STRESS ANALYSIS
OF
STEEL COAL TOWER AND BUNKER

Thesis for the Degree of B. S.

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Leonard A. Robert

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STRESS ANALYSIS

OF

STEEL COAL TOWER AND BUNKER

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FOREWORD

The coal tower and bunker are to be constructed by the Detroit Edison Company at their Marysville plant and are to occupy a portion of a building known as the crusher house.

The Detroit Edison Company supplied the attached construction plans, and it is the purpose of the thesis to analyse the stresses in this structure and to check their design.

The library was searched but no direct information was available on the design or construction of such a structure; therefore Ketchum's text on Walls, Bins and Grain Elevators was used. In this text Ketchum explains the design of tall circular grain elevators as developed by Janssen, Airy, the Germans and others, and states that the above named are the most used and that they check actual values very closely. The connection between the stresses in a structure designed to store grain and those of a structure designed to store coal can best be explained by referring to Ketchum's text, page 308. It is quite evident that the development of the equations for pressures to be used is a purely static analysis of a granular mass confined within a circular container, and in the final analysis will not differ for any granular mass except in that of the values for unit weight, angle of repose, angle of friction and value of k; these can only be obtained for any one granular substance under given conditions by experiment.

The value of k is approximately equal to $\frac{1 - \sin \phi}{1 + \sin \phi}$ and is actually equal to L/V; where L is the lateral pressure and V is the vertical pressure. On page 214 of Ketchum's text are shown the results of experiments on coal by the Portland Cement Association and by the Link Belt Company in which they have determined bituminous coal to weigh 50 pounds per cubic foot, the angle of repose to be 30 to 45 degrees depending on conditions, corresponding values of k to equal .33 and .17 and the angle of friction to equal 18 degrees.

The smaller angle of repose gives the greater lateral pressure, and the greater angle of repose gives the greater vertical pressure. It is therefore advisable to design the tower and hopper for maximum conditions affecting their design. The tower is designed for a coal with angle of repose of 30 degrees and the hopper for a coal with angle of repose of 45 degrees.

In the analysis the worst condition derived from the methods of Janssen, Airy and the Germans is used.

CONTENTS

NOMENCLATURE

The following nomenclature will be used.

L	lateral pressure
R	hydraulic radius
u'	angle of friction
k	L/V (a constant for any one granular material under given conditions)
h	height
w	weight
e	2.718
N	normal pressure
P	pressure of coal in vertical direction supported by shell
ϕ'	angle of friction = u'
y	height = h
V	vertical pressure
F	frictional force
R_h	horizontal reaction
R_v	vertical reaction

PART I

ANALYSIS OF TOWER SHELL

Janssen's:

$$L = \frac{wR}{u'}\left[1 - e^{\frac{-ku'h}{R}}\right]$$

$$L = \frac{50 \text{ x } 9.5}{.31}\left[1 - e^{\frac{-.33 \text{ x } .31 \text{ x } 81.5}{9.5}}\right]$$

$L = 892\#$

$L = kV \qquad V = \frac{892}{.33} = 2,700\#/\text{sq. ft.}$

$P = \frac{Nu'h}{2} = \left(\frac{892 \text{ x } .305 \text{ x } 81.5}{2}\right) = 11,100\#/\text{lin.ft.}$

German Practice:

$$P = wR\left[y - \frac{R}{ku'}\left(1 - e^{\frac{-ku'y}{R}}\right)\right]$$

$$P = 50 \text{ x } 9.5\left[81.5 - \frac{9.5}{.33 \text{ x } .305}\left(1 - \frac{1}{e^{\frac{.33 \text{ x } .305 \text{ x } 81.5}{9.5}}}\right)\right]$$

$P = 12,650\#/\text{lin. ft.}$

$L = 905\#/\text{sq. ft.}$

$V = 2,710\#/\text{sq. ft.}$

$P = 12,650\#/\text{lin. ft.}$

Airy's:

$$L = \frac{wd}{u + u'} \text{ x } \frac{1 - 2\sqrt{1 + u^2}}{2\sqrt{\frac{2h}{d}(u + u') + 1 - uu'}}$$

$L = kV$

$$L=\frac{50\times 38}{.7+.3}\left[\frac{1-\sqrt[2]{1+(.7)^2}}{\sqrt[2]{\frac{2\times 81.5}{38}(.7+.3)+1-(.7\times .3)}}\right]$$

$L = 872$#/sq. ft.

$V=\frac{872}{.33}=2{,}640$

$P=\frac{872\times .305\times 81.5}{2}=11{,}000$#/lin. ft.

TABLE I

Summary of Three Methods

	L/sq.ft.	V/sq.ft.	P/lin.ft.	Total P
Janssen's	892	2,691	12,420	1,492,000
German	905	2,710	12,650	1,505.000
Airy's	872	2,640	11,000	1,310,000

In analysis use L = 905#   V = 2,710#   P = 12,650#

When a hopper is emptied from the side, the lateral pressure on the opposite side will increase as much as four times. (Ketchum, page 351)

These hoppers do not empty from the side, but they do empty from a point considerably off center; in order to compensate for this a proportion of actual offset to radial distance was used as a factor by which to multiply four and obtain a reasonable factor of increase for this problem.

$F=\frac{14.5}{19}\times 4=3.05$

$$s = \frac{19 \times 12 \times 905 \times 3}{144} = 4,300\#/\text{sq. in.}$$

$$t = \frac{4,300}{12,000} = .358 \qquad \text{Use } .375$$

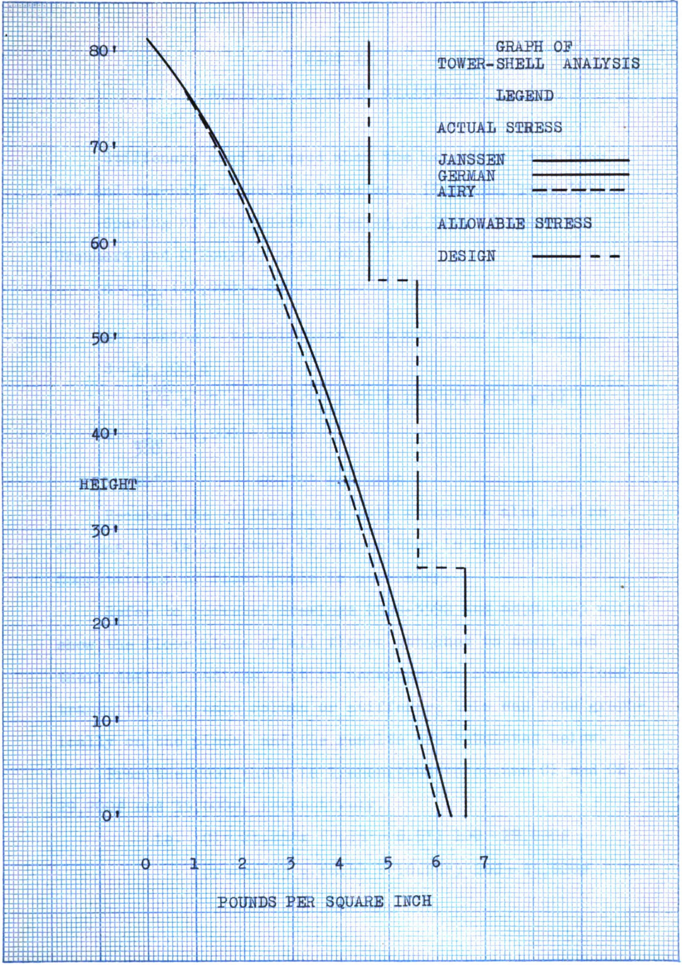
.375 was used

In order to show the allowable stress for material used and the actual stress imposed upon it, for the full height of the tower, a Table and Graph are included on the following pages.

