ARTHUR ERNEST BAYLISS

THESIS

ANALYSIS OF A REINFORCED CONCRETE SCHOOL BUILDING

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1917

Copil

Analysis of a Reinforced Concrete School 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, 1917.

THESIS

Cop. 1

ANALYSIS OF THE REINFORCED CONCRETE GRAMMAR SCHOOL BUILDING AT OWOSSO,MICHIGAN.

INTRODUCTION.

The authors as a basis for thos thesis have various reasons for selecting the analysis of a reinforced concrete building. First:there is a large field for this type of construction, and they are especially interested in it. Second:they have had some experience along this line of work, and expect to specialize in concrete construction. Third:the Kahn reinforcing used in this building is practically new and affords excellent data for investigation. Therefore an effort has been made to determine if it is a type of construction which will stand the tests of the best specifications.

This building was constructed in the year 1915, and the authors were not able to analyze it as it was being built. So, in this investigation the plans, which were loaned by the courtesy of Mr. S.D.Butterworth, of Lansing, Mich.,who is the Architect of the building, were carefully followed.

INDEX.

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Nomeclature.

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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 steel 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=tension unit stress in steel in lbs/sq.in.

f_c=comp. " " " concrete in lbs/sq.in.

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

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

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 of tension steel.

ΣP=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.

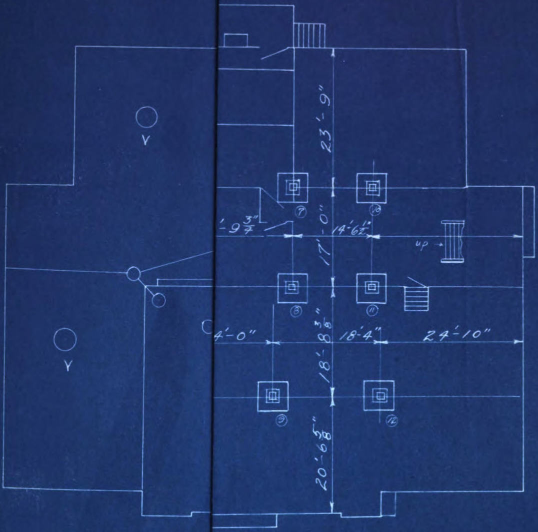
h=total depth of beam.

C= a constant.

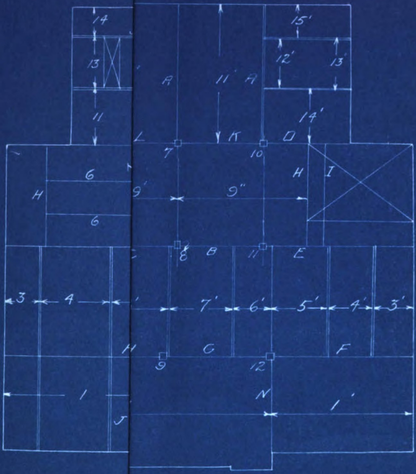
L=Live Load.

D=Dead Load.

S=Snow Load.



ENT PLAN.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G
F
12
1
N
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J

FLOOR

Formulas for Beams.

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

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

$p=A_s/bd$.

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

$J=1-k/3$.

$f_s=M/A_sJd$

$f_c=2M/d^2bJk$

Beams with steel in top and bottom.

—o—

$p'=p_1+p_2$

$p_2=p''(k-d')/(1-k)$.

$M=M_1+M_2$.

$M_1/f_sbd^2=p_1J_1$.

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

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

$v=V/bjd$.

$u=V/EPJd$.

Formulas for T Beams.

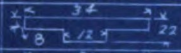
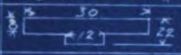
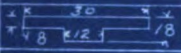
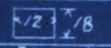
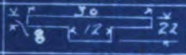
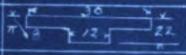
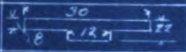
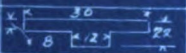
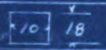
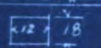
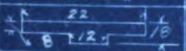
$kd= 2ndA_s + bt^2/2nA_s + 2bt$.

