PDHonline Course S184 (2 PDH)

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.

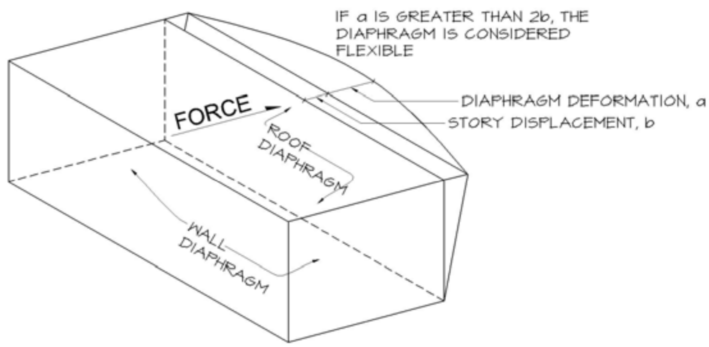


Figure 1 illustrates the basic concept for a roof and wall diaphragm. In Figure 1 wind pressure on the walls parallel to length L act on the building. A line load reaction of this pressure is distributed to the top of the wall at the roof plane via the vertical spanning capacity of the cladding over 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.

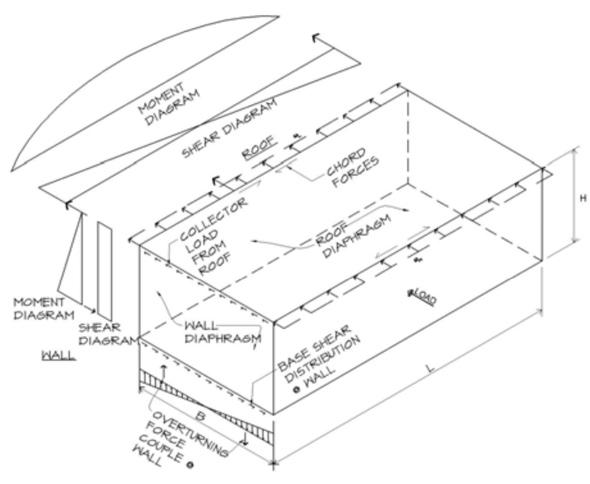


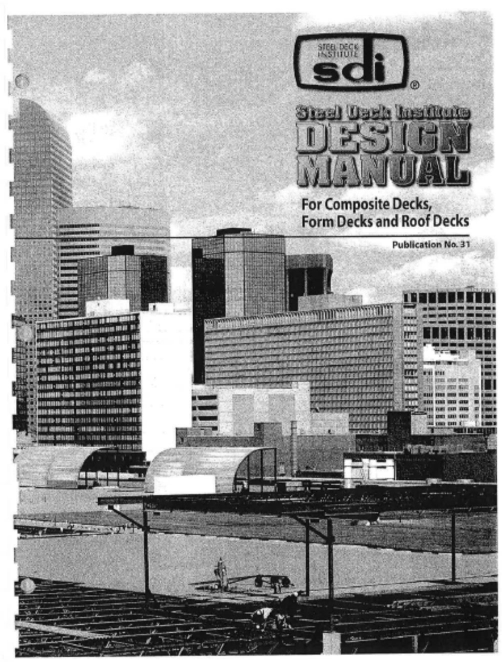
FIGURE 1

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 $((w_w + w_L) \times L)/2$. R divided by length B equals the collector load or maximum uniform shear force in the diaphragm. The maximum roof diaphragm moment (M) equals $((w_w + w_L) \times L^2)/8$. 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 x 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, <u>Design Manual for Composite Decks, Form Decks and Roof Decks</u>. 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.

