THESIS

LANSING ENGINEER'S PLANS FOR A BRIDGE AND GRADE CROSSING AT KALAMAZOO STREET, LANSING

Harold George Bauerle

1924

LANSING ENGINEER'S PLANS for a BRIDGE and

GRADE CROSSING at KALAMAZOO STREET, LANSING.

A thesis submitted to the FACULTY of the

DEPARTMENT of CIVIL ENGINEERING, M.A.C.

by

Harold George Bauerle

Candidate for the degree Bachelor of Science.

June 1924

S P E C I F I C A T I O N S

for

D E S I G N

of a reinforced concrete bridge over Grand River

and N.Y.C.R.R. at Kalamazoo Street.

L A N S I N G , M I C H I G A N .

J A N U A R Y 1 9 2 4

1. The design shall provide for :

   (a). Roadway - 36 feet from curb to curb.

   (b). Two sidewalks - 7 feet from curb to inside of rail.

   (c). Railing on each side - 3'-6" high.

   The design shall provide for the base for the pavement over the entire length of the bridge allowing 3" for the top surface.

2. Clearance.

   At least 18 feet vertical clearance must be allowed from top of rail over the N.Y.C. tracks and six feet side clearance from the rail.

SPECIFICATIONS

2.(a). <u>Dead Loads</u>

The dead loads shall consist of the computed weight of the material used in the construction.

In determining weights the following units will be used:

Pavement(including base)	130 #/sq. ft.
Reinforced concrete	150 #/cu. ft.
Earth Fill	100 #/ cu. ft.
Gravel and Broken Stone	150 #/cu. ft.

(b). <u>Live Load.</u>

Whichever of the following live loads as will give maximum stresses.

A. <u>Concentrated Live Load.</u>

Motor Trucks of the following dimensions and weights, occupying as much of the roadway as is necessary for maximum stresses :

Total weight	40,000 lbs.
Weight on rear axle	20,000 "
Distance between axles	12 ft.
Width of tread of rear wheels	24 inches
Distance berween centers of rear wheels	6 ft.
Roadway space occupied, width	10 ft.
" " " , length	30 ft.

B. <u>Uniformly Distributed Live Load</u>

One hundred forty(140) pounds per sq.foot

The above uniform live loading may be assumed to occupy all or such portions of the roadway area not occupied by the motor truck loading as is necessary for maximum stresses.

3. TEMPERATURE

Stresses due to an average temperature variation of 60 degrees shall be provided for . When temperature stresses are added to dead and live load stresses the allowable unit stress may be increased 25 %.

4. IMPACT

For concrete arches with spandrel filling of one foot or more no allowance for impact.

On concrete slabs,girders and open spandrel arches allow 25%.

5. DISTRIBUTION of CONCENTRATED LOADS through EARTH FILLS.

Concentrated loads on earth filling two feet or more in depth shall be assumed to be uniformly distributed over an area the side of which is equal to the width of the concentrated load plus the depth of the filling.

6. UNIT STRESSES.

Unit stresses shall not exceed the values given in the following table for the class of concrete indicated:

Class A Concrete	1:1½:3 mix	2500#/sq.in.
" B "	1:2 :4 "	2000#/ " "
" C "	1:2½:5 "	1600#/ " "

	CLASS OF CONCRETE		
	A	B	C
	Pounds per square inch		
Concrete in compression (flexural stress)	800	650	500
Bearing on concrete (where surface which load is applied is at least two times the loaded area)	975	700	550
Axial Compression			
(a) For concentric compression on a plain concrete pier, the length of which does not exceed 4 times the least lateral dimension or on a column reinforced with longitudinal bars only the length of which does not exceed 12 times the least lateral dimension	550	450	350
(b) Columns with longtudinal reinforcement to the extent of not less than 1%, and not more than 4% and with lateral ties of not less than ¼" in diameter, 12" apart, nor more than 16 diameters of the longitudinal bar.	550	450	350
(c) 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 whose length is less than 10 times the least lateral dimension	850	690	540
Shear, used as measure of diagonal tension, on plain concrete sections	50	40	35
Shear, used a a measure of diagonal tension, on sections fully reinforced for diagonal tension in excess of that allowed on plain concrete	150	120	95

	CLASS OF CONCRETE		
	A	B	C
	Pounds per square in.		
Punching Shear	150	120	95
Bond on plain bars	100	80	65
Bond on deformed bars	125	100	80
Ratio of Moduli of Elasticity of steel to concrete"n"	12	15	15
Concrete in tension	0	0	0

Steel in tension 16,000 #/ sq.in.

Steel in compression(where imbedded in concrete) is "n" times the stress in surrounding concrete and in no case should exceed 16,000 #/sq.in.

7. SOIL PRESSURE

The following soil pressure shall not be exceeded :

Clay	4 tons per sq.ft.
Gravel & Sand	5 " " " "

Piling may be required

8. ELEVATION OF FOOTINGS

(a). The bottom of the footings for the abutments and piers in the river shall not be higher than elevation 90'.

For an arch bridge piers and abutments shall be carried to rock. If in the opinion of the designer it is safe to cut off these foundations at a higher elevation, the contractor may put in his bid provided these footings are carried only to a certain elevation, but in no event higher than elevation 90.00'.

(b). For that part of the bridge east of the river the footings shall be carried to the following elevations:

River to		Sta. 2-50	El. 108.00
Sta. 2-00	to	2-50	115.00
1-50		2-00	117.00
1-00		1-50	120.00
0-50		1-00	122.00

9. PIERS.

The center line of the piers and abutments shall follow approximately the line of the river at this point.

10. CLEAR WATER WAY

2600 sq. ft. of clear water way shall be provided below elevation 117.00 and at right angles to the flow of the stream.