$z= 3kd - 2t/2kd - t \times t/3$.

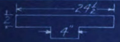
$jd= d-z$.

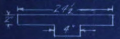
$f_s=M/A_sjd$.

$f_c=Mkd/bt(kd-1/2t)jd$.

Beam	Shape and Loading	W/D lin.ft.	W/L lin.ft.	M=W/ C	jd	Stirrups used	Stirrups needed	size	spacing	ΣP	u	v
A		1450	1330	2210	18.41		yes	Kahn-bar used		14	124	145
B		1600	1105	8600	15.82		"	"	"	12	103	106
								"	"			
C		1938	1428	1190	15.08		"	"	"	13	161	176
D		3650	1975	5700	14.85		"	"	"	6	250	125
E		1563	1350	2910	18.90					14.06	117	142
							"	"	"			
F		1850	1330	2220	18.40					14.06	147	173
G		2020	1540	1195	15.37		"	"		13.0	163	126
H		2360	1540	7650	16.234					12.0	139	140
I		1650	1200	1233	14.12					7.0	245	143
K		2320	2730	556	14.25					12.06	126	128
L		1170	1170	655	15.185							
										12	96	95

Beam	Shape and Loading	W/□ lin. ft.	W/L lin. ft.	M= w	Jd	Stirrups used	needed	size	spacing	Σ ο	u	v
A	34, 12, 22	1450	1010	1950	18.56		yes	Kahn bar used		14	108	127
B	30, 12, 18	1560	1065	848	15.28		"	"	"	12	213	212
C	36, 12, 18	1410	1070	1850	15.26		"	"	"	17	82	116
D	12, 18	1610	1205	315	15.00		"	"	"	6	132	66.3
E	30, 12, 22	1560	1216	282	18.40		"	"	"	14.12	115	136.0
F	30, 12, 22	1380	1053	1675								
					18.40		"	"	"	14.06	113	158
G	30, 12, 18	1360	1111	832	15.385		"	"	"	13	146	159
H	30, 12, 18	1385	1141	496	15.385		"	"	"	12.0	93	129
I	10, 18	661	450	573	14.76		"	"	"	7.0	108	63.2
K	30, 12, 18	1765	1200	628								
					15.00		"	"	"	16.0	89	14.3
L	38, 12, 18	1250	510	617								
					15.81		"	"	"	16.06	65	86.1
M	8, 16	439		234	12.45							
N	8, 16	439		179	12.45		"	"	"	7.0	205	38.0

Beam and Shape	Sec.	W/a lin.ft.	W/L lin.ft.	l	z	kd	jd	Stirrups used	Stirrups needed	ΣP	u	v
	1	250	83.5	16'-6"	.750	2.33	6.25	Kahn Bars Used		4.5	97	110
″	2	135.0	164	19'-6"	.810	2.77	8.29	″	yes	4.5	78	188
″	3	102.0	164	18'-8 3/8"	.748	2.33	6.252	″	″	4.5	88	98.5
″	4	253	163	16'-6"	.748	2.33	6.252	″	″	4.5	77	87.5
″	5	253	120	18'-8 3/8"	.748	2.33	6.252	″	″	4.5	124	138
″	6	102.0	163	16'-6"	.748	2.33	6.252	″	″	4.5	78	87.5
″	7	120.0	164	18'-8 3/8"	.748	2.33	6.252	″	″	4.5	94	94.5
″	8	120.0	163	17'-0"	.748	2.33	6.252	″	″	4.5	86	96.0
″	9	136	163	18'-9 3/4"	.748	2.33	6.252	″	″	4.5	100	112
″	10	102	163	14'-6 1/2"	.290	1.471	6.71	″	″	3.5	82	77
″	11	183	163	16'-6"	.748	2.33	6.252	″	″	4.5	95	108.5
″	12	102	163	9'-3"	.748	2.33	6.252	″	″	4.5	44	49
″	13	102	163	5'-0"	.495	1.665	6.505	″	″	2.0	51	25.5
″	14	102	163	15'-6"	.748	2.33	6.252	″	″	4.5	73	80.5
″	15	120	163	12'-0"	.748	2.33	6.252	″	″	4.5	83	96