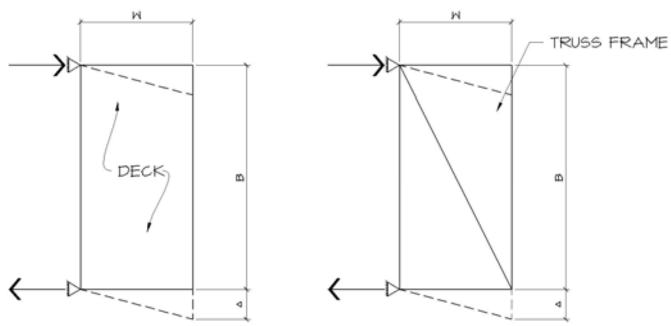
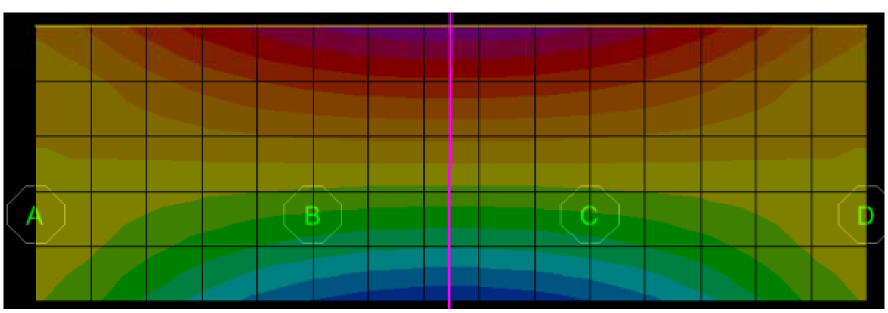


FIGURE 2

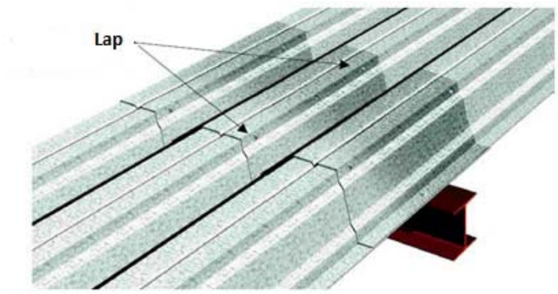
In addition to needing to resist planar shear forces, in order for a diaphragm to function properly it is also necessary to provide members along the extreme edges of the deck as required to resist the flexural tension and compression components of the deep beam member. This is analogous to the top and bottom chords of a truss resisting the compression and tension forces of the member.



Source: CSI

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.



Source: Tata Steel

Fasteners

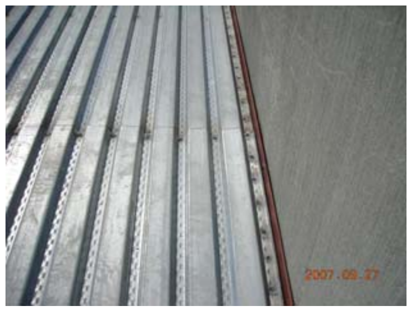
Establishing what type of fastener is to be used is a critical component of designing and detailing a diaphragm. Fasteners commonly include welds, screws and powder driven nail-like pins. Structural connections include fasteners that connect one or more sheets to heavier or thicker frame or structural 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.

Source: Hilti; Metal Deck Supply; Hobart Brothers

Welded Connections:

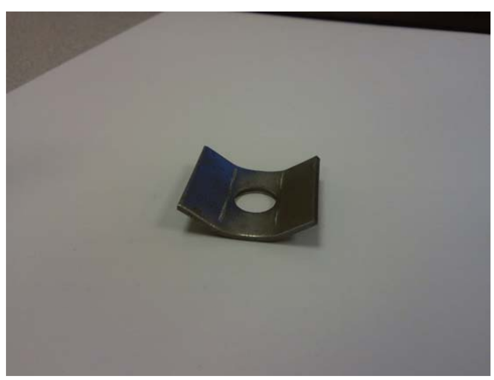
Arc-spot welds, or puddle welds, are produced by striking an arc on the upper sheet, thereby causing a hole to form, while the lower sheet is being raised to fusion temperature. With the attainment of the proper temperature, the electrode is moved in a circular pattern until the hole is filled and fusion 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.



Source: Palm Beach County Schools

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: 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.

Sidelap Screw Connector
Source: Bondek

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.

Source: Simpson

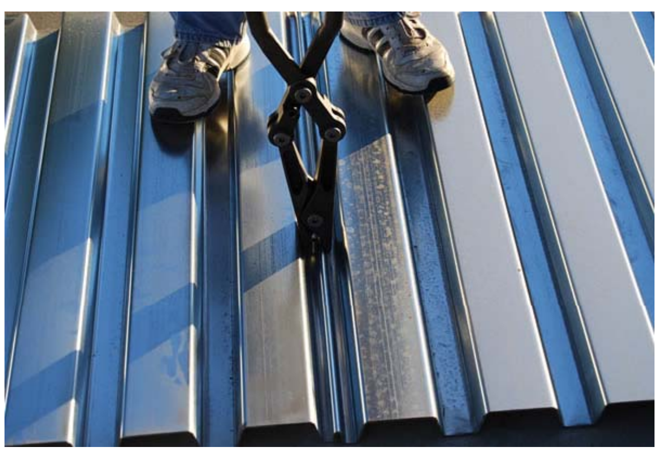
Powder-Driven Fasteners:

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.