INTRODUCTION

The last fifty years of our industrial and commercial development has worked wonders, so to speak, in the problem of transportation. This development has so changed the standard of living and industrial methods that motor cars and trucks have come, perhaps, to stay. As these vehicles have been introduced our highways, streets, and bridges have been subjected to conditions entirely different from those under which they were designed. With this situation at hand the states throughout the nation have tried, not to smother the movement for more motor vehicle transportation, but, to encourage it by improving the public highways. The improvement of the highways has lagged behind the improvement of the motor vehicle because, until recently, the automobile has been looked at as an experiment. Since the number of motor vehicles have steadily increased and the loads hauled on trucks increased in weight it becomes necessary to replace the highways, streets, and bridges of yesterday with structures that will safely withstand the loading of present day traffic. It is for this reason that the City of Lansing, Michigan has an extensive improvment program each year. In this program there is included many miles of pavement. Beside the pavement several bridges are being built. In 1921 a reinforced concrete bridge at Elm St. and Grand River was built and in 1923 a bridge and grade crossing at Shiawassee St. and the river was completed. A three span arch bridge is now being constructed at

Seymour St. and Grand River. The bridge and grade crossing at Kalamazoo St and the river has been designed but not built. Plans are under way for a new structure at the River St. river junction. Other structures are being considered but plans are not yet under way.

The present bridge at Kalamazoo St. is a single span bowstring truss having a span of 214 feet and a height at the center of 24 feet. It was built at Michigan Ave. about 50 years ago and later moved to its present site. It was once floated away during a flood and again moved to its present site. As it now stands it is approximately at right angles to the line of flow of the river which made it necessary to change the direction of the street. This was done by making an approach from River St. This arrangement is very unsatisfactory due to the abrupt turn on to River St. The bridge itself due to its age and the traffic to which it is subjected has become entirely inadequate and unsafe. The above facts are a few that assisted in convincing the city authorities that a new bredge was needed at Kalamazoo St.

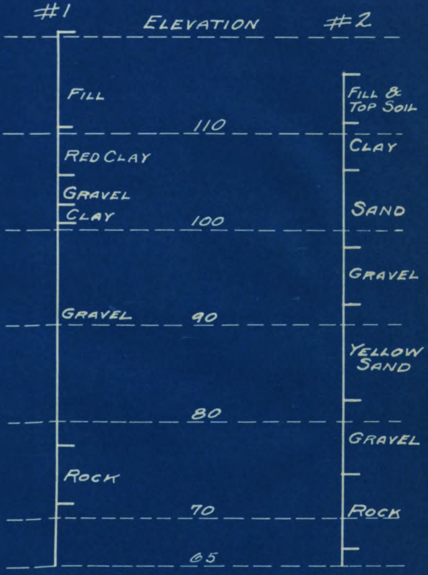
In looking ahead to the new atructure, the first obstacle was the N.Y.C.R.R. on the east side of the river. A new bridge at this point would attract more traffic and since the traffic at a bridge is highly concentrated a railroad crossing would cause serious delay. The only feasible way to avoid such a condition was to make it an overhead crossing. The next condition to be considered was the kind of soil on which the foundations were to be placed.

It was found by test holes that there were several strata of sand, clay, and gravel the total depth amounting to 45 feet on both sides of the river. This condition made an arch bridge impossible from the financial standpoint since conservative designers require that the arch have a rock foundation. The possibility of a slab bridge was eliminated because of the long spans necessary and also from the appearance such a structure presents. The type not yet considered but the one which proved to be the most economical from every point of view is the cantilever. First, it has an appearance equal to that of the arch. Second, it does not require rigid foundations since the load is applied to the foundation vertically.

SOIL CONDITIONS

TEST HOLE #1 WEST BANK

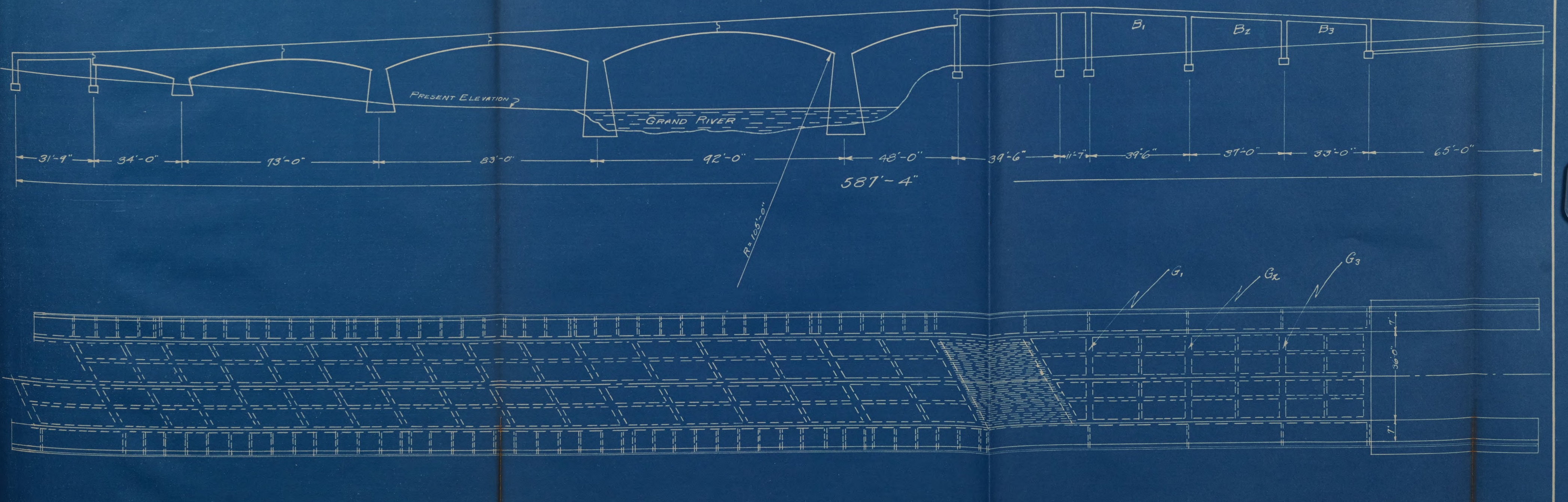
TEST HOLE #2 EAST BANK



PLAN AND SECTION THRU CENTER
PROPOSED BRIDGE AT KALAMAZOO ST. LANSING.