Beam and Shape	Sec.	W/B lin.ft.	W/D lin.ft	W	l	z	kd	jd	Stirrups used	Stirrups needed	Σp	u	v
	1'	102	123		16'-6"	.748	2.33	6.252	Kahn-Bar		4.5	99.5	112
"	2'	114	123		19'-6"	.810	2.77	8.79	"	Used yes	4.5	69	69.5
"	3'	102	123		18'-6"	.748	2.33	6.252	"	"	4.5	71.5	80.5
"	4'	102	123		16'-6"	.748	2.33	6.252	"	"	4.5	74.5	84
"	5'	102	123		18'-6$\frac{3}{8}$"	.748	2.33	6.252	"	"	4.5	74.5	84
"	6'	102	123		18'-8$\frac{3}{8}$"	.748	2.33	6.252	"	"	4.5	93	104
"	7'	157	123		16'-6"	.748	2.33	6.252	"	"	4.5	74.5	84
"	8'	102	123		18'-8$\frac{3}{8}$"	.748	2.33	6.252	"	"	4.5	74.5	84
"	9'	120	123		17'-00"	.748	2.33	6.252	"	"	4.5	73.5	83
"	9"	120	123		17'-00"	.748	2.33	6.252	"	"	4.5	73.5	83
"	10'	134	123		18'-9$\frac{3}{4}$"	.748	2.33	6.252	"	"	4.5	85.5	96.7
"	11'	102	123		14'-6$\frac{1}{2}$"	.748	1.865 2.33	6.383	"	"	3.5	73	64
"	12'	102	123		9'-3"	.748	2.33 1.865	6.252	"	"	4.5	37	141.7
"	13'	102	123		5'-0"	.512	1.760	6.438	"	"	2.5	39	19.4
"	14'	144	123		15'-6"	.748	2.33	6.252	"	"	4.5	73	82.5
"	15'	102	123		15'-6"	.748	2.33	6.252	"	"	4.5	62	70

Analysis of Stairs.

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Consider the stairs as a beam whose length is equal to the horizontal projection. Sections one foot and fifteen inches were analyzed. The live load used was seventy lbs. per sq.ft.

Formulae used on stairs.

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$M = wl^2/8$

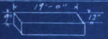
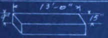
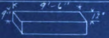
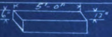
$p = A_s/bd.$

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

$J = 1 - k/3.$

$f_s = M/A_s J d.$

$f_c = 2M/bd^2 Jk.$

Shape	l	W/□	/	k	f_s	f_c
	19'-0"	150	68	.497	11330	740
	13'-0"	89	88	.418	9900	325
	9'-6"	100	'5	.311	12000	550
	5'-0"	100	5	.311	2760	121

Analysis of Columns and Footings.

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In the construction of the building there were twelve columns used, only two extending to the roof. The analysis includes one of each case.

Formulae used on Columns.

