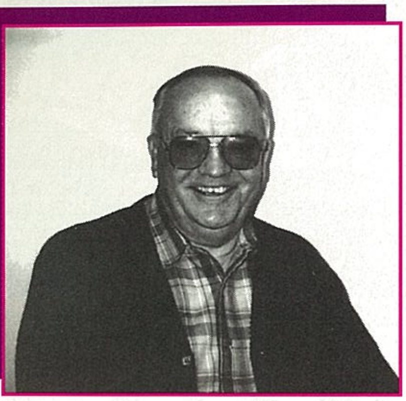
Designing Roof And Floor Diaphragms

A primer on floor and roof decks



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STEEL-FRAMED STRUCTURES MOST COMMONLY USE COLD-FORMED PROFILED STEEL PANELS FOR BOTH ROOF AND FLOOR DECKING. Such decking is manufactured to a wide range of configurations and often allows multiple spans from a single panel length. Typical roof loads may dictate spans of perhaps 6-ft., using a relatively thin panel, whereas floors may require either shorter spans or thicker decking.

Common panels have depths between 1.5 and 3 in., with steel thicknesses between 0.0295 and 0.06 in. Floor panels may have embossments on some surfaces to enhance interlocking and composite slab behavior when concrete is used to finish floors. Roof areas may be finished using insulating concrete or some other overlayment method to form a smooth surface.

The individual deck panels must be properly attached to the structure since, during construction, they must sustain transient loads including wind uplift forces. Panel-to-panel side lap or "stitch" connections may be required to prevent edge separation between adjacent units (the latter are particularly important when concrete fill is used).

The complete assembly of panels, purlins and framing members possess beam-like characteristics and may be thought of in terms of a flexible deep girder. The assembly will act as a shear diaphragm and can be used to transfer in-plane forces between units of the structure.

Shear Elements

A horizontal diaphragm assembly involves the deck panels, any overlayment or concrete fill, and structural members to which the decking is attached. Though its shear stiffness may be an order of magnitude lower than than with conventional girders, the diaphragm can be thought of as a thin-web girder. Each "girder" or diaphragm area is bounded by stiffeners, flanges on two sides and shear walls on the other two sides. The "web" shear elements require that forces be transferred to them through the boundary members to which the shear elements must be adequately attached.

The design issue usually is not one of developing all the shear potential for a diaphragm assembly but rather to design connection details so that the assembly can develop the bracing level required. The developed bracing level depends on perimeter conditions, on the connections within the diaphragm field, and on decking geometry.

The Basic Diaphragm

It is clear that most diaphragms, when viewed in plan, will fall into the short, deep-beam category as illustrated in Figure 1. Further, the diaphragm shear stiffness often is an order of magnitude lower than for a continuously connected flat plate of similar dimensions. Any analytical process then, must include shear deflections and they may well dominate the total deflections for the system.

Consider a roof area of B x L as shown in Figure 1. The diaphragm decking is not shown but is to be supported on spandrel beams between columns on the long sides and on three shear walls in the shorter B direction. The system is to be evaluated for a uniformly distributed load, q, that may have been from either a wind acting on the long walls or from an earthquake loading condition. Considering the entire roof area as a horizontal deep beam will require finding the shear wall reactions V_L, R, and V_R. Following Figure 2, these values may be found by noting that total deflection at the inner rigid shear wall must be zero.

The change in shear deflection between two points may be established from changes in the shear diagram areas between those same two points. The shaded areas on the shear diagrams, divided by the shear width B and the stiffness G′, establishes the shear deflection at the center wall. Further, presuming a beam moment of inertia, I, the bending deflections can be found from a simple analyses.

