Diaphragm Action of Cellular Steel Floor and Roof Deck Construction

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Steel Decking UCI 5

Preface

This CSSBI Information Bulletin is published as an aid to Designers and Building Code Officials. It offers a simple and practical approach to the design of steel deck diaphragms consisting of cellular steel floor or roof deck supported by horizontal steel framing systems. Such construction is capable of providing an efficient diaphragm to resist lateral forces when all components are suitably interconnected. Although the design approach is to some extent empirical, various tests have confirmed the validity of the design values given. More sophisticated methods have not yet been developed to the point where they can be simplified for everyday practical applications.

Care has been exercised to ensure that all information contained herein is factual and that numerical values are accurate. The Canadian Sheet Steel Building Institute, however, assumes no responsibility for errors or oversights in the use of the information for designs, drawings and specifications.

DIAPHRAGM ACTION OF CELLULAR STEEL FLOOR AND ROOF DECK CONSTRUCTION

1. INTRODUCTION

Building structures must resist the lateral forces to which they are subjected. The nature and magnitude of these forces depends on the location of the building site and the geometry and orientation of the building. The most common lateral force to be considered is that due to wind acting on the building surfaces, the magnitude being a function of the height, shape and area of the building as well as the wind pressure which varies with the velocity and density of the air mass. Earthquake or seismic shock is a second potential generator of lateral force in certain areas of the country, the force in this case being due to the inertia of the building mass when subjected to ground motion.

The value of both wind and seismic forces to be considered in design are usually specified in the applicable building regulations and may vary considerably from locality to locality. The National Building Code of Canada provides a comprehensive treatment of such forces and should be consulted in the absence of specific legal requirements.

Lateral forces may also be due to other effects such as moving loads, assymetric loading, sway, etc., which must be considered in the design.

The presence of lateral forces requires that at each floor and roof level there be an adequate horizontal distribution system in the plane of the floor or roof so as to distribute the imposed lateral forces to columns, walls or vertical bracing systems and thence eventually to the ground. Such a horizontal distribution system may be effected by horizontal bracing installed specifically for that purpose; by diaphragm action of the floor or roof elements alone; or by a combination of these methods.

Cellular steel floor and roof decks, combining high strength with light weight are ideally suited for use as lateral-force-distributing diaphragms in addition to their primary load carrying function. When so used, the cellular steel floor or roof deck and its method of attachment must be checked to ensure that diaphragm action will occur. In many cases the strength and stiffness of the diaphragm alone will obviate the need for an independent horizontal bracing system. The permissible lateral deflection of a diaphragm will be governed by the type of construction and the choice of materials for walls, partitions and other elements which may be affected by lateral movement.

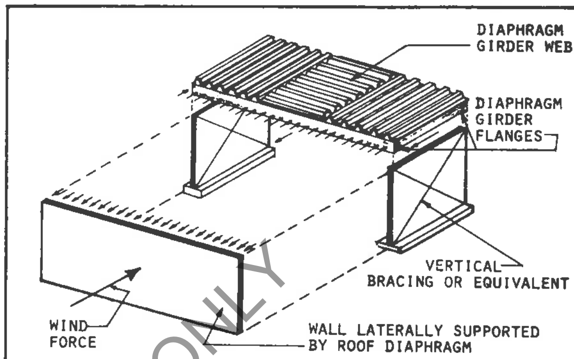


FIG. 1 - DIAPHRAGM GIRDER ANALOGY

Note: Direction of flutes is optional. See Section 2, final paragraph.

2. DESIGN ANALOGY

As depicted by Figure 1, the diaphragm is analogous structurally to a plate girder having its web (the cellular steel floor or roof deck) in a horizontal plane in order to efficiently resist the lateral forces applied to the building. The flanges of the girder consist of the perimeter supporting members required on all four sides of the diaphragm. The girder is considered to span between locations of vertical support capable of transferring the horizontal forces into the vertical plane (i.e. shear walls, braced or moment-resisting bents). To simplify design the web is considered to resist only shear forces whereas the flanges (the perimeter members parallel to the span of the girder) are assumed to resist only flexural forces, determined by the relationship:

$$P_a = M/D$$

where P_a = force in tension or compression

M = girder bending moment at the particular point investigated

D = distance between centre lines of perimeter members, measured perpendicular to the span of the girder

The ability of the web to resist shear depends not only on the cross-sectional area and profile of the steel deck but also on the type and spacing of connections. The design values given herein are applicable irrespective of whether the span direction of individual deck units runs parallel or perpendicular to the span of the diaphragm girder, or in a combination.

3. DIAPHRAGM DEFLECTION

The two components of deflection of a steel deck diaphragm are the flexural component and the web component. They are considered to be directly additive, thus total deflection is the arithmetic sum of the two component deflections. (See Fig. 2)

(1) **Flexural Deflection** ($\triangle_F$)
The flexural deflection of a diaphragm is determined by conventional beam deflection formulas as shown in Table I. The moment of inertia of the diaphragm is computed using only the flange areas; except when concrete cover over the steel deck is used, then the transformed area of concrete cover may also be utilized.

