DESIGN OF NORTH LOGAN
STREET BRIDGE

THESIS FOR THE DEGREE OF B. S.

CARL HAUSSMAN, JR.
1939

THESIS

C 2

Design of North Logan
Street Bridge

A Thesis Submitted to

The Faculty of

Michigan State College

of

Agriculture and Applied Science

by

Carl Haussman, Jr.

Canidate for the Degree of

Bachelor of Science

June 1939

THESIS

cop. 1

Acknowledgement

The author wishes to thank Professor C.A.Miller and Mr. Collins Thornton for their help in the preparation of this thesis.

In the City of Lansing there is only one bridge that is available for all traffic crossing the Grand River enroute to Grand Rapids and other places in the northwest of the State. This bridge, on U.S.16, must carry all Lansing and through traffic. Although it can, and does carry all traffic, such a situation is far from desirable due to the fact that many motorists must travel a long circuitous route in order to cross the bridge.

The City of Lansing contemplates building another bridge in the future that will alleviate the conditions as they now are. This thesis is concerning the design of such a bridge which shall be known as the North Logan Street Bridge.

Such a bridge would not only offer a cut off for Lansing traffic going to the northwest part of the State, but would also give a through route for traffic from the small towns south of Lansing that come comes into the City on South Logan Street.

North Logan Street comes to a dead end a distance from the river and due to existing property it is not feasible to continue it; however, what this thesis purposes is a jog in the route over to Alice Street which does run to the river bank. Alice Street would be available for building an approach to the bridge proper.

There are several conditions at the Alice Street location that must be considered before the bridge can be designed. What is desired is this: A bridge that will offer a direct route from Alice Street over to U.S.16 to the north. If this is accomplished the bridge will have to cross the river, and in addition, clear the Consumers Power Railway siding. The South bank of the river is twenty feet higher than the north bank so the bridge will probably have to be built on a grade.

Also it would be desirable to build the bridge so that it is perpendicular to the flow of the river.

A reinforced concrete open spandrel arch was deemed best for this location, but due to the many qualifying conditions there were many possibilities as far as types of arches were concerned. After many trials a bridge was adopted using 5 - 100 foot arches. The arch ring was to be made up of four separate rings. Instead of trying to take up the difference in grade between the two banks in the arches it was found better to build the bridge on level grade and fill up to this grade on the north side of the river.

A good deal of the fill necessary on the north side could be obtained from earth which would have to be cut between the bridge and U.S.16. A road would have to be built between these points and an attempt was made to find a grade line that would allow just as much cut as possible in the roadway which would find use as fill near the bridge.

This type of bridge will necessitate a viaduct approach on the south side which will have to be built in a slight curve. However, it was thought better to have the curve on the approach than on the bridge proper, thus simplifying problems of design and construction on the arches. The viaduct design was not considered in this thesis.

Because of the nature of this work the material consists mainly of computations. As all arches are the same the work is shown for designing one of them, the method of least work was used in designing these arches. The floor scheme of beams and girders is a typical one used by the City of Lansing in their design of the South Logan Street Bridge. A gravity type pier was designed and computations are shown for the north abutment.

It might be stated that if the bridge as here shown were to be used it would be possible to use a thinner arch ring than used by the author due to two reasons. (1) The final stress as shown are rather low, (2) A concrete railing was first contemplated which was later changed to a lighter steel one.

The starting point for the design is the use of the crown thickness formula, where dc crown thickness in inches, L clear span in feet, WL live load in pounds per square foot, WC dead load at crown in pounds per square foot. For safety's sake the author added 2" to this thickness which apparently was unnecessary.

The clearance over the railroad offered by this bridge will be the 19 feet as specified for this type of track; however, if desired the track can be lowered a few feet as is evidenced on the general site plan.

Due to the size of this structure the piers and abutments are to go down to bed rock which is at an elevation of 85.

The roadway is to be 9 inches of concrete with a 3 inch asphalt topping.

