An Analysis of a Reinforced Concrete Factory Building.

A Thesis Submitted to

The Faculty of

MICHIGAN AGRICULTURAL COLLEGE

by

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Candidates for the degree of

Bachelor of Science

June, 1920

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LIST OF PLATES.

INTRODUCTION.

The Sheet Metal Building of the Olds Motor Works consists of a one story reinforced concrete structure of the beam and girder type, 180 feet wide by 480 feet long. The roof is entirely of reinforced concrete and is a combination of flat slab and saw tooth construction. The walls are of brick to a height of about three feet. Truscon steel windows are placed above this wall and extend to the roof. A 1:2:4 mixture of concrete was used.

The building is used as a light machine shop. The beams which run longitudinally are provided with Truscon slotted inserts cast in the concrete to which the shaft hangers are attached. No provision is made for attaching machinery to the girders which are transverse of the building.

The building was designed by the H. G. Christman Co. and erected by them in the summer of 1919. The steel was furnished by the Truscon Steel Co. and consists of Kahn bars and Kahn rib bars.

It was our purpose in choosing this as the subject for a thesis to continue the study of reinforced concrete as applied to a practical design and to study the unusual problems presented in this type of a building.

The writers desire to acknowledge their indebtedness to Prof. C. L. Allen for the valuable assistance and advice given, and to Mr. Conrad of the H. G. Christman Co. for allowing free access to the building at all

INTRODUCTION.

times, for furnishing complete plans, data, and information, and for the practical advice so kindly given.

REFERENCES.

Hool, Reinforced Concrete Construction, Vol. 1 & 2.

Hool and Johnson, Concrete Engineers' Handbook.

Ketchum, Structural Engineers' Handbook.

Malcolm, Graphic Statics

Michigan State Highway Department, General Specifications for Steel and Concrete Highway Bridges.

Taylor and Thompson, Concrete Plain and Reinforced.

SPECIFICATIONS.

The following working stresses have been recommended by the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers presented before the society Jan. 17, 1917. Hool and Johnson, Concrete Engineers' Handbook, page 845-6.

	Per cent of compressive strength.	Lbs. per sq. inch.
1. Structural steel in tension		16,000
2. Concrete in compression where the surface is at least twice the loaded area	35.0	700
3. Concrete for concentric compression on a plain concrete column or pier, the length of which does not exceed 4 diameters --------------------	22.5	450
4. Compression on columns with longitudinal reinforcement to the extent of not less than 1 % and not more than 4 %, and with lateral ties of not less than ¼ inch in diameter, 12 inches apart, nor more than 16 diameters of the longitudinal bar ------------	22.5	450

5.	Per cent of compressive strength.	Lbs. per sq. inch.
5. Compression on columns reinforced with not less than 1 % and not more than 4 % of longitudinal bars and with circular hoops or spirals not less than 1 % of the volume of the concrete, the clear spacing of the hooping to be not greater than one- sixth of the diameter of the encased column and preferably not greater than one-tenth, and in no case more than 2-½ inches, where the ratio of unsupported length of column to diameter of the hooped core is not more than 10 ---------------	34.875	697.5
6. Compression on extreme fibre of a beam, calculated for constant modulus of elasticity (stresses adjacent of the support of continuous beams may be 15 % higher) -----------	32.5	650

	Per cent of compressive strength.	Lbs. per sq. inch.
7. Shear in beams with horizontal reinforcement or without reinforcement --------------	2	40
8. Shear in beams thoroughly reinforced with web reinforcement (the web reinforcement exclusive of bent-up bars to be designed to resist two-thirds the external shear) --------------------	6	120
9. Punching shear,only --------	6	120
10. Bond stress between concrete and plain reinforcing bars	4	80
11. Bond stress between concrete and drawn wire -------------	2	40

For the above a 1:2:4 mixture (Portland Cement Concrete) was used as a basis, having a strength of 2000 lbs. per square inch. (Compressive).

MOMENTS.

All steel to be allowed a working stress of 16,000 lbs. per sq. inch.

General Specifications for Steel and Concrete Highway Bridges,- Fourth Edition 1920,- Michigan State Highway Dep't.,- Spec. 179& 180,- page 14.

12. When the beam or slab is continuous over its supports, reinforcement shall be fully provided at points of negative moment, and the following stresses shall not

be exceeded; (See Spec. 237 & 238, page 17) for a 1:2:4 mixture with ultimate compressive strength per square inch of 2000 lbs. an extreme fiber stress of 650lbs. per square inch will be allowed, and adjacent to the support of continuous beams, stresses 15 % (or 747.5) higher may be used. (Agrees with Spec. 6 above).

In computing the positive and negative moments in beams and slabs continuous over several supports due to uniformly distributed loads the following rules shall be followed.

(a). That for floor slabs the bending moments at center and at supports shall be taken as $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per lineal foot and l the span.

(b) That for beams the bending moment at the center and at supports for interior spans shall be taken as $\frac{wl^2}{12}$, and for end spans $\frac{wl^2}{10}$ for center and adjourning supports, for both dead and live loads.

(c).In case of beams and slabs continuous for two spans only, the bending moment at the center support shall be taken as $\frac{wl^2}{8}$ and near the middle of the span as $\frac{wl^2}{10}$.

(d). At the ends of continuous beams the amount of negative bending moment will be left to the judgement of the designer, but it must be provided for.

(e). Continuous beams and slabs designed for concentrated loads shall have their moments calculated as if they were simply supported and the resulting moment shall then be multiplied by the factor eight-twelfths or eight-tenths to give the designing moment; the factor eight-twelfths shall be used where the coefficient of wl^2 for uniform loading is one-twelfth, and the factor eight-tenths shall be used where the coefficient of wl^2 for uniform loading is one-tenth.

LOADING.

13. Live load on floors. Hool - Vol. 2 - page 144.
Toilet room (same as public buildings) 100 lbs. per sq.ft.
Live load on beams. 500 lbs. per lineal foot of beam.
Note: Mr. Conrad of H. G. Christman Company advised us that this was the load assumed in designing the building. It includes the weight of the shaft hangers, shafting, pull of the machinery, and impact.

14. Live load on columns. Schneider's Spec. S. H. B. pg. 74.
Use specified uniform live load per square foot with minimum of 20,000 lbs. per column.

15. Loads on foundations. Schneider's Spec. S.H.B. pg. 75.
Live loads on columns shall be assumed to be the same as for the footings of columns. The areas of the bases of columns shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined live and dead loads. The dead load on the foundation shall be divided by the area thus found and this reduced pressure per square foot shall be permissable working pressure to be used for the dead load for all foundations.
Permissable pressure on foundations. S.H.B. pg. 75.
2 tons per square foot.

16. Wind load. Malcolm's Graphic Statics, pg. 73.

30 lbs. per square foot of vertical surface, the normal pressure to be the largest as determined by Duchemin's, Hutton's, or the Straight Line formula.

17. Snow on roof. Hool and Johnson, Concrete Engineers' Handbook pg. 512.

30 lbs. per square foot of horizontal surface.

DIMENSIONS.

18. The minimum width of web in beams and girders shall not be less than 1/24 of the span. Spec. 189 - pg.15 S. & C. H. Bridges.

DEAD WEIGHTS.

The following weights were used in figuring the dead loads.

Concrete - - - - - - - - - - - - - - - -	-150 lbs./cu. ft.
Brick - - - - - - - - - - - - - - - - - -	120 lbs./cu. ft.
Plaster - - - - - - - - - - - - - - - -	5 lbs./sq. ft
Roofing, (3 or 4 ply) - - - - - - - -	5 lbs./sq. ft.
Windows(Estimated) - - - - - - - - - - -	15 lbs./sq. ft.
Hollow tile Hool, Vol. 2, pg. 69	
6-12-12	22 lbs. each.
4-12-12	16 lbs. each.

LIVE and DEAD LOADS ON ROOF SLABS.

Flat Roof.

Snow load - - - - - - - - - - - - - - - - - - -	30.0	lbs. per sq.ft.
Roofing, 4 ply tar and gravel - - - - - - - - -	5.0	
Weight of slab $\frac{12 x 12 x 3 x 150}{144 x 12} =$	37.5	
Total load	72.5	lbs. per sq.ft.

Roof over saw tooth.

The roof slab as scaled from the blue print makes an angle of $28^\circ 35'$ with the horizontal.

Wind Load. Malcolm's Graphic Statics, page 73.

P=pressure per sq. ft. on a vertical surface.

P_n= pressure per sq. ft. normal to the roof surface.

A = angle the roof makes with the horizontal in degrees.

Duchemin's formula,-

$$P_n = p\,\frac{2\sin A}{1 + \sin^2 A} = \frac{30x\ 2x.47844}{1 + (.47844)^2} = 23.4 \text{ lbs. per sq. ft.}$$

Hutton's formula,-

$$P_n = P \sin A^{1.842\cos A - 1} = 30x(.47844)^{1.842x.87812-1}$$

P_n= 21 lbs. per sq. ft.

Straight Line formula,-

$$P_n = \frac{P A}{45} = \frac{30}{45}(28.58) = 19.1 \text{ lbs. per sq. ft.}$$

Use the largest of these or 23.4 lbs. per sq. ft.

Dead Load + Maximum Snow Load.

Max. snow load, 30 x cos 28° $35'$	= 26.34	lbs./ sq. ft.
Roofing, 3 ply tar and gravel	= 5.00	
Weight of slab, $\frac{2 x 12 x 12 x 150}{12 x 144}$	= 25.00	
Total vertical load	= 56.34	lbs./ sq.ft.
Total load normal to roof	= 49.47	lbs./ sq.ft.

Dead Load + Min. Snow + Wind.

Min. snow,	$15 \times \cos 28^\circ 35'$	= 13.17 lbs./ sq. ft.
Roofing, 3 ply tar and gravel,		= 5.00
Weight of slab	$\frac{2 \times 12 \times 12 \times 150}{12 \times 144}$	= <u>25.00</u>
	Total vertical load	= 43.17 lbs./sq.ft.

Normal to roof,-

$43.17 \times \cos 28^\circ 35'$	= 37.89 lbs./sq.ft.
Wind	= <u>23.40</u>
Total load normal to roof	= 61.29 lbs./sq.ft.

