ANALYSIS AND DESIGN OF THE STIFFENING SYSTEMS FOR A RECTANGULAR STEEL BIN

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ABSTRACT

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ABSTRACT

The present report deals with the analysis and design of a rectangular steel bin structure, under the influence of its internal forces. The analysis is based on the most recently available practice procedures, regarding eccentric pyramidal hopper outlets.

Two types of stiffening systems are examined in the design of the bin walls. The conventional one, considering the elastic behaviour of plates and stiffening members, and the one based on the application of the orthotropic plate theory for triangular load distribution. The results of the two stiffening options favor the application of the orthotropic plate concept, because it leads to a better and greater utilization of the provided material.

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NOTATIONS

NOTATIONS

A	area of cross-section; horizontal panel dimension
B_i	vertical panel dimension
C	compression force
C_d	overpressure factor
D_x, D_y	flexural rigidities of a plate in the x- and y-directions, respectively
D_{ij}	distribution coefficient
E	modulus of elasticity
F_a	allowable axial stress
F_b	allowable bending stress
F_y	yield stress
G	hoppr own weight
H	height of the bin walls
I	section moment of inertia
K	horizontal-to-vertical pressure ratio constant
L_{ef}	effective length
M	moment
P	pressure
R	hydraulic radius
S	section modulus; spacing dimension
T	tensile force
W	total vertical load
V	shear force

Y,X,Z	axis designation
a	the smaller of the bin wall horizontal dimensions
b	the larger of the bin wall horizontal dimensions
b_e	effective width of the plate on one side of the stiffener
c	subscript for compression; subscript for corner location
d	total height of cross-section
e_a, e_b	eccentricities regarding sides a and b, respectively
f	subscript for friction load; calculated stress
h	height of the web; subscript for horizontal direction
i,j	designation of beam supports
k	constant related to the dimensions of a panel or frame
ℓ	length of span
n	upper summation limit
p	uniformly distributed load
q	linearly distributed friction load
r	section radius of gyration
s	spacing of stiffeners
t	thickness of plating; subscript for tension
w	line load; thickness of the web
v	subscript for vertical direction
x,y	dimensions regarding X,Y-axis, respectively

Δ	deflection
α, β	angles of hopper walls inclination
γ	constant; specific weight
ε	eccentricity regarding C.G. of hopper
λ	effective width of plating on one side of the stiffener
μ'	coefficient of friction
ν	Poisson's ratio
π	constant (3.14)
σ	stress
τ	shear stress
ϕ	angle of internal friction

CHAPTER I

INTRODUCTION

CHAPTER I

INTRODUCTION

Rectangular steel bins with eccentric outlets are in the category of tubular structures which are subjected to internal loads of severe adversity. Research still continues for the exact analysis of such loads. In the present report, however, based on the quite accurate and popular Janssen's method, as well as widely applied practice procedures regarding the amplification of loads, due to the eccentricity of the outlet; the computed loads inherit all the necessary safety provisions.

The detailed analysis and design of the conventional stiffening system is based on a value of an effective width, considering 40 times the thickness of the participating plate.

On the other hand, in the orthotropically stiffened system, the effective width is based on existing valid theories which may be safely applied considering the modern welding techniques, which provide adequate continuity of stiffeners - plate cross-sections. Such an option, applied with the basic concept of the Pelikan-Esslinger method, for the analysis of orthotropic decks, results in a substantial savings in material.

The foregoing analysis of the stiffening systems considers the elastic behaviour, as well as small deflections of plates.

Finally, the drawings of the steel bin give the details of the design.

CHAPTER II

DESIGN CONCEPT AND STRUCTURAL SPECIFICATIONS

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DESIGN CONCEPT AND STRUCTURAL SPECIFICATIONS

2.1 TYPE OF STRUCTURE

The structure to be designed is a rectangular steel bin, as shown in Figure 2.1.

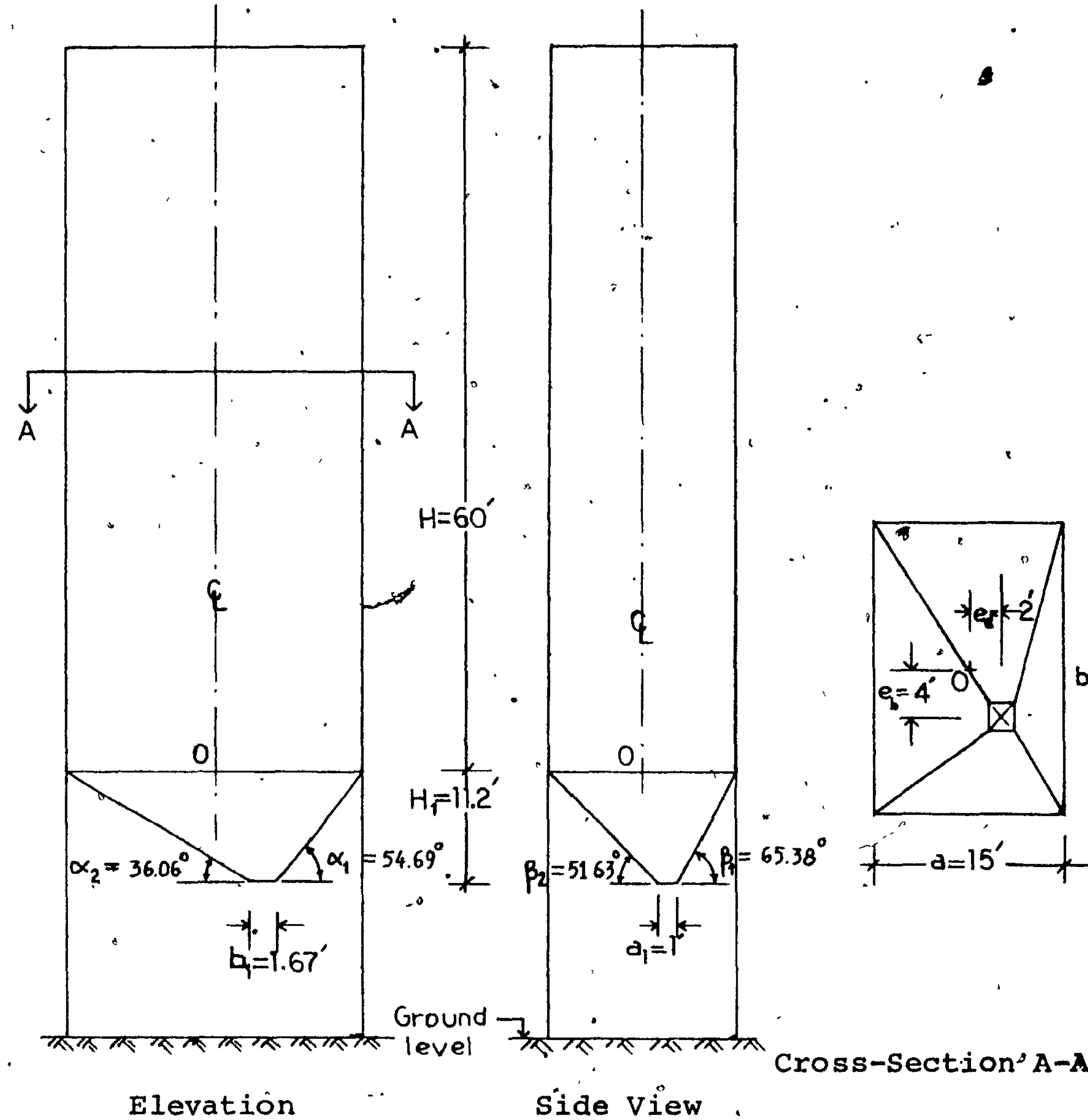

FIG. 2.1 The steel bin and its dimensions

The stored material is sand, having a specific weight: γ = 100 lbs/ft^3; considered constant over the full height of the bin [3].

The coefficient of friction between the stored material and the walls of the steel bin is: μ' = 0.5 [3].

The angle of internal friction in the sand is: $\phi = 35^o$ [3].

2.2 HOPPER GEOMETRY

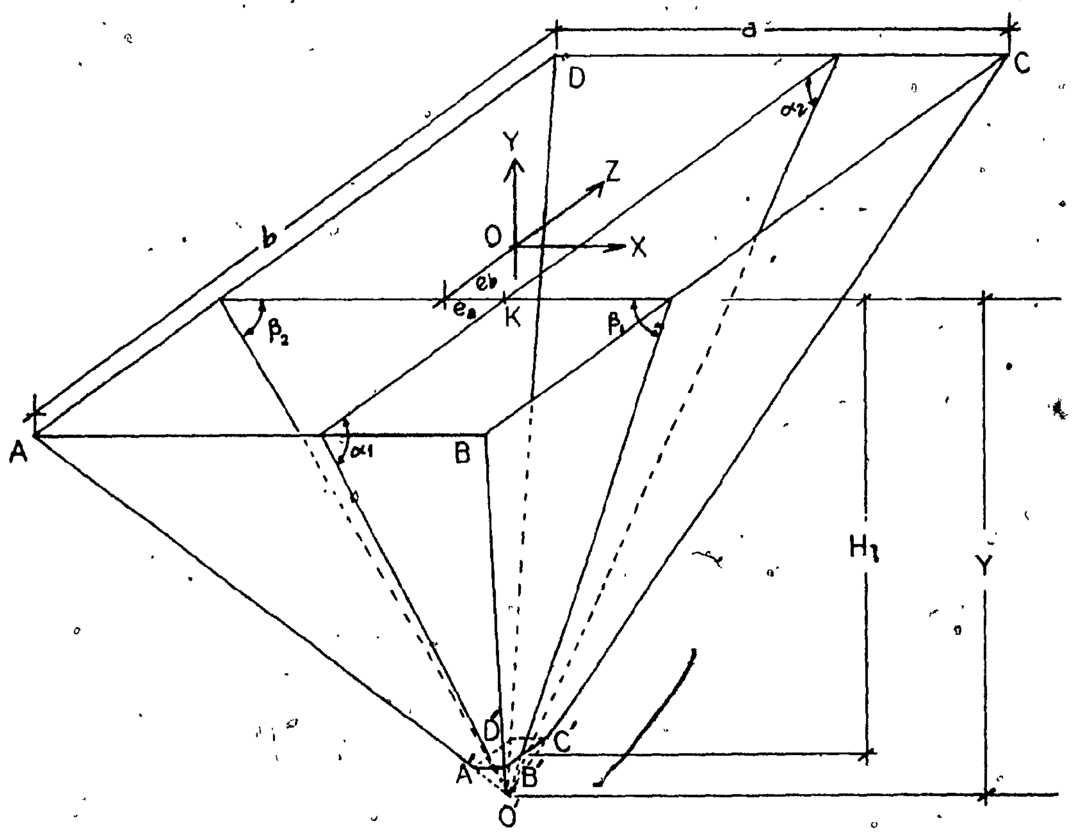

FIG. 2.2 Hopper isometric view

For the hopper as shown in Figure 2.2, the following expressions are valid:

(i) For any plane ⊥ Y axis: $\frac{A'B'}{AB} = \frac{D'C'}{DC}$ ∴ $A'B' = D'C' = a_1$

Similarly: $\frac{A'D'}{AD} = \frac{B'C'}{BC}$ ∴ $A'D' = B'C' = b_1$

(ii) The height Y may be obtained from one of the following expressions:

$$Y = (b/2-e_a)\tan\alpha_1 = (b/2+e_a)\tan\alpha_2 = (a/2-e_b)\tan\beta_1 =$$

$$= (a/2\ e_b)\tan\beta_2$$

Rearranging and substituting the values of a,b,e_a and e_b, the following expressions may be obtained:

$\tan\alpha_1 = Y/8.5$; $\tan\alpha_2 = Y/16.5$; $\tan\beta_1 = Y/5.5$ and $\tan\beta_2 = Y/9.5$.

(iii) For $a_1 = A'B'$ and $b_1 = B'C'$, from similar triangles:

$$\frac{Y-H_1}{Y} = \frac{b_1}{b} = \frac{a_1}{a}$$

Considering an (1'-0") × (1'-8") outlet for the hopper, then: $a_1 = 1.0'$ and $b_1 = 1.67'$. Assuming: $H_1 = 11.2'$ it follows that:

$$\frac{Y-11.2}{Y} = \frac{1}{15}\ ;\ \text{or}:\quad Y = 12'\text{-}0".$$

Substituting the latter in the expressions of 2.2, the values of the inclined wall angles are obtained:

$$\alpha_1 = \tan^{-1}\frac{12}{8.5} = 54.69^o; \quad \alpha_2 = \tan^{-1}\frac{12}{16.5} = 36.03^o$$

$$\beta_1 = \tan^{-1}\frac{12}{5.5} = 65.38^o; \quad \beta_2 = \tan^{-1}\frac{12}{9.5} = 51.63^o$$

All the computed angles of inclined hopper walls are in the usual practice range, i.e., $\alpha > \phi$.[3]

2.3 LOCATION OF THE CENTER OF GRAVITY IN A HOPPER

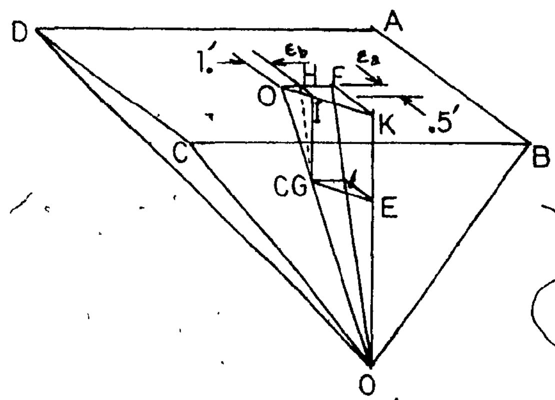

FIG. 2.3 Center of gravity of hopper

The center of gravity of the hopper is located on the locus of all cross-sectional centers of gravity, line OO', at 1/4 of the height measured from base ABCD.

From Fig.2.3: $\frac{KE}{KO} = \frac{1}{4} \quad \therefore OI = \frac{OK}{4}$

Then, if we assume that the volume of the pyramid (O'A'B'C'D') in Fig. 2.2, is very small compared with (OABCD), it follows:

$$HI = \varepsilon_a = \frac{e_a}{4} = \frac{2}{4} = 0.5'$$

$$OI = \varepsilon_b = \frac{e_b}{4} = \frac{4}{4} = 1.0'$$

2.4 WEIGHTS OF STORED MATERIAL IN BIN AND HOPPER

The volume of the bin is:

$$V_1 = 60 \times 25 \times 15 = 22{,}500 \text{ ft}^3$$

The volume of the hopper is:

$$V_2 = \text{Volume}(O'ABCD) = \frac{1}{3} \times 12 \times 25 \times 15 = 1{,}500 \text{ ft}^3$$

The weight of the sand in the upper portion of the bin is:

$$22{,}500 \times 0.1 = 2{,}250 \text{ kips}$$

The weight of the sand in the hopper is:

$$1{,}500 \times (0.1) = 150 \text{ kips}$$

The total weight in the bin is:

$$2{,}250 + 150 = 2{,}400 \text{ kips}$$

2.5 SURFACE AREAS OF HOPPER WALLS

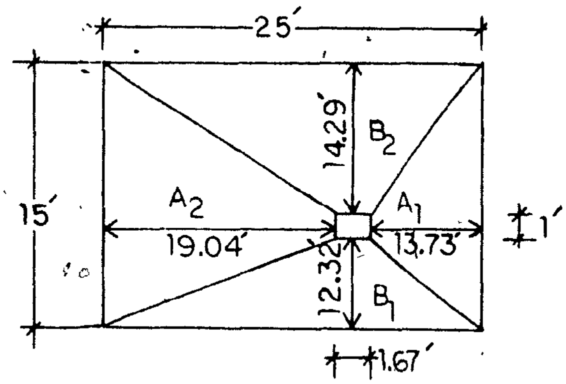

FIG. 2.4 Areas of inclined walls

From Fig. 2.4:

$$A_1 = \frac{(15+1) \times (11.2)}{2 \times \sin(54.69)} = 8(13.73) = 109.8 \text{ ft}^2$$

$$A_2 = \frac{8 \times (11.2)}{\sin(36.03)} = 8(19.40) = 152.3 \text{ ft}^2$$

$$B_1 = \frac{(25+1.67) \times (11.2)}{2 \times \sin(65.38)} = 13.34(12.32) = 164.3 \text{ ft}^2$$

$$B_2 = 13.34\frac{(11.2)}{\sin(51.63)} = 13.34(14.29) = 190.6 \text{ ft}^3$$

Total surface area = $A_1 + A_2 + A_3 + A_4 = 617 \text{ ft}^2$

CHAPTER III

INTERNAL FORCES ACTING ON BIN WALLS

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INTERNAL FORCES ACTING ON BIN WALLS

3.1 STATIC PRESSURES

The pressure distributions due to stored material, shown in Figure 3.1 (a), are calculated from Janssen's formulae [3].

$$P_{ha,b} = \frac{\gamma R_{a,b}}{\mu}\left(1-e^{-\mu' K y / R_{a,b}}\right) \qquad (3.1)$$

$$P_{va,b} = P_{ha,b}/K \qquad (3.2)$$

$$q_{a,b} = (\gamma y - 0.8 P_{va,b})\frac{A_{a,b}}{h_{a,b}} \qquad (3.3)$$

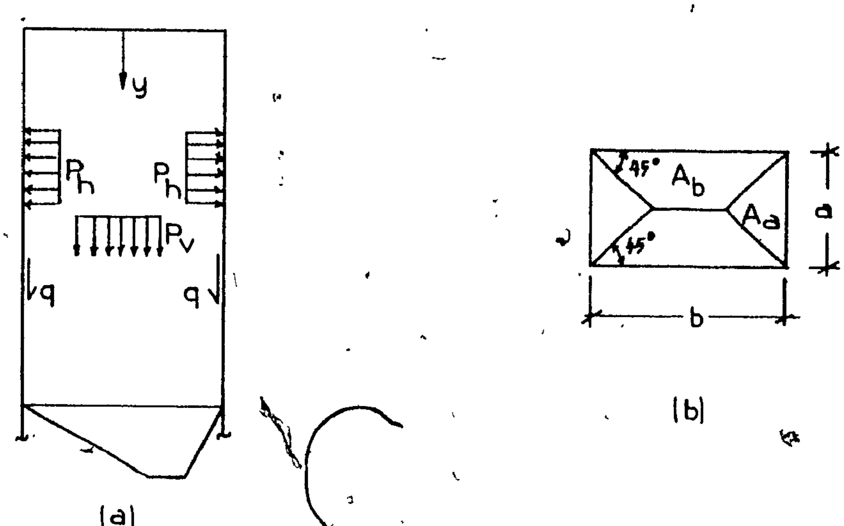

FIG. 3.1 a) Pressures acting on bin walls;
b) Effective areas for friction load

Where:

$P_{ha,b}$ = the horizontal pressure distribution (KSF)

$P_{va,b}$ = the vertical pressure distribution (KSF)

$q_{a,b}$ = the vertical friction load (KSF)

$R_{a,b}$ = the hydraulic radius (FT)

Subscripts a,b denote the corresponding side of the vertical wall.

$$K = \frac{1-\sin\phi}{1 \sin\phi} = \frac{1-\sin(35)}{1 \sin(35)} = 0.271$$

From Fig. 2.1 (b):

$$\frac{A_a}{h_a} = [(15 \times \frac{15}{2})\frac{1}{2}] \times \frac{1}{15} = 3.75'$$

$$\frac{A_b}{h_b} = [\frac{25 + (25-15)(0.5)}{2} (\frac{15}{2})] \times \frac{1}{25} = 4.5'$$

The pressures calculated from formulae (3.1,3.2 and 3.3), correspond to the bin walls with a symmetric outlet. In the present case, of eccentric outlet, there is a pressure increase which may be considered as acting on both walls, near and to the side opposite the eccentric outlet. [12].

This additional pressure is:

$$P_h' = P_{hi} - P_h \qquad (3.4)$$

where

P_{hi} is the horizontal pressure distribution considering the imaginary section developed in Fig. 3.2 [12].

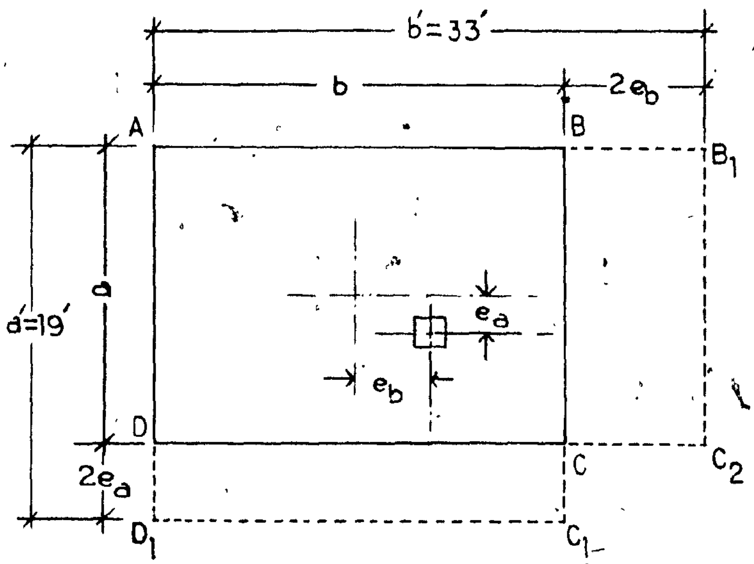

FIG. 3.2 Imaginary bin sections; AD_1C_1B for side b and ADC_2B_1 for a

3.2 DESIGN PRESSURES

The pressures computed by the method described so far correspond to static conditions. In the case, though, of filling or discharge, these pressures are amplified and depending on the depth, an appropriate value of the overpressure factor, C_d, is applied to formula (3.4).

The values of C_d, measuring height from the top are [3]:

$$0' - 20' : C_{d1} = 1.5$$

$$20' - 60' : C_{d2} = 1.65$$

$$60' - 71.2' : C_{d3} = 1.75$$

Equation (3.4) may be rewritten as:

$$(P_{h\ design}) = C_{dn}(P_h' + P_h) \text{ or } C_{dn}(P_{hi} - P_h + P_h) = C_{dn}P_{hi}$$

Then, according to formulae (3.1), (3.2) and (3.3):

$$(P_{ha,b})_{design} = C_{dn}\ [\gamma \frac{R'_{a,b}}{\mu'}\ (1-e^{-\mu' Ky/R'_{a,b}})] \quad (3.5)$$

$$(P_{va,b})_{design} = (P_{ha,b})_{design}/K \quad (3.6)$$

$$(q_{a,b})_{design} = C_d(q_{a,b}) \quad (3.7)$$

where, from Fig. 3.2:

$$R'_a = \frac{33 \times 15}{2(33+15)} = 5.16'$$

$$R'_b = \frac{19 \times 25}{2(19+25)} = 5.40'$$

Substituting the values of constants in Equations (3.5), (3.6) and (3.7):

$$(P_{ha})_d = C_{dn}\left[\frac{0.1(5.16)}{0.5} \times \left(1 - e^{-\frac{0.5(0.271)}{5.16}y}\right)\right]$$

Or: $$(P_{ha})_d = C_{dn}(1.032) \times (1 - e^{-0.0263y}) \quad (3.8a)$$

$$(P_{hb})_d = C_{dn}\left[\frac{0.1(5.4)}{0.5} \times \left(1 - e^{-\frac{0.5(0.271)}{5.4}y}\right)\right]$$

Or: $$(P_{hb})_d = C_{dn}(1.08) \times (1 - e^{-0.0251y})] \quad (3.8b)$$

$$(P_{va})_d = (P_{ha})_d/K = \frac{(P_{ha})_d}{0.271}$$

$$(P_{va})_d = C_{dn}(3.81) \times (1 - e^{-0.0263y}) \quad (3.9a)$$

$$(P_{vb})_d = C_{dn}(3.99) \times (1 - e^{-0.0251y}) \quad (3.9b)$$

$$(q_a)_d = C_{dn}[0.1y - 0.8(3.81(1 - e^{-0.0263y}))] \times (3.75)$$

Or: $$(q_a)_d = C_{dn}[0.375y + 11.43e^{-0.0263y} - 11.43] \quad (3.10a)$$

$$(q_b)_d = C_{dn}[0.1y - 0.8 \times (3.99(1 - e^{-0.025}))] \times (4.5)$$

Or: $$(q_b)_d \doteq C_{dn}[0.45y + 14.36e^{-0.0251y} - 14.36] \quad (3.10b)$$

Since the maximum difference in the horizontal pressure distribution for sides a and b is:

$$\frac{1.39 - 1.35}{1.35} \times 100 \doteq 3\%,$$

TABLE 3.1 Pressure distribution - Design values

Height	C_{dn}	(1) $(P_{ha})_d$	(2) $(P_{hb})_d$	(3) $(P_{va})_d$	(4) $(P_{vb})_d$	(5) $(q_a)_d$	(6) $(q_b)_d$
4.0	1.5	0.15	0.15	0.57	0.57	0.54	0.64
8.0	1.5	0.29	0.29	1.08	1.09	1.25	1.48
12.0	1.5	0.42	0.42	1.55	1.55	2.11	2.50
16.0	1.5	0.53	0.54	1.96	1.98	3.11	3.68
20.0	1.5	0.63	0.64	2.34	2.36	4.24	5.01
24.0	1.65	0.80	0.81	2.94	2.98	6.03	7.11
28.0	1.65	0.89	0.90	3.27	3.32	7.50	8.84
32.0	1.65	0.97	0.98	3.58	3.63	9.07	10.69
36.0	1.65	1.04	1.06	3.85	3.91	10.74	12.65
40.0	1.65	1.11	1.13	4.09	4.17	12.48	14.70
44.0	1.65	1.17	1.19	4.31	4.40	14.30	16.84
48.0	1.65	1.22	1.25	4.51	4.60	16.18	19.06
52.0	1.65	1.27	1.30	4.68	4.79	18.13	21.36
56.0	1.65	1.31	1.35	4.84	4.96	20.12	23.71
60.0	1.65	1.35	1.39	4.99	5.12	22.17	26.13
64.0	1.75	1.47	1.51	5.43	5.58	-	-
68.0	1.75	1.50	1.55	5.55	5.71	-	-
71.2	1.75	1.53	1.57	5.64	5.81	-	-

only, the values of $(P_{hb})_d$ may be used for the design of the bin vertical walls, i.e., column (2), Table 3.1.