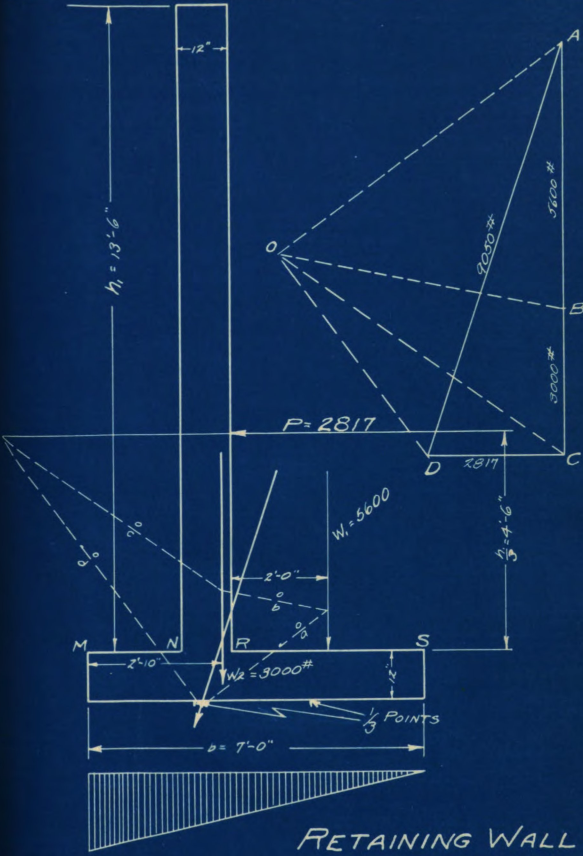
MAY - 1924

SCALE - 3/8" = 10'



RETAINING WALL

VERTICAL SECTION



RETAINING WALL

GRAPHICAL SOLUTION

ANALYSIS

Retaining Wall East End of bridge

Dimensions: (All dimensions are referred to the west or high end of the wall)

	Spacing of steel
West end to 17'-6"	Reinforcing rods spaced 6"
17'-6" to 35'-6"	" " " 9"
35'-6" to 64'-6"	" " " 12"

Section	Width of base (b)	Height (h)
0'-0"	7.[illegible]'	14.00'
9'-0"	6.[illegible]6'	13.50'
18'-0"	6.78'	12.50'
25'-0"	6.67'	11.50'
30'-6"	6.52'	11.33'
36'-0"	6.43'	10.75'
42'-6"	6.35'	10.00'
50'-0"	6.25'	9.25'
54'-0"	6.17'	9.[illegible]
64'-6"	6.0[illegible]'	[illegible].00'

Weight of earth retained is 100 #/ cu.ft.

For the stem of a retaining wall Hool & Johnson recommend the following formula: $M = \frac{f w' h_1^3}{6} \cos e$ p. 5[illegible]

f = factor of limitation

w'= equivalent liquid weight

h_1= height above base

e = angle the thrust makes with the horizontal

The Lansing Engineers through computation and experience with local soil have concluded that the above expression may be represented by : $M = \frac{1}{3}\,\frac{w\,h^2}{2}\,\frac{h}{3} = \frac{w\,h^3}{18}$

They assume the thrust to act horizontal which produces a greater moment, Since several similar walls have been constructed in the city of Lansing under the same assumption and have proved satisfactory, it is safe to use the above results in the analysis of the present structure.

Section [illegible]'-[illegible]"

$$M = \frac{w\,h^3}{18} = \frac{100 \times \overline{13}^3}{18} = 12{,}200\ '\#$$

3/4" rods spaced 6" have an area of .[illegible] sq.in.

$$f_s = \frac{M}{A_s\,jd} = \frac{12200 \times 12}{.[illegible] \times .874 \times 10} = 19{,}000\#\ \text{High}$$

[illegible]J p.277

$$K = M / bd^2 = \frac{12200 \times 12}{12 \times 100} = 122$$

Entering diagram 2 p. 360 with $K = 1[illegible]$ and $f_s = 19000$

$f_c = 725\#$ High for class B concrete

Section 9'-0"

$$M = \frac{100 \times \overline{1[illegible].5}^3}{18} = 11{,}100'\#$$

$$f_s = \frac{11{,}100 \times 12}{.5[illegible] \times .874 \times 10} = 27{,}[illegible]\#\ \text{high}$$

$$K = \frac{11{,}100 \times 12}{12 \times 100} = 111$$

From diagram 2 p. 360 f_c = [illegible]90# High

Section 1[illegible]'-[illegible]" 3/4" round rods spaced 9"

$$M = \frac{100 \times \overline{11.5}^3}{18} = 8{,}[illegible]'\#$$

$$f_s = \frac{8{,}600 \times 12}{.59 \times .874 \times 10} = 20{,}000\#\ \text{high for steel}$$

$k = 86$

From diagram [illegible] p. 3[illegible] $f_c = 610$ # Safe

$M = f_s A_s j d = 16000 \times .59 \times .874 \times 10 = 83000$ "#

The point at which the steel is not over stressed.

$M = w h^3 / 18$; $h_1^3 = 18 M / w = 18 \times 83000 / 1200 = 1250$

$h_1 = 10.75'$ This height occurs at 24'-0"

Therefore a high stress in the steel exists from 17'-6"

to 24'-0"

Section 25'-0"

$$M = \frac{1[illegible] \times \overline{10.[illegible]}^3}{18} = 64[illegible]0'"$$

$$f_s = \frac{6[illegible] \times 12}{.59 \times .[illegible]75 \times 10} = 14[illegible] \text{ \# Safe} \qquad k = 64$$

$f_c = 450$ #

Since the height of the wall is becoming less it is safe for the remainder of the section having 9" spacing.

Section 36'-0" Spacing of bars 12"

$$M = \frac{1[illegible] \times \overline{[illegible].75}^3}{1} = 5140 \text{ '\#}$$

$$f_s = \frac{5140 \times 12}{.44 \times .875 \times 10} = 16000 \text{ \# Safe}$$

$k = 51.4$ $f_c = 400$ #

The remainder of the wall is safe since the spacing of steel is constant and the height is decreasing.

BASE SLAB OF RETAINING WALL

Section of wall at west end.

Resultant falls within the middle third therefore safe against overturning. From the loads as shown in the diagram the total vertical pressure is 5600 #/ ft. which for a 7' base is 800 #/ sq.ft. Allowable soil pressure on clay varies from 1 to 4 tons minimum and our maximum pressure is only 800 #,hence the soil will stand under the wall.

Assuming the unit pressure at the heel equal to zero , the unit pressure at the toe = 2R / A = 11200/7 = 1600 #/ sq. ft. at toe .

