Flexible Metal Deck Roof Diaphragms

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Shear diaphragms are essentially planar structural systems found in roofs, floors, and walls of buildings. They are comprised of interconnected units, attached to supporting members, such that the entire assembly possesses both in-plane shear strength and stiffness. The major components of a diaphragm include the individual deck panels, the structural members to which they are connected and the connecting devices or fasteners. Fastener types include welds, screws, power driven pins, or other mechanical devices that have a predictable capacity. The strength and stiffness of a diaphragm depends on the panel properties, the span arrangements, and the quality of the connections.

Figure 1 illustrates the basic concept for a roof and wall diaphragm. In Figure 1 wind pressure on the walls parallel to length L act on the building. A line load reaction of this pressure is distributed to the top of the wall at the roof plane via the vertical spanning capacity of the cladding over height H. The roof, acting as a simple span horizontal girder of depth B, distributes the wind line load as a shear reaction along the top of the end walls perpendicular to L. The end walls in turn act as diaphragms between the roof and foundation levels.
For the roof in this example, the total uniform load on the diaphragm equals the combined effects of the windward and leeward cladding reactions (\(w_w + w_L\)). The maximum end reaction (R) of the roof diaphragm therefore is \(\left(\left(w_w + w_L\right) \times L\right)/2\). R divided by length B equals the collector load or maximum uniform shear force in the diaphragm. The maximum roof diaphragm moment (M) equals \(\left(\left(w_w + w_L\right) \times L^2\right)/8\). The corresponding diaphragm chord forces are therefore equal to M divided by the diaphragm depth B. The maximum unit shear on the wall diaphragm is also equal to the roof reaction, R, divided by B. However, the wall diaphragm moment equals R \times H.

It should also be noted that:

a. A diaphragm acts like a short deep beam.
b. The maximum average shear (R/B) occurs at the ends of the roof diaphragm.
c. Zones near the mid-span of the roof are subjected to less unit shear therefore less diaphragm strength is required.
d. The larger design shears may be resisted by using both heavier gage panels and fewer connections or by more frequently connected lighter gage panels. Therefore the most efficient use of materials may not be met by using a single diaphragm design for the entire roof area. However, from a practical constructability standpoint it is more common to use a single deck and connector type over an entire roof and increase the frequency of the connectors in order to resist greater in plane shear forces.

Roof diaphragms may be assembled from a wide variety of panel profiles including Narrow Rib (NR), Intermediate Rib (IR) and Wide Rib (WR). Each of these different deck types are described in the Steel Deck Institute (SDI) Publication No. 31, Design Manual for Composite Decks, Form Decks and Roof Decks. Roof diaphragm panels typically vary from 0.0295 inches to 0.064 inches in thickness and come in widths of 18 inches to 36 inches or more.
A diaphragm should be designed and assembled to cover a specified area in such a way that in-plane shear strength and stiffness is predictable. The principal elements determining these factors include spacing of the support framing, the size and thickness of the individual deck panels and the interconnecting fasteners. A metal roof deck diaphragm is a fairly flexible system somewhat analogous to a truss as shown in Figure 2. Prior to installing the truss diagonal or attaching the diaphragm to the supports (and each adjacent sheet via sidelap connectors), neither of the frames possess much strength or stiffness. The shear strength of a diaphragm system is limited by the strength of connections, local panel buckling and the general plate-like buckling characteristics of the entire diaphragm area.

In addition to needing to resist planar shear forces, in order for a diaphragm to function properly it is also necessary to provide members along the extreme edges of the deck as required to resist the flexural tension and compression components of the deep beam member. This is analogous to the top and bottom chords of a truss resisting the compression and tension forces of the member.
The strength of a diaphragm can also be limited by proper attention to the edge and end termination conditions. In addition, at interior positions, panels must be sufficiently overlapped to provide adequate end distances for the connector used. If panels are butted at their ends rather than end lapped, as is common with floor decks, then each panel must be individually connected at its ends with the specified pattern.

The overlapping edges of panels should be in close contact in order to minimize the eccentricity on fasteners at the lap. This is also a critical factor at sidelap conditions. In addition, fasteners used at the overlap condition should also be installed on the edge panel at the diaphragm perimeter where the sidelap connector would normally occur if there were another adjacent panel. Otherwise the shear strength along the first interior sidelap may exceed that along the perimeter member thus diminishing the contribution of the deck end connection at the supporting shearwall or vertical X-braced frame.