Let the subscripts s and b indicate shear and bending deflections respectively, while q and R represent the uniform load and interior shear wall reaction. The following individual components of deflection result for x = L/3:

$$\Delta_{sq} = \frac{L}{3BG'}\left[\frac{1}{2}\left(\frac{ql}{2} + \frac{ql}{6}\right)\right] = \frac{qL^2}{9BG'}$$

$$\Delta_{bq} = \frac{11qL^4}{972\,EI} \qquad (1)$$

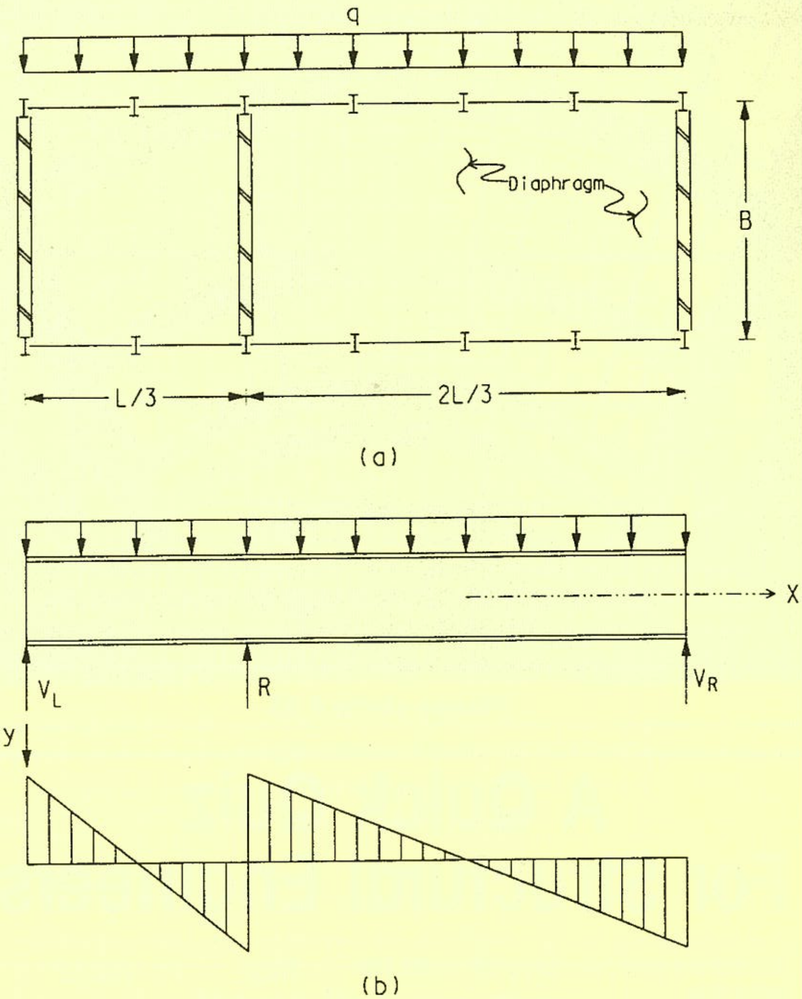
$$\Delta_{sR} = \frac{1}{BG'}\left(\frac{2R}{3}\right)\frac{L}{3} = -\frac{2RL}{9BG'} \qquad (2)$$

$$\Delta_{bR} = \frac{4RL^3}{243\,EI} \qquad (3)$$

where: L = building length
B = building shear width
E = Young's modulus
q = uniform load
G′= diaphragm shear stiffness
R = interior wall reaction
I = beam moment of inertia

Considering the shear walls to be essentially rigid and defining α = EI/BG′, the net deflection at the interior wall must be zero, resulting in:

Figure 1:Roof diaphragm.
(a) roof plan. (b) beam and shear diagram



$$R = qL\left(\frac{108 + 11\,L^2/\alpha}{216 + 16\,L^2/\alpha}\right) \qquad (4)$$

The interior reaction R is dependent on the relative stiffnesses, α. If BG′ is large relative to EI, the shear deflection component becomes small and R approaches (11/16)qL which is the reaction for a long slender beam on the indicated spans. However, if G′ is small, R approaches qL/2 and the interior wall receives shear forces only from from its tributary area, i.e., half of each roof zone framing into it..