3.3 FORCES ON THE HOPPER INCLINED WALLS

3.3.1 Normal Design Pressures

The normal design pressure for any inclined wall is the composition of the horizontal and vertical pressure, according to the angle of inclination, such as [3,22–25].

$$P_{nai} = (P_{ha})_d \sin^2\alpha_i + (P_{va})_d \cos^2\alpha_i; \text{ for short sides}$$

$$P_{nbi} = (P_{hb})_d \sin^2\beta_i + (P_{vb})_d \cos^2\beta_i; \text{ for long sides}$$

Thus, considering average values of pressure distribution in hopper from Table 3.1 and Fig. 2.1, it follows that:

(i) For wall a_1: angle $\alpha_1 = 54.69^\circ$ and

$$P_{nai} = \left(\frac{1.47+1.53}{2}\right) \sin^2 54.69 + \frac{5.43+5.64}{2} \cos^2(54.69) =$$

$$= (1.5).(0.67) + (5.54).(0.33) = 1.01 + 1.84$$

or $$P_{nai} = \underline{\underline{2.85}} \text{ KSF}$$

(ii) For wall a_2: angle $\alpha_2 = 36.06^{\circ}$ and

$$P_{na2} = (1.5)\sin^2(36.06) + (5.54)\cos^2(36.06) =$$

$$= (1.5)(0.35) + (5.54)(0.65) = 0.53 + 3.60$$

or $$P_{na2} = \underline{\underline{4.13}} \text{ KSF}$$

(iii) For wall b_1: angle $\beta_1 = 65.38^{\circ}$ and

$$P_{nb1} = \left(\frac{1.51+1.57}{2}\right)\sin^2 65.38 + \left(\frac{5.58+5.81}{2}\right)\cos^2 65.38 =$$

$$= (1.54)(0.83) + (5.7)(0.17) = 1.28 + 0.97$$

or $$P_{nb2} = \underline{\underline{2.25}} \text{ KSF}$$

(iv) For wall b_2: angle $\beta_2 = 51.63^{\circ}$ and

$$P_{nb2} = (1.54)\sin^2(51.63) + (5.7)\cos^2(51.63) =$$

$$= (1.54)(0.61) + (5.7)(0.39) = 0.94 + 2.22$$

or $$P_{nb2} = \underline{\underline{3.16}} \text{ KSF}$$

3.3.2 Meridional Forces in Hopper Walls

The hopper is supporting the full weight of the stored material as well as its own weight. The meridional forces developed are the in-plane forces in the wall resulting from

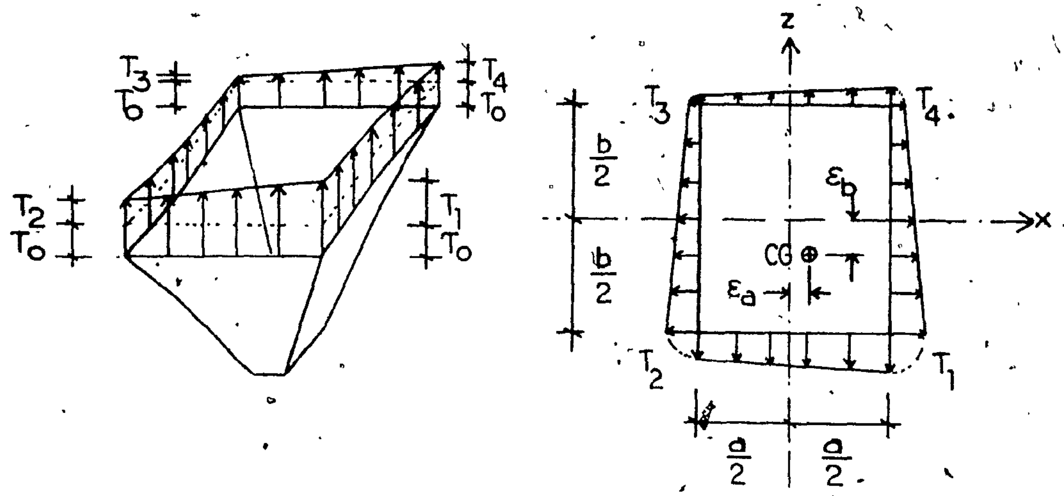

FIG. 3.3 Vertical load distribution in hopper

the ones shown in Fig. 3.3.

These vertical forces are composed of the distribution of the force reactions to the weight of the material above the hopper, designated as T_o; and the distribution of the hopper weight and contained material force reaction, indicated by T_i.

The meridional tensile forces T_i may be calculated by the following equations [11]:

$$T_1 = \frac{(W_h+G)}{2(a+b)} tx_3tz_1 \quad ; \quad T_2 = \frac{W_h+G}{2(a+b)} tx_2tz_2 \qquad (3.11a,b)$$

$$T_3 = \frac{(W_h+G)}{2(a+b)} tx_3tz_3 \quad ; \quad T_4 = \frac{W_h+G}{2(a+b)} tx_4tz_4 \qquad (3.11c,d)$$

Where:

tx_i and tz_i are the distribution coefficients computed according to Table 3.2.

W_h is the weight of the stored material in hopper alone, which from previous calculations is 150 kips, (Section 2.4).

G is the hopper own weight.

Assume a 3/4" plate for the walls of the hopper; then, for a total surface of the pyramid formed by the walls of the hopper equal to 617 + (25×15) = 992 ft^2 and having the same center of gravity with the stored material volume, it follows:

$$G = A_h \times t_h \times \gamma_{steel} = 992 \times \frac{0.75}{12} \times (0.49) \doteq 30.5 \text{ kips}$$

Of course, the weight of the plate added at the based of the pyramid is to be subtracted from the weight of the stored material above the hopper, since it has the same center of gravity with it. Being equal to:

$$25 \times 15 \times (\frac{0.75}{12}) \times 0.49 = 11.5 \text{ kips}$$

The force T_o, as shown in Fig.3.3 results from the weight of stored sand less the contribution of 11.5 kips. Plate at the base of the hopper:

$$T_o = \frac{2250 - 11.5}{2(15+25)} = 27.98 \text{ KLF}$$

TABLE 3.2 Coefficients tx_i, tz_i

Corner	tx_i	tz_i	A	B
1	$1 + A\varepsilon_b$	$1 + B\varepsilon_a$		
2	$1 + A\varepsilon_b$	$1 - B\varepsilon_a$	$\frac{6(a+b)}{b(b+3a)}$	$\frac{6(a+b)}{a(a+3b)}$
3	$1 - A\varepsilon_b$	$1 - B\varepsilon_a$		
4	$1 - A\varepsilon_b$	$1 + B\varepsilon_a$		

$$A = \frac{6(15+25)}{25(25+3\times15)} = 0.137 \; ; \quad B = \frac{6(15+25)}{15(15+3\times25)} = 0.178$$

$$tx_1 = 1 + (0.137)(1.0) = 1 + 0.137 = 1.137 = tx_2$$

$$tx_3 = 1 - 0.137 = 0.863$$

$$tx_4 = 1 - 0.137 = 0.863$$

$$tz_1 = 1 + (0.178)(0.5) = 1.089$$

$$tz_4 = 1 + 0.089 = 1.089$$

$$tz_2 = 1 - 0.089 = 0.911 = tz_3$$

Substituting values in Equations (3.11(a),(b),(c) and (d))

$$T_1 = \left(\frac{150 + 30.5}{2(25+15)}\right)(1.137)(1.089) = (2.26)(1.238) = 2.80 \text{ KLF}$$

$$T_2 = 2.26(1.137)(0.911) = (2.26)(1.04) = 2.34 \text{ KLF}$$

$$T_3 = 2.26(0.863)(0.911) = (2.26)(0.786) = 1.78 \text{ KLF}$$

$$T_4 = 2.26(0.863)(1.137) = (2.26)(0.94) = 2.12 \text{ KLF}$$

The tensile meridional forces acting in the inclined hopper walls are expressed by the average values considering forces acting in the corners, as follows:

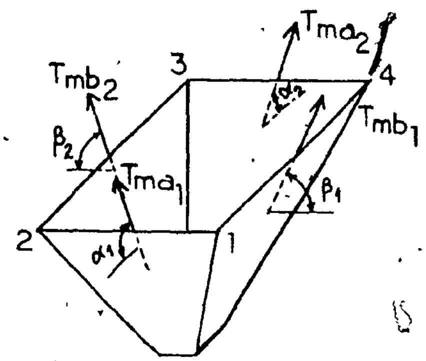

FIG. 3.4 Meridional forces in hopper walls

Wall 1-2: $T_{mai} = [T_o + (\frac{T_1+T_2}{2})] \times \frac{a}{\sin\alpha_1} =$

$$= [27.98 + \frac{2.8+2.34}{2}]\frac{15}{\sin 54.69} = 562 \text{ kips}$$

Wall 2-3: $T_{mbi} = [T_o + (\frac{T_2+T_3}{2})] \times \frac{b}{\sin\beta_2} =$

$$= [27.98 + [\frac{2.34 + 1.78}{2}]]\frac{25}{\sin 51.63} = 958 \text{ kips}$$

Wall 3-4: $T_{mai} = [T_o + (\frac{T_3+T_4}{2})]\frac{a}{\sin\alpha_2} =$

$$= [27.98 + (\frac{1.78+2.72}{2})] \frac{15}{\sin 36.06} = 763 \text{ kips}$$

Wall 4-1: $T_{mbi} = [T_o + (\frac{T_4+T_1}{2})]\frac{b}{\sin\beta_1} =$

$$= [27.98 + (\frac{2.12+2.8}{2})] \frac{25}{\sin 65.38} = 837 \text{ kips}$$

3.3.3 Horizontal Tensile Forces in Hopper Walls

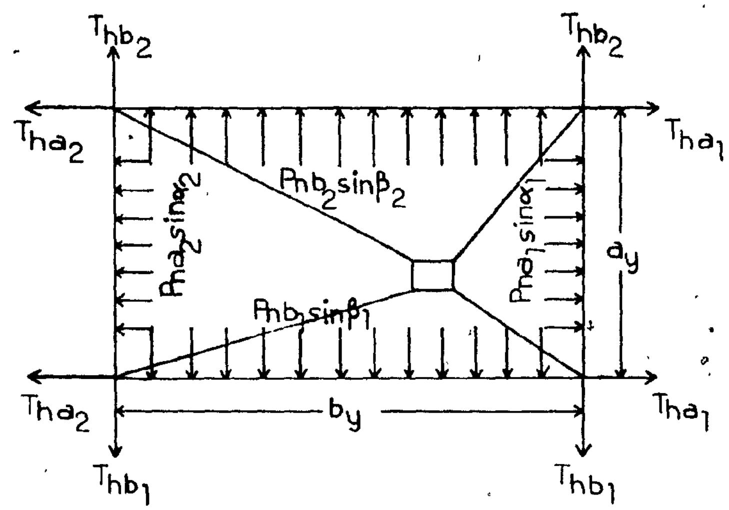

FIG. 3.5 Horizontal forces in hopper walls

The horizontal tensile forces in the inclined walls result from the horizontal component of the normal pressure acting on each wall.

Since the width of each wall varies with the depth of the hopper as:

$$a_y = 15 - y\,\frac{15 - 1}{11.2} = 15 - 1.25y$$

$$b_y = 25 - y\,\frac{25 - 1.67}{11.2} = 25 - 2.08y$$

The forces then as shown in Fig. 3.5 are:

$$T_{ha1} = \frac{1}{2}(P_{na2}\sin\alpha_2)ay = \frac{1}{2}(4.13 \sin 36.06)ay = 1.22ay \quad (3.12a)$$

$$T_{ha2} = \frac{1}{2}(P_{na1}\sin\alpha_1)ay = \frac{1}{2}(2.85 \sin 54.69)ay = 1.16ay \quad (3.12b)$$

$$T_{hb1} = \frac{1}{2}(P_{nb2}\sin\beta_2)by = \frac{1}{2}(3.16 \sin 51.63)by = 1.24by \quad (3.12c)$$

$$T_{hb2} = \frac{1}{2}(P_{nb1}\sin\beta_1)by = \frac{1}{2}(2.25 \sin 65.38)by = 1.02by \quad (3.12d)$$

Thus the final form of these forces in K/(FT of height) are:

$$T_{ha1} = 1.22(15-1.25y) = 18.3 - 1.53y$$

$$T_{ha2} = 1.16(15-1.25y) = 17.4 - 1.45y$$

$$T_{hb1} = 1.25(25-2.08y) = 31.25 - 2.6y$$

$$T_{hb2} = 1.02(25-2.08y) = 25.5 - 2.12y$$

For the above formulae, y is zero at the top of the hopper.

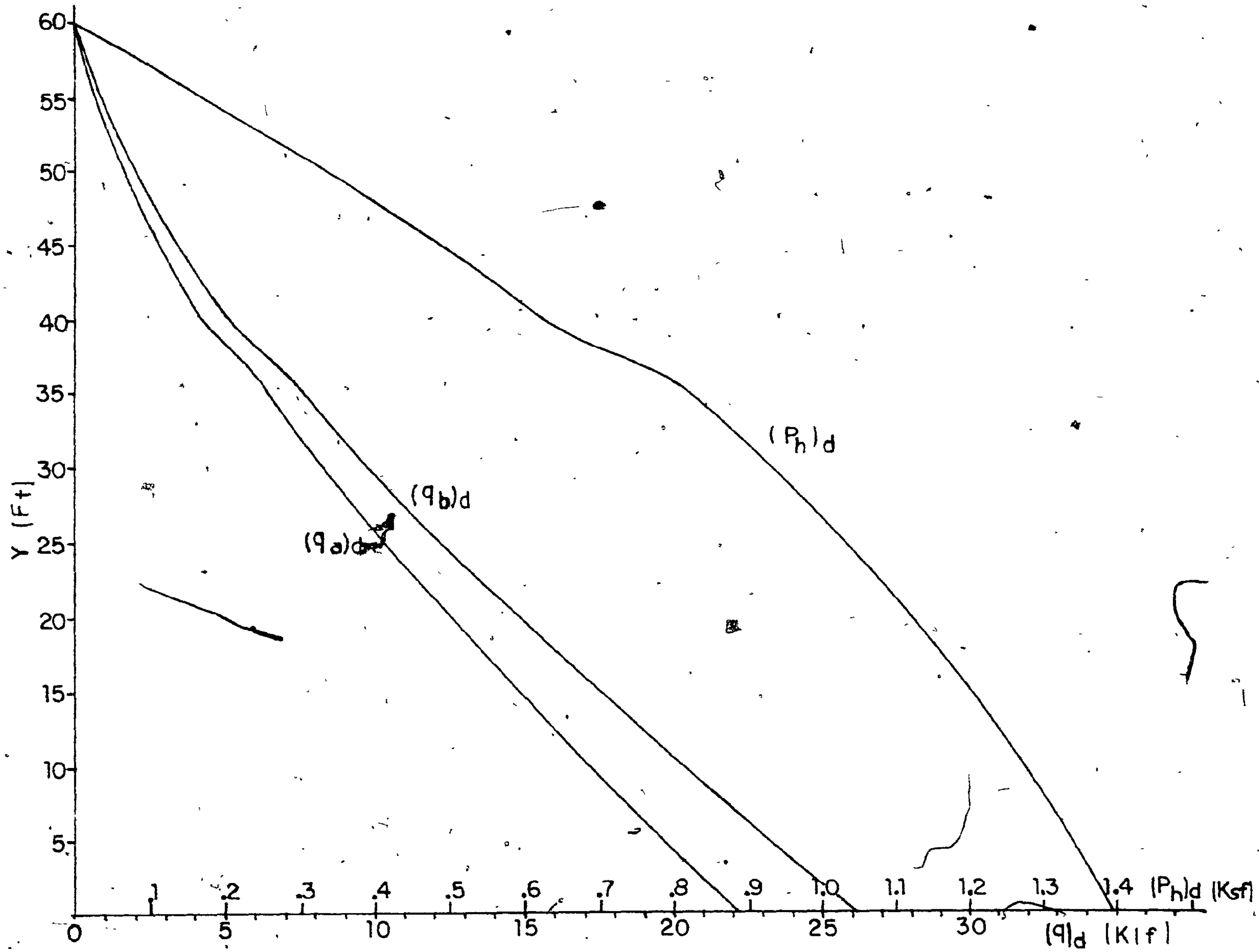

FIG. 3.6 Design curves

CHAPTER IV

DESIGN OF THE BIN WITH CONVENTIONAL STIFFEN-ING SYSTEM

CHAPTER IV

DESIGN OF THE BIN WITH CONVENTIONAL STIFFENING SYSTEM

In general, the plating is designed as a continuous plate in two directions, supported by horizontal and vertical stiffeners designed as acting separately to resist the lateral loads transmitted by the plating. The plating and the vertical stiffeners are to be checked also, for the vertical friction load which has to be transferred to the supporting columns. The general arrangement of such a stiffening system is shown in Figure 4.1.

4.1 DESIGN OF THE VERTICAL WALL PLATINGS

The type of steel used throughout the design is ASTM-A36, structural steel and, considering elastic design, its yield point is: Fy = 36 KSI [5]. The modulus of elasticity is assumed to be: $E = 29.10^3$ KSI.

The plating is designed for the action of the horizontal pressure and the vertical friction load, as shown in Fig.4.2. Furthermore, if the vertical stiffeners are spaced at 2'-6" along the perimeter of the wall and the horizontal stiffeners are spaced in such a way that the same thickness of plate may be used, then the following assumptions may be made for the analysis of the individual panels:

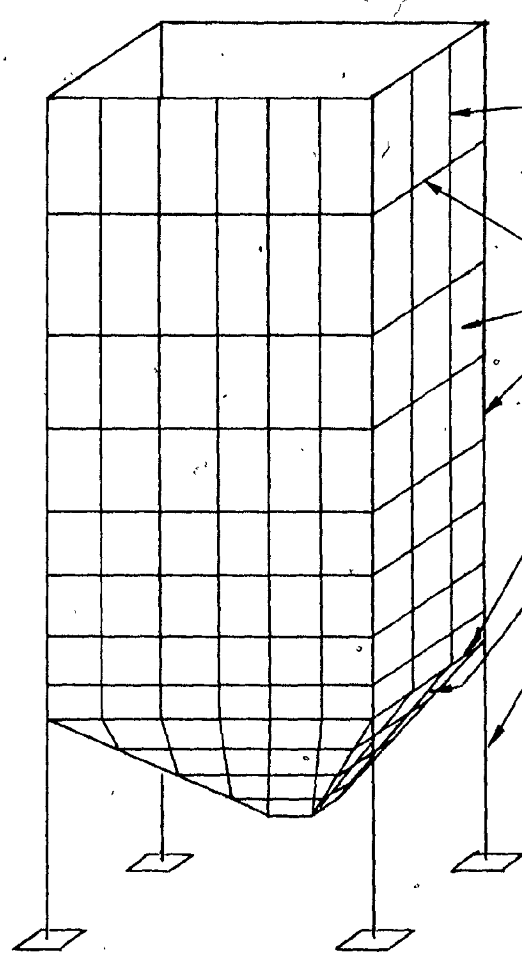

FIG. 4.1 Conventional stiffening system

(i) The uniformly distributed load is: $P_i = \frac{1}{2}(P_{hT} + P_{hB})$

(ii) The in-plane vertical load, resulting from friction load is: $q_o = \frac{1}{2}(q_T + q_B)$ at $Y = 0$.

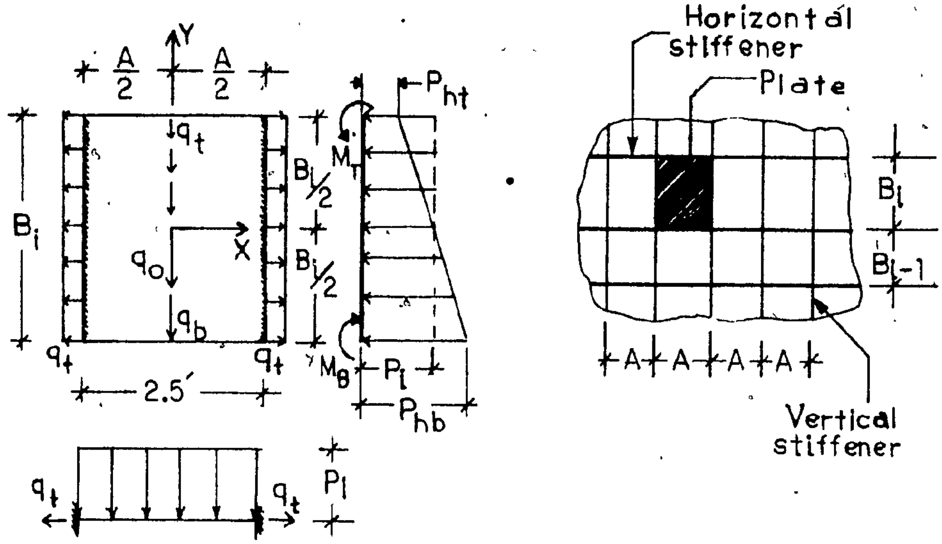

FIG.4.2 Loads and support conditions of plate wall panels

(iii) Since, for panels along the x-axis, the span and the thickness of the plate is the same, fixicity may be assumed at Y = 0 and X = ± A/2.

(iv) Following the continuity along the y-axis, and considering that $B_i \geq A$, the distributed moments M_T and M_B are equal to, or smaller than the ones produced at the clamped edges, in the horizontal direction. [11].

(v) The moments at the middle span are, in all cases, smaller by 50% to 30%, compared to the edge moments in the x-direction.

Therefore, the plate panels may be designed as rectangular plates fixed on four sides, and Table 4.2 may be used for their design.

The plate thickness is chosen on the basis of the allowable flexural stress being: $F_b = 0.6F_y = 0.6(36) = 21.6$ KSI.

The presence of the additional in-plane tensile stresses, due to the horizontal pressure on opposite walls, has to be accounted for, as well. Thus, the following stress-checking schedule may be assumed:

a) Checking the allowable tension at $x = \pm A/2$ and $Y = 0$ with the value of the tensile distributed forces are: $(q_t)_a = \frac{1}{2}(P_{ib})$ for sides a, and $(q_t)_b = \frac{1}{2}(P_{ia})$ for sides b. The values of $(q_t)_a$ will be used for this case, being larger than $(q_t)_b$, and

b) Checking the allowable compression, at $x = 0$ and $Y = 0$.

The choice of horizontal stiffener spacings may be based on the pressure the panels may carry, keeping stresses in the elastic range[6]. Thus, if a lower limit of allowable stress is assumed for bending alone, i.e., $f_b = 19$ KSI, for $f_b = \frac{6M}{t^2}$ and

$$M = (\text{coef}).P_i A^2 \quad \therefore \quad P_i = f_b t^2/6(\text{coef})A^2$$

where

t = the thickness of the plate;

or

$$P_i = 19t^2/6(\text{coef}).(2.5)^2 = \frac{0.507}{(\text{coef})} t^2 \qquad (4.1)$$

TABLE 4.1 Maximum bending moments and reactions in the panels with four edges clamped [11]

B_i/A	$(M_x)]X=\pm\frac{A}{2}$, Y=0 $M_X=-(COEF)\times P_i A^2$	$(M_y)]X=0, Y=0$ $M_Y=(COEF)\times P_i A^2$	REACTION $]X=0, Y=\pm\frac{B_i}{2}$ $R_X=(COEF)\times P_i B_i$
1.0	0.0513	0.0231	0.446
1.1	0.0581	0.0231	0.420
1.2	0.0639	0.0228	0.393
1.3	0.0687	0.0222	0.366
1.4	0.0726	0.0212	0.340
1.5	0.0757	0.0203	0.315
1.6	0.0780	0.0193	0.293
1.7	0.0799	0.0182	0.274
1.8	0.0812	0.0174	0.259
1.9	0.0822	0.0165	0.246
2.0	0.0829	0.0158	0.235
>2.0	0.0833	0.0125	0.235

Table 4.2 gives the values of average horizontal pressure for different ratios (B_i/A) based on the values of the maximum moment coefficients.

TABLE 4.2 Choice values of horizontal stiffener spacing

Spacing B_i (FT)	$\frac{B_i}{A}$	(Coef) (M_X)	t=3/8" P_i (KSF)	t=5/16" P_i (KSF)	t=1/4" P_i (KSF)
2.5'	1.0	0.0513	1.39	0.97	0.62
2.75'	1.1	0.0581	1.23	0.85	0.55
3.0'	1.2	0.0639	1.12	0.77	0.5
3.25'	1.3	0.0687	1.04	0.72	0.46
3.5'	1.4	0.0726	0.98	0.68	0.44
3.75'	1.5	0.0757	0.94	0.65	0.42
4.0'	1.6	0.0780	0.91	0.63	0.41
4.25'	1.7	0.0799	0.89	0.62	0.40
4.5'	1.8	0.0812	0.88	0.61	0.31
4.75'	1.9	0.0822	0.87	0.60	0.39
5.0'	2.0	0.829	0.86	0.60	0.38

The spacing of the horizontal stiffeners is done by comparing the values in Table 4.2, and the corresponding horizontal pressure values taken from Fig. 3.6; the spacing increases with the height from the hopper junction.