TABLE II

TABLE OF PRESSURES ACTING WITHIN THE TOWER AND ON THE SHELL

Depth	Pressure on coal	Pressure Normal to Shell	Weight of coal supported by shell, and wt. of shell
10	396	132	1,770
20	966	322	8,620
30	1,314	438	17,600
40	1,620	540	28,900
50	1,926	642	43,000
60	2,190	730	58,500
70	2,409	803	75,000
80	2,631	877	94,000
81.5	2,710	905	94,400


GRAPH OF
TOWER-SHELL ANALYSIS
LEGEND
ACTUAL STRESS
JANSSEN
GERMAN
AIRY
ALLOWABLE STRESS
DESIGN
80'
70'
60'
50'
40'
HEIGHT
30'
20'
10'
0'
0
1
2
3
4
5
6
7
POUNDS PER SQUARE INCH

PART II

ANALYSIS OF TOWER STIFFENERS

Stiffeners must be used when the height is less than two and one-half times the diameter. (Ketchum, page 359)

Spacing of stiffeners. (Design of Steel Structures, Urquhart and O'Rourke, page 178)

$$d = \frac{t}{40} (12{,}000 - S)$$

d = spacing   t = thickness   S = web shear #/sq. in.

$$S = \frac{\frac{94{,}000\ d}{12}}{3/8 \times 27 \times 12 + 5/16 \times 30 \times 12 + 1/4 \times 24 \times 12} = 26d$$

$$d = \frac{3}{320} (12{,}000 - 26d)$$

d = 90"   89.75" was used

Because the stiffeners in this structure also act as columns, it is necessary to design them in combination. Computations of floor loads is necessary.

Refer to pocketed drawings 6S58-3592 and 6S58-3593 which show the floor plans of all floors attached to tower and their design loadings. It is necessary to compute the load transmitted to each column or stiffener; this was done graphically on the plans, and the results are tabulated below.

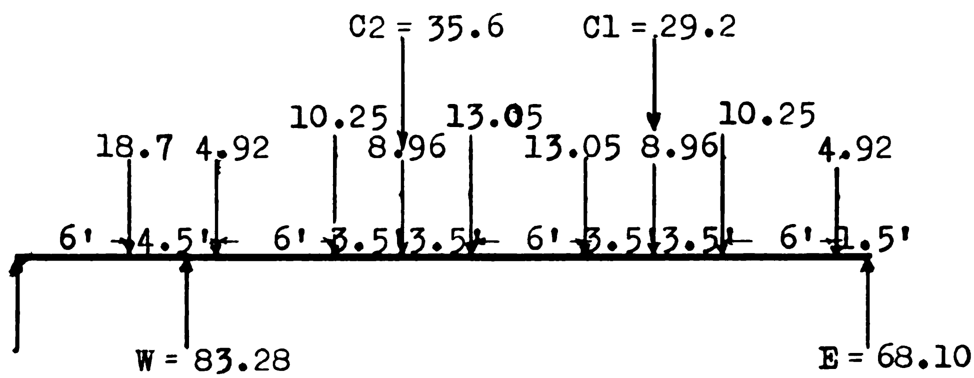
From the roof there is transmitted to column C1 and C2 29,200# and 35,600#, respectively.

ie. C1 = 21.25' x (7.25' + 6.5') 100 = 29,000#

C2 = 21.25 x (6.5' + 10.25') 100 = 35,600#

This in turn is transmitted together with a portion of the bunker floor loading to the (E) and (W) stiffeners.

ie. Loads transmitted to the (E) and (W) stiffeners at bunker floor, elevation 693'. (see Drawing 6S58-3592)



$$R_W = \frac{6}{9} \times 18.7 + 13.05 + 8.96 + 10.25 + 4.92 + \frac{24}{35} \times 35.6 + \frac{11}{35} \times 29.2 = 83.28 \text{ kips}$$

$$R_e = 13.05 + 8.96 + 10.25 + 4.92 + \frac{11}{35} \times 35.6 + \frac{24}{35} \times 29.2 = 68.10 \text{ kips}$$

All the loads shown are transmitted by the bunker floor except loads of 29.2 and 35.6 which are transmitted as shown by column C1 and C2.

There is also transmitted to each stiffener the weight of coal supported by 7.47' of shell, the weight of the shell and the weight of the stiffener.

ie. wt. of coal supported by shell per stiffener		94,000#
wt. of shell		7,470#
wt. of 10 WF 29	81.5' long	2,360#
	Total	104,230#
additional wt. of East and West stiffeners		896#
	Total	105,126#

The following table shows the floor loads brought into the columns at their respective elevations and the weight of the coal and material supported by the stiffener.

The tower is so constructed that the analysis of any quarter is an analysis of the complete structure. The table therefore shows only the West stiffener (W); the East stiffener (E); then, numbering from (E) counterclockwise, (A), (B), (C) and (N).

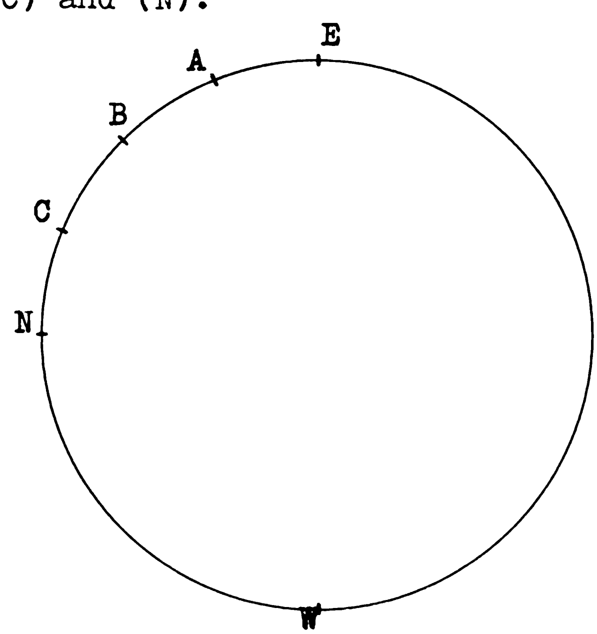


TABLE III

TABLE OF COLUMN LOADING

Column Loads in kips

Elevation	E	A	B	C	D	W
693	68.38	8.27	5.09	12.75	4.91	83.28
671'2"	---	1.7	6.7	3.0	2.8	17.8
656'9"	---	1.7	6.7	3.0	2.8	17.8
641'3"	---	1.7	6.7	3.0	2.8	17.8
624'3"	---	1.9	7.6	3.4	3.2	20.1
614'6"	---	1.7	6.7	3.0	2.8	17.8
	105.1	104.2	104.2	104.2	104.2	105.1
Total	173.31	120.84	143.36	132.35	123.51	279.68

The question arises as to why each section of stiffener is not studied and incremental relations of actual stress and allowable stress analyzed. The structure carries the same size stiffener from top to bottom, and therefore in checking the design it is not necessary to do so. If the intent is to analyze the structure in such a manner, remember that the pressures do not vary as the height but as a curve shown on graph on Plate I.

In analyzing the stress in each stiffener a number of conditions must be noted.

1. The shell is welded continuously to the stiffener on either edge of the stiffener flange.
2. The shell is therefore a curved cover plate, and a

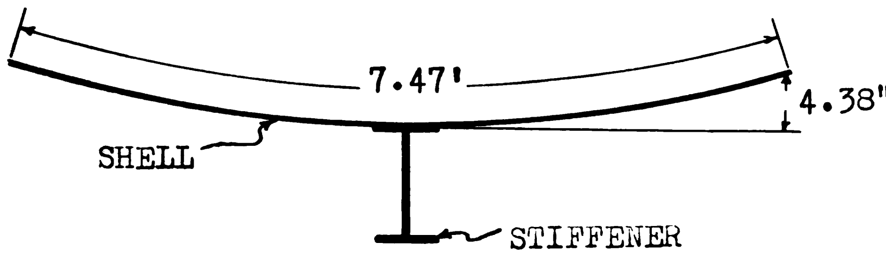
portion of it can be used as a supporting member.

3. The coal load and shell load are transmitted through the shell and therefore will act as an eccentric load, acting through the centroid of the curved section supported.
4. The floor loads are not in all cases brought into the stiffener but are transmitted to it as eccentric loadings and may act on one or more faces of the stiffener.