Use the larger of these two or 61.29 lbs. per sq. ft. as the normal load on the roof over the saw tooth.

LOADING ON BEAMS.

Beam A.

Roof (live + dead)	0.0
Weight of beam $\frac{8x16x150}{144}$ =	133.3
Live load (machinery)	500.0
Total	633.3 lbs./lin. ft.

Beam B

Roof (live + dead)	0.0
Weight of beam $\frac{8 \times 16 \times 150}{144}$ =	133.3
Live load (machinery)	500.0
Total	633.3 lbs./lin. ft.

Beam C.

Roof (live + dead) 7.5 x 72.5 =	543.75
Weight of stem $\frac{19.5 \times 8 \times 150}{144}$ =	162.50
Live load (machinery) =	500.00
Total	1206.25 lbs./lin. ft.

Beam D.

Roof (live + dead) 7.5 x 72.5 =	543.75
Weight of stem $\frac{8 \times 22 \times 150}{144}$ =	183.33
Live load (machinery) =	500.00
Total	1227.08 lbs./lin. ft.

LOADING ON BEAMS.

Beam E.

Roof (live + dead) $7.5 \times 72.5 =$ 543.75

Weight of stem $\frac{24.5 \times 8 \times 150}{144} =$ 204.00

Live load (machinery) $=$ 500.00

Total 1247.75 lbs./lin. ft.

Beam F.

Roof (live + dead) $3.5 \times 72.5 =$ 254.00

Weight of beam $\frac{12 \times 30 \times 150}{144} =$ 375.00

Live load (machinery) $=$ 500.00

Total 1129.00 lbs./lin. ft.

Beam G.

The amount of load on this beam from the slab in the saw tooth is indeterminate. We have assumed that one-fourth of a slab load will cover this load.

Roof (live + dead), flat roof, $3.75 \times 72.5 = 271.9$

Roof, saw tooth, $\frac{1}{2} \times 3.75 \times 72.5$ $= 135.9$

Weight of stem $\frac{12 \times 27 \times 150}{144}$ $= 338.0$

Live load (machinery) $= 500.0$

Total 1245.8 lbs/lin.ft.

LOADING ON BEAMS.

Beam H.

Roof (live + dead)	7.5×72.5	= 543.75
Weight of stem	$\frac{8 \times 19.5 \times 150}{144}$	= 162.50
Live load (machinery)		= 500.00
	Total	1206.25 lbs./ lin. ft.

Beam I.

Roof (live + dead)	7.5×72.5	= 543.75
Weight of stem	$\frac{8 \times 22 \times 150}{144}$	= 183.33
Live load (machinery)		= 500.00
	Total	1227.08 lbs./ lin. ft.

Beam J.

Roof(live + dead)	7.5×72.5	= 543.75
Weight of stem	$\frac{24.5 \times 8 \times 150}{144}$	= 204.00
Live load (machinery)		= 500.00
	Total	1247.75 lbs./lin. ft.

Beam K.

Under the flat roof at the east end of the building.

Roof (live + dead)	7.5×72.5	= 543.75
Weight of stem	$\frac{12 \times 27 \times 150}{144}$	= 337.00
Live load (machinery)		= 500.00
	Total	1380.75

LOADING ON BEAMS.

Beam K.

Under the saw tooth. Assume ½ slab load from the slab on the saw tooth.

Roof (live + dead)	$\frac{3}{4} \times 7.5 \times 72.5$	=	407.81
Weight of stem	$\frac{12 \times 27 \times 150}{144}$	=	324.00
Live load (machinery)		=	500.00
	Total		1231.81 lbs./lin. ft.

Beam L.

Roof (live + dead)	$\frac{3}{4} \times 7.5 \times 72.5$	=	407.81
Weight of stem	$\frac{11.5 \times 27 \times 150}{144}$	=	324.00
Live load (machinery)		=	500 .00
	Total		12.31.81 lbs./lin. ft.

In the above ½ a slab load was assumed to cover the load which might come on the beam from the slab over the saw tooth.

Beam M.

Roof (live + dead)	7.5×72.5	=	543.75
Weight of stem	$\frac{8 \times 24.5 \times 150}{144}$	=	204.00
Live load (machinery)		=	500.00
	Total		1247.75 lbs./lin. ft.

LOADING ON BEAMS.

Beam N.

Roof (live + dead)	7.5×72.5	= 543.75
Stem	$\frac{8 \times 22 \times 150}{144}$	= 183.33
Live load (machinery)		= 500.00
Total		1227.08 lbs./lin. ft.

Beam O.

Roof (live + dead)	$\frac{1}{2} \times 7.5 \times 72.5$	=271.87
Weight of 3 inch slab on the monitor	$\frac{2.75 \times 1 \times 3 \times 150}{12}$	= 103.10
Weight of sash	5×15	= 75.00
Estimated weight of slab extending under the monitor	$\frac{2.5 \times 3 \times 150}{12}$	= 94.00
Weight of stem	$\frac{8 \times 19.5 \times 150}{144}$	= 162.70
Live load (machinery)		= 500.00
Total		1206.67 lbs./ lin. ft.

Beam P.

Roof (live + dead)	$\frac{3}{4} \times 7.5 \times 72.5$	= 408.00
Weight of stem	$\frac{12 \times 27 \times 150}{144}$	= 338.00
Live load (machinery)		= 500.00
Total		1246.00 lbs./lin.ft.

The above is on the assumption that the load on the beam from the saw tooth slab is equal to one-half a slab load.

LOADING ON BEAMS.

Beam Q.

Beam supporting the toilet room located next to the stairs.

6 in. tile wall $\frac{3.5 \times 20 \times 22}{20}$ = 77.0

Weight of plaster both sides

$\frac{3.5 \times 20 \times 2 \times 5}{20}$ = 35.0

5 in. floor slab + 2½ in. granolithic

surface $\frac{7.5 \times 7.5 \times 150}{12}$ =704. 0

Live load (fixtures + people) 7.5x100= 750.0

Live load (machinery) = 500.0

Weight of stem $\frac{8 \times 16 \times 150}{144}$ = 132.0

Total 2198.0 lbs./lin. ft.

Beam Q.

Beam at center of the toilet room span.

Weight of cuttain wall located 2 ½ feet from center line o

of the beam. (4 inch tile, plastered both sides).

Weight of wall 7 x 20 x 18 = 2520

Weight of plaster 10x7x20 = 1400

3920 lbs.

Weight of wall on beam $\frac{5 \times 3920}{7.5}$ = 2610.0

Weight of slab $\frac{7.5 \times 20 \times 7.5 \times 150}{12}$ =14080.0

Live load on floor 7.5 x 20 x 100 =15000.0

Live load (machinery) 20 x 500 =10000.0

Weight of beam $\frac{8 \times 16 \times 20 \times 150}{144}$ = 2670.0

44360.0 lbs.

$\frac{44,360.0}{20}$ = 2218.0 lbs. /lin. ft.

LOADING ON BEAMS.

Beam Q.

Located at the back of the toilet room floor.

Curtain wall	$\frac{2.5 \times 1960}{7.5}$	= 655.0
Floor slab	$\frac{10 \times 3.75 \times 7.5 \times 150}{12}$	= 3520.0
Live load	3.75 x 10 x 100	= 3750.0
Live load (machinery)	10 x 500	= 5000.0
Weight of beam	$\frac{8 \times 16 \times 150 \times 20}{144}$	= 1335.0
	Total	14260.0 lbs.

$\frac{14,260.0}{10}$ = 1426.0 lbs. per lin ft.

Beam Q.

Supporting heating coils (See plate 15).

Weight of motor	½ x 1500	= 1500.0
Weight of fan	½ x 3000	=1500.0
Floor slab	$\frac{5 \times 7.5 \times 20 \times 150 \times 12}{144}$	= 9300.0
Weight of beam	$\frac{22 \times 8 \times 20 \times 150}{144}$	= 3670.0
Live load	20 x 500	=10000.0
	Total	25220.0 lbs./

$\frac{25,220.0}{20}$ = 1261.0 lbs. per lin. ft.

Beam(Qx)' (See plate 15)

Weight of heater	½ x 32688	= 16,344.0
Floor slab	$\frac{6.5 \times 3.75 \times 20 \times 150}{12}$	= 6,090.0
Weight of beam	$\frac{12 \times 22 \times 20 \times 150}{144}$	= 5,280.0
Live load (machinery)	20 x 500	= 10,000.0
	Total	37,714.0 lbs.

$\frac{37,714.0}{20}$ = 1885.0 lbs. per lin. ft.

LOADING ON BEAMS.

Beam Qx. (See plate 15)

Fan	$\frac{1}{2} \times 3000$	= 1500.0
Weight of heater	$\frac{1}{2} \times 32688$	= 16344.0
6 ½" floor slab	$\frac{6.5 \times 3.75 \times 20 \times 150}{12}$	= 6090.0
5" floor slab	$\frac{5 \times 3.75 \times 20 \times 150}{12}$	= 4800.0
Live load (machinery)	20×500	= 10000.0
Weight of beam	$\frac{12 \times 22 \times 150 \times 20}{144}$	= 5280.0
	Total	44014,0 lbs.

$\frac{44.014}{20}$ = 2201 lbs. per lin. ft.

Beam R. (See plate 15)

Located under slab supporting heaters.

Weight of motor	$\frac{1}{2} \times 1500$	= 750.0
Weight of floor slab	$\frac{\frac{1}{2} \times 5 \times 7.5 \times 20}{12}$	= 4650.0
Weight of 8" wall	$\left(\frac{8 \times 15 \times 20 \times 1500}{12}\right) - \left(\frac{5 \times 15 \times 150 \times 8}{12}\right)$	=22500.0
Weight of windows	$5 \times 15 \times 15$	= 1125.0
Weight of stem	$\frac{12 \times 28 \times 20 \times 150}{144}$	= 7000.0
Live load (machinery)	20×500	= 10000.0
	Total	46025.0 lbs.

$\frac{46.025}{20}$ = 2301.2 lbs. per lin. ft.

Beam R.

Same as beam G - - - - - - - - - - - 1245.8 lbs. per lin. ft.

LOADING ON BEAMS.

Beam T.

Roof (live + dead) 7.5 x 72.5 = 543.75

Weight of stem $\frac{8 \times 27 \times 150}{144}$ = 225.00

Live load (machinery) = 500.00

Total 1268.75 lbs./ lin. ft.

