P_{nf} = 1.17 kips (Example 1)

P_{not} = $0.85 t_c d F_{u2}$ AISI S100-10 Eq. J4.4.1-1

= 0.85(0.048)(0.216)(55) = 0.485 kips

P_{nts} = 2.7 kips (Example 1) 1)

P_{nft} = 1.17 kips maximum with uplift

$$\left(\frac{P_{nft}}{2.00(1.17)}\right)+\left(\frac{0.120}{0.485}\right)=\frac{1.15}{2.55} \qquad P_{nft} = 0.476 \text{ kips}$$

Result for Pullout: The value of P_{nft}, reduced by uplift is 0.476 kips.

$$P_{nft} = \min(0.976, 0.476, 2.7) = 0.476\text{ k}$$

Result: For purposes of calculating the diaphragm strength with uplift, P_{nft} = 0.476 kips

Calculate nominal diaphragm shear strength, S_{nf}:

Note: The simplification is that the interior support strength (P_{nf} =0.476 k) will be applied to the butt joint or end-lap strength in calculating S_{nf}.

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.827\text{ k}}{0.476\text{ k}} = 1.74$$ Connection strength ratio AISI S310 Eq. D1-6

$$\beta = 5(1.74) + 2(4)(0.556) + 4(0.556) = 15.4$$ AISI S310 Eq. D1-5

$$S_{ni} = [2(1.0)(0.871 - 1) + 15.4]\frac{0.476\text{ k}}{20\text{ ft}} = 0.360\text{ klf}$$ AISI S310 Eq. D1-1

$$S_{nc} = \left(\frac{(1.0\frac{1}{\text{ft}})^2(15.4)^2}{(20\text{ft})^2(1.0\frac{1}{\text{ft}})^2 + (15.4)^2}\right)^{0.5} 0.476 \quad \text{kips} = 0.290\text{ klf}$$ AISI S310 Eq. D1-2

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.360, 0.290) = 0.290\text{ klf}$

Controlled by corner screws at deck ends.

Ultimate design wind pressure of 50 psf reduces the nominal diaphragm strength from 0.532 klf to 0.290 klf in the Example 1 assembly.

ASD available strength = $\frac{0.290\text{ klf}}{2} = 0.145\text{ klf}$

LRFD Solution using AISI S310-2016 and AISI S100 - 2016

The LRFD uplift pressure is 1.0 times the ultimate design pressure, or 1.0(50) = 50 psf.

For the interaction of shear and pull-over:

$$P_{nft} = P_{nf}$$ Eq. D3.1.2.1-3

$$\left(\frac{\phi_d P_{nft}}{P_{nf}}\right) + \left(\frac{0.71\overline{T}}{P_{nov}}\right) = 1.1\phi$$ Eq. D3.1.2.1-4

where

ϕ = 0.65

ϕ_d = 0.80

$\overline{T}$ = 50 psf x 4 foot deck span / 1 screws per foot of panel width = 200 lbs. per screw

P_{nf} = 1.17 kips (Example 1)

P_{nov} = $1.5t_1d'_wF_{u1}$ AISI S100-10 Eq. J4.4.2-1

= 1.5(0.036)(0.500)(62) = 1.67 kips

P_{nft} = 1.17 kips maximum with uplift

$$\left(\frac{0.80P_{nft}}{1.17}\right)+\left(\frac{0.71(0.200)}{1.67}\right)=1.1(0.65) \qquad P_{nft} = 0.921 \text{ kips}$$

Result for Pullover: The value of P_{nft}, reduced by uplift is 0.921 kips.

For the interaction of shear and pull-out

$$P_{nft} = P_{nf}$$ Eq. D3.1.2.1-7

$$\left(\frac{\phi_d P_{nft}}{P_{nf}}\right)+\left(\frac{\overline{T}}{P_{not}}\right)=1.15\phi$$ Eq. D3.1.2.1-8

where

ϕ = 0.60

ϕ_d = 0.80

$\overline{T}$ = 200 lbs. per screw

P_{nf} = 1.17 kips (Example 1)

P_{not} = $0.85t_cdF_{u2}$ AISI S100-10 Eq. J4.4.1-1

= 0.85(0.048)(0.216)(55) = 0.485 kips

P_{nft} = 1.17 kips maximum with uplift

$$\left(\frac{0.80P_{nft}}{1.17}\right)+\left(\frac{0.200}{0.485}\right)=1.15(0.60) \qquad P_{nft} = 0.406 \text{ kips}$$

Result for Pullout: The value of P_{nft}, reduced by uplift is 0.406 kips.

$$P_{nft} = \min(0.921, 0.406, 2.7) = 0.406 \text{ k}$$

Result: For purposes of calculating the diaphragm strength with uplift, P_{nft} = 0.406 kips

Calculate nominal diaphragm shear strength, S_{nf}:

Note: The simplification is that the interior support strength (P_{nf} =0.406 k) will be applied to the butt joint or end-lap strength in calculating S_{nf}.

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.827\text{ k}}{0.406\text{ k}} = 2.04$$ Connection strength ratio AISI S310 Eq. D1-6

$$\beta = 5(2.04) + 2(4)(0.556) + 4(0.556) = 16.9$$ AISI S310 Eq. D1-5

$$S_{ni} = [2(1.0)(0.871 - 1) + 16.9]\frac{0.406\text{ k}}{20\text{ ft}} = 0.338\text{ klf}$$ AISI S310 Eq. D1-1

$$S_{nc} = \left(\frac{(1.0\frac{1}{\text{ft}})^2(16.9)^2}{(20\text{ft})^2(1.0\frac{1}{\text{ft}})^2 + (16.9)^2}\right)^{0.5} 0.406\text{ kips} = 0.262\text{ klf}$$ AISI S310 Eq. D1-2

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.338, 0.262) = 0.262\text{ klf}$

Controlled by corner screws at deck ends

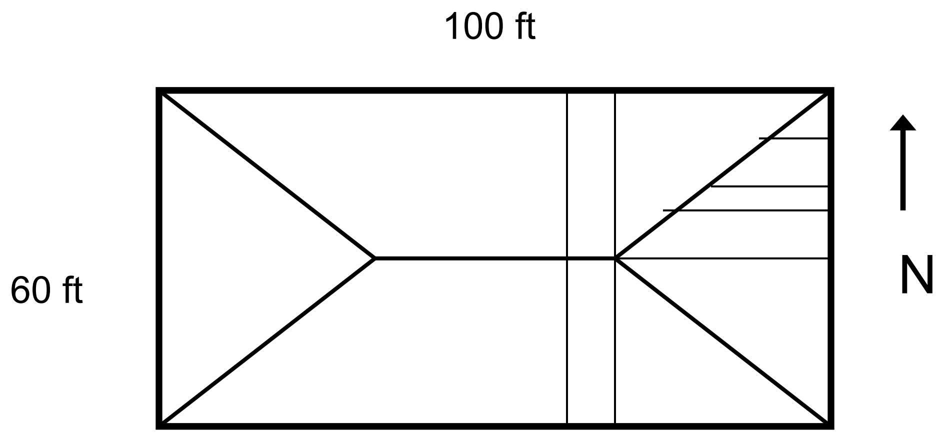
Ultimate design wind pressure of 50 psf reduces the nominal diaphragm strength from 0.532 klf to 0.262 klf in the Example 1 assembly. A 36/7 pattern will reduce the magnitude of this reduction.

The LRFD methods predicts a smaller S_{nf} relative to ASD, 0.262 klf vs. 0.290 klf.

LRFD available diaphragm strength = $0.80\,(0.262\ \text{klf}) = 0.210\ \text{klf}$

Example 4: Roof Deck (ASD)

Given:



Using the design tables, select deck, given the following conditions.

(1) Deck spans over roof trusses at 6'-00" on center

(2) Roof deck is 1-1/2" Wide Rib (WR) deck, 22 gage, F_y = 33 ksi. Self weight of deck and roofing is 12 psf.

(3) Roof trusses span in the north-south directions, with the truss top chord to be a minimum of 43 mils, F_u = 65 ksi. Truss top chord is 1-3/4 inches wide. Roof pitch is 4/12.

(4) Loads, combined per ASCE 7-10.

  (a) Dead Load of Deck and Roofing = 12 psf

  (b) Roof Live Load = 20 psf

(5) Per ASCE 7-10, the edge zone is 6 foot wide

  (a) The Component and Cladding (C&C) uplift pressures are:

    (i) Zone 1 = 40 psf uplift, 25 psf down (interior)

    (ii) Zone 2 = 70 psf uplift, 25 psf down (edge)

    (iii) Zone 3 = 100 psf uplift, 25 psf down (corner)

  (b) The Main Wind Force Resisting System (MWFRS) loads are:

    (i) Zone 2E = 46 psf uplift applied on zone width of "2a" or 12 feet.

    (ii) Zone 2 = 32 psf uplift

    (iii) Roof diaphragm load is 500 plf.

(6) Use ANSI/SDI RD-2017 for roof deck design, and ANSI/AISI S310-16 for diaphragm design.

Solution:

Controlling Gravity Loading:	Dead + Roof Live	=	32 psf
Controlling Uplift Loading (C&C):	0.6 Wind + 0.6 Dead	=	-16.8 psf (Zone 1)
		=	-34.8 psf (Zone 2)
		=	-52.8 psf (Zone 3)
Controlling Down Wind Load (C&C):		=	+13.6 psf (All)
Controlling Uplift Loading (MWFRS)	0.6 Wind + 0.6 Dead	=	-12.0 psf (Zone 2)
		=	-20.4 psf (Zone 2E)
Controlling Diaphragm Load (MWFRS)	0.6 Wind	=	300 plf

Check roof deck span using Roof Deck Design Manual (RDDM) Tables 4.

Maximum construction span =

6'-11" for multiple spans (RDDM Table 4.8) OK

5'-08" for single spans (Single span deck near ridge will need additional framing)

Maximum gravity load =

63 psf (single span) > 32 psf (RDDM Table 4.1) OK

Maximum uplift load =

78 psf (multiple span) > -52.8 psf for Zone 3 (RDDM Table 4.2) OK

66 psf (single span) > -16.8 psf for Zone 1 (RDDM Table 4.2) OK

Design the Roof Diaphragm for Wind Loading on East-West Face

Assume that the 60 foot end walls are braced to carry the roof diaphragm shear to the foundations, and assume that the full 60 foot length of the wall acts as a drag strut.

Maximum nominal diaphragm force = 500 plf (100 ft / 2) = 25,000 lbs.

ASD diaphragm force = (0.6) 25,000 = 15,000 lbs

Nominal diaphragm shear = 25,000 / 60 = 417 plf

ASD diaphragm shear = (0.6) 417 = 250 plf

Check diaphragm shear capacity without uplift.

Assuming #12 support screws and #10 sidelap screws, and referring to Table 7-34;
Screws in a 36/7 pattern and (5) sidelap screws per span (or 1 per foot).

S_{nf} = 572 plf; Ω = 2.00 S_{nf} / Ω = 285 plf > 250 plf OK

S_{nb} = 3814 plf; Ω = 2.00 S_{nb} / Ω = 1907 plf > 250 plf OK

A computer spreadsheet solution is as follows:

Sidelaps per Span	Nominal Shear Strength, S_{nf}, plf — Span, ft									
	4	4.5	5	5.5	6	6.5	7	7.5	8	
0	383	338	302	273	249	228	211	195	182	
1	481	431	386	349	318	292	270	251	234	DDM04
2	568	512	466	425	388	357	330	307	286	
3	649	588	536	493	455	421	390	362	338	
4	724	659	603	556	515	479	448	418	391	
5	793	725	667	616	572	533	499	469	442	
6	856	787	726	673	626	585	549	517	488	

The support screw spacing is less than the 18 inch maximum permitted by the ANSI/SDI RD-2017 Standard and the sidelap screw spacing is less than the 36 inch maximum permitted by the same Standard.

<u>Check diaphragm shear capacity with uplift.</u>

Check the diaphragm with #12 support screws in a 36/7 pattern and (5) sidelap screws per span.

Using a computer spreadsheet to calculate the interaction of the diaphragm shear with the Zone 2E MWFRS uplift of 20.4 psf, the ASD allowable diaphragm shear is 213 plf, which is less than the demand of 250 plf.

Net Uplift = 20.4 psf										
P_{nft}, kips	0.669	0.650	0.631	0.612	0.593	0.574	0.555	0.536	0.517	
	ASD Interactive Allowable Shear Strength, S_{nf}/W, plf									
Sidelaps per	Span, ft									
Span	4	4.5	5	5.5	6	6.5	7	7.5	8	
0	141	121	105	92	81	72	64	58	52	ASD
1	182	160	140	124	111	99	90	81	74	
2	218	193	173	155	140	127	115	105	96	
3	252	225	202	183	166	152	140	129	118	
4	282	254	229	208	190	175	161	149	138	
5	310	280	254	232	213	196	182	169	157	
6	335	304	278	255	235	217	201	187	175	

Try a 36/14 pattern in the 12 foot edge zone;

Net Uplift = 20.4 psf										
P_{nft}, kips	0.745	0.735	0.726	0.716	0.707	0.697	0.688	0.678	0.669	
	ASD Interactive Allowable Shear Strength, S_{nf}/W, plf									
Sidelaps per	Span, ft									
Span	4	4.5	5	5.5	6	6.5	7	7.5	8	
0	313	273	241	215	193	175	159	146	134	ASD
1	358	313	277	247	223	202	185	169	156	
2	396	352	312	280	253	230	210	193	178	
3	434	387	348	312	282	257	235	217	200	
4	470	420	379	344	312	284	261	240	223	
5	505	453	409	373	341	312	286	264	245	
6	539	484	439	400	367	339	311	288	267	

In the 12 foot edge zone, a 36/14 support fastener pattern, with (5) sidelap screws per span has an ASD capacity of 341 plf, which is greater than the demand of 250 plf.

Outside the 12 foot edge zone, the MWFRS ASD uplift is 12.0 psf, and the diaphragm shear is 250 (50-12)/50 = 190 plf. A 36/7 pattern with (5) sidelap screws per span has an ASD allowable diaphragm shear is 229 plf, which is greater than the demand of 190 plf.

Net Uplift = 12 psf										
P_{nft}, kips	0.731	0.720	0.709	0.698	0.687	0.675	0.664	0.653	0.642	
	ASD Interactive Allowable Shear Strength, S_{nf}/W, plf									
Sidelaps per	Span, ft									
Span	4	4.5	5	5.5	6	6.5	7	7.5	8	
0	154	134	118	105	94	85	77	70	64	
1	195	173	153	137	124	112	102	94	86	
2	232	207	187	169	153	139	128	117	109	ASD
3	266	239	216	197	181	166	153	141	131	
4	298	269	244	223	205	190	176	164	153	
5	326	296	271	248	229	212	197	184	173	
6	353	322	295	272	252	234	218	204	191	

The support screw spacing is less than the 18 inch maximum permitted by the ANSI/SDI RD-2017 Standard and the sidelap screw spacing is less than the 36 inch maximum permitted by the same Standard.

Check the anchorage of the steel deck for C&C uplift pressures:

The nominal uplift resistance U, in psf, can be calculated for a given fastener group using the following equation.

$$U = \frac{kP}{CL}$$

Where: L = 6 feet

P = min (538 pounds per Table 2 or 793 pounds per Table 3)

C = 3 (Table 8)

k = 6 for 36/7 fastener pattern; 12 for 36/14 fastener pattern (Table 8)

Ω = 3.0 (for screws)

For 36/7 pattern

$$U = \frac{kP}{CL} = \frac{6(538)}{3(6)} = 179\text{psf}$$

U/ Ω = 179/3.0 = 60 psf

For 36/14 pattern

$$U = \frac{kP}{CL} = \frac{12(538)}{3(6)} = 358\text{psf}$$

U/ Ω = 358/3.0 = 119 psf

Maximum ASD C&C uplift is 52.8 psf, so a 36/7 fastener pattern would work over the entire roof area. A 36/14 fastener pattern would also work.

Design the Roof Diaphragm for Wind Loading on North-South Face

Assume that the 100 foot side walls are braced to carry the roof diaphragm shear to the foundations, and assume that the full 100 foot length of the wall acts as a drag strut.

Maximum nominal diaphragm force = 500 plf (60 ft / 2) = 15,000 lbs.

ASD diaphragm force = (0.6) 15,000 = 9,000 lbs

Nominal diaphragm shear = 15,000 / 100 = 150 plf

ASD diaphragm shear = (0.6) 417 = 90 plf

Check diaphragm shear capacity without uplift.

Checking the previously designed 36/7 pattern and (5) sidelap screws per span (or 1 per foot).

S_{nf} = 572 plf; Ω = 2.00 S_{nf} / Ω = 285 plf > 90 plf OK

S_{nb} = 3814 plf; Ω = 2.00 S_{nb} / Ω = 1907 plf > 90 plf OK

Check diaphragm shear capacity with uplift.

In the 12 foot edge zone, the previously selected 36/14 support fastener pattern, with (5) sidelap screws per span has an ASD capacity of 341 plf, which is greater than the demand of 90 plf.

Outside the 12 foot edge zone, the MWFRS ASD uplift is 12.0 psf, and the diaphragm shear is 90 (30-12)/30 = 54 plf. A 36/7 pattern with (5) sidelap screws per span has an ASD allowable diaphragm shear is 229 plf, which is greater than the demand of 54 plf.

Check the anchorage of the steel deck for C&C uplift pressures:

The nominal uplift resistance U, in psf, can be calculated for a given fastener pattern using the following equation.

$$U = \frac{kP}{CL}$$

Where: L = 6 feet

P = min (538 pounds per Table 2 or 793 pounds per Table 3)

C = 3 (Table 8)

k = 6 for 36/7 fastener pattern; 12 for 36/14 fastener pattern (Table 8)

Ω = 3.0 (for screws)

For 36/7 pattern

$$U = \frac{kP}{CL} = \frac{6(538)}{3(6)} = 179\text{psf}$$

U/ Ω = 179/3.0 = 60 psf

For 36/14 pattern

$$U = \frac{kP}{CL} = \frac{12(538)}{3(6)} = 358\text{psf}$$

U/ Ω = 358/3.0 = 119 psf

Maximum ASD C&C uplift is 52.8 psf, so a 36/7 fastener pattern would work over the entire roof area. A 36/14 fastener pattern would also work.

Design the Roof Diaphragm Boundary (Edge) Fasteners

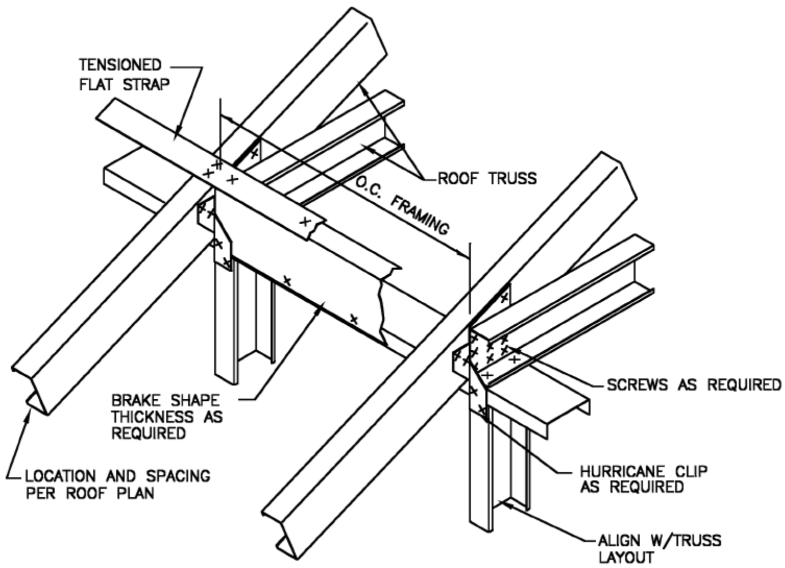
The boundary fasteners must be designed to resist the boundary force and carry that diaphragm boundary force into the drag strut.

North-South Endwalls

Along the north-south endwalls, the selected diaphragm fastening pattern (36/14) will be sufficient to carry the design diaphragm force into the endwall framing, assuming that the framing that the deck is attached to is the same as the trusses.

East-West Sidewalls

The truss tails are usually not sufficient to transfer the chord force into the wall framing. One possible detail is to use a flat strap and blocking between the truss tails.



In this case, the connection of the deck to the flat strap must have sufficient strength to transfer the diaphragm shear into the strap.

Assuming an ASD demand of 90 plf, a 43 mil, F_y = 45 ksi flat strap, and #10 screws:

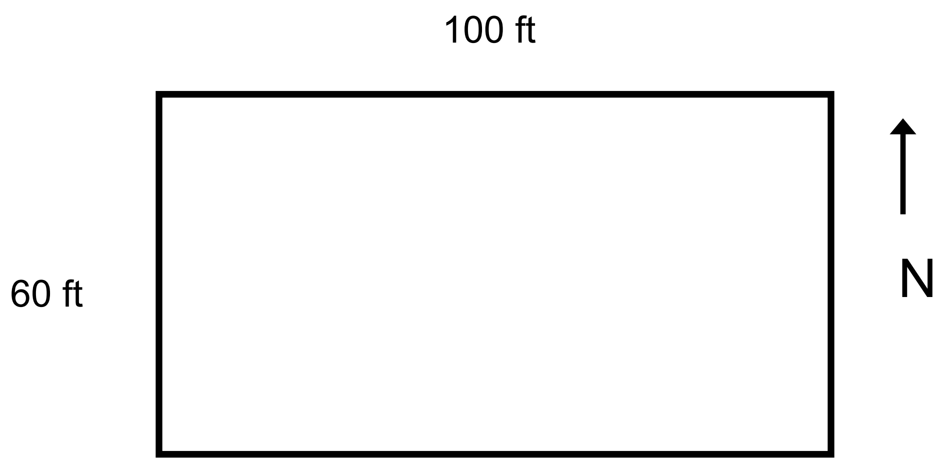
> P_n = 681 pounds; Ω = 3.00 P_n / Ω = 227 pounds
> 227 / 90 = 2.52 feet = 30 inches maximum.
> Install #10 screws at 18 inch spacing to comply with RD-2017.

Other Notes:

This example does not cover diaphragm penetrations, irregular diaphragms, or zoning of diaphragm fasteners. Refer to the SDI Diaphragm Design Manual (DDM03) for additional information.

Example 5: Floor Deck (ASD)

Given:



Using the design tables, select deck, given the following conditions.

(1) 3-span deck, deck spans over floor trusses at 6'-00" on center

(2) Floor deck is 1-1/2", galvanized, 20 gage, non-composite floor deck, F_y = 33 ksi. Deck is covered with 3000 psi normal weight concrete, total thickness is 4" (2-1/2" cover over top of deck). Self weight of deck and concrete is 40 psf.

(3) Floor trusses span in the north-south directions, with the truss top chord to be a minimum of 43 mils, F_u = 65 ksi. Truss top chord is 2-1/2 inches wide. Trusses are supported by intermediate bearing walls which do not act as shearwalls.

(4) Loads, combined per ASCE 7-10.

  (a) Dead Load of Deck and Concrete = 40 psf

  (b) Additional Dead = 10 psf (total)

  (c) Floor Live = 50 psf

(5) Wind loading on floor diaphragm is 900 plf, applied on the east-west face.

(6) Use ANSI/SDI NC-2017 for floor deck design, and ANSI/AISI S310-16 for diaphragm design.

Solution:

Controlling Gravity Loading:	Dead + Floor Live Load (D + L)	=	100 psf
Controlling Diaphragm Shear:	0.6 Wind Load (0.6 W)	=	450 plf

Check floor deck construction span using Floor Deck Design Manual (FDDM) Table 2F, with a construction live load of 20 psf.

Maximum unshored clear span = 6'-05"

Actual clear span = 6'-00" - 2.5" = 5'-9.5" OK

Design the floor slab to carry the superimposed load of 10 psf plus 50 psf. Because the deck is galvanized and is considered to be a permanent form, only the superimposed 60 psf load need be carried by the slab.

By analysis of the concrete slab as a 3-span slab using ACI 318, 6x6-W2.9xW2.9 welded wire reinforcement, located at the mid-depth of the 2-1/2" slab, is sufficient. Refer to the SDI FDDM for additional slab design information.

Design the Floor Diaphragm

Assume that the 60 foot end walls are braced to carry the floor diaphragm shear to the foundations, and assume that the full 60 foot length of the wall acts as a drag strut.

Maximum nominal diaphragm force = 900 plf (100 ft / 2) = 45,000 lbs.

ASD diaphragm force = (0.6) 45,000 = 27,000 lbs

Nominal diaphragm shear = 45,000 / 60 = 750 plf

ASD diaphragm shear = (0.6) 750 = 450 plf

<u>Option 1: Assume all diaphragm shear is carried by the deck alone.</u>

Assuming #12 support screws and #10 sidelap screws.

Screws in a 36/14 pattern and (4) sidelap screws per span.

S_{nf} = 980 plf; Ω = 2.00 S_{nf} / Ω = 490 plf > 450 plf OK

S_{nb} = 5103 plf; Ω = 2.00 S_{nf} / Ω = 2551 plf > 450 plf OK

The support screw spacing is less than the 18 inch maximum permitted by the ANSI/SDI NC-2017 Standard and the sidelap screw spacing is less than the 36 inch maximum permitted by the same Standard.

<u>Option 2: Assume all diaphragm shear is carried by the concrete alone.</u>

Referring to Table 6.1, with 3000 psi normal weight concrete, and a concrete cover of 2.5 inches;

S_{nf} = 4904 plf; Ω = 3.25 S_{nf} / Ω = 1510 plf > 450 plf OK

The support screw spacing must not be greater than the 18 inch maximum permitted by the ANSI/SDI NC-2017 Standard and the sidelap screw spacing must not exceed the 36 inch maximum permitted by the same Standard. Therefore, use a 36/3 support fastener pattern and 1 sidelap screw per span.

<u>Option 3: Assume diaphragm shear is carried by the deck and concrete combined.</u>

S_n = 980 plf (deck) + 4904 plf (concrete) = 5884 plf

However, the contribution of the deck cannot exceed 25% of the total nominal diaphragm strength;

$S_{n\ deck} \leq$ 0.25 (5884) = 1471 plf > 980 plf OK

If the deck strength exceeded the limit, then reduce the contribution of the deck appropriately.

The support screw spacing must not be greater than the 18 inch maximum permitted by the ANSI/SDI NC-2017 Standard and the sidelap screw spacing must not exceed the 36 inch maximum permitted by the same Standard. Therefore, use a 36/3 support fastener pattern and 1 sidelap screw per span.

Design the Floor Diaphragm Boundary Fasteners

The boundary fasteners must be designed to resist the boundary force and carry that diaphragm boundary force into the drag strut.

Nominal diaphragm shear (S_n)= 45,000 / 60 = 750 plf

From Table 5.2, a #12 screw in 20 gage, F_y = 33 ksi deck and a 43 mil, F_u = 65 ksi drag strut, has a nominal shear resistance, P_n = 1215 pounds.

Maximum fastener spacing ≤ 1215 / 750 = 1.62 feet = 19.4 inches; however the spacing must not be greater than the 18 inch maximum permitted by the ANSI/SDI NC-2017 Standard. Therefore, use a 36/3 support fastener pattern at the boundary, but not less than what is required by the diaphragm design.

This 36/3 minimum fastener pattern at the boundaries applies for Options 2 and 3. Option 1 will require the 36/14 pattern required for the diaphragm design.

Diaphragm Stiffness

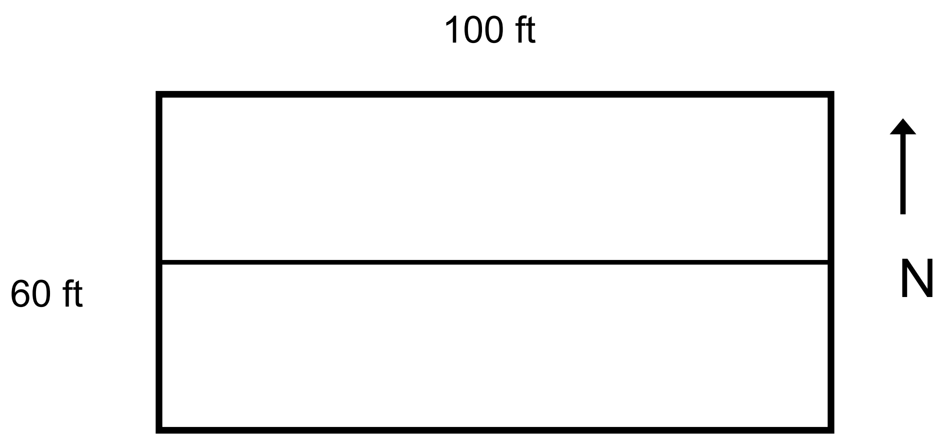
Regardless of the option chosen to design the diaphragm for strength, a concrete filled diaphragm should have the stiffness calculated considering the concrete. This calculation is covered in DDM04.

Other Notes

1. This example does not cover diaphragm penetrations, irregular diaphragms, or zoning of diaphragm fasteners. Refer to the SDI Diaphragm Design Manual (DDM03) for additional information.
2. This diaphragm would also need to be checked for loading in the east-west direction.
3. The mechanics of design of a floor diaphragm for seismic forces are similar. The factor of safety and resistance factor are different for seismic loading. The governing building code may contain additional detailing requirements specific to seismic resistance.

Example 6: Gable Roof with Open Ridge

Given:



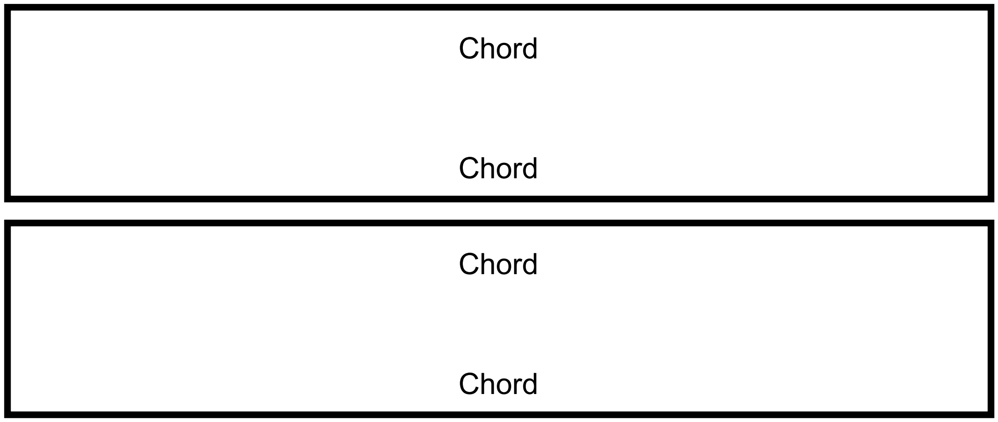
Normally, a ridge plate would be installed over the ridge of the roof to provide continuity of the diaphragm,. However, for purposes of providing a ventilated attic, the roofing design will require that the ridge of a roof be left open. In this case, the roof will behave as 2 separate 30 foot by 100 foot diaphragms, rather than a single 60 foot by 100 foot diaphragm. In this case, consider the forces that develop in the diaphragm .

Assume the load being transferred into the diaphragm is 500 plf.

Solution:

Design the Roof Diaphragm for Wind Loading on East-West Face

Assume that the roof acts as 2 independent diaphragms, each 30 foot deep and 100 foot long. 60 foot end walls are braced to carry the roof diaphragm shear to the foundations, and assume that the full 60 foot length of the wall acts as a drag strut.



Maximum total nominal diaphragm force = 500 plf (100 ft / 2) = 25,000 lbs.

For each diaphragm

Nominal diaphragm shear = (25,000/2) / 30 = 417 plf

The 417 plf nominal diaphragm shear is used to design the steel deck diaphragm, and is transferred into each endwall. In this case, there is no difference in the design of the diaphragm from when there is a ridge plate, except that the edge of each diaphragm at the ridge must be considered as a chord. A simple solution would be to consider the last full rib of the deck as the chord.

Design the Roof Diaphragm for Wind Loading on North-South Face

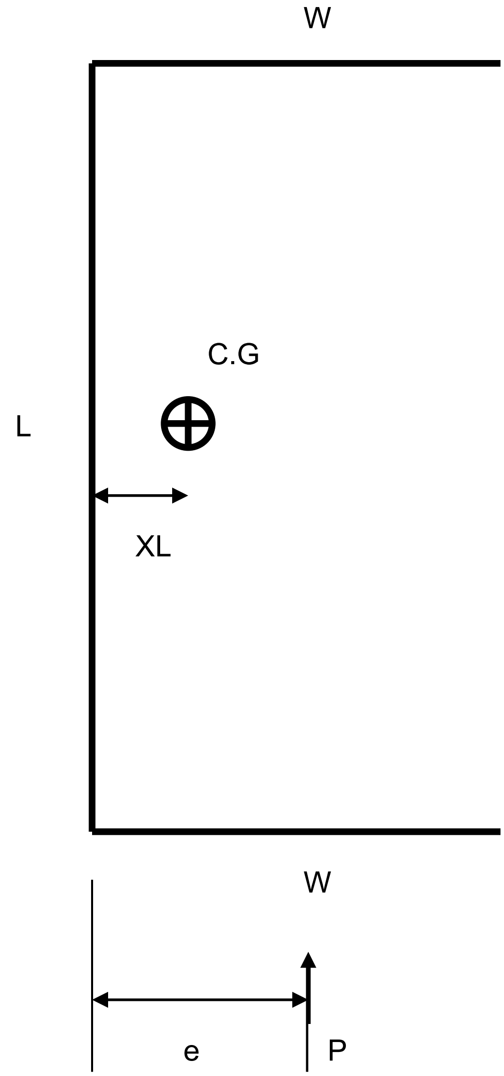
Assume that the 100 foot side walls are braced to carry the roof diaphragm shear to the foundations, and assume that the full 100 foot length of the wall acts as a drag strut.

Maximum nominal diaphragm force = 500 plf (60 ft / 2) = 15,000 lbs.

For each diaphragm

Nominal diaphragm shear = (15,000/2) / 100 = 150 plf

This is the same as when there is a ridge plate to create a single roof diaphragm, except that now there is torsion introduced into the diaphragm because the lines of support for the diaphragm are not symmetric.



Using statics, the forces in this unsymmetric diaphragm can be solved as follows:

Force in Side L	=	P	=	P / L (pounds per foot)
Force in Side W	=	P(e-XL) / L	=	P(e-XL) / W L (pounds per foot)
XL	=	W^2 / (L+2W)		
L	=	100 feet		
W	=	30 feet		
P	=	15,000 pounds		
e	=	15 feet		
XL	=	5.62 feet		

Force in L	=	15,000 pounds	=	150 plf		
Force in W	=	(15,000)(15-5.62)/(100)	=	1407 pounds	=	47 plf

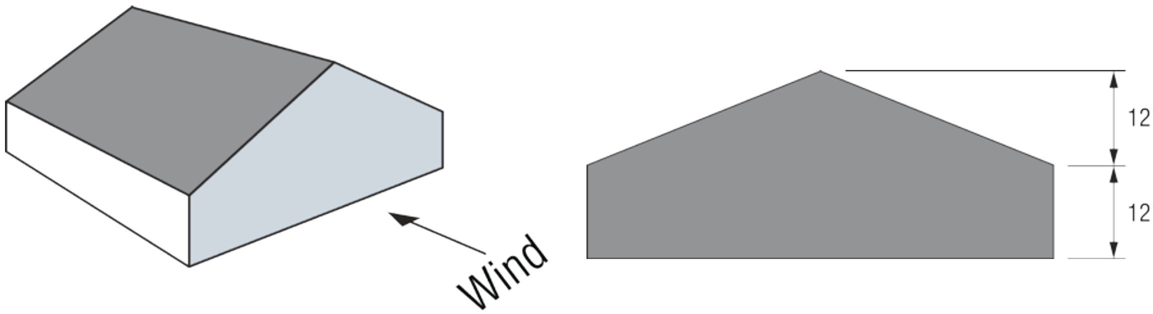
The torsion in the diaphragm results in an opposing force couple in the transverse walls that must be taken out into the drag struts at those walls.

NOTE 1: The location of the centroid of the walls is determined by statics, and the two transverse walls do not have to be of equal length.

NOTE 2: For both wind and seismic loading, the load P is located at the centroid of the load application.