(2) **Web Deflection** ($\triangle_W$)
The web deflection of a diaphragm depends not only on the shear deformation of the deck profile but also on the flexibility of the attachment of the deck to the framing members and the amount of slip of the side lap connection of the deck units. It has been determined that for practical purposes, web deflection is directly proportional to the product of the average shear (v) per unit width of the diaphragm web and the length of the diaphragm measured from the point of support to the point at which deflection is to be determined. The proportional constant (F), termed the "Flexibility Factor" of the diaphragm, is measured in micro-inches per pound. It represents the average micro-inches a diaphragm web will deflect in a span of one foot under a shear of one pound per foot of diaphragm width.

Web deflection formulas are given in Table I. Design values of the Flexibility Factor for 1½ inch deep steel deck are given in Table III.

4. SHEAR DISTRIBUTION

Shear distribution is governed by the flexibility of the diaphragm as measured by the flexibility factor (F). Diaphragm flexibility ranges from "rigid" ($F<1$) to "very flexible" ($F>150$). Diaphragms in which the flexibility factor exceeds 150 are limited in effectiveness.

Table II provides a guide to the practical limits of diaphragms, based on flexibility considerations.

With a rigid diaphragm, lateral forces applied to the diaphragm are assumed to be distributed to the vertical supporting members in proportion to the relative stiffness of the vertical supports, that is, the diaphragm is assumed to be rigid relative to its supports. In the case of a flexible diaphragm however, the vertical supports are assumed to be rigid relative to the diaphragm. Thus with a flexible diaphragm the lateral loading is assumed to be distributed to the vertical supports in proportion to the contributing diaphragm areas, neglecting continuity effects. One or the other method of distribution is usually used for diaphragms intermediate between rigid and flexible depending on which idealized situation is more closely approximated.

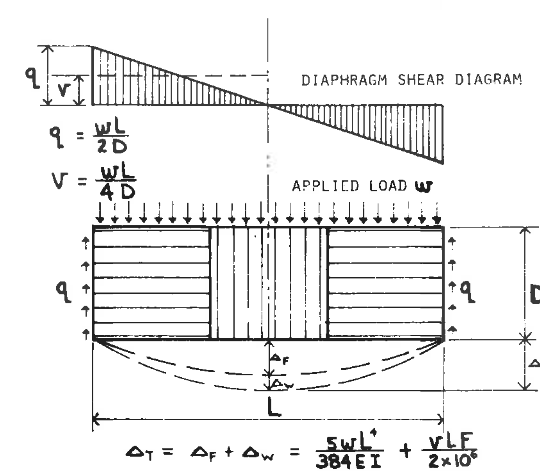


FIG. 2 - DIAPHRAGM DEFLECTION

TABLE I
FORMULAS FOR MAXIMUM DIAPHRAGM DEFLECTION

DIAPHRAGM SPAN CONDITION	DIAPHRAGM LOADING CONDITION	FLEXURAL DEFLECTION $\triangle_F$ (inches)	WEB DEFLECTION $\triangle_W$ (inches)
Simple	Uniform	$\frac{5WL^4\,(12)^3}{384EI}$	$\frac{v\,L\,F}{2 \times 10^6}$
	Load applied at centre	$\frac{PL^3\,(12)^3}{48EI}$	
	Load applied at each third point	$\frac{23PL^3(12)^3}{648EI}$	
Cantilever	Uniform	$\frac{Wa^4(12)^3}{8EI}$	$\frac{v\,a\,F}{10^6}$
	Load applied at free end	$\frac{Pa^3(12)^3}{3EI}$	
TOTAL DEFLECTION $\triangle_T = \triangle_F + \triangle_W$			

E = Modulus of elasticity of steel (29.5×10^6 psi)
I = Moment of inertia of diaphragm flange (perimeter) members about centroidal axis of diaphragm (in^4)
L = Span of simple beam (ft)
a = Span of cantilever beam (ft)
P = Concentrated lateral load (lbs)
W = Uniform lateral load (lb/ft)
F = Flexibility factor (10^6in/lb)
v = Average shear (per unit of diaphragm width) along length L/2 or a (lb/ft)

TABLE II
DIAPHRAGM LIMITS BASED ON FLEXIBILITY CONSIDERATIONS

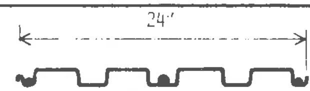
DIAPHRAGM FLEXIBILITY CATEGORY	RANGE OF FLEXIBILITY FACTOR F (10^6 in/lb)	MAXIMUM SPAN if laterally supporting Masonry or Concrete Walls (ft)	MAXIMUM DIAPHRAGM SPAN/DEPTH RATIOS			
			No Torsion On Structure		Torsion On Structure	
			Masonry or Concrete Walls	*Flexible Walls	Masonry or Concrete Walls	*Flexible Walls
Flexible	70–150	200	2 or as required for deflection	3	Not Used	2
Semi-Flex.	10–70	400	2.5 or as required for deflection	4	As required for deflection	2.5
Semi-Rigid	1–10	Not limited	3 or as required for deflection	5		3
Rigid	Less than 1	Not limited	as required for deflection	Not limited		3.5

* For cantilever diaphragms the recommended maximum diaphragm span/depth ratio is one half of the values shown.