Beam U.

Roof (live + dead) 7.5 x 72,5 = 543.75

Weight of stem $\frac{8 \times 24.5 \times 150}{144}$ = 204.00

Live load (machinery) = 500.00

Total 1247.75 lbs./lin. ft.

Beam V.

Roof (live + dead) 7.5 x 72.5 = 543.75

Weight of stem $\frac{8 \times 22 \times 150}{144}$ =183.33

Live load (machinery) =500.00

Total 1227.08 lbs./lin. ft.

Beam W.

Roof (live + dead) 7.5 x 72.5 = 543.75

Weight of stem $\frac{19.5 \times 8 \times 150}{144}$ = 163.00

Live load (machinery) = 500.00

Total 1206.75 lbs./lin. ft.

LOADINGS ON BEAMS.

Beam X.

Roof (live + dead)	7.5 x 72.5	= 543.75
Weight of stem	$\frac{12 \times 27 \times 150}{144}$	= 337.50
Live load (machinery)		= 500.00
	Total	1381.25 lbs./lin.ft.

Beam Z.

Roof (live + dead)	3.5 x 72.5	= 254.0
Weight of beam	$\frac{12 \times 30 \times 150}{144}$	= 375.0
Live load		= 500.0
	Total	1129.0 lbs./lin.ft.

LOADING ON GIRDERS.

Girder A A.

Roof (live + dead)	7.5 x 72.5	=	0.0
Weight of girder	$\frac{11.5 \times 20 \times 150}{144}$	=	240.0
Live load (machinery)		=	500.0
	Total		740.0 lbs./lin.ft.

Girder B B.

Roof (live + dead)		=	0.0
Weight of girder	$\frac{11.5 \times 20 \times 150}{144}$	=	240.0
Live load (machinery)		=	500.0
	Total		740.0 lbs./lin.ft.

Girder C C.

Load from beam F	½(20 x 1129.0)	=	11290.0
" " " C	20 x 1206.25	=	24135.0
" " " D	20 x 1227.08	=	24541.6
" " " E	20 x 1247.08	=	24955.0
" " " G	½(20 X 1245.81)	=	12458.1
Weight of girdet	$\frac{\frac{1}{2} \times 17 \times 27 \times 15.5 \times 30 \times 150}{144}$	=	10656.0
	Total		108,035.7 lbs.

$\frac{108,035.7}{30}$ = 3601.2 lbs. per lin. ft.

Girder D D.

Load from beam G	½(20x1245.810	=	12,458.1
3 beam A	3x20x633.3	=	37,998.0
beam L	½x20x1231.81	=	12,381.1
Weight of girder	$\frac{\frac{1}{2} \times 13 \times 27 \times 15.5 \times 30 \times 150}{144}$	=	9,700.0
	Total		72,537.2 lbs.

$\frac{72,537.2}{30}$ = 2417.9 lbs. per lin. ft.

LOADING ON GIRDERS.

Girder E. E.

Load from	beam L ½x20x1231.81	=	12,318.1
	beam M 20x1247.75	=	24,955.0
	beam N 20x1227.08	=	24,541.6
	beam O 20x1206.75	=	241 35.0
	beam AA ½x20x1283.75	=	12,837.5
Weight of girder	$\frac{½x17x27x15.5x30x150}{144}$	=	10,656.0
	Total		109,443.2 lbs.

$\frac{109,443.2}{30}$ = 3648.1 lbs. per lin. ft.

Girder F F.

Due to the unusual loading the moment was figured on the basis of concentrated loads.

Load at 1/4 point,

Beam Q	10x2198.0	21980.0	
Beam A	10x633.0	6330.0	
			28,310.0 lbs.

Load at 1/2 point,

Beam Q	10x2218.0	22,180.0	
Beam A	10x633.3	6,333.0	28,513.0 lbs.

Load at 3/4 point,

Beam Q	10x 1426.0	14,260.0	
Beam A	10x633.3	6,333.0	20,593.0 lbs.

Weight of beam $\frac{16 \times 30 \times 150}{144}$ = 500#/lin.ft.

Weight of wall 6x10.5x4x10.5x½x18 = tile = 1512

6x10.5x4x10.5x½x10 = plaster = 840

2352 lbs.

LOADING ON GIRDERS.

Girder G G.

Load at 1/4 point,-

Beam (Qx)'	$\frac{37,714}{2}$	=	18,857.0	
Beam A	10 x 633.3	=	6,333.0	25,190.0 lbs.

Load at 1/2 point,-

Beam (Qx)	$\frac{44,014}{2}$	=	22,007.0	
Beam A	10 x 633.3	=	6,333.0	28,340.0 lbs.

Load at 3/4 point,-

Beam Q	$\frac{25,220}{2}$	=	12,610.0	
Beam A	10 x 633.3	=	6,333.0	18,943.0 lbs.

Weight of wall $\frac{26.5 \times 11.5}{2} \times (22 + 10)$ = 4,873.5 lbs.

Weight of girder $\frac{16 \times 30 \times 30 \times 150}{144}$ = 15,000,0 lbs.

Total uniform load $\frac{4,873 + 15,000}{30}$ = 662.4 lbs./ lin. ft.

Girder H H.

Load from Beam G		$\frac{1}{2}$ x 20 x 1245.8	=	12,458.0
	Beam A	3 x $\frac{1}{2}$ x 20 x 633.3	=	18,999.0
	Beam L	$\frac{1}{2}$ x 20 x 1231.81	=	12,318.1
	Beam K	1 x 20 x 1380.75	=	27,615.0
	Beam T	3 x $\frac{1}{2}$ x 20 x 1268.75	=	38,062.5
Weight of girder		$\frac{27 \times 15.5 \times 30 \times 150}{144}$	=	13,100.0
Weight of wall		$\frac{210 \times 6 \times 150}{12}$	=	15,750.0
				138,302.6 lbs.

$\frac{138,302.6}{30}$ = 4610.9 lbs. per lin. ft.

LOADING ON GIRDERS.

Girder I I.

Loading from Beam Z $\frac{1}{2}$ x 20 x 1129 = 5645.0

Beam H $\frac{1}{2}$ x 20 x 1206.25 = 12062.5

Beam I $\frac{1}{2}$ x 20 x 12270.8 = 12270.8

Beam J $\frac{1}{2}$ x 20 x 1247.75 = 12477.5

Beam K $\frac{1}{4}$ x 20 x 1380.75 = 6903.7

Weight of girder $\frac{27 \times 15.5 \times 150 \times 30}{144}$ = 13100.00

62,459.5 lbs.

$\frac{62,459.5}{30}$ = 2081.98 lbs. per lin. ft.

Girder J J.

Loading from Beam K $\frac{1}{4}$ x 20 x 1380.75 = 6903.75

Beam T 3 x $\frac{1}{2}$ x 20 x 1268.75=27615.00

Beam K $\frac{1}{4}$ x 20 x 1380.75 = 6903.75

Weight of girder $\frac{27 \times 15.5 \times 30 \times 150}{144}$ = 13100.00

54,522.50 lbs.

$\frac{54,522.50}{30}$ = 1817.4 lbs. per lin. ft.

Girder K K.

Loading from Beam Z $\frac{1}{4}$ x 20 x 1129 = 5,645.0

Beam H $\frac{1}{2}$ x 20 x 1206.25 =12,062.5

Beam I $\frac{1}{2}$ x 20 x 1227.08 = 12,270.8

Load from the Enameling Building,-

Floor slab $\frac{10 \times 20 \times 8.5 \times 150}{12}$ =

Live load 10x20x150 = 51,200.0

LOADING ON GIRDERS.

Girder K K Cont'd.

Wall load $\frac{7.0 \times 20 \times 3.75 \times 120}{12}$	=	12,250.8
Sash 5x15x20	=	1,500.0
Total		94,928.3 lbs.

$\frac{94,928.3}{20}$ = 4746.4 lbs. per lin. ft.

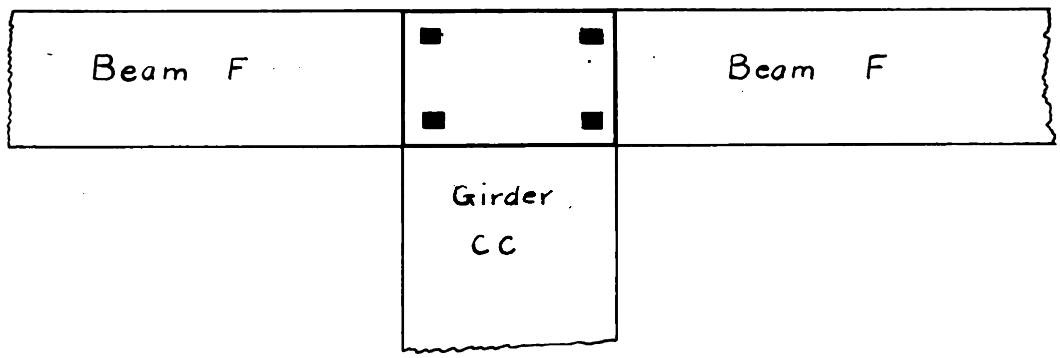
Girder L L.

Load from Beam J $\frac{1}{2}$x20x1247.75	=	12,477.5
Load from Beam K $\frac{1}{2}$ x20x1380.75	=	13,807.5
Load from Beam B $\frac{1}{2}$x20x633.3	=	6,333.0
Loads from the enameling building,-		
Floor slab (See girder K K)	=	51,200.0
Wall load $\frac{7 \times 160 \times 120}{12}$	=	11,200.0
Sash 15 x 120	=	1,800.0
Total		96,818.0 lbs.

$\frac{96,818.0}{20}$ = 4840.9 lbs. per lin. ft.

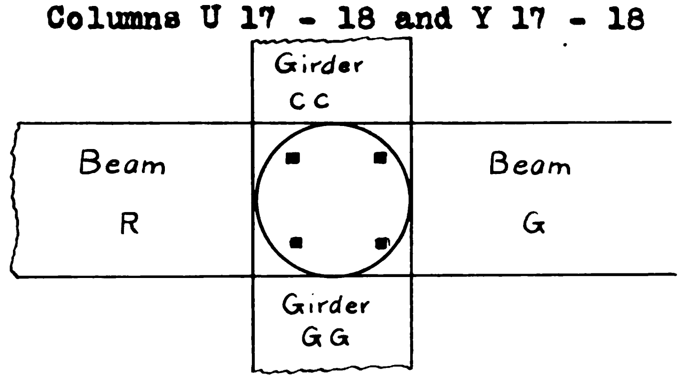
LOADING ON COLUMNS.