Example 7: Loads on Diaphragm - Gable Roof Loaded on Endwall

Given:



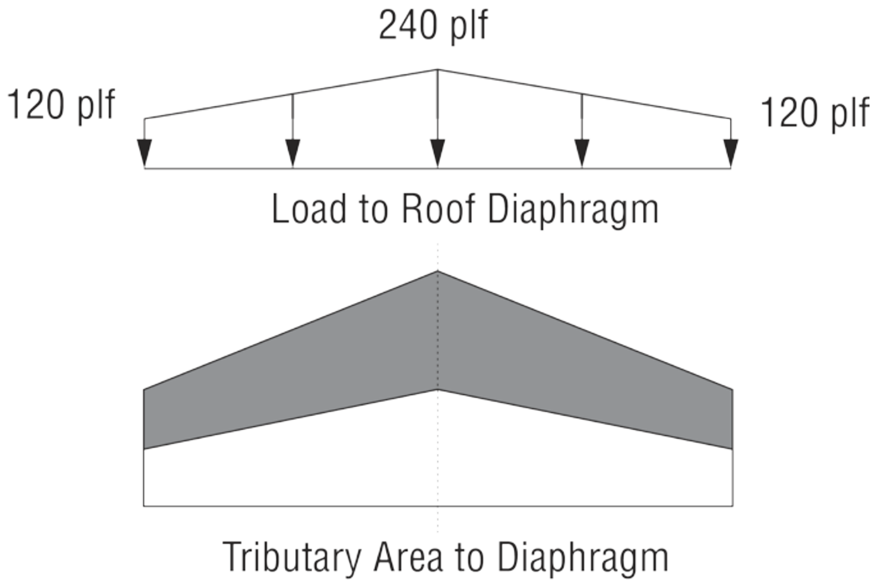
Assuming a 20 psf wind pressure applied to the gable end wall, determine the force that is applied to the diaphragm.

Solution:

The load that is applied to the diaphragm depends on how the gable end wall is framed.

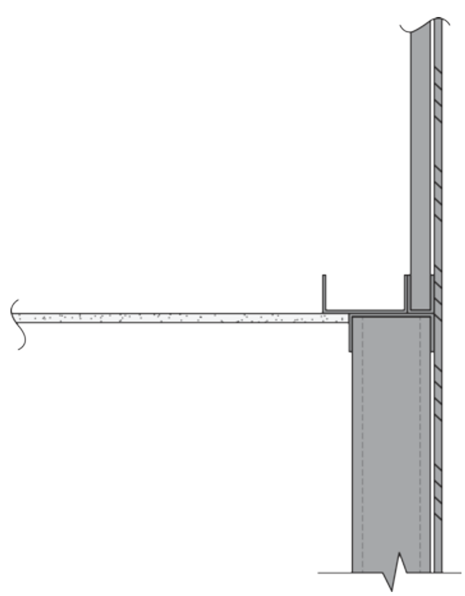
Case A: Full height gable end wall studs

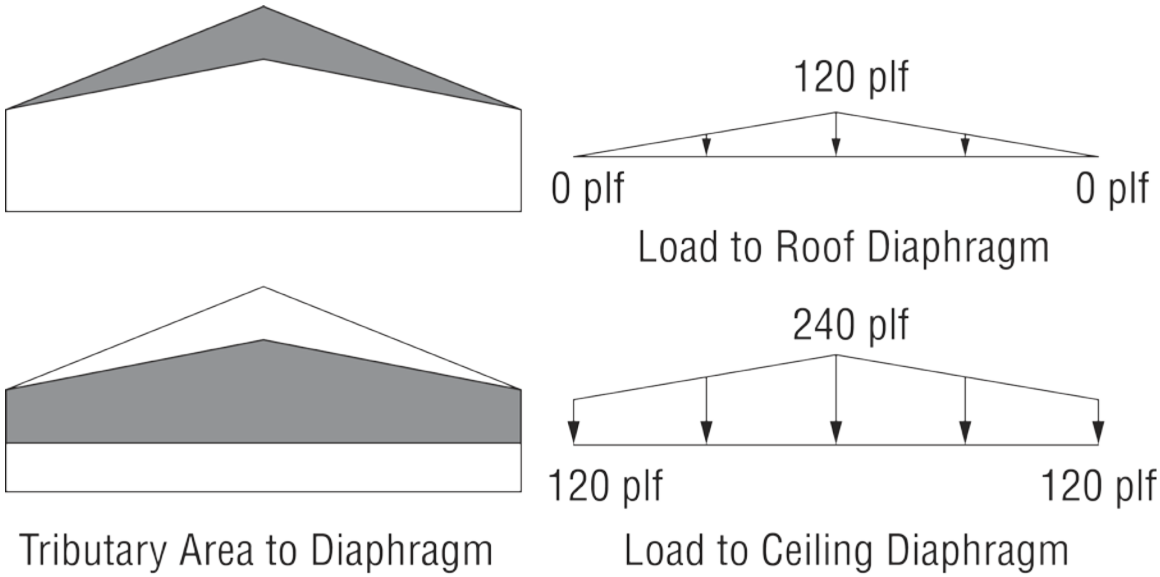
The preferred structural solution for gable end walls is where the end wall is framed with full height studs, also referred to as "balloon framing." The studs span from slab to roof edge and the load into the diaphragm is determined by the area tributary to the slab and the roof.



Case B: Uniform height wall with gable end truss braced by gypsum board ceiling

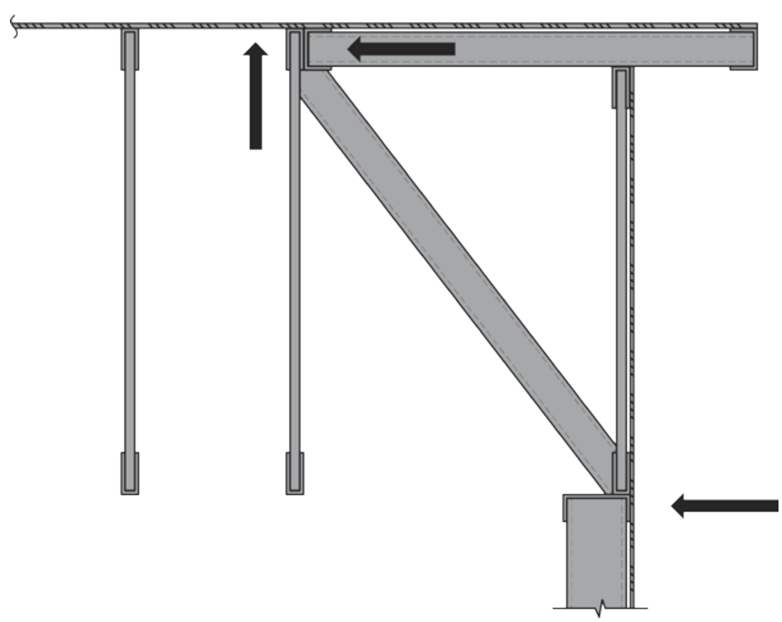
An alternate wall framing solution is to frame the end wall to the same height as the side walls and to top the wall with a gable end truss. This method is referred to as a "platform framed gable end. The resulting "hinge" at the top of the platform framed wall is braced by a ceiling diaphragm, usually the gypsum board ceiling. Similar to Case A, the loads into the diaphragms are determined by tributary areas.

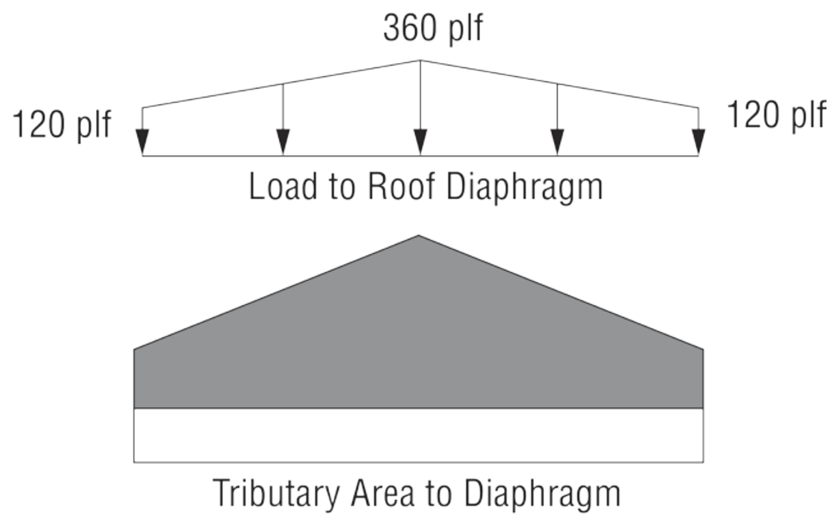




Case C: Uniform height wall with gable end truss with kickers to roof

An alternate wall framing solution is to frame the end wall to the same height as the side walls and to top the wall with a gable end truss. This requires a brace to transfer the load at the top of wall "hinge point" into the roof diaphragm. This is usually accomplished by an engineered system of kickers that must resist both tension and compression, depending on if the wall is windward or leeward.




360 plf
120 plf
120 plf
Load to Roof Diaphragm
Tributary Area to Diaphragm

FASTENER AND FRAMING TABLES

SECTION 6

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Section 6.1 List of Tables

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Table 1 Screw Dimensional and Tensile Strength
ASD ($\Omega = 3.00$) LRFD ($\Phi = 0.50$)

Screw Size	Major Diameter d (inches)	Head or Washer Diameter d_w (inches)	Average Tested Tensile Strength (lbs)
#10	**0.190**	0.400 or 0.415	2560
#12	**0.216**	0.400 or 0.430	3620
#14	**0.240**	0.480 or 0.520	4432
1/4 inch	**0.250**	0.480 or 0.520	4810

Table 2 Screw Nominal Pull-out Strength; P_n (Pounds)
ASD ($\Omega = 3.00$) LRFD ($\Phi = 0.50$)

			#10 Screw			#12 Screw			#14 Screw		
		Design	Framing Tensile Strength; F_u (ksi)								
Framing Gage	**Mil**	**thickness (inches)**	**45**	**62**	**65**	**45**	**62**	**65**	**45**	**62**	**65**
22	**27**	**0.0283**	206	283	297	234	322	338	260	358	375
20	**33**	**0.0346**	251	346	363	286	394	413	318	438	459
18	**43**	**0.0451**	328	452	473	373	513	538	414	570	598
16	**54**	**0.0566**	411	567	594	468	644	675	520	716	751
14	**68**	**0.0713**	518	714	748	589	812	851	655	902	945
12	**97**	**0.1017**	739	1018	1068	840	1158	1214	934	1286	1349

Table 3 Screw Nominal Pull-over Strength; P_n (Pounds)

ASD (Ω = 3.00) LRFD (Φ=0.50)

		Washer = 0.400 inches			Washer = 0.430 inches			Washer = 0.480 inches		
		Deck Tensile Strength; F_u (ksi)								
Deck Gage	Design thickness (inches)	45	62	65	45	62	65	45	62	65
22	0.0295	797	1097	1151	856	1180	1237	956	1317	1381
20	0.0358	967	1332	1396	1039	1432	1501	1160	1598	1675
18	0.0474	1280	1763	1849	1376	1896	1987	1536	2116	2218
16	0.0598	1615	2225	2332	1736	2391	2507	1938	2669	2799

Table 4 Sidelap Screw Nominal Shear Strength; P_n (Pounds)
ASD (Ω = 3.00) LRFD (Φ=0.50)

#10 Screw											
22 ga	20 ga	18 ga	16 ga	22 ga	20 ga	18 ga	16 ga	22 ga	20 ga	18 ga	16 ga
Deck Tensile Strength; F_u (ksi)											
45	45	45	45	62	62	62	62	65	65	65	65
417	558	850	1205	575	769	1171	1660	603	806	1228	1740

#12 Screw											
22 ga	20 ga	18 ga	16 ga	22 ga	20 ga	18 ga	16 ga	22 ga	20 ga	18 ga	16 ga
Deck Tensile Strength; F_u (ksi)											
45	45	45	45	62	62	62	62	65	65	65	65
445	595	906	1285	613	820	1249	1770	643	859	1309	1855

Table 5.1 Support Screw Nominal Shear Strength; P_n (Pounds)
ASD (Ω = 3.00) LRFD (Φ=0.50)

# 10 Screw				Deck Gage, Design Thickness (inches), and Tensile Strength, F_u (ksi)							
				22	20	18	16	22	20	18	16
Framing				0.0295	0.0358	0.0474	0.0598	0.0295	0.0358	0.0474	0.0598
Gage	Mil	Design thickness	F_u	45	45	45	45	65	65	65	65
22	27	0.0283	45	392	392	392	392	392	392	392	392
20	33	0.0346	45	548	530	530	530	561	530	530	530
18	43	0.0451	45	681	796	789	789	858	833	789	789
16	54	0.0566	45	681	826	1094	1109	984	1142	1135	1109
14	68	0.0713	45	681	826	1094	1380	984	1194	1573	1578
12	97	0.1017	45	681	826	1094	1380	984	1194	1581	1994
22	27	0.0283	62	540	540	540	540	540	540	540	540
20	33	0.0346	62	681	731	731	731	760	731	731	731
18	43	0.0451	62	681	826	1087	1087	984	1106	1087	1087
16	54	0.0566	62	681	826	1094	1380	984	1194	1535	1528
14	68	0.0713	62	681	826	1094	1380	984	1194	1581	1994
12	97	0.1017	62	681	826	1094	1380	984	1194	1581	1994
22	27	0.0283	65	567	567	567	567	567	567	567	567
20	33	0.0346	65	681	766	766	766	791	766	766	766
18	43	0.0451	65	681	826	1094	1140	984	1149	1140	1140
16	54	0.0566	65	681	826	1094	1380	984	1194	1581	1602
14	68	0.0713	62	681	826	1094	1380	984	1194	1581	1994
12	97	0.1017	65	681	826	1094	1380	984	1194	1581	1994

Table 5.2 Support Screw Nominal Shear Strength; P_n (Pounds)
ASD (Ω = 3.00) LRFD (Φ=0.50)

# 12 Screw				Deck Gage, Design Thickness (inches), and Tensile Strength, F_u (ksi)							
				22	20	18	16	22	20	18	16
Framing				0.0295	0.0358	0.0474	0.0598	0.0295	0.0358	0.0474	0.0598
Gage	Mil	Design thickness	F_u	45	45	45	45	65	65	65	65
22	27	0.0283	45	418	418	418	418	418	418	418	418
20	33	0.0346	45	589	565	565	565	605	565	565	565
18	43	0.0451	45	774	858	841	841	939	901	841	841
16	54	0.0566	45	774	940	1191	1183	1118	1250	1222	1183
14	68	0.0713	45	774	940	1244	1569	1118	1357	1714	1698
12	97	0.1017	45	774	940	1244	1569	1118	1357	1797	2267
22	27	0.0283	62	576	576	576	576	576	576	576	576
20	33	0.0346	62	774	779	779	779	818	779	779	779
18	43	0.0451	62	774	940	1159	1159	1118	1193	1159	1159
16	54	0.0566	62	774	940	1244	1569	1118	1357	1651	1630
14	68	0.0713	62	774	940	1244	1569	1118	1357	1797	2267
12	97	0.1017	62	774	940	1244	1569	1118	1357	1797	2267
22	27	0.0283	65	604	604	604	604	604	604	604	604
20	33	0.0346	65	774	817	817	817	851	817	817	817
18	43	0.0451	65	774	940	1215	1215	1118	1240	1215	1215
16	54	0.0566	65	774	940	1244	1569	1118	1357	1720	1708
14	68	0.0713	62	774	940	1244	1569	1118	1357	1797	2267
12	97	0.1017	65	774	940	1244	1569	1118	1357	1797	2267

Table 5.3 Support Screw Nominal Shear Strength; P_n (Pounds)
ASD (Ω = 3.00) LRFD (Φ=0.50)

#14 Screw				Deck Gage, Design Thickness (inches), and Tensile Strength, F_u (ksi)							
				22	20	18	16	22	20	18	16
Framing				0.0295	0.0358	0.0474	0.0598	0.0295	0.0358	0.0474	0.0598
Gage	Mil	Design thickness	F_u	45	45	45	45	65	65	65	65
22	**27**	**0.0283**	**45**	441	441	441	441	441	441	441	441
20	**33**	**0.0346**	**45**	626	596	596	596	644	596	596	596
18	**43**	**0.0451**	**45**	860	914	887	887	1012	961	887	887
16	**54**	**0.0566**	**45**	860	1044	1264	1247	1243	1348	1299	1247
14	**68**	**0.0713**	**45**	860	1044	1382	1744	1243	1508	1841	1803
12	**97**	**0.1017**	**45**	860	1044	1382	1744	1243	1508	1996	2519
22	**27**	**0.0283**	**62**	607	607	607	607	607	607	607	607
20	**33**	**0.0346**	**62**	826	821	821	821	870	821	821	821
18	**43**	**0.0451**	**62**	860	1044	1222	1222	1229	1271	1222	1222
16	**54**	**0.0566**	**62**	860	1044	1382	1718	1243	1508	1754	1718
14	**68**	**0.0713**	**62**	860	1044	1382	1744	1243	1508	1996	2440
12	**97**	**0.1017**	**62**	860	1044	1382	1744	1243	1508	1996	2519
22	**27**	**0.0283**	**65**	637	637	637	637	637	637	637	637
20	**33**	**0.0346**	**65**	860	861	861	861	905	861	861	861
18	**43**	**0.0451**	**65**	860	1044	1281	1281	1243	1320	1281	1281
16	**54**	**0.0566**	**65**	860	1044	1382	1744	1243	1508	1826	1801
14	**68**	**0.0713**	**62**	860	1044	1382	1744	1243	1508	1996	2519
12	**97**	**0.1017**	**65**	860	1044	1382	1744	1243	1508	1996	2519

Table 6.1 Support Screw Flexibility; S_f (inches/kip)
#12 Support Screw

Support thickness mils	Support F_u	Deck Gage	Deck F_u	S_f
27	45	22	45	0.0194
27	45	20	45	0.0194
27	45	18	45	0.0194
27	45	16	45	0.0194
27	45	22	62	0.0194
27	45	20	62	0.0194
27	45	18	62	0.0194
27	45	16	62	0.0194
27	45	22	65	0.0194
27	45	20	65	0.0194
27	45	18	65	0.0194
27	45	16	65	0.0194
27	62	22	45	0.0194
27	62	20	45	0.0194
27	62	18	45	0.0194
27	62	16	45	0.0194
27	62	22	62	0.0194
27	62	20	62	0.0194
27	62	18	62	0.0194
27	62	16	62	0.0194
27	62	22	65	0.0194
27	62	20	65	0.0194
27	62	18	65	0.0194
27	62	16	65	0.0194
27	65	22	45	0.0194
27	65	20	45	0.0194
27	65	18	45	0.0194
27	65	16	45	0.0194
27	65	22	62	0.0194
27	65	20	62	0.0194
27	65	18	62	0.0194
27	65	16	62	0.0194
27	65	22	65	0.0194
27	65	20	65	0.0194
27	65	18	65	0.0194
27	65	16	65	0.0194

Support thickness mils	Support F_u	Deck Gage	Deck F_u	S_f
33	45	22	45	0.0136
33	45	20	45	0.0161
33	45	18	45	0.0161
33	45	16	45	0.0161
33	45	22	62	0.0153
33	45	20	62	0.0161
33	45	18	62	0.0161
33	45	16	62	0.0161
33	45	22	65	0.0154
33	45	20	65	0.0161
33	45	18	65	0.0161
33	45	16	65	0.0161
33	62	22	45	0.0076
33	62	20	45	0.0161
33	62	18	45	0.0161
33	62	16	45	0.0161
33	62	22	62	0.0136
33	62	20	62	0.0161
33	62	18	62	0.0161
33	62	16	62	0.0161
33	62	22	65	0.0140
33	62	20	65	0.0161
33	62	18	65	0.0161
33	62	16	65	0.0161
33	65	22	45	0.0076
33	65	20	45	0.0161
33	65	18	45	0.0161
33	65	16	45	0.0161
33	65	22	62	0.0132
33	65	20	62	0.0161
33	65	18	62	0.0161
33	65	16	62	0.0161
33	65	22	65	0.0136
33	65	20	65	0.0161
33	65	18	65	0.0161
33	65	16	65	0.0161

Screw flexibility calculated in accordance with AISI S310, Appendix D5.2.2

Table 6.2 Support Screw Flexibility; S_f (inches/kip)
#12 Support Screw

Support thickness mils	Support F_u	Deck Gage	Deck F_u	S_f
43	45	22	45	0.0076
43	45	20	45	0.0093
43	45	18	45	0.0141
43	45	16	45	0.0141
43	45	22	62	0.0108
43	45	20	62	0.0124
43	45	18	62	0.0141
43	45	16	62	0.0141
43	45	22	65	0.0113
43	45	20	65	0.0127
43	45	18	65	0.0141
43	45	16	65	0.0141
43	62	22	45	0.0076
43	62	20	45	0.0069
43	62	18	45	0.0141
43	62	16	45	0.0141
43	62	22	62	0.0076
43	62	20	62	0.0093
43	62	18	62	0.0141
43	62	16	62	0.0141
43	62	22	65	0.0076
43	62	20	65	0.0100
43	62	18	65	0.0141
43	62	16	65	0.0141
43	65	22	45	0.0076
43	65	20	45	0.0069
43	65	18	45	0.0141
43	65	16	45	0.0141
43	65	22	62	0.0076
43	65	20	62	0.0084
43	65	18	62	0.0141
43	65	16	62	0.0141
43	65	22	65	0.0076
43	65	20	65	0.0093
43	65	18	65	0.0141
43	65	16	65	0.0141

Support thickness mils	Support F_u	Deck Gage	Deck F_u	S_f
54	45	22	45	0.0076
54	45	20	45	0.0069
54	45	18	45	0.0073
54	45	16	45	0.0126
54	45	22	62	0.0076
54	45	20	62	0.0082
54	45	18	62	0.0109
54	45	16	62	0.0126
54	45	22	65	0.0076
54	45	20	65	0.0087
54	45	18	65	0.0112
54	45	16	65	0.0126
54	62	22	45	0.0076
54	62	20	45	0.0069
54	62	18	45	0.0060
54	62	16	45	0.0053
54	62	22	62	0.0076
54	62	20	62	0.0069
54	62	18	62	0.0073
54	62	16	62	0.0126
54	62	22	65	0.0076
54	62	20	65	0.0069
54	62	18	65	0.0083
54	62	16	65	0.0126
54	65	22	45	0.0076
54	65	20	45	0.0069
54	65	18	45	0.0060
54	65	16	45	0.0053
54	65	22	62	0.0076
54	65	20	62	0.0069
54	65	18	62	0.0061
54	65	16	62	0.0126
54	65	22	65	0.0076
54	65	20	65	0.0069
54	65	18	65	0.0073
54	65	16	65	0.0126

Screw flexibility calculated in accordance with AISI S310, Appendix D5.2.2

Table 6.3 Support Screw Flexibility; S_f (inches/kip)
#12 Support Screw

Support thickness mils	Support F_u	Deck Gage	Deck F_u	S_f
68	45	22	45	0.0076
68	45	20	45	0.0069
68	45	18	45	0.0060
68	45	16	45	0.0053
68	45	22	62	0.0076
68	45	20	62	0.0069
68	45	18	62	0.0063
68	45	16	62	0.0090
68	45	22	65	0.0076
68	45	20	65	0.0069
68	45	18	65	0.0070
68	45	16	65	0.0094
68	62	22	45	0.0076
68	62	20	45	0.0069
68	62	18	45	0.0060
68	62	16	45	0.0053
68	62	22	62	0.0076
68	62	20	62	0.0069
68	62	18	62	0.0060
68	62	16	62	0.0053
68	62	22	65	0.0076
68	62	20	65	0.0069
68	62	18	65	0.0060
68	62	16	65	0.0053
68	65	22	45	0.0076
68	65	20	45	0.0069
68	65	18	45	0.0060
68	65	16	45	0.0053
68	65	22	62	0.0076
68	65	20	62	0.0069
68	65	18	62	0.0060
68	65	16	62	0.0053
68	65	22	65	0.0076
68	65	20	65	0.0069
68	65	18	65	0.0060
68	65	16	65	0.0053

Support thickness mils	Support F_u	Deck Gage	Deck F_u	S_f
97	45	22	45	0.0076
97	45	20	45	0.0069
97	45	18	45	0.0060
97	45	16	45	0.0053
97	45	22	62	0.0076
97	45	20	62	0.0069
97	45	18	62	0.0060
97	45	16	62	0.0053
97	45	22	65	0.0076
97	45	20	65	0.0069
97	45	18	65	0.0060
97	45	16	65	0.0053
97	62	22	45	0.0076
97	62	20	45	0.0069
97	62	18	45	0.0060
97	62	16	45	0.0053
97	62	22	62	0.0076
97	62	20	62	0.0069
97	62	18	62	0.0060
97	62	16	62	0.0053
97	62	22	65	0.0076
97	62	20	65	0.0069
97	62	18	65	0.0060
97	62	16	65	0.0053
97	65	22	45	0.0076
97	65	20	45	0.0069
97	65	18	45	0.0060
97	65	16	45	0.0053
97	65	22	62	0.0076
97	65	20	62	0.0069
97	65	18	62	0.0060
97	65	16	62	0.0053
97	65	22	65	0.0076
97	65	20	65	0.0069
97	65	18	65	0.0060
97	65	16	65	0.0053

Screw flexibility calculated in accordance with AISI S310, Appendix D5.2.2

Table 7 Sidelap Screw Flexibility; S_s (inches/kip)
#10 or #12 Sidelap Screw

Deck Gage	Deck F_u	S_s
22	Any	0.0175
20	Any	0.0159
18	Any	0.0138
16	Any	0.0123

Screw flexibility calculated in accordance with AISI S310, Section D5.2.2

Table 8 Roof and Form Deck Fastener Patterns

Attachment Patterns		C	Screws k
36/9		3	7
36/7		3	6
36/5		3	4
36/4		3	3
35/8		2.92	7
35/7		2.92	6
35/6		2.92	5
35/5		2.92	4
30/7		2.5	6
30/5		2.5	4
30/4		2.5	3

Notes:

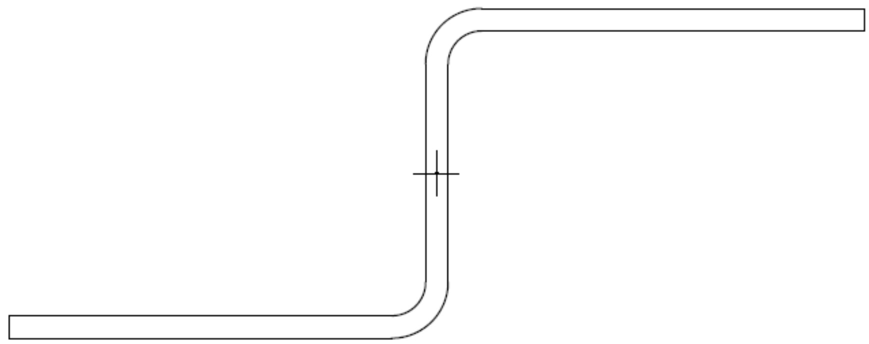
1. C is the cover width in feet. k is the effective number of connectors per deck cover width.
2. The uplift resistance U, in psf, can be calculated for a given fastener pattern using the following equation. P is obtained from Table 2 or 3. L is the deck span in feet.

$$U = \frac{kP}{CL}$$

3. The shear resistance S, in plf, can be calculated for a given fastener pattern using the following equation. <u>This equation is not applicable to the design of shear diaphragms</u>. V is obtained from Table 5.

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

Table 8 Spanning Zee



1-1/2 inch Deep Zee with 2 inch Flanges, F_y = 33 ksi

	Allowable Uniform Load (ASD) (PLF)				
Span (inches)	**20 ga**	**18 ga**	**16 ga**	**14 ga**	**12 ga**
24	167	245	345	487	809
30	107	157	221	312	518
36	74	109	153	217	360
42	54	80	113	159	264
48	42	61	86	122	202
54	33	48	68	96	160
60	27	39	55	78	129
66	22	32	46	64	107
72	19	27	38	54	90
78	16	23	33	46	77
84	14	20	28	40	66
90	12	17	25	35	58
96	10	15	22	30	51
Thickness (in)	0.0346	0.0451	0.0566	0.0713	0.1017
M_{nx} (in-lbs)	1670	2459	3458	4883	8109
M_{nx} / Ω (in-lbs)	1000	1472	2071	2924	4856
Φ M_{nx} (in-lbs)	1503	2213	3112	4395	7298
M_{ny} (in-lbs)	978	1466	2179	3400	7090
M_{ny} / Ω (in-lbs)	586	878	1305	2036	4245
Φ M_{ny} (in-lbs)	880	1319	1961	3060	6381
S_{xeff} (top) (in^3)	0.0506	0.0745	0.1048	0.1480	0.2457
S_{xeff} (bot) (in^3)	0.0960	0.1261	0.1583	0.1977	0.2726
I_{xeff} (in^4)	0.0497	0.0702	0.0946	0.1269	0.1939

	Uniform Load at Deflection = Span/120 (PLF)				
Span (inches)	**20 ga**	**18 ga**	**16 ga**	**14 ga**	**12 ga**
24	815	1151	1550	2080	3178
30	417	589	794	1065	1627
36	241	341	459	616	942
42	152	215	289	388	593
48	102	144	194	260	397
54	72	101	136	183	279
60	52	74	99	133	203
66	39	55	75	100	153
72	30	43	57	77	118
78	24	34	45	61	93
84	19	27	36	49	74
90	15	22	29	39	60
96	13	18	24	32	50

DIAPHRAGM TABLES

SECTION 7

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LOAD TABLES

THE LOAD TABLES ARE SHOWING NOMINAL STRENGTH VALUES. THE VALUES MUST NOT BE USED WITHOUT APPLYING THE PROPER SAFETY OR RESISTANCE FACTOR.

LRFD: The values of the load tables must be multiplied by a resistance factor when comparing to forces evaluated using Load and Resistance Factor Design.

ASD: The values of the load tables must be divided by a safety factor when comparing to forces evaluated using Allowable Strength Design.

The following load tables are for typical panel configurations and connector types. Different panel widths and fastening patterns are possible. Specific design applications may dictate an arrangement, not listed, which would require the designer to make direct use of the strength and stiffness formulas found in the SDI Diaphragm Design Manual (DDM). Assistance is available from SDI member companies.

The tables are arranged showing the fastener types and resistance factor at the top along with the fastener patterns as defined in Section 6. For each steel base metal design thickness given, nominal shear strengths are listed under the specific span lengths. The column "Side-lap Conn/Span" shows the number of connectors between structural supports at the sheet edge. For example, "5" would represent six even spaces or stitch fasteners at 12" centers within a 6 foot deck span.

Nominal diaphragm shears due to panel buckling are tabulated at the bottom of the pages to check whether the panel buckling governs over connector strength for diaphragm design.

Steel yield and tensile strength, and weld electrode strength (where appropriate), are shown in each table.

Table 7.1 Filled Diaphragm

Nominal Diaphragm Shear Strength, S_n, PLF
ASD (Ω = 3.25) LRFD (Φ=0.50)

Concrete Thickness Above Deck (inches)	Normalweight Concrete (145 pcf)		Lightweight Concrete (115 pcf)	
	3000 psi	4000 psi	3000 psi	4000 psi
1.5	2943	3398	2078	2400
2	3923	4530	2771	3200
2.5	4904	5663	3464	4000
3	5885	6796	4157	4800
3.5	6866	7928	4850	5600

Notes:

1. This table considers only the contribution of the concrete to the diaphragm resistance.
2. Per SDI-NC, the minimum thickness of a structural concrete slab is 1-1/2 inches above the top of the non-composite steel deck.
3. Per SDI-C, the minimum thickness of a composite concrete slab is 2 inches above the top of the composite steel deck.