To illustrate that the diaphragm deflection problem is largely one of shear only, consider a typical roof of thickness t = 0.75 mm (0.0295 in.) and with a shear stiffness G′= 7000 N/mm (40 kips/in.). The equivalent shear modulus would be G = G′/t = 9300 N/mm²(1360 kips/in².) compared to a flat plate value of 78.3 kN/mm² (11,350 kips/in².). The diaphragm is about an order of magnitude more flexible in shear than is a flat plate.

The spandrel beams in Figure 1 may not be so rigidly connected at their ends that they can act as continuous diaphragm flanges. However, their area, A_s, might be used to indicate an approximate lower-bound value for the the "girder" moment of inertia such as:

$$I = 2A_s\left(\frac{B}{2}\right)^2 \qquad (5)$$

As an example, consider the following data for the roof in Figure 1: B = 30 m (98.5 ft.), L = 90 m (295 ft.), spandrel area As = 10 x 10^{-3} m^2 (15.5 in.^{2}), E = 203 x 106 kPa (29,500 kip/in.^{2}), and G′ = 7 x 10^6 N/m (40 kips/in.).Considering the spandrel member as a diaphragm flange, moment of inertia may be approximated by:

$$I = 2A_s(\frac{B}{2})^2 = \frac{A_sB^2}{2} \quad (6)$$

$$\text{Then } \alpha = \frac{EI}{BG'} = \frac{EA_sB}{2G'} \quad (7)$$

$$= \frac{203 \times 10 \times 30}{2 \times 14} = 4350 m^2$$

$$\text{And } R = qL(\frac{108 + 11(90)^2/4350}{216 + 16(90)^2/4350} \quad (8)$$

$$= 0.523qL$$

The interior shear wall reaction is R = 0.523 qL. If there were any connection slip in the spandrel beams at the columns, the effective moment of inertia could be even smaller and the interior shear wall reaction even closer to 0.5qL. These values are typical for many diaphragms and indicate that such diaphragms can be treated as simple shear systems, ignoring bending deflections.

Diaphragm Field

The field of a diaphragm is that part away from shear walls or braced frames where behavior is dictated by individual deck panels, their dimensions, and the type of connections used. In any panel of width, w, connections may be made through some or all of the lower flat elements as in Figure 3. As external shear forces act in the plane of the diaphragm, they produce racking shear distortions as indicated in Figure 4. Resistance to the shear distortion is developed through a series of internal couples at the cross-panel support members coupled with shear transfer forces on sidelaps.

The Figure 4 panels are shown with four structural connections per panel at the end-of-panel purlins, three per panel across interior purlins, and two stitch connections per span in the sidelaps for a total of six stitch connections per panel edge. The resisting couples depend on fastener position and fastener shear strength. Extensive research and development programs (1, 2) have shown that the equlibrium of an interior panel near ultimate load can be expressed as:

$$P_u\frac{W}{L_s} = 2M_e + n_pM_p + n_sQ_sw \qquad (9)$$

where:
- M_e = end of panel couple
- M_p = interior purlin couple
- L_s = panel length
- n_p = number of interior purlins (2 shown)
- n_s = number of stitch connections/sheet (6 shown)
- Q_s = stitch connector shear strength
- w = deck element width

With a presumed linear variation in connector shear forces, F_p, across the panel and relative to the limiting fastener shear strength Q_f:

$$F_p = Q_f\left(\frac{x_p}{w/2}\right) \qquad (10)$$

with

$$M_p = \Sigma F_p x_p = \frac{2}{w}Q_f\Sigma x_p^2 \qquad (11)$$

$$M_e = \frac{2}{w}Q_f\Sigma x_e^2 \qquad (12)$$

The end-of-panel couple, M_e, may be somewhat limited depending on the ability of the edge-most corrugation to resist eccentric compression. It has been found that the connection at the compression corner is limited through:

$$\theta = 1 - \frac{d_dL_v}{C\sqrt{t}} \qquad (13)$$

Where
- d_d = decking depth, mm (in.)
- L_v = purlin spacing, m (ft.)
- t = panel thickness, mm (in.)
- C = 370 (240 in U.S. units)