Since a corrosion allowance of 1 mm is provided for the skin plating material, a nominal increase of 1/16" is assumed to be the required thicknesses.

Therefore, for the lower courses of panels, the effective thickness is:

$$t_{ef} = 7/16 - \frac{19 \times 3.281}{1000} = 0.438 - 0.039 = 0.398"$$

and for the upper courses, the effective thickness is:

$$t_{ef} = 3/8 - 0.039 = 0.375 - 0.039 = 0.336"$$

The values of the additional compression stresses in the vertical direction are: $(\sigma_c)_b = \frac{(q_o)_b}{12 \times t_{ef}}$; for side "b" panels.

The values of the additional tensile stresses in the horizontal direction are: $(\sigma_t)_a = \frac{(q_t)_a}{12 \times t_{ef}} = \frac{1}{2}(P_i \times 25)\frac{4}{12 \times (t_{ef})} = P_i \times \frac{1.04}{t_{ef}}$; for side "a" panels.

The critical buckling stress for the typical panel shown in Fig. 4.2 is [9]:

$$\sigma_{cr} = \frac{k\,\pi^2\,E\,t_{ef}^2}{12A^2(1-\nu^2)} \quad (4.2)$$

where:

k is a coefficient depending upon the ratio $\frac{B_i}{A}$.

Substituting the values in Eq. (4.2):

$$\sigma_{cr} = k(3.14)^2(29.10^3)t_{ef}^2/12(2.5\times12)^2(0.91) =$$

$$= k(29.12)t_{ef}^2 \quad \text{(KSI)}$$

TABLE 4.3 Critical compression stress in plating (KSI)

B_i/A	1.0	1.1	1.2	1.3	1.4	1.5	2.0
K	4.0	4.04	4.13	4.28	4.47	4.30	4.0
σ_{cr}	18.45	18.63	19.05	19.74	20.62	19.83	13.15

4.1.1 Checking Stresses in Vertical Wall Platings

a) Compression stresses

The plates have to be checked for local buckling under the combined bending and axial stress. Using Table 4.1 positive moment coefficients: $f_{bc} = \frac{6M}{t_{ef}^2} = \frac{6(\text{coef})(P_i)_{max}}{t_{ef}^2}(2.5)^2$

Or:

$$f_{bc} = 37.5\ (\text{coef})\frac{(P_i)_{max}}{t_{ef}^2}$$

(i) For the 2.5' panels: $t_{ef} = 0.398"$; $\frac{B_i}{A} = 1$; $(P_i)_{max} = 1.38$ KSF

The flexural stress being: $f_{bc} = 37.5(0.0231)\frac{1.38}{(0.398)^2} =$

$= 7.55$ KSI

The axial stress from Table 4.3 is $(\sigma_c)_b = 5.34$ KSI and the maximum compression stress is:

$$(\sigma_c)_{max} = f_{bc} + (\sigma_c)_b = 7.55 + 5.34 = 12.89 \text{ KSI} < \sigma_{cr}$$

TABLE 4.4 Horizontal stiffener spacings and forces in steel plates

Y (FT)	B_i (FT)	t_{ef}	P_i (KSF)	$(q_o)_b$ (KLF)	$(q_o)_a$ (KLF)	$(\sigma_c)_b$ (KSI)	$(\sigma_t)_a$ (KLF)	$(\sigma_t)_b$ (KLF)	$(\sigma_t)_a$ (KSI)
	7.0		0.13	0.70	0.50	0.17	1.63	0.98	0.40
7.0									
	6.0		0.36	2.10	1.60	0.52	4.50	2.70	1.12
13.0		0.336"							
	5.0		0.53	3.70	3.00	0.92	6.63	3.98	1.64
18.0									
	4.0		0.64	5.00	4.25	1.05	8.00	4.80	1.68
22.0									
	4.0		0.81	7.20	6.00	1.51	10.13	6.08	2.12
26.0									
	4.0		0.90	9.00	7.50	1.88	11.25	6.75	2.36
30.0									
	3.5		0.97	10.50	9.00	2.20	12.13	7.28	2.54
33.5									
	3.0		1.04	12.25	10.20	2.56	13.00	7.80	2.72
36.5									
	2.75		1.09	13.70	11.50	2.87	13.63	8.18	2.85
39.25		0.398"							
	2.75		1.13	15.00	12.75	3.14	14.13	8.48	2.96
42.0									
	2.75		1.18	16.50	14.00	3.45	14.75	8.85	3.09
44.75									
	2.75		1.22	18.0	15.30	3.77	15.25	9.15	3.19
47.5									
	2.5		1.26	19.50	16.50	4.08	15.75	9.45	3.30
50.0									
	2.5		1.29	21.00	17.80	4.40	16.13	9.68	3.38
52.5									
	2.5		1.32	22.30	19.00	4.67	16.50	9.90	3.45
55.0									
	2.5		1.35	24.0	20.30	5.03	16.88	10.13	3.53
57.5									
	2.5		1.38	25.50	21.50	5.34	17.25	10.35	3.61
60.0									

(ii) For the 2.75' panels: $t_{ef} = 0.398"$; $\frac{B_i}{A} = 1.1$;

$$(P_i)_{max} = 1.22 \text{ KSF}$$

$$\therefore \quad f_{bc} = 37.5(0.0231)\frac{(1.22)}{(0.398)^2} = 6.67 \text{ KSI}$$

and $(\sigma_c)_{max} = 6.67 + 3.77 = 10.44 \text{ KSI} < \sigma_{cr}$

(iii) For the 3.0' panels: $t_{ef} = 0.398"$; $\frac{B_i}{A} = 1.2$;

$$(P_i)_{max} = 1.04 \text{ KSF}$$

$$\therefore \quad f_{bc} = 37.5(0.0228)\frac{1.04}{(0.398)^2} = 5.61 \text{ KSI}$$

and $(\sigma_c)_{max} = 5.61 + 2.56 = 8.17 \text{ KSI} < \sigma_{cr}$

Since the positive moment coefficients, the pressures and the axial stresses decrease as the height of the panel increases from the bottom of the vertical walls, it follows that the combined stresses will be significantly smaller than the already checked ones, for the lower courses.

b) <u>Tensile stresses</u>

The values of the tension flexural stresses may be obtained from Table 4.1, using the negative moment coefficients as follows:

$$f_{bt} = \frac{6M}{t_{ef}^2} = 37.5 \frac{(P_i)_{max}}{t_{ef}^2} \text{ (coef)}$$

(i) For the 2.5' panels:

$$f_{bt} = 37.5 \frac{(1.38)(0.0513)}{(0.398)^2} = 16.76 \text{ KSI}$$

Then, the combination of the flexural and axial stresses from Table 4.4 results in: $(\sigma_t)_{max} = f_{bt} + (\sigma_t)_a =$

$$= 16.76 + 3.61$$

Or: $(\sigma_t)_{max} = 20.37 \text{ KSI} < F_t = F_b = 21.6 \text{ KSI}$

(ii) For the 2.75' panels:

$$f_{bt} = 37.5 \frac{(1.22)(0.0581)}{(0.398)^2} = 16.78 \text{ KSI}$$

$$\therefore \quad (\sigma_t)_{max} = 16.78 + 3.19 = 19.97 \text{ KSI} < F_t$$

(iii) For the 3.0' panel:

$$f_{bt} = \frac{37.5(1.04)(0.0639)}{(0.398)^2} = 15.73 \text{ KSI}$$

$$\therefore \quad (\sigma_t)_{max} = 15.73 + 2.72 = 18.45 \text{ KSI} < F_t$$

(iv) For the 3.5' panel:

$$f_{bt} = \frac{37.5(0.97)(0.0726)}{(0.398)^2} = 16.67 \text{ KSI}$$

$$\therefore \quad (\sigma_t)_{max} = 16.67 + 2.54 = 19.21 \text{ KSI} < F_t$$

(v) For the 4.0' panel:

$$f_{bt} = \frac{37.5(0.90)(0.078)}{(0.398)^2} = 16.62 \text{ KSI}$$

$$\therefore \quad (\sigma_t)_{max} = 16.62 + 2.36 = 18.98 \text{ KSI} < F_t$$

(vi) For the 5.0' panel:

$$f_{bt} = \frac{37.5(0.53)(0.0829)}{(0.336)^2} = 14.59 \text{ KSI}$$

$$\therefore \quad (\sigma_t)_{max} = 14.59 + 1.64 = 16.23 \text{ KSI} < F_t$$

c) Shear stresses

The shear stresses may be computed from Table 4.1 values for reactions. The maximum of such a force occurs in the first lower course of panels $R_x = 0.448(P_i)(B_i)$.

Or:

$$R_x = 0.448(1.38)(2.5) = 1.55 \text{ Kips/ft}$$

and:

$$\tau_x = \frac{R_x}{t_{ef}} = \frac{1.55}{12 \times 0.398} = 0.32 \text{ KSI}$$

The allowable shear stress is: $f_v = 0.4F_y = 0.4 \times 36 = 14.4$ KSI; and $f_v > \tau_x$.

The plating, then, for the vertical walls of the bin, is chosen as:

(i) For the first 18'-0" from the top of the bin, 3/8" plate, and

(ii) For the remaining lower portion, 7/16" plate, including a 1 mm corrosion allowance in both cases.

4.2 DESIGN OF THE HORIZONTAL STIFFENERS

For these stiffeners, frame action is assumed, for which the members are subjected tobending due to the horizontal pressure transmitted through the plating, and to axial tension, due to the reaction of the opposite walls of the bin to the internal pressure from the stored sand.

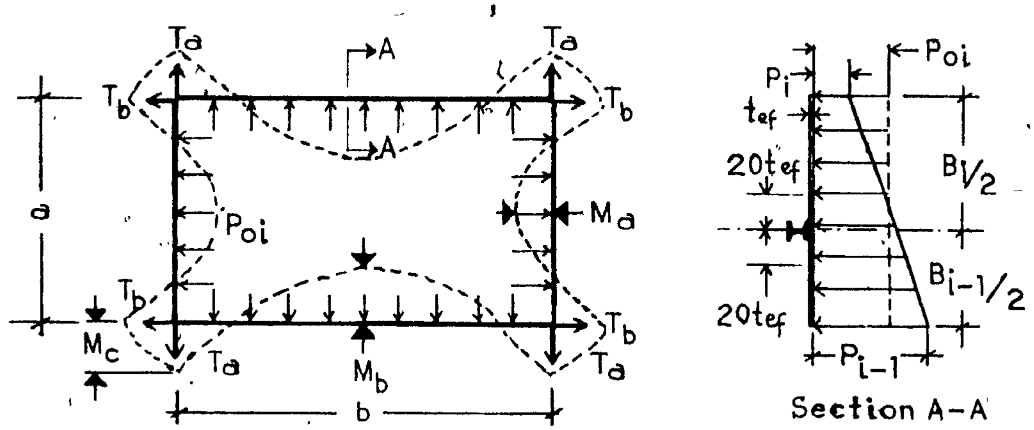

FIG. 4.3 Plan and section of horizontal stiffening frame

4.2.1 Load Computation

Considering Fig. 4.3, cross-section A-A, the distributed load acting on the sides of the frame is:

$$P_{oi} = \frac{1}{2}(P_i+P_{i-1}) .$$

The values of P_i may be obtained from Table 4.4.

The values of the axial forces in the members of the frame are:

$$T_{ai} = \frac{(P_i+P_{i-1})}{2} \frac{(B_i+B_{i-1})}{2} \frac{b}{2} \text{ or } T_{ai} = W_{hi} \frac{b}{2} \qquad (4.3)$$

and

$$T_{bi} = W_{hi} \frac{a}{2} \qquad (4.4)$$

where:

$$W_{hi} = P_{oi}\left(\frac{B_i+b_{i-1}}{2}\right) \text{ or } W_{hi} = \frac{(P_i+P_{i-1}) \times (B_i+B_{i-1})}{4}$$

The values of the bending moments in the frame members of the same cross-section are as follows: [12,24]

$$M_{ci} = -W_{hi}\left(\frac{a^3 + b^3}{12(a+b)}\right)$$

or

$$M_{ci} = -W_{hi}\frac{(a^2-ab+b^2)}{12} \qquad (4.5)$$

$$M_{aj} = W_{hi} \frac{a^2}{8} + M_c \qquad (4.6)$$

and

$$M_{bi} = W_{hi} \frac{b^2}{8} + M_c \qquad (4.7)$$

Substituting values of a and b in Equations (4.5) and (4.4):

$$T_{ai} = W_{hi}(12.5) \quad \text{(kips)} \tag{4.8}$$

$$T_{bi} = W_{hi}(7.5) \quad \text{(kips)} \tag{4.9}$$

$$-M_{ci} = W_{hi}(39.58) \text{ (Ft-K)} \tag{4.10}$$

$$M_{ai} = W_{hi}(28.13-39.58) = -W_{hi}(11.46) \text{ (Ft-K)} \tag{4.11}$$

$$M_{bi} = W_{hi}(78.13-39.58) = W_{hi}(38.55) \text{ (Ft-K)} \tag{4.12}$$

A part of the plating is assumed to participate with the provided frame members in resisting the above forces and moments and it is $L_{ef} = 40\ t_{ef}$. [1].

Table 4.5 values are computed with Equations (4.11), (4.12) and Table 4.4, value of P_i.

The stiffening frame members are subjected to loads having their intensity varying within 14–25% of the maximum values corresponding to the frame at height 30 Ft from the top of the bin; these values may be used for the design of all stiffening frames.

The design loads, then, for the frame members are:

(i) Negative moment: $M_c = -139.7$ Ft–K
along with axial tension: $T_a = 44.1$ K.

TABLE 4.5 Horizontal stiffening frames: Member Loads

Y_i	$\frac{P_i+P_{i-1}}{2}$	$\frac{B_i+B_{i-1}}{2}$	W_{hi} (KLF)	Side a		Side b		M_{ci}	L_{ef} (IN)
				T_{ai}	M_{ai}	T_{bi}	M_{bi}		
7.0	0.26	6.5	1.69	21.1	-19.4	12.7	65.1	-66.9	13.44
13.0	0.45	5.5	2.48	31.0	-28.4	18.6	95.6	-98.2	13.44
18.0	0.59	4.5	2.66	33.3	-30.5	20.0	102.5	-105.3	13.44
22.0	0.73	4.0	2.92	36.5	-33.5	21.9	112.6	-115.6	15.92
26.0	0.86	4.0	3.44	43.0	-39.4	25.8	132.6	-136.2	15.92
30.0	0.94	3.75	3.53	44.1	-40.5	26.5	136.1	-139.7	15.92
33.5	1.01	3.25	3.28	41.0	-37.6	24.6	126.4	-129.8	15.92
36.5	1.07	2.88	3.08	38.5	-35.3	23.1	118.7	-121.9	15.92
39.25	1.11	2.75	3.05	38.1	-35.0	22.9	117.6	-120.7	15.92
42.0	1.16	2.75	3.19	39.9	-36.6	23.9	123.0	-126.3	15.92
44.75	1.20	2.75	3.30	41.3	-37.8	24.8	127.2	-130.6	15.92
47.5	1.24	2.63	3.26	40.8	-37.4	24.5	125.7	-129.0	15.92
50.0	1.28	2.5	3.20	40.0	-36.7	24.0	123.4	-126.7	15.92
52.5	1.30	2.5	3.25	40.6	-37.2	24.4	125.3	-128.6	15.92
55.0	1.34	2.5	3.35	41.9	-38.4	25.1	129.1	-132.6	15.92
57.5	1.37	2.5	3.43	42.9	-39.3	25.7	132.2	-135.8	15.92

(ii) Positive moment: M_b = 136.1 Ft-K

along with axial tension: T_b = 26.5 K.

The effective width of the participating plating is L_{ef} = (40) × (0.398) = 15.92".

4.2.2 Trial and choice of sections

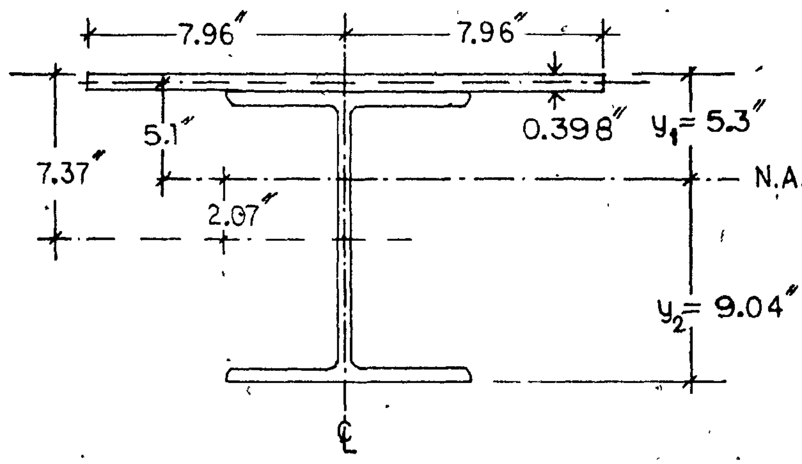

FIG. 4.4 Horizontal stiffener section geometry

The section properties of a W 14 × 53, trial section, are: A = 15.6 in^2, I = 542 in^4 and its geometry as in Fig. 4.4.

The neutral axis is located as:

$$y_1 = \frac{(0.398)(15.92)(0.2) + 15.6(7.37)}{15.6 + 0.398(15.92)} = \frac{(6.34)(0.2) + 15.6(7.37)}{21.94}$$

$$= 5.3"$$

and

$y_2 = (13.94+0.398) - 5.3 = 9.04"$, with $A_T = 21.94$ in^2.

The total moment of inertia of the plate and W-section is:

$$I_T = 542 + 15.6(7.37-5.3)^2 + 6.34(5.1)^2 = 773.75 \text{ in}^4$$

The section moduli of the whole section are:

$$S_1 = \frac{773.75}{5.3} = 146 \text{ in}^3 \;;\quad S_2 = \frac{773.75}{9.04} = 85.59 \text{ in}^3$$

The stresses in the section may be computed as follows:

(i) In the negative moment region:

$$f_{t1} = \frac{T_a}{A_T} = \frac{44.1}{21.94} = 2.01 \text{ KSI};$$ axial tension stress

$$f_{bt1} = \frac{M_e}{S_1} = \frac{139.7 \times 12}{146} = \frac{1676.4}{146} = 11.48 \text{ KSI};$$ flexural tension stress, and

$$f_{bc1} = \frac{M_e}{S_2} = \frac{1676.4}{85.59} = 19.59 \text{ KSI};$$ flexural compression stress.

(ii) In the positive moment region:

$$f_{t2} = \frac{T_b}{A_T} = \frac{26.5}{21.94} = 1.21 \text{ KSI}$$

$$f_{bt2} = \frac{M_b}{S_2} = \frac{136.1 \times 12}{85.59} \quad \frac{1633.2}{85.59} = 19.08 \text{ KSI}$$

$$f_{bc2} = \frac{M_b}{S_1} = \frac{1633.2}{146} = 11.19 \text{ KSI}$$

b) According to [5], Art. 17.2, in the case of axial tension and bending, the following condition should be satisfied by the stresses:

$$\frac{f_t}{F_t} + \frac{f_{bt}}{F_{bt}} \leq 1 \text{ ; since: } F_t = F_{bt} = 0.6F_y = 21.6 \text{ KSI}$$

Thus, in the negative moment region:

$$\frac{2.01}{21.6} + \frac{11.48}{21.6} = 0.62 < 1.0;$$

and in the positive moment region:

$$\frac{1.21}{21.6} + \frac{19.08}{21.6} = 0.94 < 1$$

Regarding local stability of the frame, the free flange of the section, as shown in Fig. 4.4, is subjected to compression, thus [5], Art.16.2.4.1 b) is applicable.

The derivation of the allowable compressive stress is as follows [5] Art.16.2.4.1 b) :

Computation of: $F_1 = \sqrt{(F_2)^2 + (F_3)^2}$ with

$F_2 = \frac{12000}{Ld/A_f}$ and $F_3 = \frac{149,000}{(L/rt)^2}$, where:

A_f = area of compression flange

L = unsupported length of compression flange, being in this case, 15 × 12 = 180", For member a.

d = depth of the section

r_t = radius of gyration of a section comprising of 1/6 of the web and the compression flange areas, being [5],pp.5–104,2.17".

From [5],pp.6–36,d/A_f = 2.63 in^{-1}; thus,

$$F_2 = \frac{12,000}{180(2.63)} = 25.35 \text{ KSI}$$

and

$$F_3 = \frac{149,000}{(180/2.17)^2} = 21.66 \text{ KSI}$$

Then, $F_1 = \sqrt{25.35 + 21.66} = 33.34$ KSI and for 2/3 $F_{bt} = \frac{2}{3}(21.6)$ = 14.4 KSI < F_1, the allowable stress is:

$$F_{bc} = 1.15\ F_{bt}(1 - \frac{0.28\ F_{bt}}{F_1}) \qquad (4.13)$$

or

$$F_{bc} = 1.15(21.6)(1 - \frac{0.28(21.6)}{33.34})$$

$$= 20.33 \text{ KSI}$$

The maximum compression stress in member "a" is:

$$f_{bc} = 19.59 - 2.01 = 17.58 \text{ KSI} < F_{bc}$$

Regarding the shear strength of the section, the maximum shear force in the frame is:

$$V_{max} = T_a = 44.1 \text{ Kips}$$

The latter is resisted by web area of the section being, [5], pp.6-36, 5.16 in^2; therefore, the shear stress is:

$$f_v = \frac{44.1}{5.16} = 8.55 \text{ KSI}$$

From section properties h/w = 41.8 and for [5], Art. 16.2.3, $380/\sqrt{Fy} = 380/\sqrt{36} = 63.33$, $h/w < 63.33$.

The allowable shear stress, then, is:

$$F_v = 0.4F_y \quad \text{or} \quad F_v = 0.4(36) = 14.4 \text{ KSI} > f_v$$

The tried section, W14 × 53 meets all the CSA S16-1969 requirements and may be used for the members of the horizontal stiffening frames.

4.3 DESIGN OF THE VERTICAL STIFFENERS

The vertical stiffeners are designed to carry the lateral loads transmitted with the plating.

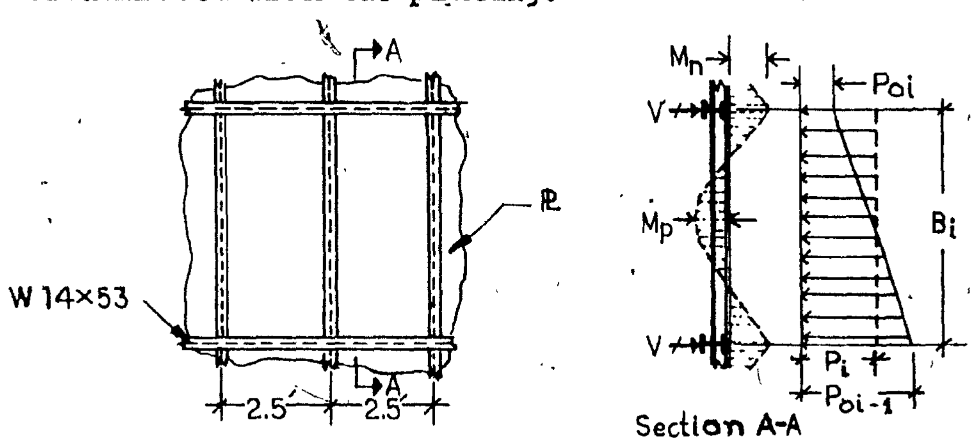

FIG. 4.5 Vertical stiffeners arrangement and loading

4.3.1 Load Computation

The average of the lateral pressure between horizontal stiffeners is considered, i.e., from Fig. 4.5:

$$P_i = \frac{1}{2}(P_{oi} + P_{oi-1})$$

Therefore, the distributed load acting on each vertical stiffener is:

$$w_{vi} = \frac{1}{2}(2.5)[P_{oi} + P_{oi-1}] = 1.25[P_{oi} + P_{oi-1}] \quad [\text{KLF}]$$

The stiffeners are considered to be continuously spanning between the horizontal frames and the bending moments may be assumed as:

$$M_{pi} = - M_{ni} = \frac{W_i \, B_i^2}{10} \quad [12]$$

Table 4.6 is prepared on the basis of Table 4.4, with reactions of $V_i = (W_{vi}\ B_i)/2$, and required minimum section modulus of

$$S_{i\ MIN} = M_p/f_{bc} = \frac{W_{vi}\ B_i^2}{10} \times 12\ \frac{1}{20.0} = 0.06\ W_{vi} B_i^2$$

considering participation of the plating.

The results from the load computations, as they appear in Table 4.6, indicate that there is not a large difference between the magnitude of the design loads. Therefore, the maximum of these values may be used for the design of all vertical stiffeners.