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

$f_c = N/A + (n-1)A_s + M/I + (n-1)I_s.$

$I + (n-1)I_s = bh^3/12 + (n-1)pbha^2.$

$M = CaP.$

$p = A_s/bd.$

$k = 2pn - (pn)^2 - pn.$

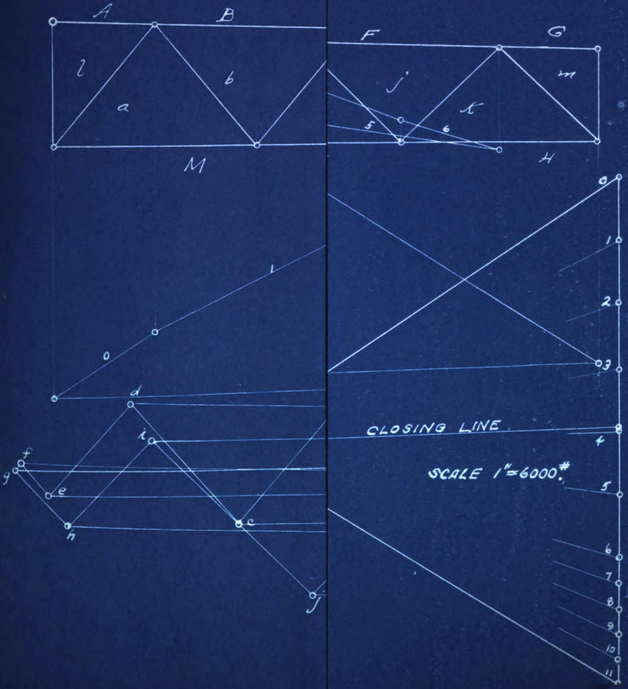
$J = 1 - k/3.$

$v = V/bjd.$

$u = v/EPJd.$

Column	Location	Wt. Truss Girder	M	$\frac{F}{A_t}$	$I_c+(n-1)I_s$	$\frac{Mc}{I}$	Max fc
5	Roof	13800	900000	340	3665	242	340
	2nd Floor		1016500	650	3000	330	650
	1st Floor		947000	640	5180	180	640
	Footing			260			
3	2nd Floor		125300	365	4050	310	365
	1st Floor		1492500	620	5160	290	620
	Footing			280			

S/S



A
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h
c
j
CLOSING LINE.
SCALE 1"=6000.#

Member	D + S	Length	/o Rivets	Value one rivet	Value rivet conn.	Size rivet
IM	25 + 30	4'-0"	3-2			$\frac{3}{4}$
I-A-B-C-D-E-F-G		3'-3.15"	2-3	4220	8440	"
Ia	Snow Load: 30	4-3"	4-4	4220	16880	"
ab	Dead Load: 25	4-2.5"	3-3	4220	12660	"
A-M-L-K-I-J-H	Ceiling " ": 20	6'-6.2"	2-3	4220	12660	"
bB	Upper P.P.: 4.0	6'-6.3"	3-3	4220	12660	"
cL	Lower " ": 1.7	6'-6.2"	3-3	4220	12660	"
bc		4'-.5"	3-3	3310	9660	"
cd		4'-1"	3-3	4220	12930	"
dC		6'-6.3"	3-2	3310	8440	"
dc		3'-10.5"	2-2	4220	6620	"
eK		6'-6.3"	3-3	3310	8440	"
ef		3'-10"	2-2	4220	6620	"
fD		6'-6.3"	2-3	3310	6660	"
fg		3'-8.5"	3-3	4220	12630	"
gJ		6'-6.2"	3-3	3310	9660	"
gh		3'-8"	3-3	4220	9930	"
hE		6'-6.3"	3-3	3310	12660	"
hi		3'-6"	3-3	4220	12660	"
iI		6'-6.2"	3-3	4220	12660	"
iJ		3'-6.5"	3-3	4220	12660	"
JF		6'-6.3"	3-3	4220	12660	"
J'K		3'-4.5"	3-4	4220	12660	"
KH		6'-6.2"	4-4	4220	12880	"
KM		3'-3.15"	3-2	4220	8440	"
MG		3'-4"	3-2			
MH		3'-00"				

Sample Computations.

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Bending moment for Beams.

Beam C- Continuous beam 1st. floor.

$M=wl^2/12. = 63300 \times 18.812 \times 12/12 = 1,190,000.$

$p=A_s/bd = 3.945/361= .01091.$

$kd=2x15x17x3.945 + 30 \times (8)^2/2 \times 15 \times 3.945 + 2 \times 30 \times 8=6.56$

$z= 3x6.56 - 2x8/2x6.56 - 8 \quad x \quad 8/3= 1.92$

$jd=(d-z)= 17-1.92=15.08$

$f_s=M/A_s jd=1,190,000/3.945 \times 15.08=20,000$

$f_c=1,190,000 \times 6.56/ 30 \times 8(6.56 - 1/2x8)15.08=845$

Negative Moment for same beam.

$k= .378$(from chart)

$J=1-.378/3=.881$

$f_s=1,190,000/12x(17)^2 -(.00659 \times .881)+ .00443 \times (1-.1)-$

$= 35,100$

$f_c= 35,100/15x \; ,378/1-.378= 1,300.$

Concrete Beams supporting floortyle are analyzed the same as the above beams.