TABLE III

ALLOWABLE DIAPHRAGM SHEAR, q, (lb/ft) AND FLEXIBILITY FACTOR, F, (10^6in/lb) FOR 1½ INCH DEEP STEEL DECK

No. of Transverse Welds/Unit=3 per support



SPAN (ft.)	Side Lap ▶	Button Punch 12 in. o.c.			Button Punch 18 in. o.c.			Button Punch 24 in. o.c.		
	Deck ga.[1] ▶	22	20	18	22	20	18	22	20	18
5	q	336	495	817	300	453	766	282	431	740
	F	$17+222R$[2]	$13+128R$	$9+54R$	$19+222R$	$14+128R$	$9+54R$	$20+222R$	$15+128R$	$10+54R$
6	q	295	433	709	259	390	657	240	368	630
	F	$19.5+185R$	$15.7+107R$	$10+45.2R$	$22+185R$	$17.2+107R$	$10.4+45.2R$	$23.2+185R$	$18+107R$	$11+45.2R$
7	q	266	386	633	230	343	580	212	321	554
	F	$21+158R$	$17+91R$	$12+38R$	$24+158R$	$19+91R$	$12+38R$	$26+158R$	$20+91R$	$13+38R$
8	q	224	350	577	208	307	520	190	286	496
	F	$24.2+138R$	$19.3+80.6R$	$13.4+33.9R$	$27.2+138R$	$21.6+80.6R$	$14.5+33.9R$	$29.4+138R$	$23.1+80.6R$	$15.3+33.9R$

No. of Transverse Welds/Unit=5 per support

SPAN (ft.)	Side Lap ▶	Button Punch 12 in. o.c.			Button Punch 18 in. o.c.			Button Punch 24 in. o.c.		
	Deck ga.[1] ▶	22	20	18	22	20	18	22	20	18
5	q	418	624	1074	382	580	1019	364	559	992
	F	$16+55R$[2]	$12+32R$	$8+13R$	$17+55R$	$13+32R$	$8+13R$	$17+55R$	$13+32R$	$8+13R$
6	q	360	533	926	326	490	870	308	469	842
	F	$18+46.3R$	$14.2+25.7R$	$9.5+11.3R$	$20+46.3R$	$15.2+25.7R$	$10+11.3R$	$21+46.3R$	$15+25.7R$	$10.2+11.3R$
7	q	322	469	821	286	426	764	268	404	735
	F	$20+39R$	$16+22R$	$11+9R$	$22+39R$	$17+22R$	$11+9R$	$24+39R$	$18+22R$	$12+9R$
8	q	290	421	733	256	378	675	238	357	546
	F	$22.2+34.7R$	$18+20R$	$12.2+8.5R$	$25+34.7R$	$19.6+20R$	$13.1+8.5R$	$27+34.7R$	$21+20R$	$13.6+8.5R$

Notes:

(1) For related steel core thickness see Table IV.
(2) R=ratio of span of deck unit to average length of deck unit supplied.

5. DATA FOR DIAPHRAGM DESIGN

Data for the design of a diaphragm employing the common type of 1½ inch deep steel deck units, without concrete cover, are given in Table III. Allowable diaphragm shear (q) is expressed in pounds per foot and values of the flexibility factor(F) are given in micro-inches per unit of shear per unit of span (10^6in/lb). When concrete cover is placed to a minimum depth of 2½ inches over the top of the deck, the flexibility factor of the resulting combination is normally less than unity and the diaphragm may be considered as being within the rigid category of Table II.

The listed values of allowable diaphragm shear, q, and flexibility factor, F, have been extrapolated from various large scale test results in which a safety factor of 3 has been incorporated.

6. ROTATION OF DIAPHRAGM DUE TO TORSION OF THE STRUCTURE

When lateral forces are applied to a diaphragm such that the centroid of these forces does not coincide with the centroid of resistance of vertical supports, the diaphragm will be subject to rotation about a vertical axis. This rotation creates torsional stresses in the supporting structure which are superimposed on the stresses existing if no rotation had occurred. Such stresses may be determined by the usual methods of torsion analysis, taking into account the centre of rotation of the structure as a whole and the torsional stiffness of the vertical supporting members, taken as a group.