Figure 2: Shear Diagram

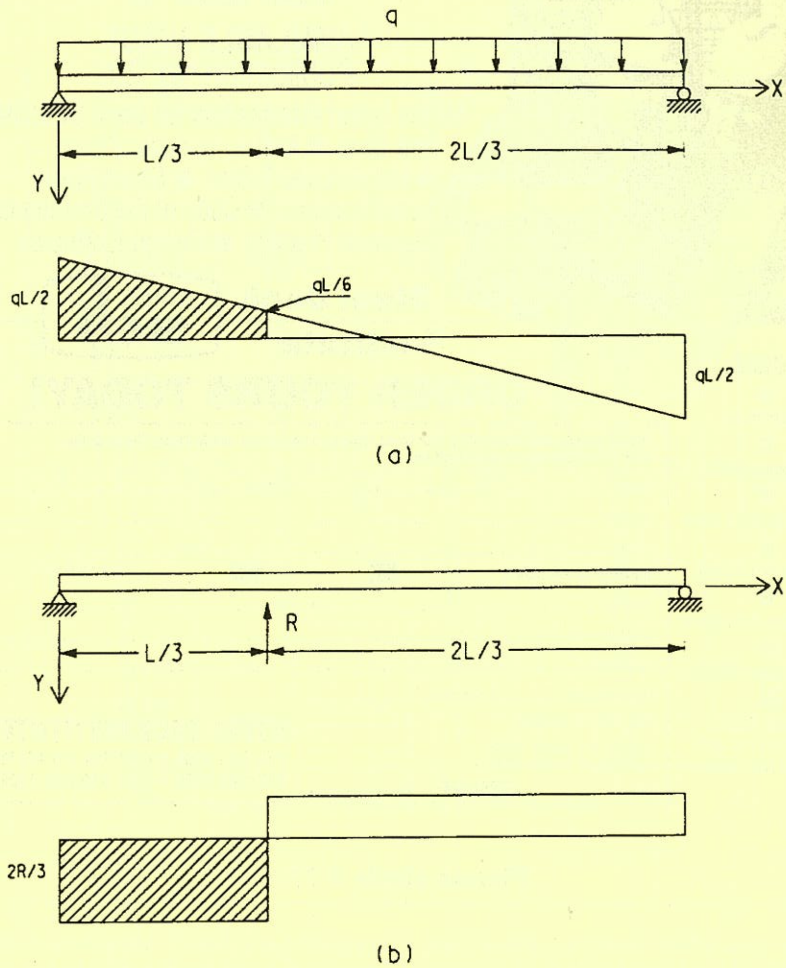
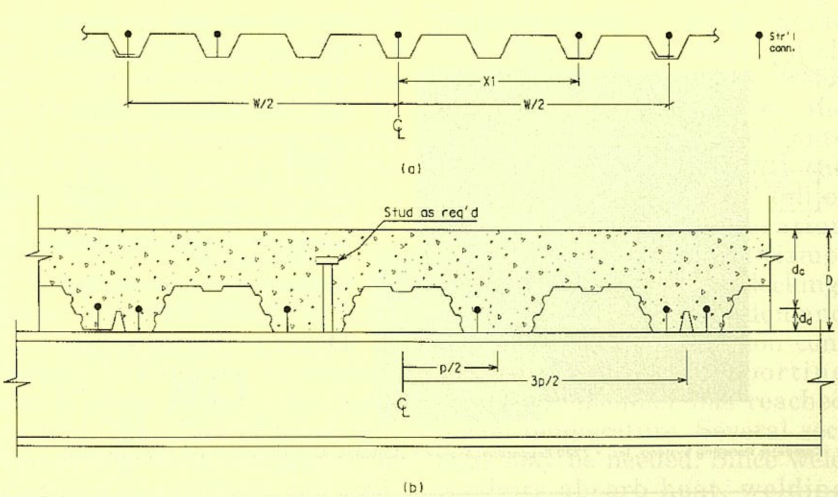