The question now arises as to how much of the shell can be used as a cover plate for the stiffener. The A. I. S. C. does not give any allowable values for this condition; so it is necessary to reason out an allowable value. For columns built up of two channels and a connecting cover plate the distance between the gauge lines must not exceed 40 times the thickness of the connecting web. The outstanding leg of flanges in compression must not be more than 12 times the thickness of the material. It is quite evident that the conditions in this problem are not as favorable as the former, but are more favorable than the latter. The former would allow a cover plate of 20", the latter a cover plate of 15"; therefore a cover plate of 18" is used.

Analysis of the 10-WF-29 stiffeners.

$A = 8.53$ sq. in. $\quad I = 157.3$ in.^{4} $\quad$ depth $= 10.22"$



$Z = 19 \times 12 - 19 \times 12 \times \cos 11^\circ 15' = 4.38''$

e_1 = distance from face to center of loading

$$e_1 = \frac{R \sin 11^\circ 15'}{11^\circ 15'}$$

R = radius

$11^\circ 15'$ is in radians

$$e_1 = \frac{19 \times 12 \times .19509}{.1963}$$

$e_1 = 1.416''$

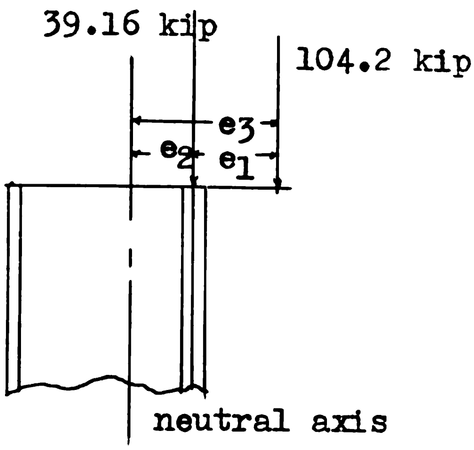
1 - Determine the location of the neutral axis.

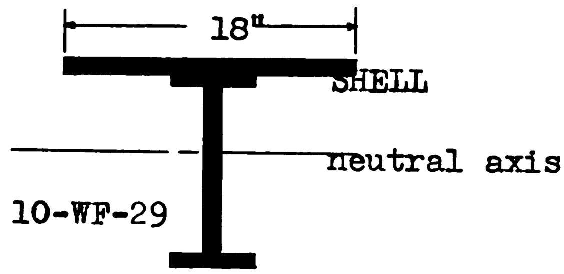
Area of cover plate $3/8 \times 18 = 6.75$ sq. in.

$$\bar{y} = \frac{5.11 \times 8.53 + 10.41 \times 6.75}{15.28} = 7.45''$$

$$I = 157.3 + 8.53 \times (2.34)^2 + .079 + 6.75 \times (2.96)^2$$
$$= 263.1''^4$$

ie. Stiffener A





$e_1 = 1.416"$

$e_2 = 10.22 - 7.45 = 2.77"$

$e_3 = 2.77 + 1.416 = 4.19"$

$$s = \frac{P}{A} \pm \frac{mc}{I}$$

$$s_m = \frac{143.36}{15.28} + \frac{(39.16 \times 2.77 + 104.2 \times 4.19)3.14}{263.1}$$

$= 15{,}920\#/\text{sq. in.}$

$$s = \underline{143.36} - \underline{(39.16 \times 2.77 + 104.2 \times 4.19)\ 7.45}$$

$= -\ 6{,}000\#/\text{sq. in.}$

$$r = \sqrt[2]{\frac{263.1}{15.92}} = 4.1$$

$$\frac{l}{r} = \frac{16 \times 12}{4.1} = 48$$

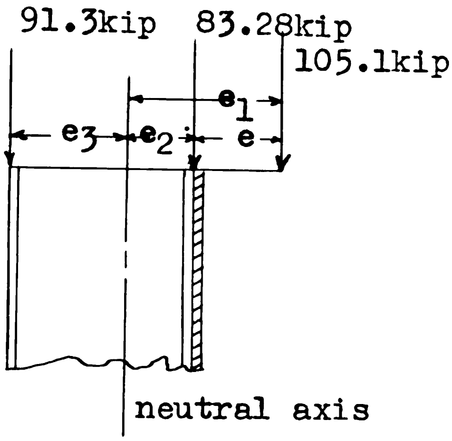
Allowable stress $17{,}000 - \dfrac{.485 \times 192 \times 192}{4.1 \times 4.1}$

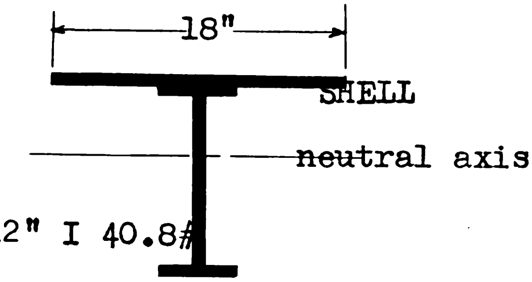
$= 16{,}000\#/\text{sq. in}$

Column B, C, and N are similarly loaded, but with lighter loads; therefore they are of sufficient size.

Check column W 12" I=40.8# A = 11.84 sq. in.

depth = 12.00" width = $5\frac{1}{4}$" I = 268.9"^{4}





$$\bar{y} = \frac{6.75 \times 12.19 + 11.84 \times 6}{18.59} = 8.26"$$

$$I = 268.9 + (2.26)^2 \times 11.84 + .079 + 6.75 \times (3.93)^2$$

$$I = 433.8"^4$$

$e_1 = 12 - 8.26 = 3.74"$ $e_2 = 3.74 + 1.41 = 5.55"$

$e_3 = 8.26"$

$$s = \frac{279.7}{18.59} + \frac{(83.28 \times 3.74 + 105.1 \times 5.15 - 91.3 \times 8.26)4.11}{433.8}$$

= 15,966#/sq. in.

$$s = 279.7 - \frac{(83.28 \times 3.74 + 105.1 \times 5.15 - 91.3 \times 8.26)8.26}{433.8}$$

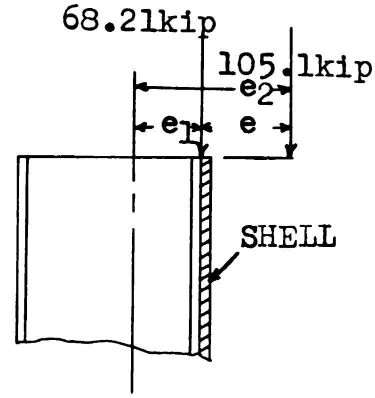
= 13.210#/sq. in.

$$r = \sqrt[2]{\frac{433.8}{18.59}} = 4.84$$

$$\frac{l}{r} = \frac{16 \times 12}{4.84} = 40.6$$

Allowable stress 17,000 - .485 x $(40.6)^2$ = 16,200#/sq. in.

Column E same as column W but loaded differently.



$e_1 = 12 - 8.26 = 3.74''$ $e_2 = 3.74 + 1.41 = 5.15''$

$$s = \frac{173.3}{18.59} + \frac{(68.21 \times 3.74 + 105.1 \times 5.15)\ 4.11}{433.8}$$

$$= 16{,}860\#/\text{sq. in.}$$

$$s = \frac{173.31}{18.59} - \frac{(68.21 \times 3.74 + 105.1 \times 5.15)8.26}{433.8}$$

$$= -5{,}830\#/\text{sq.in.}$$

A stress of 16,860#/sq. in. is allowable due to the fact that the load on the column is accumulative, and therefore the actual stress (as noted in loading table II) decreases with each foot of height and will not exceed the maximum allowable of 17,000#/sq. in.

PART III

ANALYSIS OF HOPPER PLATES

In the analysis of pressures acting on the plates of a hopper, the ellipsoidal method is normally used, but because this hopper is of such odd shape a purely static analysis is advisable. This will give the same maximum but will not give the gradual decrease to the top of the hopper. The following is an analysis on that basis.

