THESIS

ANALYSIS OF A REINFORCED CONCRETE FACTORY BUILDING

H. W. SHELDEN CHAS. PATTERSON

1917

Analysis of a Reinforced Concrete Factory Building

A Thesis Submitted to

The Faculty of

MICHIGAN AGRICULTURAL COLLEGE

By

H.W. Shelden Chas. Patterson

Candidates for the Degree of

Bachelor of Science

June, 1917

ANALYSIS OF THE REINFORCED CONCRETE FACTORY BUILDING OF THE W.K. PRUDDEN COMPANY, LANSING, MICH.

INTRODUCTION.

As a basis for this thesis the authors have various reasons for choosing the analysis of a reinforced concrete factory building, First: they are especially interested in reinforced concrete construction, Second, this parttcular type of construction (the mushroom type) is fairly new and as yet has not been very thoroughly tested. It is peculiarly adapted to large factories, warehouses and the like, where light and overhead room is essential. Therefore an attempt has been made to determine if this type of construction will stand the tests of best specifications. Thirdly, since the authors were employed in the construction of this building they were able to analyze it as it was built, and did not have to follow the plans blindly. Familiarity with the building, and aquaintance with the contractor's Supt., Mr. Groves, provides excellent promise of an effective and profitable analysis. An analysis of a purely paper design could hardly supply an equally lively interest.

-INDEX-

-oOo-

Nomeclature

M=bending moment in in. lbs.

w=uniform load/lin. ft.

l=length.

A_s=area of steel in tension.

b=breadth of rect. beam or breadth of flange of T beam.

d=distance from outer compressive fibre to c. of g. of steel.

p=ratio of area of tension to area of beam, bd.

K= " " depth of neutral axis to depth of beam, d.

J= " " distance between centers of compression

and tension steel to depth of beam, d.

f_s=tensile unit stress in steel in lbs./sq. in.

f_s=comp. " " " concrete " / " "

p'=ratio of area of steel in tension to area of beam,bd.

p"= " " " " " " comp. " " " " "

p_1=percent of steel required for desired units of $f_s$and

$f_c$for single reinforced beam.

p_2=

d'=percent of d from top of beam to compression steel.

n=E_s/E_c ratio of modulus of elasticity of steel and

concrete.

v=shearing unit stress in lbs./sq.in.

V=total shear.

u=bond unit stress in lbs./sq.in.of surface tension steel.

o=sum of perimeters of all horizontal tension steel at

section considered.

I=Total moment of inertia.

I_s= " " " " of steel reinforcing.

P=total axil load.

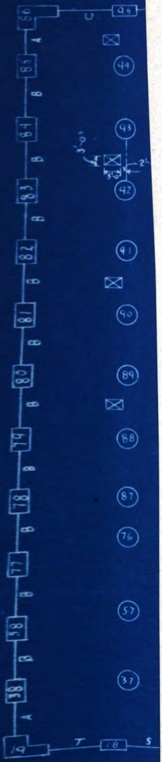
N=thrust, a component of the forces normal to the section.

A=effective area of column.

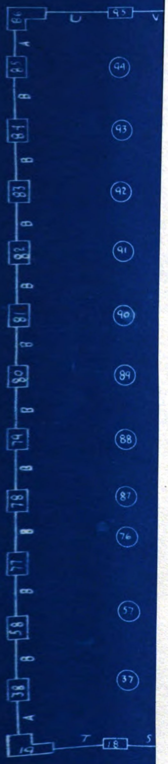
h=total depth of beam.

C=a constant.


86
U
95
85
94
84
93
83
92
82
91
81
90
80
89
79
88
78
77
58
57
38
37
19
T
18



95
86
85
84
83
82
81
80
79
78
77
58
38
19
18
A
B
C
T
S
43
42
91
90
89
88
87
76
57
37
3'-0"
2'
5'-0"



86
95
U
V
A
85
94
B
84
93
83
92
82
91
81
90
80
89
79
88
78
87
76
77
57
58
38
37
19
T
18
S

Formulas for Floor Slabs.