Rotation effects can be accommodated by rigid and semi-rigid diaphragms, including all cantilevered diaphragms, employed in conjunction with steel frames. The use of flexible diaphragms ($F > 70$) is not recommended where the diaphragm is subject to rotation. The possibility of rotation should be avoided in all diaphragms supported on unit masonry. (See Table II for additional guide information).

7. CONNECTION DETAILS

7.1 General

In order for the cellular steel floor or roof deck units to perform the required diaphragm action, it is important that all units as well as the steel frame be integrated structurally. This is achieved through proper connection details. For specific installations the design of these details is considered to be within the area of responsibility of the structural engineer. For deck profiles and conditions other than given in Table III, it is recommended that the structural engineer consult the deck supplier for the properties of the diaphragm under consideration. The final design details and information should be shown clearly on the structural drawings or in the specifications.

In general, the strength and stiffness of a profiled deck diaphragm is governed by the type of connection in three main areas. Therefore, the connection details in these areas require special consideration.
(See Figs. 3A and 3B)

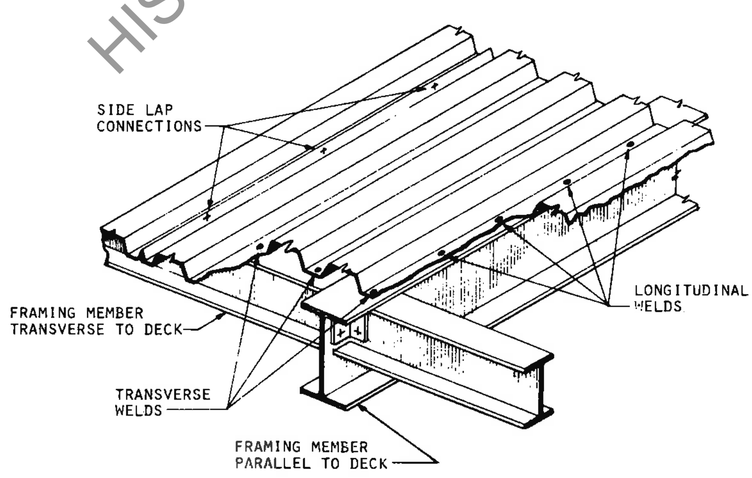


FIG. 3A - TYPICAL CONNECTION DETAILS-BEAM AND GIRDER FRAMING

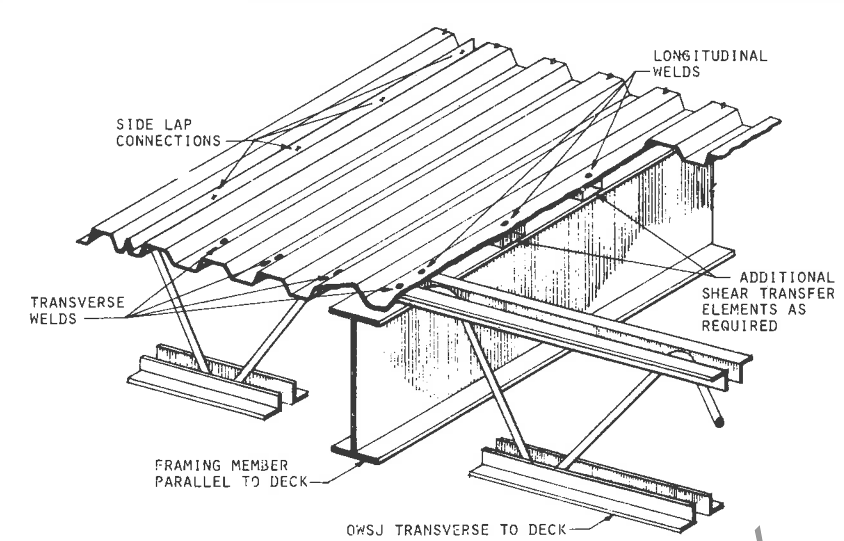


FIG. 3B - TYPICAL CONNECTION DETAILS-OPEN WEB STEEL JOIST FRAMING

7.2 Welding of the Deck to Structural Framing

The standard welds are generally referred to as puddle welds with a minimum effective diameter of ½ inch. The welding process requires melting through the steel deck, thus fusing the deck metal to the supporting structural member.

According to the location, puddle welds are divided into two groups, namely (a) the *transverse* welds, and (b) the *longitudinal* welds.

(a) The *transverse* welds are located at the valleys of the deck corrugation where the deck is in contact with the supporting member. Thus the spacing is governed by the profile of the deck itself. It is recommended that the required number of transverse welds per deck unit be specified.