This will be taken asthe unit pressure on the entire toe.

Shear along plane HR = 2817 / 144 = 20 #/ sq.in.

Safe Allowable 40 #

Wt. of earth above heel 5600 #/ft. + wt. of heel 600#/ft. = 6200#/ft. total pressure on heel.

V/A = v = 6200 / 144 = 43 # which is above the allowable for plain concrete but when thoroughly reinforced may go as high 120#

Moment at that section 2 x 6200 = 12000

Steel spaced 12" for first 12'

$f_s = \frac{12400 \times 12}{1.92 \times .874 \times 10} = 12900$ # safe.

K = 124 $\qquad f_c$ = 650 # Safe

Total pressure on toe 3200 #

K = M/bd^2 = 32

f_s = $\dfrac{3200 \times 12}{1.32 \times .874 \times 10}$ = 3330 # Safe.

f_c = 200 # low.

u = $\dfrac{6000}{7 \times .874 \times 10}$ = [100] High for plain bars, safe for deformed bars.

Section 18'-0" $\qquad$ Base width = 6.7'

Shear along section MR = 2205 / 144 = 15.4#/sq.in.

Shear along vertical plane thru R

Pressure on heel = 3.7 x 1 x 1 x 100 x 11.5 = 4250#

4250 / 144 = 30#/sqin. $\qquad$ Safe

4250 x 2 = 8500 '# moment of heel pressure.

F_s = $\dfrac{8500 \times 12}{.88 \times .874 \times 10}$ = 13 300 # $\quad$ Safe.

K = [illegible]5 $\qquad f_c$ = 550 #

The toe is of constant length therefore the same moment as in other sections = 3200

f_s = $\dfrac{3200 \times 12}{.88 \times .874 \times 10}$ = 5000 # Low

K = 32 $\qquad f_c$ = 250 #

Section 30'-0"

P = 1600 #

Shear along MR $\qquad$ 1600 / 144 = 11.1 # Safe

Pressure on heel $\qquad$ 10.75 x 1 x 3.45 x 100 = 3700 #

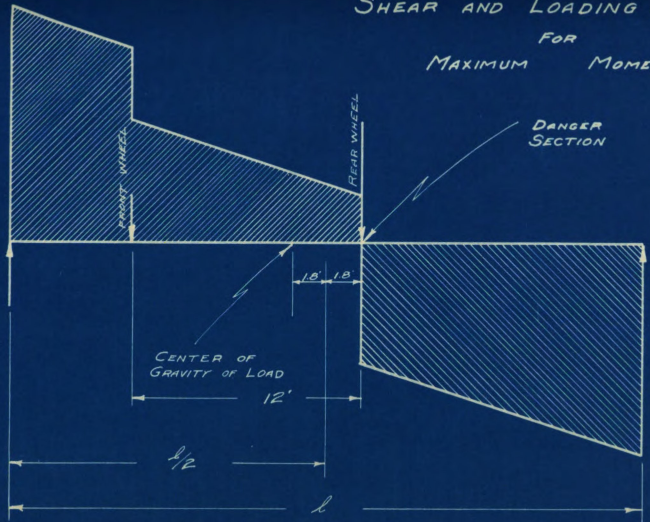
3700 / 144 = 25.8 # Safe $\qquad$ Vertical shear thru R

Moment at R = 2 x 3700 = 7400

f_s = $\dfrac{7400 \times 12}{.66 \times .874 \times 10}$ = 15 400 # $\quad$ Safe

u = $\dfrac{3700}{3.9 \times .874 \times 10}$ = [110]# High for plain bars.

Shear and Loading Diagram for Maximum Moment



Since the short span is assumed to take all the load,

M = 1535 (2.5 + 1535/ 2 x 1535) = 3 x 1535 = 4605 '#

Dead load moment = 1/10 w l^2

= 1/10 x 130 x 7 x 7

= 637 '#

Total moment = 5265 '#

$$d = \sqrt{\frac{5265}{107.7}} = 6.9'' \qquad D = 8.4'' \qquad \text{Actual } D = 8''$$

$$f_s = \frac{5265 \times 12}{.88 \times .74 \times 6.9} = 15\,900 \text{ \# Safe}$$

$$x = \frac{5265 \times 12}{12 \times 6.9 \times 6.9} = 110 \qquad f_c = 650 \text{ \# Safe}$$

BEAMS

Since the slab and beams are built monolithically Tee-beam action can be expected, in which b = ¼ span.

For continuous or restrained beams built monolithically into supports, the length of span may be taken as the clear distance between faces of supports. Hool & Johnson p.318

Beam B_5 Center l = 30' d = 21.75 " b' = 36"

Dead load - Slab = 1300 #/ft.

Beam = 975 "

2275 "

M = 1/10 w l^2 = 1/10 x 2275 x 30 x 30 = 205,000 '#

Uniform live load at 1400 #/ft.

M = 1/10 x 1400 x 30 x 30 = 126,000 '#

Total moment 331,000 '#

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

p = A_s/ b d

$j = .834$

$$kd = \frac{2\, ndA_s + bt^2}{2\, nA_s + 2bt} = 8.76$$ Therefore case II

$k = .399$

$$f_s = \frac{M}{A_s jd} = \frac{331,000 \times 12}{17.7 \times .834 \times 21.75} = 12,400 \text{ \# Safe}$$

Same beam under action of the truck load

$b = 90"$; $d = 21.75"$; $A_s = 17.7$ $b' = 36"$

The truck load occupies a space 10' x 30' therefore no uniform live load can be applied to any beam when the truck load is acting. The total truck load will be assumed to be acting on the beam.

Weight of slab and beam as before 2275 # / ft.

Dead load moment $1/10\ w\, l^2$ = 205,000 '#

The load is applied as shown on the loading diagram in which $l = 30'$

From the shear diagram the danger section occurs under the rear wheel of the truck hence moments will be taken about that point.

R_L = 28,000 # R_R = 20,000#

M = 15,000 x 12 - 28,000 x 16.8 = 290,000 '#

For continuous action use 8/10 of moment

8/10 x 290,000 = 232,000 '#

Total moment 437,000 '#

As above $j = .834$ $k = .339$

$$f_s = \frac{437,000 \times 12}{17.7 \times .834 \times 21.75} = 16,400 \text{ \#}$$ Over the support [20,700#] High.