Circular segmental arches were used and reinforcing steel is used in the top and bottom.

The live loading assumed was 200 pounds per square foot. 2500# concrete was used with n = 12.

Reference books used are "Reinforced Concrete Construction" Vol.3, by Hool, "Reinforced Concrete Structures" by Peabody, "Design of Viaduct for South Logan Street" by Thornton, "Design of Open Spandrel Reinforced Concrete Arch" by Cooke and "Reinforced Concrete Design", by Sutherland and Clifford.

In some computations several trials were necessary but only the final results are shown.

The following sheets contain the computations for the North Logan Street Bridge

Design of Arch.

$$dc = \sqrt{L} + \frac{L}{10} + \frac{W_L}{200} + \frac{Wc}{400}$$

$L = 100'$

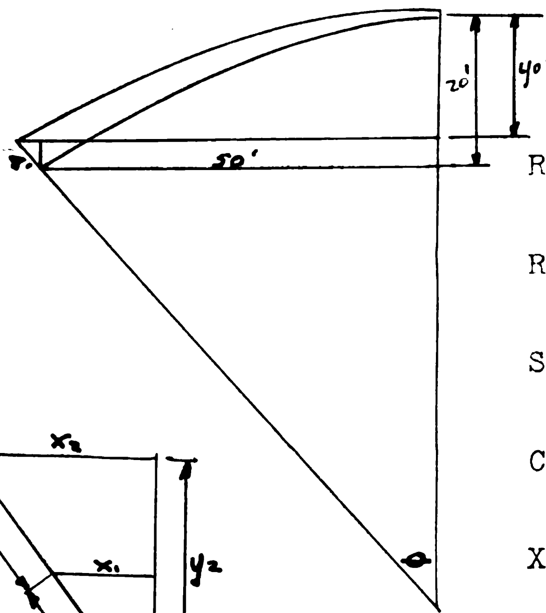
$$dc = \sqrt{100} + \frac{100}{10} + \frac{200}{200} + \frac{150}{400}$$

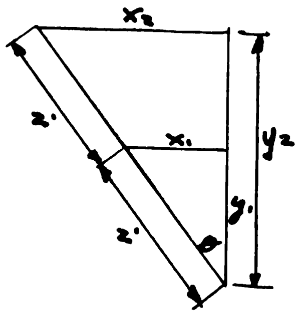
$Wc = 150$ lbs/sq.ft.
$W_L = 200$ lbs/sq.ft.

$dc = 22''$

Let crown thickness = 2'

Let springing thickness = 4'





$$R = \frac{X^2 + Y^2}{2Y}$$

$$R_1 = \frac{(50)^2 + (19)^2}{2(19)} = 75.2'$$

$$\sin\theta = \frac{50}{75.2}$$

$$\cos\theta = \frac{56.2}{75.2}$$

$X_1 = 2 \sin\theta = 1.33$

$Y_1 = 2 \cos\theta = 1.49$

$$R_2 = \frac{(51.35)^2 + (18.51)^2}{2(18.51)} = 80'$$

$$R_3 = \frac{(52.66)^2 + (18.02)^2}{2(18.02)} = 85.5'$$

$Yo = 20 - 1.49$

$Yo = 18.51'$

Distance between center lines of columns = 9'3/32" = 9.1'

Area of road crossection = sidewalk + road + beams

Wt of road crossection = 2(7/12)(5) + 38(1) + 8(1)(1) 150

= 7,770

2 railings	1,100	(550#/ft. per rail)
	8,870#lin.ft.	

Wt. per ring = $\frac{8,870\ (9.1)}{4}$ = 20,360# (4 rings per cross sec.)

Use 1ft. sq. columns they are oversize but it is not customary to go below this size.

Add to the above weight per ring the weight of girders, columns, and arch ring.