(b) The *longitudinal* welds are also located at the valley of the deck corrugation, except in this case the framing member is parallel to the corrugation. In some types of construction the framing members parallel to the span of the deck are at a lower elevation than the framing members perpendicular to the span of the deck. For example, open web steel joists usually rest on the top of supporting beams. In such cases, direct welding of the deck to parallel framing members is not possible. Therefore, special attention must be paid to the connection detail to ensure that adequate means of transferring the shear from the deck to the framing member, or vice versa, are provided. The spacing of longitudinal welds should be no greater than the maximum distance obtained from the applicable spacing factors given in Table IV.

7.3 Side Lap Connection of Deck Units

The shear induced in the plane of the diaphragm is transmitted from one deck unit to another through the side lap connections. The most common form of connection is the interlocking male and female side lap. There are two methods of fastening. One is to button punch the male and female side lap together; the other is to seam-weld. The button punch will tend to slip when subjected to high stress and therefore is better suited to situations where diaphragm shear is moderate. Seam welding is produced by fusing the metal layers in the side lap together. This type of attachment offers a stronger, stiffer connection and is more suitable for high diaphragm shear. However, it is limited to 20 gauge (0.036 in) or thicker material to ensure successful welding.

The spacing of side lap connections influences diaphragm shear capacity (See Table III) and the appropriate spacing should be specified.

When concrete is placed to a minimum thickness of 2½ inches above the steel deck, the concrete slab itself is capable of transmitting the diaphragm shear across the side lap of the deck units. Therefore, no special side lap connection need be provided other than the recommended minima. (See CSSBI Standards for Steel Roof Deck and Standards for Cellular Steel Floor)

7.4 Perimeter Framing Members

The members at the perimeter of the deck diaphragm which function as flanges of the analogous plate girder are required to resist direct tensile or compressive force resulting from flexure of the diaphragm. It is important that these members be positioned so as to allow welding of the steel deck directly to them. Special attention must be given to the interconnection of a line of these members to ensure that they will act as a continuous flange over the entire length of the diaphragm. Perimeter members which support loads additional to those resulting from diaphragm behaviour should be designed for the applicable combined stress situation.

7.5 Intermediate Framing Members

Cellular steel floor and roof deck are often supported by intermediate framing members in addition to those comprising the perimeter members and vertical supporting members of the diaphragm as a whole. Since the intermediate framing members are usually the means by which lateral loads acting on the structure are transferred into the diaphragm, the welding of the deck to intermediate framing members is just as important to proper performance of the diaphragm as the welding of the deck to the perimeter members. For safety and simplicity, it is recommended that the spacing of the welds connecting deck units to intermediate framing members be the same as required for connecting deck units to the perimeter framing members. (i.e. based on the computed diaphragm shear, q, as derived in Fig. 2).

7.6 Welding Recommendations

The following is recommended for inclusion in job specifications covering the use of cellular steel floor and/or steel roof deck as diaphragms:

"Welding shall be done only by qualified welders who shall make practice welds prior to actual job welding. Practice welds shall be made on the deck to be used to check adequacy of the welding rod amperage and burn-off rate to produce satisfactory fusion for the various welds required. Both the practice welds and actual job welds shall be inspected by the steel deck erector as to size and spacing and tested by pry tests to assure metal-to-metal fusion".

TABLE IV

LONGITUDINAL WELD SPACING FACTORS

Deck Gauge	Nominal Core Thickness	Factor ‡
22	.030 in.	1380
20	.036	1650
18	.048	2200
16	.060	2750
20–20	.036/.036	2750
20–18	.036/.048	3120

‡ Divide factor in this Table by the actual diaphragm shear to get spacing of welds in feet to framing members parallel to flutes. The spacing in no case should exceed 4 ft.

8. DIAPHRAGM DESIGN EXAMPLES

8.1 Example One—Roof Diaphragm for Steel Frame Building

Given: A one-storey steel frame building with $1\frac{1}{2}$ inch 22 ga. steel roof deck and steel sandwich panel cladding. (See Fig. 4). Wall height is 16 ft. and design wind pressure is 20 psf.

Lateral deflection of the diaphragm will not adversely affect the cladding. Therefore diaphragm deflection is assumed to be not critical and limiting stress is the design criterion.

Required: To determine the number of required transverse welds per deck unit; side lap connection spacing; longitudinal weld spacing; and the size of the diaphragm perimeter members parallel to the length of the building.