Deck F_y:	33	ksi
Deck F_{u1}:	45	ksi
Framing F_{y2}:	33	ksi
Framing F_{u2}:	45	ksi

P_{nf}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	418	589	774	774	774	774
20	418	565	858	940	940	940
18	418	565	841	1191	1244	1244
16	418	565	841	1183	1569	1569

P_{ns}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	417	417	417	417	417	417
20	558	558	558	558	558	558
18	850	850	850	850	850	850
16	1205	1205	1205	1205	1205	1205

S_f, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0178	0.0136	0.0076	0.0076	0.0076	0.0076
20	0.0178	0.0161	0.0093	0.0069	0.0069	0.0069
18	0.0178	0.0161	0.0141	0.0073	0.0060	0.0060
16	0.0178	0.0161	0.0141	0.0126	0.0053	0.0053

S_s, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0175	0.0175	0.0175	0.0175	0.0175	0.0175
20	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
18	0.0138	0.0138	0.0138	0.0138	0.0138	0.0138
16	0.0123	0.0123	0.0123	0.0123	0.0123	0.0123

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	770	605	410				0.647
36/14	1	900	725	510	380	335		0.468
α_1=α_2= 4.000	2	1015	830	595	460	405	300	0.366
α_p^2 = α_e^2= 1.556	3	1110	930	680	530	475	350	0.301
N= 4.000	4	1190	1010	755	595	535	405	0.255
A= 2	5	1255	1085	825	655	595	455	0.222
	6	1310	1150	895	715	650	505	0.196
	0	385	300	205				1.293
36/7	1	505	415	295	230	200		0.732
α_1=α_2= 2.000	2	595	505	375	295	265	200	0.511
α_p^2 = α_e^2= 0.778	3	655	575	445	355	325	250	0.392
N= 2.000	4	695	625	505	410	375	295	0.318
A= 1	5	725	665	550	460	420	335	0.268
	6	745	695	590	500	465	375	0.231
	0	325	265	185				1.552
36/5	1	410	350	265	210	190		0.809
α_1=α_2= 1.667	2	460	410	330	265	245	190	0.547
α_p^2 = α_e^2= 0.722	3	490	450	375	315	290	230	0.413
N= 1.333	4	505	475	410	355	330	270	0.332
A= 1	5	520	495	440	385	360	300	0.277
	6	525	505	460	410	390	330	0.238
	0	245	200	140				1.940
36/4	1	325	280	220	175	155		0.903
α_1=α_2= 1.333	2	360	330	270	225	205	165	0.588
α_p^2 = α_e^2= 0.556	3	380	355	310	265	250	200	0.436
N= 1.000	4	390	375	335	295	280	235	0.347
A= 1	5	395	385	355	320	305	260	0.288
	6	400	390	365	340	325	285	0.246

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
		0	1085	850	580				0.494
36/14		1	1220	975	680	505	445		0.382
α_1=α_2=	4.000	2	1345	1090	770	585	515	380	0.312
α_p^2 = α_e^2=	1.556	3	1450	1195	860	660	585	430	0.263
N=	4.000	4	1550	1290	940	730	655	485	0.228
A=	2	5	1630	1375	1020	795	715	535	0.201
		6	1705	1455	1095	860	775	590	0.179
		0	540	425	290				0.989
36/7		1	670	545	385	290	255		0.624
α_1=α_2=	2.000	2	775	645	470	365	325	240	0.455
α_p^2 = α_e^2=	0.778	3	850	725	545	430	385	295	0.359
N=	2.000	4	910	795	615	490	445	345	0.296
A=	1	5	960	850	675	545	495	390	0.252
		6	995	900	730	600	545	430	0.219
		0	460	370	265				1.186
36/5		1	550	465	345	270	240		0.697
α_1=α_2=	1.667	2	610	535	415	330	300	230	0.493
α_p^2 = α_e^2=	0.722	3	655	590	475	385	350	275	0.382
N=	1.333	4	685	625	520	435	400	320	0.311
A=	1	5	705	655	560	475	440	355	0.263
		6	720	680	590	515	480	390	0.228
		0	350	285	200				1.483
36/4		1	435	370	280	220	200		0.790
α_1=α_2=	1.333	2	485	430	345	280	255	195	0.538
α_p^2 = α_e^2=	0.556	3	515	470	390	330	300	240	0.408
N=	1.000	4	530	500	430	370	340	275	0.329
A=	1	5	545	520	460	400	375	310	0.275
		6	555	530	480	430	405	340	0.237

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1425	1120	765				0.274
36/14	1	1565	1245	865	640	565		0.236
$\alpha_1=\alpha_2=$ 4.000	2	1690	1360	960	720	635	465	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1810	1470	1045	795	705	520	0.185
N= 4.000	4	1915	1575	1135	875	775	570	0.166
A= 2	5	2010	1670	1215	945	845	620	0.151
	6	2100	1760	1295	1010	910	675	0.139
	0	710	560	380				0.549
36/7	1	845	680	480	360	315		0.414
$\alpha_1=\alpha_2=$ 2.000	2	955	785	565	435	385	285	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1050	880	645	505	455	335	0.278
N= 2.000	4	1125	960	720	570	510	390	0.239
A= 1	5	1185	1030	790	630	570	440	0.209
	6	1235	1090	855	685	625	485	0.186
	0	600	490	350				0.659
36/5	1	700	585	430	335	295		0.474
$\alpha_1=\alpha_2=$ 1.667	2	770	665	505	400	360	270	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	820	725	570	460	415	320	0.304
N= 1.333	4	860	775	625	510	465	365	0.257
A= 1	5	890	815	675	560	515	410	0.223
	6	915	850	715	605	555	445	0.197
	0	460	375	265				0.823
36/4	1	550	465	345	270	240		0.554
$\alpha_1=\alpha_2=$ 1.333	2	610	535	415	335	300	225	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	650	585	470	385	355	275	0.334
N= 1.000	4	675	620	520	435	400	320	0.279
A= 1	5	695	650	555	475	440	355	0.240
	6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1425	1120	765				0.274
36/14	1	1565	1245	865	640	565		0.236
$\alpha_1=\alpha_2=$ 4.000	2	1690	1360	960	720	635	465	0.207
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	1810	1470	1045	795	705	520	0.185
N= 4.000	4	1915	1575	1135	875	775	570	0.166
A= 2	5	2010	1670	1215	945	845	620	0.151
	6	2100	1760	1295	1010	910	675	0.139
	0	710	560	380				0.549
36/7	1	845	680	480	360	315		0.414
$\alpha_1=\alpha_2=$ 2.000	2	955	785	565	435	385	285	0.333
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1050	880	645	505	455	335	0.278
N= 2.000	4	1125	960	720	570	510	390	0.239
A= 1	5	1185	1030	790	630	570	440	0.209
	6	1235	1090	855	685	625	485	0.186
	0	600	490	350				0.659
36/5	1	700	585	430	335	295		0.474
$\alpha_1=\alpha_2=$ 1.667	2	770	665	505	400	360	270	0.370
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	820	725	570	460	415	320	0.304
N= 1.333	4	860	775	625	510	465	365	0.257
A= 1	5	890	815	675	560	515	410	0.223
	6	915	850	715	605	555	445	0.197
	0	460	375	265				0.823
36/4	1	550	465	345	270	240		0.554
$\alpha_1=\alpha_2=$ 1.333	2	610	535	415	335	300	225	0.417
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	650	585	470	385	355	275	0.334
N= 1.000	4	675	620	520	435	400	320	0.279
A= 1	5	695	650	555	475	440	355	0.240
	6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	1425	1120	765				0.274
		1	1565	1245	865	640	565		0.236
α_1=α_2=	4.000	2	1690	1360	960	720	635	465	0.207
α_p^2 = α_e^2=	1.556	3	1810	1470	1045	795	705	520	0.185
N=	4.000	4	1915	1575	1135	875	775	570	0.166
A=	2	5	2010	1670	1215	945	845	620	0.151
		6	2100	1760	1295	1010	910	675	0.139
36/7		0	710	560	380				0.549
		1	845	680	480	360	315		0.414
α_1=α_2=	2.000	2	955	785	565	435	385	285	0.333
α_p^2 = α_e^2=	0.778	3	1050	880	645	505	455	335	0.278
N=	2.000	4	1125	960	720	570	510	390	0.239
A=	1	5	1185	1030	790	630	570	440	0.209
		6	1235	1090	855	685	625	485	0.186
36/5		0	600	490	350				0.659
		1	700	585	430	335	295		0.474
α_1=α_2=	1.667	2	770	665	505	400	360	270	0.370
α_p^2 = α_e^2=	0.722	3	820	725	570	460	415	320	0.304
N=	1.333	4	860	775	625	510	465	365	0.257
A=	1	5	890	815	675	560	515	410	0.223
		6	915	850	715	605	555	445	0.197
36/4		0	460	375	265				0.823
		1	550	465	345	270	240		0.554
α_1=α_2=	1.333	2	610	535	415	335	300	225	0.417
α_p^2 = α_e^2=	0.556	3	650	585	470	385	355	275	0.334
N=	1.000	4	675	620	520	435	400	320	0.279
A=	1	5	695	650	555	475	440	355	0.240
		6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1425	1120	765				0.274
	1	1565	1245	865	640	565		0.236
α_1=α_2= 4.000	2	1690	1360	960	720	635	465	0.207
$\alpha_p^2 = \alpha_e^2$= 1.556	3	1810	1470	1045	795	705	520	0.185
N= 4.000	4	1915	1575	1135	875	775	570	0.166
A= 2	5	2010	1670	1215	945	845	620	0.151
	6	2100	1760	1295	1010	910	675	0.139
36/7	0	710	560	380				0.549
	1	845	680	480	360	315		0.414
α_1=α_2= 2.000	2	955	785	565	435	385	285	0.333
$\alpha_p^2 = \alpha_e^2$= 0.778	3	1050	880	645	505	455	335	0.278
N= 2.000	4	1125	960	720	570	510	390	0.239
A= 1	5	1185	1030	790	630	570	440	0.209
	6	1235	1090	855	685	625	485	0.186
36/5	0	600	490	350				0.659
	1	700	585	430	335	295		0.474
α_1=α_2= 1.667	2	770	665	505	400	360	270	0.370
$\alpha_p^2 = \alpha_e^2$= 0.722	3	820	725	570	460	415	320	0.304
N= 1.333	4	860	775	625	510	465	365	0.257
A= 1	5	890	815	675	560	515	410	0.223
	6	915	850	715	605	555	445	0.197
36/4	0	460	375	265				0.823
	1	550	465	345	270	240		0.554
α_1=α_2= 1.333	2	610	535	415	335	300	225	0.417
$\alpha_p^2 = \alpha_e^2$= 0.556	3	650	585	470	385	355	275	0.334
N= 1.000	4	675	620	520	435	400	320	0.279
A= 1	5	695	650	555	475	440	355	0.240
	6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	770	605	415				0.785
	1	945	760	540	410	360		0.552
$\alpha_1=\alpha_2=$ 4.000	2	1080	900	655	505	455	335	0.426
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1190	1015	755	595	535	405	0.346
N= 4.000	4	1275	1110	850	675	610	475	0.292
A= 2	5	1345	1190	935	755	685	535	0.252
	6	1395	1255	1010	825	755	590	0.222
36/7	0	385	300	205				1.569
	1	540	450	325	250	225		0.851
$\alpha_1=\alpha_2=$ 2.000	2	635	555	425	335	305	235	0.584
$\alpha_p^2=\alpha_e^2=$ 0.778	3	695	625	505	410	375	295	0.445
N= 2.000	4	735	675	565	475	435	345	0.359
A= 1	5	760	710	615	530	490	400	0.301
	6	775	735	655	570	535	445	0.259
36/5	0	325	265	185				1.883
	1	430	375	290	230	210		0.936
$\alpha_1=\alpha_2=$ 1.667	2	480	440	360	300	275	220	0.623
$\alpha_p^2=\alpha_e^2=$ 0.722	3	505	475	410	355	330	270	0.467
N= 1.333	4	520	500	445	395	370	310	0.373
A= 1	5	530	515	470	425	405	345	0.311
	6	535	520	490	450	430	375	0.266
36/4	0	245	200	145				2.354
	1	340	300	240	195	175		1.039
$\alpha_1=\alpha_2=$ 1.333	2	375	350	300	255	235	190	0.667
$\alpha_p^2=\alpha_e^2=$ 0.556	3	390	375	335	295	280	235	0.491
N= 1.000	4	400	385	360	325	310	270	0.388
A= 1	5	405	395	375	345	335	295	0.321
	6	405	400	385	360	350	315	0.274

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1040	815	560				0.710
36/14	1	1220	980	690	515	455		0.514
$\alpha_1=\alpha_2=$ 4.000	2	1370	1125	805	620	550	405	0.403
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1500	1250	915	710	640	475	0.331
N= 4.000	4	1605	1365	1020	800	720	545	0.281
A= 2	5	1695	1465	1115	885	800	615	0.244
	6	1770	1550	1200	965	870	675	0.216
	0	520	405	280				1.419
36/7	1	685	560	400	310	275		0.805
$\alpha_1=\alpha_2=$ 2.000	2	800	680	510	400	360	270	0.562
$\alpha_p^2=\alpha_e^2=$ 0.778	3	885	775	600	480	435	335	0.432
N= 2.000	4	940	845	680	555	505	395	0.350
A= 1	5	980	895	745	620	570	450	0.295
	6	1010	940	795	675	625	505	0.254
	0	440	355	255				1.703
36/5	1	555	475	360	285	255		0.889
$\alpha_1=\alpha_2=$ 1.667	2	620	555	440	360	330	255	0.602
$\alpha_p^2=\alpha_e^2=$ 0.722	3	660	605	505	425	390	310	0.455
N= 1.333	4	685	640	555	480	445	360	0.365
A= 1	5	700	665	595	520	490	405	0.305
	6	710	685	620	555	525	445	0.262
	0	335	270	195				2.129
36/4	1	435	380	295	235	215		0.993
$\alpha_1=\alpha_2=$ 1.333	2	485	445	365	305	280	220	0.647
$\alpha_p^2=\alpha_e^2=$ 0.556	3	515	480	420	360	335	270	0.480
N= 1.000	4	530	505	455	400	375	315	0.382
A= 1	5	535	520	475	430	410	350	0.317
	6	545	530	495	455	435	380	0.271

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1580	1240	850				0.409
	1	1765	1405	985	730	645		0.335
$\alpha_1=\alpha_2=$ 4.000	2	1930	1560	1105	835	740	545	0.284
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2080	1700	1220	940	830	615	0.247
N= 4.000	4	2210	1835	1330	1035	925	685	0.218
A= 2	5	2330	1955	1440	1125	1010	755	0.195
	6	2435	2065	1540	1210	1090	825	0.176
36/7	0	790	620	425				0.818
	1	965	780	550	415	370		0.568
$\alpha_1=\alpha_2=$ 2.000	2	1105	915	665	515	460	340	0.435
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1215	1030	770	605	545	410	0.353
N= 2.000	4	1300	1130	865	685	620	480	0.297
A= 1	5	1370	1210	950	765	695	540	0.256
	6	1425	1280	1025	835	765	600	0.225
36/5	0	670	545	385				0.982
	1	795	670	495	385	345		0.643
$\alpha_1=\alpha_2=$ 1.667	2	880	765	590	470	425	325	0.478
$\alpha_p^2=\alpha_e^2=$ 0.722	3	940	840	670	545	495	390	0.380
N= 1.333	4	985	895	740	610	560	445	0.316
A= 1	5	1015	940	795	670	620	495	0.270
	6	1040	975	840	725	670	545	0.236
36/4	0	510	415	295				1.228
	1	625	535	400	315	285		0.740
$\alpha_1=\alpha_2=$ 1.333	2	695	615	490	395	360	275	0.529
$\alpha_p^2=\alpha_e^2=$ 0.556	3	740	675	555	460	425	335	0.412
N= 1.000	4	770	715	610	520	480	390	0.337
A= 1	5	790	745	655	565	530	435	0.286
	6	800	765	685	605	570	475	0.248

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1725	1360	930				0.302
	1	1915	1525	1065	790	700		0.260
$\alpha_1=\alpha_2=$ 4.000	2	2085	1680	1185	895	790	585	0.228
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
36/7	0	860	680	465				0.605
	1	1040	840	590	445	395		0.456
$\alpha_1=\alpha_2=$ 2.000	2	1185	980	705	545	490	360	0.366
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
36/5	0	730	595	425				0.726
	1	860	720	535	415	370		0.522
$\alpha_1=\alpha_2=$ 1.667	2	950	825	630	500	450	340	0.408
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
36/4	0	555	455	325				0.907
	1	675	575	430	340	300		0.610
$\alpha_1=\alpha_2=$ 1.333	2	750	660	520	420	380	290	0.459
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df} S_{nf}$, $\phi_{db} S_{nb}$}

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1725	1360	930				0.302
36/14	1	1915	1525	1065	790	700		0.260
α_1=α_2= 4.000	2	2085	1680	1185	895	790	585	0.228
α_p^2 = α_e^2= 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
	0	860	680	465				0.605
36/7	1	1040	840	590	445	395		0.456
α_1=α_2= 2.000	2	1185	980	705	545	490	360	0.366
α_p^2 = α_e^2= 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
	0	730	595	425				0.726
36/5	1	860	720	535	415	370		0.522
α_1=α_2= 1.667	2	950	825	630	500	450	340	0.408
α_p^2 = α_e^2= 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
	0	555	455	325				0.907
36/4	1	675	575	430	340	300		0.610
α_1=α_2= 1.333	2	750	660	520	420	380	290	0.459
α_p^2 = α_e^2= 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1725	1360	930				0.302
	1	1915	1525	1065	790	700		0.260
$\alpha_1=\alpha_2=$ 4.000	2	2085	1680	1185	895	790	585	0.228
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
36/7	0	860	680	465				0.605
	1	1040	840	590	445	395		0.456
$\alpha_1=\alpha_2=$ 2.000	2	1185	980	705	545	490	360	0.366
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
36/5	0	730	595	425				0.726
	1	860	720	535	415	370		0.522
$\alpha_1=\alpha_2=$ 1.667	2	950	825	630	500	450	340	0.408
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
36/4	0	555	455	325				0.907
	1	675	575	430	340	300		0.610
$\alpha_1=\alpha_2=$ 1.333	2	750	660	520	420	380	290	0.459
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	770	605	415				1.039
	1	1020	835	600	465	410		0.700
$\alpha_1=\alpha_2=$ 4.000	2	1195	1020	760	575	510	380	0.527
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	1315	1155	870	650	580	435	0.423
N= 4.000	4	1400	1260	975	730	650	485	0.353
A= 2	5	1460	1340	1080	810	720	540	0.303
	6	1500	1400	1180	885	785	590	0.266
36/7	0	385	300	205				2.078
	1	595	510	380	285	255		1.054
$\alpha_1=\alpha_2=$ 2.000	2	700	630	485	365	325	240	0.706
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	750	700	590	440	390	295	0.531
N= 2.000	4	775	740	655	520	460	345	0.426
A= 1	5	790	765	700	600	530	400	0.355
	6	805	780	725	665	600	450	0.305
36/5	0	325	265	185				2.494
	1	460	410	330	250	220		1.152
$\alpha_1=\alpha_2=$ 1.667	2	505	475	415	330	290	220	0.749
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	525	505	460	405	360	270	0.555
N= 1.333	4	535	525	490	455	430	325	0.441
A= 1	5	540	530	510	480	460	375	0.365
	6	545	540	520	495	485	425	0.312
36/4	0	245	200	145				3.117
	1	360	330	275	215	190		1.269
$\alpha_1=\alpha_2=$ 1.333	2	390	375	335	295	260	195	0.797
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	400	390	365	340	325	245	0.581
N= 1.000	4	405	400	385	365	350	300	0.457
A= 1	5	410	405	395	380	370	340	0.376
	6	410	410	400	385	380	360	0.320

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1040	815	560				0.940
36/14	1	1300	1055	750	575	510		0.653
$\alpha_1=\alpha_2=$ 4.000	2	1505	1255	920	715	645	480	0.500
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1660	1425	1075	850	765	585	0.406
N= 4.000	4	1775	1555	1210	970	875	655	0.341
A= 2	5	1865	1665	1330	1085	970	730	0.294
	6	1930	1755	1435	1185	1065	800	0.259
	0	520	405	280				1.879
36/7	1	750	625	460	355	320		1.001
$\alpha_1=\alpha_2=$ 2.000	2	885	775	605	485	435	325	0.682
$\alpha_p^2=\alpha_e^2=$ 0.778	3	965	875	715	590	530	400	0.517
N= 2.000	4	1010	940	800	680	625	470	0.417
A= 1	5	1040	985	865	755	705	540	0.349
	6	1060	1015	915	815	765	610	0.300
	0	440	355	255				2.255
36/5	1	595	520	405	325	295		1.098
$\alpha_1=\alpha_2=$ 1.667	2	660	610	510	425	395	295	0.726
$\alpha_p^2=\alpha_e^2=$ 0.722	3	695	655	580	505	470	365	0.542
N= 1.333	4	710	685	625	560	530	435	0.433
A= 1	5	725	705	655	600	575	495	0.360
	6	730	715	675	630	605	535	0.308
	0	335	270	195				2.819
36/4	1	465	420	335	275	250		1.217
$\alpha_1=\alpha_2=$ 1.333	2	515	485	420	360	335	265	0.776
$\alpha_p^2=\alpha_e^2=$ 0.556	3	535	515	470	420	395	335	0.569
N= 1.000	4	545	530	495	460	440	385	0.450
A= 1	5	550	540	515	485	470	420	0.372
	6	555	545	525	500	490	445	0.317

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1545	1215	840				0.823
36/14	1	1820	1465	1030	780	690		0.594
α_1=α_2= 4.000	2	2055	1680	1210	930	830	615	0.465
α_p^2 = α_e^2= 1.556	3	2245	1875	1375	1070	960	720	0.382
N= 4.000	4	2405	2045	1530	1205	1085	825	0.324
A= 2	5	2535	2195	1675	1330	1200	925	0.282
	6	2645	2325	1810	1450	1315	1020	0.249
	0	770	605	420				1.646
36/7	1	1025	840	605	465	415		0.931
α_1=α_2= 2.000	2	1200	1020	765	600	540	410	0.649
α_p^2 = α_e^2= 0.778	3	1320	1160	905	725	655	510	0.498
N= 2.000	4	1405	1265	1020	835	760	600	0.404
A= 1	5	1465	1345	1115	930	855	680	0.340
	6	1510	1405	1195	1020	940	760	0.293
	0	655	530	380				1.975
36/5	1	830	710	540	425	385		1.027
α_1=α_2= 1.667	2	925	830	665	540	495	385	0.694
α_p^2 = α_e^2= 0.722	3	985	910	760	640	590	470	0.524
N= 1.333	4	1020	960	835	720	670	545	0.421
A= 1	5	1045	995	890	785	735	610	0.352
	6	1060	1020	930	835	790	670	0.302
	0	500	405	290				2.469
36/4	1	655	570	440	355	320		1.147
α_1=α_2= 1.333	2	730	665	550	460	420	335	0.747
α_p^2 = α_e^2= 0.556	3	765	720	625	540	505	410	0.554
N= 1.000	4	790	755	680	605	565	475	0.440
A= 1	5	800	775	715	650	615	530	0.365
	6	810	790	740	685	655	575	0.312

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2190	1725	1185				0.427
36/14	1	2470	1975	1380	1035	915		0.356
$\alpha_1=\alpha_2=$ 4.000	2	2720	2205	1565	1195	1060	780	0.305
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2940	2415	1740	1345	1200	890	0.267
N= 4.000	4	3135	2610	1910	1485	1330	995	0.237
A= 2	5	3305	2790	2065	1620	1455	1100	0.214
	6	3455	2950	2220	1750	1575	1205	0.194
	0	1095	860	590				0.854
36/7	1	1360	1100	780	595	530		0.610
$\alpha_1=\alpha_2=$ 2.000	2	1565	1305	955	740	665	495	0.475
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1725	1475	1110	875	785	600	0.389
N= 2.000	4	1845	1615	1245	995	900	700	0.329
A= 1	5	1940	1730	1370	1110	1010	790	0.285
	6	2015	1820	1480	1215	1110	875	0.252
	0	925	755	535				1.025
36/5	1	1115	945	700	550	495		0.693
$\alpha_1=\alpha_2=$ 1.667	2	1240	1085	845	675	610	475	0.524
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1325	1195	960	785	715	565	0.421
N= 1.333	4	1385	1270	1055	885	810	645	0.352
A= 1	5	1425	1330	1135	970	895	725	0.302
	6	1455	1375	1200	1040	970	795	0.265
	0	705	575	410				1.281
36/4	1	880	755	570	450	405		0.801
$\alpha_1=\alpha_2=$ 1.333	2	980	875	700	570	515	405	0.583
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1040	955	795	665	615	490	0.458
N= 1.000	4	1080	1010	875	750	695	565	0.377
A= 1	5	1105	1050	930	815	765	635	0.321
	6	1120	1080	975	870	825	695	0.279

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18
Design thickness = 0.0474 in.
$F_{y\text{-deck}}$= 33 ksi
$F_{u\text{-deck}}$= 45 ksi
Framing designation = 68 mils
Framing thickness = 0.0713 in.
$F_{y\text{-framing}}$= 33 ksi
$F_{u\text{-framing}}$= 45 ksi
Support fastening: #12 screws
Side-lap fastening: #10 screws

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	2290	1800	1240				0.348
		1	2570	2050	1435	1075	950		0.299
α_1=α_2=	4.000	2	2820	2285	1620	1235	1095	805	0.263
α_p^2 = α_e^2=	1.556	3	3045	2500	1795	1385	1235	915	0.234
N=	4.000	4	3245	2695	1965	1525	1370	1020	0.211
A=	2	5	3415	2875	2125	1660	1495	1125	0.192
		6	3570	3040	2280	1790	1615	1235	0.176
36/7		0	1145	900	620				0.696
		1	1410	1140	810	615	545		0.525
α_1=α_2=	2.000	2	1620	1345	980	760	685	510	0.422
α_p^2 = α_e^2=	0.778	3	1785	1520	1140	895	805	615	0.352
N=	2.000	4	1910	1665	1280	1020	925	715	0.303
A=	1	5	2010	1780	1405	1135	1035	805	0.265
		6	2085	1880	1520	1245	1135	895	0.236
36/5		0	970	785	560				0.835
		1	1155	980	725	570	510		0.601
α_1=α_2=	1.667	2	1285	1125	870	695	630	485	0.469
α_p^2 = α_e^2=	0.722	3	1375	1235	990	805	735	575	0.385
N=	1.333	4	1435	1315	1090	905	830	660	0.326
A=	1	5	1480	1380	1170	995	920	740	0.283
		6	1515	1425	1240	1070	995	815	0.250
36/4		0	740	600	430				1.044
		1	910	780	590	465	420		0.702
α_1=α_2=	1.333	2	1015	905	720	585	530	415	0.528
α_p^2 = α_e^2=	0.556	3	1080	990	820	685	630	500	0.424
N=	1.000	4	1120	1050	900	770	715	575	0.354
A=	1	5	1150	1090	960	840	785	645	0.304
		6	1170	1120	1010	895	845	710	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2290	1800	1240				0.348
	1	2570	2050	1435	1075	950		0.299
$\alpha_1=\alpha_2=$ 4.000	2	2820	2285	1620	1235	1095	805	0.263
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3045	2500	1795	1385	1235	915	0.234
N= 4.000	4	3245	2695	1965	1525	1370	1020	0.211
A= 2	5	3415	2875	2125	1660	1495	1125	0.192
	6	3570	3040	2280	1790	1615	1235	0.176
36/7	0	1145	900	620				0.696
	1	1410	1140	810	615	545		0.525
$\alpha_1=\alpha_2=$ 2.000	2	1620	1345	980	760	685	510	0.422
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1785	1520	1140	895	805	615	0.352
N= 2.000	4	1910	1665	1280	1020	925	715	0.303
A= 1	5	2010	1780	1405	1135	1035	805	0.265
	6	2085	1880	1520	1245	1135	895	0.236
36/5	0	970	785	560				0.835
	1	1155	980	725	570	510		0.601
$\alpha_1=\alpha_2=$ 1.667	2	1285	1125	870	695	630	485	0.469
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1375	1235	990	805	735	575	0.385
N= 1.333	4	1435	1315	1090	905	830	660	0.326
A= 1	5	1480	1380	1170	995	920	740	0.283
	6	1515	1425	1240	1070	995	815	0.250
36/4	0	740	600	430				1.044
	1	910	780	590	465	420		0.702
$\alpha_1=\alpha_2=$ 1.333	2	1015	905	720	585	530	415	0.528
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1080	990	820	685	630	500	0.424
N= 1.000	4	1120	1050	900	770	715	575	0.354
A= 1	5	1150	1090	960	840	785	645	0.304
	6	1170	1120	1010	895	845	710	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	770	605	415				1.311
36/14	1	1100	920	660	495	440		0.848
α_1=α_2= 4.000	2	1300	1135	765	575	510	380	0.627
α_p^2 = α_e^2= 1.556	3	1415	1280	870	650	580	435	0.497
N= 4.000	4	1485	1380	975	730	650	485	0.412
A= 2	5	1535	1450	1080	810	720	540	0.352
	6	1565	1495	1180	885	785	590	0.307
	0	385	300	205				2.622
36/7	1	650	565	380	285	255		1.254
α_1=α_2= 2.000	2	740	690	485	365	325	240	0.824
α_p^2 = α_e^2= 0.778	3	780	745	590	440	390	295	0.614
N= 2.000	4	800	775	695	520	460	345	0.489
A= 1	5	810	795	750	600	530	400	0.406
	6	815	805	770	675	600	450	0.348
	0	325	265	185				3.146
36/5	1	485	445	335	250	220		1.363
α_1=α_2= 1.667	2	525	505	440	330	290	220	0.870
α_p^2 = α_e^2= 0.722	3	540	525	495	405	360	270	0.639
N= 1.333	4	545	535	515	485	430	325	0.505
A= 1	5	550	540	530	510	500	375	0.417
	6	550	545	535	520	510	425	0.355
	0	245	200	145				3.933
36/4	1	380	355	290	215	190		1.492
α_1=α_2= 1.333	2	400	390	365	295	260	195	0.921
α_p^2 = α_e^2= 0.556	3	410	400	385	365	330	245	0.666
N= 1.000	4	410	405	395	385	375	300	0.521
A= 1	5	410	410	405	395	390	350	0.429
	6	415	410	405	400	395	380	0.364

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1040	815	565				1.185
36/14	1	1395	1145	825	635	570		0.794
$\alpha_1=\alpha_2=$ 4.000	2	1635	1400	1035	775	690	515	0.597
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	1800	1585	1175	880	785	585	0.478
N= 4.000	4	1910	1730	1315	985	875	655	0.399
A= 2	5	1985	1830	1460	1095	970	730	0.342
	6	2040	1910	1600	1200	1065	800	0.300
	0	520	405	280				2.371
36/7	1	815	700	515	385	345		1.194
$\alpha_1=\alpha_2=$ 2.000	2	955	865	655	490	435	325	0.798
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1020	955	800	600	530	400	0.599
N= 2.000	4	1055	1005	900	705	625	470	0.480
A= 1	5	1075	1040	955	810	720	540	0.400
	6	1090	1060	995	915	815	610	0.343
	0	440	355	255				2.845
36/5	1	625	565	455	340	300		1.303
$\alpha_1=\alpha_2=$ 1.667	2	690	650	570	445	395	295	0.845
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	715	690	630	550	490	365	0.625
N= 1.333	4	730	710	670	620	585	435	0.496
A= 1	5	735	725	690	655	635	510	0.411
	6	740	730	705	675	660	580	0.351
	0	335	270	195				3.557
36/4	1	490	450	375	290	260		1.435
$\alpha_1=\alpha_2=$ 1.333	2	530	510	460	400	355	265	0.899
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	545	535	500	465	445	335	0.654
N= 1.000	4	550	545	520	495	480	405	0.514
A= 1	5	555	550	535	515	505	470	0.424
	6	555	555	540	525	520	490	0.360

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1545	1215	840				1.038
36/14	1	1925	1555	1105	850	750		0.725
$\alpha_1=\alpha_2=$ 4.000	2	2215	1845	1350	1050	940	705	0.557
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2440	2085	1570	1235	1115	855	0.452
N= 4.000	4	2610	2285	1765	1410	1280	980	0.381
A= 2	5	2745	2445	1940	1575	1430	1085	0.329
	6	2845	2580	2095	1725	1575	1190	0.289
	0	770	605	420				2.077
36/7	1	1105	920	675	525	470		1.114
$\alpha_1=\alpha_2=$ 2.000	2	1305	1140	880	705	640	490	0.761
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1420	1290	1045	860	785	595	0.578
N= 2.000	4	1495	1385	1170	990	915	700	0.466
A= 1	5	1540	1455	1270	1100	1025	805	0.390
	6	1575	1500	1345	1185	1115	910	0.336
	0	655	530	380				2.492
36/5	1	875	770	595	475	430		1.224
$\alpha_1=\alpha_2=$ 1.667	2	980	900	745	625	575	440	0.811
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1030	970	850	735	685	545	0.606
N= 1.333	4	1055	1015	920	820	775	650	0.484
A= 1	5	1075	1040	965	880	840	725	0.403
	6	1085	1060	1000	925	890	785	0.345
	0	500	405	290				3.115
36/4	1	690	620	495	400	365		1.357
$\alpha_1=\alpha_2=$ 1.333	2	760	715	615	530	490	395	0.868
$\alpha_p^2=\alpha_e^2=$ 0.556	3	795	760	690	615	580	490	0.638
N= 1.000	4	810	785	735	675	645	560	0.504
A= 1	5	815	800	760	715	690	615	0.417
	6	825	810	780	740	720	655	0.355

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2175	1710	1185				0.927
36/14	1	2565	2060	1450	1105	975		0.669
$\alpha_1=\alpha_2=$ 4.000	2	2890	2370	1705	1315	1175	870	0.523
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3165	2645	1940	1510	1360	1020	0.430
N= 4.000	4	3390	2885	2160	1700	1530	1175	0.365
A= 2	5	3575	3095	2365	1880	1700	1310	0.317
	6	3730	3275	2550	2050	1855	1440	0.280
	0	1085	855	590				1.854
36/7	1	1445	1185	850	655	585		1.047
$\alpha_1=\alpha_2=$ 2.000	2	1695	1440	1080	850	765	585	0.729
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1865	1635	1275	1025	925	720	0.560
N= 2.000	4	1980	1785	1440	1180	1075	845	0.454
A= 1	5	2065	1895	1575	1315	1210	965	0.382
	6	2125	1980	1690	1440	1330	1075	0.330
	0	920	750	535				2.224
36/5	1	1165	1005	760	600	540		1.156
$\alpha_1=\alpha_2=$ 1.667	2	1305	1170	940	765	700	545	0.780
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1390	1280	1075	900	830	665	0.589
N= 1.333	4	1440	1355	1175	1015	945	770	0.473
A= 1	5	1470	1405	1255	1105	1040	865	0.395
	6	1495	1440	1310	1180	1115	945	0.340
	0	700	570	410				2.781
36/4	1	920	805	625	500	450		1.290
$\alpha_1=\alpha_2=$ 1.333	2	1025	940	780	650	595	475	0.839
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1080	1015	885	765	710	580	0.622
N= 1.000	4	1110	1065	955	850	800	670	0.494
A= 1	5	1130	1095	1010	915	870	750	0.410
	6	1140	1115	1045	965	925	810	0.350

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR16	
Design thickness = 0.0598 in.	
$F_{y\text{-}deck}$= 33 ksi	
$F_{u\text{-}deck}$= 45 ksi	
Framing designation = 68 mils	
Framing thickness = 0.0713 in.	
$F_{y\text{-}framing}$= 33 ksi	
$F_{u\text{-}framing}$= 45 ksi	
Support fastening: #12 screws	
Side-lap fastening: #10 screws	

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2885	2275	1570				0.391
36/14	1	3285	2625	1840	1390	1230		0.336
$\alpha_1=\alpha_2=$ 4.000	2	3635	2950	2100	1615	1430	1060	0.295
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3940	3245	2345	1815	1625	1210	0.263
N= 4.000	4	4205	3515	2580	2010	1805	1360	0.237
A= 2	5	4435	3760	2800	2200	1980	1510	0.216
	6	4630	3980	3010	2380	2150	1655	0.198
	0	1440	1135	785				0.782
36/7	1	1815	1475	1050	805	715		0.590
$\alpha_1=\alpha_2=$ 2.000	2	2100	1755	1290	1005	900	680	0.474
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2315	1990	1505	1190	1075	825	0.396
N= 2.000	4	2475	2175	1695	1360	1235	960	0.340
A= 1	5	2595	2330	1865	1520	1385	1085	0.298
	6	2690	2450	2010	1665	1525	1205	0.265
	0	1225	995	710				0.938
36/5	1	1485	1260	940	740	665		0.675
$\alpha_1=\alpha_2=$ 1.667	2	1655	1455	1135	915	830	645	0.527
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1765	1600	1300	1065	975	770	0.432
N= 1.333	4	1845	1700	1430	1200	1105	885	0.366
A= 1	5	1895	1780	1535	1315	1220	990	0.318
	6	1935	1835	1620	1415	1320	1090	0.281
	0	930	760	545				1.172
36/4	1	1170	1010	765	605	550		0.788
$\alpha_1=\alpha_2=$ 1.333	2	1305	1175	945	770	705	550	0.594
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1385	1280	1075	905	835	670	0.476
N= 1.000	4	1435	1350	1175	1015	945	775	0.397
A= 1	5	1465	1400	1250	1105	1040	865	0.341
	6	1490	1435	1310	1180	1115	950	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2885	2275	1570				0.391
36/14	1	3285	2625	1840	1390	1230		0.336
α_1=α_2= 4.000	2	3635	2950	2100	1615	1430	1060	0.295
α_p^2 = α_e^2= 1.556	3	3940	3245	2345	1815	1625	1210	0.263
N= 4.000	4	4205	3515	2580	2010	1805	1360	0.237
A= 2	5	4435	3760	2800	2200	1980	1510	0.216
	6	4630	3980	3010	2380	2150	1655	0.198
	0	1440	1135	785				0.782
36/7	1	1815	1475	1050	805	715		0.590
α_1=α_2= 2.000	2	2100	1755	1290	1005	900	680	0.474
α_p^2 = α_e^2= 0.778	3	2315	1990	1505	1190	1075	825	0.396
N= 2.000	4	2475	2175	1695	1360	1235	960	0.340
A= 1	5	2595	2330	1865	1520	1385	1085	0.298
	6	2690	2450	2010	1665	1525	1205	0.265
	0	1225	995	710				0.938
36/5	1	1485	1260	940	740	665		0.675
α_1=α_2= 1.667	2	1655	1455	1135	915	830	645	0.527
α_p^2 = α_e^2= 0.722	3	1765	1600	1300	1065	975	770	0.432
N= 1.333	4	1845	1700	1430	1200	1105	885	0.366
A= 1	5	1895	1780	1535	1315	1220	990	0.318
	6	1935	1835	1620	1415	1320	1090	0.281
	0	930	760	545				1.172
36/4	1	1170	1010	765	605	550		0.788
α_1=α_2= 1.333	2	1305	1175	945	770	705	550	0.594
α_p^2 = α_e^2= 0.556	3	1385	1280	1075	905	835	670	0.476
N= 1.000	4	1435	1350	1175	1015	945	775	0.397
A= 1	5	1465	1400	1250	1105	1040	865	0.341
	6	1490	1435	1310	1180	1115	950	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

Deck F_y:	33	ksi
Deck F_{u1}:	45	ksi
Framing F_{y2}:	50	ksi
Framing F_{u2}:	65	ksi

P_{nf}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	604	774	774	774	774	774
20	604	817	940	940	940	940
18	604	817	1215	1244	1244	1244
16	604	817	1215	1569	1569	1569

P_{ns}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	417	417	417	417	417	417
20	558	558	558	558	558	558
18	850	850	850	850	850	850
16	1205	1205	1205	1205	1205	1205

S_f, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0178	0.0076	0.0076	0.0076	0.0076	0.0076
20	0.0178	0.0161	0.0069	0.0069	0.0069	0.0069
18	0.0178	0.0161	0.0141	0.0060	0.0060	0.0060
16	0.0178	0.0161	0.0141	0.0126	0.0053	0.0053

S_s, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0175	0.0175	0.0175	0.0175	0.0175	0.0175
20	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
18	0.0138	0.0138	0.0138	0.0138	0.0138	0.0138
16	0.0123	0.0123	0.0123	0.0123	0.0123	0.0123