As explained in the foreword the following pressures apply to the hopper.

$$V = \frac{50 \times 9.15}{.17 \times .31}\left[1 - \frac{1}{e^{\frac{.17 \times .31 \times 81.5}{9.5}}}\right] = 3,320\#/\text{sq. ft.}$$

V (base) = 3320 + 12 x 50 = 3,920#sq. ft.

V (analysis) = 3,920 + 392 = 4,312#/sq. ft.

The value of 10% added to actual pressure is recommended to take care of added pressure caused by opening and closing of hopper outlet.

In the analysis of the plate supports the vertical pressure is 3,320# at the top and 3,920# at the bottom. An average of these, plus 10%, will give 3,982#. The average value was used as a uniform pressure on the supporting members. The 10% is an imperical value, so that the results obtained by finer analysis would be a waste of time. A value of 4,000#/sq. ft. is used.

In the design of the hopper parts it is common practice to allow for a factor of safety of 4; this value is to take care of usage. If the hopper will not be subjected to maximum wear a smaller factor may be used. This value is taken care of in allowable stress and is entirely up to the designer. An allowable stress value of 13,700#/ sq. in. is recommended by most authors.

ie. Analysis of Plate 2. (see Index of Plates in pocket)

(see prints 6S58-3590 and 6S58-3591 for dimensions)

For a horizontal projection area of 1 sq. ft. the vertical projection area = 5.98 sq. ft. The actual area = 6.17 sq. ft.

The plate makes an angle of 9 degrees and 18' with the vertical. $\sin \phi = .162 \quad \cos \phi = .97$

$V = 4{,}312\#/\text{sq. ft.}$

$L = 1{,}420\#/\text{sq. ft.}$

$N = (V \sin^2 \phi + L \cos^2 \phi)\ \#/\text{sq. ft. of plate}$

$N = 4{,}312 \times .162 \times .162 + 1{,}420 \times .97 \times .97 = 1{,}445\#/\text{sq.ft.}$

Maximum unsupported length = 16" (see plate 6S58-3590)

$s = \frac{mc}{I} \quad m = \frac{wl^2}{8} \quad c = 5/16" \quad I = 1/12 \times 1 \times (5/8)^3 = .0204"^4$

$$s = \frac{10.15 \times 16 \times 16 \times 5}{8 \times 16 \times .0204} = 5{,}000\#/\text{sq. in.}$$

TABLE III

TABLE OF PLATE RESISTANCE

Plate	Area	ø	Cos ø	Sin ø	L	V	N
1	21	45-00	.707	.707	1200	3652	2500
2	11.5	9-18	.97	.162	1420	4312	1445
3	2.6	9-18	.97	.162	1420	4312	1445
4	18.6	45-00	.707	.707	1200	3652	2500
5	24.7	18-10	.95	.312	1420	4312	1700
6	53.5	23-40	.915	.401	1420	4312	1860
7	31.4	23-40	.915	.401	1420	4312	1860
8	53.5	23-40	.915	.401	1420	4312	1860
9	36.7	37-45	.790	.613	1420	4312	2500
10	20.8	23-40	.915	.401	1420	4312	1800
11	7.0	37-45	.790	.613	1200	3652	2110

Area per sq. ft. of horizontal projection Vertical	Actual	Maximum span inch	Stress kips
1	1.41	16	8.0
5.98	6.17	16	5.0
5.98	6.17	16	5.0
1	1.41	21	14.6
3.04	3.21	21	10.2
2.29	2.50	19	9.1
2.29	2.50	18	8.2
2.29	2.50	18	8.2
1.19	1.56	18	11.3
2.29	2.50	18	8.2
1.19	1.56	18	12.1

PART IV

ANALYSIS OF THE PLATE SUPPORTS

It is first necessary to determine the coal pressure and plate resistance balance. To do this it is necessary to compute the resistance of each plate to a maximum loading of moving coal. The vertical pressures and the plate resistances should balance in order that too great a pressure is not transmitted to the hopper outlet. Too great a pressure at the hopper outlet would cause arching and binding.

ie. Computation of the vertical resistance of plate 2 (see index of plates in pocket) and tabulation of results for the other plates.

Plate 2 Average vertical $= 4,000\#$ Horizontal $= 1,320\#$

$N =$ (normal) $V = \sin \phi$ $L = \cos \phi$ $\phi =$ angle plate makes vertical. $F =$ friction force

$N = 4,000 \times .162 \times .162 + 1,320 \times .97 \times .97 = 1,344\#/\text{sq. ft. of plate}$

$F = .325 \times 1,344 = 437\#/\text{sq. ft.}$

$R_v =$ vertical resistance per sq. ft. of plate.

$R_v = N \sin \phi + F \cos \phi$

In plate 2 the plate makes an angle of 9 degrees and 18 minutes with the vertical and at its center makes an angle of 9 degrees and 18 minutes with the outlet, but at its extremities the angle of discharge with the vertical is increased to 25 degrees on south and 13 degrees and 30 minutes on the north. (See 6S58-3580) Therefore, the vertical resistance of the plate varies from:

$1,344 \times .162 + 437 \times .97 = 640$ at center

$1,344 \times .423 + 437 \times .573 = 820$ at south end

$1,344 \times .234 + 437 \times .957 = 730$ at north end

Total resistance of plate in vertical direction = $R_v \times A$, where A is the area of the plate.

R_v of the central section $= \frac{(1.67 \times 2.08)640}{1.62} = 14,200\#$

R_v of south section $= \frac{1}{2}(2.08 \times 5.67 \times \frac{(820 + 640)}{2} = 27,500\#$

R_v of north end $= \frac{1}{2}(13.3 \times 2.08)\ \frac{640 + 730}{2} = 9,500\#$

Total vertical resistance of the plate = 51,200#

The results of complete hopper analysis of plates are shown in tabulation below and on drawing plate in pocket.

TABLE IV

TABULATION OF VERTICAL RESISTANCE OF HOPPER PLATE ANALYSIS

Plate	Normal	Friction	Angle at center	Angle at left	Angle at right
1	2,425	785	45 - 00	----	----
2	1,285	416	9 - 18	25 - 00	13 - 30
3	1,285	416	9 - 18	13 - 30	----
4	1,285	785	45 - 00	----	----
5	1,580	513	18 - 10	----	41 - 40
6	1,740	565	23 - 40	41 - 40	31 - 45
7	1,740	565	23 - 40	31 - 45	25 - 00
8	1,740	750	23 - 40	31 - 45	41 - 40
9	2,323	750	37 - 45	41 - 40	----
10	1,740	565	23 - 40	----	31 - 45
11	2,110	685	37 - 45	----	----

TABLE IV (CONT.)

Plate	Ctr. sin.	Angle cos.	Lft. sin.	Angle cos.	Rt. sin.	Angle cos.
1	.707	.707	-	-	-	-
2	.162	.970	.422	.905	.234	.955
3	.162	.970	.234	.955	-	-
4	.707	.707	-	-	-	-
5	.312	.950	-	-	.665	.747
6	.401	.915	.665	.747	.525	.85
7	.401	.915	.525	.850	.422	.905
8	.401	.915	.525	.850	.665	.747
9	.613	.790	.665	.747	-	-
10	.401	.915	-	-	.525	.850
11	.613	.790	-	-	-	-

Resistance Per Unit			Total for
Center	Left	Right	Plate
2,265	--	---	67,200
611	916	696	86,000
611	696	---	10,300
2,265	--	---	59,700
1,240	--	1,635	114,000
1,214	1,580	1,393	179,800
1,214	1,393	1,244	99,300
1,214	1,393	1,580	179,800
2,012	2,100	---	122,000
1,214	--	1,393	66,750
1,830	--	---	21,000
Hopper outlets 1.67 x 1.67 x 4,000 x 1.5........................			16,700
		Total	1,022,850

Surcharge $= \frac{1135}{4} \times 3320 = 942{,}000$

Wt. of coal in hopper:

End hopper $[\frac{1}{2}(9.417 \quad 17.25)9.5 \quad (1.67 \times 1.67)\frac{1}{2}]12.25$

845 cu. ft.