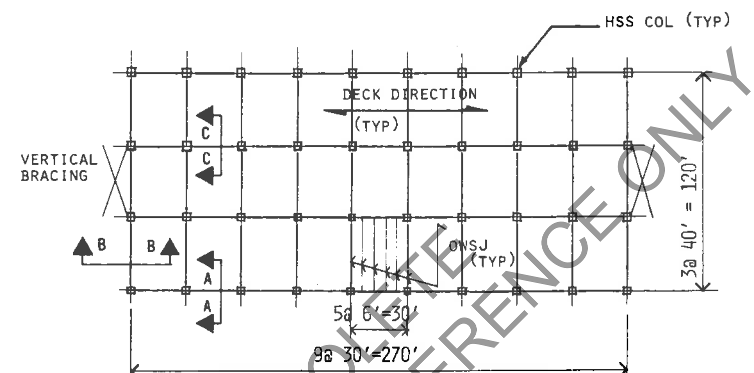


FIG. 4 – BUILDING PLAN – DESIGN EXAMPLE ONE

Solution:

PROCEDURE	CALCULATIONS
(1) Determine the lateral load due to wind acting on the roof diaphragm. Assume this to be a uniform load, W, equal to the wind pressure times half the wall height.	(1) $W = 20 \times \frac{1}{2} \times 16 = \underline{160}$ lb/ft
(2) Determine the maximum diaphragm shear, q, with diaphragm spanning between end-walls. (See Fig. 2) $q = \frac{WL}{2D}$	(2) $q = \frac{160 \times 270}{2 \times 120} = \underline{180}$ lb/ft
(3) Determine the required transverse weld spacing and side lap connection spacing to resist the maximum diaphragm shear, q, (See Table III)	(3) From Table III, for $1\frac{1}{2}$ inch deck, 22 ga., 6'-0 span, using 3 transverse welds/unit and side lap button punched at 24 in. o.c., allowable diaphragm shear is $\underline{240}$ lb/ft $q = 180 < 240$ $\underline{\text{OK}}$
(4) Determine the longitudinal weld spacing, a_w, required. (See Table IV)	(4) From Table IV, factor is 1380 $a_w = \frac{1380}{180} = 7.67$ ft; $7.67 > 4.0 = a_w$ max. Use $a_w = \underline{4.0}$ ft

Solution: (cont'd)

PROCEDURE	CALCULATIONS

PROCEDURE

(5) Select a trial perimeter member.

Determine:

(a) axial stress due to diaphragm flexure (f_a).

$$M = \frac{WL^2}{8}$$

$$P_a = \frac{M}{D}$$

$$f_a = \frac{P_a}{A}$$

(b) local bending stress due to wind load acting on perimeter member over its unsupported length equal to the joist spacing of 6'-0. Assume the two perimeter members transverse to the wind direction each take one half the total lateral load on the diaphragm

$$W_1 = \tfrac{1}{2} W_2$$

$$M_1 = \frac{W_1 L_1}{10} \text{ (for cont. member)}$$

$$f_b = \frac{M_1}{S}$$

(c) Combined stress

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$$

CALCULATIONS

(5) Try L 3 x 3 x 3/16 continuous
$A = 1.09$ in.^{2}; $r = 0.94$ in.; $S = 0.44$ in^3

(a) $M = \dfrac{160 \times 270^2}{8} = 1{,}458{,}000$ ft-lb

$P_a = \dfrac{1{,}458{,}000}{120} = 12{,}150$ lb

$f_a = \dfrac{12{,}150}{1.09} = 11{,}120$ psi

(b) $W_1 = \tfrac{1}{2} \times 160$
$= \underline{80}$ lb/ft

$M_1 = \dfrac{80 \times 6^2}{10}$
$= \underline{288}$ ft-lb

$f_b = \dfrac{288 \times 12}{0.44}$
$= \underline{7855}$ psi

(c) $\dfrac{KL_1}{r} = \dfrac{1 \times 6 \times 12}{0.94} = 77$

whence $F_a = \underline{18{,}700}$ psi
(assuming $F_y = 44{,}000$ psi for L)

$F_b = 0.6\ F_y$
$= 0.6 \times 44{,}000 = \underline{26{,}400}$ psi

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{11{,}120}{18{,}700} + \frac{7{,}855}{26{,}400}$$

$= 0.595 + 0.297$
$= \underline{0.892} < 1.0$ OK

Use L 3 x 3 x 3/16 continuous

Summary:

1½ in. 22 ga. steel roof deck — 5 span @ 6'-0
No. of transverse welds per deck unit — 3
Side lap connection — button punch @ 24 in. o.c.
Longitudinal weld spacing — 48 in.
Perimeter framing member — L 3 x 3 x 3/16 continuous
Typical details — See Figs. 5, 6 and 7.

8.2 Example Two — Roof Diaphragm for Masonry Wall Building

Given: A one-storey masonry wall building with 1½ inch 22 ga. steel roof deck. (See Fig. 8). The loading, spans and general design are the same as for Example One, with the exception of the 10 inch thick masonry perimeter walls. In this example the roof diaphragm is tied to, and provides lateral support for the masonry walls. Therefore diaphragm deflection must be limited so as to prevent excessive stresses in the walls due to lateral displacement. It is assumed that the limiting deflection of a masonry wall is given by the expression [1]

$$\triangle_{max} = \frac{100h^2F_c}{E_w t_w}$$

Where $\triangle_{max}$ = maximum permissible deflection of the wall (in.)

h = height of wall between horizontal supports (ft.)