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1110	875	595				0.647
	1	1250	995	695	515	455		0.468
$\alpha_1=\alpha_2=$ 4.000	2	1370	1110	785	595	525	385	0.366
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1480	1215	875	675	595	440	0.301
N= 4.000	4	1575	1310	955	740	665	490	0.255
A= 2	5	1660	1400	1035	810	725	540	0.222
	6	1735	1480	1110	875	785	595	0.196
36/7	0	555	435	295				1.293
	1	685	555	390	295	260		0.732
$\alpha_1=\alpha_2=$ 2.000	2	785	655	475	370	330	245	0.511
$\alpha_p^2=\alpha_e^2=$ 0.778	3	865	740	555	435	390	295	0.392
N= 2.000	4	930	810	625	495	450	345	0.318
A= 1	5	975	865	685	555	505	390	0.268
	6	1015	915	740	605	555	435	0.231
36/5	0	470	380	270				1.552
	1	560	475	350	275	245		0.809
$\alpha_1=\alpha_2=$ 1.667	2	625	545	420	335	305	235	0.547
$\alpha_p^2=\alpha_e^2=$ 0.722	3	670	600	480	390	360	280	0.413
N= 1.333	4	700	640	530	440	405	320	0.332
A= 1	5	720	670	570	485	445	360	0.277
	6	735	690	605	520	485	395	0.238
36/4	0	355	290	205				1.940
	1	440	380	285	225	200		0.903
$\alpha_1=\alpha_2=$ 1.333	2	495	440	350	285	260	200	0.588
$\alpha_p^2=\alpha_e^2=$ 0.556	3	525	480	400	335	305	245	0.436
N= 1.000	4	545	510	435	375	345	280	0.347
A= 1	5	555	530	465	410	380	315	0.288
	6	565	545	490	435	410	345	0.246

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in⁴/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1425	1120	765				0.274
	1	1565	1245	865	640	565		0.236
$\alpha_1=\alpha_2=$ 4.000	2	1690	1360	960	720	635	465	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1810	1470	1045	795	705	520	0.185
N= 4.000	4	1915	1575	1135	875	775	570	0.166
A= 2	5	2010	1670	1215	945	845	620	0.151
	6	2100	1760	1295	1010	910	675	0.139
36/7	0	710	560	380				0.549
	1	845	680	480	360	315		0.414
$\alpha_1=\alpha_2=$ 2.000	2	955	785	565	435	385	285	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1050	880	645	505	455	335	0.278
N= 2.000	4	1125	960	720	570	510	390	0.239
A= 1	5	1185	1030	790	630	570	440	0.209
	6	1235	1090	855	685	625	485	0.186
36/5	0	600	490	350				0.659
	1	700	585	430	335	295		0.474
$\alpha_1=\alpha_2=$ 1.667	2	770	665	505	400	360	270	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	820	725	570	460	415	320	0.304
N= 1.333	4	860	775	625	510	465	365	0.257
A= 1	5	890	815	675	560	515	410	0.223
	6	915	850	715	605	555	445	0.197
36/4	0	460	375	265				0.823
	1	550	465	345	270	240		0.554
$\alpha_1=\alpha_2=$ 1.333	2	610	535	415	335	300	225	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	650	585	470	385	355	275	0.334
N= 1.000	4	675	620	520	435	400	320	0.279
A= 1	5	695	650	555	475	440	355	0.240
	6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	1425	1120	765				0.274
		1	1565	1245	865	640	565		0.236
α_1=α_2=	4.000	2	1690	1360	960	720	635	465	0.207
α_p^2 = α_e^2=	1.556	3	1810	1470	1045	795	705	520	0.185
N=	4.000	4	1915	1575	1135	875	775	570	0.166
A=	2	5	2010	1670	1215	945	845	620	0.151
		6	2100	1760	1295	1010	910	675	0.139
36/7		0	710	560	380				0.549
		1	845	680	480	360	315		0.414
α_1=α_2=	2.000	2	955	785	565	435	385	285	0.333
α_p^2 = α_e^2=	0.778	3	1050	880	645	505	455	335	0.278
N=	2.000	4	1125	960	720	570	510	390	0.239
A=	1	5	1185	1030	790	630	570	440	0.209
		6	1235	1090	855	685	625	485	0.186
36/5		0	600	490	350				0.659
		1	700	585	430	335	295		0.474
α_1=α_2=	1.667	2	770	665	505	400	360	270	0.370
α_p^2 = α_e^2=	0.722	3	820	725	570	460	415	320	0.304
N=	1.333	4	860	775	625	510	465	365	0.257
A=	1	5	890	815	675	560	515	410	0.223
		6	915	850	715	605	555	445	0.197
36/4		0	460	375	265				0.823
		1	550	465	345	270	240		0.554
α_1=α_2=	1.333	2	610	535	415	335	300	225	0.417
α_p^2 = α_e^2=	0.556	3	650	585	470	385	355	275	0.334
N=	1.000	4	675	620	520	435	400	320	0.279
A=	1	5	695	650	555	475	440	355	0.240
		6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1425	1120	765				0.274
36/14	1	1565	1245	865	640	565		0.236
α_1=α_2= 4.000	2	1690	1360	960	720	635	465	0.207
α_p^2 = α_e^2= 1.556	3	1810	1470	1045	795	705	520	0.185
N= 4.000	4	1915	1575	1135	875	775	570	0.166
A= 2	5	2010	1670	1215	945	845	620	0.151
	6	2100	1760	1295	1010	910	675	0.139
	0	710	560	380				0.549
36/7	1	845	680	480	360	315		0.414
α_1=α_2= 2.000	2	955	785	565	435	385	285	0.333
α_p^2 = α_e^2= 0.778	3	1050	880	645	505	455	335	0.278
N= 2.000	4	1125	960	720	570	510	390	0.239
A= 1	5	1185	1030	790	630	570	440	0.209
	6	1235	1090	855	685	625	485	0.186
	0	600	490	350				0.659
36/5	1	700	585	430	335	295		0.474
α_1=α_2= 1.667	2	770	665	505	400	360	270	0.370
α_p^2 = α_e^2= 0.722	3	820	725	570	460	415	320	0.304
N= 1.333	4	860	775	625	510	465	365	0.257
A= 1	5	890	815	675	560	515	410	0.223
	6	915	850	715	605	555	445	0.197
	0	460	375	265				0.823
36/4	1	550	465	345	270	240		0.554
α_1=α_2= 1.333	2	610	535	415	335	300	225	0.417
α_p^2 = α_e^2= 0.556	3	650	585	470	385	355	275	0.334
N= 1.000	4	675	620	520	435	400	320	0.279
A= 1	5	695	650	555	475	440	355	0.240
	6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	1425	1120	765				0.274
		1	1565	1245	865	640	565		0.236
α_1=α_2=	4.000	2	1690	1360	960	720	635	465	0.207
α_p^2 = α_e^2=	1.556	3	1810	1470	1045	795	705	520	0.185
N=	4.000	4	1915	1575	1135	875	775	570	0.166
A=	2	5	2010	1670	1215	945	845	620	0.151
		6	2100	1760	1295	1010	910	675	0.139
36/7		0	710	560	380				0.549
		1	845	680	480	360	315		0.414
α_1=α_2=	2.000	2	955	785	565	435	385	285	0.333
α_p^2 = α_e^2=	0.778	3	1050	880	645	505	455	335	0.278
N=	2.000	4	1125	960	720	570	510	390	0.239
A=	1	5	1185	1030	790	630	570	440	0.209
		6	1235	1090	855	685	625	485	0.186
36/5		0	600	490	350				0.659
		1	700	585	430	335	295		0.474
α_1=α_2=	1.667	2	770	665	505	400	360	270	0.370
α_p^2 = α_e^2=	0.722	3	820	725	570	460	415	320	0.304
N=	1.333	4	860	775	625	510	465	365	0.257
A=	1	5	890	815	675	560	515	410	0.223
		6	915	850	715	605	555	445	0.197
36/4		0	460	375	265				0.823
		1	550	465	345	270	240		0.554
α_1=α_2=	1.333	2	610	535	415	335	300	225	0.417
α_p^2 = α_e^2=	0.556	3	650	585	470	385	355	275	0.334
N=	1.000	4	675	620	520	435	400	320	0.279
A=	1	5	695	650	555	475	440	355	0.240
		6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
		0	1425	1120	765				0.274
36/14		1	1565	1245	865	640	565		0.236
α_1=α_2=	4.000	2	1690	1360	960	720	635	465	0.207
α_p^2 = α_e^2=	1.556	3	1810	1470	1045	795	705	520	0.185
N=	4.000	4	1915	1575	1135	875	775	570	0.166
A=	2	5	2010	1670	1215	945	845	620	0.151
		6	2100	1760	1295	1010	910	675	0.139
		0	710	560	380				0.549
36/7		1	845	680	480	360	315		0.414
α_1=α_2=	2.000	2	955	785	565	435	385	285	0.333
α_p^2 = α_e^2=	0.778	3	1050	880	645	505	455	335	0.278
N=	2.000	4	1125	960	720	570	510	390	0.239
A=	1	5	1185	1030	790	630	570	440	0.209
		6	1235	1090	855	685	625	485	0.186
		0	600	490	350				0.659
36/5		1	700	585	430	335	295		0.474
α_1=α_2=	1.667	2	770	665	505	400	360	270	0.370
α_p^2 = α_e^2=	0.722	3	820	725	570	460	415	320	0.304
N=	1.333	4	860	775	625	510	465	365	0.257
A=	1	5	890	815	675	560	515	410	0.223
		6	915	850	715	605	555	445	0.197
		0	460	375	265				0.823
36/4		1	550	465	345	270	240		0.554
α_1=α_2=	1.333	2	610	535	415	335	300	225	0.417
α_p^2 = α_e^2=	0.556	3	650	585	470	385	355	275	0.334
N=	1.000	4	675	620	520	435	400	320	0.279
A=	1	5	695	650	555	475	440	355	0.240
		6	710	670	590	510	475	390	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1110	875	595				0.785
36/14	1	1290	1035	730	545	480		0.552
$\alpha_1=\alpha_2=$ 4.000	2	1445	1180	845	650	575	425	0.426
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1580	1310	955	745	665	495	0.346
N= 4.000	4	1690	1430	1060	830	750	565	0.292
A= 2	5	1780	1530	1160	915	825	635	0.252
	6	1860	1620	1250	995	900	700	0.222
	0	555	435	295				1.569
36/7	1	720	590	420	325	285		0.851
$\alpha_1=\alpha_2=$ 2.000	2	845	715	530	415	375	280	0.584
$\alpha_p^2=\alpha_e^2=$ 0.778	3	930	810	625	495	450	350	0.445
N= 2.000	4	990	885	705	570	520	405	0.359
A= 1	5	1035	940	775	640	585	465	0.301
	6	1065	985	830	700	645	515	0.259
	0	470	380	270				1.883
36/5	1	585	500	375	295	270		0.936
$\alpha_1=\alpha_2=$ 1.667	2	655	585	465	375	340	265	0.623
$\alpha_p^2=\alpha_e^2=$ 0.722	3	700	640	530	440	405	320	0.467
N= 1.333	4	725	680	580	495	460	375	0.373
A= 1	5	745	705	620	545	505	420	0.311
	6	755	725	655	580	545	460	0.266
	0	355	290	205				2.354
36/4	1	460	400	310	245	220		1.039
$\alpha_1=\alpha_2=$ 1.333	2	515	470	385	315	290	230	0.667
$\alpha_p^2=\alpha_e^2=$ 0.556	3	545	510	440	375	345	280	0.491
N= 1.000	4	560	535	475	420	390	325	0.388
A= 1	5	570	550	505	450	430	365	0.321
	6	580	560	520	480	455	395	0.274

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1500	1180	810				0.710
36/14	1	1685	1345	940	700	620		0.514
$\alpha_1=\alpha_2=$ 4.000	2	1850	1500	1060	805	710	525	0.403
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2000	1640	1175	910	805	595	0.331
N= 4.000	4	2130	1770	1290	1000	900	665	0.281
A= 2	5	2240	1885	1395	1090	980	735	0.244
	6	2340	1995	1495	1175	1060	805	0.216
	0	750	590	405				1.419
36/7	1	925	750	530	400	355		0.805
$\alpha_1=\alpha_2=$ 2.000	2	1065	885	645	500	450	330	0.562
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1170	995	745	585	530	400	0.432
N= 2.000	4	1255	1090	840	670	605	465	0.350
A= 1	5	1320	1170	920	745	675	530	0.295
	6	1370	1235	995	815	745	585	0.254
	0	635	515	365				1.703
36/5	1	760	640	475	375	335		0.889
$\alpha_1=\alpha_2=$ 1.667	2	845	735	570	455	410	315	0.602
$\alpha_p^2=\alpha_e^2=$ 0.722	3	900	810	650	530	480	380	0.455
N= 1.333	4	940	865	715	595	545	435	0.365
A= 1	5	970	905	770	650	600	485	0.305
	6	995	935	815	700	655	535	0.262
	0	485	395	280				2.129
36/4	1	595	510	385	305	275		0.993
$\alpha_1=\alpha_2=$ 1.333	2	665	595	470	380	350	270	0.647
$\alpha_p^2=\alpha_e^2=$ 0.556	3	710	650	540	450	410	325	0.480
N= 1.000	4	735	690	590	505	465	380	0.382
A= 1	5	755	715	630	550	515	425	0.317
	6	765	735	660	590	555	465	0.271

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1725	1360	930				0.302
36/14	1	1915	1525	1065	790	700		0.260
$\alpha_1=\alpha_2=$ 4.000	2	2085	1680	1185	895	790	585	0.228
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
	0	860	680	465				0.605
36/7	1	1040	840	590	445	395		0.456
$\alpha_1=\alpha_2=$ 2.000	2	1185	980	705	545	490	360	0.366
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
	0	730	595	425				0.726
36/5	1	860	720	535	415	370		0.522
$\alpha_1=\alpha_2=$ 1.667	2	950	825	630	500	450	340	0.408
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
	0	555	455	325				0.907
36/4	1	675	575	430	340	300		0.610
$\alpha_1=\alpha_2=$ 1.333	2	750	660	520	420	380	290	0.459
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1725	1360	930				0.302
	1	1915	1525	1065	790	700		0.260
$\alpha_1=\alpha_2$= 4.000	2	2085	1680	1185	895	790	585	0.228
$\alpha_p^2 = \alpha_e^2$= 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
36/7	0	860	680	465				0.605
	1	1040	840	590	445	395		0.456
$\alpha_1=\alpha_2$= 2.000	2	1185	980	705	545	490	360	0.366
$\alpha_p^2 = \alpha_e^2$= 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
36/5	0	730	595	425				0.726
	1	860	720	535	415	370		0.522
$\alpha_1=\alpha_2$= 1.667	2	950	825	630	500	450	340	0.408
$\alpha_p^2 = \alpha_e^2$= 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
36/4	0	555	455	325				0.907
	1	675	575	430	340	300		0.610
$\alpha_1=\alpha_2$= 1.333	2	750	660	520	420	380	290	0.459
$\alpha_p^2 = \alpha_e^2$= 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1725	1360	930				0.302
36/14	1	1915	1525	1065	790	700		0.260
α_1=α_2= 4.000	2	2085	1680	1185	895	790	585	0.228
α_p^2 = α_e^2= 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
	0	860	680	465				0.605
36/7	1	1040	840	590	445	395		0.456
α_1=α_2= 2.000	2	1185	980	705	545	490	360	0.366
α_p^2 = α_e^2= 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
	0	730	595	425				0.726
36/5	1	860	720	535	415	370		0.522
α_1=α_2= 1.667	2	950	825	630	500	450	340	0.408
α_p^2 = α_e^2= 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
	0	555	455	325				0.907
36/4	1	675	575	430	340	300		0.610
α_1=α_2= 1.333	2	750	660	520	420	380	290	0.459
α_p^2 = α_e^2= 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1725	1360	930				0.302
	1	1915	1525	1065	790	700		0.260
$\alpha_1=\alpha_2=$ 4.000	2	2085	1680	1185	895	790	585	0.228
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2235	1825	1305	1000	885	655	0.203
N= 4.000	4	2375	1960	1415	1095	980	720	0.183
A= 2	5	2495	2085	1525	1185	1065	790	0.167
	6	2605	2200	1630	1275	1145	860	0.153
36/7	0	860	680	465				0.605
	1	1040	840	590	445	395		0.456
$\alpha_1=\alpha_2=$ 2.000	2	1185	980	705	545	490	360	0.366
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1300	1100	815	635	570	430	0.306
N= 2.000	4	1395	1200	910	720	650	500	0.263
A= 1	5	1470	1290	1000	800	725	560	0.230
	6	1530	1365	1080	875	795	620	0.205
36/5	0	730	595	425				0.726
	1	860	720	535	415	370		0.522
$\alpha_1=\alpha_2=$ 1.667	2	950	825	630	500	450	340	0.408
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1015	900	715	575	525	410	0.334
N= 1.333	4	1060	965	785	645	590	465	0.283
A= 1	5	1100	1010	845	710	650	520	0.246
	6	1125	1050	895	765	705	570	0.217
36/4	0	555	455	325				0.907
	1	675	575	430	340	300		0.610
$\alpha_1=\alpha_2=$ 1.333	2	750	660	520	420	380	290	0.459
$\alpha_p^2=\alpha_e^2=$ 0.556	3	800	725	590	490	445	350	0.368
N= 1.000	4	830	770	650	550	505	405	0.307
A= 1	5	855	805	695	600	555	455	0.264
	6	870	830	735	645	600	500	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	1110	875	600				1.039
		1	1375	1115	790	605	535		0.700
α_1=α_2=	4.000	2	1585	1320	960	745	670	500	0.527
α_p^2 = α_e^2=	1.556	3	1745	1490	1120	880	795	605	0.423
N=	4.000	4	1870	1630	1255	1005	910	700	0.353
A=	2	5	1965	1745	1380	1120	1020	780	0.303
		6	2035	1840	1490	1225	1120	855	0.266
36/7		0	555	435	300				2.078
		1	790	660	480	370	335		1.054
α_1=α_2=	2.000	2	935	815	625	500	455	350	0.706
α_p^2 = α_e^2=	0.778	3	1015	920	745	610	560	425	0.531
N=	2.000	4	1070	990	835	705	650	500	0.426
A=	1	5	1105	1040	905	780	730	575	0.355
		6	1125	1075	960	845	795	650	0.305
36/5		0	470	380	270				2.494
		1	625	550	425	340	305		1.152
α_1=α_2=	1.667	2	700	640	535	445	410	315	0.749
α_p^2 = α_e^2=	0.722	3	735	695	605	525	490	390	0.555
N=	1.333	4	755	725	655	585	550	465	0.441
A=	1	5	770	745	690	630	600	515	0.365
		6	780	760	715	660	635	560	0.312
36/4		0	355	290	205				3.117
		1	495	440	350	285	260		1.269
α_1=α_2=	1.333	2	545	510	440	375	350	285	0.797
α_p^2 = α_e^2=	0.556	3	570	545	490	440	415	350	0.581
N=	1.000	4	580	565	525	480	460	400	0.457
A=	1	5	585	575	545	510	490	440	0.376
		6	590	580	560	530	515	470	0.320

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1500	1180	815				0.940
36/14	1	1775	1425	1005	760	675		0.653
$\alpha_1=\alpha_2=$ 4.000	2	2005	1645	1185	915	815	600	0.500
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2195	1835	1350	1050	945	710	0.406
N= 4.000	4	2350	2005	1505	1185	1065	815	0.341
A= 2	5	2480	2150	1645	1310	1185	915	0.294
	6	2585	2275	1775	1430	1295	1005	0.259
	0	750	590	405				1.879
36/7	1	1000	820	590	455	405		1.001
$\alpha_1=\alpha_2=$ 2.000	2	1175	1000	750	590	530	405	0.682
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1290	1135	885	715	645	500	0.517
N= 2.000	4	1370	1240	1000	820	750	590	0.417
A= 1	5	1430	1315	1095	920	845	675	0.349
	6	1470	1370	1175	1000	930	750	0.300
	0	635	515	365				2.255
36/5	1	805	695	525	415	375		1.098
$\alpha_1=\alpha_2=$ 1.667	2	905	810	650	530	485	380	0.726
$\alpha_p^2=\alpha_e^2=$ 0.722	3	960	890	745	630	580	465	0.542
N= 1.333	4	995	940	815	705	655	535	0.433
A= 1	5	1015	970	870	770	720	605	0.360
	6	1035	995	910	820	775	660	0.308
	0	485	395	280				2.819
36/4	1	635	560	435	345	315		1.217
$\alpha_1=\alpha_2=$ 1.333	2	710	650	540	450	415	330	0.776
$\alpha_p^2=\alpha_e^2=$ 0.556	3	745	705	615	530	495	405	0.569
N= 1.000	4	765	735	665	590	555	470	0.450
A= 1	5	780	755	700	635	605	520	0.372
	6	790	770	725	670	645	565	0.317

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2235	1760	1210				0.823
36/14	1	2515	2010	1405	1055	935		0.594
$\alpha_1=\alpha_2=$ 4.000	2	2765	2240	1590	1215	1075	795	0.465
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2990	2455	1765	1365	1215	900	0.382
N= 4.000	4	3185	2650	1935	1505	1350	1005	0.324
A= 2	5	3355	2830	2095	1640	1475	1110	0.282
	6	3505	2995	2245	1770	1595	1220	0.249
	0	1115	880	605				1.646
36/7	1	1380	1120	795	605	535		0.931
$\alpha_1=\alpha_2=$ 2.000	2	1590	1325	965	750	675	500	0.649
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1750	1495	1120	885	795	610	0.498
N= 2.000	4	1875	1635	1260	1005	910	705	0.404
A= 1	5	1970	1755	1385	1125	1020	795	0.340
	6	2045	1850	1495	1230	1125	885	0.293
	0	945	770	550				1.975
36/5	1	1135	960	715	560	505		1.027
$\alpha_1=\alpha_2=$ 1.667	2	1260	1105	855	685	620	480	0.694
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1350	1210	975	795	725	570	0.524
N= 1.333	4	1410	1290	1070	895	820	655	0.421
A= 1	5	1450	1355	1155	980	905	730	0.352
	6	1485	1400	1220	1055	980	805	0.302
	0	720	590	420				2.469
36/4	1	895	765	580	460	415		1.147
$\alpha_1=\alpha_2=$ 1.333	2	995	890	710	575	525	410	0.747
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1060	970	810	675	620	495	0.554
N= 1.000	4	1100	1030	885	760	705	570	0.440
A= 1	5	1125	1070	945	825	775	640	0.365
	6	1145	1095	990	885	835	700	0.312

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df},\ S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf},\ \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2290	1800	1240				0.348
36/14	1	2570	2050	1435	1075	950		0.299
α_1=α_2= 4.000	2	2820	2285	1620	1235	1095	805	0.263
α_p^2 = α_e^2= 1.556	3	3045	2500	1795	1385	1235	915	0.234
N= 4.000	4	3245	2695	1965	1525	1370	1020	0.211
A= 2	5	3415	2875	2125	1660	1495	1125	0.192
	6	3570	3040	2280	1790	1615	1235	0.176
	0	1145	900	620				0.696
36/7	1	1410	1140	810	615	545		0.525
α_1=α_2= 2.000	2	1620	1345	980	760	685	510	0.422
α_p^2 = α_e^2= 0.778	3	1785	1520	1140	895	805	615	0.352
N= 2.000	4	1910	1665	1280	1020	925	715	0.303
A= 1	5	2010	1780	1405	1135	1035	805	0.265
	6	2085	1880	1520	1245	1135	895	0.236
	0	970	785	560				0.835
36/5	1	1155	980	725	570	510		0.601
α_1=α_2= 1.667	2	1285	1125	870	695	630	485	0.469
α_p^2 = α_e^2= 0.722	3	1375	1235	990	805	735	575	0.385
N= 1.333	4	1435	1315	1090	905	830	660	0.326
A= 1	5	1480	1380	1170	995	920	740	0.283
	6	1515	1425	1240	1070	995	815	0.250
	0	740	600	430				1.044
36/4	1	910	780	590	465	420		0.702
α_1=α_2= 1.333	2	1015	905	720	585	530	415	0.528
α_p^2 = α_e^2= 0.556	3	1080	990	820	685	630	500	0.424
N= 1.000	4	1120	1050	900	770	715	575	0.354
A= 1	5	1150	1090	960	840	785	645	0.304
	6	1170	1120	1010	895	845	710	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
F_{y-deck}=	33	ksi
F_{u-deck}=	45	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y-framing}$=	50	ksi
$F_{u-framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2290	1800	1240				0.348
	1	2570	2050	1435	1075	950		0.299
$\alpha_1=\alpha_2=$ 4.000	2	2820	2285	1620	1235	1095	805	0.263
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3045	2500	1795	1385	1235	915	0.234
N= 4.000	4	3245	2695	1965	1525	1370	1020	0.211
A= 2	5	3415	2875	2125	1660	1495	1125	0.192
	6	3570	3040	2280	1790	1615	1235	0.176
36/7	0	1145	900	620				0.696
	1	1410	1140	810	615	545		0.525
$\alpha_1=\alpha_2=$ 2.000	2	1620	1345	980	760	685	510	0.422
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1785	1520	1140	895	805	615	0.352
N= 2.000	4	1910	1665	1280	1020	925	715	0.303
A= 1	5	2010	1780	1405	1135	1035	805	0.265
	6	2085	1880	1520	1245	1135	895	0.236
36/5	0	970	785	560				0.835
	1	1155	980	725	570	510		0.601
$\alpha_1=\alpha_2=$ 1.667	2	1285	1125	870	695	630	485	0.469
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1375	1235	990	805	735	575	0.385
N= 1.333	4	1435	1315	1090	905	830	660	0.326
A= 1	5	1480	1380	1170	995	920	740	0.283
	6	1515	1425	1240	1070	995	815	0.250
36/4	0	740	600	430				1.044
	1	910	780	590	465	420		0.702
$\alpha_1=\alpha_2=$ 1.333	2	1015	905	720	585	530	415	0.528
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1080	990	820	685	630	500	0.424
N= 1.000	4	1120	1050	900	770	715	575	0.354
A= 1	5	1150	1090	960	840	785	645	0.304
	6	1170	1120	1010	895	845	710	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	2290	1800	1240				0.348
		1	2570	2050	1435	1075	950		0.299
α_1=α_2=	4.000	2	2820	2285	1620	1235	1095	805	0.263
α_p^2 = α_e^2=	1.556	3	3045	2500	1795	1385	1235	915	0.234
N=	4.000	4	3245	2695	1965	1525	1370	1020	0.211
A=	2	5	3415	2875	2125	1660	1495	1125	0.192
		6	3570	3040	2280	1790	1615	1235	0.176
36/7		0	1145	900	620				0.696
		1	1410	1140	810	615	545		0.525
α_1=α_2=	2.000	2	1620	1345	980	760	685	510	0.422
α_p^2 = α_e^2=	0.778	3	1785	1520	1140	895	805	615	0.352
N=	2.000	4	1910	1665	1280	1020	925	715	0.303
A=	1	5	2010	1780	1405	1135	1035	805	0.265
		6	2085	1880	1520	1245	1135	895	0.236
36/5		0	970	785	560				0.835
		1	1155	980	725	570	510		0.601
α_1=α_2=	1.667	2	1285	1125	870	695	630	485	0.469
α_p^2 = α_e^2=	0.722	3	1375	1235	990	805	735	575	0.385
N=	1.333	4	1435	1315	1090	905	830	660	0.326
A=	1	5	1480	1380	1170	995	920	740	0.283
		6	1515	1425	1240	1070	995	815	0.250
36/4		0	740	600	430				1.044
		1	910	780	590	465	420		0.702
α_1=α_2=	1.333	2	1015	905	720	585	530	415	0.528
α_p^2 = α_e^2=	0.556	3	1080	990	820	685	630	500	0.424
N=	1.000	4	1120	1050	900	770	715	575	0.354
A=	1	5	1150	1090	960	840	785	645	0.304
		6	1170	1120	1010	895	845	710	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1110	875	605				1.311
	1	1470	1205	865	665	595		0.848
$\alpha_1=\alpha_2=$ 4.000	2	1720	1465	1095	830	735	550	0.627
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	1895	1660	1255	940	835	625	0.497
N= 4.000	4	2015	1810	1405	1055	935	700	0.412
A= 2	5	2100	1925	1560	1170	1040	780	0.352
	6	2160	2010	1710	1280	1140	855	0.307
36/7	0	555	435	300				2.622
	1	860	730	545	415	365		1.254
$\alpha_1=\alpha_2=$ 2.000	2	1005	905	700	525	465	350	0.824
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1080	1005	855	640	570	425	0.614
N= 2.000	4	1120	1065	945	755	670	500	0.489
A= 1	5	1145	1100	1005	865	770	575	0.406
	6	1160	1125	1045	960	870	650	0.348
36/5	0	470	380	270				3.146
	1	665	595	475	360	320		1.363
$\alpha_1=\alpha_2=$ 1.667	2	730	690	595	475	425	315	0.870
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	760	730	665	590	525	390	0.639
N= 1.333	4	775	755	705	650	625	465	0.505
A= 1	5	785	770	730	690	665	545	0.417
	6	790	780	750	715	695	620	0.355
36/4	0	355	290	205				3.933
	1	520	475	395	310	275		1.492
$\alpha_1=\alpha_2=$ 1.333	2	565	540	485	425	380	285	0.921
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	580	565	530	490	470	360	0.666
N= 1.000	4	590	580	555	525	510	435	0.521
A= 1	5	595	585	570	545	535	495	0.429
	6	595	590	575	560	550	515	0.364

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1500	1180	815				1.185
36/14	1	1875	1520	1080	830	735		0.794
$\alpha_1=\alpha_2=$ 4.000	2	2165	1805	1320	1030	925	695	0.597
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2385	2045	1540	1215	1095	845	0.478
N= 4.000	4	2555	2235	1735	1390	1260	950	0.399
A= 2	5	2680	2395	1905	1550	1405	1050	0.342
	6	2780	2525	2055	1695	1540	1155	0.300
	0	750	590	405				2.371
36/7	1	1080	900	660	515	460		1.194
$\alpha_1=\alpha_2=$ 2.000	2	1275	1115	865	695	630	475	0.798
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1390	1260	1025	845	770	575	0.599
N= 2.000	4	1455	1355	1150	975	900	680	0.480
A= 1	5	1500	1420	1245	1080	1010	780	0.400
	6	1530	1465	1315	1165	1095	880	0.343
	0	635	515	365				2.845
36/5	1	855	750	585	465	425		1.303
$\alpha_1=\alpha_2=$ 1.667	2	955	875	730	615	565	430	0.845
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1000	950	830	725	675	530	0.625
N= 1.333	4	1030	990	895	805	760	635	0.496
A= 1	5	1045	1015	940	860	825	715	0.411
	6	1055	1030	975	905	870	770	0.351
	0	485	395	280				3.557
36/4	1	675	605	485	395	360		1.435
$\alpha_1=\alpha_2=$ 1.333	2	740	695	605	520	485	385	0.899
$\alpha_p^2=\alpha_e^2=$ 0.556	3	770	740	675	605	570	485	0.654
N= 1.000	4	785	765	715	660	630	550	0.514
A= 1	5	795	780	740	695	675	605	0.424
	6	800	790	760	725	705	645	0.360

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	2235	1760	1215				1.038
		1	2625	2110	1485	1125	995		0.725
$\alpha_1=\alpha_2$=	4.000	2	2955	2420	1735	1340	1200	885	0.557
$\alpha_p^2=\alpha_e^2$=	1.556	3	3230	2695	1975	1535	1380	1040	0.452
N=	4.000	4	3460	2940	2195	1725	1555	1190	0.381
A=	2	5	3650	3155	2400	1905	1720	1330	0.329
		6	3805	3340	2590	2075	1880	1460	0.289
36/7		0	1115	880	605				2.077
		1	1475	1210	865	670	600		1.114
$\alpha_1=\alpha_2$=	2.000	2	1730	1470	1095	860	775	595	0.761
$\alpha_p^2=\alpha_e^2$=	0.778	3	1900	1670	1295	1035	940	730	0.578
N=	2.000	4	2025	1820	1460	1195	1090	855	0.466
A=	1	5	2110	1935	1600	1335	1225	975	0.390
		6	2175	2020	1720	1460	1350	1085	0.336
36/5		0	945	770	550				2.492
		1	1195	1025	775	610	555		1.224
$\alpha_1=\alpha_2$=	1.667	2	1335	1195	955	780	710	555	0.811
$\alpha_p^2=\alpha_e^2$=	0.722	3	1420	1310	1095	915	845	675	0.606
N=	1.333	4	1475	1385	1200	1030	960	780	0.484
A=	1	5	1510	1435	1280	1125	1055	875	0.403
		6	1530	1475	1340	1200	1135	960	0.345
36/4		0	720	590	420				3.115
		1	940	825	635	510	460		1.357
$\alpha_1=\alpha_2$=	1.333	2	1050	960	795	660	605	480	0.868
$\alpha_p^2=\alpha_e^2$=	0.556	3	1105	1040	900	775	720	590	0.638
N=	1.000	4	1140	1090	975	865	815	680	0.504
A=	1	5	1160	1120	1030	935	885	760	0.417
		6	1170	1140	1070	985	945	825	0.355

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2885	2275	1570				0.927
	1	3285	2625	1840	1390	1230		0.669
$\alpha_1=\alpha_2=$ 4.000	2	3635	2950	2100	1615	1430	1060	0.523
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3940	3245	2345	1815	1625	1210	0.430
N= 4.000	4	4205	3515	2580	2010	1805	1360	0.365
A= 2	5	4435	3760	2800	2200	1980	1510	0.317
	6	4630	3980	3010	2380	2150	1655	0.280
36/7	0	1440	1135	785				1.854
	1	1815	1475	1050	805	715		1.047
$\alpha_1=\alpha_2=$ 2.000	2	2100	1755	1290	1005	900	680	0.729
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2315	1990	1505	1190	1075	825	0.560
N= 2.000	4	2475	2175	1695	1360	1235	960	0.454
A= 1	5	2595	2330	1865	1520	1385	1085	0.382
	6	2690	2450	2010	1665	1525	1205	0.330
36/5	0	1225	995	710				2.224
	1	1485	1260	940	740	665		1.156
$\alpha_1=\alpha_2=$ 1.667	2	1655	1455	1135	915	830	645	0.780
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1765	1600	1300	1065	975	770	0.589
N= 1.333	4	1845	1700	1430	1200	1105	885	0.473
A= 1	5	1895	1780	1535	1315	1220	990	0.395
	6	1935	1835	1620	1415	1320	1090	0.340
36/4	0	930	760	545				2.781
	1	1170	1010	765	605	550		1.290
$\alpha_1=\alpha_2=$ 1.333	2	1305	1175	945	770	705	550	0.839
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1385	1280	1075	905	835	670	0.622
N= 1.000	4	1435	1350	1175	1015	945	775	0.494
A= 1	5	1465	1400	1250	1105	1040	865	0.410
	6	1490	1435	1310	1180	1115	950	0.350

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	33	ksi
$F_{u\text{-deck}}$=	45	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2885	2275	1570				0.391
36/14	1	3285	2625	1840	1390	1230		0.336
$\alpha_1=\alpha_2=$ 4.000	2	3635	2950	2100	1615	1430	1060	0.295
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3940	3245	2345	1815	1625	1210	0.263
N= 4.000	4	4205	3515	2580	2010	1805	1360	0.237
A= 2	5	4435	3760	2800	2200	1980	1510	0.216
	6	4630	3980	3010	2380	2150	1655	0.198
	0	1440	1135	785				0.782
36/7	1	1815	1475	1050	805	715		0.590
$\alpha_1=\alpha_2=$ 2.000	2	2100	1755	1290	1005	900	680	0.474
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2315	1990	1505	1190	1075	825	0.396
N= 2.000	4	2475	2175	1695	1360	1235	960	0.340
A= 1	5	2595	2330	1865	1520	1385	1085	0.298
	6	2690	2450	2010	1665	1525	1205	0.265
	0	1225	995	710				0.938
36/5	1	1485	1260	940	740	665		0.675
$\alpha_1=\alpha_2=$ 1.667	2	1655	1455	1135	915	830	645	0.527
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1765	1600	1300	1065	975	770	0.432
N= 1.333	4	1845	1700	1430	1200	1105	885	0.366
A= 1	5	1895	1780	1535	1315	1220	990	0.318
	6	1935	1835	1620	1415	1320	1090	0.281
	0	930	760	545				1.172
36/4	1	1170	1010	765	605	550		0.788
$\alpha_1=\alpha_2=$ 1.333	2	1305	1175	945	770	705	550	0.594
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1385	1280	1075	905	835	670	0.476
N= 1.000	4	1435	1350	1175	1015	945	775	0.397
A= 1	5	1465	1400	1250	1105	1040	865	0.341
	6	1490	1435	1310	1180	1115	950	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df},\ S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf},\ \phi_{db} S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	33	ksi
$F_{u\text{-}deck}$=	45	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2885	2275	1570				0.391
36/14	1	3285	2625	1840	1390	1230		0.336
α_1=α_2= 4.000	2	3635	2950	2100	1615	1430	1060	0.295
α_p^2 = α_e^2= 1.556	3	3940	3245	2345	1815	1625	1210	0.263
N= 4.000	4	4205	3515	2580	2010	1805	1360	0.237
A= 2	5	4435	3760	2800	2200	1980	1510	0.216
	6	4630	3980	3010	2380	2150	1655	0.198
	0	1440	1135	785				0.782
36/7	1	1815	1475	1050	805	715		0.590
α_1=α_2= 2.000	2	2100	1755	1290	1005	900	680	0.474
α_p^2 = α_e^2= 0.778	3	2315	1990	1505	1190	1075	825	0.396
N= 2.000	4	2475	2175	1695	1360	1235	960	0.340
A= 1	5	2595	2330	1865	1520	1385	1085	0.298
	6	2690	2450	2010	1665	1525	1205	0.265
	0	1225	995	710				0.938
36/5	1	1485	1260	940	740	665		0.675
α_1=α_2= 1.667	2	1655	1455	1135	915	830	645	0.527
α_p^2 = α_e^2= 0.722	3	1765	1600	1300	1065	975	770	0.432
N= 1.333	4	1845	1700	1430	1200	1105	885	0.366
A= 1	5	1895	1780	1535	1315	1220	990	0.318
	6	1935	1835	1620	1415	1320	1090	0.281
	0	930	760	545				1.172
36/4	1	1170	1010	765	605	550		0.788
α_1=α_2= 1.333	2	1305	1175	945	770	705	550	0.594
α_p^2 = α_e^2= 0.556	3	1385	1280	1075	905	835	670	0.476
N= 1.000	4	1435	1350	1175	1015	945	775	0.397
A= 1	5	1465	1400	1250	1105	1040	865	0.341
	6	1490	1435	1310	1180	1115	950	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