Figure 3: Deck connections
(a) 36/5 fastener layout, w=914mm (36 in.)
(b) floor deck layout



Equation 12 then can be reduced to the average ultimate shear strength per unit length:

$$S_u = \frac{P_u}{L_s} = [2(\theta - 1) + B]\frac{Q_f}{L_s} \qquad (14)$$

$$\text{with } B = n_s\alpha_s + \frac{1}{w^2}(2n_p\Sigma x_p^2 + 4\Sigma x_e^2) \qquad (15)$$

$$\alpha_s = Q_s / Q_f \qquad (16)$$

Structural concrete fill causes several changes in behavior. The concrete bonds to the decking, retards panel warping and side lap slip, and provides secondary shear transfer paths. Corner buckling is essentially eliminated by the forced alignment and the θ term of Eq. 13 approaches unity. The additional shear path in the field can greatly increase shear strength and lead to a system similar in shear strength to that of a conventional flat slab. With a minimum concrete cover depth, d_c = 63 mm (2.5 in.), the shear equation is modified to:

$$S_u = \frac{BQ_f}{L_s} + \frac{w_c^{1.5}\sqrt{f_c}}{C_d} \qquad (17)$$

Where: S_u = shear strength, kN/m (kips/ft)
w_c = concrete unit weight, kg/m3 (lbs/ft3)
f_c = concrete compressive strength, kPa (lbs/in^3)
C_d = 225,000 (19500 in U.S. units)

Light weight insulating concrete and other finishing overlayments also provide additional shear paths but at considerably lower levels than is indicated for normal concrete by Equation 17. The Steel Deck Institute *Diaphragm Design Manual* contains formulations for such systems.

Diaphragm Perimeter

Figure 4 indicates two perimeter conditions, one is at the deck ends and the other over a shear wall. The decking end fastener arrangement already has been given consideration as part of the field evaluation for M_e in Equation 12.

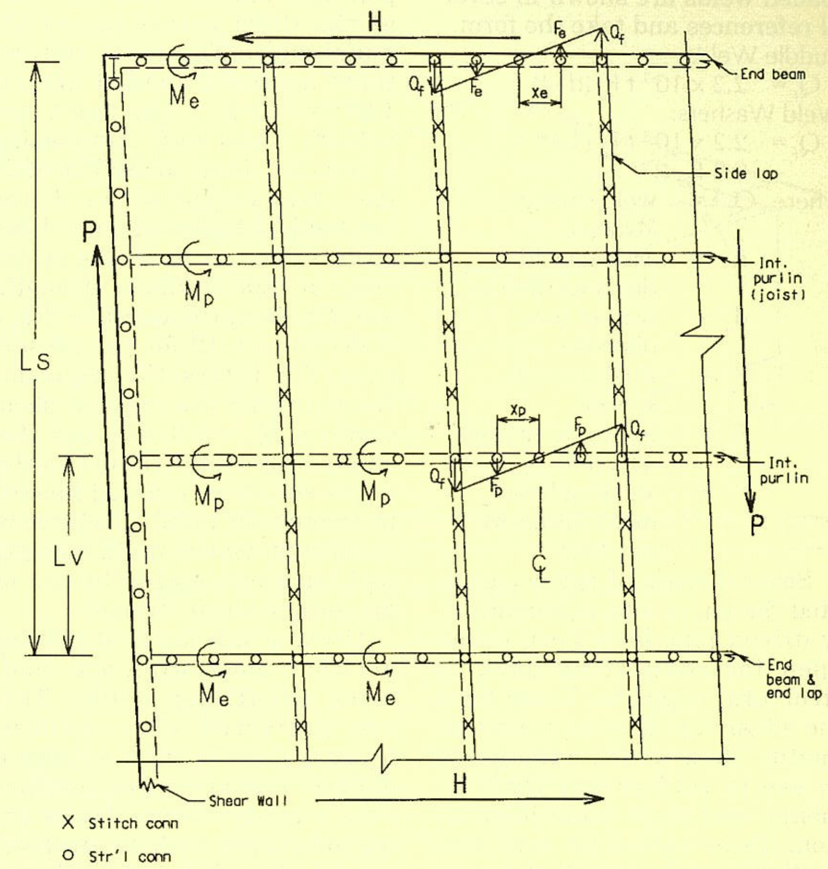
The decking connections along the shear wall can be critical to shear transfer especially if the diaphragm is concrete filled and the concrete is terminated at the wall. The first deck unit must transfer its shears into the field through couples at the purlins and through the sidelap connections along the first interior side lap. On the other side of the first sheet, it is essential that panel-to-wall connections develop the needed capacity. The figure indicates edge fasteners with the same spacing as those along the first side lap. Connections from a panel to the supporting framework usually are stronger than sidelap connections and the Figure 4 layout would be conservative for non-filled diaphragms. With concrete filled diaphragms, shear studs may be required.