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.



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.

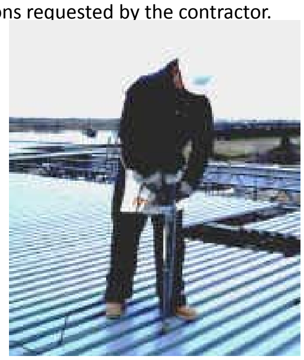
4. The exposed underside appearance of the deck may be important. If appearance is critical sidelap screws or even very weak button punches may be required even though an increased number of these connectors would be needed over that provided by welding.

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.



Source: Barsin Products

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.

$$Q_f = 2.2 * t * F_u * (d\text{-}t) \text{ [in kips]}$$

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 TEKS 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 additional 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 joist 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 ¼ 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.

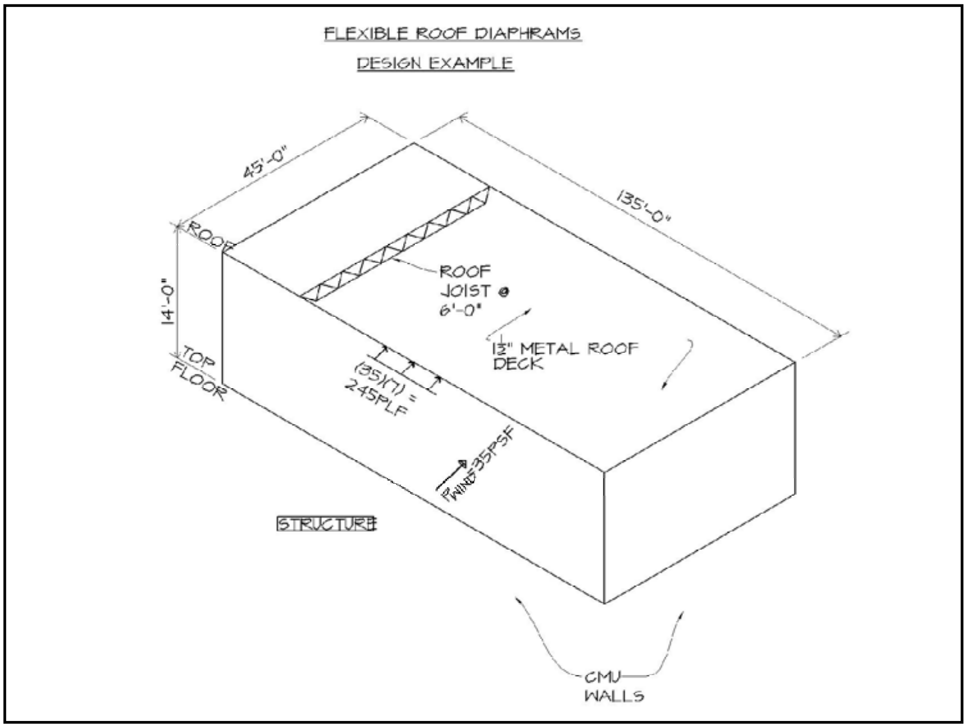
Detailing of the connection of the metal roof deck to the end shear walls is not covered in this example, however typically a continuous embedded steel plate in the top of the wall would be provided in order for the deck to be secured to the top of the wall.