Deck F_y:	50	ksi
Deck F_{u1}:	65	ksi
Framing F_{y2}:	33	ksi
Framing F_{u2}:	45	ksi

P_{nf}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	418	605	939	1118	1118	1118
20	418	565	901	1250	1357	1357
18	418	565	841	1222	1714	1797
16	418	565	841	1183	1698	2267

P_{ns}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	603	603	603	603	603	603
20	806	806	806	806	806	806
18	1228	1228	1228	1228	1228	1228
16	1740	1740	1740	1740	1740	1740

S_f, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0178	0.0154	0.0113	0.0076	0.0076	0.0076
20	0.0178	0.0161	0.0127	0.0087	0.0069	0.0069
18	0.0178	0.0161	0.0141	0.0112	0.0070	0.0060
16	0.0178	0.0161	0.0141	0.0126	0.0094	0.0053

S_s, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0175	0.0175	0.0175	0.0175	0.0175	0.0175
20	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
18	0.0138	0.0138	0.0138	0.0138	0.0138	0.0138
16	0.0123	0.0123	0.0123	0.0123	0.0123	0.0123

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	770	605	410				0.647
36/14	1	955	775	550	415	365		0.468
$\alpha_1=\alpha_2=$ 4.000	2	1100	920	670	520	465	345	0.366
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	1215	1040	780	615	555	420	0.301
N= 4.000	4	1300	1135	880	705	635	485	0.255
A= 2	5	1365	1215	965	785	715	540	0.222
	6	1415	1280	1040	860	785	590	0.196
	0	385	300	205				1.293
36/7	1	550	460	335	260	230		0.732
$\alpha_1=\alpha_2=$ 2.000	2	650	565	440	350	315	240	0.511
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	705	640	520	430	390	295	0.392
N= 2.000	4	745	690	585	495	455	345	0.318
A= 1	5	765	725	630	545	510	400	0.268
	6	780	745	670	590	555	450	0.231
	0	325	265	185				1.552
36/5	1	435	380	295	235	215		0.809
$\alpha_1=\alpha_2=$ 1.667	2	485	445	370	310	285	220	0.547
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	510	480	420	365	340	270	0.413
N= 1.333	4	525	505	455	405	385	325	0.332
A= 1	5	535	515	480	435	415	360	0.277
	6	540	525	495	460	440	390	0.238
	0	245	200	140				1.940
36/4	1	345	305	245	200	180		0.903
$\alpha_1=\alpha_2=$ 1.333	2	380	355	305	260	245	195	0.588
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	395	380	340	305	290	245	0.436
N= 1.000	4	400	390	365	335	320	280	0.347
A= 1	5	405	395	375	355	340	305	0.288
	6	410	400	385	370	355	325	0.246

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1110	875	595				0.560
	1	1305	1050	740	550	485		0.421
$\alpha_1=\alpha_2=$ 4.000	2	1470	1205	865	665	585	435	0.337
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1610	1345	985	765	685	510	0.281
N= 4.000	4	1725	1465	1095	860	775	585	0.241
A= 2	5	1815	1570	1195	950	860	660	0.211
	6	1895	1665	1290	1035	940	730	0.187
36/7	0	555	435	295				1.120
	1	735	600	430	330	290		0.674
$\alpha_1=\alpha_2=$ 2.000	2	860	730	545	430	385	290	0.482
$\alpha_p^2=\alpha_e^2=$ 0.778	3	945	830	645	515	470	365	0.375
N= 2.000	4	1005	905	730	595	545	425	0.307
A= 1	5	1050	965	800	665	610	485	0.260
	6	1080	1005	855	725	670	540	0.225
36/5	0	470	380	270				1.344
	1	595	510	385	305	275		0.749
$\alpha_1=\alpha_2=$ 1.667	2	665	595	475	385	350	275	0.519
$\alpha_p^2=\alpha_e^2=$ 0.722	3	705	650	545	455	420	335	0.397
N= 1.333	4	735	690	595	515	475	390	0.321
A= 1	5	750	715	635	560	525	435	0.270
	6	760	735	665	595	565	480	0.233
36/4	0	360	290	205				1.681
	1	470	410	315	250	230		0.842
$\alpha_1=\alpha_2=$ 1.333	2	520	475	395	330	300	240	0.562
$\alpha_p^2=\alpha_e^2=$ 0.556	3	550	515	450	385	360	295	0.422
N= 1.000	4	565	540	485	430	405	340	0.337
A= 1	5	575	555	510	465	440	380	0.281
	6	580	565	530	490	470	410	0.241

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1725	1360	925				0.410
	1	1925	1535	1075	795	700		0.330
α_1=α_2= 4.000	2	2105	1700	1205	910	800	590	0.276
α_p^2 = α_e^2= 1.556	3	2270	1855	1330	1020	905	665	0.237
N= 4.000	4	2415	2000	1450	1125	1005	740	0.208
A= 2	5	2540	2130	1565	1220	1100	815	0.185
	6	2655	2250	1675	1315	1185	890	0.167
36/7	0	860	680	460				0.819
	1	1050	850	600	455	400		0.552
α_1=α_2= 2.000	2	1205	1000	725	560	500	370	0.416
α_p^2 = α_e^2= 0.778	3	1325	1125	835	655	590	445	0.334
N= 2.000	4	1420	1230	940	745	675	520	0.279
A= 1	5	1495	1320	1035	830	755	585	0.239
	6	1555	1395	1115	910	830	650	0.210
36/5	0	730	595	425				0.983
	1	865	730	540	425	375		0.621
α_1=α_2= 1.667	2	960	835	645	515	465	350	0.454
α_p^2 = α_e^2= 0.722	3	1025	915	730	595	540	425	0.358
N= 1.333	4	1075	980	805	665	610	485	0.295
A= 1	5	1110	1025	865	730	675	540	0.251
	6	1135	1065	915	785	730	595	0.219
36/4	0	555	455	325				1.229
	1	680	580	440	345	305		0.711
α_1=α_2= 1.333	2	760	675	530	430	390	300	0.501
α_p^2 = α_e^2= 0.556	3	805	735	605	505	460	365	0.386
N= 1.000	4	840	780	665	565	525	420	0.314
A= 1	5	860	815	710	620	575	475	0.265
	6	875	835	750	660	620	520	0.229

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2055	1620	1105				0.274
36/14	1	2260	1795	1255	925	815		0.236
$\alpha_1=\alpha_2=$ 4.000	2	2445	1965	1385	1040	915	675	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
	0	1025	810	550				0.549
36/7	1	1220	980	690	520	455		0.414
$\alpha_1=\alpha_2=$ 2.000	2	1380	1135	815	630	560	410	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
	0	870	710	505				0.659
36/5	1	1010	845	625	485	430		0.474
$\alpha_1=\alpha_2=$ 1.667	2	1115	960	730	580	520	390	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
	0	665	540	385				0.823
36/4	1	790	675	500	395	345		0.554
$\alpha_1=\alpha_2=$ 1.333	2	880	770	600	480	435	330	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2055	1620	1105				0.274
36/14	1	2260	1795	1255	925	815		0.236
α_1=α_2= 4.000	2	2445	1965	1385	1040	915	675	0.207
$\alpha_p^2 = \alpha_e^2$= 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
	0	1025	810	550				0.549
36/7	1	1220	980	690	520	455		0.414
α_1=α_2= 2.000	2	1380	1135	815	630	560	410	0.333
$\alpha_p^2 = \alpha_e^2$= 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
	0	870	710	505				0.659
36/5	1	1010	845	625	485	430		0.474
α_1=α_2= 1.667	2	1115	960	730	580	520	390	0.370
$\alpha_p^2 = \alpha_e^2$= 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
	0	665	540	385				0.823
36/4	1	790	675	500	395	345		0.554
α_1=α_2= 1.333	2	880	770	600	480	435	330	0.417
$\alpha_p^2 = \alpha_e^2$= 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2055	1620	1105				0.274
36/14	1	2260	1795	1255	925	815		0.236
$\alpha_1=\alpha_2=$ 4.000	2	2445	1965	1385	1040	915	675	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
	0	1025	810	550				0.549
36/7	1	1220	980	690	520	455		0.414
$\alpha_1=\alpha_2=$ 2.000	2	1380	1135	815	630	560	410	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
	0	870	710	505				0.659
36/5	1	1010	845	625	485	430		0.474
$\alpha_1=\alpha_2=$ 1.667	2	1115	960	730	580	520	390	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
	0	665	540	385				0.823
36/4	1	790	675	500	395	345		0.554
$\alpha_1=\alpha_2=$ 1.333	2	880	770	600	480	435	330	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df},\ S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf},\ \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	770	605	415				0.785
	1	1010	825	590	455	405		0.552
$\alpha_1=\alpha_2=$ 4.000	2	1180	1000	745	575	510	380	0.426
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1300	1135	870	650	580	435	0.346
N= 4.000	4	1385	1240	975	730	650	485	0.292
A= 2	5	1445	1320	1080	810	720	540	0.252
	6	1490	1380	1170	885	785	590	0.222
36/7	0	385	300	205				1.569
	1	590	500	370	285	255		0.851
$\alpha_1=\alpha_2=$ 2.000	2	690	620	485	365	325	240	0.584
$\alpha_p^2=\alpha_e^2=$ 0.778	3	745	690	585	440	390	295	0.445
N= 2.000	4	770	730	645	520	460	345	0.359
A= 1	5	790	760	690	600	530	400	0.301
	6	800	775	720	655	600	450	0.259
36/5	0	325	265	185				1.883
	1	455	405	325	250	220		0.936
$\alpha_1=\alpha_2=$ 1.667	2	505	470	405	330	290	220	0.623
$\alpha_p^2=\alpha_e^2=$ 0.722	3	525	505	455	405	360	270	0.467
N= 1.333	4	535	520	485	445	425	325	0.373
A= 1	5	540	530	505	470	455	375	0.311
	6	545	535	515	490	475	425	0.266
36/4	0	245	200	145				2.354
	1	360	325	270	215	190		1.039
$\alpha_1=\alpha_2=$ 1.333	2	390	370	330	295	260	195	0.667
$\alpha_p^2=\alpha_e^2=$ 0.556	3	400	390	365	335	320	245	0.491
N= 1.000	4	405	400	380	360	350	300	0.388
A= 1	5	410	405	390	375	365	335	0.321
	6	410	405	395	385	375	355	0.274

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1040	815	560				0.710
	1	1290	1045	740	565	500		0.514
α_1=α_2= 4.000	2	1485	1240	905	705	630	465	0.403
α_p^2 = α_e^2= 1.556	3	1635	1400	1050	830	745	570	0.331
N= 4.000	4	1755	1530	1185	945	855	655	0.281
A= 2	5	1840	1640	1300	1055	960	730	0.244
	6	1910	1730	1405	1155	1055	800	0.216
36/7	0	520	405	280				1.419
	1	740	620	450	350	315		0.805
α_1=α_2= 2.000	2	875	765	590	470	425	325	0.562
α_p^2 = α_e^2= 0.778	3	955	865	700	575	525	400	0.432
N= 2.000	4	1005	930	785	665	610	470	0.350
A= 1	5	1035	975	850	735	685	540	0.295
	6	1055	1010	900	795	745	610	0.254
36/5	0	440	355	255				1.703
	1	585	515	400	320	290		0.889
α_1=α_2= 1.667	2	655	600	500	420	385	295	0.602
α_p^2 = α_e^2= 0.722	3	690	650	570	495	460	365	0.455
N= 1.333	4	710	680	615	550	520	435	0.365
A= 1	5	720	700	645	590	560	485	0.305
	6	730	710	670	620	595	525	0.262
36/4	0	335	270	195				2.129
	1	465	415	330	270	245		0.993
α_1=α_2= 1.333	2	510	480	415	355	330	265	0.647
α_p^2 = α_e^2= 0.556	3	530	510	460	410	390	330	0.480
N= 1.000	4	540	530	490	450	430	375	0.382
A= 1	5	550	540	510	480	460	410	0.317
	6	550	545	525	495	485	440	0.271

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1655	1305	890				0.558
	1	1920	1535	1080	810	715		0.429
$\alpha_1=\alpha_2=$ 4.000	2	2145	1750	1250	960	850	625	0.349
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2335	1940	1410	1095	985	725	0.294
N= 4.000	4	2500	2110	1565	1225	1100	830	0.254
A= 2	5	2640	2260	1705	1350	1215	930	0.223
	6	2755	2395	1840	1465	1325	1025	0.199
36/7	0	825	650	445				1.115
	1	1070	875	625	480	425		0.697
$\alpha_1=\alpha_2=$ 2.000	2	1250	1055	780	610	550	415	0.507
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1375	1195	920	730	660	510	0.399
N= 2.000	4	1470	1310	1035	840	765	600	0.328
A= 1	5	1535	1395	1140	940	860	680	0.279
	6	1585	1460	1225	1030	945	755	0.243
36/5	0	700	570	405				1.338
	1	870	745	560	440	395		0.778
$\alpha_1=\alpha_2=$ 1.667	2	975	865	685	555	505	395	0.549
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1035	950	785	650	595	475	0.424
N= 1.333	4	1080	1005	860	735	680	550	0.345
A= 1	5	1105	1045	920	800	750	615	0.291
	6	1125	1075	965	855	805	675	0.252
36/4	0	535	435	310				1.673
	1	685	595	460	365	330		0.881
$\alpha_1=\alpha_2=$ 1.333	2	765	695	570	470	430	340	0.598
$\alpha_p^2=\alpha_e^2=$ 0.556	3	810	755	645	550	510	415	0.452
N= 1.000	4	835	795	705	615	580	480	0.364
A= 1	5	850	820	745	670	630	535	0.304
	6	860	835	775	710	675	585	0.262

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2300	1810	1240				0.384
	1	2570	2045	1435	1065	940		0.318
α_1=α_2= 4.000	2	2810	2270	1605	1215	1075	790	0.272
α_p^2 = α_e^2= 1.556	3	3025	2475	1775	1370	1210	895	0.237
N= 4.000	4	3215	2665	1935	1500	1345	995	0.210
A= 2	5	3385	2840	2090	1630	1465	1095	0.189
	6	3540	3000	2240	1755	1580	1195	0.172
36/7	0	1150	905	620				0.768
	1	1405	1135	800	605	535		0.544
α_1=α_2= 2.000	2	1605	1330	965	750	670	495	0.421
α_p^2 = α_e^2= 0.778	3	1770	1500	1120	875	790	595	0.343
N= 2.000	4	1895	1640	1255	995	900	695	0.290
A= 1	5	1995	1760	1380	1110	1005	785	0.251
	6	2070	1860	1490	1215	1105	870	0.221
36/5	0	975	790	565				0.922
	1	1155	975	720	565	505		0.617
α_1=α_2= 1.667	2	1280	1115	860	685	620	470	0.463
α_p^2 = α_e^2= 0.722	3	1370	1225	975	795	720	565	0.371
N= 1.333	4	1430	1305	1075	890	815	645	0.309
A= 1	5	1480	1370	1155	975	900	720	0.265
	6	1510	1420	1225	1050	975	795	0.232
36/4	0	740	605	430				1.153
	1	910	775	585	460	415		0.712
α_1=α_2= 1.333	2	1010	900	710	575	520	405	0.515
α_p^2 = α_e^2= 0.556	3	1075	985	810	670	615	490	0.403
N= 1.000	4	1120	1045	890	755	700	565	0.331
A= 1	5	1150	1085	950	825	770	630	0.281
	6	1170	1115	1000	885	830	695	0.244

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2495	1965	1345				0.302
	1	2765	2205	1540	1145	1010		0.260
α_1=α_2= 4.000	2	3010	2425	1715	1295	1145	845	0.228
α_p^2 = α_e^2= 1.556	3	3230	2635	1885	1445	1280	945	0.203
N= 4.000	4	3430	2830	2045	1585	1415	1045	0.183
A= 2	5	3605	3010	2205	1715	1540	1145	0.167
	6	3765	3180	2355	1845	1660	1245	0.153
36/7	0	1245	980	670				0.605
	1	1505	1210	855	645	570		0.456
α_1=α_2= 2.000	2	1715	1415	1020	790	705	520	0.366
α_p^2 = α_e^2= 0.778	3	1880	1590	1175	920	830	620	0.306
N= 2.000	4	2015	1735	1315	1045	940	720	0.263
A= 1	5	2125	1860	1445	1160	1050	815	0.230
	6	2210	1970	1560	1265	1150	900	0.205
36/5	0	1055	860	610				0.726
	1	1240	1045	770	600	540		0.522
α_1=α_2= 1.667	2	1370	1190	910	725	655	495	0.408
α_p^2 = α_e^2= 0.722	3	1465	1305	1030	835	760	590	0.334
N= 1.333	4	1535	1390	1135	935	855	675	0.283
A= 1	5	1585	1460	1220	1025	940	750	0.246
	6	1625	1515	1295	1105	1020	825	0.217
36/4	0	805	655	470				0.907
	1	975	830	625	490	435		0.610
α_1=α_2= 1.333	2	1085	960	750	605	550	420	0.459
α_p^2 = α_e^2= 0.556	3	1155	1050	855	705	645	510	0.368
N= 1.000	4	1200	1115	940	795	730	585	0.307
A= 1	5	1235	1160	1005	865	805	655	0.264
	6	1260	1195	1060	930	870	720	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2495	1965	1345				0.302
	1	2765	2205	1540	1145	1010		0.260
α_1=α_2= 4.000	2	3010	2425	1715	1295	1145	845	0.228
α_p^2 = α_e^2= 1.556	3	3230	2635	1885	1445	1280	945	0.203
N= 4.000	4	3430	2830	2045	1585	1415	1045	0.183
A= 2	5	3605	3010	2205	1715	1540	1145	0.167
	6	3765	3180	2355	1845	1660	1245	0.153
36/7	0	1245	980	670				0.605
	1	1505	1210	855	645	570		0.456
α_1=α_2= 2.000	2	1715	1415	1020	790	705	520	0.366
α_p^2 = α_e^2= 0.778	3	1880	1590	1175	920	830	620	0.306
N= 2.000	4	2015	1735	1315	1045	940	720	0.263
A= 1	5	2125	1860	1445	1160	1050	815	0.230
	6	2210	1970	1560	1265	1150	900	0.205
36/5	0	1055	860	610				0.726
	1	1240	1045	770	600	540		0.522
α_1=α_2= 1.667	2	1370	1190	910	725	655	495	0.408
α_p^2 = α_e^2= 0.722	3	1465	1305	1030	835	760	590	0.334
N= 1.333	4	1535	1390	1135	935	855	675	0.283
A= 1	5	1585	1460	1220	1025	940	750	0.246
	6	1625	1515	1295	1105	1020	825	0.217
36/4	0	805	655	470				0.907
	1	975	830	625	490	435		0.610
α_1=α_2= 1.333	2	1085	960	750	605	550	420	0.459
α_p^2 = α_e^2= 0.556	3	1155	1050	855	705	645	510	0.368
N= 1.000	4	1200	1115	940	795	730	585	0.307
A= 1	5	1235	1160	1005	865	805	655	0.264
	6	1260	1195	1060	930	870	720	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	770	605	415				1.039
36/14	1	1105	925	660	495	440		0.700
α_1=α_2= 4.000	2	1305	1145	765	575	510	380	0.527
$\alpha_p^2 = \alpha_e^2$= 1.556	3	1420	1290	870	650	580	435	0.423
N= 4.000	4	1490	1385	975	730	650	485	0.353
A= 2	5	1535	1455	1080	810	720	540	0.303
	6	1570	1500	1180	885	785	590	0.266
	0	385	300	205				2.078
36/7	1	650	570	380	285	255		1.054
α_1=α_2= 2.000	2	745	690	485	365	325	240	0.706
$\alpha_p^2 = \alpha_e^2$= 0.778	3	785	750	590	440	390	295	0.531
N= 2.000	4	800	780	695	520	460	345	0.426
A= 1	5	810	795	750	600	530	400	0.355
	6	815	805	770	675	600	450	0.305
	0	325	265	185				2.494
36/5	1	485	450	335	250	220		1.152
α_1=α_2= 1.667	2	525	505	440	330	290	220	0.749
$\alpha_p^2 = \alpha_e^2$= 0.722	3	540	525	495	405	360	270	0.555
N= 1.333	4	545	535	515	485	430	325	0.441
A= 1	5	550	545	530	510	500	375	0.365
	6	550	545	535	520	515	425	0.312
	0	245	200	145				3.117
36/4	1	380	355	290	215	190		1.269
α_1=α_2= 1.333	2	400	390	365	295	260	195	0.797
$\alpha_p^2 = \alpha_e^2$= 0.556	3	410	400	385	370	330	245	0.581
N= 1.000	4	410	405	400	385	380	300	0.457
A= 1	5	415	410	405	395	390	350	0.376
	6	415	410	405	400	395	385	0.320

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1040	815	560				0.940
	1	1400	1150	830	640	570		0.653
α_1=α_2= 4.000	2	1645	1405	1035	775	690	515	0.500
α_p^2 = α_e^2= 1.556	3	1805	1595	1175	880	785	585	0.406
N= 4.000	4	1915	1735	1315	985	875	655	0.341
A= 2	5	1995	1840	1460	1095	970	730	0.294
	6	2045	1920	1600	1200	1065	800	0.259
36/7	0	520	405	280				1.879
	1	820	700	515	385	345		1.001
α_1=α_2= 2.000	2	955	865	655	490	435	325	0.682
α_p^2 = α_e^2= 0.778	3	1020	960	800	600	530	400	0.517
N= 2.000	4	1055	1010	905	705	625	470	0.417
A= 1	5	1080	1045	960	810	720	540	0.349
	6	1090	1065	995	915	815	610	0.300
36/5	0	440	355	255				2.255
	1	630	565	455	340	300		1.098
α_1=α_2= 1.667	2	690	655	570	445	395	295	0.726
α_p^2 = α_e^2= 0.722	3	715	690	635	550	490	365	0.542
N= 1.333	4	730	710	670	625	585	435	0.433
A= 1	5	735	725	695	655	635	510	0.360
	6	740	730	705	680	660	580	0.308
36/4	0	335	270	195				2.819
	1	495	455	380	290	260		1.217
α_1=α_2= 1.333	2	530	510	465	400	355	265	0.776
α_p^2 = α_e^2= 0.556	3	545	535	505	470	450	335	0.569
N= 1.000	4	550	545	525	500	485	405	0.450
A= 1	5	555	550	535	515	505	470	0.372
	6	560	555	545	530	520	495	0.317

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1545	1215	840				0.823
36/14	1	1930	1565	1110	850	750		0.594
$\alpha_1=\alpha_2=$ 4.000	2	2225	1855	1355	1055	950	710	0.465
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	2450	2100	1580	1245	1125	860	0.382
N= 4.000	4	2625	2300	1780	1425	1290	980	0.324
A= 2	5	2755	2460	1955	1590	1445	1085	0.282
	6	2860	2590	2110	1740	1585	1190	0.249
	0	770	605	420				1.646
36/7	1	1110	925	675	525	475		0.931
$\alpha_1=\alpha_2=$ 2.000	2	1310	1150	890	710	645	490	0.649
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1430	1295	1055	870	790	595	0.498
N= 2.000	4	1500	1395	1180	1000	925	700	0.404
A= 1	5	1545	1460	1275	1110	1035	805	0.340
	6	1575	1505	1350	1195	1125	910	0.293
	0	655	530	380				1.975
36/5	1	880	770	600	480	435		1.027
$\alpha_1=\alpha_2=$ 1.667	2	980	900	750	630	580	440	0.694
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	1030	975	855	740	690	545	0.524
N= 1.333	4	1060	1015	920	825	780	650	0.421
A= 1	5	1075	1045	970	885	845	730	0.352
	6	1085	1060	1000	930	895	790	0.302
	0	500	405	290				2.469
36/4	1	695	620	495	405	370		1.147
$\alpha_1=\alpha_2=$ 1.333	2	765	715	620	535	495	395	0.747
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	795	765	690	620	585	495	0.554
N= 1.000	4	810	790	735	675	650	565	0.440
A= 1	5	820	800	765	715	690	620	0.365
	6	825	810	780	745	725	660	0.312

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2250	1770	1220				0.650
	1	2645	2125	1495	1130	1000		0.499
α_1=α_2= 4.000	2	2980	2440	1755	1355	1205	890	0.404
α_p^2 = α_e^2= 1.556	3	3260	2720	1995	1555	1395	1045	0.340
N= 4.000	4	3490	2970	2220	1745	1570	1200	0.294
A= 2	5	3680	3185	2430	1930	1745	1345	0.258
	6	3840	3370	2620	2100	1905	1480	0.230
36/7	0	1125	885	610				1.300
	1	1490	1220	875	675	600		0.809
α_1=α_2= 2.000	2	1745	1485	1110	870	785	600	0.587
α_p^2 = α_e^2= 0.778	3	1920	1685	1310	1050	950	740	0.461
N= 2.000	4	2040	1835	1480	1210	1105	865	0.379
A= 1	5	2125	1950	1620	1350	1240	990	0.322
	6	2190	2040	1735	1475	1365	1100	0.280
36/5	0	950	775	550				1.560
	1	1205	1035	780	620	560		0.902
α_1=α_2= 1.667	2	1345	1205	965	785	715	560	0.635
α_p^2 = α_e^2= 0.722	3	1430	1320	1105	925	855	680	0.490
N= 1.333	4	1485	1395	1210	1045	970	790	0.399
A= 1	5	1520	1450	1290	1135	1065	885	0.336
	6	1540	1485	1350	1210	1145	970	0.290
36/4	0	725	590	420				1.950
	1	950	830	645	515	465		1.021
α_1=α_2= 1.333	2	1060	970	800	665	610	485	0.691
α_p^2 = α_e^2= 0.556	3	1115	1050	910	785	730	595	0.522
N= 1.000	4	1145	1095	985	875	825	690	0.420
A= 1	5	1165	1130	1040	945	895	770	0.351
	6	1180	1150	1075	995	950	835	0.302

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
		0	3155	2485	1710				0.406
36/14		1	3560	2845	1990	1495	1320		0.341
α_1=α_2=	4.000	2	3920	3175	2255	1725	1525	1130	0.294
α_p^2 = α_e^2=	1.556	3	4240	3485	2510	1940	1730	1280	0.259
N=	4.000	4	4520	3765	2750	2140	1920	1435	0.231
A=	2	5	4760	4020	2980	2335	2100	1590	0.208
		6	4975	4255	3200	2520	2275	1740	0.190
		0	1575	1240	855				0.812
36/7		1	1960	1585	1125	860	760		0.589
α_1=α_2=	2.000	2	2260	1880	1375	1070	960	715	0.462
α_p^2 = α_e^2=	0.778	3	2485	2125	1600	1260	1135	870	0.380
N=	2.000	4	2660	2325	1800	1440	1305	1010	0.323
A=	1	5	2795	2490	1975	1605	1460	1140	0.280
		6	2900	2625	2135	1755	1605	1265	0.248
		0	1335	1085	775				0.974
36/5		1	1605	1360	1010	795	715		0.670
α_1=α_2=	1.667	2	1785	1565	1215	975	880	685	0.510
α_p^2 = α_e^2=	0.722	3	1910	1720	1385	1135	1035	815	0.412
N=	1.333	4	1995	1835	1525	1275	1170	935	0.345
A=	1	5	2055	1920	1640	1395	1295	1045	0.297
		6	2100	1980	1735	1505	1400	1150	0.261
		0	1020	830	595				1.218
36/4		1	1265	1090	825	650	585		0.776
α_1=α_2=	1.333	2	1410	1260	1005	820	745	585	0.570
α_p^2 = α_e^2=	0.556	3	1500	1380	1150	960	885	705	0.450
N=	1.000	4	1555	1460	1260	1080	1005	815	0.372
A=	1	5	1590	1515	1345	1180	1105	915	0.317
		6	1615	1555	1405	1255	1185	1000	0.276

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	3305	2605	1790				0.348
		1	3715	2965	2075	1555	1375		0.299
$\alpha_1=\alpha_2$=	4.000	2	4075	3300	2340	1785	1580	1165	0.263
$\alpha_p^2=\alpha_e^2$=	1.556	3	4400	3610	2595	2005	1785	1320	0.234
N=	4.000	4	4685	3895	2835	2205	1980	1475	0.211
A=	2	5	4935	4155	3070	2400	2160	1630	0.192
		6	5155	4395	3290	2590	2335	1780	0.176
36/7		0	1650	1300	895				0.696
		1	2035	1650	1170	890	790		0.525
$\alpha_1=\alpha_2$=	2.000	2	2340	1945	1415	1100	990	735	0.422
$\alpha_p^2=\alpha_e^2$=	0.778	3	2575	2195	1645	1295	1165	890	0.352
N=	2.000	4	2760	2405	1850	1475	1335	1030	0.303
A=	1	5	2900	2575	2030	1640	1495	1165	0.265
		6	3015	2715	2195	1800	1640	1290	0.236
36/5		0	1400	1140	810				0.835
		1	1675	1415	1050	825	740		0.601
$\alpha_1=\alpha_2$=	1.667	2	1860	1625	1255	1005	910	705	0.469
$\alpha_p^2=\alpha_e^2$=	0.722	3	1985	1785	1430	1165	1065	835	0.385
N=	1.333	4	2075	1900	1575	1310	1205	955	0.326
A=	1	5	2140	1990	1695	1435	1330	1070	0.283
		6	2190	2060	1790	1545	1440	1175	0.250
36/4		0	1065	870	620				1.044
		1	1315	1130	855	675	605		0.702
$\alpha_1=\alpha_2$=	1.333	2	1470	1310	1040	845	770	600	0.528
$\alpha_p^2=\alpha_e^2$=	0.556	3	1560	1430	1185	990	910	725	0.424
N=	1.000	4	1620	1515	1300	1110	1030	835	0.354
A=	1	5	1660	1575	1390	1215	1135	935	0.304
		6	1690	1620	1460	1295	1220	1025	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	770	605	415				1.311
36/14	1	1205	990	660	495	440		0.848
α_1=α_2= 4.000	2	1405	1150	765	575	510	380	0.627
α_p^2 = α_e^2= 1.556	3	1505	1305	870	650	580	435	0.497
N= 4.000	4	1560	1460	975	730	650	485	0.412
A= 2	5	1590	1535	1080	810	720	540	0.352
	6	1610	1570	1180	885	785	590	0.307
	0	385	300	205				2.622
36/7	1	700	575	380	285	255		1.254
α_1=α_2= 2.000	2	780	730	485	365	325	240	0.824
α_p^2 = α_e^2= 0.778	3	805	785	590	440	390	295	0.614
N= 2.000	4	815	800	695	520	460	345	0.489
A= 1	5	820	810	785	600	530	400	0.406
	6	825	820	800	675	600	450	0.348
	0	325	265	185				3.146
36/5	1	510	480	335	250	220		1.363
α_1=α_2= 1.667	2	535	525	440	330	290	220	0.870
α_p^2 = α_e^2= 0.722	3	545	540	520	405	360	270	0.639
N= 1.333	4	550	545	535	485	430	325	0.505
A= 1	5	550	550	540	530	500	375	0.417
	6	550	550	545	535	530	425	0.355
	0	245	200	145				3.933
36/4	1	390	375	290	215	190		1.492
α_1=α_2= 1.333	2	405	400	385	295	260	195	0.921
α_p^2 = α_e^2= 0.556	3	410	410	400	370	330	245	0.666
N= 1.000	4	415	410	405	400	395	300	0.521
A= 1	5	415	415	410	405	400	350	0.429
	6	415	415	410	405	405	400	0.364

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1040	815	565				1.185
36/14	1	1515	1265	895	670	595		0.794
$\alpha_1=\alpha_2=$ 4.000	2	1785	1550	1035	775	690	515	0.597
$\alpha_p^2 = \alpha_e^2=$ 1.556	3	1940	1765	1175	880	785	585	0.478
N= 4.000	4	2030	1895	1315	985	875	655	0.399
A= 2	5	2090	1980	1460	1095	970	730	0.342
	6	2130	2045	1600	1200	1065	800	0.300
	0	520	405	280				2.371
36/7	1	890	775	515	385	345		1.194
$\alpha_1=\alpha_2=$ 2.000	2	1015	945	655	490	435	325	0.798
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1065	1020	800	600	530	400	0.599
N= 2.000	4	1085	1060	940	705	625	470	0.480
A= 1	5	1100	1080	1025	810	720	540	0.400
	6	1105	1090	1050	915	815	610	0.343
	0	440	355	255				2.845
36/5	1	665	610	455	340	300		1.303
$\alpha_1=\alpha_2=$ 1.667	2	715	685	595	445	395	295	0.845
$\alpha_p^2 = \alpha_e^2=$ 0.722	3	730	715	680	550	490	365	0.625
N= 1.333	4	740	730	705	655	585	435	0.496
A= 1	5	740	735	720	695	680	510	0.411
	6	745	740	725	710	700	580	0.351
	0	335	270	195				3.557
36/4	1	515	485	390	290	260		1.435
$\alpha_1=\alpha_2=$ 1.333	2	545	530	500	400	355	265	0.899
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	555	545	525	505	450	335	0.654
N= 1.000	4	555	555	540	525	515	405	0.514
A= 1	5	560	555	550	535	530	475	0.424
	6	560	560	550	545	540	520	0.360

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1545	1215	840				1.038
36/14	1	2060	1690	1215	940	840		0.725
α_1=α_2= 4.000	2	2420	2060	1540	1155	1025	770	0.557
α_p^2 = α_e^2= 1.556	3	2660	2340	1750	1310	1165	875	0.452
N= 4.000	4	2825	2550	1960	1470	1305	980	0.381
A= 2	5	2945	2710	2170	1630	1445	1085	0.329
	6	3030	2825	2380	1785	1585	1190	0.289
	0	770	605	420				2.077
36/7	1	1210	1030	770	575	510		1.114
α_1=α_2= 2.000	2	1410	1275	980	735	650	490	0.761
α_p^2 = α_e^2= 0.778	3	1515	1410	1190	890	790	595	0.578
N= 2.000	4	1565	1495	1330	1050	930	700	0.466
A= 1	5	1600	1545	1410	1205	1075	805	0.390
	6	1620	1575	1470	1350	1215	910	0.336
	0	655	530	380				2.492
36/5	1	930	835	670	505	450		1.224
α_1=α_2= 1.667	2	1025	965	840	665	590	440	0.811
α_p^2 = α_e^2= 0.722	3	1065	1025	935	820	730	545	0.606
N= 1.333	4	1085	1055	990	920	870	650	0.484
A= 1	5	1095	1075	1025	970	935	755	0.403
	6	1100	1085	1050	1000	975	860	0.345
	0	500	405	290				3.115
36/4	1	730	670	555	435	385		1.357
α_1=α_2= 1.333	2	790	760	685	595	525	395	0.868
α_p^2 = α_e^2= 0.556	3	810	790	745	690	660	500	0.638
N= 1.000	4	820	810	775	735	715	605	0.504
A= 1	5	825	820	795	765	750	695	0.417
	6	830	825	805	785	770	725	0.355