TABLE 4.6 Vertical stiffeners loading schedules

B_i (FT)	P_i (KSF)	W_{vi} (KLF)	V_i (K)	M_{pi} or M_{ni} (Ft-K)	S_i MIN (IN^3)	L_{ef} (IN)
7.0	0.13	0.33	1.14	1.59	0.96	13.44
6.0	0.36	0.90	2.70	3.24	1.94	13.44
5.0	0.53	1.33	3.31	3.31	1.99	13.44
4.0	0.64	1.60	3.20	2.56	1.54	15.92
4.0	0.81	2.03	4.05	3.24	1.94	15.92
4.0	0.90	2.25	<u>4.50</u>	<u>3.60</u>	<u>2.16</u>	<u>15.92</u>
3.5	0.97	2.43	4.24	2.97	1.78	15.92
3.0	1.04	2.60	3.90	2.34	1.40	15.92
2.75	1.09	2.73	3.75	2.06	1.24	15.92
2.75	1.13	2.83	3.88	2.14	1.28	15.92
2.75	1.18	2.95	4.06	2.23	1.34	15.92
2.75	1.22	3.05	4.19	2.31	1.38	15.92
2.5	1.26	3.15	3.94	1.97	1.18	15.92
2.5	1.29	3.23	4.03	2.02	1.21	15.92
2.5	1.32	3.30	4.13	2.06	1.24	15.92
2.5	1.35	3.38	4.22	2.11	1.27	15.92
2.5	1.38	3.45	4.31	2.16	1.29	15.92

The design loads are: M_p or M_n = 3.6 Ft-K

V = 4.5 kips

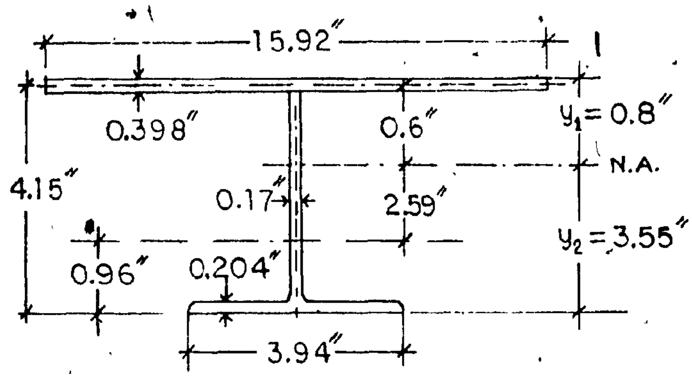

FIG. 4.6 Vertical stiffener section geometry

4.3.2 Trial and Choice of Section [5,6-65]

The section properties of a WT4 × 5 section are $A = 1.48$ in^2, $I = 2.15$ in^4, $t_{flange} = 0.204"$, $t_{web} = 0.17"$ and its geometry as in Fig. 4.6.

The neutral axis of the combined cross-section is located at:

$$Y_2 = \frac{1.48(0.96) + 6.34(4.15)}{1.48 + 6.34} = \frac{27.73}{7.82} = 3.55"$$

and

$$Y_1 = 4.35 - 3.55 = 0.8"$$

The moment of inertia of the whole section about its neutral axis is:

$$I_T = 2.15 + 1.48(2.59)^2 + 6.34(0.6)^2 = 14.36 \text{ in}^4$$

The section moduli of the above section are:

$$S_i = \frac{I_T}{y_1} = \frac{14.36}{0.8} = 17.95 \text{ in}^3; \quad S_2 = \frac{14.36}{3.55} = 4.05 \text{ in}^3$$

The maximum stresses in the section then may be computed as follows:

(i) Maximum tensile stress:

$$f_{bt} = \frac{M_p}{S_2} = \frac{3.16 \times 12}{4.05} = 10.67 \text{ KSI} < F_{bt}$$

(ii) Maximum compression stress:

$$f_{bc} = \frac{M_n}{S_2} = \frac{3.6 \quad 12}{4.05} = 10.67 \text{ KSI}$$

Following the same procedure as in previous section (4.2.2):

$$F_2 = \frac{12{,}000}{Ld/A_f} = \frac{12{,}000}{(48)(4.25)/(3.94)(0.204)} = 47.28 \text{ KSI}$$

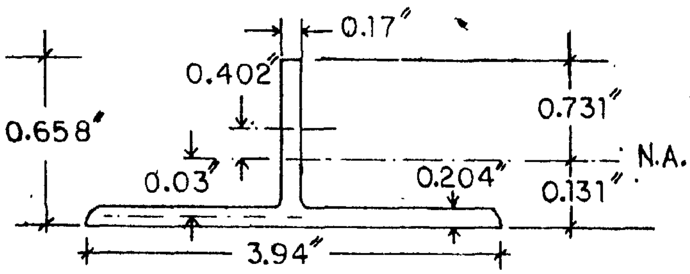

FIG. 4.7 Compression flange geometry of vertical stiffener sections

In order to compute r_t, the geometric properties of the compression flange section are required [5],pp.1-29,then from Fig. 4.7:

$$y_1' = \frac{[(0.204)(3.94)](0.102) + [(0.17)(1/6)(3.95)](0.431)}{(0.204)(3.94) + (0.17)(1/6)(3.95)}$$

or

$$y_1' = \frac{(0.806)(0.102) + (0.077)(0.431)}{0.883} = 0.131"$$

The moment of inertia of that section is:

$$I' = \frac{(0.204)^3(3.94)}{12} + (0.806)(0.03)^2 + \frac{(0.658)^3(0.17)}{12} +$$

$$+ 0.077(0.402)^2 = 0.002 \text{ in}^4$$

The radius of gyration about the neutral axis is, then:

$$r_t = \sqrt{I'/A'} = \sqrt{0.02/0.883} = 0.15"$$

As a result:

$$F_2 = \frac{149,000}{(L/r_t)^2} = \frac{149,000}{(48/0.15)^2} = 1.45 \text{ KSI}$$

and

$$F_1 = \sqrt{F_2^2 + F_3^2} = \sqrt{47.28^2 + 1.45^2} = 47.3 \text{ KSI}$$

$$(2/3)\ F_{bt} = 2/3(21.6) \quad \text{and} \quad F_1 > \frac{2}{3} F_{bt}$$

Therefore, using Equation (4.13), F_{bc} is computed as:

$$F_{bc} = (1.15)(21.6)(1 - \frac{0.28(21.6)}{47.3}) = 21.6 \text{ KSI}$$

Comparing stresses, then:

$$F_{bc} \not> f_{bc}$$

(iii) Checking shear stresses:

$$f_v = \frac{V}{A_{web}} = \frac{4.5}{(3.95)(0.17)} = 6.7 \text{ KSI}$$

According to [5] Art.16.2.3:

$$380/\sqrt{F_y} = 63.33 \quad \text{and} \quad \frac{h}{w} = \frac{3.95}{0.17} = 23.23$$

and $h/w < 380/\sqrt{F_y}$; therefore: $F_v = 14.4 > f_v$.

Section WT4 * 5 may be used for all vertical stiffeners of the vertical walls of the bin.

4.4 VERTICAL LOADS ANALYSIS

The vertical loads transmitted to the supporting columns, regarding the upper part of the bin, are the own weight of the wall plating and stiffeners, as well as the vertical friction load expressed in its total value per unit width of the bin wall, in Figure 3.6.

4.4.1 Vertical loads analysis

There are 32 vertical stiffeners at 5 lbs/ft, 17 horizontal stiffening frames at 53 lbs/ft; 18' of 3/8" plate and 42' of 7/16" plate. Then the total weight of the walls is:

$$W_w = \frac{32(60)5 + 17(80)(53)}{1000} + (0.49)\frac{(80)}{12}(18(\frac{3}{8}) + 42(\frac{7}{16})) =$$

$$= 164 \text{ kips}$$

4.4.2 Vertical friction load

From Fig. 3.6 at the bottom of the walls, the total friction load is:

(i) Wall a : 15 × 22.2 = 33 kips

(ii) Wall b : 25 × 26.25 = 656 kips

Then the total friction load is:

$$W_f = (333 + 656)(2) = 1979 \text{ kips}$$

Therefore, the total vertical load, in the vicinity of the hopper junction, to be transmitted through the walls to the supporting columns is:

$$W_T = 1979 + 164 + 17 = 2160 \text{ kips}$$

adding 17 kips for roof weight and attachments.

Considering Equation (4.2) for the lower course plating, the critical compression stress is

$$\sigma_{cr} = 4(29.12)(0.398)^2 = 18.45 \text{ ksi}$$

Consulting Table 4.4, the compression stress in the plating was found to be 7.55 ksi.

Then the vertical force that may be transmitted through the plating is:

$$W_p = (18.45-7.55)(0.398)(80) \times 12 = 4{,}165 \text{ kips}$$

Since $W_p > W_T$ and the combined axial stress in the plating does not change considerably along the height of the wall, the corner stiffeners do not carry axial forces.

Angles 4 × 4 × 7/16 (× 11.3 lbs/ft) are provided at the corners to facilitate construction and they have to be checked for the bending moments transmitted from the wall plating, on sides a and b.

The maximum bending moments that would occur on the sides of the angle have a magnitude of: $M = w\ell^2(0.0513)$ [11]

where:

w = the distributed load from Figure 3.6 (1.39 KSF)

and ℓ = the spacing of the vertical stiffeners (2.5 FT)

Therefore,

$$M = 1.39 = (2.5)^2(0.0513) = 0.446 \text{ FT-K/FT}$$

and the maximum bending stress in the plate of the angle is:

$$f_b = \frac{0.446 \times 6}{(7/16)^2} = 14 \text{ KSI} < F_b$$

CHAPTER V

APPLICATION OF THE ORTHOTROPIC PLATE CONCEPT TO THE DESIGN OF THE STIFFENED WALL PLATE

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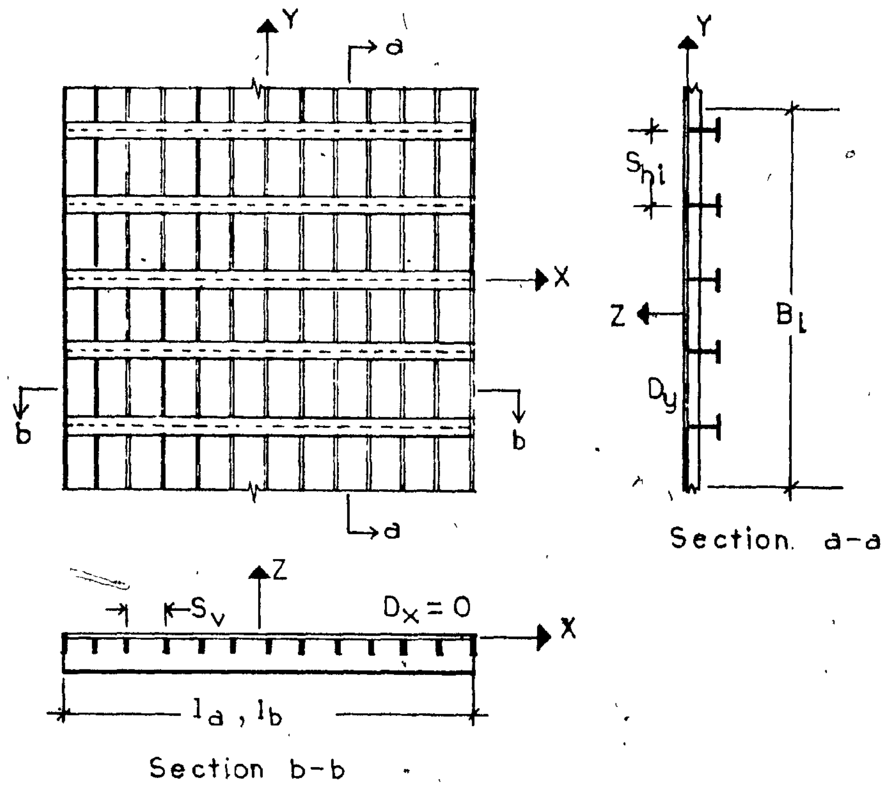

FIG. 5.1 The orthotropic plate stiffeners system

5.1 ANALYSIS APPROACH AND ASSUMPTIONS

For the design of the wall plating as an orthotropic plate, it is necessary to consider the manufacturing process details, i.e., continuous plate-to-stiffener welds, so that, both economy and safety may be achieved.

In order to arrive at a practical solution to the stiffeners arrangement problem, the triangular pressure distribution should be averaged for certain intervals, along the height of the bin.

Regarding Fig. 5.1 and latter approximation, then, if the horizontal stiffeners are equally spaced, within the interval B_i, in such a way that S_{hi}/S_v = 4.5 to 7.0, the analysis may be based on the one used for the orthotropic bridge decks [8].

The basic simplifications, adopting bridge deck analysis to be considered in the present case, are:

(a) there are no moving live loads;

(b) the pressure is constant within the 20'-0" intervals along theheight of the bin.

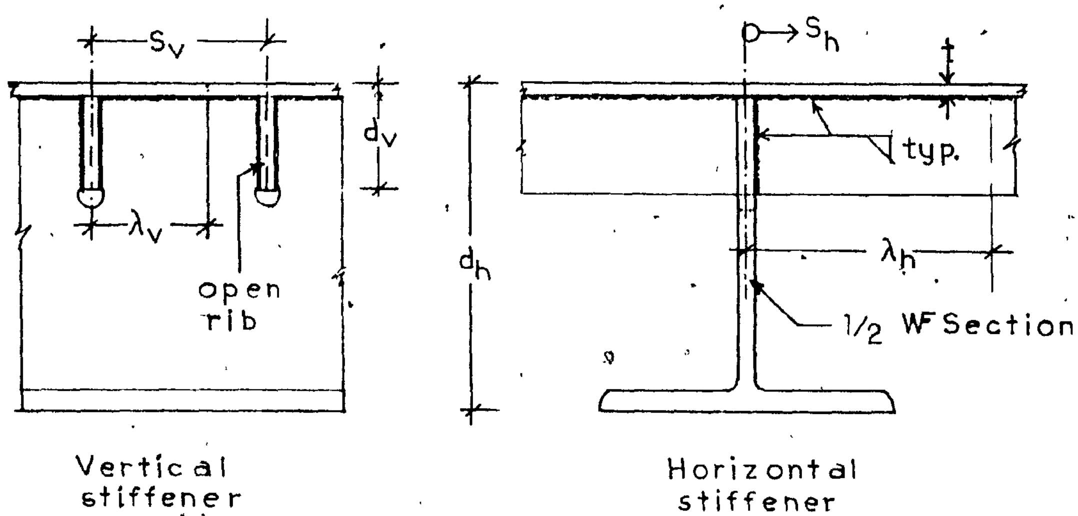

FIG. 5.2 Stiffeners and Plate cross-section details

Assuming a stiffening system as in Fig. 5.1, with attachment details as in Fig. 5.2, and considering uniform pressure within interval B_i, the following points are of valid consideration to the analysis and design of the stiffened plate:

a) The stiffened plate behaves in the same fashion but independently regarding x and y-axes, i.e., the interaction of plate eccentricities, e_x, e_y, as well as flexural rigidities D_x, D_y, are minimal, due to the criterion of stiffener spacing $S_h > 4.5\ S_{hi}$.

Due to its construction, the stiffened plate is considered to be spanning continuously over the horizontal stiffeners and is to be designed with $D_x = 0$, or the rigidity of the plate alone is to be neglected. [1]

Based on that assumption, the Huber's Equation [1]:

$D_x \dfrac{\partial^4 w}{\partial x^4} + 2H\dfrac{\partial^4 w}{\partial x^2 \partial y^2} + D_y\dfrac{\partial^4 w}{\partial y^4} = 0$; with $H = \sqrt{D_x D_y}$, representing the elastic curve relationship to the rigidities of the plate in both axes, may be reduced to the form, which may ensure behavior of the ribbed plate similar to that of the beam between the supports; of course, considering a **fully** elastic behavior and small deformations of the plate.

Thus, considering a uniform load intensity of P_i:

$$D_y \frac{\partial^4 w}{\partial y^4} = P_i$$

The vertical stiffeners are equally spaced in all panels resulting, of course, in different plate thickness choice.

Their design is basically governed by the two following conditions regarding horizontal stiffener behavior:

(i) Continuously spanning over rigid supports, and

(ii) Continuously spanning over flexible supports.

The stresses developed in the stiffener-plate cross-section, as derived from the above loading conditions, are to be superimposed.

The design of the plating is based on the allowable deflection formula: $t = S_v\ 0.0065 \sqrt[3]{P_{iv}}$, developed by Kloeppel [2],p.71, for which a cylindrical surface is assumed for the deflection curve of the plate between vertical stiffeners.

b) The horizontal stiffeners are designed as frame members, for which at all corners equal -ve moments exist. Their loading is constant within the panels but their deflections are to be considered varying linearly between the panel heights. This assumption is justified due to the actual, triangular, load application.

The vertical and horizontal stiffening elements are acting integrally with the plate, at least for a certain minimum width which is referred to as the effective width λ_v or λ_h [1], for each side of the stiffener.

In general, such a condition may be considered valid for any ribbed plate. In the present case, however, it makes the whole difference from the conventional design, when an effective width of 40 t_{ef} was employed.

Following Von Karman's theory and its rigorous applications to specific loading and span arrangements [1] it is certain that, a far more than the above-mentioned conventional effective width of plating may be employed for the design of the vertical ribs and the horizontal frame members. The latter are half W-sections, shown in Figure 5.2.

The type of construction of an orthotropic, open rib, plate favors the effective width theory application, because the continuous longitudinal welding of the stiffener webs to the plate ensures a rather continuous section and stress development, according to the theoretical analysis [2].

For the preliminary spacing, Beschkine's analysis [14] presented in Reference [1], is employed. For the actual design of the stiffeners, the assumed and the finally adopted values of the effective widths are derived from formulae strictly pertaining to the (a) type of stiffener, and (b) type of loading.

In the presence of transverse and in-plane forces, the orthotropic plate stiffening system adopted in the design of the vertical walls of a bin, has the advantage of providing extra stability to the plating alone. [9]

5.2 ANALYSIS AND DESIGN OF THE STIFFENED PLATE WALLS

5.2.1 Loads and stiffeners preliminary spacing

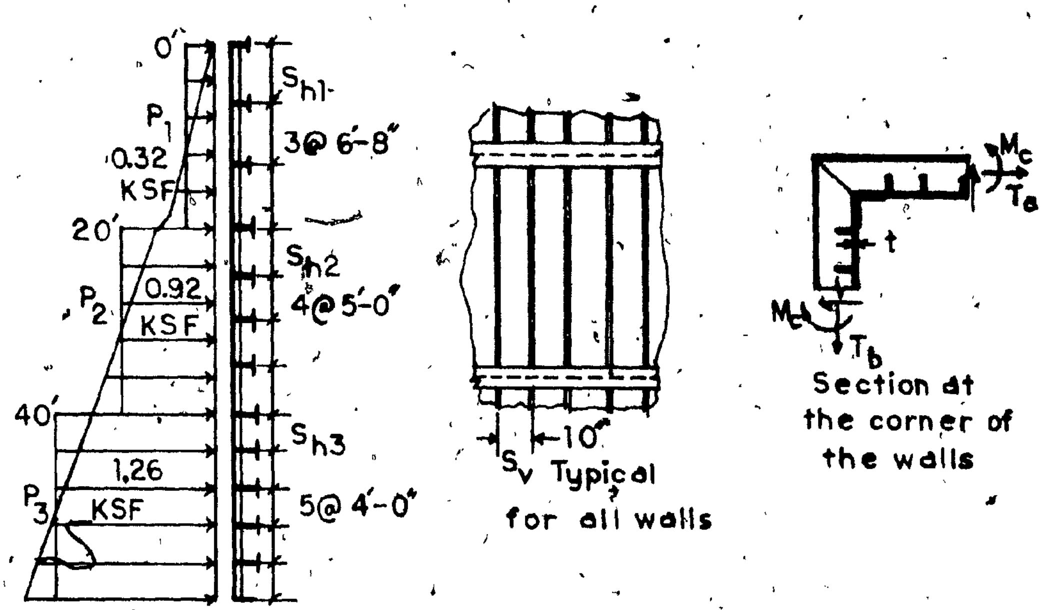

FIG. 5.3 Spacing and detailed arrangement of stiffeners

The walls are divided into three parts, as shown in Figure 5.3, each one carrying uniform pressure taken from Figure 3.6. Thus, p_1 = 0.32 ksf, p_2 = 0.92 ksf and p_3 = 1.26 ksf and in PSI:

$$p_1 = \frac{1000(0.32)}{144} = 6.94(0.32) = 2.22 \text{ psi}$$

$$p_2 = 6.94(0.92) = 6.39 \text{ psi, and}$$

$$p_3 = 8.75 \text{ psi}$$

The thickness of the plate may be computed from the formula : $t_i = s_v\ 0.0065 \sqrt[3]{p_i}$, with s_v = 10".

Then, for the first panel 0'-20':

(i) $$t_1 = 10(0.0065)\sqrt[3]{2.22} = \underline{0.085"}$$

Considering 1 mm corrosion allowance:

0.085 + 0.039 = 0.124 or 1/8" nominal

(ii) For the middle panel = 20'-40':

$$t_2 = 10(0.0065)\sqrt[3]{6.39} = 0.121"$$

or 0.121 + 0.039 = 0.16" or 3/16" nominal

$$t_2 = 3/16 - 0.039 = \underline{0.149"}$$

(iii) For the bottom panel: 40'-60':

$$t_3 = 10(0.0065)\sqrt[3]{8.75} = 0.134"$$

or 0.134 + 0.039 = 0.173 or 3/16" nominal

$$t_3 = \underline{0.149"}$$

The stiffener spacing is chosen from the effective width analysis results, by Beschkine's study, relating the ratio of the effective width to the span to the ratio of the spacing to the span of continuous stiffened plates [1].

Thus, for: $S_v/S_{hi} = 10/80 = 0.125$

and $S_v/S_h = 10/48 = 0.24$

λ_v/S_h is between 0.1 and 0.160, which is a reasonable estimate compared with other results [8].

Approximately, then, if the full effective width is 2λ, then it should be between 0.1(80).2 = 16" and 0.16(48) × 2 = 15.4", exceeding the spacing of 10".

For the horizontal stiffeners:

$$S_{h1}/b = 6.67/25 = 0.267 \text{ and } 2_h/b = 0.18$$

or $$2\lambda_h = (0.18)(25)(2) = 9' > 6.67'$$

and

$$S_{h3}/b = 4/25 = 0.16 \text{ and } \lambda_h/b = 0.132$$

or

$$2\lambda_h = (0.132)(25)2 = 6.6' > 4'$$

Therefore, in the calculation of the exact effective width, it should be anticipated that there is a value very close to the actual spacing of the stiffeners, thus substantial savings in material.

5.2.2 Analysis and design of the horizontal stiffeners

The horizontal stiffeners are designed for the moment loads computed considering frame action and for the horizontal tensile forces resulting from the action of the horizontal pressure on the vertical walls.

The design loads are:

(i) first panel: $0.32 \times 6.67 = w_1 = 2.13$ KLF

(ii) middle panel: $0.92(5) = w_2 = 4.6$ KLF; and

(iii) bottom panel: $1.26(4) \cong w_3 = 5.4$ KLF

From Figure 4.3:

$$T_{b1} = 2.13 \times 15/2 = 15.98^k$$

$$T_{a1} = 2.13 \times 25/2 = 26.63^k$$

$$T_{b2} = 4.6(15/2) = 34.5^k;\ T_{a2} = 4.6(25/2) = 57.5^k$$

$$T_{b3} = 5.04(15/2) = 37.8^k;\ T_{a3} = 5.04(25/2) = 63^k$$

The corner bending moments are computed from the formula:

$$M_c = -\frac{w(a^2k+b^2)}{12(1+k)} \qquad (5.1)$$

as shown in Reference [12], where:

$k = (I_b/I_a)a/b$; assuming $I_b/I_a = 1$, then $k = 0.6$.

Therefore:

$$M_{c1} = -2.13\frac{(225\ (0.6)+625)}{12(7.6)} = -2.73(39.58) = -84.31'^k$$

$$M_{a1} = \frac{2.13(15)^2}{8} - 84.31 = -24.40'^k$$

$$M_{b1} = \frac{2.13(625)}{8} - 84.31 = +82.10'^k$$

$$M_{c2} = -(4.6)(39.58) = -182.07'^k$$

$$M_{a2} = \frac{4.6(225)}{8} - 182.07 = -52.70'^k$$

$$M_{b2} = \frac{4.6(625)}{8} - 182.07 = 177.3'^{k}$$

$$M_{c3} = -5.04(39.58) = -199.48'^{k}$$

$$M_{a3} = \frac{5.04(225)}{8} - 199.48 = -57.73'^{k}$$

$$M_{b3} = \frac{5.04(625)}{8} - 199.48 = 194.27'^{k}$$

It is obvious then, that the middle span moment in all sides a is negative and the plating is in tension in all panels a in the horizontal direction.

Considering zero moment location in spans of side b the general equation of moment may be written as:

$$M_x = -M_c + \frac{w\ell^2}{2}\left(\frac{x}{\ell} - \left(\frac{x}{\ell}\right)^2\right) \qquad (5.2)$$

and for $M_x = 0$ for the first panel

$$-84.31 + \frac{2.13(625)}{2}\left(\frac{x}{25} - \frac{x^2}{625}\right) = 0$$

$$x^2 - 25x + 79.16 = 0$$

Therefore

$$x = 3.72'$$

or

$$x = \frac{3.72}{25} b = 0.149b$$

for all spans.