![Lap connection](source:Tata Steel)

**Fasteners**

Establishing what type of fastener is to be used is a critical component of designing and detailing a diaphragm. Fasteners commonly include welds, screws and powder driven nail-like pins. Structural connections include fasteners that connect one or more sheets to heavier or thicker frame or structural members. Sidelap connections include fasteners that connect adjacent panels to each other and not to the supporting frame members. Stitch connections are another name for sidelap connections.

![Fasteners](source:Hilti; Metal Deck Supply; Hobart Brothers)
**Welded Connections:**

Arc-spot welds, or puddle welds, are produced by striking an arc on the upper sheet, thereby causing a hole to form, while the lower sheet is being raised to fusion temperature. With the attainment of the proper temperature, the electrode is moved in a circular pattern until the hole is filled and fusion attained on the arc-puddle perimeter. The relative strength in a series of welds can vary significantly by modest changes in welding times.

Arc-spot welds to structural members (i.e. welding of thinner sheets to thicker structural members) require direct contact between the units for proper heat transfer. In addition, a proper balance between the welding time and the electrode burn-off rate is essential to good quality welding. Welding machine power settings required usually are well below those needed for welding in hot-rolled steels. The time required per weld may vary between 3 and 6 seconds or more depending of the properties of parts being connected.

Weld washers function as a heat sink allowing hole formation in thinner panels without excessive growth of the hole, as the temperature of the underlying panel is increased. The washer is subsequently filled with the weld stem growing into the lower panel which becomes anchored on the washer’s hole perimeter. Upon cooling, the washer is clamped down on the attached sheet. Weld washers are recommended for panels thinner than 0.0280-inches.

*Source: Palm Beach County Schools*

*Source: Metal Deck Supply*
All welds should be made by qualified operators following AWS D1.3 Specifications. As indicated previously, welding thin material usually requires a much lower power setting and lower burn-off rate than with heavy steel units. Particular care is also required when welding deck to joists in order to avoid damage to the joist chords. Preliminary field quality control checks can be made by placing a pair of welds in adjacent valleys at one end of a panel. A visual inspection will show if the weld material is fused properly and in contact with the underlying panel. Intermittent contact may indicate excessive power settings. Separation between the panels may indicate insufficient welding time and poor fusion. The termination of the welding operation may not permit complete fusion around the entire perimeter. Weld fusion should be visible over no less than three-quarters of the weld perimeter.

In adjacent diaphragm panels with nestable or flat overlapping edges, sheet-to-sheet or stitch connections are typically required away from the supporting members. The placement of arc spot welds at such sidelaps is difficult. In addition, the thinner the material, the more difficult the welding operation becomes. Welding of sidelaps is not recommended for material of 0.0295-inch or thinner. The amount of slip or movement experienced as welds are loaded in shear in thin steel elements is very small relative to that for most other mechanical connectors. The movement is essentially limited to panel distortion around the weld.

**Screw Connections:**

Screws must be installed using properly calibrated tools in order to avoid overdriving which can strip the threads at side laps or sever the screw when it is placed into a thicker supporting member. The quality of screw connectors in general is typically not a major problem although these types of connectors can sometimes fracture when being driven into a thicker supporting material. The most common problem with screw connectors involves allowing the screw to thread-up on the upper sheet before becoming engaged in the lower sheet thus leaving major gaps between the separate panels. Such screws should be removed and redriven nearby while forcing sheet layers to remain in direct contact.
**Powder-Driven Fasteners:**

Powder driven fasteners must be installed following the manufacturer’s recommendations. Care must be exercised in setting the driving force to obtain the proper depth of penetration. Once driven properly, these nail-like fasteners are very resistant to extraction by uplift forces. In uplift tests on sheet material, the usual mode of failure involves tearing the sheet around the head or washer leaving the fastener in place.

![Image of powder-driven fasteners](source: Hilti)

**Button-Punched Sidelaps:**

The quality of button punching is difficult to maintain because the attachment depends on the care and the energy used by the installer and the tool that is used.

![Image of button-punched fasteners](source: Metaldeck.com)
Miscellaneous:

1. Strength vs. Cost: It is typically better to achieve the required diaphragm strength with a greater number of weaker but less expensive fasteners. However it is recommended that the designer should check the deflection of the diaphragm.