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2175	1710	1185				0.927
	1	2715	2205	1565	1205	1065		0.669
$\alpha_1=\alpha_2=$ 4.000	2	3135	2615	1915	1490	1340	1005	0.523
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3455	2960	2230	1760	1585	1220	0.430
N= 4.000	4	3695	3240	2510	2010	1820	1375	0.365
A= 2	5	3880	3465	2760	2245	2035	1525	0.317
	6	4025	3650	2980	2455	2230	1675	0.280
36/7	0	1085	855	590				1.854
	1	1565	1305	955	745	670		1.047
$\alpha_1=\alpha_2=$ 2.000	2	1845	1620	1255	1005	910	685	0.729
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2010	1825	1490	1225	1115	835	0.560
N= 2.000	4	2110	1965	1665	1410	1305	985	0.454
A= 1	5	2175	2055	1800	1565	1460	1130	0.382
	6	2220	2120	1905	1690	1590	1280	0.330
36/5	0	920	750	535				2.224
	1	1240	1085	845	675	615		1.156
$\alpha_1=\alpha_2=$ 1.667	2	1380	1270	1060	890	815	620	0.780
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1450	1375	1205	1045	975	770	0.589
N= 1.333	4	1490	1435	1300	1165	1100	915	0.473
A= 1	5	1515	1470	1365	1250	1190	1035	0.395
	6	1530	1495	1410	1315	1265	1115	0.340
36/4	0	700	570	410				2.781
	1	975	875	700	570	520		1.290
$\alpha_1=\alpha_2=$ 1.333	2	1075	1010	875	755	700	555	0.839
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1120	1075	975	875	825	700	0.622
N= 1.000	4	1140	1110	1035	955	915	800	0.494
A= 1	5	1150	1130	1075	1010	975	875	0.410
	6	1160	1145	1100	1050	1020	930	0.350

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	3125	2460	1700				0.688
	1	3685	2960	2085	1585	1405		0.535
$\alpha_1=\alpha_2=$ 4.000	2	4155	3410	2450	1890	1695	1255	0.438
$\alpha_p^2=\alpha_e^2=$ 1.556	3	4550	3805	2790	2175	1955	1475	0.370
N= 4.000	4	4870	4150	3110	2445	2205	1690	0.321
A= 2	5	5140	4455	3405	2705	2445	1890	0.283
	6	5360	4715	3675	2950	2675	2080	0.253
36/7	0	1560	1230	850				1.377
	1	2075	1705	1225	945	845		0.875
$\alpha_1=\alpha_2=$ 2.000	2	2435	2075	1555	1220	1100	845	0.642
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2680	2355	1835	1475	1335	1040	0.507
N= 2.000	4	2845	2565	2070	1700	1550	1220	0.418
A= 1	5	2965	2725	2270	1895	1745	1390	0.356
	6	3050	2845	2430	2070	1915	1550	0.310
36/5	0	1325	1075	765				1.652
	1	1675	1445	1095	865	780		0.979
$\alpha_1=\alpha_2=$ 1.667	2	1880	1685	1350	1100	1005	790	0.696
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1995	1840	1545	1300	1195	960	0.540
N= 1.333	4	2070	1945	1695	1460	1360	1110	0.441
A= 1	5	2115	2020	1805	1590	1495	1245	0.372
	6	2145	2070	1885	1695	1605	1365	0.323
36/4	0	1010	820	585				2.065
	1	1325	1160	900	720	650		1.111
$\alpha_1=\alpha_2=$ 1.333	2	1475	1350	1120	935	860	685	0.760
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1550	1465	1275	1100	1025	840	0.577
N= 1.000	4	1595	1530	1380	1225	1155	970	0.466
A= 1	5	1620	1570	1450	1320	1255	1080	0.390
	6	1640	1600	1500	1390	1330	1170	0.336

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	4170	3285	2270				0.391
36/14	1	4745	3795	2660	2010	1780		0.336
α_1=α_2= 4.000	2	5250	4260	3035	2330	2070	1530	0.295
$\alpha_p^2 = \alpha_e^2$= 1.556	3	5690	4690	3390	2620	2350	1750	0.263
N= 4.000	4	6070	5080	3725	2905	2610	1965	0.237
A= 2	5	6405	5430	4045	3175	2860	2185	0.216
	6	6690	5750	4350	3440	3105	2390	0.198
	0	2085	1640	1135				0.782
36/7	1	2625	2130	1515	1165	1035		0.590
α_1=α_2= 2.000	2	3035	2540	1860	1450	1305	980	0.474
$\alpha_p^2 = \alpha_e^2$= 0.778	3	3345	2875	2175	1720	1550	1195	0.396
N= 2.000	4	3575	3145	2450	1965	1785	1385	0.340
A= 1	5	3755	3365	2690	2195	2000	1570	0.298
	6	3885	3540	2905	2405	2205	1745	0.265
	0	1765	1435	1025				0.938
36/5	1	2145	1825	1360	1070	960		0.675
α_1=α_2= 1.667	2	2390	2105	1645	1320	1200	930	0.527
$\alpha_p^2 = \alpha_e^2$= 0.722	3	2555	2310	1875	1540	1410	1110	0.432
N= 1.333	4	2665	2460	2065	1735	1600	1280	0.366
A= 1	5	2740	2570	2215	1900	1765	1435	0.318
	6	2795	2650	2340	2040	1910	1575	0.281
	0	1345	1100	785				1.172
36/4	1	1690	1460	1110	880	795		0.788
α_1=α_2= 1.333	2	1885	1695	1365	1115	1015	795	0.594
$\alpha_p^2 = \alpha_e^2$= 0.556	3	2000	1850	1555	1310	1205	965	0.476
N= 1.000	4	2075	1955	1700	1470	1370	1120	0.397
A= 1	5	2120	2025	1810	1600	1500	1255	0.341
	6	2150	2075	1890	1705	1610	1370	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df} S_{nf}$, $\phi_{db} S_{nb}$}

Deck F_y:	50	ksi
Deck F_{u1}:	65	ksi
Framing F_{y2}:	50	ksi
Framing F_{u2}:	65	ksi

P_{nf}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	604	851	1118	1118	1118	1118
20	604	817	1240	1357	1357	1357
18	604	817	1215	1720	1797	1797
16	604	817	1215	1708	2267	2267

P_{ns}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	603	603	603	603	603	603
20	806	806	806	806	806	806
18	1228	1228	1228	1228	1228	1228
16	1740	1740	1740	1740	1740	1740

S_f, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0178	0.0136	0.0076	0.0076	0.0076	0.0076
20	0.0178	0.0161	0.0093	0.0069	0.0069	0.0069
18	0.0178	0.0161	0.0141	0.0073	0.0060	0.0060
16	0.0178	0.0161	0.0141	0.0126	0.0053	0.0053

S_s, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22	0.0175	0.0175	0.0175	0.0175	0.0175	0.0175
20	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
18	0.0138	0.0138	0.0138	0.0138	0.0138	0.0138
16	0.0123	0.0123	0.0123	0.0123	0.0123	0.0123

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1110	875	595				0.647
	1	1305	1045	735	550	485		0.468
$\alpha_1=\alpha_2=$ 4.000	2	1470	1205	865	665	585	430	0.366
$\alpha_p^2=\alpha_e^2=$ 1.556	3	1605	1340	980	765	685	510	0.301
N= 4.000	4	1720	1465	1095	860	775	585	0.255
A= 2	5	1815	1570	1195	950	855	660	0.222
	6	1895	1660	1290	1035	940	725	0.196
36/7	0	555	435	295				1.293
	1	735	600	430	330	290		0.732
$\alpha_1=\alpha_2=$ 2.000	2	860	730	545	430	385	290	0.511
$\alpha_p^2=\alpha_e^2=$ 0.778	3	945	830	645	515	470	360	0.392
N= 2.000	4	1005	905	725	595	545	425	0.318
A= 1	5	1050	960	795	665	610	485	0.268
	6	1080	1005	855	725	670	540	0.231
36/5	0	470	380	270				1.552
	1	595	510	385	305	275		0.809
$\alpha_1=\alpha_2=$ 1.667	2	665	595	475	385	350	275	0.547
$\alpha_p^2=\alpha_e^2=$ 0.722	3	705	650	545	455	420	335	0.413
N= 1.333	4	730	690	595	515	475	390	0.332
A= 1	5	750	715	635	560	525	435	0.277
	6	760	730	665	595	565	475	0.238
36/4	0	355	290	205				1.940
	1	470	410	315	250	230		0.903
$\alpha_1=\alpha_2=$ 1.333	2	520	475	395	325	300	240	0.588
$\alpha_p^2=\alpha_e^2=$ 0.556	3	550	515	450	385	360	290	0.436
N= 1.000	4	565	540	485	430	405	340	0.347
A= 1	5	575	555	510	465	440	375	0.288
	6	580	565	530	490	470	410	0.246

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1565	1230	840				0.494
	1	1765	1410	985	730	645		0.382
$\alpha_1=\alpha_2=$ 4.000	2	1940	1575	1115	845	745	550	0.312
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2100	1725	1240	960	845	625	0.263
N= 4.000	4	2235	1860	1360	1055	945	700	0.228
A= 2	5	2355	1990	1475	1150	1035	775	0.201
	6	2465	2105	1580	1245	1120	850	0.179
36/7	0	780	615	420				0.989
	1	970	785	555	420	370		0.624
$\alpha_1=\alpha_2=$ 2.000	2	1115	930	680	525	470	350	0.455
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1230	1050	790	620	560	425	0.359
N= 2.000	4	1315	1150	890	710	640	495	0.296
A= 1	5	1385	1230	975	790	720	560	0.252
	6	1435	1300	1055	865	790	625	0.219
36/5	0	660	540	385				1.186
	1	795	675	500	390	350		0.697
$\alpha_1=\alpha_2=$ 1.667	2	885	775	600	480	435	335	0.493
$\alpha_p^2=\alpha_e^2=$ 0.722	3	945	850	685	560	510	400	0.382
N= 1.333	4	990	905	755	630	580	460	0.311
A= 1	5	1020	950	810	690	640	515	0.263
	6	1040	980	855	740	690	565	0.228
36/4	0	505	410	290				1.483
	1	625	540	405	320	290		0.790
$\alpha_1=\alpha_2=$ 1.333	2	700	625	495	405	370	285	0.538
$\alpha_p^2=\alpha_e^2=$ 0.556	3	740	680	570	475	435	345	0.408
N= 1.000	4	770	720	620	535	495	400	0.329
A= 1	5	790	750	665	580	545	450	0.275
	6	800	770	695	620	585	495	0.237

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2055	1620	1105				0.274
36/14	1	2260	1795	1255	925	815		0.236
$\alpha_1=\alpha_2=$ 4.000	2	2445	1965	1385	1040	915	675	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
	0	1025	810	550				0.549
36/7	1	1220	980	690	520	455		0.414
$\alpha_1=\alpha_2=$ 2.000	2	1380	1135	815	630	560	410	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
	0	870	710	505				0.659
36/5	1	1010	845	625	485	430		0.474
$\alpha_1=\alpha_2=$ 1.667	2	1115	960	730	580	520	390	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
	0	665	540	385				0.823
36/4	1	790	675	500	395	345		0.554
$\alpha_1=\alpha_2=$ 1.333	2	880	770	600	480	435	330	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2055	1620	1105				0.274
36/14	1	2260	1795	1255	925	815		0.236
$\alpha_1=\alpha_2=$ 4.000	2	2445	1965	1385	1040	915	675	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
	0	1025	810	550				0.549
36/7	1	1220	980	690	520	455		0.414
$\alpha_1=\alpha_2=$ 2.000	2	1380	1135	815	630	560	410	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
	0	870	710	505				0.659
36/5	1	1010	845	625	485	430		0.474
$\alpha_1=\alpha_2=$ 1.667	2	1115	960	730	580	520	390	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
	0	665	540	385				0.823
36/4	1	790	675	500	395	345		0.554
$\alpha_1=\alpha_2=$ 1.333	2	880	770	600	480	435	330	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2055	1620	1105				0.274
	1	2260	1795	1255	925	815		0.236
$\alpha_1=\alpha_2=$ 4.000	2	2445	1965	1385	1040	915	675	0.207
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
36/7	0	1025	810	550				0.549
	1	1220	980	690	520	455		0.414
$\alpha_1=\alpha_2=$ 2.000	2	1380	1135	815	630	560	410	0.333
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
36/5	0	870	710	505				0.659
	1	1010	845	625	485	430		0.474
$\alpha_1=\alpha_2=$ 1.667	2	1115	960	730	580	520	390	0.370
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
36/4	0	665	540	385				0.823
	1	790	675	500	395	345		0.554
$\alpha_1=\alpha_2=$ 1.333	2	880	770	600	480	435	330	0.417
$\alpha_p^2=\alpha_e^2=$ 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR22		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2055	1620	1105				0.274
36/14	1	2260	1795	1255	925	815		0.236
α_1=α_2= 4.000	2	2445	1965	1385	1040	915	675	0.207
α_p^2 = α_e^2= 1.556	3	2615	2125	1515	1150	1020	750	0.185
N= 4.000	4	2765	2275	1635	1265	1120	825	0.166
A= 2	5	2905	2415	1755	1365	1220	900	0.151
	6	3030	2545	1870	1460	1315	975	0.139
	0	1025	810	550				0.549
36/7	1	1220	980	690	520	455		0.414
α_1=α_2= 2.000	2	1380	1135	815	630	560	410	0.333
α_p^2 = α_e^2= 0.778	3	1515	1270	935	730	655	485	0.278
N= 2.000	4	1625	1385	1045	820	740	560	0.239
A= 1	5	1710	1490	1145	910	825	635	0.209
	6	1785	1575	1235	995	900	700	0.186
	0	870	710	505				0.659
36/5	1	1010	845	625	485	430		0.474
α_1=α_2= 1.667	2	1115	960	730	580	520	390	0.370
α_p^2 = α_e^2= 0.722	3	1190	1050	825	660	600	465	0.304
N= 1.333	4	1245	1120	905	740	675	530	0.257
A= 1	5	1290	1180	975	810	745	590	0.223
	6	1320	1225	1035	875	805	645	0.197
	0	665	540	385				0.823
36/4	1	790	675	500	395	345		0.554
α_1=α_2= 1.333	2	880	770	600	480	435	330	0.417
α_p^2 = α_e^2= 0.556	3	935	845	680	560	510	400	0.334
N= 1.000	4	975	900	750	630	575	460	0.279
A= 1	5	1005	940	805	685	635	515	0.240
	6	1025	970	850	740	690	565	0.210

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.173	34328	19310	8582	4827	3814	2146

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20			
Design thickness = 0.0358 in.			
$F_{y\text{-}deck}$=	50	ksi	
$F_{u\text{-}deck}$=	65	ksi	
Framing designation = 27 mils			
Framing thickness = 0.0283 in.			
$F_{y\text{-}framing}$=	50	ksi	
$F_{u\text{-}framing}$=	65	ksi	
Support fastening: #12 screws			
Side-lap fastening: #10 screws			

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
		0	1110	875	595				0.785
36/14		1	1365	1100	780	590	525		0.552
α_1=α_2=	4.000	2	1565	1300	945	735	655	485	0.426
α_p^2 = α_e^2=	1.556	3	1720	1465	1095	860	775	585	0.346
N=	4.000	4	1845	1605	1230	980	885	685	0.292
A=	2	5	1940	1720	1350	1090	990	770	0.252
		6	2015	1815	1460	1195	1090	855	0.222
		0	555	435	295				1.569
36/7		1	780	650	470	365	325		0.851
α_1=α_2=	2.000	2	920	800	615	490	440	340	0.584
α_p^2 = α_e^2=	0.778	3	1005	905	730	595	545	425	0.445
N=	2.000	4	1060	980	820	685	630	500	0.359
A=	1	5	1095	1030	890	765	710	575	0.301
		6	1120	1065	945	825	775	640	0.259
		0	470	380	270				1.883
36/5		1	620	540	420	335	300		0.936
α_1=α_2=	1.667	2	695	635	525	435	400	315	0.623
α_p^2 = α_e^2=	0.722	3	730	690	595	515	475	390	0.467
N=	1.333	4	755	720	645	575	540	450	0.373
A=	1	5	765	740	680	620	585	505	0.311
		6	775	755	705	650	625	545	0.266
		0	355	290	205				2.354
36/4		1	490	435	345	280	255		1.039
α_1=α_2=	1.333	2	540	505	430	370	340	275	0.667
α_p^2 = α_e^2=	0.556	3	565	540	485	430	405	340	0.491
N=	1.000	4	575	560	520	475	450	390	0.388
A=	1	5	585	570	540	505	485	430	0.321
		6	590	580	555	525	510	460	0.274

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	1500	1180	810				0.710
		1	1760	1415	995	750	660		0.514
α_1=α_2=	4.000	2	1985	1625	1165	900	795	585	0.403
α_p^2 = α_e^2=	1.556	3	2165	1810	1325	1030	925	685	0.331
N=	4.000	4	2320	1970	1470	1155	1040	790	0.281
A=	2	5	2450	2115	1610	1275	1155	890	0.244
		6	2555	2240	1735	1390	1260	980	0.216
36/7		0	750	590	405				1.419
		1	990	810	580	450	395		0.805
α_1=α_2=	2.000	2	1160	985	735	575	520	395	0.562
α_p^2 = α_e^2=	0.778	3	1275	1120	865	695	630	490	0.432
N=	2.000	4	1360	1220	980	800	730	575	0.350
A=	1	5	1415	1295	1075	895	820	655	0.295
		6	1460	1355	1150	975	905	730	0.254
36/5		0	635	515	365				1.703
		1	800	690	520	410	370		0.889
α_1=α_2=	1.667	2	895	800	640	520	475	370	0.602
α_p^2 = α_e^2=	0.722	3	955	880	735	615	565	450	0.455
N=	1.333	4	990	930	805	690	640	525	0.365
A=	1	5	1010	965	855	755	705	585	0.305
		6	1030	990	900	805	760	645	0.262
36/4		0	485	395	280				2.129
		1	630	550	425	340	310		0.993
α_1=α_2=	1.333	2	705	645	530	440	405	320	0.647
α_p^2 = α_e^2=	0.556	3	740	700	605	520	485	395	0.480
N=	1.000	4	765	730	655	580	545	455	0.382
A=	1	5	775	750	690	625	595	510	0.317
		6	785	765	715	660	630	550	0.271

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2280	1795	1230				0.409
	1	2550	2030	1420	1060	935		0.335
α_1=α_2= 4.000	2	2790	2255	1595	1210	1070	785	0.284
α_p^2 = α_e^2= 1.556	3	3005	2460	1765	1360	1205	890	0.247
N= 4.000	4	3195	2650	1925	1495	1335	990	0.218
A= 2	5	3365	2825	2080	1625	1460	1090	0.195
	6	3515	2985	2225	1750	1575	1190	0.176
36/7	0	1140	895	615				0.818
	1	1395	1125	795	605	535		0.568
α_1=α_2= 2.000	2	1595	1325	960	745	665	495	0.435
α_p^2 = α_e^2= 0.778	3	1755	1490	1110	875	785	595	0.353
N= 2.000	4	1880	1630	1250	995	900	695	0.297
A= 1	5	1980	1750	1370	1105	1005	780	0.256
	6	2060	1845	1480	1210	1105	865	0.225
36/5	0	965	785	560				0.982
	1	1145	965	715	560	500		0.643
α_1=α_2= 1.667	2	1270	1110	855	680	615	470	0.478
α_p^2 = α_e^2= 0.722	3	1360	1215	970	790	720	560	0.380
N= 1.333	4	1420	1295	1065	885	810	645	0.316
A= 1	5	1465	1360	1150	970	895	720	0.270
	6	1500	1410	1215	1045	970	790	0.236
36/4	0	735	600	430				1.228
	1	900	770	580	460	410		0.740
α_1=α_2= 1.333	2	1005	895	705	570	520	400	0.529
α_p^2 = α_e^2= 0.556	3	1070	975	805	670	615	485	0.412
N= 1.000	4	1110	1035	885	750	695	560	0.337
A= 1	5	1140	1080	945	820	765	630	0.286
	6	1160	1110	995	880	825	690	0.248

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2495	1965	1345				0.302
36/14	1	2765	2205	1540	1145	1010		0.260
$\alpha_1=\alpha_2=$ 4.000	2	3010	2425	1715	1295	1145	845	0.228
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3230	2635	1885	1445	1280	945	0.203
N= 4.000	4	3430	2830	2045	1585	1415	1045	0.183
A= 2	5	3605	3010	2205	1715	1540	1145	0.167
	6	3765	3180	2355	1845	1660	1245	0.153
	0	1245	980	670				0.605
36/7	1	1505	1210	855	645	570		0.456
$\alpha_1=\alpha_2=$ 2.000	2	1715	1415	1020	790	705	520	0.366
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1880	1590	1175	920	830	620	0.306
N= 2.000	4	2015	1735	1315	1045	940	720	0.263
A= 1	5	2125	1860	1445	1160	1050	815	0.230
	6	2210	1970	1560	1265	1150	900	0.205
	0	1055	860	610				0.726
36/5	1	1240	1045	770	600	540		0.522
$\alpha_1=\alpha_2=$ 1.667	2	1370	1190	910	725	655	495	0.408
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1465	1305	1030	835	760	590	0.334
N= 1.333	4	1535	1390	1135	935	855	675	0.283
A= 1	5	1585	1460	1220	1025	940	750	0.246
	6	1625	1515	1295	1105	1020	825	0.217
	0	805	655	470				0.907
36/4	1	975	830	625	490	435		0.610
$\alpha_1=\alpha_2=$ 1.333	2	1085	960	750	605	550	420	0.459
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1155	1050	855	705	645	510	0.368
N= 1.000	4	1200	1115	940	795	730	585	0.307
A= 1	5	1235	1160	1005	865	805	655	0.264
	6	1260	1195	1060	930	870	720	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	2495	1965	1345				0.302
		1	2765	2205	1540	1145	1010		0.260
α_1=α_2=	4.000	2	3010	2425	1715	1295	1145	845	0.228
α_p^2 = α_e^2=	1.556	3	3230	2635	1885	1445	1280	945	0.203
N=	4.000	4	3430	2830	2045	1585	1415	1045	0.183
A=	2	5	3605	3010	2205	1715	1540	1145	0.167
		6	3765	3180	2355	1845	1660	1245	0.153
36/7		0	1245	980	670				0.605
		1	1505	1210	855	645	570		0.456
α_1=α_2=	2.000	2	1715	1415	1020	790	705	520	0.366
α_p^2 = α_e^2=	0.778	3	1880	1590	1175	920	830	620	0.306
N=	2.000	4	2015	1735	1315	1045	940	720	0.263
A=	1	5	2125	1860	1445	1160	1050	815	0.230
		6	2210	1970	1560	1265	1150	900	0.205
36/5		0	1055	860	610				0.726
		1	1240	1045	770	600	540		0.522
α_1=α_2=	1.667	2	1370	1190	910	725	655	495	0.408
α_p^2 = α_e^2=	0.722	3	1465	1305	1030	835	760	590	0.334
N=	1.333	4	1535	1390	1135	935	855	675	0.283
A=	1	5	1585	1460	1220	1025	940	750	0.246
		6	1625	1515	1295	1105	1020	825	0.217
36/4		0	805	655	470				0.907
		1	975	830	625	490	435		0.610
α_1=α_2=	1.333	2	1085	960	750	605	550	420	0.459
α_p^2 = α_e^2=	0.556	3	1155	1050	855	705	645	510	0.368
N=	1.000	4	1200	1115	940	795	730	585	0.307
A=	1	5	1235	1160	1005	865	805	655	0.264
		6	1260	1195	1060	930	870	720	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR20		
Design thickness = 0.0358 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2495	1965	1345				0.302
36/14	1	2765	2205	1540	1145	1010		0.260
$\alpha_1=\alpha_2=$ 4.000	2	3010	2425	1715	1295	1145	845	0.228
$\alpha_p^2=\alpha_e^2=$ 1.556	3	3230	2635	1885	1445	1280	945	0.203
N= 4.000	4	3430	2830	2045	1585	1415	1045	0.183
A= 2	5	3605	3010	2205	1715	1540	1145	0.167
	6	3765	3180	2355	1845	1660	1245	0.153
	0	1245	980	670				0.605
36/7	1	1505	1210	855	645	570		0.456
$\alpha_1=\alpha_2=$ 2.000	2	1715	1415	1020	790	705	520	0.366
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1880	1590	1175	920	830	620	0.306
N= 2.000	4	2015	1735	1315	1045	940	720	0.263
A= 1	5	2125	1860	1445	1160	1050	815	0.230
	6	2210	1970	1560	1265	1150	900	0.205
	0	1055	860	610				0.726
36/5	1	1240	1045	770	600	540		0.522
$\alpha_1=\alpha_2=$ 1.667	2	1370	1190	910	725	655	495	0.408
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1465	1305	1030	835	760	590	0.334
N= 1.333	4	1535	1390	1135	935	855	675	0.283
A= 1	5	1585	1460	1220	1025	940	750	0.246
	6	1625	1515	1295	1105	1020	825	0.217
	0	805	655	470				0.907
36/4	1	975	830	625	490	435		0.610
$\alpha_1=\alpha_2=$ 1.333	2	1085	960	750	605	550	420	0.459
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1155	1050	855	705	645	510	0.368
N= 1.000	4	1200	1115	940	795	730	585	0.307
A= 1	5	1235	1160	1005	865	805	655	0.264
	6	1260	1195	1060	930	870	720	0.231

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.210	45927	25834	11482	6459	5103	2870

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	1110	875	600				1.039
36/14	1	1475	1210	870	670	595		0.700
α_1=α_2= 4.000	2	1730	1470	1100	830	735	550	0.527
α_p^2 = α_e^2= 1.556	3	1900	1670	1255	940	835	625	0.423
N= 4.000	4	2020	1820	1405	1055	935	700	0.353
A= 2	5	2105	1935	1560	1170	1040	780	0.303
	6	2170	2020	1710	1280	1140	855	0.266
	0	555	435	300				2.078
36/7	1	865	735	550	415	365		1.054
α_1=α_2= 2.000	2	1010	910	700	525	465	350	0.706
α_p^2 = α_e^2= 0.778	3	1085	1010	855	640	570	425	0.531
N= 2.000	4	1120	1070	950	755	670	500	0.426
A= 1	5	1145	1105	1010	865	770	575	0.355
	6	1160	1130	1050	965	870	650	0.305
	0	470	380	270				2.494
36/5	1	665	595	480	360	320		1.152
α_1=α_2= 1.667	2	735	690	600	475	425	315	0.749
α_p^2 = α_e^2= 0.722	3	760	735	670	590	525	390	0.555
N= 1.333	4	775	755	710	655	625	465	0.441
A= 1	5	785	770	735	690	670	545	0.365
	6	790	780	750	715	700	620	0.312
	0	355	290	205				3.117
36/4	1	520	480	395	310	275		1.269
α_1=α_2= 1.333	2	565	540	485	425	380	285	0.797
α_p^2 = α_e^2= 0.556	3	580	565	530	490	470	360	0.581
N= 1.000	4	590	580	555	525	510	435	0.457
A= 1	5	595	585	570	545	535	495	0.376
	6	595	590	575	560	550	520	0.320

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1500	1180	815				0.940
	1	1880	1525	1085	830	735		0.653
$\alpha_1=\alpha_2=$ 4.000	2	2175	1815	1330	1035	930	695	0.500
$\alpha_p^2=\alpha_e^2=$ 1.556	3	2395	2055	1550	1225	1105	850	0.406
N= 4.000	4	2565	2250	1750	1400	1270	950	0.341
A= 2	5	2690	2410	1920	1565	1405	1050	0.294
	6	2790	2535	2075	1715	1540	1155	0.259
36/7	0	750	590	405				1.879
	1	1085	905	665	515	465		1.001
$\alpha_1=\alpha_2=$ 2.000	2	1280	1125	875	700	635	475	0.682
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1395	1265	1035	855	770	575	0.517
N= 2.000	4	1460	1360	1160	985	905	680	0.417
A= 1	5	1505	1425	1250	1090	1015	780	0.349
	6	1535	1470	1320	1175	1105	880	0.300
36/5	0	635	515	365				2.255
	1	855	755	585	470	425		1.098
$\alpha_1=\alpha_2=$ 1.667	2	955	880	735	620	570	430	0.726
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1005	950	835	730	680	530	0.542
N= 1.333	4	1030	990	900	810	765	635	0.433
A= 1	5	1045	1015	945	865	830	720	0.360
	6	1055	1035	975	910	875	775	0.308
36/4	0	485	395	280				2.819
	1	675	605	485	395	360		1.217
$\alpha_1=\alpha_2=$ 1.333	2	745	700	605	525	485	385	0.776
$\alpha_p^2=\alpha_e^2=$ 0.556	3	770	745	675	610	575	485	0.569
N= 1.000	4	785	765	720	665	635	555	0.450
A= 1	5	795	780	745	700	675	610	0.372
	6	800	790	760	725	705	645	0.317

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	2235	1760	1210				0.823
	1	2630	2115	1490	1125	995		0.594
α_1=α_2= 4.000	2	2965	2430	1745	1350	1200	890	0.465
α_p^2 = α_e^2= 1.556	3	3245	2710	1990	1550	1390	1040	0.382
N= 4.000	4	3475	2960	2210	1740	1565	1195	0.324
A= 2	5	3665	3170	2420	1925	1740	1340	0.282
	6	3825	3360	2615	2095	1900	1475	0.249
36/7	0	1115	880	605				1.646
	1	1480	1215	870	675	600		0.931
α_1=α_2= 2.000	2	1735	1480	1105	870	780	595	0.649
α_p^2 = α_e^2= 0.778	3	1910	1680	1305	1045	950	735	0.498
N= 2.000	4	2030	1830	1475	1205	1100	865	0.404
A= 1	5	2115	1945	1615	1350	1240	985	0.340
	6	2180	2030	1730	1470	1360	1100	0.293
36/5	0	945	770	550				1.975
	1	1195	1030	780	615	555		1.027
α_1=α_2= 1.667	2	1340	1200	960	785	715	560	0.694
α_p^2 = α_e^2= 0.722	3	1425	1315	1100	925	850	680	0.524
N= 1.333	4	1475	1390	1205	1040	965	790	0.421
A= 1	5	1510	1440	1285	1130	1065	885	0.352
	6	1535	1480	1345	1210	1140	970	0.302
36/4	0	720	590	420				2.469
	1	945	825	640	510	465		1.147
α_1=α_2= 1.333	2	1055	965	800	665	610	485	0.747
α_p^2 = α_e^2= 0.556	3	1110	1045	905	785	730	595	0.554
N= 1.000	4	1140	1090	980	870	820	690	0.440
A= 1	5	1160	1125	1035	940	895	765	0.365
	6	1170	1145	1070	990	950	830	0.312

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	3165	2490	1715				0.427
	1	3570	2850	2000	1500	1325		0.356
$\alpha_1=\alpha_2=$ 4.000	2	3930	3185	2260	1730	1530	1130	0.305
$\alpha_p^2=\alpha_e^2=$ 1.556	3	4250	3490	2515	1945	1735	1285	0.267
N= 4.000	4	4530	3775	2755	2145	1925	1440	0.237
A= 2	5	4775	4030	2985	2340	2105	1590	0.214
	6	4990	4265	3205	2525	2280	1745	0.194
36/7	0	1580	1245	855				0.854
	1	1965	1590	1130	865	765		0.610
$\alpha_1=\alpha_2=$ 2.000	2	2265	1885	1375	1070	960	720	0.475
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2495	2130	1600	1260	1140	870	0.389
N= 2.000	4	2670	2330	1800	1440	1305	1010	0.329
A= 1	5	2805	2500	1980	1605	1460	1140	0.285
	6	2910	2635	2140	1760	1610	1270	0.252
36/5	0	1340	1090	775				1.025
	1	1610	1365	1015	795	715		0.693
$\alpha_1=\alpha_2=$ 1.667	2	1790	1570	1220	975	885	685	0.524
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1915	1725	1390	1135	1035	815	0.421
N= 1.333	4	2000	1840	1530	1275	1175	935	0.352
A= 1	5	2060	1925	1645	1400	1295	1050	0.302
	6	2105	1990	1735	1505	1405	1150	0.265
36/4	0	1020	835	595				1.281
	1	1270	1090	825	650	590		0.801
$\alpha_1=\alpha_2=$ 1.333	2	1415	1265	1010	820	750	585	0.583
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1505	1385	1150	965	885	705	0.458
N= 1.000	4	1560	1465	1260	1085	1005	815	0.377
A= 1	5	1595	1520	1345	1180	1105	915	0.321
	6	1620	1560	1410	1260	1190	1005	0.279

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	3305	2605	1790				0.348
		1	3715	2965	2075	1555	1375		0.299
α_1=α_2=	4.000	2	4075	3300	2340	1785	1580	1165	0.263
$\alpha_p^2 = \alpha_e^2$=	1.556	3	4400	3610	2595	2005	1785	1320	0.234
N=	4.000	4	4685	3895	2835	2205	1980	1475	0.211
A=	2	5	4935	4155	3070	2400	2160	1630	0.192
		6	5155	4395	3290	2590	2335	1780	0.176
36/7		0	1650	1300	895				0.696
		1	2035	1650	1170	890	790		0.525
α_1=α_2=	2.000	2	2340	1945	1415	1100	990	735	0.422
$\alpha_p^2 = \alpha_e^2$=	0.778	3	2575	2195	1645	1295	1165	890	0.352
N=	2.000	4	2760	2405	1850	1475	1335	1030	0.303
A=	1	5	2900	2575	2030	1640	1495	1165	0.265
		6	3015	2715	2195	1800	1640	1290	0.236
36/5		0	1400	1140	810				0.835
		1	1675	1415	1050	825	740		0.601
α_1=α_2=	1.667	2	1860	1625	1255	1005	910	705	0.469
$\alpha_p^2 = \alpha_e^2$=	0.722	3	1985	1785	1430	1165	1065	835	0.385
N=	1.333	4	2075	1900	1575	1310	1205	955	0.326
A=	1	5	2140	1990	1695	1435	1330	1070	0.283
		6	2190	2060	1790	1545	1440	1175	0.250
36/4		0	1065	870	620				1.044
		1	1315	1130	855	675	605		0.702
α_1=α_2=	1.333	2	1470	1310	1040	845	770	600	0.528
$\alpha_p^2 = \alpha_e^2$=	0.556	3	1560	1430	1185	990	910	725	0.424
N=	1.000	4	1620	1515	1300	1110	1030	835	0.354
A=	1	5	1660	1575	1390	1215	1135	935	0.304
		6	1690	1620	1460	1295	1220	1025	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR18		
Design thickness = 0.0474 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	3305	2605	1790				0.348
36/14	1	3715	2965	2075	1555	1375		0.299
$\alpha_1=\alpha_2=$ 4.000	2	4075	3300	2340	1785	1580	1165	0.263
$\alpha_p^2=\alpha_e^2=$ 1.556	3	4400	3610	2595	2005	1785	1320	0.234
N= 4.000	4	4685	3895	2835	2205	1980	1475	0.211
A= 2	5	4935	4155	3070	2400	2160	1630	0.192
	6	5155	4395	3290	2590	2335	1780	0.176
	0	1650	1300	895				0.696
36/7	1	2035	1650	1170	890	790		0.525
$\alpha_1=\alpha_2=$ 2.000	2	2340	1945	1415	1100	990	735	0.422
$\alpha_p^2=\alpha_e^2=$ 0.778	3	2575	2195	1645	1295	1165	890	0.352
N= 2.000	4	2760	2405	1850	1475	1335	1030	0.303
A= 1	5	2900	2575	2030	1640	1495	1165	0.265
	6	3015	2715	2195	1800	1640	1290	0.236
	0	1400	1140	810				0.835
36/5	1	1675	1415	1050	825	740		0.601
$\alpha_1=\alpha_2=$ 1.667	2	1860	1625	1255	1005	910	705	0.469
$\alpha_p^2=\alpha_e^2=$ 0.722	3	1985	1785	1430	1165	1065	835	0.385
N= 1.333	4	2075	1900	1575	1310	1205	955	0.326
A= 1	5	2140	1990	1695	1435	1330	1070	0.283
	6	2190	2060	1790	1545	1440	1175	0.250
	0	1065	870	620				1.044
36/4	1	1315	1130	855	675	605		0.702
$\alpha_1=\alpha_2=$ 1.333	2	1470	1310	1040	845	770	600	0.528
$\alpha_p^2=\alpha_e^2=$ 0.556	3	1560	1430	1185	990	910	725	0.424
N= 1.000	4	1620	1515	1300	1110	1030	835	0.354
A= 1	5	1660	1575	1390	1215	1135	935	0.304
	6	1690	1620	1460	1295	1220	1025	0.266