Columns T 6 - 28 and Z 6 - 28.



Beam F	20 x 1129.0	= 22,580
Girder C C		
Beam C	24,135	
½ Beam D	12,270	
½ weight of girder	5,328	= 41,733
Weight of column	$\frac{12 \times 16 \times 14 \times 150}{144}$	= 2,800
	Total	67,113 lbs.

Columns U 17 - 18 and Y 17 - 18



Beam G	10 x 1246.8	= 12,468
Beam R	10 x 1246.8	= 12,468
Girder C C (See page 29)		= 42,553
Girder G G (See page 25)	½x92346.	= 46,173
Strut I36,800 cos 30		= 31,869
Weight of column	$\frac{3.1416 \times 64 \times 14 \times 150}{144}$	= 2,916
	Total	145,964 lbs.

LOADING ON COLUMNS.

Columns U 9 - 16 & 19 - 25

Y 9 - 16 & 19 - 25

Beam G	20 x 1246.8		= 24,936
Girder C C			
Beam E	20x1247.75	= 24,955	
½ Beam D		= 12,270	
½ weight of girder		= 5,328	= 42,553
Girder D D			
Beam A	20x633.3	= 12,666	
½ Beam A	10x633.3	= 6,333	
½ weight of girder		= 4,850	= 23,849
Strut I	36,800 cos 30		= 31,869
Weight of column	$\frac{3.1416 \times 64 \times 14 \times 150}{144}$		= 2,916
		Total	166,123 lbs.

LOADING ON COLUMNS.

Columns U 28 and Y 28.

For typical columns seepages 28 and 29.

Beam G	=	12,468
Beam K	=	13,808
Girder C C	=	42,553
Girder H H	=	40,331
Strut I	=	18,153
Weight of column	=	2,916
Total		130,229 lbs.

Columns U 7 - 8 and 26 - 27
Y 7 - 8 and 26 - 27

Beam G	=	12,468
Beam P	=	12,460
Girder C C	=	42,553
Girder F F	=	47,907
Strut I	=	31,869
Weight of column	=	2,916
Total		150,173 lbs.

Columns U 8 -16 and 19 -25
Y 8 - 16 and 19 - 25.

Assuming maximum load from girder CC. No machinery load from girder DD.

Beam G	=	24,936
Girder C C	=	42,553
Girder D D	=	8,849
Strut I	=	31,869
Weight of column	=	2,916
Toaal		111,123 lbs.

LOADING ON COLUMNS.

Columns V 7 - 8 and 26 - 27.
X 7 - 8 and 26 -27.

Beam L	= 12,318
Beam P	= 12,460
Girder E E	= 42,554
Girder F F	= 37,279
Beam H (Saw tooth) 52707 cos 30	= 45,644
Weight of column	= 2,916
Total	153,171 lbs.

Columns V 9 - 16 and 19 - 25.
X 9 - 16 and 19 - 25.

Beam L	= 24,636
Girder E E	= 42,554
Girder D D	= 23,849
Beam H (Saw tooth)	= 45,644
Weight of column	= 2,916
Total	139,599 lbs.

Columns V 17 - 18
X 17 - 18

Beam L	= 12,318
Beam R	= 28,512
Girder E E	= 42,554
Girder G G	= 45,739
Beam H (Saw tooth)	= 45,644
Weight of column	= 2,916
Total	177,683 lbs.

LOADING ON COLUMNS.

Columns V 28 and X 28.

Beam L	= 12,318
Beam K	= 13,808
Girder E E	= 42,554
Girder H H	= 35,080
Beam H (Saw tooth)	= 45,644
Weight of Column	= 2,916
Total	152,320 lbs.

Columns V 9 - 16 and 19 - 25.
X 9 - 16 and 19 - 25.

Assuming maximum load on girder E E and no machinery load on girder D D.

Beam L	= 24,636
Girder EE	= 42,554
Girder D D	= 8,849
Beam H (Saw tooth)	= 45,644
Weight of column	= 2,916
Total	124,599 lbs.

Columns W 7 - 27.

Girder A A	= 14,800
Girder E E	= 85,108
Weight of column	= 2,916
Total	102,824 lbs.

LOADING ON COLUMNS.

Columns W 7 - 27.

Assume a maximum load on one girder E E and no machinery load on the other girder E E.

Girder A A		=	14,800
Girder E E		=	42,554
Girder E E		=	27,554
Weight of column	Total	=	2,916
	Total		87,814 lbs.

Columns T 29 and Z 29.

Beam Z		=	11,290
Girder I I		=	24,748
Weight of column	$\frac{16x16x14x150}{144}$	=	3,735
	Total		39,773 lbs.

Columns U 29 and Y 29.

Beam K		=	13,807.5
Girder I I		=	25,162.9
Girder J J		=	25,581.2
Weight of column		=	2,916.0
	Total		67,467.6 lbs.

Columns V 29 and X 29.

Beam K		=	13,807.5
Girder J J		=	54,522.5
Weight of column		=	2,916.0
	Total		71,246.0 lbs.

LOADING ON COLUMNS.

Column W 29.

Beam X	=	13,812.5
Girder J J	=	54,522.5
Weight of column	=	2,916.0
Total		71,251.0 lbs.

ANALYSIS OF ROOF SLAB.

In the analysis of the roof slab the formulas for rectangular beams given in Hool's Reinforced Concrete Construction Vol. 1, pages 112 and 113 were used. The entire load was assumed to be carried the short way of the slab. This is in accordance with the recommendation of Hool (Vol. 1, pg.140) that when the ratio of the length to breadth is more than 1.5, the load is carried entirely by the reinforcement the short way of the span.

NOTATION.

f_c = unit compressive stress in outside fiber of concrete.

f_s = unit tensile stress in steel.

n = ratio of modulus of elasticity of steel in tension to modulus of elasticity of concrete in compression.

a_s = area of cross-section of steel.

b = breadth of beam.

d = distance from compression surface to axis of reinforcement.

M = bending moment.

p = steel ratio.

k = ratio of depth of neutral axis to depth of steel.

j = ratio of lever arm of resisting couple to depth of steel.

V = total shear.
v = unit shear.

v' = unit working shear.

u = unit bond.

o = circumference of one bar.

w = uniform load in pounds per foot.

l = span of beam in feet.

ANALYSIS OF ROOF SLAB.

Reinforcing = 7/32 inch bars spaced 5" c. to c.

Span = l = 7.5 ft.

Loading = w = 72.5 lbs.

d = 2.5

$M = wl^2/12 = 72.5 \times 7.5 \times 7.5 \times 12/12 = 4080$ in. lbs.

$$p = \frac{a_s}{bd} = \frac{.03758}{5 \times 2.5} = .003$$

$$k = \sqrt{2pn + (pn)^2} - pn = \sqrt{2 \times .003 \times 15 + (.003 \times 15)^2}$$

$$- .003 \times 15$$

k = .2583

$$j = 1 - \frac{k}{3} = 1 - \frac{.2583}{3} = .9139$$

$$a_s = \frac{44}{20} \times .03758 = .08267$$

$$f_s = \frac{M}{a_s jd} = \frac{4080}{.08267 \times .9139 \times 2.5} = 21{,}600 \text{ lbs. per sq. in}$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2 \times 4080}{.9139 \times .2583 \times 12 \times (2.5)^2} = 461 \text{ lbs./sq. in.}$$

$$v = \frac{V}{bjd} = \frac{3.75 \times 72.5}{12 \times .9139 \times 2.5} = 9.9 \text{ lbs. per sq. in.}$$

$$u = \frac{V}{jd\,sum\,o} = \frac{3.75 \times 72.5}{.9139 \times 2.5 \times 1.515} = 5.76 \text{ lbs. per sq. in.}$$

sum o = 3.1416 x 7/32 x 2.2 = 1.515

Over the Support.

Reinforceing consists of alternate bars bent up over the support, or 7/32 in bars spaced 10" c to c.

$$p = \frac{a_s}{bd} = \frac{.03758}{10 \times 2.5} = .0015$$

$$k = \sqrt{2pn + (pn)^2} - pn = \sqrt{2x.0015x15 + (.0015x15)^2} - .0015x15$$

k = .179

j = 1 - k/3 = 1 - .179/3 = .9403

$$f_s = \frac{M}{a_s jd} = \frac{4080}{.0414 \times .9403 \times 2.5} = 42{,}000 \text{ lbs. per sq. in}$$

ANALYSIS OF ROOF SLAB.

$$f_c = \frac{2M}{jkbd^2} = \frac{2 \times 4080}{.9403 \times .179 \times (2.5)^2 \times 12}$$

f_c = 646 lbs. per sq. in.

$$u = \frac{V}{j\ d\ \text{sum of } o} = \frac{3.75 \times 72.5}{.9403 \times 2.5 \times 3.1416 \times 2.2 \times 7/32}$$

u = 78.2 lbs. per sq in.

End panels.

Reinforceing,- 7/32 in. bars spaced 5" c to c.

12 - 7/32 in. bars spaced equally.

$M = wl^2/10 = 72.5 \times 7.5 \times 7.5 \times 12/10 = 4890$ in. lbs.

$$a_s = \frac{52}{20} \times .03758 = .0977$$

$$p = \frac{a_s}{bd} = \frac{.0977}{12 \times 2.5} = .00327$$

k = .267

j =.911

$$f_s = \frac{M}{a_s jd} = \frac{4890}{.0977 \times .911 \times 2.5} = 21{,}950 \text{ lbs. per sq. in.}$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2 \times 4890}{.911 \times .267 \times 12 \times (2.5)^2} = 536 \text{ lbs./sq/in.}$$

$$v = \frac{V}{bjd} = \frac{3.75 \times 72.5}{.0911 \times 12 \times 2.5} = 9.94 \text{ lbs. per sq. in.}$$

$$u = \frac{V}{jd\ \text{sum } o} = \frac{3.75 \times 72.5}{.911 \times 2.5 \times 1.79} = 66.5 \text{ lbs. per sq. in.}$$

ANALYSIS OF BEAMS.