Oblong panels

l_1=width of floor panel.

l_2=length of floor panel.

c=diameter of column head.

Negative bending moment.

$1/17wl_2(l_1-2/3c)^2$ as bending moment parallel to width.

$1/17wl_1(l_2-2/3c)^2$ " " " " " length.

Of this moment 85% should be provided for in the col. head and the remaining 15% in the middle section.

Positive bending moment.

$1/30\ wl_2(l_1-2/3c)^2$ as moment through center parallel to length.

$1/30\ wl_1(l_2-2/3c)^2$ " " " " " to width.

Of this moment not more than 60% should be placed in outer section.

For end panels add 20% to bending moment at first interior col. head and at center of span for section for section parallel to wall.

Ro… / Sla… / 5-6-2…	Comp and C		1	2	3	4	5	…
	3.30	p	00777	00690	.00[illegible]	.00251	.00448	.00
		b	129.0	122.5	[illegible]	122.5	129.0	12
		d	6.5	6.5	6.5	6.5	6.5	6
		j	.867	.879	.911	.921	.898	.9
		K	.400	.363	.269	[illegible]	.306	.29
		f_s	15,900	18700	17400	[illegible]	8600	67
		f_c	616	710	425	[illegible]	244	1

Ro… / Sl… / 25-2…	Comp and C		1	2	3	4	5	…
	2.97	p	.00703	.00624	.00298	.00251	.004[illegible]	.00
		b	129.0	122.5	129.0	122.5	129.0	12
		d	6.5	6.5	6.6	6.6	6.6	6
		k	.367	.349	.258	.237	.283	.2
		j	.878	.864	.914	.921	.906	.9
		f_s	14500	20600	15900	16200	7900	74
		f_c	553	737	366	300	225	19

Ro… / Sl… / 17-18…	Comp and C		1	2	3	4	5	…
	3.62	p	.00634	.00703	.00240	.00331	.00446	.00
		b	156.0	145.0	142.5	139.0	142.5	13
		d	6.5	6.5	6.5	6.5	6.5	6
		k	.352	.346	.235	.269	.305	.34
		j	.883	.885	.922	.910	.898	.88
		f_s	22100	18200	20900	18000	7630	54
		f_c	783	740	423	445	223	12

Roof-	omp.						
Slab	nd C		1	2	3	4	5	6
18-19-38-	5	p	.00805	.00696	.00284	.00234	.00556	.006
		b	141.0	140.5	135.0	131.5	135.0	131.5
		d	6.5	6.5	6.5	6.5	6.5	6.5
		K	.386	.364	.253	.233	.338	.305
		j	.871	.879	.916	.922	.887	.899
		f_s	15100	19200	18650	21300	6560	665
		f_c	630	730	422	424	216	22

	omp.							
Roof	nd C		1	2	3	4	5	6
Slab	12	p	.00945	.00603	.00372	.00197	.00560	.0042
30-37-5		b	129.0	156.0	129.0	156.0	129.0	148
		d	6.5	6.5	6.5	6.5	6.5	6.
		K	.393	.344	.283	.230	.335	.301
		j	.869	.885	.906	.923	.888	.90
		f_s	16000	21500	16650	23700	7520	762
		f_c	760	765	438	400	250	21

	omp.							
1st Fl	nd C		1	2	3	4	5	6
Slab	284	p	.00956	.00761	.00415	.00262	.00541	.005
5-6-2		b	129.0	122.5	129.0	122.5	129.0	122
		d	7.0	7.0	7.0	7.0	7.0	7.
		K	.412	.378	.297	.247	.330	.32
		j	.863	.874	.901	.918	.890	.893
		f_s	31500	41500	33500	40500	17350	136
		f_c	1460	1670	937	860	670	427