Beam D- Simple beam 1st. floor.

$M=wl^2/8= 44,584 \times 8.5 \times 12/8=570000$"#

$p=1.58/18 \times 12=.00775.$

$k=\sqrt{2x15x.00775 + (15x0077)^2} - (15 \times .0077) = .380$

$Jd=17 \times (1-.380/3)= 14.85$

$f_s=M/A_s jd= 570000/1.58 \times 14.85=24,300$

$f_c= 2 \times 570000/17 \times 12 \times 14.85 \times .380=985.$

Stairs

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Hor. projection.= 19400"

$M=wl^2/8=4180 \times 19 \times 12/8=1,19300$

$p=1.58/12 \times 8=.0165.$

$k=\sqrt{2 \times 15 \times .0165+(.0165 \times 15)^2}-.0165 \times 15=.497$

$jd=8(1-.497/3)=6.68$

$f_s= 119300/1.58 \times 6.68= 113300$

$f_c= 2 \times 119300/12 \times 8(6.68 \times .497)=740$

Columns.

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Col.#5 second floor.

A_s=6 bars x 5/8"= 2.3436

$A_c=11 \times 11=121$

$p=2.3436/121=.0194$

Area to be added= 141 x .0194 x 144=39.2

Total area of concrete = 144 + 39.2= 183.2

P/A_t= 119,665/183.2=650#

In analyzing the columns sections were taken between the floors.

References.

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For references the authors had excess to the following: "Taylor & Thompson", "Turneure & Morse", "Ketchums Structural Handbook", and "Kahn Building Specifications". Special acknowledgment is due to Professors H. K. Vedder, and C. A. Melick.

SUMMARY AND CONCLUSIONS.

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In this analysis we did not deal directly with the building as we found it, but tried to find out the specifications for which it was designed. Not being familiar with the actual construction, we were not able to analyze some parts as they exist. But as we had the actual plans to work from, we had to analyze the various parts as they were shown.

Not being able to obtain the roof plans, we found it necessary, to measure the members of the roof truss, and estimate values to a more or less degree. As a whole the truss was found to be amply safe, although occasionally a member was found to be slightly overstressed. Each of the three trusses supporting the roof carried a uniform load, there being no concentrated loads on them. The graphical analysis used in finding the stresses in the members proved the truss safe for imposed loading.

The second floor beams on the average were found to be within the safe values of 16,000 for f_s and 650 for f_c, which are recommended by most authorities. Occasionally a decided overstress in the steel as well as in the concrete was found occuring at the points of negative bending moment, at the supports. However, in actual construction this overstress may have been taken care of by additional reinforcement. For instance Beams E on the first and second floors, according to our figures shows the largest overstress at the supports. This shows a very poor point in the design.

On the first floor the beams were stressed over the conservative values stated above especially so at the supports, as was found on the second floor. The beams are overstressed 30- to 35% in both concrete and steel. This shows a decided lack of steel over the supports for continuous beams. As previously stated in the construction extra steel may have been used, as our analysis adhered to the data on the plans.

The concrete beams supporting floors were stressed above the conservative values at the supports, but at the centers averaged within the allowed unit. The concrete beams supporting the floors of sections 1 to 9 inclusive, also 11 and 14, on both floors are decidedly overstressed, indicating poor design. These like the floor beams show a lack of negative steel over the supports.

According to our figures the whole structure is fairly well balanced, although there seems a lack of reinforcing steel in some beams. The beams which present the largest over-stress are supported wholly or in part by tile walls. Taking some of these points into consideration tends to lower the stresses in the concrete and steel. The stairs and columns are well designed and carry their live load well within the allowed units. The building as a whole has been well designed, and can be considered safe for its live load. The building represents a neat and strong appearance and is absolutely fireproof.

For analysis we have used "Taylor and Thompson" text on reinforced concrete and the findings of the "Joint Committee". These references afford the very latest formulae for testing structures and are the best in use at the present time.

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