At the ends of the decking, fasteners may be much more closely spaced than along diaphragm edges over shear walls. These spacing effects are considered within the formulations above except when edge connectors differ from those at deck ends. The design issue is simply to determine the ultimate average shear force along the top of the wall and to uniformly space an adequate number of connectors of known strength. Within the first decking panel, all other fastener arrangements should match those in the second panel.

Two special edge conditions arise. When the diaphragm is supported on joists, which rest on top of the shear wall, there is no ready method for installing edge connections ***between*** purlins. It is common practice to extend a thin steel angle along the shear wall and weld it to each joist top. The angle can then support deck edges and provide locations for frequent edge connections. The shear transfer throught the joist end may require some increases in welding for the joist seat.

A second edge condition arises

Figure 4: Layout indicating internal resisting couples



when concrete filled diaphragms are used. The first interior sidelap, and other components in the field, are strengthened by the concrete shear path. The concrete will terminate at the shear wall leaving this shear line weaker than others. Some edge enhancement may be developed by spacing the edge connections closer but, if very high shears are present, shear studs may be required to transfer concrete shear forces directly into the shear walls.

Connector Shear Strength

The most common method for connecting deck panels to steel framing is by arc welding or "puddle" welding. An electrode is used to strike an arc on the decking with the effect of blowing a hole at the weld location. The operation is continued by "puddling" the weld material in the hole. Such operations require intimate contact between connected parts for facilitating heat transfer and the operation requires considerable skill. Such a welding process typically is limited to panels of 22 gage [0.75 mm (0.0295-in.)] or heavier.

Structural welds through thinner decking can be made using welding washers which have prepunched holes. As welding through the hole begins, the washer absorbs heat and limits burn-out in the deck. Once the washer hole is "puddled" full of weld material, the operation stops, and the weld cools clamping the washer onto the decking. All welding operations demand that the welding operation continues until the supporting structural member has reached fusion temperature. Several seconds may be needed. Since weld washers absorb heat, welding with washers commonly requires more welding time than does arc puddle welding.

Values for diaphragm shear-loaded welds are shown in several references and take the form:

Puddle Welds:

$$Q_f = 2.2 \times 10^{-3}\, t\, F_u\, (d - t) \quad (18)$$

Weld Washers:

$$Q_f = 2.2 \times 10^{-3}\, t\, F_u\, (1.33\, d_o + 0.3\, F_{xx}\, t) \quad (19)$$

where Q_f = weld strength, kN(kips)
d = visible weld diameter, mm (in.)
d_o = washer hole diameter, mm (in.)
F_u = steel strength, kPa(ksi)
F_{xx} = electrode strength, kPa (ksi)
t = decking base metal thickness, mm (in.)

Several types of powder actuated fasteners and pneumatically driven pins have been developed for connecting deck to structural supports. These have the advantage of more consistent quality than welds, adaptablity to use in cold or wet environments, and more rapid installation. Shear values for such connections can be supplied by the manufacturer.