1st F
S/
25-2

Comp Band C							
		1	2	3	4	5	6
2.97	P	.00735	.00608	.00325	.00262	.00375	.003
	b	129.0	122.5	129.0	122.5	129.0	122
	d	7.0	7.0	7.0	7.0	7.0	7.0
	k	.368	.345	.265	.245	.286	.27
	j	.877	.885	.912	.918	.906	.90
	fs	33,700	51000	32200	40300	20600	1940
	fc	1345	1800	863	865	540	488

3d F
S/
17-18

Comp. and C							
		1	2	3	4	5	6
3.62	P	.00722	.00700	.00376	.00360	.00414	.0037
	b	156.0	145.0	142.5	139.0	142.5	139.
	d	7.0	7.0	7.0	7.0	7.0	7.0
	k	.343	.366	.285	.279	.296	.28
	j	.876	.878	.905	.907	.901	.90
	fs	64,500	92,500	68,200	44,300	40,000	28,8
	fc	2,700	3540	1600	1140	1,060	75

1st F
S/
18-19

Comp and C							
		1	2	3	4	5	6
.073	P	.0126	.01024	.00344	.00222	.00975	.0087
	b	141.0	140.5	136.0	131.5	136.0	131.5
	d	7.0	7.0	7.0	7.0	7.0	7.0
	k	.455	.4214	.272	.227	.414	.348
	j	.848	.860	.909	.924	.862	.867
	fs	28,000	35,000	41,000	63,500	10,200	11,15
	fc	1545	1700	1040	1210	480	49

	1	2	3	4	5	6
p	.01047	.00758	.00443	00320	.00605	.00461
b	129.0	156.0	129.0	156.0	129.0	148.0
d	7.0	7.0	7.0	7.0	7.0	7.0
k	.426	.377	.304	.266	.345	.371
j	.858	.8744	.899	.911	.885	.873
fs	39,100	46,000	37,000	39600	18,400	20,100
fc	1920	1850	1080	955	645	500

Analysis of Roof Slab 44-43-63-64.

On account of the shape and supporting of this slab it was necessary to analyze it in two parts. In the analysis of eachsection we took a strip a foot wide and the length of the section long and analyzed it as a beam. One section was 20'-5" long and 9'-7" wide and the other 13'-1" by 9'4".

Section 20'-5" by 9'-7"

A_s/ft.width=5285 sq.in. w=123.77#

$M=1/12wl^2$=4300"#

p=.00677 K=.360 J=.88 f_s=17000 f_c=386

Section 13'-1"by 9'-4"

A_s/ft. width=.297 sq.in. W= 1619#

$M=1/12\ Wl^2$=21200"#

p=.00381 K=.286 J=.905 f_s=12100 f_c=322

Analysis of Second Floor Slab 44-43-63-64

This slab is 20'-5" by 9'-7"and was analyzed in the same way as the corresponding one on the roof.

A_s/ft.width=.505 sq.in. w=129#

$M=1/12\ wl^2$=53700"#

p=.006 K=.345 J=.882 f_s=17200 f_c=595

Sample Computations.

Bending moment floor slabs

Neg. Moment

$1/17x350x23.33(20.42-3.66)^2x12=1613887"\#$

$20\%=1936700"\#$

$1/17x350x20.42(23.3303.66)^2x12=1946000"\#$

Pos. moment

$1/30x350x23.33(20.42-3.66)^2x12=914536"\#$

$1/30x350x20.42(23.33-3.66)^2x12=1103000"\#$

$-20\%=1323600"\#$

For either floor slabs or beams.

M=1646300"#

p=8.64/903=.oo956

$K=\sqrt{(30x.00956)+(oo956x15)}-(00956x15)=.4116$

J=1-.4116/3=.863

$f_s=1646200/8.64x.863x7=31500\#/sq.in.$

$f_c=2x1646200/6321x.863x.412=1460\#/sq.\ in.$

Bending Mom for Beams.

$M=\frac{wl^2}{12}=11650\#\ 332\#x23.33')23.33'\frac{x12}{12}=452000$

11650=total load on beam

332=wt.per lin.ft. of beam

23.33=length of beam.

Formulas for Beams

$M=1/12wl^2$ for interior continuous beams.