Thus, the effective span or the portion of the span for which the moment is positive is:

$$b_1 = b - 2(0.149)b \quad \text{or} \quad b_1 = 0.702b = 17.55'$$

Considering sides b, the moment diagram of the stiffeners resembles the one of a continuous beam member, for which the general equation for the effective width, from Fukuda's analysis [15] from Reference [1], is:

$$\frac{\lambda}{\ell} = \frac{4}{\frac{1}{6}[1-6\frac{x}{\ell}+6(\frac{x}{\ell})^2][1 + (3+\nu)4n\pi\gamma]\pi^2 \sum_n \frac{n^2}{\gamma\cos\frac{2n\pi x}{\ell}}} \tag{5.3}$$

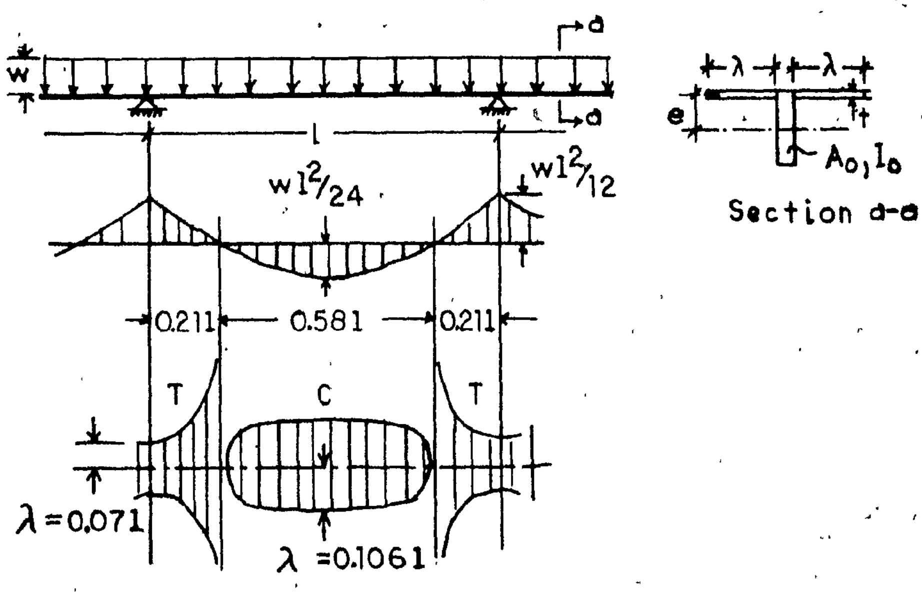

FIG. 5.4 Loading, moment and effective width distribution for a continuous multi-span ribbed plate

with: $$\gamma = \frac{A_o I_o}{8t\ell(I_o+e^2A_o)} .$$

Considering the case for which the plate thickness has little effect, i.e., $t \to 0$, then $\gamma \to \infty$. Then, the limit of Equation (5.3), for the effective width, becomes:

$$\frac{\lambda}{\ell} = \frac{6}{(3+\nu)\pi^3[1 - 6\frac{x}{\ell} + (\frac{x}{\ell})^2]} \sum_n \frac{1}{n^3} \cos \frac{2n\pi}{\ell} x \qquad (5.4)$$

Applying the values of: $\frac{x}{\ell} = 0$ and $n = 7$, the value of $\frac{\lambda}{\ell} = 0.07$ is obtained for the negative moment region effective width. Again, for $\frac{x}{\ell} = \frac{1}{2}$ and $n = 5$, $\frac{\lambda}{\ell} = 0.106$ is obtained for the positive moment region.

Although the moment diagrams in the sides b of the bin walls and the one shown in Figure 5.4, have the same shape, i.e., the series expansion of the moment equation [8] has the same form as the one used in the equation for the effective width derivation, the positive moment region of the span is slightly different, being 0.7b in the present analysis.

Using the diagram values developed for a simply supported stiffened plate, then at the middle span for: $t \to 0$, $\nu = 0.3$ and $\lambda/\ell = 0.197$ [1], the effective width for an effective span of 0.7b is:

$$\lambda_1 = 0.197(0.7)b = 0.138b$$

Using Figure 5.4 values, we obtain:

$$\lambda_2 = 0.106b = 0.106(25) = 2.65' < \lambda_1.$$

For both sides of the horizontal stiffeners, the total effective width of the plate is $2\lambda_2 = 2(2.65) = 5.3'$. Considering the spacing of 4'-6.7', the above value of 2λ is an acceptable estimate, i.e., the plate material is fully utilized with the conservative assumption that the thickness of the plate is very small compared with the other dimensions of the stiffener.

The case of a continuous plate under uniform distributed load is very close to the present case, where the triangular loading is averaged to a uniform one, along the height of the bin.

In accordance with the above reasoning, the following values of effective widths from Figure 5.4, may be used for the preliminary design of the horizontal stiffeners:

(i) Positive moment $2\lambda \doteq 2.65 \times 12 = 63.6"$

(ii) Negative moment $2\lambda = (0.07)(25)(12)(2) = 42"$.

Regarding sides "a" design moments and effective width, since there is no change of moment sign between supports, where the moment is maximum, the effective width of a cantilever case [1] may be considered; which, for at least

TABLE 5.1 Design loads and effective widths for horizontal stiffeners

Panel	M_c ('k)	M_b ('k)	T_a (k)	T_b (k)	$2\lambda_c$ (in)	$2\lambda_b$ (in)
Top	-84.31	82.10	26.63	15.98	42.0"	63.6"
Middle	-182.07	177.30	57.3	34.50	42.0"	63.6"
Bottom	-199.48	194.27	63.0	37.80	42.0"	63.6"

half the span, justifies: $2\lambda = 0.264\ell$ or $2\lambda = 0.264(15) \times 12 = 47.52" > 2\lambda_c$, from Table 5.1.

The final choice of stiffener section, is the one for which, based upon the stiffener properties now, results in an equal or larger effective width, according to the theoretical value.

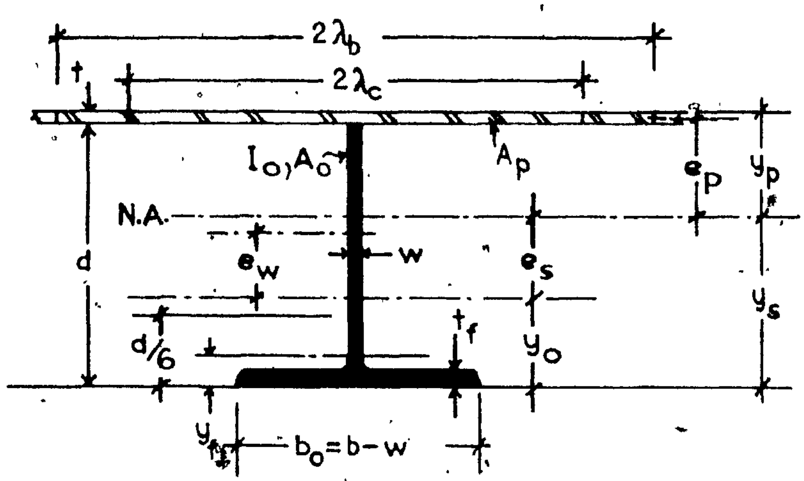

FIG. 5.5 Typical horizontal stiffener section geometry

The geometric properties of the stiffeners are :

d = 1/2 depth of section ; w = thickness of the web

t_f = thickness of the stiffener flange;

b_o = net width of flange

A = total area of section; A_f = area of stiffener flange

λ_c, λ_b = effective widths; t = thickness of plate

y_o = distance to neutral axis of stiffener alone

e_s, e_p = stiffener and plate section eccentricities in respect to N.A.

y_p, y_s = distances to outer fibers from N.A.

A_o, A_p = cross-sectional areas of stiffener and plate

I_o = moment of inertia of stiffener; y_f = distance to neutral axis of compression zone regarding local buckling.

(a) Stiffeners of the first panel: 0' - 20"

A half-section W24 × 61 [5], may be tried, having the following properties, according to Figure 5.5.

$$2\lambda_c = 42"; \; 2\lambda_b = 63.6"; \; t = 0.085"$$

$$d = \frac{23.72}{2} = 11.86"; \; dw = \frac{994}{2} = 4.97 \text{ in}^2; \; w = 0.419"$$

$$A_o = 18/2 = 9 \text{ in}^2; \; b_o t_f = A_o - dw = 9 - 4.97 = 4.03 \text{ in}^2$$

$$t_f = 0.591; \; t_f/2 = 0.295; \; d/6 = 1.98"; \; b = 7.02"$$

Considering the arrangement in Figure 5.5:

$$y_o = \frac{b_o t_f^2 + wd^2}{2A_o} = \frac{4.03(0.591) + 4.97(11.86)}{2(9)} = 3.41 \text{ in}$$

$$I_o = \frac{wd^3}{12} + b_o t_f (y_o - t_f/2)^2 + dw(\frac{d}{2} - y_o)^2 =$$

$$= \frac{4.97(11.86)^2}{12} + 4.03(3.12)^2 + 4.97(2.52)^2 = 129.04 \text{ in}^4$$

(i) <u>In the negative moment region:</u>

$$A_p = 2\lambda_c.t = 42(0.085) = 3.57 \text{ in}^2$$

$$A = A_p + A_o = 3.57 + 9 = 12.57 \text{ in}^2$$

$$y_s = \frac{A_p d + A_o y_o}{A_T} = \frac{3.57(11.86) + 9(3.41)}{12.57} = 5.81 \text{ in}$$

$$y_p = 11.86 - 5.81 = 6.05 \text{ in}$$

$$e_p = y_p - t/2 = 6.05 - 0.04 = 6.01 \text{ in}$$

$$e_s = y_s - y_o = 5.81 - 3.41 = 2.40 \text{ in}$$

$$I = A_p e_p^2 + A_o e_s^2 + I_o = 3.57(6.01)^2 + 9(2.4)^2 +$$

$$+ 129.04 = 309.93 \text{ in}^2$$

Therefore,

$$S_p = \frac{I}{y_p} = \frac{309.93}{6.05} = 51.23 \text{ in}^3; \; S_s = \frac{309.93}{5.81} = 53.34 \text{ in}^3$$

and

$$f_{bt} = \frac{M_c}{S_p} = \frac{84.31 \times 12}{51.23} = 19.75 \text{ KSI};$$

$$f_{bc} = \frac{M_c}{S_s} = \frac{84.31 \times 12}{53.34} = 18.97 \text{ KSI}$$

Due to the axial tension $f_t = \frac{T_a}{A_T} = \frac{26.63}{12.57} = 2.12$ KSI, and the net stress in the outer fibers are:

$$f_t' = 19.75 + 2.12 = 21.87 \text{ KSI, in the plate and}$$

$$f_c' = 18.97 - 2.12 = 16.75 \text{ KSI, in the flange.}$$

The compression flange, assuming an unsupported length of 180 in, has to be checked for local buckling [5].

Then from [5]: $r_t = 1.72$; $d/A_f = \frac{11.86}{4.03} = 2.94$.

$$F_2 = \frac{12,000}{180(2.94)} = 22.68 \text{ KSI}$$

$$F_3 = \frac{149,000}{(180/173)^2} = 13.76 \text{ KSI}$$

$$F_1 = \sqrt{(F_2)^2 \ (F_3)^2} = \sqrt{(22.68)^2 \ (13.76)^2} = 26.52 \text{ KSI}$$

$$2/3\ F_{bt} = 2/3(0.6)(36) = 14.4 < F_1$$

Therefore,

$$F_{bc} = 1.15\ F_{bt}(1-\frac{0.28F_{bt}}{F_1}) = 1.15(21.6)(1-\frac{0.28(21.6)}{26.52})$$

or

$$F_{bc} = 19.18 \text{ KSI}$$

Comparing allowable and existing stresses, stiffener section 1/2W 24 × 61 is adequate for preliminary design.

(ii) <u>In the positive moment region:</u>

$$A_p = 63.6(0.085) = 5.41 \text{ in}^2$$

$$A = 5.41 + 9 = 14.41 \text{ in}^2$$

$$y_s = \frac{5.41(11.86) + 9(3.41)}{14.41} = 6.58 \text{ in}$$

$$y_p = 11.86 - 6.58 = 5.28 \text{ in}$$

$$I = 5.41(5.24)^2 + 9(3.17)^2 + 129.04 = 368.03 \text{ in}^4$$

Therefore

$$S_p = \frac{368.03}{5.28} = 69.7 \text{ in}^3$$

$$S_s = \frac{368.03}{6.58} = 55.93 \text{ in}^3$$

The maximum tension occurs in the flange of the stiffener for which

$$f_t' = \frac{M_b}{S_s} + \frac{T_b}{A_T} = \frac{82.10 \times 19}{55.93} + \frac{15.98}{14.41} = 18.72 \text{ KSI}$$

with a maximum compression in the equivalent T-section of

$$f_c' = \frac{M_b}{S_p} - \frac{T_b}{A_T} = \frac{82.10 \times 12}{69.7} - \frac{15.98}{14.41} = 13.03 \text{ KSI}$$

Therefore, the trial section is appropriate for the middle span of sides "b".

(iii) For the exact computation of the value of the effective width at the midspan, where compression exists in the plate, the following formulae, for uniformly distributed load over multispan stiffened plate, may be applied.[ ]

$$\frac{\lambda_1}{L} = \frac{\Sigma}{1-q\Sigma}\,\frac{2}{\pi(3-\nu)(1+\nu)} \qquad (5.5)$$

where

$$q = \frac{4tL}{\pi(3-\nu)(1+\nu)}\left(\frac{e_o^2}{I_o} + \frac{1}{A_o}\right) \qquad (5.6)$$

and

$$\Sigma = \sum_{n=1}^{\infty} \frac{12(-1)^{n+1}}{n^2\pi^2\left\{q + \frac{n}{\sinh\frac{n\pi a}{L}}\left[\cosh\frac{n\pi a}{L} - \left(\frac{1+\nu}{3-\nu}\right)\frac{n\pi a}{L\sinh\frac{n\pi a}{L}}\right]\right\}} \qquad (5.7)$$

Using the following values:

$t = 0.085"$; $L = \frac{25}{2} \times 12 = 150"$; $a = \frac{6.67}{2} \times 12 = 40"$

$e = 11.82 - 3.41 = 8.41"$; $I_o = 129.04\ in^4$; $A_o = 9\ in^2$

Therefore,

$$q = \frac{4(0.085)(150)}{\pi(2.7)(1.3)}\left(\frac{(8.41)^2}{129.04} + \frac{1}{9}\right) = 3.05$$

$$a/L = \frac{40}{150} = 0.267; \quad \frac{1+\nu}{3-\nu} = \frac{1+a3}{3-0.3} = 0.481$$

$$\frac{2}{\pi(3-\nu)(1+\nu)} = \frac{2}{\pi(2.7)(1.3)} = 0.181$$

Then Equation (5.3) becomes:

$$\Sigma = \sum_{n=1}^{\infty} \frac{12(-1)^{n+1}}{n^2\pi^2\left\{1 + \frac{n}{\sinh n\pi(0.267)}\left[\cosh n\pi(0.267) - \frac{n(0.481)(0.267)}{\sinh n\pi(0.267)}\right]\right.}$$

For $\quad n = 26 \quad \Sigma = 0.254$

and $$\frac{\lambda_1}{L} = \frac{0.254}{1 - 3.05(0.254)}(0.181) = 0.207$$

or $$2\lambda_1 = 2(150)(0.207) = 62.1"$$

This value is almost equal to the assumed value of $2\lambda_b$ = 63.6" because in the latter, the thickness of the plate was considered infinitesimally small. The stress computations are not to be repeated, since the difference between the two effective widths is less than 1%.

(iv) For maximum deflection calculations of the stiffening frame members, the smaller between the average of the effective widths along the span and, the spacing between the frames, will be considered.

For panel 0'-20" the spacing is at 6'-8" or 80" and the average of the effective widths is smaller than the moment of inertia for deflection calculations and is:

$$I_a = (368.03+309.93)\frac{1}{3} = 339 \text{ in}^4$$

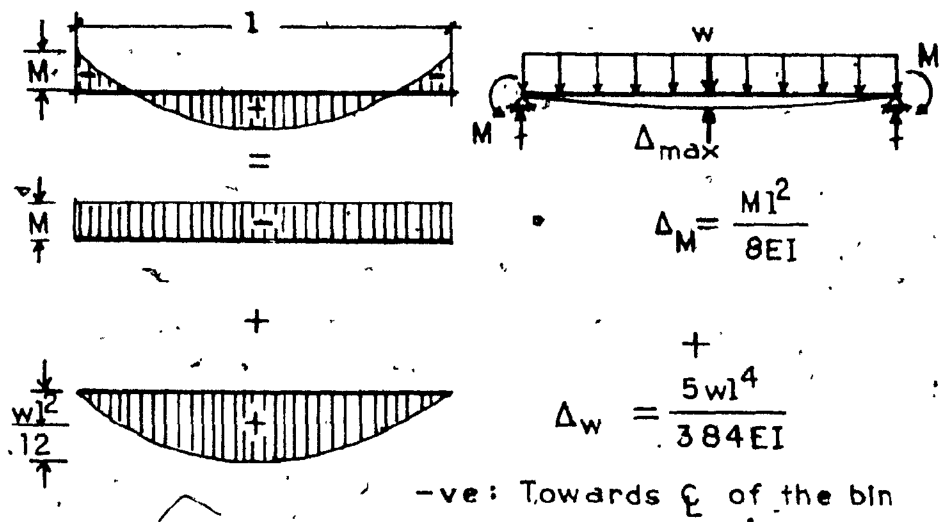

FIG. 5.6 Loading and corresponding moment diagrams for the horizontal stiffening frame members

The maximum deflection of a simply supported beam subjected to moments at its supports as shown in Fig.5.6 may be considered valid for the present case [4]:

where:

Δ_{max} = The maximum deflection at middle span of beams on "2" or "b" side;

Δ_M = The maximum deflection due to equal and opposite support moments;

Δ_W = The deflection due to a uniform load.

For the panel 0'-20" frame members and from Fig.5.6:

$$M = 84.31 \text{ Ft.K} \quad \text{and} \quad w = 0.32(6.67) = 2.13 \text{ KLF}$$

(a) For members a: $\ell = 15'$

$$\Delta_{max} = \frac{5wL^4}{384EI} - \frac{M\ell^2}{8EI} = \frac{\ell^2}{8EI}\left(\frac{5w\ell^2}{48} - M\right) \qquad (5.8)$$

and

$$\Delta_{max} = \frac{(12\times15)^2 \times 12}{8(29.10^3)(339)}\left(\frac{5(2.13)(15)^2}{48} - 84.31\right) =$$

$$= - 0.170"$$

(b) For members b: $\ell = 25'$

$$\Delta_{max} = \frac{(12\times25)^2 \times 12}{8(29.10^3)(339)}\left(\frac{5(2.13)(25)^2}{48} - 84.31\right) =$$

$$= 0.747"$$

Considering deflection requirements, the allowable deflections are: $\frac{15\times12}{200} = 0.9"$ for walls "a" and $\frac{25\times12}{200} = 1.5"$ for walls "b", with: $\frac{60\times12}{480} = 1.5"$, sway, in any location on the walls. [5]

Thus, 1/2 W24×61 may be used for the horizontal stiffening frame members in the first panel, at 6'-8" spacing.

(b) Stiffeners of the middle panel

A half W27×145 section may be tried having the following properties, according to Fig. 5.5 [5]

$$2\lambda_c = 42"; \quad 2\lambda_b = 63.6"; \quad t = 0.149"$$

$$d = \frac{26.88}{2} = 13.44"; \quad dw = 16.1/2 = 8.05 \text{ in}^2; \quad w = 0.6"$$

$$A_o = \frac{42.7}{2} = 21.35 \text{ in}^2; \quad b_o . t_f = A_o - dw = 13.3 \text{ in}^2$$

$$t_f = 0.975 \text{ in}; \quad t_f/2 = 0.49"; \quad d/A_f = 1.01; \quad r_t = 3.71 \ [5]$$

Considering the arrangement in Fig.5.5:

$$y_o = \frac{13.3(0.975) + 8.05(13.44)}{2(21.35)} = 2.84"$$

$$I_o = \frac{8.05(13.44)^2}{12} + 13.3(2.35)^2 + 8.05(3.88)^2 = 316.0 \text{ in}^4$$

(i) In the negative moment region:

$$A_p = 42.(0.149) = 6.26 \text{ in}^2$$

$$A = 6.26 + 21.35 = 27.61 \text{ in}^2$$

$$y_o = \frac{6.26(13.44) + 21.35(2.84)}{27.61} = 5.24 \text{ in.}$$

$$y_p = 13.44 - 5.24 = 8.20 \text{ in}$$

$$I = 316 + 6.26(8.13)^2 + 21.35(2.4)^2 = 852.74 \text{ in}^4$$

$$S_p = \frac{852.74}{8.2} = 104 \text{ in}^3; \; S_s = \frac{852}{5.24} = 162.6 \text{ in}^3$$

For $b/2t_f = \frac{13.97}{2(0.975)} = 7.16$ and $\frac{64}{\sqrt{F_y}} = 10.67 > \frac{b}{2t_f}$ from [5], the section may be considered as compact with $F_{bt} = 0.66(36) = 23.76$ KSI. The negative moment being, from Table 5.1,

$M_c = 182.07$'K, therefore

$$f_{bt} = \frac{(182.07)(12)}{104} = 21.01 \text{ KSI, and}$$

$$f'_c = 13.44 - 2.08 = 11.36 \text{ KSI} = 0.32 F_y$$

(ii) <u>In the positive moment region:</u>

$$A_p = 63.6(0.149) = 9.48 \text{ in}^2; \; A = 9.48 + 21.35 = 30.83 \text{ in}^2$$

$$y_s = \frac{9.48(13.44) + 21.35(2.84)}{30.83} = 6.1 \text{ in}$$

$$y_p = 13.44 - 6.1 = 7.34 \text{ in}$$

$$I = 316.0 + 21.35(3.26)^2 + 9.48(7.27)^2 = 1044 \text{ in}^4$$

$$S_p = \frac{1044}{7.34} = 142.23 \text{ in}^3; \; S_s = \frac{1044}{6.1} = 171.15 \text{ in}^3$$

For $M_b = 177.3$'K and $T_b = 57.5$ KSI

$$f'_t = \frac{177.3\times12}{171.15} + \frac{57.5}{30.83} = 14.3 \text{ KSI}$$

$$f'_c = \frac{177.3\times12}{142.23} - 1.87 = 13.09 \text{ KSI}$$

(iii) Considering Equation (5.3:

$$q = \frac{4(0.149)(150)}{\pi(2.7)(1.3)}\left(\frac{(10.6)^2}{316} + \frac{1}{21.35}\right) = 3.26$$

$$\frac{a}{L} = \frac{5}{25} = 0.2; \quad \frac{\pi a}{L} = 0.628$$

and

$$\sum_{n=1}^{\infty} = \frac{12(-1)^{n+1}}{n^2\pi^2\left\{3.26 + \frac{n}{\sinh n(0.628)}\left[\cosh n(0.628) - \frac{n(0.265)}{\sinh n(0.628)}\right]\right.}$$

For $n = 26$ $\Sigma = 0.216$

and

$$\frac{\lambda_1}{L} = \frac{(0.216) \times (0.181)}{1 - 3.26(0.216)} = 0.223$$

Therefore

$$2\lambda_1 = 25 \times 12 \times (0.223) = 66.9" > 63.6"$$

(iv) $I_A = (1044+852.7)\frac{1}{2} = 948 \text{ in}^4$

$$w = 0.92 \times 5 = 4.6 \text{ KLF};$$

Then, for sides "a" deflection:

(a) $\Delta_{max} = \frac{(12\times15)^2 \times 12}{8(29.10^3)948}\left(\frac{5(4.6)(15)^2}{48} - 182.07\right) = -0.131"$

and for sides "b" deflection:

(b) $\Delta_{max} = \frac{(12\times25)^2 \times 12}{8(29.10^3)948}\left(\frac{5(4.6)(25)^2}{48} - 182.07\right) = 0.577"$

A $\frac{1}{2}$ W27 × 145 may be used for the middle panel horizontal stiffener frame members, at 5'-0" spacing.