Beam C - Typical Tee-beam.

Reinforcing,- 1 - 3/4 x 2 3/16 kahn bar 0.79 sq. in.

1 - 5/8 rib bar 0.3906 "

Total area 1.1806 sq. in.

Loading = 1206.25 lbs. per lin. ft.

span = 20 ft.

According to the recommendations of the joint committee;-

(1). b = $\frac{1}{4}$ span of beam = $\frac{1}{4}$ x 20 = 5 ft.

(2). b = six times the thickness of the slab on each side of the stem = 2 x(3 x 6) + 7½ = 43½ inches.

(3). b = distance between beams = 7 ½ ft.

Use the least of these three conditions, or b = 43 ½ in.

$$kd = \frac{2nda_s + bt^2}{2na_s + 2bt} = \frac{2 \times 15 \times 21 \times 1.1806 + 43.5 \times 9}{2 \times 15 \times 1.1806 + 2 \times 43.5 \times 3}$$

kd = 3.83

Therefore the neutral axis falls in the stem, and the formulas for case 2 apply.

$$p = \frac{a_s}{bd} = \frac{1.1806}{43.5 \times 21} \quad .001293$$

$$j = \frac{6 - 6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 + \left(\frac{t}{d}\right)^3\left(\frac{1}{2pn}\right)}{6 - 3\left(\frac{t}{d}\right)}$$

$$j = \frac{6 - 6\left(\frac{3}{21}\right) + 2\left(\frac{3}{21}\right)^2 + \left(\frac{3}{21}\right)^3 \frac{1}{(2 \times .001293 \times 15)}}{6 - 3\left(\frac{3}{21}\right)}$$

j = .945

$M = wl^2/12$ = 1206.25 x 20 x 20 x 12/12 = 482,500 in. lbs.

$$f_s = \frac{M}{a_s jd} = \frac{482,500}{1.1806 \times .945 \times 21} = 20,600 \text{ lbs./ sq. in.}$$

ANALYSIS OF BEAMS.

Beam C - Tee-beam.

$$f_c = \frac{M}{\left(1 - \frac{t}{2kd}\right)btjd} = \frac{482{,}500}{\left(1 - \frac{3}{2x3.83}\right) 43.5x3x.945x21}$$

f_c = 306 lbs. per sq. in.

$$v = \frac{V}{b'jd} = \frac{10 \times 1206.25}{7.5 \times .945 \times 21} = 80.9 \text{ lbs./ sq. in.}$$

$$u = \frac{V}{jd \text{ sum } o} = \frac{10 \times 1206.25}{.945 \times 21 \times 7.7} = 78.8 \text{ lbs./ sq. in.}$$

$$u = \frac{a_s f_s}{01'} = \frac{.126 \times 16000}{2 \times .81 \times 12.05} = 103 \text{ lbs. per sq. in.} = \text{bond}$$

in the web reinforcing.

$$l' = \frac{.6d}{.707} - \left(\frac{d}{.707} - 24\right) = 12.05 = \text{effective length of}$$

web reinforcing when inclined at an angle of 45.

$$x' = \frac{l}{2} - \frac{v'bjd}{w} = \frac{20}{2} - \frac{40 \times 7.5 \times .945 \times 21}{1206.25} = 5.07 \text{ ft.}$$

= distance from support to where web reinforcement is needed.

$$s = \frac{3a_s f_s jd}{1.4\ V} = \frac{3x.126x16000x.945x21}{10x\ 1206.25\ x1.4} = 7.12 \text{ in.} = \text{required}$$

spacing of the web reinforcement.

$$f_s = 2/3 \times \frac{0.7Vs}{a_s jd} = \frac{2x0.7x10x1206.25x12}{3x.126x.945x21}$$

f_s = 27,000 lbs. per sq. in. = stress on the web bars from diagonal tension.

ANALYSIS OF BEAMS.

Beam A- Rectangular beam.

Reinforcement,-

1 - 3/4 x 2 3/16 kahn bar

1 - 1/2 rib bar

Span 20 ft.

Loading - 633 lbs. per lin. ft.

a_s = 1.04 sq. in.

$$p = \frac{a_s}{bd} = \frac{1.04}{7.5 \times 14.5} = .00956$$

$$k = \sqrt{2pn + (pn)^2} - pn$$

$$k = \sqrt{2x.00956x15 + (.00956x15)^2} - .00956x15$$

$$k = .4112$$

$$j = 1 - k/3 = 1 - .4112/3 = .863$$

$$M = wl^2/12 = 633.3 \times 20 \times 20 \times 12/12 = 253,320 \text{ in. lbs./ sq}$$

$$f_s = \frac{M}{pjbd^2} = \frac{M}{a_s jd} = \frac{253,320}{1.04 \times .863 \times 14.5}$$

f_s = 19,400 lbs. per sq. in.

$$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 253,320}{.863 \times .411 \times 7.5 \times 14.5^2}$$

f_c = 907 lbs. per sq. in.

$$v = \frac{V}{bjd} = \frac{10 \times 633.3}{7.5 \times .863 \times 14.5} = 67 \text{ lbs. sq. in.}$$

$$u = \frac{V}{jd \text{ sum } 0} = \frac{10 \times 633.3}{.863 \times 14.5 \times 7.2} = 70 \text{ lbs. per sq. in.}$$

l' = 15.8 in. = effective lenght of the web reinforceing.

$$u = \frac{a_s f_s}{0\ l'} = \frac{.126 \times 16000}{1.62 \times 15.8} = 77.2 \text{ lbs per sq. in.}$$

$$x_, = \frac{l}{2} - \frac{v'bjd}{w} = \frac{20}{2} - \frac{40 \times 7.5 \times .863 \times 14.5}{633.3}$$

$x_,$ = 4.05 ft. = distance from the support to the point where web reinforcement is needed.

ANALYSIS OF BEAMS.

Beam A continued.

$$s = \frac{3a_s f_s jd}{1.4\ V} = \frac{3x.126x16000x.863x14.5}{1.4x10x633.3} = 8.6 \text{ in.}$$ = required

spacing of the wing bars.

$$f_s = 2/3\ \frac{0.7\ x\ Vs}{a_s jd} = \frac{2x0.7x10x633.3x12}{.126x.863x14.5} = 23{,}020 \text{ lbs. per}$$

sq. in. = tensile stress in the web reinforcement.

The negative moments for both the beams and girders were computed as above for a rectangular beam.

ANALYSIS OF COLUMNS.

Columns U and Y 17 and 18.

Reinforceing,- 4 - 5/8 in. square bars.

Length - 14 ft.

Gross diameter - 16 inches.

Diameter inside reinforceing - 13 inches.

Total load - 145,964 lbs.

$$p = \frac{1.5624}{132.7} = .01178$$

$$f_c = \frac{P}{A(1 + (15 - 1)p)} = \frac{145,964}{132.7(1 + (15 - 1) \times .01178)}$$

f_c = 945 lbs. per sq. in.

f_s = 15 x 945 = 14200 lbs. per sq. in.

Considering Eccentricity.

Columns W 7 - 27.

The method of analysis outlined in Hool's Reinforced Concrete Construction, pages 395-413, of Vol. 2 was used. The formulas for beams continuous over three spans were used. This is not exactly correct but Hool states that this method may be used in the case of beams continuous over four or more spans without any great error.

B = ratio of moment of inertia to length for a beam.

C= ratio of moment of inertia to lenght for a column.

α = the slope or angle made by the neutral surface at the end of a member with its original position.

E = modulus of elasticity.

K = constant in equation $M = KCE\alpha$

I_b = moment of inertia of beam.

I_c = moment of inertia of column.

ANALYSIS OF COLUMNS.

Column W 7 - 27.

$$B = \frac{.11 \times (28)^3 \times 15.5}{30 \times 12} = 73.5$$

(See Fig. 317, pg 405 Vol.2 of Hool.)

$$C = \frac{4125}{12 \times 14} = 24.5$$

$$\alpha = \frac{l^2}{12E} \frac{1.5\, W_1 - W_2}{KC + 5B} = \frac{(30)^2\,(12)^2}{12 \times E} \frac{1.5 \times 3468 - 2135}{4 \times 24.5 + 5 \times 73.5}$$

$$\alpha = \frac{7100}{E}$$

$$M = 4EC\alpha = 4E \frac{I_c}{12 \times 14} \frac{7100}{E} = 169 I_c$$

$$\text{stress} = \frac{Mx}{I_c} = \frac{169 I_c \times 4.5}{I_c} = 760 \text{ lbs. per sq. in.}$$

ANALYSIS OF THE SAW TOOTH.

2 inch roof slab.

Reinforceing - none.

Span - 12 inches.

Load - 61.29 lbs. per sq. ft.

$$Shear = \frac{\frac{1}{2} \times 61.29}{2 \times 12} = 1.28 \text{ lbs. per sq. in.}$$

$$M = wl^2/\ 12 = \frac{61.29 \times 1 \times 12}{12} = 61.29 \text{ in. lbs.}$$

$$f_c = \frac{My}{I} = \frac{61.29 \times 1}{.667 \times 12} = 7.65 \text{ lbs per sq. in.}$$

ANALYSIS OF THE JOIST.

Reinforceing - 1 - 3/8" rib bar

1 - 1/2" rib bar

Span - 20 ft. Size 4" x 8"

Loading normal to the joist -

Roof slab(live + dead load) 16/12x61.29	=	81.72
Tile - 12x12x6	= 22.0	
Plaster	= 5.0	
Weight of joist	= 25.0	
52.0 cos 28 35	=	45.66
Total .		127.38 lbs.

b = (2 x 6)2 + 4 = 16"/

$$kd = \frac{2nda_s + bt^2}{2na_s + 2bt} = \frac{2x15x7x.3906 + 16\ x4}{2x15x.3906 + 2x16x4} = 1.93$$

Therefore the neutral axis is in the flange and the formulas for rectangular beams apply.

ANALYSIS OF SAW TOOTH.