2. The insurance fire rating or wind uplift requirements of a building may not allow a substitution of fasteners or change in spacing as governed by design.

3. The common availability of any given deck attachment tool, equipment or power source may dictate the choice of fastener for any given project.

4. The exposed underside appearance of the deck may be important. If appearance is critical sidelay screws or even very weak button punches may be required even though an increased number of these connectors would be needed over that provided by welding.

5. At this time there are no standard tests for uplift loadings on fasteners. However, uplift failure of a deck panel under service loading is very uncommon. This is because the spacing of fasteners as dictated by diaphragm design or insurance requirements (Factory Mutual or Underwriters Laboratories) typically exceeds any net uplift forces.

How to Fasten Steel Deck

Fastening of metal deck to the supports at one time was only done by welding until self-drilling screws became available. In addition, welding of deck has also been replaced to some degree by fasteners that are shot in (either air or powder actuated). All three of these methods of attachment perform the same function of holding the deck in place and thus allowing shear strength and stiffness (that can be predicted) to be developed.

Because the connection of the deck (both side laps and to the supports) is a very important component of the diaphragm capacity, for both structural stability and insurance and fire rating approvals, it is up to the designer to choose the correct fasteners to be specified and be able to judge the merits of substitutions requested by the contractor.
Welded Connections:

Welding, when properly executed, provides the strongest and stiffest deck connections. Welding also requires the most skill and therefore should be inspected thoroughly. Because most visible weld diameters used with steel deck are between ½-inches and ¾-inches neither AISI nor AWS capacity formulas should be used. However, testing performed by the SDI has established strengths for weld diameters between ½-inch and ¾-inches. The ultimate strength formula was determined to be that shown on this slide. The corresponding AISI formula indicates that greater weld strengths can result with multiple metal thickness (such as at overlaps), however the SDI tests suggests that for metal deck this effect should be ignored. The SDI also recommends, based on the distribution of the test results, that a safety factor of 2.75 be used for these welds rather than a safety factor of 2.56 as suggested by the AISI Specifications.

\[ Q_u = 2.2 \times t \times F_u \times (d-t) \text{ [in kips]} \]

Where:
- \( d \) = average visible diameter [inches]
- \( F_u \) = steel strength [ksi]
- \( t \) = base metal thickness [inches]

It is good practice to use weld washers for decks with a metal thickness less than 0.0280 inches. It is also recommended to use 5/8-inch puddle welds without washers for material thicknesses greater than 0.0280 inches. In no case should washers be used on interior sidelaps.

In adjacent deck panels with nestable or flat overlapping edges, sheet-to-sheet (stitch) connections are required. The installation of arc spot welds at sidelaps can be problematic. In addition, the thinner the material, the more difficult the welding operation. Welding of sidelaps is not recommended for material thicknesses of 0.0295 inches or thinner.

Source: Cordeck
Screw Connections:

Screw connections, such as Buildex TEK screws, may be either self-drilling or the self-tapping type that requires a predrilled hole prior to installation. The most commonly used screw size to attach deck to bar joists or structural steel are No. 12 and No. 14. Smaller No. 8 and No. 10 screws are more commonly used for sidelap connections.

When connecting thin metal deck to thicker structural members such as bar joists or beam flanges, little difference exists in the shear strength for No. 12 and No. 14 screws. This is because the failure mode of the deck tends to involve a “roll up” or crushing of the deck on the bearing side of the screw with fracture lines subsequently developing in the decking.

For stitch connections between sheets (at the sidelaps) a different type of failure occurs. The screw, when not anchored into a thicker more rigid element, tips over more easily and is more flexible. Although the resulting strength may be limited by bearing and tearing in the metal deck, with sufficient rotation of the sidelap, a combined tearing and pull out failure may also occur.

The SDI screw studies indicate that stitch screw shear capacity is independent of the yield strength of steel deck that is commonly used for flexible diaphragms. The most common premature failure problem with sidelap attachments occurs when the screw is allowed to thread-up on the upper sheet before becoming fully engaged in the lower, thus creating major gaps between the adjacent units. Such improperly installed screws should be removed and redriven while forcing the overlapping sheets to remain in contact.