This value is larger than the stress caused by the uniform live load therefore the truck load will be used for the following analysis.

$$f_c = \frac{16{,}400 \times .339}{15(1 - .339)} = \frac{f_s\, k}{n(1-k)} = 605\ \#\quad \text{Safe.}$$

Live load reaction 28,000#

Dead " " 33,000#

61,000 # = Shear

$$\text{Unit bond stress, } u = \frac{V}{o \times j \times d} = \frac{61{,}000}{63 \times .843 \times 21.75}$$

$$= 53.5\ \#\ \text{Safe Allowable } 80\ \#\ \text{plain bars}$$

$$\text{Unit shearing stress, } v = \frac{V}{b \times j \times d} = \frac{61{,}000}{36 \times .834 \times 21.75}$$

$$= 93.5\ \#\ \text{High, web reinforcement needed.}$$

Spacing of turned up bars, where A_s = area of turned up bars

$$s = \frac{3 \times A_s \times f_s \times j \times d}{2 \times V} = \frac{3 \times 3.8 \times 16{,}400 \times .834 \times 2175}{2 \times 61{,}000}$$

$$= 27''$$

Bars are spaced 18" therefore they are able to take the shear.

Beam - B_3 - Intermediate

Live load moment 232,000 '#

w_s = 1300 #/ft.

$$w_D = \frac{(19.75 + 4.25) \times 36}{144} \times 150 = 900\ \#/\text{ft.}$$

Total w = 2200#/ft.

$$M = 1/10 \times 2200 \times 30 \times 30 = 198{,}000\ '\#$$

Total live and dead load moment 430,000 '#

j = .995 Formula 6 p. 308

k = .417 " 2 p. 308

$f_s = \dfrac{432,000 \times 12}{17.7 \times .995 \times 19.75}$ = 14,900 # Safe

Over support $\boxed{19,000 \text{ \# ?}}$

$f_c = \dfrac{14,900 \times .417}{15(1 - .417)}$ = 1710 # High

V = 61,000 #

$u = \dfrac{61,000}{63 \times .995 \times 19.75}$ = 49.5 # Allowable 60#

$v = \dfrac{61,000}{36 \times .995 \times 19.75}$ = 86.2 # " 40 #

x_1 = the distance beyond which no web reinforcement is needed Hool & Johnson p.286

$= 1/2 \; (1 - v_1/v) = 30/2(1 - 40/86.2) = 8.05'$

First bars are turned up 3'-6" from support therefore safe

$s = \dfrac{[illegible].56 \times 14,900 \times .995 \times 19.75}{2 \times 61,000}$ = 19.25"

Three bars are turned up at the support where the maximum shear occurs and although this spacing for two bars is slightly in excess of the actual, the beam is well reinforced where the greatest shear acts, namely, the support. Stirrups are an added factor of safety in this beam.

Beam- B_3 - Outside.

Live load moment	232,000 '#
Slab (½ only)	650# /ft.
Beam	1020 "
Walk	700 "
Rail	375 "
Total	2745 #/ft.

M = 1/10 x 2745 x 900 = 247,000 '#

Total live and dead load moment 479,000 '#

$j = .874 \qquad k = .395$

$f_s = \dfrac{479,000 \times 12}{17.7 \times .874 \times 23} = 16,300$ # Safe

Over support 20,900 # High

$f_c = \dfrac{16,350 \times .325}{15 \times .605} = 713$ # High

Reactions LL 28,000
DL 41,200

Total 69,200 = V

$u = \dfrac{69,200}{65 \times .874 \times 23} = 54.6$ # Safe

$v = \dfrac{69,200}{36 \times .874 \times 23} = 96$ # Allowable 40 #

$x_1 = 15 \ (1 - .4175) = 8.75'$

Turned up bars take care of the shear for 8.5' and the inclined stirrups provide the web reinforcement for the remainder of the distance.

$s = \dfrac{3 \times 2.54 \times 16,350 \times .874 \times 23}{2 \times 69,200} = 18.05$ "

Rods are spaced 18" therefore safe for turned up rods.

Inclined stirrups have an area of 6 x .1963 = 1.18 sq. in.

For stirrup spacing $s = \dfrac{3 \times 1.18 \times 16,350 \times .874 \times 23}{2 \times 69,200}$

$= 12"$

Stirrups are spaced 15" This value of s is small since the value of V at the support was used when in the actual beam the shear out a distance of 8.5' would be much less giving a greater spacing.

Beam - B_2 - Center

d = 24.75"; b = 102" Clear span = 34'

Live load moment as computed in previous beam 272,000'#

Dead load- Slab 1300 #/ft

Beam 1090 #/ft.

Total dead load 2390 #/ft.

$M = 1/10 \times \overline{34}^2$ = 276,000 '#

Total moment 548,000 '#

j = .923 k = .41

$f_s = \dfrac{548,000 \times 12}{22.75 \times .892 \times 24.75}$ = 14,200 # Safe

Over support 17,000 # High

$f_c = \dfrac{14,200 \times .41}{15 \times .59}$ 655 # Safe.

Reactions L.L. 27,600
D.L. 40,600

68,200 = V

$u = \dfrac{68,200}{61 \times .892 \times 24.75}$ = 41.5 # Safe.

$v = \dfrac{68,200}{36 \times .893 \times 24.75}$ = 93 # Allowable 40#

x_1 = 17(1 - 40/93) = 17 x .57 9.7'

3 - 1 1/8" square bars are turned up in each groupe or an area of 3.8 sq. in.

$s = \dfrac{3 \times 3.8 \times 14,200 \times .893 \times 24.75}{2 \times 68,200}$ = 24.2"

Bars are turned up at intervals of 18" and extend 8½' from support. 6 - ½" round bars at 1' -6" furnish the rest of the web reinforcement.

Shear at end of turned up bars, 68,200 - (2390 x 8½)

= 47,900#

Spacing of stirrups $= \dfrac{3 \times 1.18 \times 14,200 \times .893 \times 24.75}{2 \times 47,900}$

= 10.75 "

Actual spacing of stirrups 18".