Wt. of girder on 1 ring = $\frac{3(2)(40)(150)}{4}$ = 9,000#

total wt. per ring 29,400#

Wt.@ 6 per ring = 29,400 + 10,800 = 40,200

5 per ring = 29,400 + 3,150 = 32,450

4 per ring = 29,400 + 1,860 = 31,260

3 per ring = 29,400 + 900 = 30,300

2 per ring = 29,400 + 300 = 29,700

1 per ring = 29,400 + 300 = 29,700

total weight applied to ring per column per 1'width of ring

		ring weight		total wt.	wt. per ft. of ring
6	40,200#	(9.1)(7)(150)(3.6)	= 34,400#	74,600	10,700
5	32,450	9560(3.2)	= 30,600	63,050	9,000
4	31,260	" (2.7)	= 25,800	57,060	8,100
3	30,300	" (2.4)	= 22,900	53,200	7,600
2	29,700	" (2.1)	= 20,100	49,800	7,100
1	29,700	" (2)	= 19,100	48,800	7,000
				total	49,500#

The rings are 7' wide the loads as shown are applied thru the columns. The load at 6 acts down to the pier.

Dead Load Equilibrium Polygon

Load	Arm	Moment	Section
		0	A
A 7,000	9.1	63,700	B
13 7,100			
14,100	9.1	128,310	
		192,010	C
C 7,600			
21,700	9.1	197,470	
		389,480	D
D 8,100			
29,800	9.1	271,180	
		660,660	E
E 9,000			
38,800	9.1	353,080	
F 10,700			
49,500	1.25	61,875	
		1,075,615	Springing

Location of Reference Axis

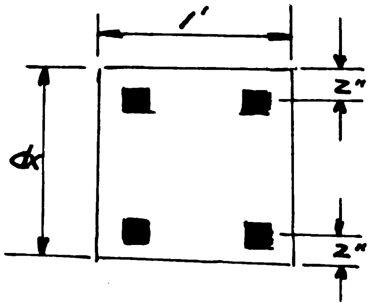
Sect.	Zo y	dx	dx3/12	dx .333	.06(d .33)	I	Zo/I	$\frac{1}{I}$
1	18.58	2.05	.6179	1.72	.1775	.7954	23.359	1.257
2	18.18	2.10	.6717	1.77	.1880	.8597	21.115	1.116
3	17.35	2.15	.7282	1.82	.1987	.9269	18.718	1.078
4	16.21	2.29	1.0008	1.96	.2305	1.2313	13.165	.812
4	14.65	2.45	1.2258	2.12	.2696	1.4954	9.796	.669
6	12.72	2.65	1.5500	2.32	.3229	1.8729	6.792	.534
7	10.45	2.88	1.9907	2.55	.3902	2.3809	4.389	.420
8	7.87	3.17	2.6546	2.84	.4840	3.1386	2.507	.319
9	4.92	3.47	3.4818	3.14	.5916	4.0734	1. 208	.245
10	1.60	3.80	4.5727	3.47	.7225	5.2952	.302	.189
							101.351	6.639

$$t = \frac{\Sigma \frac{Z_o}{I}}{\Sigma \frac{1}{I}} = \frac{101.351}{6.639} = 15.30'$$

$$y_e = r - t = 20 - y - t$$

$$y_e = 20 - 1.49 - 15.30$$

$$y_e = 3.21$$



$7/8"$ sq.bar = .785 sq.inches

4 bars = 3.14 sq. inches

$$A = \frac{dx + (n-1)(3.14)}{144} = .2399 + dx \text{ sq.ft.}$$

$$I = \frac{dx^3}{12} + .2399\left(\frac{dx}{2} - \frac{1}{6}\right)^2$$

$$= \frac{dx^3}{12} + .06(dx - .333)^2$$

Arch Constants

Sect.	X	Y = -t	X^2/I	Y^2/I	1/A
1	2.81	3.01	9.925	11.388	.436
2	8.35	2.61	27.810	7.600	.427
3	13.81	1.78	205.736	3.417	.418
4	19.30	.64	302.462	3.329	.395
5	24.72	.92	408.811	.569	.371
6	29.95	2.85	399.033	4.336	.345
7	35.06	5.12	516.266	11.008	.320
8	40.01	7.70	510.655	18.914	.293
9	44.70	10.65	489.532	27.815	.269
10	49.20	13.97	457.501	36.855	.248
			3,377.731	125.261	3.522