F_c = allowable flexural compressive stress of wall material (psi.)

E_w = modulus of elasticity of wall material (psi.)

t_w = thickness of wall (in.)

(1) *"Recommended Practice for Engineered Brick Masonry"—Structural Clay Products Institute—1969.*

Required: To determine the number of transverse welds per deck unit; side lap connection spacing; longitudinal weld spacing; and the size of the diaphragm perimeter members parallel to the length of the building.

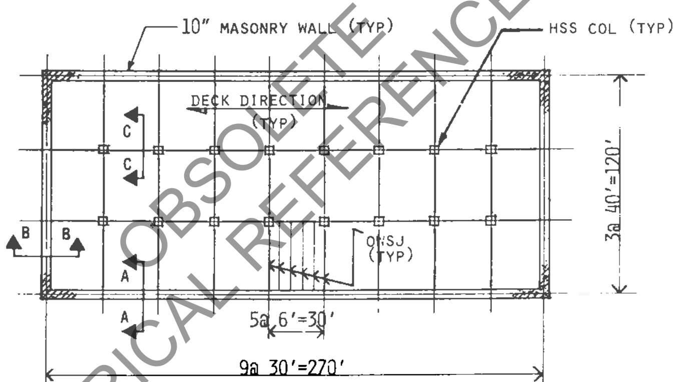


FIG. 8 - BUILDING PLAN - DESIGN EXAMPLE TWO

Solution:

PROCEDURE	CALCULATIONS
(1) Follow steps 1 to 5 inclusive of Example One. This results in a diaphragm design which is sufficiently strong but not necessarily with sufficient stiffness.	(1) See Example One
(2) Determine limiting deflection of the 16 ft. high masonry wall. Assume 1500 psi. concrete block units with types M or S mortar. (See Tables 4.4.3 C and D National Building Code of Canada 1970) $\triangle_{max} = \frac{100h^2F_c}{E_w t_w}$ in.	(2) $t_w = 10$ in. $E_w = 1.15 \times 10^6$ psi. $F_c = 0.32 \times 1{,}150 = 368$ psi. $h = 16$ ft. $\triangle_{max} = \frac{100 \times 16^2 \times 368}{1.15 \times 10^6 \times 10} = \underline{0.820}$ in.

Solution: (cont'd)

PROCEDURE — CALCULATIONS

Procedure

(3) Compute diaphragm deflection. (See Table I)

(a) Flexural deflection

A = area of perimeter member in^2.

$$I = \frac{AD^2(12)^2}{2} \text{ in.}^4$$

$$\Delta_F = \frac{5wL^4(12)^3}{384\ E\ I} \text{ in.}$$

(b) Web deflection

$$v = \frac{WL}{4D} \text{ lb/ft}$$

R = ratio of deck span to deck length

F = flexibility factor (Table III)

$$\Delta_w = \frac{vLF}{2 \times 10^6} \text{ in.}$$

(c) Total deflection

$$\Delta_T = \Delta_F + \Delta_W$$

Calculations

(3)

(a) For L 3 x 3 x 3/16; A = 1.09 in.^{2}

$$I = \frac{1.09 \times 120^2 \times 12^2}{2} \text{ in.}^4$$

$$\Delta_F = \frac{5 \times 160 \times 270^4 \times 12^3 \times 2}{384 \times 29.5 \times 10^6 \times 1.09 \times 120^2 \times 12^2}$$

$= \underline{0.573}$ in.

(b) $v = \dfrac{160 \times 270}{4 \times 120} = \underline{90}$ lb/ft

$R = \dfrac{6}{30} = \underline{0.2}$

$F = 23.2 + 185\ (0.2) = \underline{60.2}$

$$\Delta_w = \frac{90 \times 270 \times 60.2}{2 \times 10^6}$$

$= \underline{0.731}$ in.

(c) $\Delta_T = 0.573 + 0.731 = \underline{1.304}$ in.

$\Delta_T > \Delta_{max}$ (0.820 in.) $\underline{\text{NG}}$

Diaphragm stiffness must be increased.

Procedure

(4) Re-design diaphragm so that $\Delta_T \leq \Delta_{max}$

Calculations

(4) Try:
L 3 x 3 x ¼ as perimeter member;
5 transverse welds per deck unit;
side lap—button punch @ 12'' o.c.

Procedure

(5) Repeat step 3 using new values.

Calculations

(5)

(a) $$\Delta_F = \frac{5 \times 160 \times 270^4 \times 12^3 \times 2}{384 \times 29.5 \times 10^6 \times 1.44 \times 120^2 \times 12^2}$$

$= \underline{0.435}$ in.

(b) $F = 18 + 46.3(0.2) = \underline{27.3}$

$$\Delta_w = \frac{90 \times 270 \times 27.3}{2 \times 10^6}$$

$= \underline{0.332}$ in.