(c) Stiffeners of the bottom panel

A $\frac{1}{2}$ W33 × 152 section may be tried having the following properties:

$$2\lambda_c = 42"; \quad 2\lambda_b = 63.6"; \quad t = 0.149"; \quad t/2 = 0.07"$$

$$d = 33.5/2 = 16.75"; \; dw = 21.3/2 = 10.65"; \; w = 0.635"$$

$$A_o = 44.8/2 = 22.4 \text{ in}^2; \; b_o t_f = 22.4 - 10.65 = 11.75"$$

$$t_f = 1.06; \; t_f/2 = 0.53"; \; d/A_f = 2.87; \; r_t = 2.96"$$

$$y_o = \frac{11.75(0.53) + 10.65(8.38)}{22.4} = 4.26"$$

$$I_o = \frac{(16.75)^2(10.65)}{12} + 10.65(4.12)^2 + 11.75(3.73)^2 =$$

$$= 593.25 \text{ in}^4$$

(i) In the negative moment region

$$A_p = 6.26 \text{ in}^2; \qquad A = 6.26 + 22.4 = 28.66 \text{ in}^2$$

$$y_s = \frac{6.26(16.82) + 22.4(4.26)}{28.66} = 7.0"$$

$$y_p = 16.9 - 7 = 9.7"$$

$$I = 593.25 + 22.4(2.74)^2 + 6.26(7.12)^2 = 1088.7 \text{ in}^4$$

$$S_p = \frac{1088.7}{9.7} = 112.24 \text{ in}^3; \quad S_s = \frac{1088.7}{7.0} =$$

$$= 155.53 \text{ in}^3$$

$$b/2t_f = \frac{11.57}{2(1.06)} = 5.46 < 64/\sqrt{F_y} \quad [5]$$

Therefore, $F_{bt} = 0.66 F_y = 23.76$ KSI, as previously considered. $M_c = 199.48^K$; $T_a = 63^K$ and the combined stresses are:

$$f'_t = \frac{199.48 \times 12}{112.24} + \frac{63}{28.66} = 21.33 + 2.2 = 23.53 \text{ KSI}$$

$$f'_c = \frac{199.48 \times 12}{155.53} - 2.2 = 13.19 \text{ KSI}$$

(ii) <u>In the positive moment region</u>

$$A_p = 9.48 \text{ in}^2; \quad A = 9.48 + 22.4 = 31.88 \text{ in}^2$$

$$y_s = \frac{9.48(16.82) + 22.4(4.26)}{31.88} = 8 \text{ in}$$

$$y_p = 16.9 - 8 = 8.9 \text{ in}$$

$$I = 593.25 + 22.4(2.74)^2 + 9.48(7.12)^2 = 1242 \text{ in}^4$$

$$S_p = \frac{1242}{8.9} = 139.55 \text{ in}^3; \quad S_s = \frac{1242}{8} = 155.25 \text{ in}^3$$

$$M_b = 194.27 \text{ Ft-K} \; ; \; T_b = 37.8 \text{ K}$$

$$f'_t = \frac{194.27 \times 12}{155.25} + \frac{37.8}{31.88} = 16.2 \text{ KSI}$$

$$f'_c = \frac{194.27 \times 12}{139.55} - 1.19 = 15.52 \text{ KSI}$$

(iii) Check for exact effective width:

$$A_o = 22.4; \quad I_o = 593.25; \quad t = 0.149; \quad L = 150 \; ; \quad e = 12.56$$

Therefore

$$q = \frac{4(0.149)(150)}{\pi(2.7(1.3)}\left(\frac{(12.56)^2}{593.25} + \frac{1}{22.4}\right) = 2.52$$

$$\frac{a}{l} = \frac{4}{25} = 0.16; \quad \pi\frac{a}{L} = 0.503$$

and

$$\Sigma = \sum_{n=1}^{\infty} \frac{12(-1)^{n+1}}{n^2\pi^2\left\{2.52 + \frac{n}{\sinh n(0.503)}\left[\cosh n(0.503) - \frac{n(0.242)}{\sinh n(0.503)}\right]\right\}}$$

For $n = 26$ $\Sigma = 0.295$

and $$\frac{\lambda_1}{L} = \frac{0.295}{1 - 2.52(0.295)}(0.181) = 0.207$$

or $$2\lambda_1 = (300)(0.111) = 62.1"$$

(iv) Since the theoretical value of the effective width exceeds the spacing of the beams and the average of the effective widths used in the design of the sections, is larger than the spacing, as well, for deflection calculations the effective width of the plate is considered 48".

The moment of inertia may be computed as follows:

With $a_o = 22.4 \text{ in}^2$; $A_p = 48(0.149) = 7.15 \text{ in}^2$; $I_o = 593.25 \text{ in}^4$

and $y_s = \frac{7.15(16.82) + 22.4(4.26)}{29.55} = 7.30$ in

$$y_p = 16.9 - 7.3 = 9.6''$$

$$I = 593.25 + 22.4(3.04)^2 + 7.15(9.53)^2 = 1450 \text{ in}^4$$

$$w = 1.26 \times 4 = 5.04 \text{ KLF}$$

(a) For members side "a":

$$\Delta_{max} = \frac{(12\times15)^2 12}{8(29.10^3)1450}\left(\frac{5(5.04)(15)^2}{48} - 199.48\right) = -0.094''$$

(b) For members side "b":

$$\Delta_{max} = \frac{(15\times25)^2 12}{8(29.10^3)1450}\left(\frac{5(5.04)(25)^2}{48} - 199.48\right) = 0.413''$$

$\frac{1}{2}$ W33 × 152 may be used for the lower panel horizontal stiffening frame members, at 4'-0" spacing.

The deflections computed, so far, are the ones to be considered, with the condition that the stiffener frame members at the top of the walls do not deflect. In fact, this is not the case, because the assumed distributed load from the roof and its attachments does not provide any lateral frictional restraint to lateral displacements of the top beams. Therefore, revisiting Figure 5.6 and accounting for $I = I_o$ of the first panel, the requirements for Equations (5.1) and (5.8) are:

$$w = 0.13 \times \frac{6.67}{2} = 0.43 \text{ KLF}; \quad I_o = 129.0 \text{ in}^4$$

$$M_c = -\frac{0.434(225(0.6) + 625)}{12(1 + 0.6)} = 17.18 \text{ ft K}$$

$$\Delta_{max} = \frac{12(12 \times 15)^2}{8(29.10^3)(129.0)} \times [\frac{5(0.43) \times (225)}{48} - 17.18] =$$

$$= - 0.09"$$

for side "a", and

$$\Delta_{max} = \frac{12(12 \times 25)^2}{8(29.10^3)(129.0)}[\frac{5(0.43)(625)}{48} - 17.18] = 0.397"$$

for side "b".

The diagrams on Figure 5.7 have been constructed on the basis that the computed deflections occur at the middle, approximately of the panels and with the contribution of the vertical ribs and plate they assume the heavy lined curves on the left and right of the y-axis. Dashed lines are the representation of the center line of the middle spans under unloaded condition, which have to be shifted graphically towards the y-axis to construct the real displacement pattern, assuming infinitely flexible ribs. This assumed deflection pattern is according to the previously made approximation of uniform pressure acting on each of the three panels.

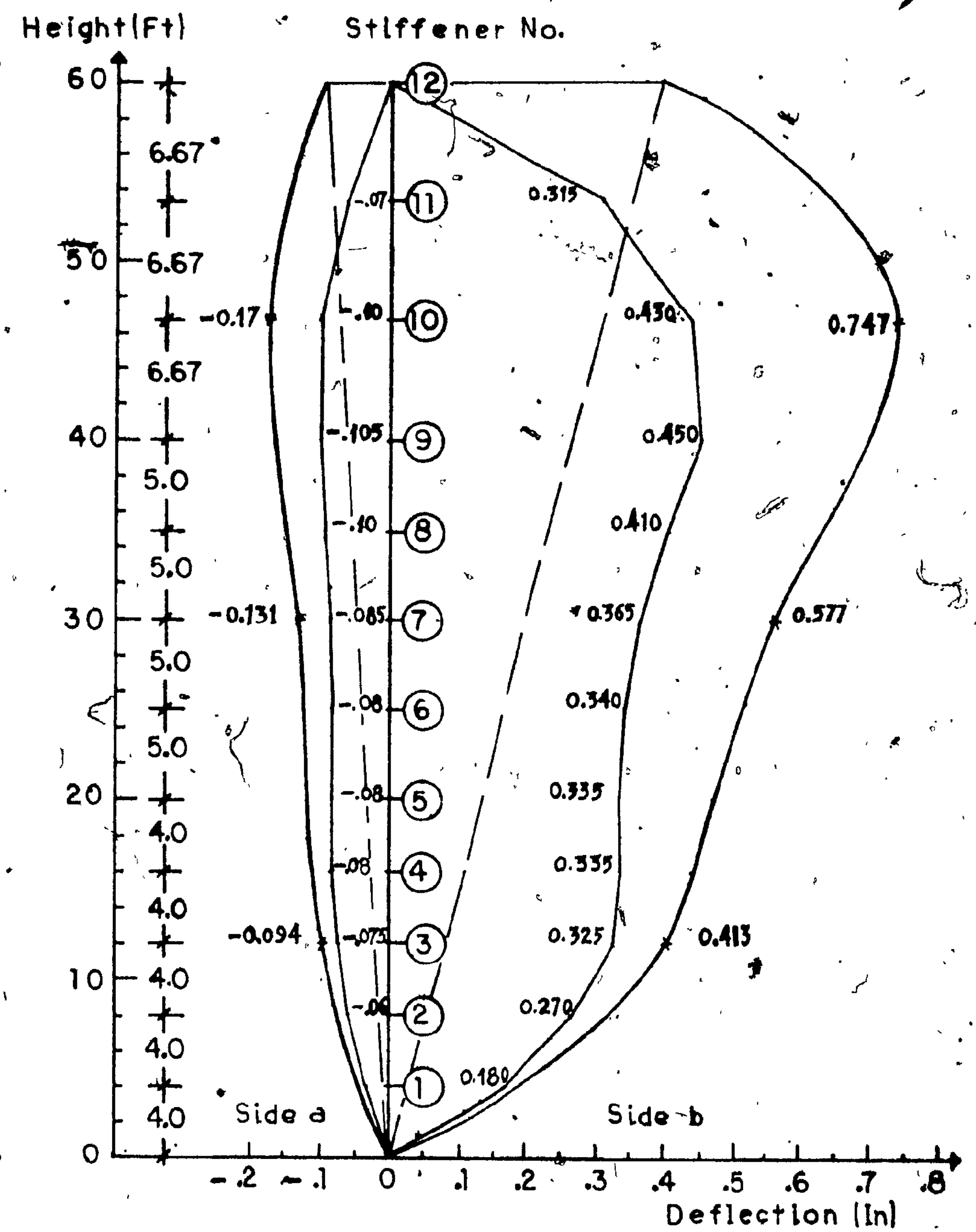

FIG. 5.7 Graphical representation of horizontal stiffeners midspan deflections

5.2.3 Analysis and design of the vertical stiffeners

The vertical stiffeners arranged as in Figure 5.3, are designed for the lateral and vertical loads and they have rectangular cross-sections, open ribs, acting integrally with the effective width of the plating, forming a T-section. This system resembles the one provided as a stiffening system of an orthotropic plate deck of a bridge. It is, therefore, appropriate to adopt the basic analysis technique used for orthotropic bridge decks, having in mind, though, the loading in the present case has a simpler form and it is considered acting uniformly throughout the deck.

Starting with the choice of the effective width, as in the case of the orthotropic decks with $D_x = 0$ and open ribs, the effective width of the plating acting with the rib in resisting the bending moments and shear is considered constant throughout the span. [2,6]

Two systems of rib action will be considered, as it was mentioned before; the one, with the horizontal beams being deflected and the one being rigid, following the assumed pattern of Figure 5.7.

The spacing of the ribs, for the derivation of the plate thickness, has been chosen 10" c/c, however, considering denting of the plating due to negative (suction) pressure induced by the eccentric location of the outlet [12], without changing the plate thickness, the spacing of the ribs may be finalized, at this point, to 9" c/c.

a) Ribs section geometry and allowable stresses

In the design of orthotropic steel decks the chosen effective width for a ribbed plate is considered being the same at the center of the span as well as over the supports of the ribbed plate. [6]

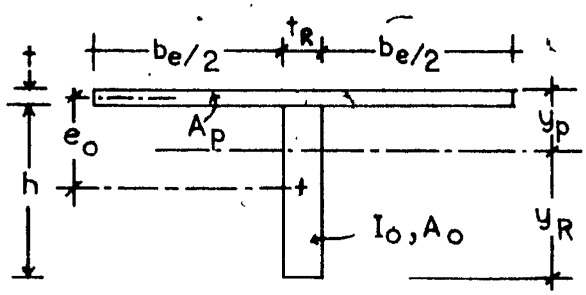

FIG. 5.8 Typical rib and plate element geometry

For the condition of flexible supports the effective width adopted in practice is up to 10% larger than the spacing of the ribs, assuming infinite length of effective span. [6]

In the present case, however, formula (5.7) may be used for effective width computations, in both support conditions provided that the adopted values do not exceed the actual spacing more than 10%.

The span lengths of the ribs are very closely proportioned to the pressure distribution in the panels of the wall, therefore, considering the different values of effective widths in each wall panel the same rib section may be adopted for the full height of the bin walls.

Following Canadian Steel Construction standards, usually adopted in the design of several industrial structures such as the present one and considering lateral support is provided to the ribs over the beam supports, the value of $64/\sqrt{Fy}$ may be used in deciding for the allowable compression stress safety factor in that region. [5]

The chosen rib section is 3 × 5/16".

The proportional limit stress for A36 steel is:

$$f_p = F_y/0.75 = 36 \times 4/3 = 48 \text{ KSI}$$

Following the analysis used in orthotropic bridge decks for derivation of allowable compressive stresses: [2]

$$t_R/h = \frac{5}{16\times3} = 0.104;$$

the ideal buckling stress is expressed as:

$$f_i = k(26,200)(\frac{t_R}{h})^2 \quad (5.8)$$

$$f_i = (1.0)(26,200)(0.104)^2 = 283 \text{ KSI} > f_p$$

Then, in computing the critical compression stress in the rib free end, the following formula may be adopted:

$$\frac{f_{cr}}{F_Y} = \frac{1}{1 + 0.1875(\frac{F_Y}{f_i})^2},$$

which yields

$$f_{cr} = 36/1 + (0.1875)(\frac{36}{283})^2 = 35.89 \text{ KSI}$$

For the chosen rib cross-section:

$$h/t_R = \frac{3\times16}{5} = 9.6 \text{ and } 64/\sqrt{F_Y} = 10.67$$

With $h/t_R < 64/\sqrt{F_Y}$, an allowable stress of 0.66 F_Y = 23.76 KSI may be used; the latter yields a safety factor:

$$S.F. = \frac{35.89}{23.76} = 1.51$$

In the design of orthotropic steel deck bridges safety factors of 1.5 to 1.85 are recommended. In the present case, however, considering already amplification of static loads and uniform pressure distribution, a factor of safety of 1.50 is acceptable. [2]

Following pressure distribution, as in Figure 5.3:

(f) For the first panel:

$$w_1 = 0.32(9/12) = 0.24 \text{ KLF}$$

and $$2L = 80'' \text{ or } 6.67'$$

with: $$e_o = 1.54''; \; I_o = \frac{27(5)}{16\times 12} = 0.703 \text{ in}^4;$$

$$A_o = \frac{5\times 3}{16} = 0.938 \text{ in}^2; \; t = 0.085''$$

and

$$\frac{2}{\pi(3-\nu)(1+\nu)} = 0.181;$$

the value of q from Equation (5.6) becomes:

$$q = 2(0.085)(40)(0.181)\left(\frac{(1.54)^2}{0.703} + \frac{1}{0.938}\right) = 5.46$$

with: $$\pi a/L = \frac{\pi(4.5)}{40} = 0.353 \text{ and } (1+\nu/1-\nu) = 0.481,$$

substituting in Equation (5.7) and summing for 26 terms the value of the effective width obtained as

$$b_e = 0.115(80) = 9.2''$$

Then, following cross-sectional geometry in Figure 5.8:

$$A_p = (9.2+5/16)(0.085) = 0.809 \text{ in}^2$$

$$A_p + A_o = 0.809 + 0.938 = 1.747 \text{ in}^2$$

$$y_p = \frac{0.809(0.043) + 0.938(1.585)}{1.747} = 0.871"$$

$$y_R = 3.085 - 0.871 = 2.214"$$

and

$$I_1 = 0.703 + 0.809(0.829)^2 + 0.938(0.714)^2 = 1.737 \text{ in}^4$$

Therefore

$$S_{p1} = \frac{1.737}{0.871} = 1.994 \text{ in}^3; \; S_{R1} = \frac{1.737}{2.214} = 0.785 \text{ in}^3$$

with:

$$\frac{I_1}{\ell_1} = \frac{1.737}{80} = 0.022.$$

(ii) <u>For the middle panel:</u>

$$w_2 = 0.92(9/12) = 0.69 \text{ KLF and } 2l_2 = 60" \text{ or } 5'$$

with: $e_o = 1.57"$; $I_o = 0.703 \text{ in}^3$; $A_o = 0.938 \text{ in}^2$

$$t = 0.149"; \; q = 2(0.149)(30)(0.181)\left[\frac{(1.57)^2}{0.703} + \frac{1}{0.938}\right]$$

or $$q = 7.4 \text{ and } \pi a/L = 0.471$$

From Equation (5.7) and n = 26:

$$b_e = 0.146(60) = 8.76"$$

Therefore:

$$A_p = 0.149 \times (8.76+5/16) = 1.352 \text{ in}^2$$

$$A_o = 0.938 \text{ in}^2; \quad A_o + A_p = 1.352 + 0.938 = 2.29 \text{ in}^2$$

$$y_p = \frac{0.938(1.649) + 1.352(0.075)}{2.29} = 0.720"$$

$$y_R = 3.149 - 0.720 = 2.429"$$

$$I_2 = 0.703 + 1.352(0.649)^2 + 0.938(0.929)^2 = 2.082 \text{ in}^4$$

and

$$S_{P2} = \frac{2.082}{0.720} = 2.892 \text{ in}^3; S_{R2} = \frac{2.082}{2.429} = 0.857 \text{ in}^3$$

$$I_2/\ell_2 = 2.082/60 = 0.035$$

(iii) <u>For the bottom panel:</u>

$$w_3 = 1.26(9/12) = 0.945 \text{ KLF and } 2L = 48" \text{ or } 4'$$

with

$$e_o = 1.57"; \; I_o = 0.703 \text{ in}^3; \; A_o = 0.938 \text{ in}^2$$

$$t = 0.149; \; q = 2(0.149)(24)(0.181)\left[\frac{(1.57)^2}{0.703} + \frac{1}{0.938}\right] = 5.92$$

with:

$$\pi a/L = \frac{\pi(4.5)}{24} = 0.589$$

From Equation (5.7) and n = 26:

$$b_e = 0.171(48) = 8.21"$$

$$A_p = 0.149(8.21+5/16) = 1.270 \text{ in}^2$$

$$A_o = 0.938 \text{ in}^2;\ A_o + A_p = 1.27 + 0.938 = 2.208 \text{ in}^2$$

$$\bar{y}_p = \frac{0.938(1.649) + 1.270(0.075)}{2.208} = 0.747"$$

$$y_R = 3.149 - 0.747 = 2.405"$$

$$I_3 = 0.703 + 1.27(0.673)^2 + 0.938(0.905)^2 = 2,046 \text{ in}^4$$

$$S_p = \frac{2.046}{0.747} = 2.740 \text{ in}^3;\ S_R = \frac{2.046}{2.405} = 0.851$$

$$I_3/\ell_3 = 2.046/48 = 0.043$$

b) Support moments

i) Rigid supports system - System I

The moment diagrams resulting from this analysis are applicable for all stiffeners at the vicinity of the corners of the bin walls and may be extended slightly beyond the inflection points of the deflected members of the horizontal stiffening frames.

Then, considering the application of the moment distribution method, the fixed end moments in the spans, as is shown in Figure 5.7, assigning clockwise direction as negative, are:

$$(0\text{-}1):\quad FEM = -w_3\ell^2/8 = \frac{0.945(4)^2}{8} \times 12 = -22.68\text{"K}$$

$$(1\text{-}2)\text{ to }(4\text{-}5):\quad FEM = \pm w_3\ \ell^2/12 = \pm 22.68\left(\frac{8}{12}\right) = \pm 15.12\text{"K}$$

$$(5\text{-}6)\text{ to }(8\text{-}9):\quad FEM = \pm \frac{0.69(5)^2}{12}(12) = \pm 17.25\text{"K}$$

$$(9\text{-}10)\text{ to }(10\text{-}11):\quad FEM = \pm 0.24(6.67)^2 = \pm 10.68\text{"K}$$

$$(11\text{-}12):\quad FEM = +\frac{0.24(6.57)^2 \times 12}{8} = 16.02\text{"K}$$

The distribution factors between spans of equal lengths are equal to 0.5. Accounting for pin support at the ends:

$$D_{1-0} = \frac{3/4(I_1/\ell_1)}{(I_1/\ell_1) + 3/4(I_1/\ell_1)} = 0.43;\ D_{1-2} = 0.57$$

as well as $D_{11-12} = 0.32$ and $D_{11-10} = 0.57$.

Since change of section and length occurs at supports 5 and 9, it follows:

$$D_{5-4} = \frac{I_3/\ell_3}{I_3/\ell_3 + I_2/\ell_2} = \frac{0.043}{0.043 + 0.035} = 0.55;$$

Similarly,

$$D_{9-8} = \frac{0.035}{0.035 + 0.022} = 0.61.$$

TABLE 5.2 Moment distribution, System I

No.	[1]		[2]		[3]		[4]		[5]		[6]
	-.43	-.57	-.5	-.5	-.5	-.5	-.5	-.5	-.55	-.45	-.5
1	-22.68	+15.12	-15.12	+15.12	-15.12	-15.12	-15.12	-15.12	-15.12	+17.25	-17.25
	-3.25	+4.31	0	0	0	0	0	0	-1.17	-0.96	0
2	0	0	+2.16	0	0	0	0	-0.59	0	0	-0.48
	0	0	-1.08	-1.08	0	0	+0.29	+0.29	0	0	+0.24
3	0	-0.54	0	0	-0.54	+0.14	0	0	+0.14	+0.12	0
	+0.23	+0.31	0	0	+0.2	+0.2	0	0	+0.14	-0.12	0
4	0	0	+0.16	+0.1	0	0	+0.1	-0.07	0	0	-0.06
	0	0	-0.13	-0.13	0	0	-0.08	-0.08	0	0	-0.02
5	0	-0.07	0	0	-0.06	-0.04	0	0	-0.04	-0.01	0
	+0.03	+0.04	0	0	+0.05	+0.05	0	0	+0.03	+0.02	0
6	0	0	+0.02	+0.02	0	0	+0.02	+0.02	0	0	+0.01
	0	0	-0.02	-0.02	0	0	-0.02	-0.02	0	0	0
Σ	-19.16		-14.01		-15.47		-14.81		-16.29		-17.57

TABLE 5.2 (continued)

No.	[6]	[7]		[8]		[9]		[10]		[11]	
	-.5	-.5	-.5	-.5	-.5	-.6	-.39	-.5	-.5	-.57	-.43
1	+17.25	-17.25	+17.25	-17.25	+17.25	-17.25	+10.68	-10.68	+10.68	-10.68	+16.02
	0	0	0	0	0	+4.01	+2.56	0	0	-3.04	-2.3
2	0	0	0	0	+2.0	0	0	+1.28	-1.52	0	0
	+0.24	0	0	-1.0	-1.0	0	0	+0.12	+0.12	0	0
3	0	+0.12	-0.5	0	0	-0.5	0.6	0	0	0.06	0
	0	+0.19	+0.19	0	0	0.27	0.17	0	0	-0.03	-0.03
4	+0.09	0	0	+0.09	+0.14	0	0	+0.08	-0.02	0	0
	-0.02	0	0	-0.11	-0.11	0	0	-0.03	-0.03	0	0
5	0	-0.01	-0.05	0	0	-0.05	-0.03	0	0	-0.02	0
	0	+0.03	+0.03	0	0	+0.05	+0.03	0	0	+0.01	0
6	-0.01	0	0	+0.02	+0.02	0	0	+0.01	-0.01	0	0
	0	0	0	-0.02	-0.02	0	0	0	0	0	0
Σ		-16.92		-18.27		-13.47		-9.23		-13.70	

ii) Flexible Supports System - System II

a) Side a

The deflections of the horizontal stiffeners are presented in Fig.5.7, and they correspond to support rises of the continuous vertical stiffeners. Considering direct analogy of flexural deformations to induced moments at the supports of the ribs and following Fig.5.9 designations, the fixed end moments for the members are tabulated in Table 5.3 [4].

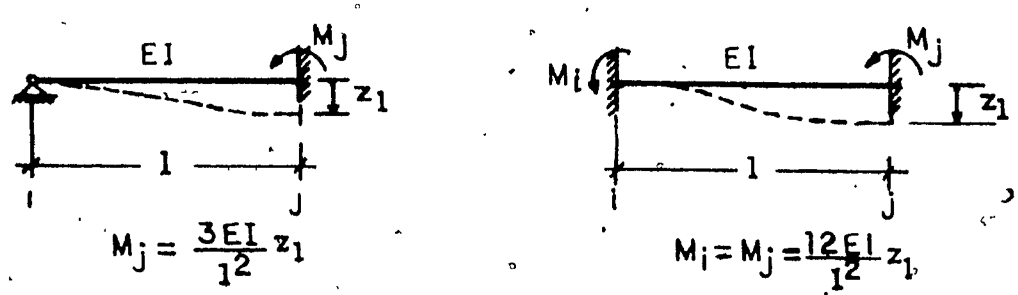

FIG. 5.9 Fixed end moments of beams subjected to settlement of support

b) Side b

In a similar manner the corresponding fixed end moments for the ribs on wall "b" are tabulated in Table 5.3. The settlement z_ℓ in Fig.5.9, drawn in the positive direction, is the difference between settlements at the joints i and j, also counterclockwise moments are considered positive.

In order to obtain the final support moments for sides a and b, moment distribution is applied, as carried out in Tables 5.4 and 5.5.