Moments due to dead load

M1 = 0
M2 = 7(3.80) = 26.6 kip feet

M3 = 7(9.26) + 7.1(.16) = 65.815

M4 = 7(14.75) + 7.1(5.65) = 143.365

M5 = M4 + 7.6(1.97) + 14.1(5.42) = 234.756

M6 = M5 + 21.7(5.23) = 348.250

$M7 = M6 + 21.7(5.11) + 8.1(3.21) = 485.138$

$M8 = M7 + 29.8(4.95) = 632.648$

$M9 = M8 + 29.8(4.69) + 9(3.75) = 806.060$

$M10 = M9 + 38.8(4.50) = 980.660$

Dead Stresses

Sect.	$M_L = MR$	$\frac{M + MR}{I}$	$\left(\frac{M_L + MR}{I}\right) Y$
1	0	0	0
2	26.600	59.311	154.802
3	65.815	141.897	252.577
4	143.565	232.825	149.008
5	234.759	314.108	288.979
6	348.250	371.397	1,058.481
7	485.138	407.516	2,086.482
8	532.648	403.629	3,107.943
9	806.060	394.969	4,206.420
10	980.660	370.689	5,178.525
	3,723.295	2,696.341	15,370.443

Dead Load Stresses

$$Mo = \frac{\left(\frac{M + MR}{I}\right)}{2\Sigma \frac{1}{I}} = \frac{2,696.341}{13.278} = 203.068 \text{ kip feet}$$

$$Mo = Ho = \frac{-\Sigma \left(\frac{M_L + MR}{I}\right)(Y)}{2\Sigma \frac{Y^2}{I} + 2\Sigma \frac{1}{A}} = \frac{15,370.443}{250.322 \quad F.044} = 59.723 \text{ kips}$$

$Mc = Mo - Ho\ (Yo) = +16.352$ kip feet eccentricity = e

$$e = \frac{16,357(12)}{59,723}$$

$e = +3.29"$

$Ms = -1,075,615 + 59,723(18.51) = 29,858$ ft. lbs.

$Ts = 59,723\ (\cos\theta) + 49,500\ (\sin\theta)$

$Ts = 77,562\#$

eccentricity $= \frac{29,858}{77,562}(12) = 4.62"$

Crown Stresses

$fc = \frac{P}{A} \pm \frac{Mc}{I}$

$= \frac{59.723}{322.55} \pm \frac{16.357(12)(12)}{17.274} = 321.51\#/\text{sq.in.}\ \&\ 48.5\ \frac{\#}{\text{sq.in.}}$

Springing Stresses

$fc = \frac{77.562}{610.55} \pm \frac{29.858(12)(24)}{133,350}$

$= 152.9$ lbs/sq.in and 101.1 lbs/sq.in.

Live Load Influence Table

Load at AL

Sect.	$M_L = ML + MR$ $ML - MR$	$(ML + MR)\frac{1}{I}$	$(ML - MR)\frac{X}{I}$	$(ML + MR)\frac{Y}{I}$
2	3.80	4.777	59.89	12.48
3	9.26	9.823	135.66	17.48
4	14.75	11.977	231.16	7.67
5	20.17	13.494	333.57	− 12.41
6	25.40	13.564	406.24	− 38.66
7	30.50	12.814	449.26	− 65.61
8	35.46	11.312	452.59	− 87.10
9	40.15	9.837	439.71	− 104.76
10	44.65	8.439	415.20	− 117.89
		96,037	2,903.28	− 388.80