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.279	70070	39414	17517	9854	7786	4379

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 27 mils		
Framing thickness = 0.0283 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
		0	1110	875	605				1.311
36/14		1	1590	1325	955	715	635		0.848
α_1=α_2=	4.000	2	1880	1640	1105	830	735	550	0.627
α_p^2 = α_e^2=	1.556	3	2045	1855	1255	940	835	625	0.497
N=	4.000	4	2150	1995	1405	1055	935	700	0.412
A=	2	5	2215	2090	1560	1170	1040	780	0.352
		6	2260	2160	1710	1280	1140	855	0.307
		0	555	435	300				2.622
36/7		1	940	820	550	415	365		1.254
α_1=α_2=	2.000	2	1075	995	700	525	465	350	0.824
α_p^2 = α_e^2=	0.778	3	1130	1080	855	640	570	425	0.614
N=	2.000	4	1155	1120	1005	755	670	500	0.489
A=	1	5	1170	1145	1085	865	770	575	0.406
		6	1180	1160	1115	980	870	650	0.348
		0	470	380	270				3.146
36/5		1	700	645	485	360	320		1.363
α_1=α_2=	1.667	2	760	730	635	475	425	315	0.870
α_p^2 = α_e^2=	0.722	3	780	760	715	590	525	390	0.639
N=	1.333	4	785	775	745	700	625	465	0.505
A=	1	5	790	785	765	735	720	545	0.417
		6	795	790	775	755	740	620	0.355
		0	355	290	205				3.933
36/4		1	545	515	415	310	275		1.492
α_1=α_2=	1.333	2	580	565	525	425	380	285	0.921
α_p^2 = α_e^2=	0.556	3	590	580	560	530	480	360	0.666
N=	1.000	4	595	590	575	555	545	435	0.521
A=	1	5	595	595	585	570	565	510	0.429
		6	600	595	590	580	575	555	0.364

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 33 mils		
Framing thickness = 0.0346 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	1500	1180	815				1.185
	1	2015	1655	1190	920	825		0.794
α_1=α_2= 4.000	2	2365	2020	1495	1120	995	745	0.597
$\alpha_p^2 = \alpha_e^2$= 1.556	3	2600	2295	1700	1275	1130	850	0.478
N= 4.000	4	2760	2495	1905	1425	1270	950	0.399
A= 2	5	2870	2650	2105	1580	1405	1050	0.342
	6	2950	2760	2310	1735	1540	1155	0.300
36/7	0	750	590	405				2.371
	1	1180	1010	745	560	495		1.194
α_1=α_2= 2.000	2	1380	1245	950	710	635	475	0.798
$\alpha_p^2 = \alpha_e^2$= 0.778	3	1475	1380	1155	865	770	575	0.599
N= 2.000	4	1525	1455	1300	1020	905	680	0.480
A= 1	5	1555	1505	1380	1170	1040	780	0.400
	6	1575	1535	1435	1325	1175	880	0.343
36/5	0	635	515	365				2.845
	1	905	815	655	490	435		1.303
α_1=α_2= 1.667	2	995	940	820	645	570	430	0.845
$\alpha_p^2 = \alpha_e^2$= 0.722	3	1035	1000	915	795	710	530	0.625
N= 1.333	4	1050	1030	965	900	845	635	0.496
A= 1	5	1065	1045	1000	945	915	735	0.411
	6	1070	1055	1020	980	955	835	0.351
36/4	0	485	395	280				3.557
	1	710	655	545	425	375		1.435
α_1=α_2= 1.333	2	770	740	670	575	510	385	0.899
$\alpha_p^2 = \alpha_e^2$= 0.556	3	790	770	725	675	645	485	0.654
N= 1.000	4	800	785	755	720	700	585	0.514
A= 1	5	805	795	775	745	730	680	0.424
	6	805	800	785	760	750	710	0.360

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-}deck}$=	50	ksi
$F_{u\text{-}deck}$=	65	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	2235	1760	1215				1.038
36/14	1	2780	2250	1600	1230	1085		0.725
α_1=α_2= 4.000	2	3200	2670	1950	1515	1360	1020	0.557
α_p^2 = α_e^2= 1.556	3	3525	3015	2265	1785	1610	1240	0.452
N= 4.000	4	3775	3300	2550	2040	1845	1415	0.381
A= 2	5	3965	3535	2805	2275	2070	1565	0.329
	6	4115	3725	3025	2490	2275	1720	0.289
	0	1115	880	605				2.077
36/7	1	1600	1335	975	755	680		1.114
α_1=α_2= 2.000	2	1885	1650	1275	1020	920	705	0.761
α_p^2 = α_e^2= 0.778	3	2055	1860	1510	1245	1135	860	0.578
N= 2.000	4	2160	2005	1695	1430	1320	1010	0.466
A= 1	5	2230	2100	1835	1590	1480	1160	0.390
	6	2275	2170	1940	1715	1610	1315	0.336
	0	945	770	550				2.492
36/5	1	1265	1110	860	690	625		1.224
α_1=α_2= 1.667	2	1415	1300	1080	905	830	640	0.811
α_p^2 = α_e^2= 0.722	3	1485	1405	1230	1065	990	790	0.606
N= 1.333	4	1530	1465	1325	1185	1120	940	0.484
A= 1	5	1550	1505	1395	1275	1215	1050	0.403
	6	1570	1530	1445	1340	1285	1135	0.345
	0	720	590	420				3.115
36/4	1	1000	895	715	580	530		1.357
α_1=α_2= 1.333	2	1100	1035	890	765	710	570	0.868
α_p^2 = α_e^2= 0.556	3	1145	1100	995	890	840	710	0.638
N= 1.000	4	1170	1135	1060	975	930	810	0.504
A= 1	5	1180	1160	1100	1030	995	890	0.417
	6	1190	1170	1125	1070	1040	950	0.355

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df},\ S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf},\ \phi_{db} S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
	0	3145	2475	1710				0.927
36/14	1	3705	2975	2100	1595	1410		0.669
α_1=α_2= 4.000	2	4180	3425	2460	1900	1700	1260	0.523
α_p^2 = α_e^2= 1.556	3	4570	3820	2805	2185	1960	1480	0.430
N= 4.000	4	4895	4170	3120	2455	2210	1695	0.365
A= 2	5	5165	4470	3415	2715	2455	1895	0.317
	6	5385	4735	3685	2960	2685	2085	0.280
	0	1570	1235	855				1.854
36/7	1	2090	1710	1230	950	850		1.047
α_1=α_2= 2.000	2	2445	2085	1560	1225	1105	845	0.729
α_p^2 = α_e^2= 0.778	3	2690	2365	1840	1480	1340	1040	0.560
N= 2.000	4	2860	2580	2080	1705	1555	1225	0.454
A= 1	5	2980	2740	2280	1905	1750	1395	0.382
	6	3070	2860	2440	2080	1925	1555	0.330
	0	1330	1080	770				2.224
36/5	1	1685	1450	1100	870	785		1.156
α_1=α_2= 1.667	2	1885	1695	1355	1105	1010	790	0.780
α_p^2 = α_e^2= 0.722	3	2005	1850	1550	1305	1200	960	0.589
N= 1.333	4	2080	1955	1700	1465	1365	1115	0.473
A= 1	5	2125	2030	1810	1600	1500	1250	0.395
	6	2160	2080	1895	1705	1610	1370	0.340
	0	1015	825	590				2.781
36/4	1	1330	1165	905	725	655		1.290
α_1=α_2= 1.333	2	1485	1360	1125	940	860	685	0.839
α_p^2 = α_e^2= 0.556	3	1560	1470	1280	1105	1030	840	0.622
N= 1.000	4	1605	1540	1385	1230	1160	970	0.494
A= 1	5	1630	1580	1455	1325	1260	1080	0.410
	6	1650	1610	1510	1395	1340	1175	0.350

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14	0	4170	3285	2270				0.391
$\alpha_1=\alpha_2=$ 4.000	1	4745	3795	2660	2010	1780		0.336
$\alpha_p^2=\alpha_e^2=$ 1.556	2	5250	4260	3035	2330	2070	1530	0.295
N= 4.000	3	5690	4690	3390	2620	2350	1750	0.263
A= 2	4	6070	5080	3725	2905	2610	1965	0.237
	5	6405	5430	4045	3175	2860	2185	0.216
	6	6690	5750	4350	3440	3105	2390	0.198
36/7	0	2085	1640	1135				0.782
$\alpha_1=\alpha_2=$ 2.000	1	2625	2130	1515	1165	1035		0.590
$\alpha_p^2=\alpha_e^2=$ 0.778	2	3035	2540	1860	1450	1305	980	0.474
N= 2.000	3	3345	2875	2175	1720	1550	1195	0.396
A= 1	4	3575	3145	2450	1965	1785	1385	0.340
	5	3755	3365	2690	2195	2000	1570	0.298
	6	3885	3540	2905	2405	2205	1745	0.265
36/5	0	1765	1435	1025				0.938
$\alpha_1=\alpha_2=$ 1.667	1	2145	1825	1360	1070	960		0.675
$\alpha_p^2=\alpha_e^2=$ 0.722	2	2390	2105	1645	1320	1200	930	0.527
N= 1.333	3	2555	2310	1875	1540	1410	1110	0.432
A= 1	4	2665	2460	2065	1735	1600	1280	0.366
	5	2740	2570	2215	1900	1765	1435	0.318
	6	2795	2650	2340	2040	1910	1575	0.281
36/4	0	1345	1100	785				1.172
$\alpha_1=\alpha_2=$ 1.333	1	1690	1460	1110	880	795		0.788
$\alpha_p^2=\alpha_e^2=$ 0.556	2	1885	1695	1365	1115	1015	795	0.594
N= 1.000	3	2000	1850	1555	1310	1205	965	0.476
A= 1	4	2075	1955	1700	1470	1370	1120	0.397
	5	2120	2025	1810	1600	1500	1255	0.341
	6	2150	2075	1890	1705	1610	1370	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

1.5WR16		
Design thickness = 0.0598 in.		
$F_{y\text{-deck}}$=	50	ksi
$F_{u\text{-deck}}$=	65	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 24"	32"	48"	64"	72"	96"	K_1 1/ft
36/14		0	4170	3285	2270				0.391
		1	4745	3795	2660	2010	1780		0.336
α_1=α_2=	4.000	2	5250	4260	3035	2330	2070	1530	0.295
α_p^2 = α_e^2=	1.556	3	5690	4690	3390	2620	2350	1750	0.263
N=	4.000	4	6070	5080	3725	2905	2610	1965	0.237
A=	2	5	6405	5430	4045	3175	2860	2185	0.216
		6	6690	5750	4350	3440	3105	2390	0.198
36/7		0	2085	1640	1135				0.782
		1	2625	2130	1515	1165	1035		0.590
α_1=α_2=	2.000	2	3035	2540	1860	1450	1305	980	0.474
α_p^2 = α_e^2=	0.778	3	3345	2875	2175	1720	1550	1195	0.396
N=	2.000	4	3575	3145	2450	1965	1785	1385	0.340
A=	1	5	3755	3365	2690	2195	2000	1570	0.298
		6	3885	3540	2905	2405	2205	1745	0.265
36/5		0	1765	1435	1025				0.938
		1	2145	1825	1360	1070	960		0.675
α_1=α_2=	1.667	2	2390	2105	1645	1320	1200	930	0.527
α_p^2 = α_e^2=	0.722	3	2555	2310	1875	1540	1410	1110	0.432
N=	1.333	4	2665	2460	2065	1735	1600	1280	0.366
A=	1	5	2740	2570	2215	1900	1765	1435	0.318
		6	2795	2650	2340	2040	1910	1575	0.281
36/4		0	1345	1100	785				1.172
		1	1690	1460	1110	880	795		0.788
α_1=α_2=	1.333	2	1885	1695	1365	1115	1015	795	0.594
α_p^2 = α_e^2=	0.556	3	2000	1850	1555	1310	1205	965	0.476
N=	1.000	4	2075	1955	1700	1470	1370	1120	0.397
A=	1	5	2120	2025	1810	1600	1500	1255	0.341
		6	2150	2075	1890	1705	1610	1370	0.299

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 24"	32"	48"	64"	72"	96"
WR	0.353	99437	55933	24859	13983	11049	6215

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

Deck F_y=	60	ksi
Deck F_{u1}=	62	ksi
Framing F_{y2}=	33	ksi
Framing F_{u2}=	45	ksi

P_{nf}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
26			647	647	647	647
24			855	864	864	864
22			921	1067	1067	1067

P_{ns}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22			272	272	272	272
20			419	419	419	419
18			575	575	575	575

S_f, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22			0.0097	0.0097	0.0097	0.0097
20			0.0088	0.0084	0.0084	0.0084
18			0.0108	0.0076	0.0076	0.0076

S_s, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22			0.0224	0.0224	0.0224	0.0224
20			0.0194	0.0194	0.0194	0.0194
18			0.0175	0.0175	0.0175	0.0175

9/16" x 2 1/2" Form Deck Design thickness = 0.0179 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 43 mils Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	790	665	520	465	355	285	0.385
35/8		1	890	760	600	545	420	340	0.300
$\alpha_1=\alpha_2$=	2.286	2	980	845	680	615	480	390	0.245
$\alpha_p^2=\alpha_e^2$=	0.857	3	1050	920	750	685	535	440	0.208
N=	2.400	4	1115	985	815	745	590	485	0.180
A=	1	5	1165	1045	875	805	645	530	0.159
		6	1210	1095	930	860	695	575	0.142
		0	750	640	505	455	350	280	0.410
35/7		1	840	725	580	530	410	335	0.315
$\alpha_1=\alpha_2$=	2.143	2	910	800	650	595	465	380	0.255
$\alpha_p^2=\alpha_e^2$=	0.847	3	970	865	715	655	520	430	0.215
N=	2.057	4	1020	920	775	715	570	475	0.185
A=	1	5	1060	965	825	765	620	515	0.163
		6	1095	1005	870	815	665	560	0.146
		0	555	480	385	350	270	220	0.560
35/5		1	625	550	455	415	325	265	0.396
$\alpha_1=\alpha_2$=	1.571	2	675	610	515	475	380	315	0.306
$\alpha_p^2=\alpha_e^2$=	0.663	3	715	655	565	525	425	355	0.250
N=	1.371	4	745	690	605	570	470	395	0.211
A=	1	5	770	720	640	605	510	435	0.182
		6	785	745	675	640	545	470	0.161
		0	735	615	475	430	325	260	0.513
30/7		1	845	715	560	505	390	310	0.387
$\alpha_1=\alpha_2$=	2.000	2	935	805	640	580	450	365	0.311
$\alpha_p^2=\alpha_e^2$=	0.778	3	1015	885	715	650	505	415	0.260
N=	2.400	4	1085	955	785	715	565	460	0.223
A=	1	5	1140	1015	845	775	615	510	0.195
		6	1190	1070	905	835	670	555	0.174
		0	565	475	375	335	260	205	0.684
30/5		1	650	565	450	410	320	260	0.477
$\alpha_1=\alpha_2$=	1.500	2	720	635	520	475	375	305	0.366
$\alpha_p^2=\alpha_e^2$=	0.625	3	775	695	585	535	430	355	0.297
N=	1.600	4	820	745	635	590	475	395	0.250
A=	1	5	855	785	685	635	525	440	0.216
		6	880	820	725	680	565	480	0.190
		0	470	405	325	295	225	180	0.770
30/4		1	545	480	395	360	285	230	0.517
$\alpha_1=\alpha_2$=	1.333	2	595	540	455	420	335	280	0.390
$\alpha_p^2=\alpha_e^2$=	0.556	3	635	580	505	470	385	320	0.313
N=	1.200	4	660	615	545	510	425	360	0.261
A=	1	5	680	645	575	545	460	395	0.224
		6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0239 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1045	880	685	620	475	380	0.466
35/8		1	1200	1025	815	735	565	460	0.359
α_1=α_2=	2.286	2	1325	1150	930	845	660	535	0.292
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1430	1260	1035	945	745	610	0.246
N=	2.400	4	1520	1355	1130	1040	830	680	0.213
A=	1	5	1590	1440	1220	1125	905	750	0.187
		6	1650	1510	1295	1205	980	815	0.167
		0	990	845	665	600	465	375	0.497
35/7		1	1125	975	785	715	555	450	0.377
α_1=α_2=	2.143	2	1230	1085	890	815	640	525	0.304
$\alpha_p^2 = \alpha_e^2$=	0.847	3	1315	1180	985	905	725	595	0.254
N=	2.057	4	1385	1255	1070	990	800	665	0.219
A=	1	5	1440	1320	1145	1065	870	730	0.192
		6	1485	1375	1210	1130	935	790	0.171
		0	735	635	510	460	355	290	0.678
35/5		1	840	745	615	560	445	365	0.473
α_1=α_2=	1.571	2	915	825	700	650	525	435	0.363
$\alpha_p^2 = \alpha_e^2$=	0.663	3	970	890	775	725	595	500	0.295
N=	1.371	4	1010	940	835	785	655	560	0.248
A=	1	5	1035	980	885	835	710	610	0.214
		6	1060	1010	925	880	760	660	0.188
		0	975	810	630	565	430	345	0.622
30/7		1	1135	965	760	685	525	425	0.464
α_1=α_2=	2.000	2	1275	1100	880	800	620	505	0.370
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1390	1215	990	905	710	580	0.307
N=	2.400	4	1485	1320	1090	1000	795	650	0.263
A=	1	5	1560	1405	1180	1090	875	720	0.230
		6	1625	1480	1265	1170	950	790	0.204
		0	745	630	495	445	340	275	0.829
30/5		1	880	760	615	560	435	355	0.570
α_1=α_2=	1.500	2	980	870	720	660	520	425	0.434
$\alpha_p^2 = \alpha_e^2$=	0.625	3	1055	955	810	745	600	495	0.351
N=	1.600	4	1115	1025	885	820	670	565	0.294
A=	1	5	1160	1080	950	890	740	625	0.253
		6	1195	1120	1000	945	795	680	0.222
		0	625	535	430	390	300	240	0.933
30/4		1	735	650	535	490	390	320	0.617
α_1=α_2=	1.333	2	805	730	625	575	465	385	0.461
$\alpha_p^2 = \alpha_e^2$=	0.556	3	860	795	695	650	535	450	0.368
N=	1.200	4	895	840	750	705	595	510	0.306
A=	1	5	920	875	795	755	650	560	0.262
		6	940	900	830	795	695	605	0.229

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1130	945	740	665	510	410	0.708
35/8		1	1330	1145	910	825	640	520	0.503
α_1=α_2=	2.286	2	1495	1310	1065	970	760	620	0.390
α_p^2 = α_e^2=	0.857	3	1620	1445	1200	1105	875	720	0.319
N=	2.400	4	1720	1555	1320	1220	985	815	0.269
A=	1	5	1800	1650	1425	1325	1080	905	0.233
		6	1860	1725	1515	1415	1175	990	0.205
		0	1070	910	720	650	500	405	0.755
35/7		1	1245	1085	880	800	625	510	0.526
α_1=α_2=	2.143	2	1380	1225	1020	935	740	610	0.404
α_p^2 = α_e^2=	0.847	3	1480	1340	1135	1050	850	705	0.328
N=	2.057	4	1555	1430	1240	1155	945	795	0.276
A=	1	5	1615	1500	1325	1240	1035	875	0.238
		6	1660	1560	1395	1320	1115	955	0.209
		0	790	680	545	495	385	315	1.030
35/5		1	930	825	690	630	500	415	0.647
α_1=α_2=	1.571	2	1020	930	800	745	605	505	0.471
α_p^2 = α_e^2=	0.663	3	1080	1005	890	835	695	590	0.371
N=	1.371	4	1120	1060	955	905	775	665	0.306
A=	1	5	1150	1100	1010	965	840	730	0.260
		6	1170	1130	1050	1010	895	790	0.226
		0	1050	875	680	610	465	370	0.944
30/7		1	1270	1080	855	775	595	480	0.644
α_1=α_2=	2.000	2	1445	1255	1015	925	720	585	0.489
α_p^2 = α_e^2=	0.778	3	1580	1400	1160	1060	840	690	0.394
N=	2.400	4	1690	1520	1280	1180	950	785	0.330
A=	1	5	1775	1620	1390	1290	1050	875	0.284
		6	1840	1705	1485	1385	1145	960	0.249
		0	800	680	535	480	370	295	1.259
30/5		1	980	855	695	630	495	405	0.777
α_1=α_2=	1.500	2	1105	990	830	760	610	500	0.561
α_p^2 = α_e^2=	0.625	3	1190	1090	940	870	710	595	0.440
N=	1.600	4	1255	1165	1025	965	800	680	0.361
A=	1	5	1295	1220	1100	1040	880	755	0.307
		6	1330	1265	1155	1100	950	825	0.266
		0	675	580	460	420	325	260	1.417
30/4		1	815	725	605	555	440	365	0.834
α_1=α_2=	1.333	2	900	830	715	665	545	455	0.591
α_p^2 = α_e^2=	0.556	3	955	895	795	750	630	535	0.457
N=	1.200	4	995	945	860	815	705	610	0.373
A=	1	5	1015	975	905	870	760	670	0.315
		6	1035	1000	940	910	810	725	0.273

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0179 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 54 mils Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
	0	790	665	520	465	355	285	0.385
35/8	1	890	760	600	545	420	340	0.300
$\alpha_1=\alpha_2=$ 2.286	2	980	845	680	615	480	390	0.245
$\alpha_p^2=\alpha_e^2=$ 0.857	3	1050	920	750	685	535	440	0.208
N= 2.400	4	1115	985	815	745	590	485	0.180
A= 1	5	1165	1045	875	805	645	530	0.159
	6	1210	1095	930	860	695	575	0.142
	0	750	640	505	455	350	280	0.410
35/7	1	840	725	580	530	410	335	0.315
$\alpha_1=\alpha_2=$ 2.143	2	910	800	650	595	465	380	0.255
$\alpha_p^2=\alpha_e^2=$ 0.847	3	970	865	715	655	520	430	0.215
N= 2.057	4	1020	920	775	715	570	475	0.185
A= 1	5	1060	965	825	765	620	515	0.163
	6	1095	1005	870	815	665	560	0.146
	0	555	480	385	350	270	220	0.560
35/5	1	625	550	455	415	325	265	0.396
$\alpha_1=\alpha_2=$ 1.571	2	675	610	515	475	380	315	0.306
$\alpha_p^2=\alpha_e^2=$ 0.663	3	715	655	565	525	425	355	0.250
N= 1.371	4	745	690	605	570	470	395	0.211
A= 1	5	770	720	640	605	510	435	0.182
	6	785	745	675	640	545	470	0.161
	0	735	615	475	430	325	260	0.513
30/7	1	845	715	560	505	390	310	0.387
$\alpha_1=\alpha_2=$ 2.000	2	935	805	640	580	450	365	0.311
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1015	885	715	650	505	415	0.260
N= 2.400	4	1085	955	785	715	565	460	0.223
A= 1	5	1140	1015	845	775	615	510	0.195
	6	1190	1070	905	835	670	555	0.174
	0	565	475	375	335	260	205	0.684
30/5	1	650	565	450	410	320	260	0.477
$\alpha_1=\alpha_2=$ 1.500	2	720	635	520	475	375	305	0.366
$\alpha_p^2=\alpha_e^2=$ 0.625	3	775	695	585	535	430	355	0.297
N= 1.600	4	820	745	635	590	475	395	0.250
A= 1	5	855	785	685	635	525	440	0.216
	6	880	820	725	680	565	480	0.190
	0	470	405	325	295	225	180	0.770
30/4	1	545	480	395	360	285	230	0.517
$\alpha_1=\alpha_2=$ 1.333	2	595	540	455	420	335	280	0.390
$\alpha_p^2=\alpha_e^2=$ 0.556	3	635	580	505	470	385	320	0.313
N= 1.200	4	660	615	545	510	425	360	0.261
A= 1	5	680	645	575	545	460	395	0.224
	6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0239 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 54 mils Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1060	890	695	625	480	385	0.445
35/8		1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2$=	2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1445	1275	1045	955	750	615	0.240
N=	2.400	4	1535	1370	1140	1045	835	685	0.208
A=	1	5	1605	1450	1225	1135	910	755	0.184
		6	1665	1520	1305	1210	985	820	0.164
		0	1005	855	675	610	470	380	0.474
35/7		1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2$=	2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2 = \alpha_e^2$=	0.847	3	1330	1190	995	915	730	600	0.248
N=	2.057	4	1395	1265	1080	1000	805	670	0.214
A=	1	5	1450	1335	1150	1070	875	735	0.188
		6	1495	1390	1215	1140	945	795	0.168
		0	740	640	515	465	360	295	0.647
35/5		1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2$=	1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2 = \alpha_e^2$=	0.663	3	975	900	780	730	600	500	0.289
N=	1.371	4	1015	950	840	790	660	560	0.244
A=	1	5	1045	990	890	845	715	615	0.211
		6	1070	1020	930	885	765	665	0.186
		0	985	820	635	570	435	345	0.593
30/7		1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2$=	2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1400	1225	1000	910	715	585	0.300
N=	2.400	4	1495	1330	1100	1010	800	655	0.258
A=	1	5	1575	1415	1190	1095	880	725	0.226
		6	1640	1490	1270	1180	955	795	0.201
		0	750	635	500	450	345	275	0.791
30/5		1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2$=	1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2 = \alpha_e^2$=	0.625	3	1065	965	815	750	605	500	0.344
N=	1.600	4	1125	1030	890	830	675	565	0.289
A=	1	5	1170	1085	955	895	740	625	0.250
		6	1205	1130	1010	950	800	685	0.220
		0	630	545	435	390	305	245	0.890
30/4		1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2$=	1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2 = \alpha_e^2$=	0.556	3	865	800	700	655	540	455	0.361
N=	1.200	4	900	845	755	710	600	510	0.301
A=	1	5	930	880	800	760	650	565	0.259
		6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1305	1100	860	770	590	475	0.494
35/8		1	1515	1295	1030	930	720	585	0.385
$\alpha_1=\alpha_2=$	2.286	2	1685	1465	1190	1080	845	690	0.315
$\alpha_p^2=\alpha_e^2=$	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
		0	1240	1055	835	750	580	470	0.527
35/7		1	1420	1230	995	905	705	575	0.404
$\alpha_1=\alpha_2=$	2.143	2	1560	1380	1140	1040	820	675	0.328
$\alpha_p^2=\alpha_e^2=$	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
		0	915	790	635	575	445	365	0.719
35/5		1	1055	940	780	715	565	465	0.508
$\alpha_1=\alpha_2=$	1.571	2	1155	1050	895	830	670	560	0.393
$\alpha_p^2=\alpha_e^2=$	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
		0	1215	1015	785	705	540	430	0.659
30/7		1	1440	1220	965	870	670	540	0.497
$\alpha_1=\alpha_2=$	2.000	2	1620	1405	1130	1025	795	650	0.399
$\alpha_p^2=\alpha_e^2=$	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
		0	930	785	620	555	430	345	0.878
30/5		1	1110	965	780	710	555	450	0.613
$\alpha_1=\alpha_2=$	1.500	2	1245	1110	920	845	670	550	0.470
$\alpha_p^2=\alpha_e^2=$	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
		0	780	670	535	485	375	305	0.988
30/4		1	925	820	680	625	495	405	0.664
$\alpha_1=\alpha_2=$	1.333	2	1020	930	795	740	600	500	0.500
$\alpha_p^2=\alpha_e^2=$	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in⁴/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0179 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
	0	790	665	520	465	355	285	0.385
35/8	1	890	760	600	545	420	340	0.300
α_1=α_2= 2.286	2	980	845	680	615	480	390	0.245
α_p^2 = α_e^2= 0.857	3	1050	920	750	685	535	440	0.208
N= 2.400	4	1115	985	815	745	590	485	0.180
A= 1	5	1165	1045	875	805	645	530	0.159
	6	1210	1095	930	860	695	575	0.142
	0	750	640	505	455	350	280	0.410
35/7	1	840	725	580	530	410	335	0.315
α_1=α_2= 2.143	2	910	800	650	595	465	380	0.255
α_p^2 = α_e^2= 0.847	3	970	865	715	655	520	430	0.215
N= 2.057	4	1020	920	775	715	570	475	0.185
A= 1	5	1060	965	825	765	620	515	0.163
	6	1095	1005	870	815	665	560	0.146
	0	555	480	385	350	270	220	0.560
35/5	1	625	550	455	415	325	265	0.396
α_1=α_2= 1.571	2	675	610	515	475	380	315	0.306
α_p^2 = α_e^2= 0.663	3	715	655	565	525	425	355	0.250
N= 1.371	4	745	690	605	570	470	395	0.211
A= 1	5	770	720	640	605	510	435	0.182
	6	785	745	675	640	545	470	0.161
	0	735	615	475	430	325	260	0.513
30/7	1	845	715	560	505	390	310	0.387
α_1=α_2= 2.000	2	935	805	640	580	450	365	0.311
α_p^2 = α_e^2= 0.778	3	1015	885	715	650	505	415	0.260
N= 2.400	4	1085	955	785	715	565	460	0.223
A= 1	5	1140	1015	845	775	615	510	0.195
	6	1190	1070	905	835	670	555	0.174
	0	565	475	375	335	260	205	0.684
30/5	1	650	565	450	410	320	260	0.477
α_1=α_2= 1.500	2	720	635	520	475	375	305	0.366
α_p^2 = α_e^2= 0.625	3	775	695	585	535	430	355	0.297
N= 1.600	4	820	745	635	590	475	395	0.250
A= 1	5	855	785	685	635	525	440	0.216
	6	880	820	725	680	565	480	0.190
	0	470	405	325	295	225	180	0.770
30/4	1	545	480	395	360	285	230	0.517
α_1=α_2= 1.333	2	595	540	455	420	335	280	0.390
α_p^2 = α_e^2= 0.556	3	635	580	505	470	385	320	0.313
N= 1.200	4	660	615	545	510	425	360	0.261
A= 1	5	680	645	575	545	460	395	0.224
	6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0239 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1060	890	695	625	480	385	0.445
35/8		1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2=$	2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2=\alpha_e^2=$	0.857	3	1445	1275	1045	955	750	615	0.240
N=	2.400	4	1535	1370	1140	1045	835	685	0.208
A=	1	5	1605	1450	1225	1135	910	755	0.184
		6	1665	1520	1305	1210	985	820	0.164
		0	1005	855	675	610	470	380	0.474
35/7		1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2=$	2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2=\alpha_e^2=$	0.847	3	1330	1190	995	915	730	600	0.248
N=	2.057	4	1395	1265	1080	1000	805	670	0.214
A=	1	5	1450	1335	1150	1070	875	735	0.188
		6	1495	1390	1215	1140	945	795	0.168
		0	740	640	515	465	360	295	0.647
35/5		1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2=$	1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2=\alpha_e^2=$	0.663	3	975	900	780	730	600	500	0.289
N=	1.371	4	1015	950	840	790	660	560	0.244
A=	1	5	1045	990	890	845	715	615	0.211
		6	1070	1020	930	885	765	665	0.186
		0	985	820	635	570	435	345	0.593
30/7		1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2=$	2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2=\alpha_e^2=$	0.778	3	1400	1225	1000	910	715	585	0.300
N=	2.400	4	1495	1330	1100	1010	800	655	0.258
A=	1	5	1575	1415	1190	1095	880	725	0.226
		6	1640	1490	1270	1180	955	795	0.201
		0	750	635	500	450	345	275	0.791
30/5		1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2=$	1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2=\alpha_e^2=$	0.625	3	1065	965	815	750	605	500	0.344
N=	1.600	4	1125	1030	890	830	675	565	0.289
A=	1	5	1170	1085	955	895	740	625	0.250
		6	1205	1130	1010	950	800	685	0.220
		0	630	545	435	390	305	245	0.890
30/4		1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2=$	1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2=\alpha_e^2=$	0.556	3	865	800	700	655	540	455	0.361
N=	1.200	4	900	845	755	710	600	510	0.301
A=	1	5	930	880	800	760	650	565	0.259
		6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 68 mils Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
35/8		0	1305	1100	860	770	590	475	0.494
		1	1515	1295	1030	930	720	585	0.385
$\alpha_1=\alpha_2$=	2.286	2	1685	1465	1190	1080	845	690	0.315
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
35/7		0	1240	1055	835	750	580	470	0.527
		1	1420	1230	995	905	705	575	0.404
$\alpha_1=\alpha_2$=	2.143	2	1560	1380	1140	1040	820	675	0.328
$\alpha_p^2 = \alpha_e^2$=	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
35/5		0	915	790	635	575	445	365	0.719
		1	1055	940	780	715	565	465	0.508
$\alpha_1=\alpha_2$=	1.571	2	1155	1050	895	830	670	560	0.393
$\alpha_p^2 = \alpha_e^2$=	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
30/7		0	1215	1015	785	705	540	430	0.659
		1	1440	1220	965	870	670	540	0.497
$\alpha_1=\alpha_2$=	2.000	2	1620	1405	1130	1025	795	650	0.399
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
30/5		0	930	785	620	555	430	345	0.878
		1	1110	965	780	710	555	450	0.613
$\alpha_1=\alpha_2$=	1.500	2	1245	1110	920	845	670	550	0.470
$\alpha_p^2 = \alpha_e^2$=	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
30/4		0	780	670	535	485	375	305	0.988
		1	925	820	680	625	495	405	0.664
$\alpha_1=\alpha_2$=	1.333	2	1020	930	795	740	600	500	0.500
$\alpha_p^2 = \alpha_e^2$=	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0179 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	33	ksi
$F_{u\text{-framing}}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
	0	790	665	520	465	355	285	0.385
35/8	1	890	760	600	545	420	340	0.300
α_1=α_2= 2.286	2	980	845	680	615	480	390	0.245
$\alpha_p^2 = \alpha_e^2$= 0.857	3	1050	920	750	685	535	440	0.208
N= 2.400	4	1115	985	815	745	590	485	0.180
A= 1	5	1165	1045	875	805	645	530	0.159
	6	1210	1095	930	860	695	575	0.142
	0	750	640	505	455	350	280	0.410
35/7	1	840	725	580	530	410	335	0.315
α_1=α_2= 2.143	2	910	800	650	595	465	380	0.255
$\alpha_p^2 = \alpha_e^2$= 0.847	3	970	865	715	655	520	430	0.215
N= 2.057	4	1020	920	775	715	570	475	0.185
A= 1	5	1060	965	825	765	620	515	0.163
	6	1095	1005	870	815	665	560	0.146
	0	555	480	385	350	270	220	0.560
35/5	1	625	550	455	415	325	265	0.396
α_1=α_2= 1.571	2	675	610	515	475	380	315	0.306
$\alpha_p^2 = \alpha_e^2$= 0.663	3	715	655	565	525	425	355	0.250
N= 1.371	4	745	690	605	570	470	395	0.211
A= 1	5	770	720	640	605	510	435	0.182
	6	785	745	675	640	545	470	0.161
	0	735	615	475	430	325	260	0.513
30/7	1	845	715	560	505	390	310	0.387
α_1=α_2= 2.000	2	935	805	640	580	450	365	0.311
$\alpha_p^2 = \alpha_e^2$= 0.778	3	1015	885	715	650	505	415	0.260
N= 2.400	4	1085	955	785	715	565	460	0.223
A= 1	5	1140	1015	845	775	615	510	0.195
	6	1190	1070	905	835	670	555	0.174
	0	565	475	375	335	260	205	0.684
30/5	1	650	565	450	410	320	260	0.477
α_1=α_2= 1.500	2	720	635	520	475	375	305	0.366
$\alpha_p^2 = \alpha_e^2$= 0.625	3	775	695	585	535	430	355	0.297
N= 1.600	4	820	745	635	590	475	395	0.250
A= 1	5	855	785	685	635	525	440	0.216
	6	880	820	725	680	565	480	0.190
	0	470	405	325	295	225	180	0.770
30/4	1	545	480	395	360	285	230	0.517
α_1=α_2= 1.333	2	595	540	455	420	335	280	0.390
$\alpha_p^2 = \alpha_e^2$= 0.556	3	635	580	505	470	385	320	0.313
N= 1.200	4	660	615	545	510	425	360	0.261
A= 1	5	680	645	575	545	460	395	0.224
	6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck			
Design thickness = 0.0239 in.			
$F_{y\text{-}deck}$=	60	ksi	
$F_{u\text{-}deck}$=	62	ksi	
Framing designation = 97 mils			
Framing thickness = 0.1017 in.			
$F_{y\text{-}framing}$=	33	ksi	
$F_{u\text{-}framing}$=	45	ksi	
Support fastening: #12 screws			
Side-lap fastening: #10 screws			