$M=1/10wl^2$ for end continuous beams.

$p=A_s/bd$

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

$J=1-K/3$

$f_s=M/A_sJd$

$f_c=2M/bd^2JK$

Beams with Steel in Top and Bottom.

$p'=p_1\ p_2$

$p_2=p''(K-d')/(1-K)$

$M=M_1\ M_2$

$M_1/f_sbd^2=p_1J_1$

$M_2/f_sbd^2=p_2(1-d')$

$f_c=f_s/n \times k/(1-K)$

$v=V/bJd$

$u=V/\Sigma oJd$

Bc
A
B
C
D
E
F
G
H

BO
A
B
C
D
E
F
G
H

L
N
N
C
F
6

L

M

M

O

F

G

Be
R
S
T
U
V
W
X
Y
Z
1
2
3
4
6

Estimated Maximum Live Floor Load

Maxium load to consist of autotruck wheels at 125 lbs. per wheel.

There are to be 10 wheels in a pile and each 30" wheel covers 5 sq.ft. of floor space.

Maxium live load=125 x10/5=250 lbs. per sq.ft.

Analysis of Retaining Wall.

A section between two of the columns was tested. This section was taken as 1' wide x 9" thick x 20' long, and considered as a simple beam.

$M=1/8\ wl^2=1110''\#$

$p=002$ $K=.194$ $J=.93$ $f_s=6630$ $f_c=152$

Formulas for Columns.

$f_c = P/A\ (n-1)A_s$

$f_c - N/A\ (n-1)A_s\ M/I\ (n-1)l_s$

$I\ (n-1)I_s = bh^3/12\ (n-1)pbha^3$

$M = C_F aP$

$p = A_s/bd$

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

$J = 1-K/3$

$f_s = M/A_s Jd$

$v = V/bJd$

$u = v/EoJd$

Shear at Col. Head					Punching Shear				
Col. Head					Col. Head				
ia.	Circum.	t	ct	v	Dia.	Circum.	t	ct	u
-6" □	38'	7.5"	3420	14.3	5.5'	17.3'	13.0"	2700	25
6" □	38'	8.0	3650	39.7	5.5'	17.3	13.5'	2800	67
-6" □	38'	8.0	3650	39.7	5.5	17.3	13.5	2800	67
-6" □	38'	8.0	3650	39.7	5.5	17.3	13.5	2800	67
-0" □	16'	24.0	4608	23.0	4'-0" □	16.0	24.0	4608	110
-6" □	38'	7.5	3420	14.3	5.5'	17.3'	13.0"	2700	26
6" □	38	8.0	3650	39.7	5.5'	17.3	13.5	2800	67
6" □	38	8.0	3650	39.7	5.5	17.3	13.5	2800	67
-6" □	38	8.0	3650	39.7	5.5	17.3	13.6	2800	67
-6 □	14	24.0	4030	41.0	3.6"	14.0	24.0	4030	130
5'x8.75'	33'	7.5"	2970	17.7	4.25	13.37'	13.0	2090	32.4
5x8.75	33'	8.0	3170	49.0	4.25	13.37'	13.6	2170	86.5
5x8.75	33'	8.0	3170	49.0	4.25	13.37	13.5	2170	86.5
5x8.75	33'	8.0	3170	49.0	4.25	13.37	13.5	2170	86.5
'-8" □	14.67'	24.0	4230	17.3	3.67	14.67	24.0	4230	79.5
					29x23	56	7.5	420	91.0
					29x23	56	8.0	448	123.0
					29x23	56	8.0	448	123.0
					29x23	56	8.0	448	123.0
					44x34	13'	19.0	2960	89.6
					29x23	56	7.5	420	91.0
					29x23	56	8.0	448	123.0
					29x23	56	8.0	448	123.0
					29x23	56	8.0	448	123.0
					44x34	13'	19.0	2960	89.6

Analysis of Stairs.

Consider the stairs as a beam whose length is equal to the horizontal projection. When they rested on stringers one of the stringers was analyzed. When they were poured as a slab a section one foot wide was analyzed.