$$Mo = \frac{96.037}{13.278} = 7.23$$

$$Vo = \frac{2903.28}{6,755.46} = .430$$

$$Ho = -\frac{388.80}{257.36} = -1.511$$

Load at B_L

3	.16	.172	2.38	.31
4	5.65	4.588	88.55	2.94
5	11.07	7.406	183.08	− 6.81
6	16.50	8.704	260.68	−24.81
7	21.41	8.992	315.26	− 46.04
8	26.36	8.409	336.44	− 64.75
9	31.05	7.607	340.03	− 81.01
10	35.55	6.719	330.57	− 93.86
		52.597	1,856.99	− 314.03

$$Mo = \frac{52.597}{13.278} = 3.961$$

$$Vo = \frac{1,856.99}{6,755.46} = .275$$

$$Ho = -\frac{314.03}{257.36} = -1.220$$

Load at C_L

Sect.	ML = ML+MR = ML-MR	$(ML+MR)\frac{1}{I}$	$(ML-MR)(\frac{X}{I})$	$(ML+MR)\frac{Y}{I}$
5	1.97	1.318	32.58	− 1.21
6	7.20	3.845	115.16	− 10.96
7	12.31	5.770	181.26	− 26.47
8	17.26	5.506	220.30	− 42.40
9	21.95	5.378	240.40	− 57.28
10	26.45	4.999	295.95	− 69.84
		26.216	1,035.65	− 208.16

$$Mo = \frac{26.216}{13,278} = 1.97$$

$$Vo = \frac{1,035.65}{6755.46} = .153$$

$$Ho = -\frac{208.163}{257.36} = -.080$$

Load at D_L

7	3.21	1.348	47.26	− 6.90
8	8.16	2.603	104.15	− 10.38
9	12.85	3.148	140.72	− 33.53
10	17.35	3.279	161.33	− 45.81
		10.378	453.46	− 96.62