Beam - B_2 - Intermediate

Beam - B_2 - Intermediate

Dead load - Slab 1300#/ft. $d = 22.75$

Beam 870 "

2170 "

D.L. Moment $= 1/10 \times 2170 \times \overline{34}^2 = 251,000$ '#

Live load moment 272,000 "

Total moment 523,000 '#

$j = .867$ $k = .42$

$f_s = \dfrac{523,000 \times 12}{22.75 \times .867 \times 22.75} =$ 14,000 # Safe
Over support 16,000# Safe.

$f_c = \dfrac{14,000 \times .42}{15 \times .58} =$ 674 # Safe.

Reactions L.L. 27,600
D.L. 36,900

64,500 = V

$u = \dfrac{64,500}{81 \times .867 \times 22.75} =$ 40.5 # Safe.

$v = \dfrac{64,500}{36 \times .867 \times 22.75} =$ 91 # Allowable 40#

$x_1 = 17 \ (1 - 40/91) = 17 \times .56 = 9.5'$

Bent up bars take care of 8½' . Stirrups furnish remainder of web reinforcement.

Spacing of bars $= \dfrac{3 \times 3.8 \times 14,000 \times .867 \times 22.75}{2 \times 64,500}$

$= 24.4''$ Bars are spaced 12" - Safe.

Spacing of stirrups $= \dfrac{3 \times 1.13 \times 14,000 \times .867 \times 22.75}{2 \times 45,100}$

$= 10.6''$ Stirrups are spaced 12" Low.

Beam - B_2 - Outside

Dead load - Slab 650 #/ft. d = 28"

Beam 1200

Walk 700

Rail 375

2925 #/ft.

$M = 1/10 \times 2925 \times \overline{34}^2 = 339{,}000$ '#

L.L.Moment 272,000

Total moment 611,000 '#

$j = .93$ $k = .397$

$f_s = \dfrac{611{,}000 \times 12}{24.7 \times .93 \times 28} = 11{,}800$ # Over support 14,200# Safe.

$f_c = \dfrac{11{,}800 \times .397}{15 \times .603} = 507$ # Safe.

$V = 50{,}000 + 27{,}600 = 77{,}600$ #

$u = \dfrac{77{,}600}{81 \times .93 \times 28} = 35$ # Safe.

$v = \dfrac{77{,}600}{36 \times .93 \times 28} = 83$ # Allowable 40 #

$x_1 = 17(1 - 40/83) = 17 \times .4025 = 8.2$ '

Turned up bars provide the web reinforcement for 8½' from support, hence the stirrups are an added factor of safety in this beam.

Beam - B_1 - Center

b = 108 " d = 24.75 '' Clear span = 36'

Deadload - Slab 1500 #/ft.

beam 1090 "

2590 "

D.L.Moment $1/10 \times 2590 \times \overline{36}^2 = 310{,}000$ '#

live load moment 292,000 "

Total live and dead load moment 602,000 '#

$j = .863$ $k = .41$

$f_s = \dfrac{602,000 \times 12}{22.78 \times .863 \times 24.75} = 15,600$ Over support 1[illegible],700#

High over support.

$f_c = \dfrac{15,600 \times .41}{15 \times .59} = 724$ # High.

$V = 43,000 + 27,500 = 70,500$ #

$u = \dfrac{70,500}{61 \times .863 \times 24.75} = 42.8$ Safe.

$v = \dfrac{70,500}{36 \times .863 \times 24.75} = 96.5$ Allowable 40#

$x' = 13(1 - 40/96.5) = 13 \times .585 = 10.5'$

In this beam the first turned up bar occurs at 10.5' from support therefore the stirrups are an added safety.

Beam -B' - Intermediate

$j = .867$ $k = .42$ $d = 22.75"$

Dead load - slab	1300 #/ft	
beam	1010	"
	2310 #/ft.	

D.L. Moment $1/10 \times 2310 \times 36^2 = 299,000$ '#

Live load moment 292,000 "

total moment 591,000 "

$f_s = \dfrac{591,000 \times 12}{22.78 \times .867 \times 22.75} = 15,800$ Over support 19,000#
High over support.

$f_c = \dfrac{15,800 \times .42}{15 \times .58} = 763$ # High

$V = 27,500 + 41,600 = 69,100$ #

$u = \dfrac{69,100}{61 \times .867 \times 22.75} = 43$ # Safe.

$v = \dfrac{69,100}{36 \times .867 \times 22.75} = 97.5$ #

x_1 = 18 (1 - 40/97.5) = 18 x .59 = 10.6'

Rods are turned up at 10.5'.

Spacing of bars = $\dfrac{3 \times 3.8 \times 15,000 \times .867 \times 22.75}{2 \times 59,100}$

= 25.7"

Bars are turned up at 24" intervals. O.K.

Beam - B_1 - Outside

j = .93 k = .397 d = 28"

Dead load - slab 650 #/ft.

beam 1200 "

walk 700

rail 375

2925 #/ft.

Dead load moment 1/10 x 2925 x 36^2 = 380,000'#

Live load moment 292,000 "

total moment 672,000 '#

f_s = $\dfrac{672,000 \times 12}{22.78 \times .93 \times 28}$ = 12,900 # Over support 15,500 Safe.

f_c = $\dfrac{15,500 \times .397}{15 \times .603}$ = 680 # Safe.

V = 52,700 + 27,500 = 80,200 #

u = $\dfrac{80,200}{31 \times .93 \times 28}$ = 36.1 Safe

v = $\dfrac{80,200}{36 \times .93 \times 28}$ = 81.2 #

x_1 = 18 x .509 = 9.15 ' Bars are bent up 10½' from

support spaced = $\dfrac{3 \times 3.8 \times 12,900 \times .93 \times 28}{2 \times 80,200}$ = 25.2"

Turned up bars are spaced 24".

GIRDER ANALYSIS

Span 19.5' d' = 30" d = 36"

Dead load concentrated at the center of the girder:

Beam B_3	900#/ft. x 15'	=	13,500 #
" B_2	870 # x 17	=	14,800 #
32' of slab at 1300#/ft.		=	41,600 #
			69,900 #

Wt. of girder $\frac{31 \times 36}{144}$ x 150 = 1240 #

Moment of concentrated dead load $\frac{P\,l}{4}$ = $\frac{69,900 \times 19.5}{4}$
= 341,000 '#

Moment of uniform dead load 1/10 wl^2 = 1/10 x 1240 x $\overline{19.5}^2$
= 47,200 '#

The truck was found to give the maximum live load stress.