TABLE 5.3 Support displacements and induced fixed end moments (in-K)

Span	Side a		Side b		I	ℓ	M_a	M_b
	Z_j	Z_ℓ	Z_j	Z_ℓ				
0-1	-0.03	0.03	+0.18	+0.18	2.046	48	-2.27	+13.59
1-2	-0.06	-0.03	+0.270	+0.09	2.046	48	-9.06	+27.18
2-3	-0.075	0.015	+0.325	+0.053	2.046	48	-4.53	+16.01
3-4	-0.08	-0.015	+0.335	+0.010	2.046	48	-1.51	+6.04
4-5	-0.08	0	+0.335	0.0	2.046	48	0	0
5-6	-0.08	0	+0.340	+0.005	2.082	60	0	+0.99
6-7	-0.085	-0.005	+0.365	+0.025	2.082	60	-0.99	+4.95
7-8	-0.100	-0.015	+0.410	+0.045	2.082	60	-2.9	+8.91
8-9	-0.105	-0.005	+0.450	+0.040	2.082	60	-0.99	+7.92
9-10	-0.10	+0.005	+0.430	-0.020	1.737	80	+0.47	-1.87
10-11	-0.07	+0.03	+0.315	-0.075	1.737	80	+2.82	-10.76
11-12	0	+0.07	+0.0	-0.315	1.737	80	+1.65	-7.37

TABLE 5.4 Moments in ribs at supports, System II, side a

	1		2		3		4		5		6		7		8		9		10		11	
	-.43	-.57	-.5	-.5	-.5	-.5	-.5	-.5	-.5	-.45	-.5	-.5	-.5	-.5	-.5	-.5	-.61	-.39	-.5	-.5	-.57	-.43
1	-227 +487	-906 +647	-906 +680	-453 +680	-453 +302	-151 +302	-151 +76	0 +76	0 0	0 0	0 +45	-99 +45	-99 +195	-290 +195	-290 +195	-99 +195	-99 +32	+47 +20	+47 -165	+282 -165	+282 -255	+165 -192
2	0 -146	+340 -194	+324 -237	+151 -237	+340 -189	+38 -189	+151 -76	0 -76	0 -13	+23 -10	0 -49	+98 -49	+23 -61	+98 -61	+98 -57	+16 -57	+98 -9	-83 -6	+10 +59	-127 +59	-83 +47	0 +36
3	0 +51	-118 +67	-97 +96	-95 +96	-118 +78	-38 +78	-95 +51	-6 +51	-38 +34	-24 +28	-5 +18	-30 +18	-25 +26	-28 +26	-30 +18	-5 +18	-28 0	+28 0	-3 -10	+24 -10	+29 -17	0 -12
4	0 -21	+48 -27	+34 -36	+39 -36	+48 -37	+26 -37	+39 -28	+17 -28	+26 -19	+9 -16	+14 -13	+13 -13	+9 -9	+9 -9	+13 -6	0 -6	+9 -2	-5 -2	0 +4	-8 +4	-5 +2	0 +2
5	0 +8	-18 +10	-13 +16	-19 +16	-18 +16	-14 +16	-18 +14	-10 +14	-14 +11	-6 +9	-8 +6	-4 +6	-6 +5	-3 +5	-4 +2	-1 +2	-3 0	+2 0	-2 0	+1 0	2 -1	0 -1
6	0 -3	+8 -5	+5 -6	+8 -6	+8 -8	+7 -8	+8 -7	+6 -7	+7 -6	+3 -4	+4 -3	+2 -3	+3 -2	+1 -2	+2 -1	0 -1	+1 0	0 0				
7	0 +1	-3 +2	-2 +3	-4 +3																		
Σ/100	+1.50		-1.40		-0.31		-0.36		-0.12		+0.09		+0.59		- 0.47		0		-0.60		0	

TABLE 5.5 Moments in ribs at supports, System II, side b

	1		2		3		4		5	
	-.43	-.57	-.5	-.5	-.5	-.5	-.5	-.5	-.55	-.45
1	+1369 -1757	+2718 -2330	+2116 -1859	+1601 -1859	+1601 -1103	+604 -1103	+604 -302	0 -302	0 -54	+99 -45
2	0 +400	-930 +530	-1165 +858	-552 +858	-930 +540	-151 +540	-552 +290	-27 +290	-151 +165	-149 +135
3	0 -184	+429 -245	+265 -268	+270 -268	+429 -287	+145 -287	+270 -179	+88 -179	+145 -131	+93 -107
4	0 +58	-134 +76	-123 +133	-143 +133	-134 +112	-89 +112	-143 +104	-66 +104	-89 +78	-53 +64
5	0 -28	+66 -38	+38 -47	+56 -47	+66 -59	+52 -59	+56 -48	+39 -48	+52 -43	+27 -36
6	0 +10	-24 +14	-16 +23	-29 +23	-24 +24	-24 +24	-29 +25	-22 +25	-24 +23	-18 +19
7	0 5	+12 -6	+7 -10	+12 -10	+12 -12	+12 -12	+12 -12	+12 -12	+12 -11	+7 -9
Σ/100	-1.37		-0.48		+2.35		+0.96		-0.28	

	6		7		8		9		10		11	
	-.5	-.5	-.5	-.5	-.5	-.5	-.61	-.39	-.5	-.5	-.57	-.43
1	+99 -297	+495 -297	+495 -693	+891 -693	+891 -842	+792 -842	+792 -369	-187 -236	-187 +632	-1076 +632	-1076 +1033	-737 +780
2	-23 +185	-347 +185	-149 +285	-421 +285	-347 +266	-185 +266	-421 +456	-316 +291	-116 -200	+516 -200	+316 -180	0 -136
3	+68 -105	+143 -105	+93 -113	+133 -113	+143 -186	+228 -186	+133 -20	-100 -13	+146 -28	-90 -28	-100 +57	0 +43
4	-53 +55	-56 +55	-53 +76	-98 +76	-56 +33	-10 +33	-98 +68	-14 +44	-6 -11	+28 -11	-14 +8	0 +6
5	+32 -35	+38 -35	+27 -22	+16 -22	+38 -36	+34 -36	+16 -7	-5 -4	+22 -13	+4 -13	-5 +3	0 +2
6	-18 +15	-11 +15	-18 +18	-18 +18	-11 +7	-4 +7	-18 +15	-6 +15	-2 0	+2 0	-6 +3	0 +3
7	+10 -10	+9 -10	+5 +1	+3 +1	+9 -8	+7 -8	+3 -2	0 -1	+7 -4	+1 -4	0 0	0 0
Σ/100	-0.77		-0.58		-0.99		+5.48		+2.40		-0.61	

TABLE 5.6 Superposition of bending moments, Systems I and II (in-K)

Support	System Ia,b		IIa	(I+II)a		IIb	(I+II)b	
	M-ve	M+ve	@Support	M-ve	M+ve	@Support	M-ve	M+ve
0	0	+13.22	0	0	+13.97	0	0	+12.54
1	-19.16	+6.22	+1.50	-17.66	+6.22	-1.37	-20.53	+5.29
2	-14.01	+8.06	-1.40	-15.51	+7.16	-0.48	-14.49	+9.0
3	-15.47	+7.66	-0.31	-15.78	+7.33	+2.35	-13.12	+9.32
4	-14.81	+7.25	-0.36	-15.17	+7.01	+0.96	-13.85	+7.64
5	-16.29	+8.95	-0.12	-16.41	+8.94	-0.28	-16.47	+8.48
6	-17.57	+8.68	+0.09	-17.48	+8.98	-0.77	-18.34	+7.96
7	-16.92	+8.29	+0.06	-16.33	+7.85	-0.58	-17.50	+7.5
8	-18.27	+10.01	-0.47	-18.74	+9.28	-0.99	-19.26	+12.26
9	-13.47	+4.67	0	-13.47	+4.37	+5.48	-7.99	+8.61
10	-9.23	+4.56	-0.60	-9.83	+4.26	+2.40	-6.83	+5.46
11	-13.70	+9.17	0	-13.70	+9.17	-0.61	-14.31	+8.87
12	0		0	0		0	0	

TABLE 5.7 Results of vertical stiffeners analysis

Span	System I Side a or b Moments		Systems I+II Side a Moments		Systems I+II Side b Moments		Maximum Stresses at Support (KSI)			Maximum Stresses at Midspan (KSI)			Section Moduli in^3	
	Max -ve	Avg +ve	Max -ve	Avg +ve	Max -ve	Avg +ve	Max -ve	Plate	Rib	Max +ve	Plate	Rib	S_P	S_R
0-1	-19.16	+13.22	-17.66	+13.97	-20.53	+12.54	-20.53	+7.49	-24.12	+13.97	-5.10	+16.42	2.740	0.851
1-2	-19.16	+6.22	-17.66	+6.22	-20.53	+5.29	-20.53	+7.49	-24.12	+6.22	-2.27	+7.31	2.740	0.851
2-3	-15.47	+8.06	-15.78	+7.16	-14.49	+9.00	-15.78	+5.76	-18.54	+9.00	-3.28	+10.58	2.740	0.851
3-4	-15.47	+7.66	-15.78	+7.33	-13.85	+9.32	-15.78	+5.76	-18.54	+9.32	-3.40	+10.95	2.740	0.851
4-5	-16.29	+7.25	-15.17	+7.01	-16.47	+7.64	-16.47	+6.01	-19.35	+7.64	-2.79	+8.98	2.740	0.851
5-6	-17.57	+8.95	-17.48	+8.94	-18.34	+8.48	-18.34	+8.89	-21.40	+8.95	-4.30	+10.44	2.082	0.857
6-7	-17.57	+8.69	-17.48	+8.98	-18.34	+7.96	-18.34	+8.89	-21.40	+8.98	-4.31	+10.48	2.082	0.857
7-8	-18.27	+8.29	-19.74	+7.85	-19.26	+7.50	-19.74	+9.48	-23.30	+8.29	-3.98	+9.67	2.082	0.857
8-9	-18.27	+10.01	-19.74	+9.28	-19.26	+12.26	-19.74	+9.48	-23.30	+12.26	-5.89	+14.31	2.082	0.857
9-10	-13.47	+4.67	-13.47	+4.37	-7.99	+8.61	-13.47	+6.76	-17.16	+8.61	-4.32	+10.97	1.994	0.785
10-11	-13.70	+4.56	-13.70	+4.26	-14.31	+5.46	-14.31	+7.18	-18.23	+5.46	-2.74	+6.96	1.994	0.785
11-12	-13.70	+9.17	-13.70	+9.17	-14.31	+8.87	-14.31	+7.18	-18.23	+9.17	-4.60	+11.68	1.994	0.785

The average value of the positive moment in the spans is computed as:

$$M_{+ve} = \frac{w^2}{8} - |M_i + M_j|\frac{1}{2} \qquad \text{(in-K)}$$

where:

w = the acting load on the span between supports i and j; and

M_i, M_j = the negative support moments for Systems (I) and (I+II).

5.2.4 Analysis of vertical loads and stiffened plate capacity

The vertical stiffeners attached to the plate not only subdivide the plate, thus increasing its stability alone, but they act together with the plate in transferring in-plane compressive forces. Following the analysis of a plate with multiple longitudinal stiffeners as shown in Figure 5.10, the following formula may be applied for the computation of the critical stress: [1]

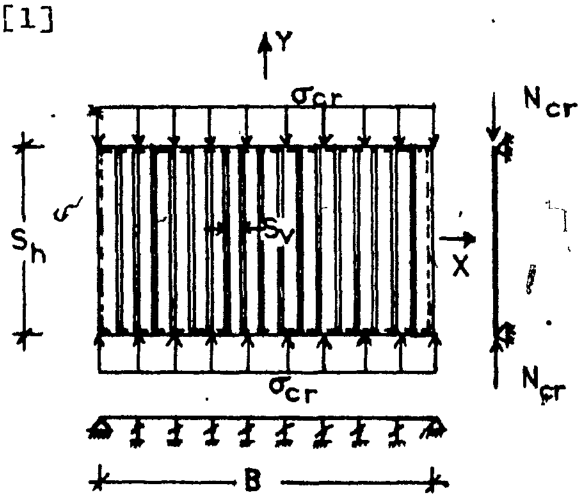

FIG. 5.10 Stiffened plate in compression

$$\sigma_{cr} = \frac{\pi^2 D}{B^2 t} \; \frac{(1+\beta^2)^2 + 2\sum_i \gamma_i \sin^2 \frac{\pi c_i}{B}}{\beta^2 (1 + 2\sum_i \delta_i \sin \frac{\pi c_i}{B})} \qquad (5.9)$$

where

$$D = \frac{Et^3}{12(1-\nu^2)};$$

t = the thickness of the plate;

c_i the distance of each stiffener from the support at $x = + \frac{B}{2}$, in the present case.

$$c_i = 9i; \; \beta = \frac{S_h}{B}; \; \delta_i = \frac{A_i}{Bt}$$

with $A = 0.938$

$$\gamma_i = \frac{EI_i}{BD} \;;$$

with $I_i = 0.703 \text{ in}^2$,

neglecting the increase of the moment of inertia taken about the plane of contact between the stiffener and the plate [1].

a) <u>Bottom panel critical stress computation</u>

(i) <u>Side a</u>:

$$B = 15 \times 12 = 180; \; \beta = \frac{4\times 12}{180} = 0.267; \; \delta = \frac{0.938}{180(0.149)} = 0.035$$

$$D = \frac{29000(0.149)^3}{12(0.91)} = 8.79 \text{ K-in}; \gamma = \frac{29.10^3(0.703)}{180(8.79)} = 12.89$$

$$\frac{\pi c_i}{B} = \frac{\pi(9)_i}{180} = 0.157i \text{ (rad)}$$

$$\sum_{i=1}^{20} \sin^2 0.157\, i = 10.0 \text{ , then}$$

$$\sigma_{cr} = \frac{\pi^2(8.79)[(1 + (0.267)^2)^2 + 2(12.89)(10)]}{(180)^2(0.149)(0.267)^2(1 + 2(0.035)10)} = 38.4 \text{ KSI}$$

Since yielding occurs at 36 KSI, it is appropriate to assume a factor of safety of 1.5 and the critical stress then becomes:

$$(\sigma_c)_{all.} = 38.4/1.5 = 25.6 \text{ KSI}$$

Consulting the final stress results in Table 5.7, the maximum compressive stress in the plate is 5.10 KSI. Since the ribs have already reached their compressive stress limit, 24 KSI, it is necessary to consider that the plate alone, will carry the in-plane compressive forces. Therefore, the compressive force that may be safely transmitted through the side a walls is:

$$W_{pa} = (25.6-5.10)(15) \times (12) = 2(0.149) = 1100 \text{ KIPS}$$

(ii) Side b

$$B = 25 \times 12 = 300;\ \beta = \frac{4\times12}{300} = 0.16;\ \delta = \frac{0.938}{300(0.149)} = 0.021$$

$$D = 8.79 \text{ K-in};\ \gamma = \frac{29.10^3(0.703)}{300(8.79)} = 7.73$$

$$\frac{\pi c_i}{B} = \frac{9.\pi}{300}\, i = 0.094\, i \text{ (rad)};\ i_{max} = \frac{25\times12}{9} = 33$$

$$\sum_{i=1}^{33} = \sin^2(0.094)i = 16.71,$$ substituting into Equation (5.9).

$$\sigma_{cr} = \frac{\pi^2(8.79)}{(300)^2(0.149)} \frac{((1 + (0.16)^2)^2 + 2(7.73)(16.71))}{(0.16)^2(1 + 2(0.021)(16.71))} =$$

$$= 38.52 \text{ KSI}$$

Again, $(\sigma_c)_{all.} = 38.52/1.5 = 25.68$ KSI

Following the same reasoning as for side a walls, the plate alone may transmit a compressive force of:

$$W_{pb} = (25.68-5.10)25 \times 12(0.149).2 = 1840 \text{ KIPS}$$

through walls b.

Then, the total compression in the ribbed walls may safely reach the value of:

$$W_p = 1840 + 1100 = 2940 \text{ KIPS}$$

The own weight of the structure is:

(i) Plating:

$$(3/16\times 40+1/8\ 20)\frac{80}{12}\times(0.49) = 33 \text{ KIPS}$$

(ii) Vertical stiffness:

$$(0.938)\frac{60}{144}\times\left(\frac{80\times 12}{9}\right)\times 0.49 = 21 \text{ KIPS}$$

(iii) Horizontal stiffness;

$$\frac{[(152\times 5)+(145\times 4)+(61\times 3)]}{2}\ \frac{80}{1000} = 61 \text{ KIPS}$$

$$W_w = 115 \text{ KIPS}$$

With reference to Section 4.3, the vertical frictional load at the bottom of the wall was found to be:

$$W_f = 1979 \text{ KIPS}$$

As a result, the total vertical load to be transmitted to the columns from the upper part of the bin is:

$$W_T = W_w + W_f + W_R = 115 + 1979 + 17 = 2110 \text{ KIPS}$$

Since $W_T < W_p$, there is no need of vertical corner stiffeners; however, an angle 4 × 4 × 7/16 (@ 11.3#/FT) will be provided at the corners of the bin walls.

CHAPTER VI

DESIGN OF THE HOPPER COMPRESSION FRAME

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DESIGN OF THE HOPPER COMPRESSION FRAME

6.1. DESIGN LOAD COMPUTATIONS

The members of the hopper compression frame located at the top of the hopper are designed for the horizontal components of the meridional hopper forces computed in Section

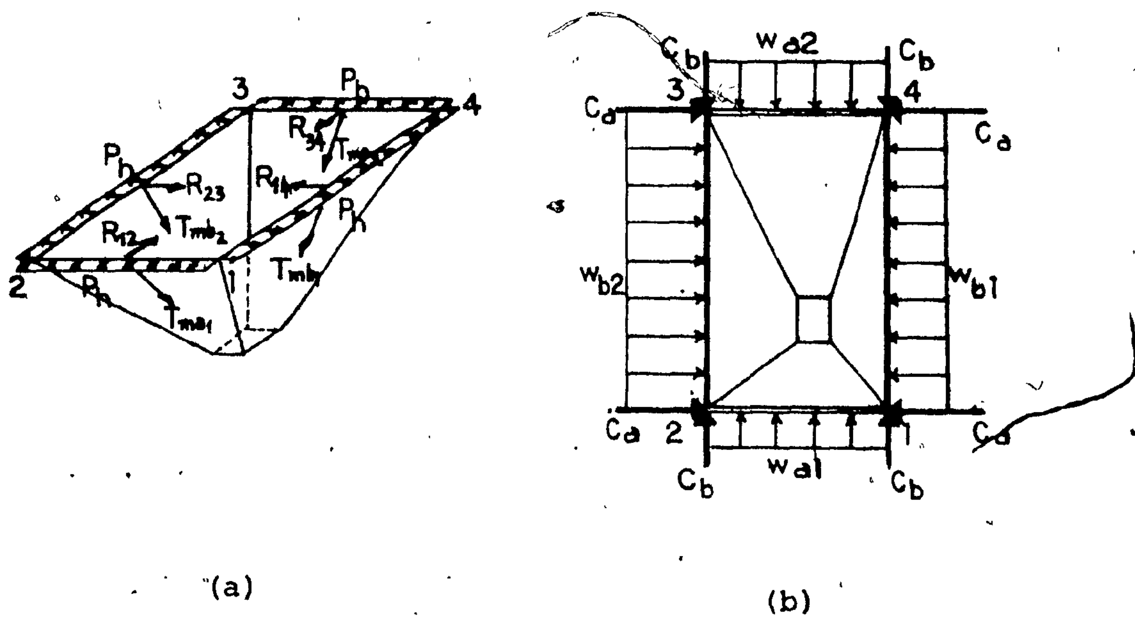

FIG.-6.1 Design loads for compression frame

3.3.2, as well as the effects of the horizontal pressure at the bottom of the vertical walls. These loads are presented in Figure 6.1(a) and their values may be computed as follows:

$$R_{12} = T_{ma_1} \cos\alpha_1 = 562 \cos(54.69) = 324.8K$$

$$R_{23} = T_{mb_2} \cos\beta_2 = 958 \cos(51.63) = 594.7K$$

$$R_{34} = T_{ma_2} \cos\alpha_2 = 763 \cos(36.06) = 616.8K$$

$$R_{14} = T_{mb_1} \cos\beta_1 = 837 \cos(65.38) = 348.7K$$

The lateral pressure from Figure 3.6 is 1.39 KSF; then if an effective height of 2.0' is considered for the compression frame, the values of the distributed load, acting outwards are:

$$p_h = 1.39(2) = 2.78 \text{ KLF}$$

Therefore, the net distributed loads acting on the frame members, shown in Figure 6.1(b), are:

$$w_{a1} = \frac{324.8}{15} - 2.78 = 18.87 \text{ KLF}$$

$$w_{b2} = \frac{594.7}{25} - 2.78 = 21.01 \text{ KLF}$$

$$w_{a2} = \frac{616.8}{15} - 2.78 = 38.34 \text{ KLF}$$

$$w_{b1} = 348.7 - 2.78 = 11.17 \text{ KLF}$$

The maximum compression forces that result from the above computed loads are:

$$C_a = -(w_{b2})\frac{b}{2} = -\ 21.01 \times \frac{25}{2} = -\ 262.6K$$

and

$$C_b = -(w_{a2})\frac{a}{2} = -\ 38.34 \times \frac{15}{2} = -\ 287.6K$$

Considering the frame action and moment distribution method, the respective fixed end moments for the frame members are:

$$(FEM)_{12} = -(FEM)_{21} = (wa_1)\frac{a^2}{12} = (18.87)(18.75) = 353.8'K$$

$$(FEM)_{23} = -(FEM)_{32} = (wb_2)\frac{b^2}{12} = (21.01)(52.08) = 10.94.3'K$$

$$(FEM)_{34} = -(FEM)_{43} = (wa_2)\frac{a^2}{12} = (38.34)(18.75) = 718.9'K$$

$$(FEM)_{41} = -(FEM)_{14} = (wb_1)\frac{b^2}{12} = (11.17)(52.08) = 581.7'K$$

For equal cross-sections of the frame members, the distribution factors at the corners are:

$$D_{12} = D_{21} = D_{34} = D_{43} = -\frac{1/15}{1/15 + 1/25} = -\ 0.62$$

and

$$D_{32} = D_{23} = D_{41} = D_{14} = -\ 1 + 0.62 = -\ 0.38$$

TABLE 6.1 Moment distribution for compression frame

1		2		3		4	
-0.38	-0.62	-0.62	-0.38	-0.38	-0.62	-0.62	-0.38
-581.7 +86.6	+353.8 +141.3	-353.8 -459.1	+1094.3 - 281.4	-1094.3 +142.7	+718.9 +232.7	-718.9 +85.1	+581.7 +52.1
+26.1 +77.3	+229.6 +126.1	+70.7 -88.1	+71.4 -54.0	-140.7 +37.3	+42.5 +60.9	+116.4 -99.0	+43.3 -60.7
-30.4 +28.3	-44.1 +46.2	+63.1 -50.7	+18.7 -31.1	-27.0 +29.1	-49.5 +47.1	+30.5 -42.7	+38.4 -26.2
-13.2 +14.7	-25.4 +23.9	+23.1 -23.4	+14.6 -14.3	-15.6 +14.1	-21.4 +22.9	+23.6 -23.4	+14.2 -14.4
-7.2 +7.2	-11.7 +11.7	+11.9 -11.8	+7.1 -7.2	-7.2 +7.2	-11.7 +11.8	+11.5 -11.7	+7.4 -7.2
-392.3		-818.2		-1054.4		-628.6	

The midspan moments are:

(i) Member 1-2: $\frac{(18.87)(15)^2}{8} - \left(\frac{392.3+818.2}{2}\right) = -74.5\text{'K}$

(ii) Member 2-3: $\frac{(21.01)(25)^2}{8} - \left(\frac{818.2+1054.4}{2}\right) = 705.1\text{'K}$

(iii) Member 3-4: $\frac{(38.34)(15)^2}{8} - \left(\frac{1054.4+628.6}{2}\right) = 236.8\text{'K}$

(iv) Member 4-1: $\frac{(11.17)(25)^2}{8} - \left(\frac{628.6+392.3}{2}\right) = 403.6\text{'K}$

The design loads, then, are:

Positive moment : 705.1'K

Negative moment : -1054.4'K

Axial load : -287.6'K (Compression)

5.2 CHOICE OF SECTION

A section formed from a WWF(M)20 × 230 and 2 plates of 17-3/8" may be tried as per Figure 6.2. [5].

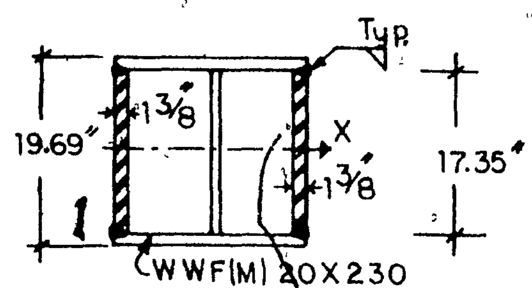

FIG. 6.2 Compression frame, built-up section

The moment of inertia of the plates is:

$$I_{xp} = \frac{(17.35)^3}{12} \times 2.75 = 1197 \text{ in}^4$$

The WWF section moment of inertia is: 4890 in^4; and the total moment of inertia is:

$$4890 + 1197 = 6807 \text{ in}^4$$

Thus, the sectional modulus is:

$$S_x = \frac{6087}{19.69} \times 2 = 618.28 \text{ in}^3$$

The total cross-sectional area is:

$$67.9 + 2.75(17.35) = 115.6 \text{ in}^2$$

with a compression stress due to the axial load equal to:

$$287.6/115.6 = 2.48 \text{ KSI}$$

The maximum compression stress due to bending is:

$$\frac{1054.4 \times 12}{618.28} = 20.46 \text{ KSI}$$

Thus, the maximum compression stress in the frame becomes:

$$20.46 + 2.48 = 22.94 \text{ KSI}$$

Since there is full lateral support and the section may be considered as compact with $F_c = 0.66(36) = 23.76$ KSI, the built-up section in Figure 6.2 may be used for the compression frame members.