$$Mo = \frac{10.378}{13.278} = .782$$

$$Vo = \frac{453.46}{6,755.96} = .067$$

$$Ho = \frac{-96.62}{257.36} = -.375$$

Load at E_L

9	3.75	.919	41.08	− 9.80
10	8.25	1.559	76.70	− 21.78
		2,478	117.78	− 31.58

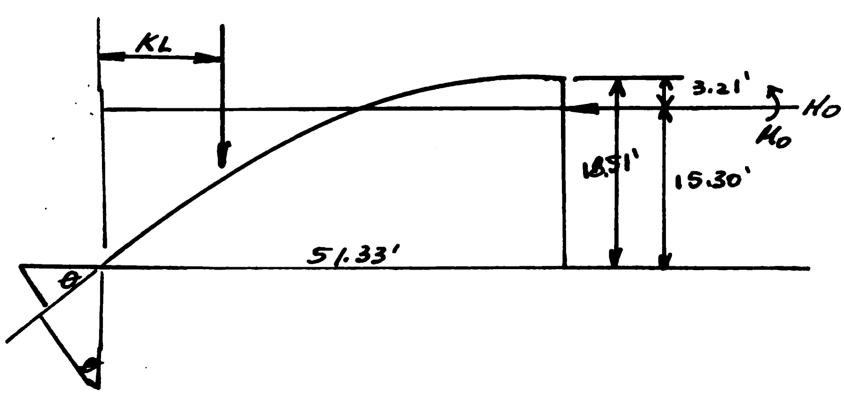
$$Mo = \frac{2.478}{13,278} = .187$$

$$Vo = \frac{117.78}{6,755.46} = .017$$

$$Ho = \frac{-31.58}{257.36} = -.123$$

Influence table

	Unit load at				
	A L	B L	C L	D L	E L
Mo	7.23	3.961	1.970	.782	.187
Ho	1.511	1.220	.808	.375	.123
Vo=Vr	.430	.275	.153	.067	.017
$V_L = 1 - V_o$	.570	.725	.847	.933	.983
-(3.21)Ho	-4.850	-3.916	-2.594	-1.204	-.395
M crown	+ 2.380	+.045	-.624	-.422	-.208
15.3(Ho)	23.118	18.666	12.362	5.738	1.882
51.33(Vo)	22.071	14.116	7.853	3.439	.873
Sum	52.419	36.743	22.185	9.959	2.942
KL	-46.780	-37.680	-28.580	-19.480	-10.380
Msleft	+ 5.639	-.937	-6.395	-9.521	-7.438
MsRight	+8.277	+8.511	+6.479	+3.081	+1.196
Ho(Cosθ)	1.129	.922	.614	.290	.150
V (Sinθ)	.379	.482	.564	.621	.655
tsleft	1.508	1.404	1.178	.911	.805
Ho(Cosθ)	1.129	.922	.614	.290	.150
V_R (Cosθ)	.286	.183	.102	.045	.011
Tsright	1.415	1.105	.716	.335	.161



$$\cos\theta = \frac{56.2}{75.2} = .7473$$

$$\sin\theta = \frac{50}{75.2} = .6649$$

Mc = Mo-3.21 Ho

Ms = Mo+15.3Ho+51.33(Vo)-KL

Ms right = Mo + 15.3Ho-51.33Vo

Live load = 200#/□'

Live panel load = 200(9.1) = 1820#

Maximum Live Moment and Thrust at Crown and Springing

Data from influence table

	Loading	Unit load values	Live load Stresses
Movements +	$A_L A_R B_L B_R$	4.850	8,827ft.#
At Crown --	$C_L C_R D_L D_R E_L E_R$	-2.508	-4,565ft.#
Ho with + Mc		5.462	9,941#
Ho with - Mc		2.612	4,754#
Movements at	$A_L A_R B_R C_R D_R E_R$	33.183	60,393ft.#
Springing -	$B_L C_L D_L E_L$	-24.291	-44,210ft.#
Thrust + Ms		5.240	9,537#
Thrust -Ms		4.298	7,822#

Temperature Stress 60 fall

$$Ho = \frac{\Sigma tnE}{\Sigma \frac{Y^2}{I} + \Sigma \frac{1}{A}} \qquad \frac{-000006(60)(10)(2,500,000)(144)}{128.783} = -10,063\text{lbs.}$$

Mc = Ho(3.21) = 32,302ft.#

Ts = Ho(Cos.ϕ) = -7,5[illegible]0#

Ms = Ho(15.30) = -153,964ft.#

20° Rise

Ho = 3,335
Mc = -10,767
Ms = 2,507
Ms = 51,[illegible]21

for 20 rise divide by -3 to get these values

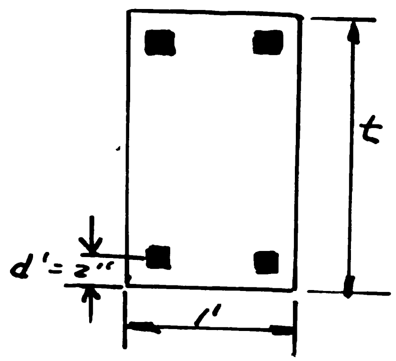
Stress Summary and Unit Stresses

Springing		Thrust Ts	Movement Ms	e	e/t	t/e	fc	fs
							in lbs/sq in	in lbs/□"
Max	Dead	77,562	29,858					
+ M	Live	9,537	60,393					
		87,099	80,251	11.1"				
	temp. rise	2507	51,321					
		89,606	131,572	17.6"				
Max	Dead	77,562	29,858					
-M	Live	7,822	-44,210					
		85,384	-14,352	-2"				
	temp. fall	-7,520	-153,964					
		77,864	-168,316	21.8"	.514	[illegible].8	562	11,600

Stress Summary & Unit Stresses

Crown		Thruse Ho	Movement Mc	e	e/t	t/e	fc	fs
							#/□"	#/□"
Max	Dead	59,723	16,357					
M	Live	9,941	8,827					
		69,664	25,184	4.35"				
	temp. fall	-10,063	32,302					
		-59,601	57,486	11.6"	.484	2.06	800	14,400
Max	Dead	59,723	16,357					
M	Live	4,754	-4,565					
		54,969	11,792	2.54"				
	temp. rise	3,354	-10,767					
		58,3[illegible]3	1,025	.24"				

Computation of Final Stresses



In computing these stresses tables in Peabody's "Reinforced Concrete Design" were used. Only the maximum values at the crown and springing were computed.