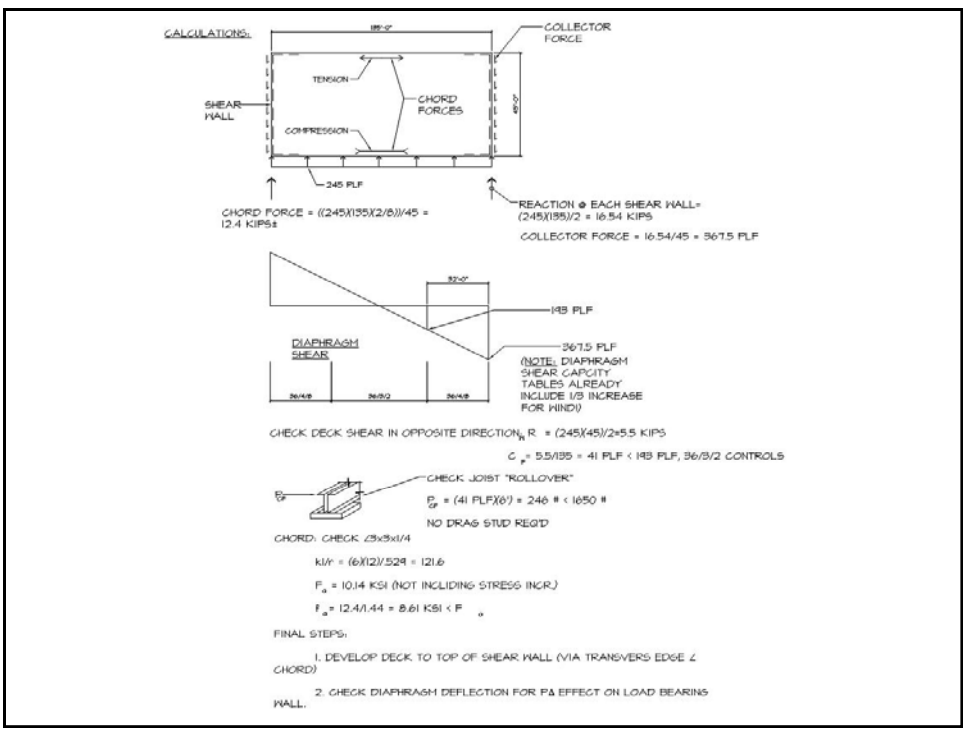
Shear in the deck diaphragm is also checked for wind loads against the short side of the building in which the diaphragm only has to span 45-feet between the 135-feet long CMU shearwalls. The calculated maximum unit collector diaphragm shear force of 41 PLF is less than the minimum 193 PLF shear capacity 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 joist 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 joist bearing assemblies. A maximum “roll-over” force of 246 pounds is calculated at each joist, which is less than the typical industry standard joist bearing assembly capacity of 1,650 pounds, therefore no collector block is required between each joist 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_{max} = ((245 PLF) * (135 FT)^2)/8 = 558.14 Kip Feet; Chord Force = 558.14 KF/45 FT = 12.4 Kips

Assuming that compression controls, the maximum allowable compressive stress of 10.14 ksi is calculated for a 36 ksi, continuous L3x3x¼ braced 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 DIAPHRAMS
DESIGN EXAMPLE
45'-0"
135'-0"
ROOF
14'-0"
TOP FLOOR
ROOF JOIST @ 6'-0"
1½" METAL ROOF DECK
(35)(7) = 245PLF
P WIND 35PSF
STRUCTURE
CMU WALLS


CALCULATIONS:
COLLECTOR FORCE
TENSION
SHEAR WALL
CHORD FORCES
COMPRESSION
245 PLF
CHORD FORCE = ((245)(135)(2/8))/45 = 12.4 KIPS±
REACTION @ EACH SHEAR WALL= (245)(135)/2 = 16.54 KIPS
COLLECTOR FORCE = 16.54/45 = 367.5 PLF
193 PLF
DIAPHRAGM SHEAR
367.5 PLF
(NOTE: DIAPHRAGM SHEAR CAPCITY TABLES ALREADY INCLUDE 1/3 INCREASE FOR WIND)
CHECK DECK SHEAR IN OPPOSITE DIRECTION, R = (245)(45)/2=5.5 KIPS
C P= 5.5/135 = 41 PLF < 193 PLF, 36/3/2 CONTROLS
CHECK JOIST "ROLLOVER"
P CP = (41 PLF)(6') = 246 # < 1650 #
NO DRAG STUD REQ'D
CHORD: CHECK ∠3x3x1/4
kl/r = (6)(12)/.529 = 121.6
Fa = 10.14 KSI (NOT INCLIDING STRESS INCR.)
fa= 12.4/1.44 = 8.61 KSI < Fa
FINAL STEPS:
1. DEVELOP DECK TO TOP OF SHEAR WALL (VIA TRANSVERS EDGE ∠ CHORD)
2. CHECK DIAPHRAGM DEFLECTION FOR PΔ EFFECT ON LOAD BEARING WALL.