CHAPTER VII

DESIGN OF THE HOPPER

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DESIGN OF THE HOPPER

7.1 STIFFENERS SPACING AND DESIGN OF PLATES

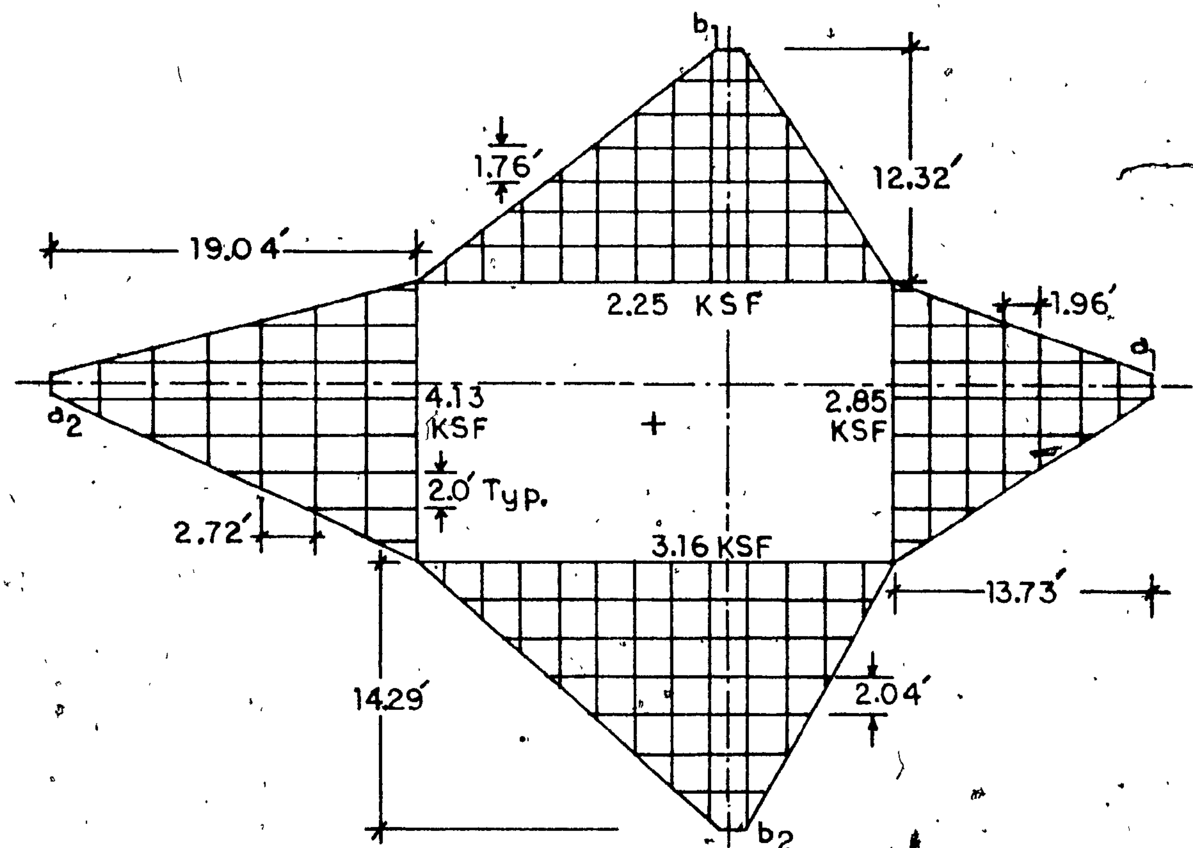

FIG. 7.1 Hopper stiffeners arrangement

The hopper inclined walls are composed of the plating and the horizontal and meridional stiffeners. The horizontal stiffeners being at the same elevation in all walls are dividing the developed surface of the walls into seven equal parts, as shown in Figure 7.1.

The meridional stiffeners are spaced at 2' distance in all walls. The plate is designed to carry the bending stresses due to the normal pressure, as well as the tensile stresses induced by the meridional and horizontal tensile forces.

However, considering that the stiffening members acting a portion of the plate, will carry all in-plane forces; the plate within the stiffeners may be designed as simply supported, at its allowable bending stress limit of 21.6 KSI.

Using Table values for uniformly loaded and simply supported rectangular plates [11], and from Figure 7.1:

(i) <u>For wall a_1:</u>

$$\frac{b}{a} = 2/1.96 = 1.02; \quad M_{max} = \beta w a^2 = 0.0479(2.85)(1.96)^2 =$$

$$= 0.524 \text{ K-in/in}$$

and from Section 3.1: $t = \sqrt{6M/f_b}$

and

$$t_{a1} = \sqrt{6(0.524)/21.6} = 0.382"$$

considering 1 mm corrosion allowance, then:

$$t_{a1} = \frac{1}{2}" \quad ((t_{a1})_{ef} = 0.46")$$

(ii) <u>For wall a_2:</u>

$$\frac{b}{a} = 2.72/2 = 1.36; \quad M_{max} = 0.0731(4.13)(2)^2 = 1.208 \text{ in K/in}$$

and

$$t_{a2} = \sqrt{6(1.208)/21.6} = 0.579";$$

with corrosion allowance:

$$t_{a2} = 5/8" \quad ((t_{a2})_{ef} = 0.586")$$

(iii) <u>For wall b_1:</u>

$$\frac{b}{a} = 2/1.76 = 1.14; \quad M_{max} = 0.0583(2.25).(1.76)^2 = 0.406$$

$$t_{b1} = \sqrt{6(0.406)/21.6} \doteq 0.336" \text{ or } t_{b1} = \frac{1}{2}"((t_{b1})_{ef} = 0.46")$$

(iv) <u>For wall b_2:</u>

$$\frac{b}{2} = 2.04/2 = 1.0; \quad M_{max} = 0.0479(3.16)4 = 0.605 \text{ in-K/in}$$

$$t_{b2} = \sqrt{6(0.605)/21.6} \doteq 0.41" \text{ or } t_{b2} = \frac{1}{2}"((t_{b2})_{ef} = 0.460")$$

7.2 DESIGN OF THE MERIDIONAL STIFFENERS

The design loads for these stiffeners are more severe in wall a_2, where:

(i) The tension per stiffener is, from Section 2.3.2:

$$T = 763/6 = 127 \text{ K}$$

and

(ii) The bending moment $M = \pm \frac{(4.13\times 2)(2.72)^2}{10} = 6.11'\text{K}$

An effective width of $40(t_{ef})$ may be assumed.

$$b_e = 40(0.586) \cong 20"; \; A_p = 20(0.586) = 11.72 \text{ in}^2$$

A section WT5 × 7.5 may be tried having [5]

$$A_s = 2.2 \text{ in}^2; \; I_s = 5.46 \text{ in}^4 \text{ and } y = 1.57"; \; d = 5.0"$$

The neutral axis is located at:

$$y_s = \frac{2.2(1.57) + 5.29(11.72)}{13.92} = 4.7"$$

$$y_p = 5.586 - 4.7 = 0.886"$$

The total moment of inertia is:

$$I_T = 5.46 + 2.2(3.14)^2 + 11.72(0.593)^2 = 31.27 \text{ in}^4$$

and the minimum section modulus is $S_p = \frac{31.27}{4.7} = 6.65 \text{ in}^3$.

The combined tensile stress is then:

$$f_t = \frac{127}{13.92} + \frac{6.11 \times 12}{6.65} = 20.15 \text{ KSI} < 21.6 \text{ KSI}$$

which occurs in the stiffener.

Therefore, WT5 × 7.5 may be used for the meridional stiffeners of the hopper walls.

7.3 DESIGN OF THE HORIZONTAL STIFFENERS

These stiffeners are designed as simply-supported beams, since the inclination of the walls varies and the corner moments are not distributed in a predictable fashion. They are carrying flexural stresses as well as axial tension due to the horizontal pressure exerted on the hopper walls.

Consulting results in Section 2.3.3, for the wall b_2 the first stiffener from the top of the hopper at

$$y = 2.04/\sin(65.38) = 2.24'$$

the tensile force is:

$$(T_{hal})(2.24) = (18.3 - 1.53(2.24))(2.24)$$

or

$$T = 33.32 \text{ K}$$

The span of the stiffener is: $b_Y = 25 - 2.08(2.24)$ or

or

$$\ell = 20.34 \text{ ft}$$

and the distributed load due to normal pressure on wall b_2: $w = 3.16(2.04) = 6.45$ KLF

Thus, the bending moment is:

$$M = \frac{wl^2}{8} = \frac{6.45(20.34)^2}{8} = 333.4 \text{ Ft-K}$$

Assuming an effective width of participating plate:

$$b_e = 40(t_{ef}) = 40(0.46) = 18"$$

then:

$$A_p = 18(0.46) = 8.28 \text{ in}^2$$

A section W16 × 96 may be tried with the following properties:

$$A_s = 28.2 \text{ in}^2;\ I_s = 1360 \text{ in}^4;\ y = \frac{16.32}{2} = 8.16 \text{ in}$$

The neutral axis is located at:

$$y_s = \frac{28.2(8.16) + 8.28(16.39)}{36.48} = 10.03"$$

$$y_p = 16.62 - 10.03 = 6.59"$$

The total moment of inertia is:

$$I = 1360 + 8.28(6.36)^2 + 28.2(1.87)^2 = 1,794 \text{ in}^4$$

and the section modulus for the stiffener part is:

$$S_s = 1794/10.03 = 178.82 \text{ in}^3$$

Therefore, the maximum tensile combined stress is:

$$f_t = \frac{333.4\ 12}{178.82} + \frac{33.32}{36.48} = 22.37 + 0.91 = 23.29 \text{ KSI}$$

The section is compact and F_t = 23.76 KSI, then W16 × 96 may be used for the horizontal stiffeners of the hopper walls.

CHAPTER VIII

SUMMARY

CHAPTER VIII

SUMMARY

The purpose of this report is to provide a safe and economical stiffening system for the walls of a rectangular steel bin and its hopper.

Regarding the results of the two options of the provided systems, there is a substantial advantage of the orthotropic one which, from derived results, from sections 5.2.4 and 4.3, a $(\frac{164-115}{164}100=)$ 29% savings in the steel weight may be achieved. Of course, the amount of welding is greatly increased, but nevertheless, the overall economic advantage is apparent.

The design of the hopper is somehow, conservative but considering adversity of the load application in that region, due to the eccentricity of the outlet, safety is of greater importance than anything else.

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REFERENCES

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APPENDIX A

DRAWINGS

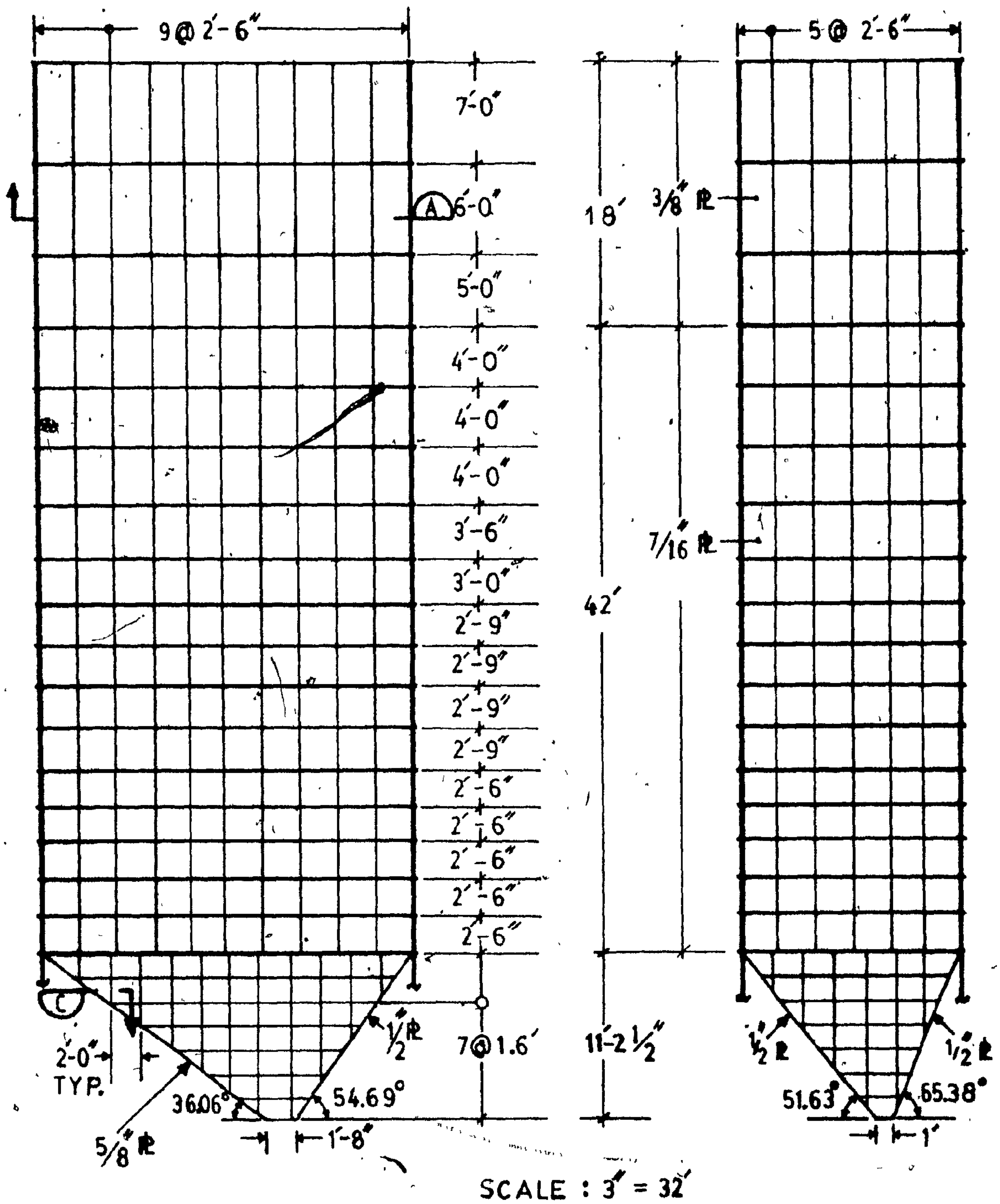

FIG. A.1 Conventional stiffening system - elevations

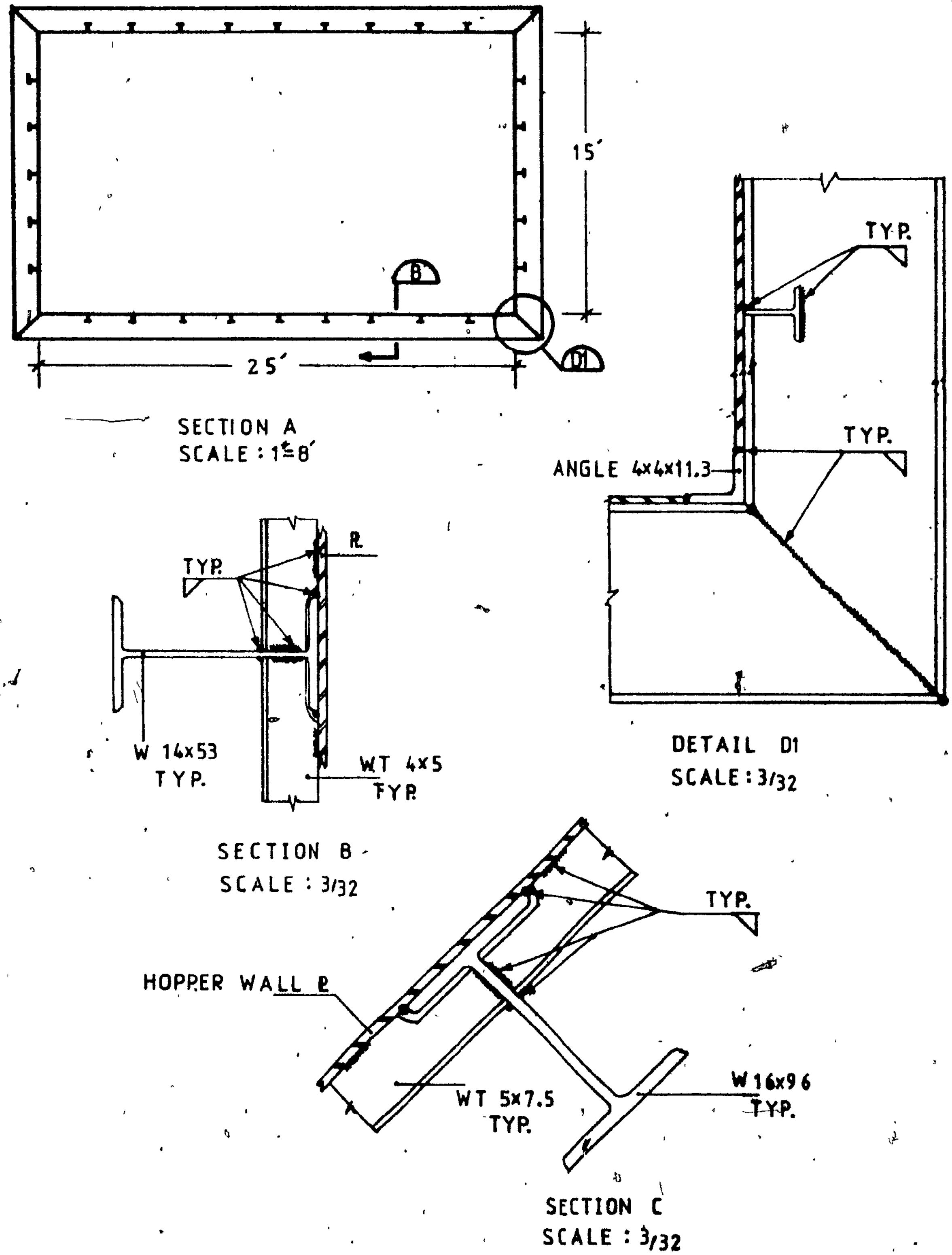

FIG. A.2 Cross-sections and details (A.1)

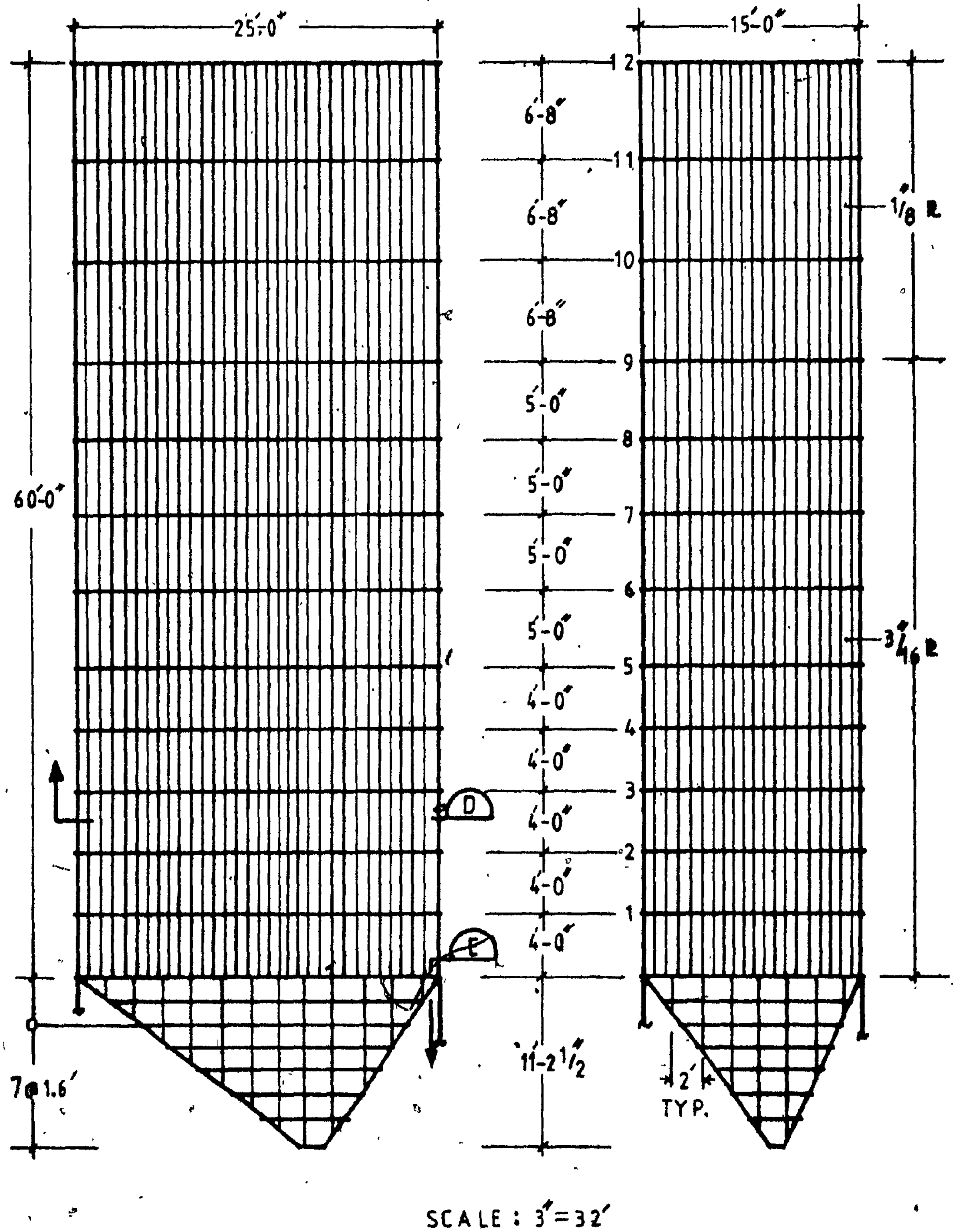

FIG. A.3 Orthotropic system - elevations

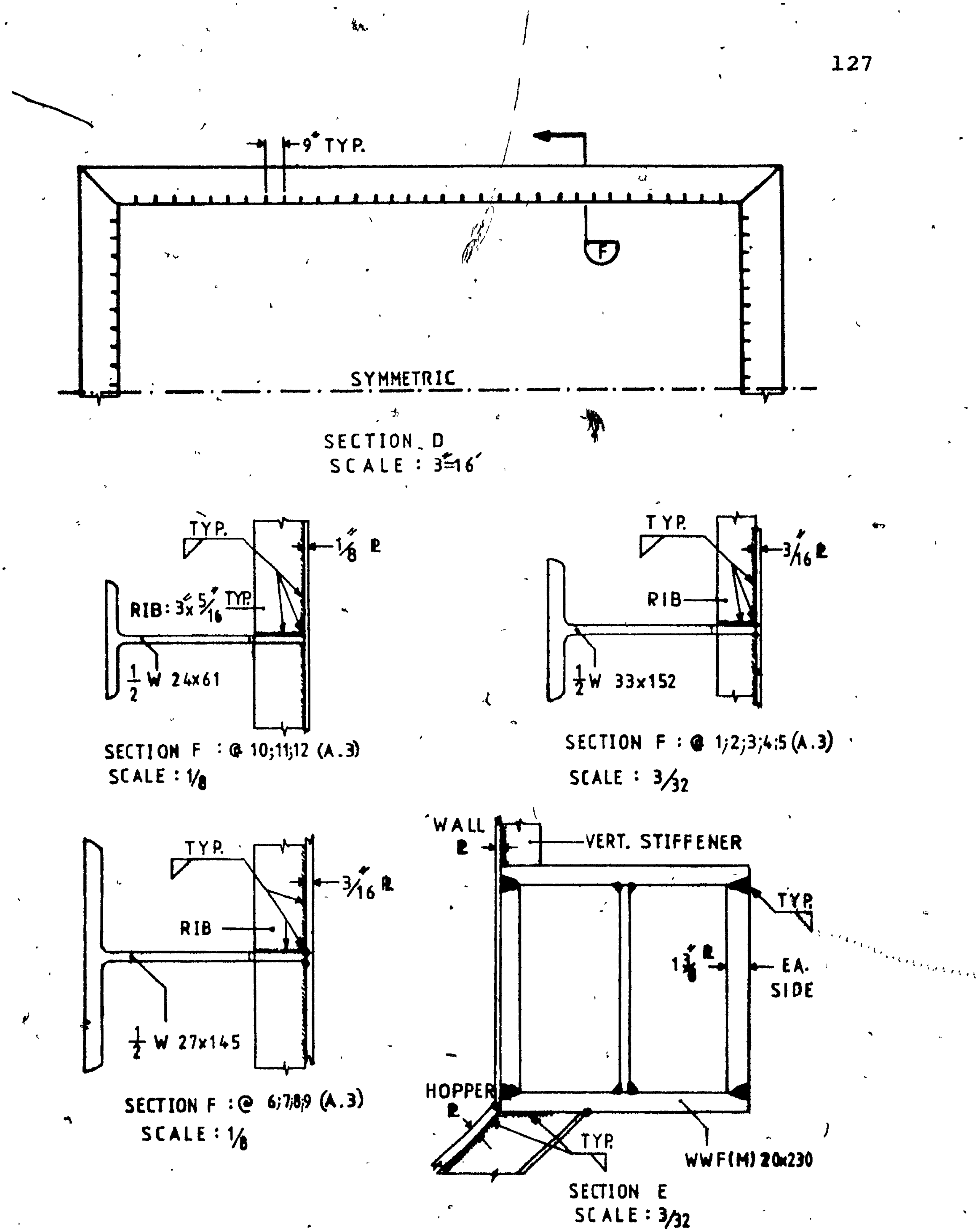

FIG. A.4 Cross-sections and details (A.3)