(c) $\Delta_T = 0.435 + 0.332 = \underline{0.767}$ in.

$\Delta_T < \Delta_{max}$ (0.820 in.) $\underline{\text{OK}}$

Diaphragm is adequate

Summary:

1½ in. 22 ga. steel roof deck—5 span@6'-0
No. of transverse welds per deck unit—5
Side lap connection—button punch @ 12 in. o.c.
Longitudinal weld spacing—48 in.
Perimeter framing member—L 3 x 3 x ¼ continuous
Typical details—See Figs. 7, 9 and 10.

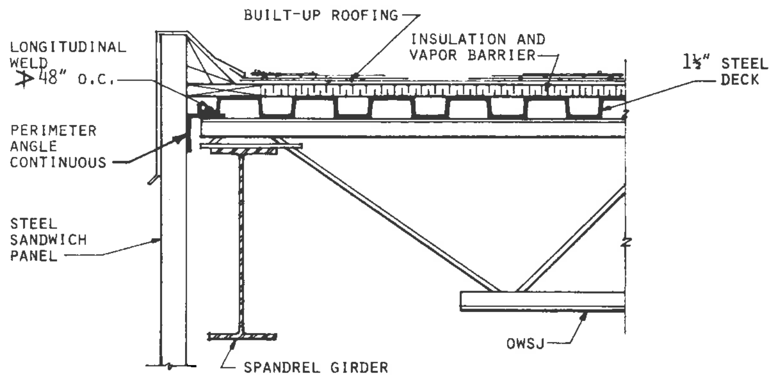


FIG. 5 - SECTION A-A - DESIGN EXAMPLE ONE

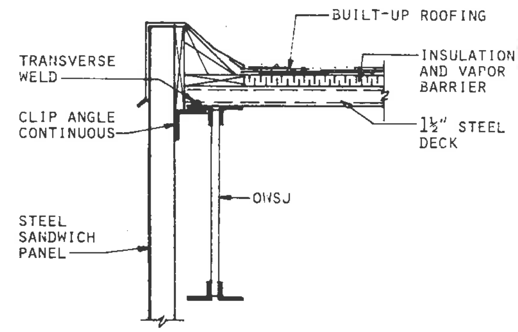


FIG. 6 - SECTION B-B - DESIGN EXAMPLE ONE

Sections

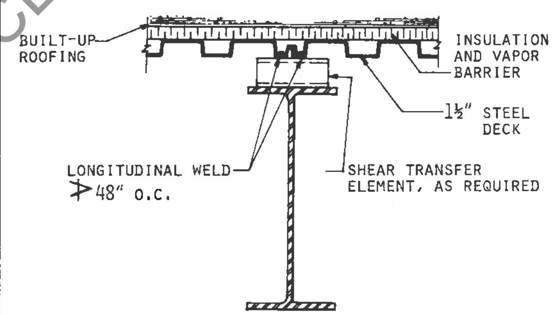


FIG. 7 - SECTION C-C - DESIGN EXAMPLES ONE AND TWO

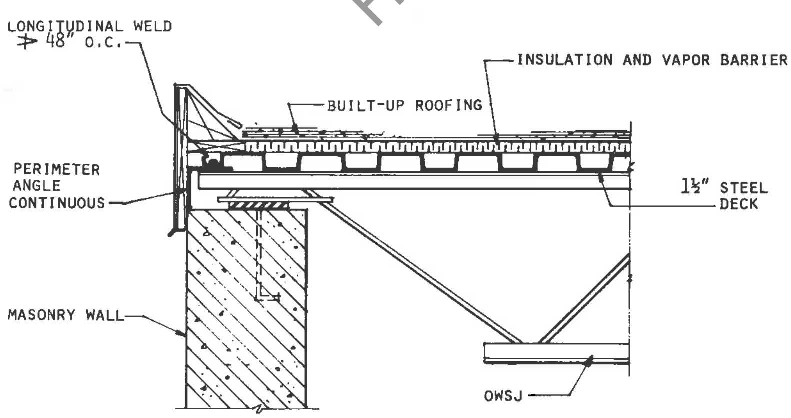


FIG. 9 - SECTION A-A - DESIGN EXAMPLE TWO

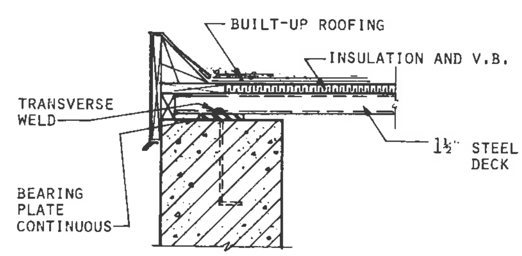


FIG. 10 - SECTION B-B - DESIGN EXAMPLE TWO