In addition to in-plane shear forces, deck structural connections may be subject to uplift, or tension loading. Such tension loads usually are small relative to shear loads.Current projects are underway at WVU to address shear strength reductions that might be associated with external tension forces. Preliminary results support an interactive form

$$\left(\frac{Q_s'}{Q_s}\right)^n + \left(\frac{T}{T_o}\right)^n \le 1 \quad (20)$$

where: n = 1.75 (preliminary)
Q_s' = available fastener shear strength
Q_s = fastener shear strength
T = external tension force
T_o = fastener tensile strength

Consider a 900 mm (3ft.)wide deck spanning 1.5 m (4.9 ft.) and with an uplift of 2000 Pa (42 psf). Structural connections are at the third points across the panel width.The three connections anchor a loaded area of 1.35 m² (15 ft.²). The uplift load is 2700 N and each fastener has a tension load of 900 N (200 lbs.).Many typical weld and mechanical fasteners have hold-down strengths in sheets on the order of 7500 N. Shear strengths are of a similar magnitude.T/T_o = 900/7500 = 0.12 for the present case..The interaction equation suggests that the reduced shear capacity, $Q'_s = 0.975\, Q_s$ and that shear-only diaphragms tables might simply be reduced linearly to account for uplift. Under most conditions, typical uplift loadings will have minimal influence on diaphragm shear strength.

Decking side lap or stitch connections most often are made using self-drilling screws. They are preferred over welding because sheet-to-sheet welding is difficult and always of questionable quality. Some decks are manufactured with an upstanding overlap that is suitable to button punching along such side laps. In the button punching operation, a crimping tool is used to form a series of nested cone-like indentations. On removing the tool, elastic rebound will allow some loosening of the cones.Such connections are adequate to prevent vertical separation at the decking edges but will be much weaker in shear that screws.

Sheet-to-sheet screws under shear load usually are limited in strength by tearing or splitting in the sheet around the screw. The screw strength shows little sensitivity to the material strength and may be represented by:

$$Q_s = C_s\,(dt) \quad (21)$$

where Qs = screw strength, kN (kips)
Cs = 0.8 (115 with inch units)
d = screw diameter, mm (in.)
t = decking thickness, mm (in.)

For a typical No. 12 screw, Q_s = 4.29 t, kN (24.3t, kips). Screw manufacturers can supply the designer with specific screw data.

Load Factors

Detailed studies have been made on full scale diaphragm tests conducted over the past 30 years at West Virginia University. These studies have involved systems with welds, screw connections, and power driven pins. Concrete filled diaphragms and others with mixed connection means have been included in the studies. These led the SDI to relate working shears S from transient loads to the foregoing formulations for ultimate strength as follows:

$$S = \frac{S_u}{SF} \quad (22)$$

where SF = safety factor
SF = 2.75 for welded diaphragms
SF = 2.35 for mechanical connections
SF = 2.75 for weldscrew combinations
SF = 3.25 for concrete filled systems

Diaphragms with Sloping Segments

The building of Figure 5 has a trussed roof system supported on simple columns and the roof diaphragm surfaces are identical. Two horizontal line loads, qw and qL, are acting and the trusses are sufficiently rigid that both eave lines move laterally by the same amount. Consider the diaphragm shears in each roof segment.

Since both surfaces experience the same horizontal eave movement and are of equal stiffness, they share the acting loads equally. Along the rake line A-B, a shear S is developed and it produces a total reaction force

$$R = S\frac{B/2}{\cos\varphi} \quad (23)$$

where φ is the roof slope. The

inclined R-force has a horizontal component of R(Cos φ) = S(B/2). Then for horizontal equilibrium using the four shear lines:

$$4\frac{SB}{2} = (q_L + q_w)L \text{ and} \qquad (24)$$
$$S = (q_L + q_w)\frac{L}{2B}$$

It is obvious then that the end wall shear in the diaphragms is exactly the same as if the roof had been dead flat and continuous across the B width. A note of caution is added. There must be a ridge cap device that can transfer shear from one roof surface to the other and there must be provision for transferring shears, S, from the roof surface and into the end walls.