Center hopper $\frac{1}{4}[17.25 \times 13 \quad 1.67 \times 1.67]12.92$

735 cu. ft.

wt. (735 845)50 79,000#

Total pressure 942,000 79,000 1,021,000#

This proves to be a good balance for the ¼ section as a whole, but looking at the problem as per hopper:

Area of corner hopper surcharged is 160 sq. ft.

Pressure 160 x 3320 845 x 50 589,250#

Resistance 612,200#

There is an excess of resistance in the corner hopper; therefore it will operate satisfactorily except for a pocketing of coal under certain conditions between plates 5 and 6.

The balance of the ¼ leaves an excess of pressure in the center hopper of some 28,000#. Whether or not this would be sufficient to cause arching at the outlet and therefore stoppage of flow, is something of which there is no definite proof. It seems however that there is a possibility of such a thing happening. Rivets will be placed in these plates to increase the coefficient of friction, thereby decreasing the pressure at center.

PART V

STRESS ANALYSIS OF THE SUPPORTING MEMBERS

The supporting members are stressed by a load acting normal to them and by an axial load transmitted to them by the member to which they connect to form the trough for the support of the plates. Each member carries a portion of the vertical and horizontal reaction, at point of connection, proportional to its angle of inclination to the vertical or horizontal.

ie. Analysis of member #6. (see drawing plate in pocket) The vertical reaction at the hopper outlet is taken by one-half the members at 9 degrees 18 minutes and one-half at 23 degrees 40 minutes.

Where the load is carried by V_1 at 9 degrees 18 minutes and V_2 at 23 degrees 40 minutes:

$V_1 + V_2 = 100\%$ of load

To find the percentage of load taken by each equate $V_1 \sin \phi$ to $V_2 \sin \phi$

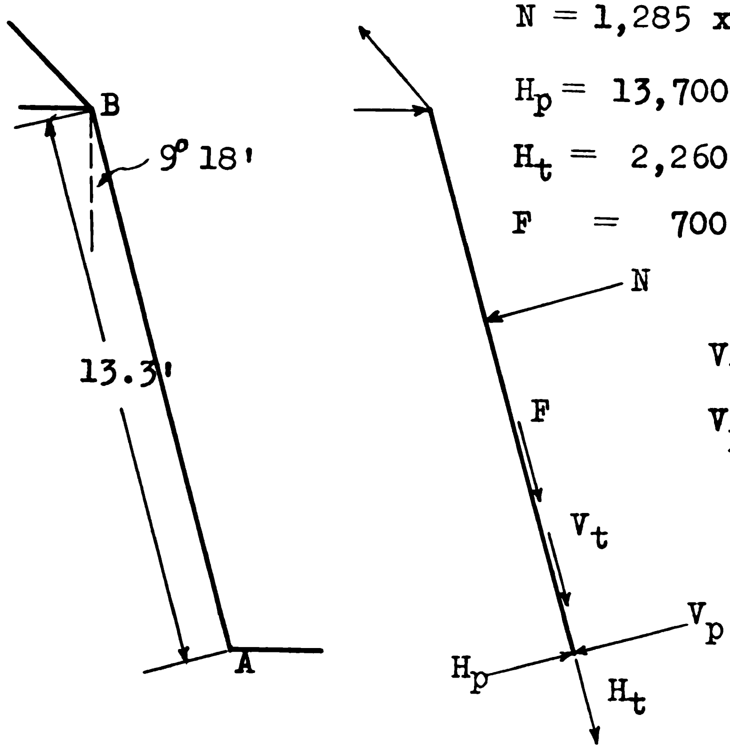
Then $.162V_1 = .401V_2$

$$V_1 = 2.47V_2$$

$$2.47\ V_2 + V_2 = 100\%$$

$$V_2 = 28.8\% \qquad V_1 = 71.2\%$$

The total vertical reaction at hopper outlet is 95.5 kips. 71.2% is carried by $\frac{1}{2}$ the members and equals 68 kips. The portion allotted to #6 is $68/4 = 17$ kips. The horizontal is taken care of by an angle bar running from hopper to column. For member #6 as shown by table VII is 14 kips.



$N = 1,285 \times 22.45 = 2,170$#/lin.ft.

$H_p = 13,700$#

$H_t = 2,260$#

$F = 700$#/lin.ft.

$V_t = 16,700$#

$V_p = 2,760$#

Member #6 is supported as shown, fixed to hopper base plate at A and to a supporting member at B, acted upon by a uniform load N normal to the axis, a vertical load V at the outer fibre, making an angle of 9 degrees and 18 minutes with the axis, and a frictional force F applied along the outer fibre through the hopper plates.

Member #6 is a 12 WF 53 section $S' = 70.7$ $A = 12.06$

Stress $= \frac{Mc}{I} = \frac{M}{S'}$ Where M is maximum, stress is maximum

Maximum Bending Moment will occur at B and is equal to:

$$\frac{22.45 \times 1,285 \times 13.3 \times 12}{2} + \frac{416 \times 22.45 \times 12}{2} +$$

$$\frac{17{,}000 \times .97 \times 12}{2} +$$

$$14{,}000 \times .162 \times 12 \times \tfrac{1}{2} + 17{,}000 \times .162 \times 13.3 \times 12 -$$

$$14{,}000 \times .97 \times 13.3 \times 12 = 740{,}500 \text{ in. lb.}$$

$$S = \frac{18{,}758.00}{12.06} + \frac{740{,}500}{70.7} = 12{,}000\#/\text{sq. in.}$$

The truth of this is best represented by the graph on the following page.

MOMENT DIAGRAM
MEMBER # 6

SCALE $\frac{1}{2}$" 1'

USE WITH DIAGRAM ON
PREVIOUS PAGE

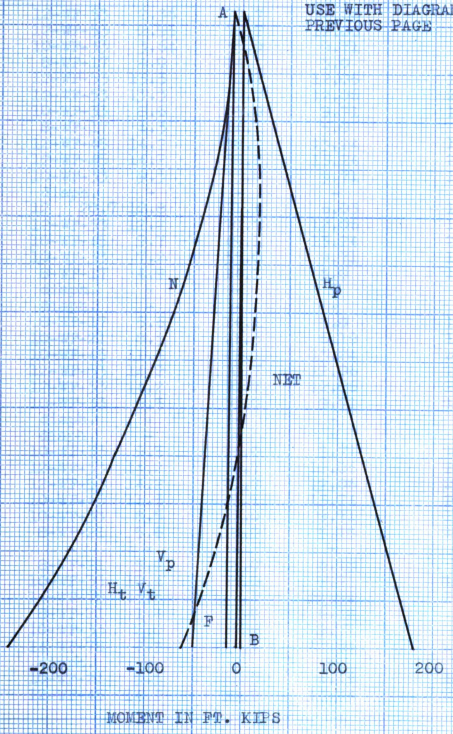


TABLE V

INDEX OF MEMBERS SUPPORTING HOPPER PLATES
(use with plate I in pocket)