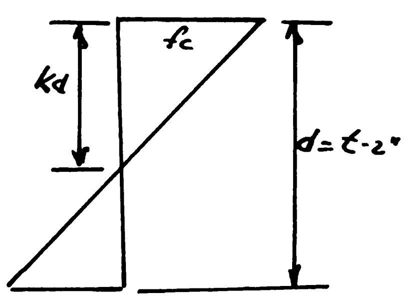
Maximum stresses at springing

$\frac{d'}{t} = \frac{2}{48} = .0416$

$p_o = \frac{A_s}{bt} = .00545 \qquad n p_o = 12(.00545) = .0655$

by plot 16 $K = .58, \quad C = 9.2$

$f_c = \frac{M}{bt^2}(C) = \frac{140{,}000(12)}{12(48)(48)}(9.2) = 560$ lbs./sq.in.



$kd = .58(46) = 26.7$

$\frac{560}{X} = \frac{26.7}{46}$

$f_s = N X = 11{,}600$ lbs./ sq.in.

Maximum Stresses at the Crown

$\frac{d'}{t} = \frac{2}{24} = .0833$

$p_o = .01190 \qquad (n-1)\, p_o = .131$

by Plot $C = 8 \quad K = .4$

$f_c = \frac{M}{bt}(C) = \frac{57500(12)}{12(24)(24)}(8) = 800$ lbs/sq.in.

$kd = .4\ (22) = 8.8$

$\frac{800}{X} = \frac{8.8}{13.2}$

$f_s = X N = 14{,}400$ lbs/sq.in.

Abutment Design

Pavement and live load surcharge $\frac{350}{120} = 2.92'$

Weight above abutment $= 26(23.92)(120) = 74,600$

$$\frac{26(24)(120)}{2} = \frac{36,400}{111,000\#}$$

Wt. of triangles of concrete

(1) $\frac{150(23.6)(4)}{2} = 10,610\#$

(2) $\frac{150(23.6)(4)}{2} = 8,490$

(3) $\frac{150(21.[illegible])(3.1)}{2} = 48,400$

(4) $\frac{150(18.3)(6.5)}{2} = \frac{8,920}{76,400\#}$

$111,000\ (a) = 74,600(13) + 36,400(\frac{26}{3})$

$a = 11.60'$

$187,400(\bar{X}) = 111,000(11.6) + 10,610(18.5) + 8,490(19.2)$

$8920(6.3) + 98,400(15.6)$

$\bar{X} = 13.10\text{feet}$

$P = 1/2\,CeWh$

$P = 1/2\left[(50.42)^2 - (23.9\ )^2\right](120)(.25)$

$\simeq 15(1930) = $ [illegible]$9{,}000$ lbs

Point of application of P

$$X = \frac{h^2 + 3hh_1}{3(h + h)} = \frac{30.5 + 3(30.5)(23.92)}{3(30.5 + 23.92)}$$

$X = 15$ feet

$$S = \frac{P}{A} \pm \frac{Mc}{I} = \frac{245{,}000}{19} \pm \frac{245{,}000(.8)(9.5)}{\frac{(19)}{12}}$$

$S = 16{,}400$ lbs./sq.ft. and $9{,}400$ lbs./sq. ft.

$$S = \frac{243{,}000}{19} \pm \frac{243{,}000(1.3)(9.5)}{\frac{(19)^3}{12}}$$

$S \simeq 18{,}050$ lbs./sq.ft. and 7550 lbs./sq.ft.

These values are not too high as the abutment goes down to a rock foundation.

[redacted]

Pocket has:
1 Plan