Somewhat similar questions arise with roofs having locally changing slopes for drainage purposes. Consider the steel framed building in Figure 6 loaded with eave line loads as indicated. The wall at line a-b is a shear wall and all columns are simple. The total shear force along line a-b is $(q_w + q_L)L/2$ and the horizontal shear acting on quarter-point section c-e-d is half that at the end line a-b. The framing along d-e-c will force points d and c move about the same amount while the center column maintain the angle φ. The shears, S, along each inclined segment produce resultant forces R = S(B/2)Cos φ which have horizontal components of S(B/2). Then it is obvious that the average shear along the longer inclined components is exactly equal to what would have developed on shorter components in a similar flat roof.

Figure5: Building with sloped roof

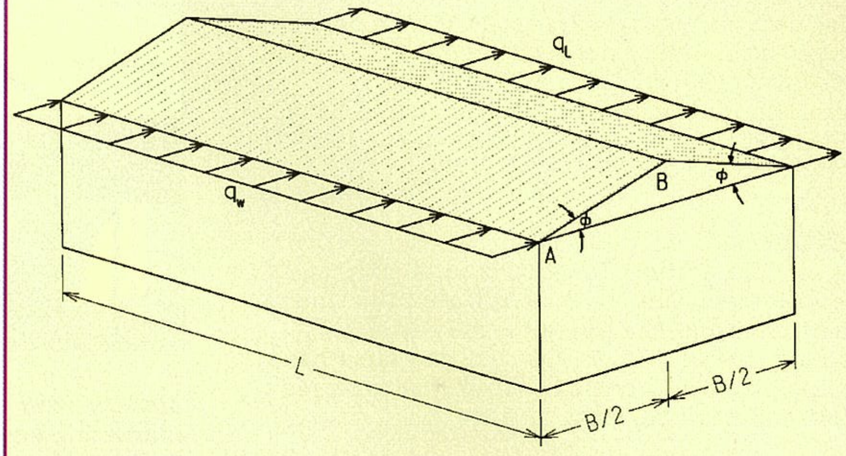
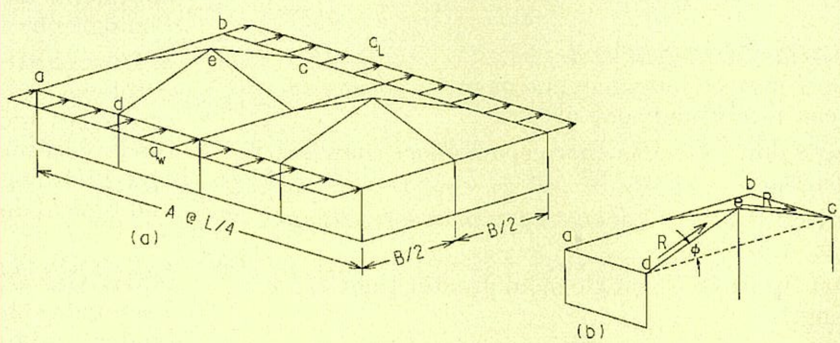


Figure 6: Roof with multiple roof slopes



Commentary

The design of a steel deck diaphragm is not particularly difficult when the problem is kept in proper perspecitve. The diaphragm simply is a large girderlike system subject to all the limitations of any other shear system. Care must be taken in the design with particular attention given to connections and load transfer lines.It is a structural system and must be designed and erected with all the care given to any other part of the structural system.

Sources of information in this article include:

- *Diaphragm Design Manual*, 1st Edition, Steel Deck Institute (1981)
- *Diaphragm Design Manual*, 2nd Edition, Steel Deck Institute (1987)
- *Cold Formed Steel in Tall Buildings*, McGraw-Hill (1993)
- "Deck Fasteners Under Combined Loadings," West Virginia University Research (193)