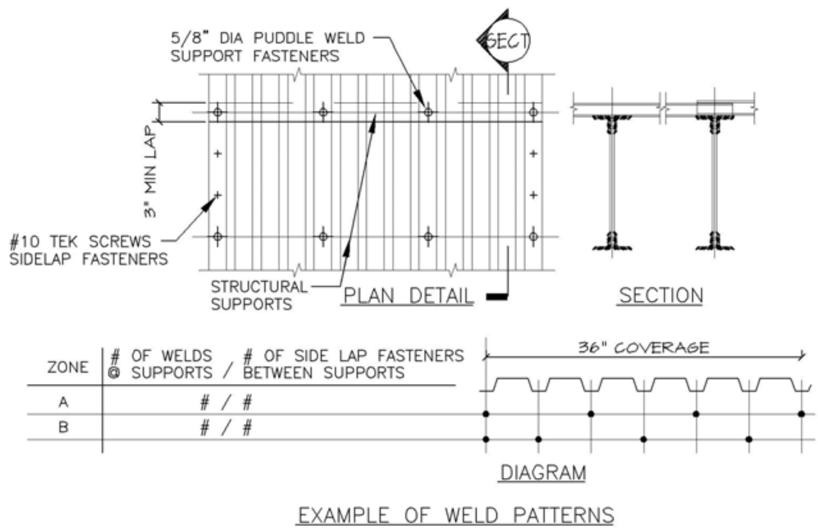
1.5 (B, F, A) 22 ALLOWABLE DIAPHRAGM SHEAR STRENGTH (PLF)
SUPPORT FASTENERS: 5/8" puddle welds[1]
SIDELAP FASTENERS: #10 TEK screws

CAPACITY TABLE

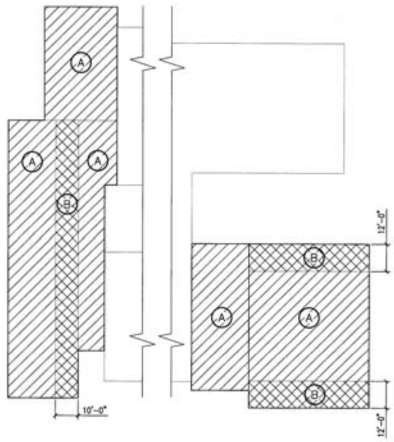
Factor of safety = 2.75

Fastener Layout	*	3'-0	3'-6	4'-0	4'-6	5'-0	5'-6	$\boxed{6'-0}$	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	10'-0	K1
	0	413	359	313	276	247										0.486
	1	476	416	369	327	293	265	242								0.377
	2	535	470	418	376	339	307	280	257	238	221	206	193	182		0.309
	3	590	521	466	420	382	349	319	293	271	252	235	220	207	185	0.261
	4	642	570	511	462	421	387	357	328	304	282	264	247	233	202	0.226
36/7	5	689	615	554	503	459	422	391	363	337	313	293	275	249	202	0.199
	6	732	658	595	541	496	457	423	394	368	344	316	280	249	202	0.178
	7	772	697	633	578	531	490	455	424	397	359	316	280	249	202	0.161
	8	809	734	669	613	565	523	486	453	412	359	316	280	249	202	0.147
	9	842	768	704	647	597	554	515	478	412	359	316	280	249	202	0.136
	10	873	800	736	678	628	583	544	478	412	359	316	280	249	202	0.126
	11	901	830	766	706	657	612	561	478	412	359	316	280	249	202	0.117
					DB = 129		DF = 226		DA = 356		K2 = 870					
	0	366	322	286	255	228										0.583
	1	419	371	333	301	274	248	226								0.434
	2	466	417	376	341	312	287	264	243	225	209	195	182	171		0.345
	3	508	456	415	379	348	321	298	277	257	239	223	209	197	176	0.287
	4	545	495	452	414	382	353	328	307	287	270	252	236	222	199	0.245
36/5	5	577	528	485	447	413	384	358	335	314	296	280	264	248	202	0.214
	6	604	558	515	477	443	412	386	362	340	321	303	280	249	202	0.190
	7	629	584	542	504	470	439	412	387	365	345	316	280	249	202	0.171
	8	650	607	567	530	496	465	437	411	388	359	316	280	249	202	0.155
	9	669	628	589	553	519	488	460	434	411	359	316	280	249	202	0.142
	10	685	646	609	574	541	510	482	456	412	359	316	280	249	202	0.131
	11	699	663	627	593	561	530	502	476	412	359	316	280	249	202	0.122
					DB = 758		DF = 886		DA = 974		K2 = 870					
	0	280	247	219	193	172										0.729
	1	331	295	265	240	218	197	179								0.509
	2	375	338	306	279	256	236	218	200	184	171	159	149	140		0.392
	3	412	375	342	314	290	268	249	233	217	202	188	176	166	148	0.318
	4	443	407	375	346	321	298	278	261	245	231	217	203	191	171	0.268
$\boxed{36/4}$	5	468	434	403	374	349	326	305	287	270	255	241	229	217	194	0.231
	6	490	458	427	400	374	351	330	311	294	278	264	251	239	202	0.203
	7	507	478	449	422	397	374	353	333	316	300	285	271	249	202	0.182
	$\boxed{8}$	522	495	468	442	418	395	$\boxed{374}$	354	336	320	305	280	249	202	0.164
	9	535	510	484	460	436	414	393	374	356	339	316	280	249	202	0.149
	10	546	522	499	475	453	431	411	392	374	357	316	280	249	202	0.137
	11	555	533	511	489	468	447	427	408	390	359	316	280	249	202	0.127
					DB = 1072		DF = 1216		DA = 1282		K2 = 870					
	0	234	209	189	171	153										0.972
	1	273	248	226	208	191	177	164								0.617
	$\boxed{2}$	303	279	258	239	222	207	$\boxed{193}$	181	170	159	148	138	130		0.452
	3	325	304	284	265	248	233	219	206	195	184	175	165	155	138	0.357
	4	343	323	305	287	271	255	241	228	216	206	196	186	178	161	0.295
$\boxed{36/3}$	5	356	339	322	306	290	275	261	248	236	225	215	205	196	180	0.251
	6	367	351	336	321	306	292	279	266	254	243	232	222	213	197	0.219
	7	375	361	348	334	320	307	294	281	270	259	248	238	229	202	0.194
	8	381	370	357	344	332	319	307	295	284	273	263	253	244	202	0.174
	9	387	376	365	353	342	330	319	307	296	286	276	266	249	202	0.158
	10	391	382	372	361	350	339	329	318	308	297	288	278	249	202	0.144
	11	395	386	377	368	358	348	338	327	318	308	298	280	249	202	0.133
					DB = 2209		DF = 2428		DA = 2442		K2 = 870					