$p = \frac{a_s}{bd} = \frac{.3906}{16 \times 7} = .003495$

$k = \sqrt{2pn + (pn)^2} - pn = \sqrt{2x.003495x15 + (.003495x15)^2} - .00349x15$

$k = .2755$

$j = 1 - k/3 = 1 - .2755/3 = .908$

$M = wl^2/12 = 127.38 \times 20 \times 20 \times 12/12 = 50,952$ in. lbs.

$f_s = \frac{M}{a_s jd} = \frac{50,952}{.3906 \times .9082 \times 7} = 20,500$ ~~in.~~ lbs./ sq. in.

$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 50952}{.2755 \times .908 \times 16 \times 49} = 520$ lbs. per sq. in.

$v = \frac{V}{bjd} = \frac{10 \times 127.38}{7 \times .908 \times 4} = 50.1$ lbs. per sq. in.

$u = \frac{V}{jd \text{ sum } 0} = \frac{10 \times 127.38}{.908 \times 7 \times 3.5} = 57.3$ lbs. per sq. in.

Stresses in joist due to tangential force of the load.

Reinforceing - for tension 1 - 1/2" rib bar

for compression 1 - 3/8" rib bar

Loading,- snow + roofing + slab = $\frac{16}{12}$ 43.17	= 57.56
Tile - 1 - 6x12x12	= 22.00
Plaster 16/12 x 5.00	= 5.00
Weight of joist $\frac{4x6}{144}$ x 150	= 25.00
	111.56

111.56 x cos 28 35 53.37 lbs./ lin. ft.

$M = wl^2/12 = 53.37 \times 400 \times 12/12 = 21,348$ in. lbs.

$p = \frac{a_s}{bd} = \frac{.25}{6x4} = .01$

$p' = \frac{.1406}{6x4} = .00585$

ANALYSIS OF SAW TOOTH.

$$k = \sqrt{2n(p + p'\frac{d'}{d}) + n^2(p + p')^2} - n(p + p')$$

$$k = \sqrt{2x15(.01 + .00585x1/3) + 225\ x.00159} - 15x.01585$$

$$k = .3872$$

$$z = \frac{1/3k^3d + 2p'nd'(k - d'/d)}{k^2 + 2p'n(k - d'/d}$$

$$z = \frac{1/3(.2378)^3x3 + 30x.00585(.3872-1/3)}{(.2378)^2 + 2x15(.3872 - 1/3)} = .0095$$

$$jd = d - z = 3 - .0095 = 2.99$$

$$f_c = \frac{6M}{bd^2(3k - k^2 + \frac{6p'n}{k}(k - d'/d)(1 - d'/d)}$$

f_c = 640 lbs. per sq. in.

$f_s = \frac{M}{pjbd^2}$ = 39,600 lbs. per sq. in.

$f'_s = nf_c \frac{k - d'/d}{k}$ =1345 lbs. per sq in. = compression in steel.

Considering the stresses from both the normal and tangential loads,-

Maximum stress in the concrete = 640 + 520 = 1160 lbs./ sq. in.

Maximum stress in the steel = 39,600 + 20500= 60100 " " "

This is the worst possible condition that could exist and the loading as assumed probably never would act in this way. The roof slab would take at least a part of the tangential load and it is possible that it would take the entire load. It is believed that it would be necessary to consider only the normal load, in the design of the joist.

ANALYSIS OF THE SAW TOOTH.

Alternate joist.

Reinforceing - 1 - 3/8" rib bar

1 - 3/8" rib bar

The computations are the same as given for the previous case, and the following results were obtained,-

Due the normal load.

f_s= 27,950 lbs. per sq. in.

f_c = 587 lbs. per sq. in.

$v = \frac{V}{bjd}$ = 49.4 lbs. per sq. in.

u = 65.8 lbs. per sq. in.

Due to the tangential load.

f_s = 58,700 lbs. per sq. in.

f_c = 643 lbs. per sq. in.

BEAM H.

Reinforceing,- 2 - 1½ x 2¼ kahn bars

2 - 7/8" rib bars

Span 24.5 ft.   Size - 12 x 20

Loading,-(roof slab + snow + wind)=24.5x20x61.29 =30,032.1

Joist 19x20x25 = 9,500

tile 19 x20x22 = 8,360

plaster24.5x20x5 = 2,450

beam$\frac{12\text{x}18}{144}$x150x24.5= 5,512

25,822 x cos 28 35   22,674.8

Total   52,706.9 lbs.

$\frac{52,706.9}{24.5}$ = 2150 lbs. per ~~sq. in.~~ lin. ft.

Computing as a tee-beam with a 2" concrete slab the following results were obtained,-

ANALYSIS OF SAW TOOTH.

Beam H continued.

f_s = 20,200 lbs. per sq. in.

f_c = 1395 lbs. per sq. in.

v = 130.5 lbs. per sq. in.

u = 78 lbs. per sq. in.

In the web reinforceing,-

u = 74.8 lbs. per sq. in.

f_s = 11,950 lbs. per sq. in.

f_s

Considering beam H as a strut supporting the tangential load.

$$f_c = \frac{P}{A(1 + (n - 1)p)}$$

P = 25,996 lbs.

$$p = \frac{a_s}{A} = .01812$$

$$f_c = \frac{25,996}{12x20(1 - (15 - 1).01812)}$$ 86.4 lbs. per sq. in.

Considering both the normal and tangential loads, the maximum stress in the concrete is found to be 1481.4 lbs./sq.in.

Beam D.

Considering beam D in the same manner, the following results are obtained,-

Reinforceing, -2 - 3/4 x 2 3/16 kahn bars

2 - 3/4" rib bars

Span - 24.5 ft. Size - 10 x 20

ANALYSIS OF THE SAW TOOTH.

Beam D.

Loading,-	slab + snow + wind		15,016.0
joist	19 x 10 x 25	=4750	
tile	19 x 10 x 22	= 4180	
plaster	24.5x10x5	= 1225	
beam	$\frac{12\times18}{144}\times150\times24.5$	= 2756	
		12,911 cos 28 35	11,337.4
			26,353.4

$\frac{26,353.4}{24.5}$ = 2330 lbs. per lin .ft.

Computing as a rectangular beam.

$p = .0154$

$k = .484$

$j = .839$

$M = wl^2/10 = 959,000$ in. lbs.

$f_s = 22,850$ lbs. per sq. in.

$f_c = 1454$ lbs. per sq. in.

$v = 110.4$ lbs. per sq. in.

$u = 93.6$ lbs. per sq. in.

Stresses in the wing bars.

$u = 92.3$ lbs. per sq. in.

$f_s = 70,000$ lbs. per sq. in.

Considering beam D.as a strut supporting the tangential load.

Loading = 13,871 lbs.

$p = .0135$

$f_c = 28.6$ lbs. per sq. in.

Maximum stress in the concrete considering both loads

= 1454 + 28.6 = 1482.6 lbs. per sq. in.

ANALYSIS OF THE SAW TOOTH.

Strut I.

In the analysis of the struts the maximum stresses were found to exist as a result of the dead weight of the roof and snow load above and at the same time the wind blowing against the sash produceing a bending moment. Hence only these results will be given.

Loading,-

Roof slab + snow = 30 x 20 x 37.89	=	22,734
Weight of joist, tile, etc	=	25,248

Only one-half of the roof load comes on the strut, the balance resting on the girder.

1/2 x(25,248 + 22,734) =		23,982
Weight of strut and bracket	=	2,634
Weight of sash	=	3,160
Total load	=	29,785 lbs.

Reinforceing,- 4 - 1/2" rib bars.

Size,- 12"x 12" within reinforceing bars - 9"x9"

Length,- 11 ft.

$$p = \frac{a_s}{A} = .01235$$

$$f_c = \frac{P}{A(1 + (n - 1)p)} = \frac{29,785}{81(1 + 14 \times .01235)} = 313.5 \text{ lbs./sq.in.}$$

f_s = 15 x 313.5 = 3291 lbs. per sq. in.

Considering the wind pressure against the sash.

Wind pressure against the upper sash = $6 \times \frac{20}{2} \times 30 = 1800$

Normal pressure = 1800 x cos 30 = 1559

Angle of lower sash with horizontal is 60 degrees.

$$P_n = \frac{30 \times 2 \times .866}{1 + .866^2} = 29.7 \text{ lbs. per sq. ft. of surface.}$$

ANALYSIS OF THE SAW TOOTH.

Strut I.

Pressure on lower sash= 6 x 10 x 29.7 =		1782 lbs.
1/2 x (1559 + 1782) = pressure at mid point	=	1670
Weight of sash at same point	=	430
Total load	=	2100 lbs.
The reaction = 6/11 x 2100	=	1145

M = 10/12 x 6 x 1145 x 12 = 68,700 in. lbs.

$f_c = \frac{6 \times 68,700}{1160} = 355$ lbs. per sq in.

$f_s = \frac{68,700}{.00397 \times .91 \times 12 \times (10.5)^2} = 14,400$ lbs. per sq. in.

Maximum stresses,-

$f_c = 355 + 313.5 = 668.5$ lbs. per sq. in.

$f_s = 14,400 - 15 \times 313.5 = 14,071$ lbs. per sq. in.

Strut J.

Since there is no beam resting on the top of strut J none of the foof load will be considered as coming on it.

Reinforceing - 4 - 1/2" rib bars.

Size - 12" x 12" Size of core 9" x 9".

Length - 12 ft.

Loading due to the wind pressure against the sash,-

2100 lbs. (See strut I).

Reaction = 1/2 x 2100 = 1050 lbs.

M = 6 x 1050 x 12 x 10/12 = 63,000 in. lbs.

$f_c = 326$ lbs. per sq. in.

$f_s = \frac{63,000}{.00397 \times .91 \times 12 \times (10.5)^2} = 13,200$ lbs. per sq.in.

CONCLUSION.

At the time the analysis was started very little machinery was placed. Hence it was necessary to adopt the loading for which the building was said to be designed. This live load of 500 lbs. per lineal foot of beam which was adopted is believed to be extremely high. Probably 200 lbs. per lineal foot will cover the total load existing on the beams and provide an impact coefficient of 100 per cent. If the loads are reduced in this ratio practically all the apparent over stresses shown on the stress sheet will disappear.

According to some authorities the snow load of 30 lbs. per square foot of horizontal surface might be reduced to 20 lbs. per square foot. The construction and location of the building makes it possible for a large amount of snow to accumulate and be held on the roof and it is thought best to use this higher load.