Powder Driven Fasteners:

Metal deck to structural support connections can also be made by using nail-like fasteners, driven either pneumatically or with powder actuated tools. Such fasteners are made from hardened steel and usually have a heat-treated shaft to enhance anchorage. The shaft, which may have a slight taper, can be fitted with washers, concave to the driving direction to absorb the final driving energy and clamp the sheet in position.

Since there are no predrilled holes required with these types of connectors, the installation process displaces material and leaves it locked under the washer, resulting in very stiff connections. The driving depth is controlled by the power selection for the tool used. Fastener strength is influenced by the driving depth. Tests conducted by West Virginia University also indicate that the thickness of the underlying material has no effect on shear strength since the thinner sheet material will control performance.

A driven fastener should be installed so the head projects outward, from the attached part, to limits set by the manufacturer. The axis of the fastener must be perpendicular to the sheet prior to driving, usually within plus or minus 10 degrees of vertical. In addition, it is recommended that edge and end fasteners have a minimum sidelap edge distance of 3/8-inch and a minimum end and endlap distance of 1-inch.
**Button-Punched Sidelaps:**

In certain panels, one edge has an upstanding single element, while the opposite side has a folded-over double element. As panels are placed, the single element is inserted into the double element, producing an upstanding sidelap that can be button punched to provide interlocking of the two adjacent sheets.

A hand operated punching tool forms a three layer nest of small cones that is left in a slightly loose state, because of elastic rebound, as the forming force is removed. Manual button punched sidelaps do stabilize panel edges but contribute little diaphragm strength as they can vary greatly in shape and effectiveness. The quality of hand installed button punches is difficult to maintain. The attachment depends on the care and the energy used by the installer and the tool used. A safety factor of three is recommended.

**Miscellaneous:**

- At conditions where joists span perpendicular to and terminate at a shear wall, the edge-most diaphragm panel may not be in direct contact with the wall. At these conditions the required perimeter diaphragm connections can be accomplished by installing a block spacer or collector along the top of the wall between the joists that matches the depth of the bearing assembly. A continuous angle attached directly to the top of and perpendicular to the joist may also be used as long as the bearing assembly of the joists is capable of transferring the collector force to the top of the wall below.
- Allowable load tables are available from the SDI and a number of different deck manufacturers. Load tables are typically categorized by the means of connection, the panel width and thickness, span lengths, the fastening pattern and the type of fastener and side lab connector.
- The stiffness of a diaphragm is a direct indication of how susceptible the deck is to distortions under the influence of in plane shear forces. The need to know the magnitude of movement is particularly important when assessing the transfer of forces, through a diaphragm, between adjacent frames or shear walls. In addition, the horizontal deflection of the diaphragm can have an impact on the sway of exterior cladding members, such as precast or tilt-up panels, in addition to the localized flexural deflection of the same as a result of wind loads perpendicular to the span of the diaphragm.
- Steel deck diaphragms may be reinforced with overlayments such as insulating concrete, structural concrete, or by directly attaching flat panels such as plywood to produce a flat surface. The supplemental material provides additional paths through which shear forces may traverse the diaphragm.
- It is also possible to install horizontal diagonal steel bracing or straps beneath the deck in order to supplement or supplant the shear capacity of the deck.
**Design Example**

The following design example is for a simple one-story rectangular building exposed to a total windward and leeward pressure of 35 PSF. It should be noted that in this example the windward and leeward pressure reactions at the top of the cladding and along the edge of the diaphragm are shown as a combined line load of 245 PLF.

As indicated in the example, the diaphragm reaction of 16.54 kips is divided by the length of the end wall to arrive at the unit collector diaphragm shear force of 367.5 PLF. At the same time a shear diagram is developed for the entire diaphragm span in order to assist in the establishment of different fastener zones as required to satisfy the varying magnitudes of planar shear resistance resulting from the imposed loads.

An Allowable Diaphragm Shear Strength Table (available from most of the roof deck manufactures as well as the SDI) is then referred to in order to assess the most appropriate fastener spacing for the applied loads. In this example, 36-inch wide, 1½-inch, Type B, 22 Gage metal roof deck using 5/8-inch puddle welds and #10 TEK screw sidelap fasteners have been pre-selected as the type of deck and fasteners to be used. It should be noted that the capacities in the Shear Strength Table already include a 1/3 increase for wind.