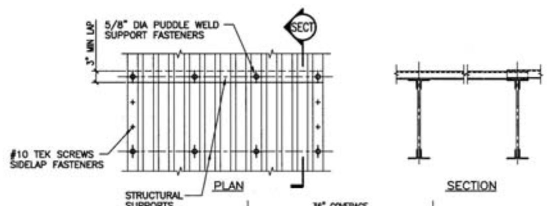
Source: SDI



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.



ROOF DECK ATTACHMENT DIAGRAM



36" COVERAGE

ZONE	# OF WELDS @ SUPPORTS / # OF SIDE LAP FASTENERS BETWEEN SUPPORTS	
A	(4)-5/8" DIA WELDS / (4)-#10 TEK SCREWS	ALLOW SHEAR = 317 PLF (NOTE 5)
B	(4)-5/8" DIA WELDS / (8)-#10 TEK SCREWS	ALLOW SHEAR = 451 PLF (NOTE 5)

1 1/2" TYPE-B

ALTERNATE S-2

24" COVERAGE

ZONE	# OF WELDS @ SUPPORTS / # OF SIDE LAP FASTENERS BETWEEN SUPPORTS	
A	(4)-5/8" DIA WELDS / (6)-#10 TEK SCREWS	ALLOW SHEAR = 317 PLF (NOTE 5)
B	(4)-5/8" DIA WELDS / (10)-#10 TEK SCREWS	ALLOW SHEAR = 451 PLF (NOTE 5)

3" TYPE-N

NOTES:

1. DECK SHALL BE ATTACHED TO ALL STRUCTURAL SUPPORTS WITH 5/8" DIA PUDDLE WELDS.
2. SIDE LAPS BETWEEN STRUCTURAL SUPPORTS SHALL BE FASTENED BY #10 TEK SCREWS.
3. DECK SHALL BE ATTACHED TO ALL PERIMETER SUPPORTS WITH 5/8" DIA PUDDLE WELDS @ 6"OC MAX
4. END LAPS SHALL BE A MINIMUM OF 3" AND SHALL OCCUR OVER SUPPORTS.
5. CAPACITY BASED ON 6'-0" DECK SPAN.

TYPICAL DETAIL
ROOF DECK ATTACHMENT SCHEDULE