MEMBER	SIZE	LENGTH (feet)	SECTION MODULUS	AREA OF SECTION (sq. in.)	AREA OF PLATE SUPPORTED
1	12-WF-53	2.36	70.7	12.06	4.12
2	12-WF-53	2.95	70.7	12.06	5.5
3	10-WF-33	3.64	35.0	9.71	7.18
4	8-WF-17	4.0	14.1	5.0	7.48
5	12-WF-53	13.3	70.7	12.06	15.5
6	12-WF-53	13.3	70.7	12.06	22.45
7	10-WF-33	8.6	35.0	9.71	17.15
8	8-WF-17	4.3	14.1	5.0	7.73
9	12-WF-45	14.2	58.2	13.24	17.85
10	12-WF-45	14.2	58.2	13.24	24.90
11	10-WF-33	9.5	35.0	9.71	18.30
12	diaph.	-	-	-	8.37
13	12 20.7	12.8	21.4	6.03	10.9
14	12-WF-53	12.2	70.7	12.06	10.90
15	10-WF-45	9.7	49.1	13.24	20.50
16	10-WF-33	7.2	35.0	9.71	17.4
17	8-WF-17	4.9	14.1	5.0	11.8
18	diaph.	-	-	-	6.23
19	diaph.	-	-	-	5.72
20	8-WF-17	5.6	14.1	5.0	11.35
21	10-WF-21	8.5	21.5	6.19	17.0
22	10-WF-45	11.3	49.1	13.24	20.1
23	12-WF-53	14.2	70.7	12.06	26.30
24	12-WF-53	14.2	70.7	12.06	24.90
25	10-WF-33	9.5	35.0	9.71	18.30
26	diaph.	-	-	-	8.37
27	diaph.	-	-	-	8.37
28	10-WF-33	9.5	35.0	9.71	18.30
29	12-WF-53	14.2	70.7	12.06	24.90
30	12-WF-53	14.2	70.7	12.06	26.30
31	10-WF-45	11.3	49.1	13.24	20.1
32	10-WF-33	8.5	35.0	9.71	17.0
33	8-WF-21	5.6	18.0	6.18	11.35
34	-	-	-	-	5.72
35	8-WF-21	5.2	18.0	6.18	9.82
36	10-WF-33	10.4	35.0	9.71	19.65
37	14-WF-78	15.6	121.1	22.94	26.5
38	8-WF-21	1.4	18.0	6.18	2.37
39	10-WF-33	2.0	35.0	9.71	3.19
40	14-WF-78	2.3	121.1	22.94	
41	diaph.	-	-	-	8.37
42	10-WF-21	9.5	21.5	6.19	18.3
43	12-WF-45	14.2	58.2	13.24	24.9

TABLE VI

TABLE OF VERTICAL REACTIONS
(use with plate I in pocket)

MEMBER	UNIT R_V	TOTAL R_V (kips)	PLATE WEIGHT (kips)	MEMBER WEIGHT (kips)	GROSS R_V (kips)	$R_V/2$ (kips)	PLATE AREA SUPPORTED (sq.ft.)
1	2265	9.0	.1	.1	9.2	4.6	4.1
2	2265	12.5	.1	.2	12.8	6.4	5.5
3	2265	16.3	.2	.1	16.6	8.3	7.2
4	2265	17.0	.2	.1	17.3	8.6	7.5
J	2265	7.9	-	-	7.9	4.0	3.5
L_2	2265	3.9	-	-	3.9	2.0	1.9
5	630	9.7	.3	.7	10.7	5.3	15.5
6	640	14.4	.5	.7	15.6	7.8	22.45
7	790	13.5	.4	.3	14.2	7.1	17.15
8	880	6.8	.2	.1	7.1	3.5	7.73
I	910	.7	-	-	.7	.4	.82
K_2	640	9.8	-	-	9.8	4.9	15.35
K	1240	8.8	.2	-	9.0	4.5	7.1
9	1220	22.1	.4	.7	23.2	11.6	18.1
10	1230	30.8	.6	.7	32.1	16.0	25.0
11	1300	24.2	.4	.3	24.9	12.5	18.6
12	1350	11.9	.2	-	12.1	6.1	8.8
P	1380	1.1	-	-	1.1	.5	.8
13	1300	14.3	-	.7	15.0	7.5	11.0
14	1260	13.9	.2	.6	14.7	7.3	11.0
15	1390	28.5	.5	.4	29.4	14.7	20.5
16	1480	25.9	.4	.2	26.5	13.3	17.5
17	1540	18.5	.2	.1	18.8	9.4	12.0
18	1590	9.9	.1	-	10.0	5.0	6.2
I_2	1620	2.4	.4	-	2.8	1.4	1.5
19	1550	8.8	.1	-	8.9	4.5	5.7
20	1500	18.7	.2	.1	19.0	9.5	11.4
21	1420	24.2	.4	.2	24.8	12.4	17.0
22	1340	27.0	.5	.5	28.0	14.0	20.1
23	1230	31.1	.6	.7	32.4	16.2	26.3
24	1230	30.6	.6	.7	31.9	16.0	24.9
25	1300	23.8	.5	.3	24.6	12.3	18.3
26	1350	11.3	.2	-	11.5	5.7	8.4
Q	1380	.8	-	-	.8	.4	.6
G	1560	1.1	-	-	1.1	.6	.7
27	1350	11.3	.2	-	11.5	5.7	8.4
28	1300	23.8	.4	.3	24.5	12.3	18.3
29	1230	30.6	.6	.7	31.9	16.0	24.9
30	1230	31.1	.6	.7	32.4	16.2	26.3
31	1340	27.0	.5	.5	28.0	14.0	20.1
32	1420	24.2	.4	.3	24.9	12.5	17.0
33	1500	18.7	.2	.1	19.0	9.5	11.4
34	1550	8.8	.1	-	8.9	4.9	5.7
G_2	1380	..8	.4	-	1.2	.6	.6
H_2	1560	1.1	.2	-	1.3	.6	.7

TABLE VI (cont.)

MEMBER	UNIT R_v	TOTAL R_v (kips)	PLATE WEIGHT (kips)	MEMBER WEIGHT (kips)	GROSS R_v (kips)	$R_v/2$ (kips)	PLATE AREA SUPPORTED (sq. ft.)
35	2080	21.8	.2	.1	22.1	11.0	10.5
36	2060	42.0	.1	.3	42.4	21.2	20.3
37	2030	55.2	.6	1.2	57.0	28.5	27.2
V_1	2100	37.9	-	-	3.8	1.9	1.8
V	650	5.2	.2	-	5.4	2.7	8.1
A_1	2265	59.5	.6	-	60.1	30.0	26.3
38	1830	5.6	.1	-	5.7	2.9	3.1
39	1830	7.3	.5	.1	7.9	3.9	4.0
40	1830	7.9	.6	.2	8.7	4.4	4.3
41	1350	11.3	-	-	11.3	5.6	8.4
42	1300	23.8	-	-	23.8	11.9	18.3
43	1230	30.7	.2	.7	31.6	15.8	24.9
F_2	1380	.8	-	-	.8	.4	.6

TABLE VII

TABLE OF HORIZONTAL REACTIONS
(use with plate I in pocket)

MEMBER	NORMAL (pounds)	UNIT R_h (pounds)	TOTAL R_h (kips)	$R_h/2$ (kips)
1	2425	1710	7.0	3.5
2	2425	1710	9.4	4.7
3	2425	1710	12.3	6.2
4	2425	1710	12.8	6.4
J	2425	1710	6.0	3.0
L_2	2425	1710	3.2	1.6
5	1285	1245	19.3	9.6
6	1285	1245	28.0	14.0
7	1285	1245	22.4	11.2
8	1285	1245	9.6	4.8
I	1285	1245	1.0	.5
K_2	1285	1245	19.1	9.6
K	1740	1590	11.3	5.6
9	1740	1590	28.8	14.4
10	1740	1590	39.8	19.9
11	1740	1590	29.6	14.8
12	1740	1590	14.0	7.0
P	1740	1590	1.3	.7
13	1580	1500	16.5	8.3
14	1580	1500	16.5	8.2
15	1580	1500	30.8	15.4
16	1580	1500	26.3	13.2
17	1580	1500	18.0	9.0
18	1580	1500	9.3	4.7
I_2	1580	1500	2.3	1.2
19	1740	1590	9.1	4.6
20	1740	1590	18.2	9.1
21	1740	1590	27.1	13.6
22	1740	1590	32.0	16.0
23	1740	1590	41.7	21.8
24	1740	1590	39.6	19.8
25	1740	1590	29.1	14.6
26	1740	1590	13.4	6.7
Q	1740	1590	.9	.5
G	1740	1590	1.1	.5
27	1740	1590	13.4	6.7
28	1740	1590	29.1	14.6
29	1740	1590	39.7	19.8
30	1740	1590	41.7	20.8
31	1740	1590	32.0	16.0
32	1740	1590	27.1	13.6
33	1740	1590	18.2	9.1
34	1740	1590	9.1	4.5
G_2	1740	1590	.9	.5
H_2	1740	1590	1.1	.5
35	2323	1830	19.2	9.6

TABLE VII (cont.)