Live load moment = 17,500 x 6.75 = 118,000 '#

Total live and dead load moment 506,2000 '#

Since the girder has a varying depth the weakest place will be analyzed, namely, where d = 30"

$f_s = \frac{506,200 \times 12}{13.9 \times .875 \times 30}$ = 16,700 # Safe.

$K = M/bd^2$ = 506,200/ 36 x 900 = 156

f_c = 700 # Diagram 2 p. 360.

V = 64,550 #

$u = \frac{64,550}{49.5 \times .874 \times 30}$ = 50 # Safe.

$v = \frac{64,550}{36 \times .874 \times 30}$ = 68.5 # Web reinforcement needed

x_1 = 19.5/2 (1 - 40/68.5) = 4.05 '

First bars are turned up at 7' from center of support which provides sufficient web reinforcement.

At the first section two bars are turned up and at other sections three bars are turned up in a groupe. the maximum shear occours at the support where there are three bars at each section. The spacing will then be based of the three bars.

$$s = \frac{3 \times 3.9 \times 16000 \times .874 \times 30}{2 \times 64,500}$$

$= 37"$ Bars are spaced 24"

Girder G_2

Since the girder and columns are built monolithically into the supports as in the case of the beams the spans may be taken as the clear distance between faces of supports, or 16.5'

Dead loads at center -	Beam B_2	14,[illegible]
	" B_1	13,800
	Slab	46,[illegible]
Total load at center		[illegible],[illegible] #

Wt. of girder 1240 #/ft.

Moment of forces at center $\frac{72,[illegible] \times 16.5}{4}$ = 330,000 '#

Moment of uniform load 1/8 x 1240 x $\overline{16.5}^2$ = 42,200

Total dead load moment 372,000

Live load moment 17,[illegible] x 3.75 =

65,600

Total live and dead load moment 437,[illegible] '#

$f_s = \frac{437,[illegible] \times 12}{13.[illegible] \times .874 \times 30}$ = 14,400 # Safe.

$K = M/bd^2$ = 437,[illegible]/ 36 x 900 = 16[illegible]

f_c = $\boxed{[illegible]}$ # High.

$V = 69{,}500\ \#$

$u = \dfrac{69{,}500}{49.5 \times .874 \times 30} = 53.5\ \#$ Safe

$v = \dfrac{69{,}500}{36 \times .874 \times 30} = 73.5\ \#$

$x' = 16.5/2(1 - 40/73.5) = 3.7'$

$s = \dfrac{3 \times 3.7 \times 16{,}000 \times .874 \times 30}{2 \times 69{,}500} = 34''$

Bars are spaced 24" Safe.

GIRDER G,

Dead loads at center

8'	Slab at	1300#/ft		10,400 #
18'	" "	"		23,400
17'	Beam at	870	"	14,800
8'	" "	"	"	6,900
		Total		55,500

$M = \dfrac{55{,}500 \times 16.5}{4} = 229{,}000\ '\#$

Uniform weight of girder 955 #/ft.

$M = \dfrac{955 \times \overline{16.5}^2}{8} = \quad 32{,}500\ '\#$

Live load moment 65,600

327,100 '#

$f_s = \dfrac{327{,}100 \times 12}{12.65 \times .874 \times 21}$ 16,900 # Safe

$K = M/b\,d^2 = \dfrac{327{,}100}{36 \times 21} = 206$

$f_c = \boxed{1000}\ \#$ High

Reactions 27,775
17,500
8000

53,275 = V

$u = \dfrac{53,875}{45 \times .874 \times 21} = 65\#$ Safe.

$v = \dfrac{53,875}{36 \times .874 \times 21} = 82\#$

$x_1 = 8.25\,(1 - 40/82) = 4.25'$

First bent up bars occur at 5.5' from face of support

$s = \dfrac{3 \times 3.9 \times 16,000 \times .874 \times 21}{2 \times 53,875} = 33''$

Bars are spaced 24"

Columns

Overall dimensions 36 x 36   3" protection on all sides

8 - 1 1/8" square bars   4 on each side of column

Center column

Supporting Girder G_3, Beams b_2 & B_3.

Loads	Girder reactions	129,100
	Beam "	61,000
		68,200
		258,300 = P

$f_c = \dfrac{P}{A(1 + p(n - 1))}$ $p = \dfrac{10.1}{900} = .0112$

$= \dfrac{258,300}{30 \times 30\,(1 + .0112(15 - 1))}$

$= 248\#$ Allowable 450#

Outside column

Supporting G_3 Beams B_2 & B_3

Loads	Reaction	G_3	64,550
	"	B_3	77,200
	"	B_2	69,500
	"	Walk	34,600
	Live load on walk		23,500
	Total (P)		269,350

$f_c = \dfrac{269,350}{900 \times 1.157} = 293\#$ Safe.

Center column

Supporting C_2 - Beams B_2 & B_1

Reaction	C_2	69,500
		69,500
"	B_2	6[illegible],200
"	B_1	70,500
		277,700 = P

$f_c = \frac{277,700}{900 \times 1.0157} = 307$ # Safe.

Outside column

Supporting C_2 - Beams B_2 & B_1

Reaction	C_2	69,500
"	B_2	77,600
"	B_1	[illegible],200
35.5' walk slab at [illegible]		23,300
" rail at 375 #		13,000
" live load at 7[illegible] #		24,000
		2[illegible],400 = P

$f_c = \frac{2[illegible],400}{900 \times 1.0157} = 318$ # Safe.

Center column

Supporting Girder C_1 - Beam B_1 and short beam from girder along R.R.

Reaction	C_1	53,[illegible]75
		53,[illegible]75
"	B_1	70,500
"	Short beam	13,270
		191,270

$f_c = \frac{191,270}{900 \times 1.0157} = 21[illegible]$ # Safe.

MAIN CANTILEVER

CANTILEVER

Supporting walks along main cantilevers

Span 7'-0" Depth 30"

Rail concentrated at end 8 x 375 = 3000 #

M = Pl = 3000 x 7 = 21,000 '#

Slab uniformly distributed 750 #/ft.