Stairs Col. 43-64

Basement to first floor.

Length=6'-1" Total w/ft. width=228#

M=12700"# A_s/ft.width=.368 in. sq.

p=.0041 K=.294 J=.902 f_s=50000 f_c=140

First floor to landing.

Length=10' Total w/ft. width=228#

M=34200"# A /ft. width=.546 in.sq.

p=.00607 K=.345 J=.885 f_s=9400 f_c=330

Stairs Col.15-54.

Stairs rest on two stringers.

Basement to first floor.

Length=14.5' Total w=353# M=111200"# A_s=.638 in.sq.

p=.00709 K=.367 J=.878 f_s=26200 f_c=1020

First to second floor.

Length=16.5' W=12841# M=320000"# A_s=3.16 in.sq.

p=.0439 K=.508 J=.831 f_s=13500 f_c=2340

Entrance Stairs.

Length=9' Total w/ft.width=145#

M=20000"# A /ft.width=.334 sq.in.

p=.0039 K=.289 J=.903 f_s=13300 f_c=254

SUMMARY AND CONCLUSIONS.

--oOo--

In this analysis we have tried to deal directly with the building as we found it, than try to find out the specifications for which it was designed. Being somewhat familiar with the actual construction we were able to analyze some parts as they exist, which appear to be different from the original design. But we are not familiar with all points of the construction. There may have been some addition to the reinforcing, which, if it had been introduced would have reduced the apparent over-stress. But as we had the very latest plans to work from, we had to analze the various parts as they were given.

In some instances we have had to take values that were more or less guess work. The irregular panels and the beams supporting stairs and elevator were especially hard to figure. One of the elevator beams for instance, had ten different loads coming to it. varying from full uniform to concentrated. It is very hard to tell just what part of the load of a panel, having beams on three sides and the reinforcing running into all of them, will be carried by each beam. Only by actual measurements can the stresses in some of these beams be determined. Some of the beams which we figured the moment for as simple beams, were fixed

to a certain degree. How much we could not determine. Also the formula for reinforced concrete of this type are more or less empirical and hard to adapt to special cases.

Although this type of building construction (the mushroom type) has been in use for may years, it has not been very thoroughly investigated. Very few complete tests have been made on buildings of this type.

For our work we have used the findings of the Joint Committee and "Taylor and Thompson" text on reinforced concrete which give the very latest formula for testing such structures. The values which we used are considered the best and most conservative ones in use at the present time.

Our values show a decided over-stress in the steel and a lesser but still high value for compression of concrete. In the floor slabs the compression in the concrete at the column head was very high, but some of this is taken care of by the drop panel which we did not consider in our calculations. The stress in the concrete at other points was not much above the conservative value. The negative steel at the column heads and the positive steel in the rectangular bands was found to be very inadequate. The slabs showed a very poor balance in the placing of the reinforcing steel. Likewise the practice of carrying all of the steel to the top at the column head is not to

be commended.

The beams are over-stressed 50 to 200 percent in both concrete and steel. There is a very decided lack of negative steel over the supports for the continuous beams, and the stresses in both concrete and steel at the center of the beam are far above the safe values of 16,000 for f_s and 650 for f_c which are recommended by most authorities.

In fact according to our figures, the whole structures shows very high stresses and a very decided lack of reinforcing steel. The beams which show the greatest over-stresses, though, are partially supported by the steel sash windows, and some of them are supported in whole or part by brick walls. Taking some of these points into consideration, would tend to make the building safer by lowering the stresses in both steel and concrete. These beams should have been designed so as to carry their load without any support from underneath. Therefore in our analysis, we were not allowed to consider these points.

Of the whole building, the columns and stairs alone, seem to have been designed to carry their load with perfect safty. In fact the analysis has proven very disappointing. The building seemed to be an ideal construction and very simple to build, but unless there is something decidely wrong with out work it can not be considered safe with more than 100 or 150 lbs. live load on its floors.

d.mc
7/98