MEMBER	NORMAL (pounds)	UNIT R_h (pounds)	TOTAL R_h (kips)	$R_h/2$ (kips)
36	2323	1830	37.1	18.5
37	2323	1830	49.7	24.8
V_1	2323	1830	3.3	1.7
V	1285	1240	10.0	5.0
A_1	2425	1710	45.0	22.5
38	2110	1670	5.2	2.6
39	2110	1670	6.7	3.3
40	2110	1670	7.2	3.6
41	1740	1590	13.4	6.7
42	1740	1590	29.2	14.6
43	1740	1590	39.6	19.8
F_2	1740	1590	1.0	.5

TABLE VIII

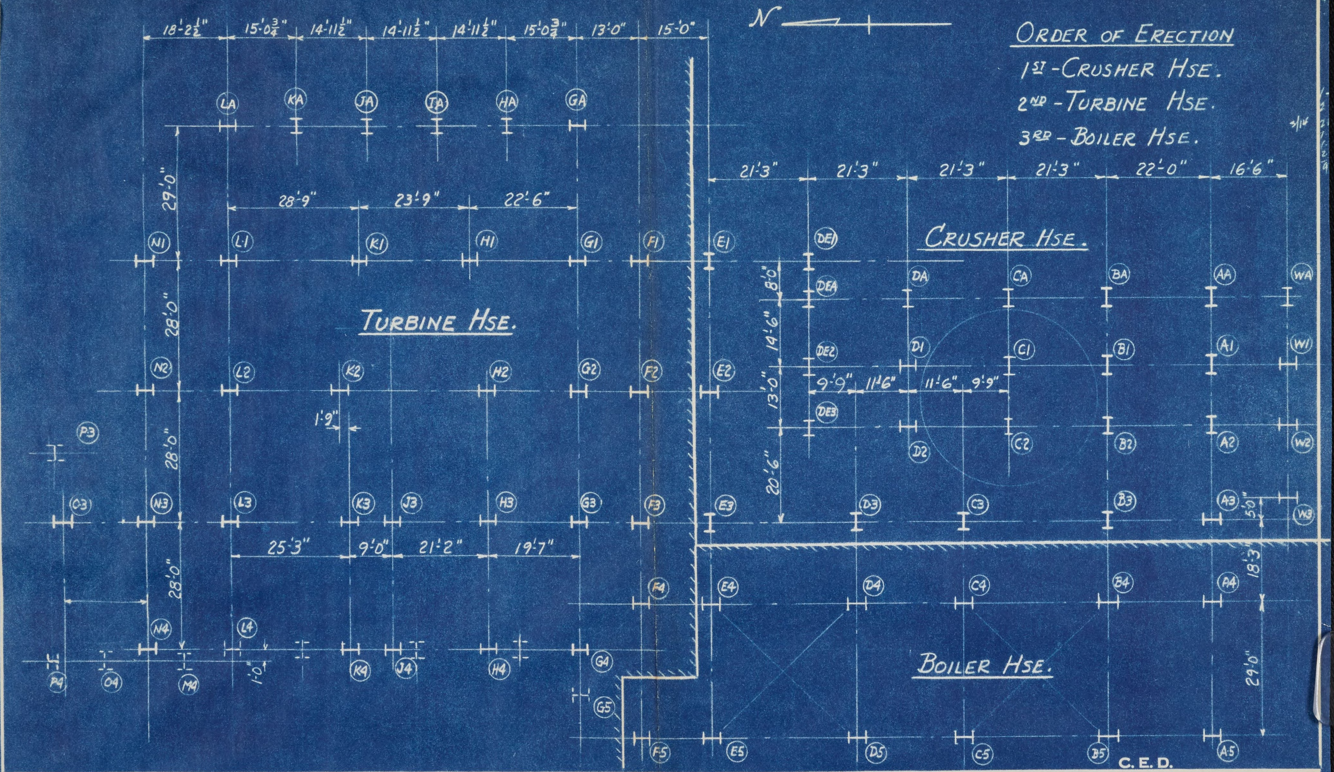
STRESSES IN SUPPORTING MEMBERS

MEMBER	UNIT NORMAL LOAD (pounds)	PLATE AREA SUPPORTED (sq.ft.)	TOTAL NORMAL LOADING (kips)	AXIAL LOADS V (kips)	AXIAL LOADS F (kips)	STRESS (kips/sq.in.)
1	2425	4.1	10.0	22.3	3.2	10.5
2	2425	5.5	13.5	24.8	4.4	14.0
3	2425	7.2	17.6	15.7	5.7	13.0
4	2425	7.5	18.4	10.0	6.0	8.94
5	1285	15.5	19.9	17.0	6.5	8.75
6	1285	22.5	28.9	17.0	9.4	12.0
7	1285	17.2	22.1	1.5	7.2	10.4
8	1285	7.7	9.9	3.0	3.3	7.34
9	1740	18.1	31.5	6.8	10.3	11.6
10	1740	25.0	43.5	6.8	14.2	14.5
11	1740	18.6	32.4	12.4	10.5	13.2
12	1740	8.8	15.3	5.9	5.0	-
13	1580	11.0	51.4	8.5	16.7	13.0
14	1580	11.0	17.4	8.5	5.6	13.0
15	1580	20.5	32.4	14.0	10.5	10.6
16	1580	17.5	27.6	13.3	9.0	10.0
17	1580	12.0	19.0	9.5	6.2	12.7
18	1580	6.2	9.8	5.0	3.2	-
19	1740	5.7	9.9	5.0	3.3	-
20	1740	11.4	19.8	9.5	6.5	13.7
21	1740	17.0	29.6	12.3	9.6	18.4*
22	1740	20.1	35.0	14.0	11.4	11.8
23	1740	26.3	45.7	20.0	14.9	13.3
24	1740	24.9	43.3	10.0	14.1	12.9
25	1740	18.3	31.8	12.3	10.3	13.0
26	1740	8.4	14.6	5.9	4.7	-
27	1740	8.4	14.6	5.7	4.7	-
28	1740	18.3	31.8	12.1	10.3	13.0
29	1740	24.9	43.3	10.0	14.1	11.8
30	1740	26.3	45.7	26.0	14.9	13.9
31	1740	20.1	35.0	33.2	11.4	13.9
32	1740	17.0	29.6	18.0	9.6	15.2
33	1740	11.4	19.8	15.0	6.5	12.2
34	1740	5.7	9.9	5.0	3.3	-
35	2323	10.5	24.4	5.0	7.9	11.1
36	2323	20.3	47.2	15.0	15.3	19.9*
37	2323	27.2	63.2	30.2	20.5	12.6
38	2110	3.1	6.6	0	2.1	-
39	2110	4.0	8.5	0	2.8	-
40	2110	4.3	9.1	0	3.0	-
41	1740	8.4	14.6	5.7	4.7	-
42	1740	18.3	31.8	12.0	10.4	20.5*
43	1740	24.9	43.3	26.0	14.1	16.0

*Upon checking with the designers to determine the reason for excess stress in these members, it was found that they had increased their size when making up the final bill of material.

Dept Ins: 7 Plans

[redacted]



NOTE REVISIONS: DESTROY ALL PRINTS MADE PREVIOUS TO LAST REVISION

	MADE BY	DATE	CH'K BY	APP. BY	APP'D BY CH. D'FTS'N
C					
DESCRIPTION:					
A					
DESCRIPTION:					
D					
DESCRIPTION:					
B					
DESCRIPTION:					

REFERENCES

W. O. NO.

BILL OF MATERIAL NO.

REFERENCE DRAWINGS

JOB NO.

NAME	DATE		NAME	DATE
Bhill	3-13-41	MADE BY	PERKINS	3-12-41
Blair	3-13-41	TRACED BY		
Meyer	3/13/41	CK'ED BY	Kennington	3-12-41
Lorimer	3/13/41	APP'D BY		
Dmyrk	3-14-41	APP'D BY CH. D'FTS'N		

USE DIMENSIONS ONLY

TITLE PLAN OF CRUSHER HSE. & ADDITION TO BOILER & TURB. HSES.