In the roof slab the concrete was found to be within the allowable working stress. The steel at the center of the slab is over stressed. Over the support the stress was found to be 42,000 lbs. per sq. in. Only every alternate bar was bent up and hence the stress was nearly doubled. According to the Joint Committee the span length may be taken as the clear distance between the faces of the support for slabs and beams built monolithic with the supports. (H.& J. pg. 318, Art. 44). If this rule were followed as the designers state was done, the span length would

CONCLUSION.

be reduced to 6 1/2 feet instead of 7 1/2 feet thus reducing the stresses.

In the beams the compression on the concrete was low except in the beams A, B, R, and W. Beams R and W were analized as rectangular beams due the their irregular shape. It may be permissable, as was done in designing the beams, to consider these as tee-beams. If this were done the stresses would be reduced to a considerable extent. However, as the roof slab on one side is at an angle of 30 or 60 degrees, it is not thought advisable to do this. The steel in all the beams shows a slight over stress. This may be accounted for by the fact that the span length used in design was the clear space between faces of the supports and also by the fact that the live load is high. The stresses in the steel due to the negative bending moment at the supports are not correct as additional steel not shown on the steel plans was placed in the beams at these points. This was not known until the analysis was completed and the amount of steel is not known.

In the girders the stresses were found to be higher. In designing the building the live load on the girders was reduced 20 %. In the analysis the full live load was used based on Hool, Vol. 2 page 211 where he states that the full live load should be used on the columns for a factory building.

The stress in the concrete over the supports is high.

CONCLUSION.

The girders may have been designed as doubly reinforced. This is not permissable as the steel at the bottom of the girders does not continue through the center of support, leaving a space of about three inches where there is no steel. The negative moment is a maximum at the center of support and decreases rapidly toward the edge of the column. Hence there will only be a small space which will be under high stress.

Assuming that a kahn bar is a deformed bar the bond stress at the center of the span was found to be safe for both the beams and girders. The values found for bond over the supports apparently show over stress. The hook on the end of the bars may be counted on to reduce this to the allowable value.

The Joint Committee recomment an allowable shear value of 120 lbs. per sq. in. when the beam is thoroughly reinforced for diagonal tension. These beams and girders are not fully reinforced since the wing bars are spaced too far apart to take the required 2/3 of the diagonal tension. It can only be said that the shear value is higher than recommended.

The columns were analized first, considering only the total load, second, considering the possible eccentricty. The over stresses found when the load was considered as centrally applied are undoubtly due to the high machinery

CONCLUSION.

load and to the fact that the building was designed on the basis of a 20 % reduction of live load in the girders and a 10 % further reduction in this live load to the columns. In a building of this type the effect of eccentricity shoud not be overlooked. The load on the column from girder C C may be double that from D D. This produces large stresses which are equal to if not greater than those due to the dead weight alone.

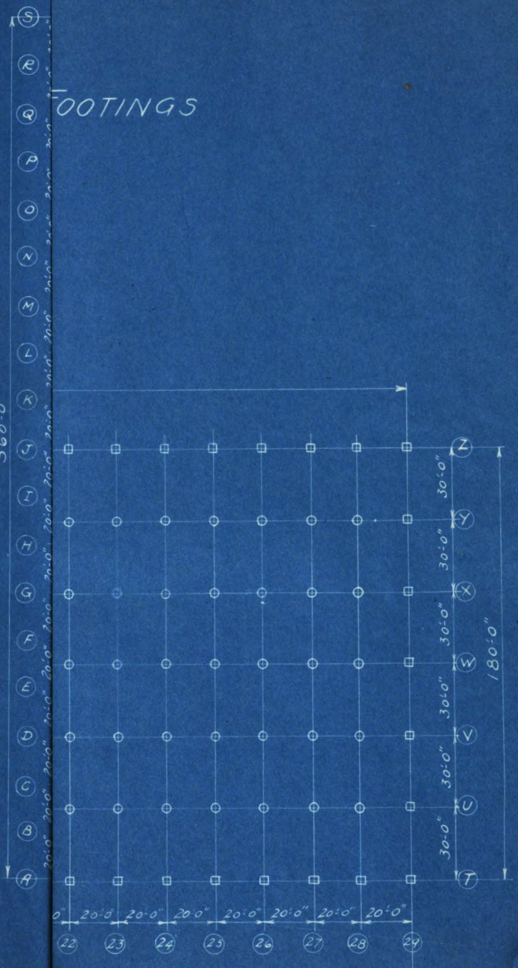
In the saw tooth the steel, in the joist was found to be over stressed somewhat due to the normal load alone. While it is possible that the joist would be called on to bear the tangential load it is not at all likely. The roof slab is a part of the flat roof slab and may be depended upon to take considerable of this load in compression.

The steel in the beams D and H in the saw tooth was found to be over stressed. The concrete in the same beams was found to be highly over stressed, beam H having a stress of 1395 lbs. per sq. in. due to the normal component alone.

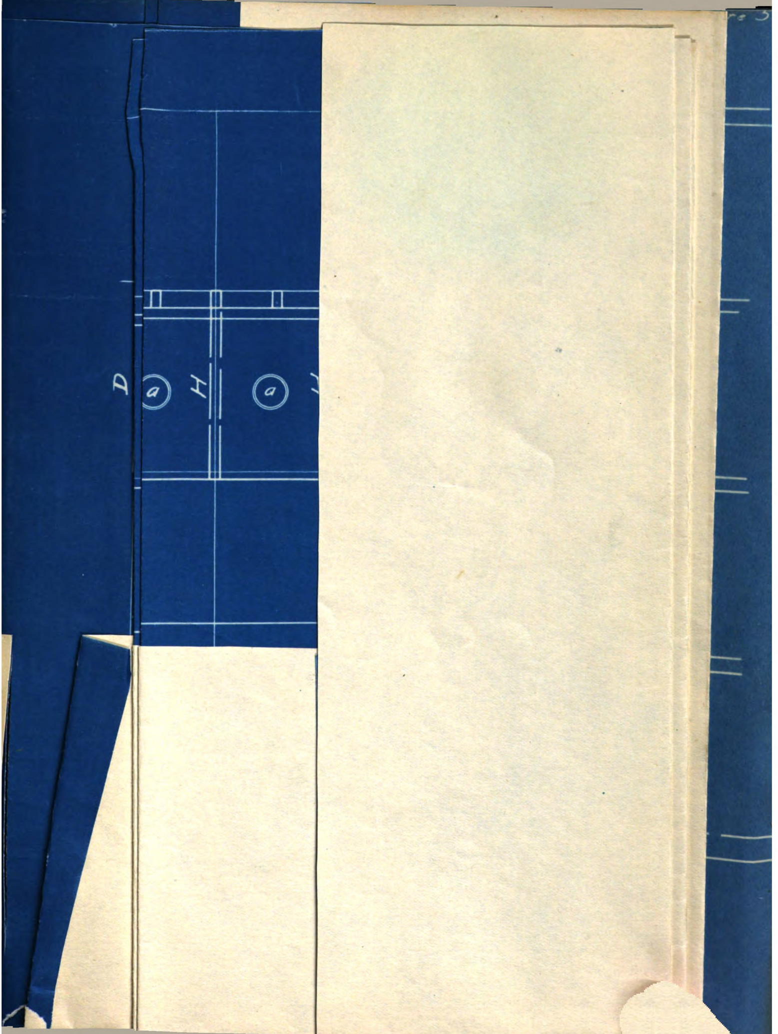
In the struts the maximum stresses were found to be produced by the wind blowing against the sah which are fastened to the struts near the center and the dead load of the roof and snow above. The stresses in both the steel and concrete were found to be well within the allowable working values.

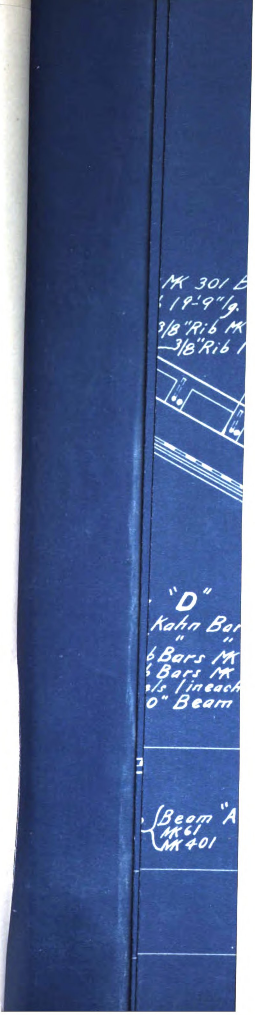
Plate I.

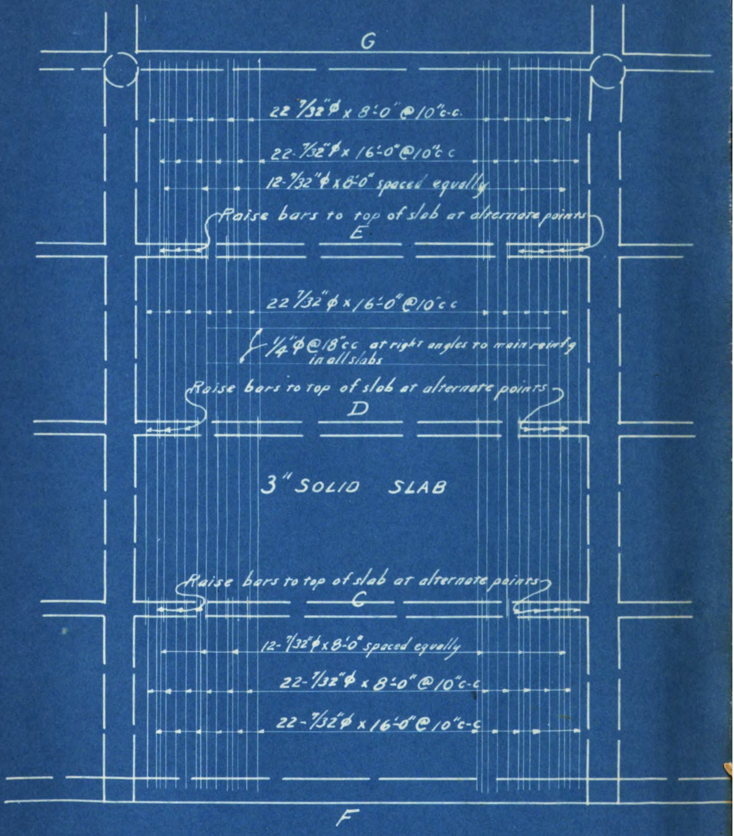
FOOTINGS




F
2
CC
15
DD
7
EE


D
a
H
a


MK 301 B
19'-9" lg.
3/8" Rib MK
3/8" Rib
"D"
Kahn Bar
" "
Bars MK
Bars MK
ls 1 in each
0" Beam
Beam "A
MK 61
MK 401