For a joint spacing of 6-feet and a pre-selected weld pattern of 4 uniformly spaced puddle welds across the 36-inch wide panel, a shear capacity of 374 PLF can be obtained using 8 uniformly spaced sidelap screws along the 6-feet span of the deck. This value is greater than the 367.5 PLF end shear reaction calculated in the example.

Next at an arbitrary location approximately 1/3 of the diaphragm span from the end support wall is analyzed in order to allow for a reduction in the number of fasteners over the middle half of the roof. The equivalent uniform shear load at this location on the diaphragm is 193 PLF. Referring again to the Allowable Shear Strength Table indicates that using a weld pattern of 3 uniformly spaced puddle welds across the 36-inch wide panel, a shear capacity of 193 PLF can be obtained using 2 uniformly spaced sidelap screws along the 6-feet span of the deck.

**Detailing**

Detailing of the connection of the metal roof deck to the end shear walls is not covered in this example, however typically a continuous embedded steel plate in the top of the wall would be provided in order for the deck to be secured to the top of the wall.

Shear in the deck diaphragm is also checked for wind loads against the short side of the building in which the diaphragm only has to span 45-feet between the 135-feet long CMU shearwalls. The calculated maximum unit collector diaphragm shear force of 41 PLF is less than the minimum 193 PLF shear capacity of the deck in the other orthogonal direction, therefore the fastener pattern developed in the first part of the example is adequate for wind loads in the opposite direction. However, as the joint bearing assembly separates the top edge of the deck from the CMU shearwall it is necessary for the collector force to be transmitted from the diaphragm to the top of the wall through the joint bearing assemblies. A maximum “roll-over” force of 246 pounds is calculated at each joint, which is less than the typical industry standard joist bearing assembly capacity of 1,650 pounds, therefore no collector block is required between each joint to transfer the shear forces to the top of the wall.

In order for the diaphragm to function properly, in addition to an analysis of the planar shear forces, it is necessary to provide chord members along the extreme edges of the deck to provide resistance to the extreme flexural tension and compression components of the deep beam diaphragm analogy. For this example the maximum chord force is calculated as follows:

\[ M_{\text{max}} = \frac{(245 \text{ PLF}) \times (135 \text{ FT})^2}{8} = 558.14 \text{ Kip Feet}; \text{ Chord Force} = 558.14 \text{ KF/45 FT} = 12.4 \text{ Kips} \]

Assuming that compression controls, the maximum allowable compressive stress of 10.14 ksi is calculated for a 36 ksi, continuous L3x3x1/2 inch long welds at 6-feet on center were it is connected to the top of the joist bearing assemblies. The actual calculated compressive stress of 8.61 ksi in the L3x3 is less than the allowable. It is important to understand that this chord member must be provided along both 135–feet long edges of the deck with butt welds provided at all joints of the L3x3. The chord member for wind forces in the opposite direction is provided by the continuous embedded steel plate located in the top of the 45-feet long end shearwalls as described in the previous slide.
FLEXIBLE ROOF DIAGRAMS
EXAMPLE

REACT. 20Ft. WALL

7-1/2 FT. DECK

REACT. 20Ft. WALL

REACT. 20Ft. WALL

CHECK SHEAR IN OPPOSITE DIRECTION = 524 kips

CHECK JOINT STRESS

M = 12kips-ft / 52

P = 3kips / 12" wide panel

Fy = 40ksi / 52kips

FINAL STEPS:

1. DEVELOP DECK TO TOP OF SHEAR WALL (A/V TRANSFER SEIZE 2)

CHECK:

2. CHECK DIAGRAM SHEAR FOR PS EFFECT ON LOAD BEARING WALL.
### 1.6 (B, F, A) 22 ALLOWABLE DIAPHRAGM SHEAR STRENGTH (PLF)

**SUPPORT FASTENERS: 5/8" puddle welds**

**SIDELAP FASTENERS: #10 TEK screws**

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**Source:** SDI

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**Diagram: 5/8" Dia Puddle Weld Support Fasteners**

**#10 TEK Screws**

**SIDELAP FASTENERS**

**STRUCTURAL SUPPORTS**

**PLAN DETAIL**

**SECTION**

**ZONE**

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**2'6" COVERAGE**

**DIAGRAM**

**EXAMPLE OF WELD PATTERNS**

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The following are illustrations of roof deck attachment diagrams and schedules from an example project. This information should be provided on all projects in order to provide clear instructions to the deck installer concerning the required fastener type, spacing and extent.