PLANT MARYSVILLE P.H.

THE DETROIT EDISON COMPANY

DRAFTING & SURVEYING DETROIT, MICH.

SCALE =1 FT. DATE PRINTED

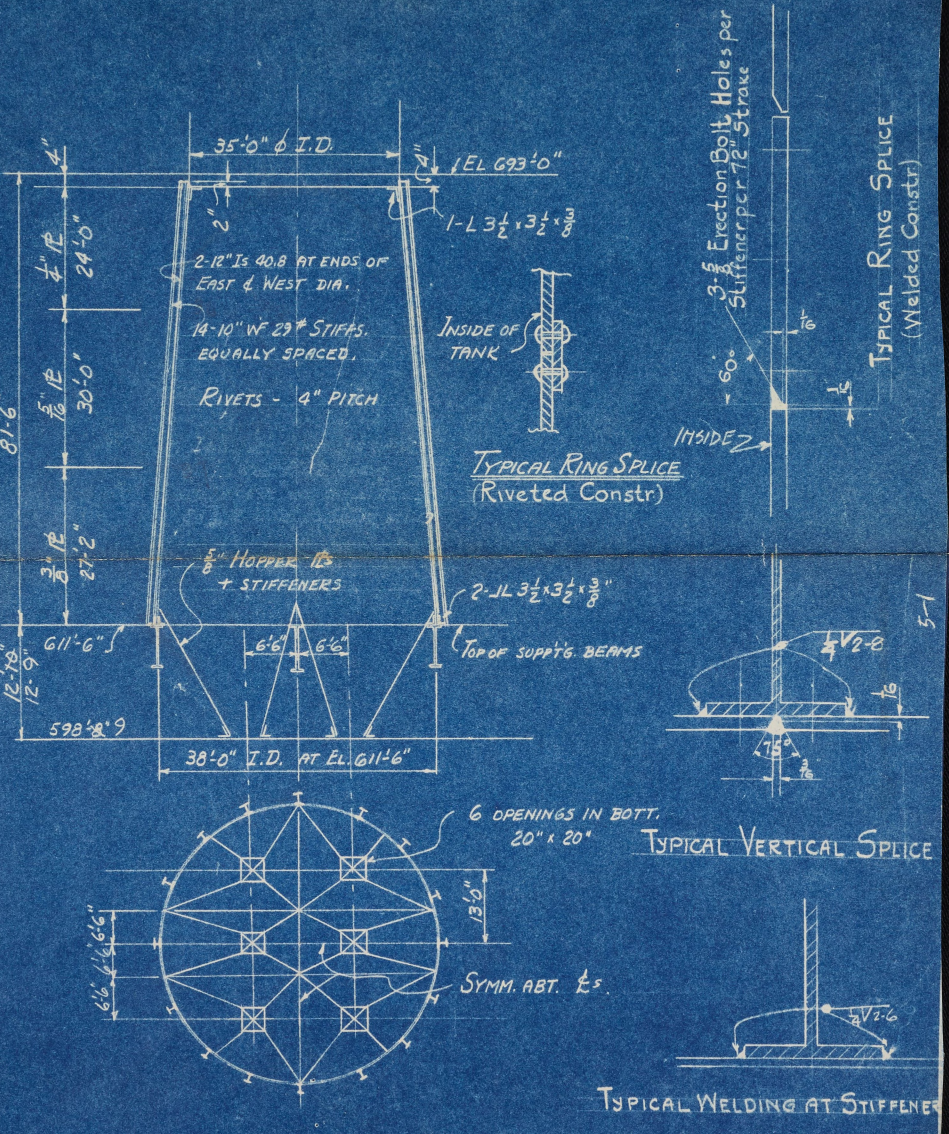
No. 3XS58-16

D E CO FORM DS 62 F

IN1M-01

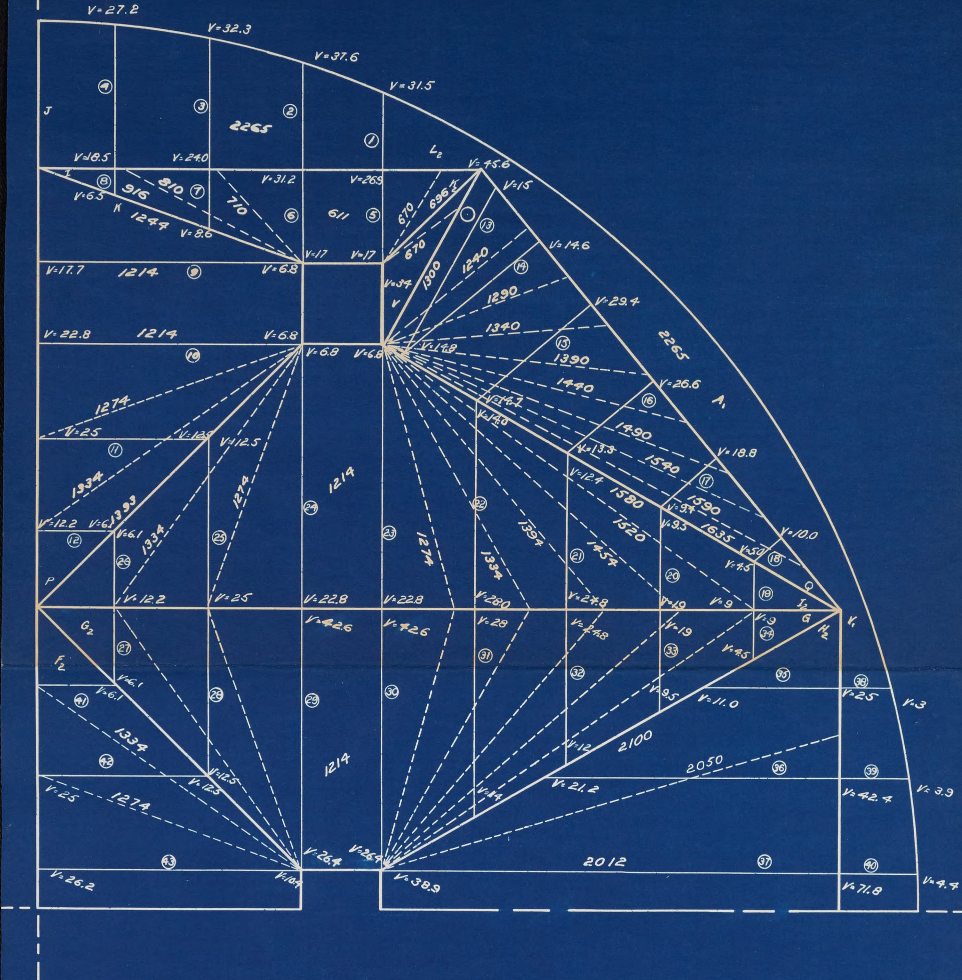
MARYSVILLE

2000 TON COAL SILO (APPROX.) 2-26-41



MARYSVILLE P.H.

MT 3X568-14



HOPPER SECTION

showing relative values to be used in stress analysis

Vertical Reactions - V= 000

Vertical Resistance sectional variation -- #/sq ft ------

Member ⊗

USE WITH PLATE 6558-3592

J.A.R.

DRAWINGS	ARCH.	Plau	3/26/41	MADE BY	Merriman
	BOILER			TRACED BY	
	CONC.	Chill	3-27-41	CH'K BY	Merriman 3-24-41
	ELEC.	[illegible]	3-27-41	APP'D BY	Thompson 3-28-41
	MECH.	Tunn	3-25-41	APP'D BY	Thornton 3-31-41
	PIPING	Welch	3/26/41	C. E. D. NO.	43518
	STEEL	Allen	3/31/41		

PLANT Marysville Power House.

THE DETROIT EDISON COMPANY

DRAFTING & SURVEYING BUREAU

DETROIT, MICH.

DRAWING NUMBER

6558-3590

SCALE 1 FT.

DATE PRINTED [illegible] 10 1941

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