PLAN OF SLAB NO. 2

LAYOUT OF STEEL FOR OTHER SLABS SIMILAR

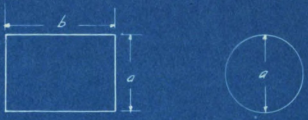
Scale 1/4"=1'-0"

No	MK	Size	Lgth	A	B	C	D
92	310	3/8	8-8		See		Sket
114	401	1/2"	29-6		7-0	1-5	12-8
6	402		25-0		2-7	1-5	12-7
88	403		30-3		7-0	2-6	11-3
88	404		30-3		7-0	2-10	10-7
22	405		27-0		4-3	3-1	12-4
4	407		25-9		7-0	2-3	11-6
44	408		30-0		7-0	2-3	11-6
2	409		26-3		7-0	3-1	10-5
22	410		29-9		7-0	1-11	12-0
82	414	1/2"	30-9		5-6	3-1	13-7
44	503	5/8"	30-0		7-0	2-3	11-6
8	504		26-0		7-0	2-6	11-3
8	505		26-0		7-0	2-10	10-8
18	506		26-3		7-0	3-1	10-5
20	507		30-9		7-0	3-1	10-7
4	508		25-9		7-0	2-3	11-6
1	509		25-9		3-0	1-11	12-0
46	526	5/8"	30-9		5-6	3-1	13-7
2	611	3/4"	30-9		7-0	3-1	10-7
14	612		38-3		4-3	3-1	23-7
2	613		36-0	6"	1-3	3-1	23-10
2	614		27-3	6"	1-9	3-1	12-0
2	615		25-0	6"	1-0	3-1	13-10
7	626	3/4"	30-9		5-6	3-1	13-7
2	701	7/8"	39-0	6	1-10	3-1	21-0
4	702	7/8"	44-6		9-6	3-1	19-4
46	800	1"	38-9	6"	1-9	1-11	21-10
50	801		44-6		9-6	3-1	19-4
46	802		44-3		9-6	1-1	20-3
46	803		35-9	6"	1-3	1-11	23-9
46	804		38-0		4-3	1-11	24-6
4	805	1"	38-3		4-3	3-1	23-7

Bent Rib Bar Diagrams

Be

COLUMN SECTIONS & REINFORCING



Column No	a	b	R-Bars			Stays		
			No	Diam	Lgth	No	Diam	Lgth
T6-T28	12	16	4	5/8 ϕ	15'-4"	15	1/4 ϕ	4-8
Z6-T28	12	16	" "	" "	15'-4"	15	" "	4-8
U6-U29	16		" "	" "	16-6	16	" "	4-4
V6-V29	" "		" "	" "	16-6	16	" "	4-4
X6-X29	" "		" "	" "	16-6	16	" "	4-4
Y6-Y29	" "		" "	" "	16-6	16	" "	4-4
W6-W29	" "		" "	" "	15-8	15	" "	4-4
T29	" "		" "	" "	15-4	15	" "	5-4
Z29	" "		" "	" "	15-4	15	" "	5-4

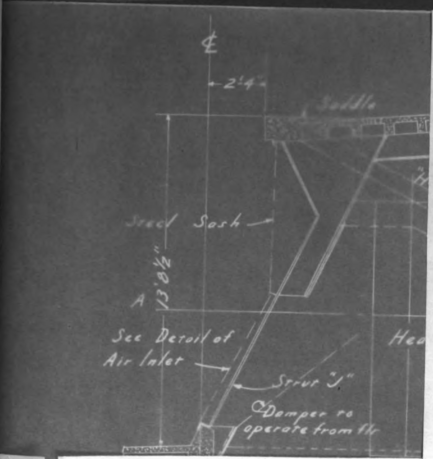
ROOF SLAB STRESSES

Span	Load per sq. ft	Max Shear	Max.+ Moment	Shear	At Center				Over Support				
					a_s	f_s	f_c	Bond	Max Moment	a_s	f_s	f_c	Bond.
7-6	72.5	272	4080	9.9	.083	*21600	461	78	4080	.041	*42000	646	78

	Over Support						
fs Wing	Max −M $\frac{wl^2}{10}$ $\frac{wl^2}{12}$	b	d	p%	k	j	a_s
A 23020	253320	7½	14½	.0121	.448	.851	1.32
B 23760	303984	7½	14½	.0121	.448	.851	1.32
C 27000	482500	7½	21	.00315	.263	.912	.7812
D 24740	490832	7½	23½	.00285	.252	.916	.50
E 22340	499100	7½	26	.002565	2365	.921	.50
F 42800	451500	11½	23½	.00153	.1941	.935	.50
G 20900	498324	11½	28½	.00153	.1941	.935	.50
H 27000	599000	7½	20	.00428	.300	.900	.6406
I 27600	589000	7½	23½	.00366	262	.906	6406
J 22340	598000	7½	26	.00329	269	.910	6406
K 22760	591264	11½	28½	.00196	215	.929	6406
L 46400	492724	11½	28½	.00238	.234	.929	7812
M 22340	499100	7½	26	.002365	2365	.921	.50
N 24740	490832	7½	23½	.002835	.252	.916	50
O 29000	482500	7½	21	.00317	.265	.912	.50
P 21000	494500	11½	23½	.00238	.234	.922	.7812
Q 12680							
R 49700	1140000	11½	28½	.00272	.248	.917	.8906
S							
T 20600							
U 25200	598000	7½	26	.00329	.269	.910	.6406
V 55100	589000	7½	23½	.00366	.282	.906	.6406
W 65200	585000	7½	21	.00406	.293	.902	.6406
X 27000							
Z 19900	542000	11.5	28½	.00191	.2133	.9299	.6406

YSIS

Unit shear	ΣO	Web Reenfor			
		Kahn Wing B			
		Size	l'	Space Actual	Req'd
38.6	7.4	.4 x 14	13.5	12	4.3
39.2	7.7	.6 X .21	13.5	12	9.58
185.5	31.78	.7 x 25	23.0	18	4.2
95.8	14.1	6 X .25	20.2	18	3.48
186.8	29.28	.7 x .25	23.0	18	4.14
133	28.5	.7 x 25	20.2	18	5.8
120	33.4	.7 x .25 .6 x .25 .6 x 25	20.2	18	9.2
185	24.74	.7 X .25	20.2	18	4.39
83.5	18.15	.7 x .25	20.2	18	4.63
72.4	20.4	.6 x .21	20.2	18	8.85
123.5	16.4	.6 x .21	20.2	18	4.57
124.8	13.4	.6 x 21	20.2	18	4.45


℄
2'-4"
Saddle
"H
Steel Sash
13'-8½"
A
See Detail of
Air Inlet
Strut "J"
Hea
℄ Damper to
operate from flr

COLUMN STRESSES

Column	Total Load P	a_s	p	A	f_s	f_c	Considering eccentricity α	C	B	M	f_s	f_c	Total f_s	Total f_c
T+Z 6-28	67113	1.562	.01336	117	5082	484	$\frac{13500}{E}$		73.5	$322 I_c$	15214	1449	x20294	x1933
U+Y 17-18	145964	" "	.01178	133	9922	945							9922	x945
U+Y 9-16+19-25	166123	" "	" "	" "	11319	1078	$\frac{11600}{E}$	24.5	46.7	$277 I_c$	13083	1246	x24402	x1966
U+Y 28	130229	" "	" "	" "	8851	843							8851	x843
U+Y 7-8+26-27	150173	" "	" "	" "	10200	972							10200	x972
V+X 7-8+26-27	153171	" "	" "	" "	10416	992							10416	x992
V+X 9-16+19-25	139599	" "	" "	" "	9492	904	$\frac{7600}{E}$	24.5	46.7	$181 I_c$	8557	815	x18049	x1719
V+X-17-18	177683	" "	" "	" "	12075	1150	$\frac{8910}{E}$	24.5	42.1	$212 I_c$	10022	955	x22097	x2105
V+X 28	158320	" "	" "	" "	10353	986							10353	x986
V+X 9-16+19-25	124599	" "	" "	" "	8473	807	$\frac{12350}{E}$	24.5	46.7	$294 I_c$	13860	1320	x22333	x2127
W 7-27	87814	" "	" "	" "	5953	567	$\frac{1100}{E}$	24.5	73.5	$169 I_c$	7980	760	13933	x1327
T+Z 29	39773	" "	.00925	169	2184	208	$\frac{1950}{E}$ $\frac{1550}{E}$	170	65.8 61.0	$59 I_c$	5722	545	7906	x753
U+Y 29	67468	" "	.01178	133	4578	436	$\frac{2440}{E}$	24.5	88.7	$82 I_c$	3864	368	8992	x804
V+X 29	71246	" "	" "	" "	4840	461	$\frac{2440}{E}$	24.5	88.7	$82 I_c$	3864	368	8704	x829
W 29	71251	" "	" "	" "	4840	461	$\frac{5350}{E}$	24.5	50.0	$127 I_c$	6016	573	10856	x1034

Stresses marked x indicate overstress.

Plate 16

CONTENTS OF POCKET.

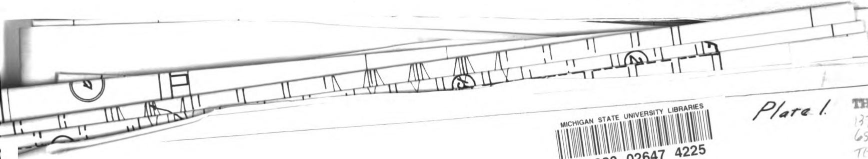


Plate 1.

…ATION DIAGRAM OF FOOTINGS
Scale- $\frac{1}{4}''=10'0''$