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1060	890	695	625	480	385	0.445
35/8		1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2$=	2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1445	1275	1045	955	750	615	0.240
N=	2.400	4	1535	1370	1140	1045	835	685	0.208
A=	1	5	1605	1450	1225	1135	910	755	0.184
		6	1665	1520	1305	1210	985	820	0.164
		0	1005	855	675	610	470	380	0.474
35/7		1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2$=	2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2 = \alpha_e^2$=	0.847	3	1330	1190	995	915	730	600	0.248
N=	2.057	4	1395	1265	1080	1000	805	670	0.214
A=	1	5	1450	1335	1150	1070	875	735	0.188
		6	1495	1390	1215	1140	945	795	0.168
		0	740	640	515	465	360	295	0.647
35/5		1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2$=	1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2 = \alpha_e^2$=	0.663	3	975	900	780	730	600	500	0.289
N=	1.371	4	1015	950	840	790	660	560	0.244
A=	1	5	1045	990	890	845	715	615	0.211
		6	1070	1020	930	885	765	665	0.186
		0	985	820	635	570	435	345	0.593
30/7		1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2$=	2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1400	1225	1000	910	715	585	0.300
N=	2.400	4	1495	1330	1100	1010	800	655	0.258
A=	1	5	1575	1415	1190	1095	880	725	0.226
		6	1640	1490	1270	1180	955	795	0.201
		0	750	635	500	450	345	275	0.791
30/5		1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2$=	1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2 = \alpha_e^2$=	0.625	3	1065	965	815	750	605	500	0.344
N=	1.600	4	1125	1030	890	830	675	565	0.289
A=	1	5	1170	1085	955	895	740	625	0.250
		6	1205	1130	1010	950	800	685	0.220
		0	630	545	435	390	305	245	0.890
30/4		1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2$=	1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2 = \alpha_e^2$=	0.556	3	865	800	700	655	540	455	0.361
N=	1.200	4	900	845	755	710	600	510	0.301
A=	1	5	930	880	800	760	650	565	0.259
		6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0295 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-}framing}$=	33	ksi
$F_{u\text{-}framing}$=	45	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1305	1100	860	770	590	475	0.494
35/8		1	1515	1295	1030	930	720	585	0.385
α_1=α_2=	2.286	2	1685	1465	1190	1080	845	690	0.315
α_p^2 = α_e^2=	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
		0	1240	1055	835	750	580	470	0.527
35/7		1	1420	1230	995	905	705	575	0.404
α_1=α_2=	2.143	2	1560	1380	1140	1040	820	675	0.328
α_p^2 = α_e^2=	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
		0	915	790	635	575	445	365	0.719
35/5		1	1055	940	780	715	565	465	0.508
α_1=α_2=	1.571	2	1155	1050	895	830	670	560	0.393
α_p^2 = α_e^2=	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
		0	1215	1015	785	705	540	430	0.659
30/7		1	1440	1220	965	870	670	540	0.497
α_1=α_2=	2.000	2	1620	1405	1130	1025	795	650	0.399
α_p^2 = α_e^2=	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
		0	930	785	620	555	430	345	0.878
30/5		1	1110	965	780	710	555	450	0.613
α_1=α_2=	1.500	2	1245	1110	920	845	670	550	0.470
α_p^2 = α_e^2=	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
		0	780	670	535	485	375	305	0.988
30/4		1	925	820	680	625	495	405	0.664
α_1=α_2=	1.333	2	1020	930	795	740	600	500	0.500
α_p^2 = α_e^2=	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

Deck F_y=	60	ksi
Deck F_{u1}=	62	ksi
Framing F_{y2}=	50	ksi
Framing F_{u2}=	65	ksi

P_{nf}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
26			647	647	647	647
24			864	864	864	864
22			1067	1067	1067	1067

P_{ns}, lbs	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22			272	272	272	272
20			419	419	419	419
18			575	575	575	575

S_f, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22			0.0097	0.0097	0.0097	0.0097
20			0.0084	0.0084	0.0084	0.0084
18			0.0076	0.0076	0.0076	0.0076

S_s, in/kip	Framing, mils					
Deck Ga.	27	33	43	54	68	97
22			0.0224	0.0224	0.0224	0.0224
20			0.0194	0.0194	0.0194	0.0194
18			0.0175	0.0175	0.0175	0.0175

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0179 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
	0	790	665	520	465	355	285	0.385
35/8	1	890	760	600	545	420	340	0.300
α_1=α_2= 2.286	2	980	845	680	615	480	390	0.245
α_p^2 = α_e^2= 0.857	3	1050	920	750	685	535	440	0.208
N= 2.400	4	1115	985	815	745	590	485	0.180
A= 1	5	1165	1045	875	805	645	530	0.159
	6	1210	1095	930	860	695	575	0.142
	0	750	640	505	455	350	280	0.410
35/7	1	840	725	580	530	410	335	0.315
α_1=α_2= 2.143	2	910	800	650	595	465	380	0.255
α_p^2 = α_e^2= 0.847	3	970	865	715	655	520	430	0.215
N= 2.057	4	1020	920	775	715	570	475	0.185
A= 1	5	1060	965	825	765	620	515	0.163
	6	1095	1005	870	815	665	560	0.146
	0	555	480	385	350	270	220	0.560
35/5	1	625	550	455	415	325	265	0.396
α_1=α_2= 1.571	2	675	610	515	475	380	315	0.306
α_p^2 = α_e^2= 0.663	3	715	655	565	525	425	355	0.250
N= 1.371	4	745	690	605	570	470	395	0.211
A= 1	5	770	720	640	605	510	435	0.182
	6	785	745	675	640	545	470	0.161
	0	735	615	475	430	325	260	0.513
30/7	1	845	715	560	505	390	310	0.387
α_1=α_2= 2.000	2	935	805	640	580	450	365	0.311
α_p^2 = α_e^2= 0.778	3	1015	885	715	650	505	415	0.260
N= 2.400	4	1085	955	785	715	565	460	0.223
A= 1	5	1140	1015	845	775	615	510	0.195
	6	1190	1070	905	835	670	555	0.174
	0	565	475	375	335	260	205	0.684
30/5	1	650	565	450	410	320	260	0.477
α_1=α_2= 1.500	2	720	635	520	475	375	305	0.366
α_p^2 = α_e^2= 0.625	3	775	695	585	535	430	355	0.297
N= 1.600	4	820	745	635	590	475	395	0.250
A= 1	5	855	785	685	635	525	440	0.216
	6	880	820	725	680	565	480	0.190
	0	470	405	325	295	225	180	0.770
30/4	1	545	480	395	360	285	230	0.517
α_1=α_2= 1.333	2	595	540	455	420	335	280	0.390
α_p^2 = α_e^2= 0.556	3	635	580	505	470	385	320	0.313
N= 1.200	4	660	615	545	510	425	360	0.261
A= 1	5	680	645	575	545	460	395	0.224
	6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0239 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 43 mils		
Framing thickness = 0.0451 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
	0	1060	890	695	625	480	385	0.445
35/8	1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2=$ 2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2 = \alpha_e^2=$ 0.857	3	1445	1275	1045	955	750	615	0.240
N= 2.400	4	1535	1370	1140	1045	835	685	0.208
A= 1	5	1605	1450	1225	1135	910	755	0.184
	6	1665	1520	1305	1210	985	820	0.164
	0	1005	855	675	610	470	380	0.474
35/7	1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2=$ 2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2 = \alpha_e^2=$ 0.847	3	1330	1190	995	915	730	600	0.248
N= 2.057	4	1395	1265	1080	1000	805	670	0.214
A= 1	5	1450	1335	1150	1070	875	735	0.188
	6	1495	1390	1215	1140	945	795	0.168
	0	740	640	515	465	360	295	0.647
35/5	1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2=$ 1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2 = \alpha_e^2=$ 0.663	3	975	900	780	730	600	500	0.289
N= 1.371	4	1015	950	840	790	660	560	0.244
A= 1	5	1045	990	890	845	715	615	0.211
	6	1070	1020	930	885	765	665	0.186
	0	985	820	635	570	435	345	0.593
30/7	1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2=$ 2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2 = \alpha_e^2=$ 0.778	3	1400	1225	1000	910	715	585	0.300
N= 2.400	4	1495	1330	1100	1010	800	655	0.258
A= 1	5	1575	1415	1190	1095	880	725	0.226
	6	1640	1490	1270	1180	955	795	0.201
	0	750	635	500	450	345	275	0.791
30/5	1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2=$ 1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2 = \alpha_e^2=$ 0.625	3	1065	965	815	750	605	500	0.344
N= 1.600	4	1125	1030	890	830	675	565	0.289
A= 1	5	1170	1085	955	895	740	625	0.250
	6	1205	1130	1010	950	800	685	0.220
	0	630	545	435	390	305	245	0.890
30/4	1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2=$ 1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2 = \alpha_e^2=$ 0.556	3	865	800	700	655	540	455	0.361
N= 1.200	4	900	845	755	710	600	510	0.301
A= 1	5	930	880	800	760	650	565	0.259
	6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0295 in.	
$F_{y\text{-deck}}$=	60 ksi
$F_{u\text{-deck}}$=	62 ksi
Framing designation = 43 mils Framing thickness = 0.0451 in.	
$F_{y\text{-framing}}$=	50 ksi
$F_{u\text{-framing}}$=	65 ksi
Support fastening: #12 screws Side-lap fastening: #10 screws	

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1305	1100	860	770	590	475	0.494
35/8		1	1515	1295	1030	930	720	585	0.385
$\alpha_1=\alpha_2=$	2.286	2	1685	1465	1190	1080	845	690	0.315
$\alpha_p^2=\alpha_e^2=$	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
		0	1240	1055	835	750	580	470	0.527
35/7		1	1420	1230	995	905	705	575	0.404
$\alpha_1=\alpha_2=$	2.143	2	1560	1380	1140	1040	820	675	0.328
$\alpha_p^2=\alpha_e^2=$	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
		0	915	790	635	575	445	365	0.719
35/5		1	1055	940	780	715	565	465	0.508
$\alpha_1=\alpha_2=$	1.571	2	1155	1050	895	830	670	560	0.393
$\alpha_p^2=\alpha_e^2=$	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
		0	1215	1015	785	705	540	430	0.659
30/7		1	1440	1220	965	870	670	540	0.497
$\alpha_1=\alpha_2=$	2.000	2	1620	1405	1130	1025	795	650	0.399
$\alpha_p^2=\alpha_e^2=$	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
		0	930	785	620	555	430	345	0.878
30/5		1	1110	965	780	710	555	450	0.613
$\alpha_1=\alpha_2=$	1.500	2	1245	1110	920	845	670	550	0.470
$\alpha_p^2=\alpha_e^2=$	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
		0	780	670	535	485	375	305	0.988
30/4		1	925	820	680	625	495	405	0.664
$\alpha_1=\alpha_2=$	1.333	2	1020	930	795	740	600	500	0.500
$\alpha_p^2=\alpha_e^2=$	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0179 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 54 mils		
Framing thickness = 0.0566 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
35/8		0	790	665	520	465	355	285	0.385
		1	890	760	600	545	420	340	0.300
α_1=α_2=	2.286	2	980	845	680	615	480	390	0.245
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1050	920	750	685	535	440	0.208
N=	2.400	4	1115	985	815	745	590	485	0.180
A=	1	5	1165	1045	875	805	645	530	0.159
		6	1210	1095	930	860	695	575	0.142
35/7		0	750	640	505	455	350	280	0.410
		1	840	725	580	530	410	335	0.315
α_1=α_2=	2.143	2	910	800	650	595	465	380	0.255
$\alpha_p^2 = \alpha_e^2$=	0.847	3	970	865	715	655	520	430	0.215
N=	2.057	4	1020	920	775	715	570	475	0.185
A=	1	5	1060	965	825	765	620	515	0.163
		6	1095	1005	870	815	665	560	0.146
35/5		0	555	480	385	350	270	220	0.560
		1	625	550	455	415	325	265	0.396
α_1=α_2=	1.571	2	675	610	515	475	380	315	0.306
$\alpha_p^2 = \alpha_e^2$=	0.663	3	715	655	565	525	425	355	0.250
N=	1.371	4	745	690	605	570	470	395	0.211
A=	1	5	770	720	640	605	510	435	0.182
		6	785	745	675	640	545	470	0.161
30/7		0	735	615	475	430	325	260	0.513
		1	845	715	560	505	390	310	0.387
α_1=α_2=	2.000	2	935	805	640	580	450	365	0.311
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1015	885	715	650	505	415	0.260
N=	2.400	4	1085	955	785	715	565	460	0.223
A=	1	5	1140	1015	845	775	615	510	0.195
		6	1190	1070	905	835	670	555	0.174
30/5		0	565	475	375	335	260	205	0.684
		1	650	565	450	410	320	260	0.477
α_1=α_2=	1.500	2	720	635	520	475	375	305	0.366
$\alpha_p^2 = \alpha_e^2$=	0.625	3	775	695	585	535	430	355	0.297
N=	1.600	4	820	745	635	590	475	395	0.250
A=	1	5	855	785	685	635	525	440	0.216
		6	880	820	725	680	565	480	0.190
30/4		0	470	405	325	295	225	180	0.770
		1	545	480	395	360	285	230	0.517
α_1=α_2=	1.333	2	595	540	455	420	335	280	0.390
$\alpha_p^2 = \alpha_e^2$=	0.556	3	635	580	505	470	385	320	0.313
N=	1.200	4	660	615	545	510	425	360	0.261
A=	1	5	680	645	575	545	460	395	0.224
		6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in⁴/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0239 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 54 mils Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout	Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
35/8	0	1060	890	695	625	480	385	0.445
	1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2=$ 2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2=\alpha_e^2=$ 0.857	3	1445	1275	1045	955	750	615	0.240
N= 2.400	4	1535	1370	1140	1045	835	685	0.208
A= 1	5	1605	1450	1225	1135	910	755	0.184
	6	1665	1520	1305	1210	985	820	0.164
35/7	0	1005	855	675	610	470	380	0.474
	1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2=$ 2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2=\alpha_e^2=$ 0.847	3	1330	1190	995	915	730	600	0.248
N= 2.057	4	1395	1265	1080	1000	805	670	0.214
A= 1	5	1450	1335	1150	1070	875	735	0.188
	6	1495	1390	1215	1140	945	795	0.168
35/5	0	740	640	515	465	360	295	0.647
	1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2=$ 1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2=\alpha_e^2=$ 0.663	3	975	900	780	730	600	500	0.289
N= 1.371	4	1015	950	840	790	660	560	0.244
A= 1	5	1045	990	890	845	715	615	0.211
	6	1070	1020	930	885	765	665	0.186
30/7	0	985	820	635	570	435	345	0.593
	1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2=$ 2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2=\alpha_e^2=$ 0.778	3	1400	1225	1000	910	715	585	0.300
N= 2.400	4	1495	1330	1100	1010	800	655	0.258
A= 1	5	1575	1415	1190	1095	880	725	0.226
	6	1640	1490	1270	1180	955	795	0.201
30/5	0	750	635	500	450	345	275	0.791
	1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2=$ 1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2=\alpha_e^2=$ 0.625	3	1065	965	815	750	605	500	0.344
N= 1.600	4	1125	1030	890	830	675	565	0.289
A= 1	5	1170	1085	955	895	740	625	0.250
	6	1205	1130	1010	950	800	685	0.220
30/4	0	630	545	435	390	305	245	0.890
	1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2=$ 1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2=\alpha_e^2=$ 0.556	3	865	800	700	655	540	455	0.361
N= 1.200	4	900	845	755	710	600	510	0.301
A= 1	5	930	880	800	760	650	565	0.259
	6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 54 mils Framing thickness = 0.0566 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1305	1100	860	770	590	475	0.494
35/8		1	1515	1295	1030	930	720	585	0.385
$\alpha_1=\alpha_2$=	2.286	2	1685	1465	1190	1080	845	690	0.315
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
		0	1240	1055	835	750	580	470	0.527
35/7		1	1420	1230	995	905	705	575	0.404
$\alpha_1=\alpha_2$=	2.143	2	1560	1380	1140	1040	820	675	0.328
$\alpha_p^2 = \alpha_e^2$=	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
		0	915	790	635	575	445	365	0.719
35/5		1	1055	940	780	715	565	465	0.508
$\alpha_1=\alpha_2$=	1.571	2	1155	1050	895	830	670	560	0.393
$\alpha_p^2 = \alpha_e^2$=	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
		0	1215	1015	785	705	540	430	0.659
30/7		1	1440	1220	965	870	670	540	0.497
$\alpha_1=\alpha_2$=	2.000	2	1620	1405	1130	1025	795	650	0.399
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
		0	930	785	620	555	430	345	0.878
30/5		1	1110	965	780	710	555	450	0.613
$\alpha_1=\alpha_2$=	1.500	2	1245	1110	920	845	670	550	0.470
$\alpha_p^2 = \alpha_e^2$=	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
		0	780	670	535	485	375	305	0.988
30/4		1	925	820	680	625	495	405	0.664
$\alpha_1=\alpha_2$=	1.333	2	1020	930	795	740	600	500	0.500
$\alpha_p^2 = \alpha_e^2$=	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0179 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	790	665	520	465	355	285	0.385
35/8		1	890	760	600	545	420	340	0.300
α_1=α_2=	2.286	2	980	845	680	615	480	390	0.245
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1050	920	750	685	535	440	0.208
N=	2.400	4	1115	985	815	745	590	485	0.180
A=	1	5	1165	1045	875	805	645	530	0.159
		6	1210	1095	930	860	695	575	0.142
		0	750	640	505	455	350	280	0.410
35/7		1	840	725	580	530	410	335	0.315
α_1=α_2=	2.143	2	910	800	650	595	465	380	0.255
$\alpha_p^2 = \alpha_e^2$=	0.847	3	970	865	715	655	520	430	0.215
N=	2.057	4	1020	920	775	715	570	475	0.185
A=	1	5	1060	965	825	765	620	515	0.163
		6	1095	1005	870	815	665	560	0.146
		0	555	480	385	350	270	220	0.560
35/5		1	625	550	455	415	325	265	0.396
α_1=α_2=	1.571	2	675	610	515	475	380	315	0.306
$\alpha_p^2 = \alpha_e^2$=	0.663	3	715	655	565	525	425	355	0.250
N=	1.371	4	745	690	605	570	470	395	0.211
A=	1	5	770	720	640	605	510	435	0.182
		6	785	745	675	640	545	470	0.161
		0	735	615	475	430	325	260	0.513
30/7		1	845	715	560	505	390	310	0.387
α_1=α_2=	2.000	2	935	805	640	580	450	365	0.311
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1015	885	715	650	505	415	0.260
N=	2.400	4	1085	955	785	715	565	460	0.223
A=	1	5	1140	1015	845	775	615	510	0.195
		6	1190	1070	905	835	670	555	0.174
		0	565	475	375	335	260	205	0.684
30/5		1	650	565	450	410	320	260	0.477
α_1=α_2=	1.500	2	720	635	520	475	375	305	0.366
$\alpha_p^2 = \alpha_e^2$=	0.625	3	775	695	585	535	430	355	0.297
N=	1.600	4	820	745	635	590	475	395	0.250
A=	1	5	855	785	685	635	525	440	0.216
		6	880	820	725	680	565	480	0.190
		0	470	405	325	295	225	180	0.770
30/4		1	545	480	395	360	285	230	0.517
α_1=α_2=	1.333	2	595	540	455	420	335	280	0.390
$\alpha_p^2 = \alpha_e^2$=	0.556	3	635	580	505	470	385	320	0.313
N=	1.200	4	660	615	545	510	425	360	0.261
A=	1	5	680	645	575	545	460	395	0.224
		6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0239 in.		
$F_{y\text{-}deck}$=	60	ksi
$F_{u\text{-}deck}$=	62	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-}framing}$=	50	ksi
$F_{u\text{-}framing}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
35/8		0	1060	890	695	625	480	385	0.445
		1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2$=	2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2=\alpha_e^2$=	0.857	3	1445	1275	1045	955	750	615	0.240
N=	2.400	4	1535	1370	1140	1045	835	685	0.208
A=	1	5	1605	1450	1225	1135	910	755	0.184
		6	1665	1520	1305	1210	985	820	0.164
35/7		0	1005	855	675	610	470	380	0.474
		1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2$=	2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2=\alpha_e^2$=	0.847	3	1330	1190	995	915	730	600	0.248
N=	2.057	4	1395	1265	1080	1000	805	670	0.214
A=	1	5	1450	1335	1150	1070	875	735	0.188
		6	1495	1390	1215	1140	945	795	0.168
35/5		0	740	640	515	465	360	295	0.647
		1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2$=	1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2=\alpha_e^2$=	0.663	3	975	900	780	730	600	500	0.289
N=	1.371	4	1015	950	840	790	660	560	0.244
A=	1	5	1045	990	890	845	715	615	0.211
		6	1070	1020	930	885	765	665	0.186
30/7		0	985	820	635	570	435	345	0.593
		1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2$=	2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2=\alpha_e^2$=	0.778	3	1400	1225	1000	910	715	585	0.300
N=	2.400	4	1495	1330	1100	1010	800	655	0.258
A=	1	5	1575	1415	1190	1095	880	725	0.226
		6	1640	1490	1270	1180	955	795	0.201
30/5		0	750	635	500	450	345	275	0.791
		1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2$=	1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2=\alpha_e^2$=	0.625	3	1065	965	815	750	605	500	0.344
N=	1.600	4	1125	1030	890	830	675	565	0.289
A=	1	5	1170	1085	955	895	740	625	0.250
		6	1205	1130	1010	950	800	685	0.220
30/4		0	630	545	435	390	305	245	0.890
		1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2$=	1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2=\alpha_e^2$=	0.556	3	865	800	700	655	540	455	0.361
N=	1.200	4	900	845	755	710	600	510	0.301
A=	1	5	930	880	800	760	650	565	0.259
		6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 68 mils		
Framing thickness = 0.0713 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1305	1100	860	770	590	475	0.494
35/8		1	1515	1295	1030	930	720	585	0.385
$\alpha_1=\alpha_2=$	2.286	2	1685	1465	1190	1080	845	690	0.315
$\alpha_p^2=\alpha_e^2=$	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
		0	1240	1055	835	750	580	470	0.527
35/7		1	1420	1230	995	905	705	575	0.404
$\alpha_1=\alpha_2=$	2.143	2	1560	1380	1140	1040	820	675	0.328
$\alpha_p^2=\alpha_e^2=$	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
		0	915	790	635	575	445	365	0.719
35/5		1	1055	940	780	715	565	465	0.508
$\alpha_1=\alpha_2=$	1.571	2	1155	1050	895	830	670	560	0.393
$\alpha_p^2=\alpha_e^2=$	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
		0	1215	1015	785	705	540	430	0.659
30/7		1	1440	1220	965	870	670	540	0.497
$\alpha_1=\alpha_2=$	2.000	2	1620	1405	1130	1025	795	650	0.399
$\alpha_p^2=\alpha_e^2=$	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
		0	930	785	620	555	430	345	0.878
30/5		1	1110	965	780	710	555	450	0.613
$\alpha_1=\alpha_2=$	1.500	2	1245	1110	920	845	670	550	0.470
$\alpha_p^2=\alpha_e^2=$	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
		0	780	670	535	485	375	305	0.988
30/4		1	925	820	680	625	495	405	0.664
$\alpha_1=\alpha_2=$	1.333	2	1020	930	795	740	600	500	0.500
$\alpha_p^2=\alpha_e^2=$	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

9/16" x 2 1/2" Form Deck			
Design thickness = 0.0179 in.			
$F_{y\text{-}deck}$=	60	ksi	
$F_{u\text{-}deck}$=	62	ksi	
Framing designation = 97 mils			
Framing thickness = 0.1017 in.			
$F_{y\text{-}framing}$=	50	ksi	
$F_{u\text{-}framing}$=	65	ksi	
Support fastening: #12 screws			
Side-lap fastening: #10 screws			

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	790	665	520	465	355	285	0.385
35/8		1	890	760	600	545	420	340	0.300
$\alpha_1=\alpha_2$=	2.286	2	980	845	680	615	480	390	0.245
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1050	920	750	685	535	440	0.208
N=	2.400	4	1115	985	815	745	590	485	0.180
A=	1	5	1165	1045	875	805	645	530	0.159
		6	1210	1095	930	860	695	575	0.142
		0	750	640	505	455	350	280	0.410
35/7		1	840	725	580	530	410	335	0.315
$\alpha_1=\alpha_2$=	2.143	2	910	800	650	595	465	380	0.255
$\alpha_p^2 = \alpha_e^2$=	0.847	3	970	865	715	655	520	430	0.215
N=	2.057	4	1020	920	775	715	570	475	0.185
A=	1	5	1060	965	825	765	620	515	0.163
		6	1095	1005	870	815	665	560	0.146
		0	555	480	385	350	270	220	0.560
35/5		1	625	550	455	415	325	265	0.396
$\alpha_1=\alpha_2$=	1.571	2	675	610	515	475	380	315	0.306
$\alpha_p^2 = \alpha_e^2$=	0.663	3	715	655	565	525	425	355	0.250
N=	1.371	4	745	690	605	570	470	395	0.211
A=	1	5	770	720	640	605	510	435	0.182
		6	785	745	675	640	545	470	0.161
		0	735	615	475	430	325	260	0.513
30/7		1	845	715	560	505	390	310	0.387
$\alpha_1=\alpha_2$=	2.000	2	935	805	640	580	450	365	0.311
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1015	885	715	650	505	415	0.260
N=	2.400	4	1085	955	785	715	565	460	0.223
A=	1	5	1140	1015	845	775	615	510	0.195
		6	1190	1070	905	835	670	555	0.174
		0	565	475	375	335	260	205	0.684
30/5		1	650	565	450	410	320	260	0.477
$\alpha_1=\alpha_2$=	1.500	2	720	635	520	475	375	305	0.366
$\alpha_p^2 = \alpha_e^2$=	0.625	3	775	695	585	535	430	355	0.297
N=	1.600	4	820	745	635	590	475	395	0.250
A=	1	5	855	785	685	635	525	440	0.216
		6	880	820	725	680	565	480	0.190
		0	470	405	325	295	225	180	0.770
30/4		1	545	480	395	360	285	230	0.517
$\alpha_1=\alpha_2$=	1.333	2	595	540	455	420	335	280	0.390
$\alpha_p^2 = \alpha_e^2$=	0.556	3	635	580	505	470	385	320	0.313
N=	1.200	4	660	615	545	510	425	360	0.261
A=	1	5	680	645	575	545	460	395	0.224
		6	700	665	605	575	495	430	0.196

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.013	5510	3525	1980	1565	880	560

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min {S_{nf} / Ω_{df}, S_{nb} / Ω_{db}}

LRFD Required strength (Factored Applied Load) <= Min {$\phi_{df}S_{nf}$, $\phi_{db}S_{nb}$}

9/16" x 2 1/2" Form Deck		
Design thickness = 0.0239 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 97 mils		
Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws		
Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1060	890	695	625	480	385	0.445
35/8		1	1210	1035	820	740	575	465	0.346
$\alpha_1=\alpha_2$=	2.286	2	1340	1160	940	850	665	540	0.283
$\alpha_p^2 = \alpha_e^2$=	0.857	3	1445	1275	1045	955	750	615	0.240
N=	2.400	4	1535	1370	1140	1045	835	685	0.208
A=	1	5	1605	1450	1225	1135	910	755	0.184
		6	1665	1520	1305	1210	985	820	0.164
		0	1005	855	675	610	470	380	0.474
35/7		1	1135	985	795	720	560	455	0.364
$\alpha_1=\alpha_2$=	2.143	2	1240	1095	900	820	645	530	0.295
$\alpha_p^2 = \alpha_e^2$=	0.847	3	1330	1190	995	915	730	600	0.248
N=	2.057	4	1395	1265	1080	1000	805	670	0.214
A=	1	5	1450	1335	1150	1070	875	735	0.188
		6	1495	1390	1215	1140	945	795	0.168
		0	740	640	515	465	360	295	0.647
35/5		1	845	750	620	565	450	365	0.458
$\alpha_1=\alpha_2$=	1.571	2	920	835	710	655	525	435	0.354
$\alpha_p^2 = \alpha_e^2$=	0.663	3	975	900	780	730	600	500	0.289
N=	1.371	4	1015	950	840	790	660	560	0.244
A=	1	5	1045	990	890	845	715	615	0.211
		6	1070	1020	930	885	765	665	0.186
		0	985	820	635	570	435	345	0.593
30/7		1	1150	975	770	690	530	430	0.447
$\alpha_1=\alpha_2$=	2.000	2	1285	1110	890	805	625	510	0.359
$\alpha_p^2 = \alpha_e^2$=	0.778	3	1400	1225	1000	910	715	585	0.300
N=	2.400	4	1495	1330	1100	1010	800	655	0.258
A=	1	5	1575	1415	1190	1095	880	725	0.226
		6	1640	1490	1270	1180	955	795	0.201
		0	750	635	500	450	345	275	0.791
30/5		1	885	770	620	565	440	355	0.552
$\alpha_1=\alpha_2$=	1.500	2	990	875	725	665	525	430	0.423
$\alpha_p^2 = \alpha_e^2$=	0.625	3	1065	965	815	750	605	500	0.344
N=	1.600	4	1125	1030	890	830	675	565	0.289
A=	1	5	1170	1085	955	895	740	625	0.250
		6	1205	1130	1010	950	800	685	0.220
		0	630	545	435	390	305	245	0.890
30/4		1	740	655	540	495	390	320	0.598
$\alpha_1=\alpha_2$=	1.333	2	815	740	630	580	470	390	0.450
$\alpha_p^2 = \alpha_e^2$=	0.556	3	865	800	700	655	540	455	0.361
N=	1.200	4	900	845	755	710	600	510	0.301
A=	1	5	930	880	800	760	650	565	0.259
		6	950	910	835	800	700	610	0.227

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.017	8370	5355	3010	2380	1335	855

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df} S_{nf}, \phi_{db} S_{nb}\}$

9/16" x 2 1/2" Form Deck Design thickness = 0.0295 in.		
$F_{y\text{-deck}}$=	60	ksi
$F_{u\text{-deck}}$=	62	ksi
Framing designation = 97 mils Framing thickness = 0.1017 in.		
$F_{y\text{-framing}}$=	50	ksi
$F_{u\text{-framing}}$=	65	ksi
Support fastening: #12 screws Side-lap fastening: #10 screws		

	S310-13		S310-16	
Loading	ϕ_{df}	Ω_{df}	ϕ_{df}	Ω_{df}
Seismic	0.65	2.50	0.70	2.30
Wind	0.70	2.35	0.80	2.00
Other	0.65	2.50	0.70	2.30

Fastener Layout		Side-lap Conn/Span	Nominal Shear Strength, S_{nf}, plf[1,2] Span 19.2"	24"	32"	36"	48"	60"	K_1 1/ft
		0	1305	1100	860	770	590	475	0.494
35/8		1	1515	1295	1030	930	720	585	0.385
α_1=α_2=	2.286	2	1685	1465	1190	1080	845	690	0.315
α_p^2 = α_e^2=	0.857	3	1820	1615	1330	1215	960	790	0.267
N=	2.400	4	1935	1735	1455	1340	1075	885	0.231
A=	1	5	2025	1840	1570	1455	1175	980	0.204
		6	2100	1930	1670	1555	1275	1065	0.183
		0	1240	1055	835	750	580	470	0.527
35/7		1	1420	1230	995	905	705	575	0.404
α_1=α_2=	2.143	2	1560	1380	1140	1040	820	675	0.328
α_p^2 = α_e^2=	0.847	3	1670	1500	1265	1165	935	770	0.276
N=	2.057	4	1755	1605	1375	1275	1035	865	0.238
A=	1	5	1825	1685	1470	1370	1130	950	0.209
		6	1880	1755	1550	1455	1215	1030	0.187
		0	915	790	635	575	445	365	0.719
35/5		1	1055	940	780	715	565	465	0.508
α_1=α_2=	1.571	2	1155	1050	895	830	670	560	0.393
α_p^2 = α_e^2=	0.663	3	1225	1135	990	925	765	645	0.321
N=	1.371	4	1275	1195	1070	1005	850	725	0.271
A=	1	5	1310	1245	1130	1075	920	795	0.234
		6	1335	1280	1180	1125	985	860	0.206
		0	1215	1015	785	705	540	430	0.659
30/7		1	1440	1220	965	870	670	540	0.497
α_1=α_2=	2.000	2	1620	1405	1130	1025	795	650	0.399
α_p^2 = α_e^2=	0.778	3	1770	1560	1275	1165	915	750	0.334
N=	2.400	4	1895	1690	1410	1295	1030	850	0.286
A=	1	5	1990	1800	1525	1410	1135	945	0.251
		6	2070	1895	1630	1515	1235	1035	0.223
		0	930	785	620	555	430	345	0.878
30/5		1	1110	965	780	710	555	450	0.613
α_1=α_2=	1.500	2	1245	1110	920	845	670	550	0.470
α_p^2 = α_e^2=	0.625	3	1345	1220	1040	960	775	645	0.382
N=	1.600	4	1415	1305	1135	1060	875	735	0.321
A=	1	5	1470	1375	1220	1145	960	815	0.277
		6	1510	1425	1285	1215	1035	890	0.244
		0	780	670	535	485	375	305	0.988
30/4		1	925	820	680	625	495	405	0.664
α_1=α_2=	1.333	2	1020	930	795	740	600	500	0.500
α_p^2 = α_e^2=	0.556	3	1085	1010	890	835	690	585	0.401
N=	1.200	4	1130	1065	960	910	770	660	0.335
A=	1	5	1160	1105	1015	965	840	730	0.287
		6	1185	1140	1055	1015	895	790	0.252

[1] Nominal shear strength shown above may be limited by shear buckling. See Table below.

	S310-13		S310-16	
Loading	ϕ_{db}	Ω_{db}	ϕ_{db}	Ω_{db}
Buckling	0.80	2.00	0.80	2.00

Deck Profile	I in^4/ft	Nominal Shear Due to Panel Buckling, S_{nb}, plf[2] Span 19.2"	24"	32"	36"	48"	60"
WR	0.021	11485	7350	4135	3265	1835	1175

[2] Design Strengths:

ASD Required strength (Service Applied Load) <= Min $\{S_{nf} / \Omega_{df}, S_{nb} / \Omega_{db}\}$

LRFD Required strength (Factored Applied Load) <= Min $\{\phi_{df}S_{nf}, \phi_{db}S_{nb}\}$

REFERENCES

SECTION 8

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REFERENCES

American Iron and Steel Institute (AISI)

AISI S100-13, North American Specification for the Design of Cold-Formed Steel Structural Members

AISI S100-16, North American Specification for the Design of Cold-Formed Steel Structural Members

AISI S202-15, Code of Standard Practice for Cold-Formed Steel Structural Framing

AISI S240-15, North American Standard for Cold-Formed Steel Structural Framing

AISI S310-13, North American Standard for the Design of Profiled Steel Diaphragm Panels

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

Steel Deck Institute (SDI)

ANSI/SDI QA/QC-2017, Standard for Quality Control and Quality Assurance for Installation of Steel Deck

ANSI/SDI NC-2017, Standard for Noncomposite Steel Floor Deck

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

ANSI/SDI C-2017, Standard for Composite Steel Floor Deck-Slabs

Diaphragm Design Manual, 4th Edition, 2015

SDI Floor Deck Design Manual

SDI Roof Deck Design Manual

SDI COSP-2017, Code of Standard Practice

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

Useful General Guides to Cold-Formed Framing Design Related to Deck on Framing

American Iron and Steel Institute (AISI)

AISI S240-15, North American Standard for Cold-Formed Steel Structural Framing

AISI D110-16 - Cold-Formed Steel Framing Design Guide

AISI D112-13: Brick Veneer Cold-Formed Steel Framing Design Guide

Cold Formed Steel Engineers Institute

Cold Formed Steel Truss to Bearing Connections; CFSEI Tech Note F501-11

Chase the Loads: Load Path Considerations for Cold-Formed Steel Light Frame Construction; CFSEI Tech Note G200-15

Roof Framing Anchorage Forces: MWFRS or C&C; CFSEI Tech Note L200-09

Design Guide: Cold-Formed Steel Framed Wood Panel or Steel Sheet Sheathed Shear Wall Assemblies, 2009

North American Steel Framing Alliance

Low-Rise Residential Construction Details; Publication NT6-00, 2000

(Available through CFSEI)

CONVERSIONS

US Customary to SI Unit Conversion Table

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

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