Live load " " 1120 "

1870

Wt. of beam $\frac{12 \times 20}{144}$ x 150 250

Total uniform load 2120 "

$M = \frac{w \times l^2}{2} = \frac{2120 \times 49}{2} =$ 52,000 '#

Moment of rail 21,000

73,000

$f_s = \frac{73,000 \times 12}{2.41 \times .874 \times 30}$ 13,900 # Safe.

K = 81 f_c = 550 # Safe.

Distance from end	Shear
1' -0"	5,120 #
2'	7,240
3'	9,360
4'	11,480
5'	13,600
6'	15,720
7'	17,840

Spacing of vertical stirrups at 7'

$s = \frac{3 \times .0[illegible] \times 16000 \times .874 \times 30}{2 \times 17,840} = 3.46"$

Stirrups are spaced 2" at this point. Safe.

MAIN CANTILEVER 48' -0"

The pier is 8' wide , hence the length of cantilever to be considered is 44'. Radius of under surface 105'

Steel at the sections

Section	Number & Size of Rods		Area.
10'	6 - 1 1/4"	Sq.	9.37 sq. in.
20	10	"	15.62
30	14	"	21.87
34	16	"	25.00
40	18	"	28.13
44	18	"	28.13

LOADS

DEAD - Slab , uniformly distributed using $M = wx^2/2$.

x = distance from unsupported end.

Section	Moment
10'	65,000 '#
20'	260,000
30	585,000
34	752,000
40	1,040,000
44	1,260,000

Moments caused by slab load

Weight of cantilever - Thickness at end - 4'

" " Support -14'

Width - 2'

W(total weight) $= \frac{14 + 4}{2} \times 44 \times 2 \times 150 = 119{,}000$ #

When the load on a cantilever increases toward the support use $M = W x^3/3l^2$

Section	Moment due to wt. of cantilever	Total dead load Moment.
10	20,500 ft. lbs.	85,500 ft. lbs.
20	164,000	424,000
30	552,000	1,137,500
34	805,000	1,557,000
40	1,310,000	2,350,000
44	1,740,000	3,000,000

Section	Live load moment using uniform load and $M = wx^2/2$
10	70,000 ft.lbs.
20	280,000
30	630,000
34	810,000
40	1,190,000
44	1,350,000

By trial it was found that the maximum moment occurs when the rear wheel of the truck is placed at the end of the cantilever , the front wheel on the next section,and the remaining area covered with the uniform live load.

Section	Moment due to a concentrated load of 35,000 # at the end
10	350,000 ft. lbs.
20	700,000
30	1,050,000
34	1,190,000
40	1,400,000
44	1,540,000

Section	Moment due to partial live load (all but 9'at end)	Total moment
10	700 ft. lbs.	436,000 ft.lbs.
20	70,000	1,194,000
30	280,000	2,467,000
34	404,000	3,151,000
40	630,000	4,380,000
44	810,000	5,350,000

The cantilever can be considered a special form of reinforced concrete beam and can be analyzed by the method given in The Concrete Engineer's Handbook by Hool & Johnson p.315. In this method long and cumbersome formulas are avoided by using the equivalent homogeneous section ,by multiplying the area of the compressive and tensile steel by n.
These transformed areas are used to compute the distance from the compression face to the neutral axis or in other words to locate the neutral axis.
The following is the statical moments equation of tensile and compressive areas:

$$\frac{by^2}{2} + \text{transformed compressive area}\,(y - \text{protection}) = \text{transformed tensile area}\,(d - y)$$

y = distance from compression face to neutral axis

b = breadth (24" inches for all sections of the cantilever)

Section	Depth	y
10	53"	13.8 "
20	72	18.25
30	101	21.2
34	117	22.25
40	147	25.25
44	166	25.25

Sample computation (section 10' - 0")

$$\frac{24\,y^2}{2} + 15 \times 9.37(y - 3) = 15 \times 9.37(53 - y)$$

$$12\,y^2 + 140.5(y - 3) = 140.5\,(53 - y)$$

$$12\,y^2 + 281\,y = 7866.5$$

$$y = 13.8''$$

$$I = I_c + nI_s + nI'_s$$

$$nI'_s = n\,A_s\,(y - 3)^2 = 15 \times 9.37\ (10.8)^2 = 1641$$

$$nI_s = n\,A_s\,(D - y)^2 = 15 \times 9.37\ (42.2)^2 = 2500$$

$$I_c = b\,y^3/3 = 24 \times (13.8)^3/3 = 22400$$

$$I = 26541$$

$f_c = M\,y/I \quad 436,200 \times 13.8 / 26,541 = 227$ # Safe.

$f'_s = \dfrac{n\,M(y - 3)}{I} = \dfrac{15 \times 436,200 \times 10.8}{26,541} = 2660$ # Safe.

$f_s = \dfrac{n\,M(D - y)}{I} = \dfrac{15 \times 436,200 \times 39.3}{26,541} = 9,850$ # Safe.

SECTION 20'

$I = 119,170 \qquad M = 1,194,000 \qquad y = 18.25$

$f_c = \dfrac{1,194,000 \times 18.25}{119,170} = 183$ # Safe.

$f'_s = \dfrac{15 \times 1,194,000 \times 15.25}{119,170} = 2950$ # Safe.

$f_s = \dfrac{15 \times 1,194,000 \times 53.75}{119,170} = 8100$ # Safe.

Section 30'

$I = 332,500 \qquad M = 2,467,000 \qquad y = 21.2$

$f_c = \dfrac{2,467,000 \times 21.2}{332,000} = 157.5 \#$ Safe.

$f'_s = \dfrac{15 \times 2,467,000 \times 18.2}{332,000} = 2030 \#$ Safe.

$f_s = \dfrac{15 \times 2,467,000 \times 79.2}{332,000} = 7900 \#$ Safe.

Section 34'

$I = 476,000 \qquad M = 3,151,000 \qquad y = 22.25$

$f_c = \dfrac{3,151,000 \times 22.25}{476,000} = 147 \#$ Safe.

$f'_s = \dfrac{15 \times 3,151,000 \times 19.25}{476,000} = 1910 \#$ Safe.

$f_s = \dfrac{15 \times 3,151,000 \times 94.75}{476,000} = 9450 \#$ Safe.

Section 40'

$I = 817,750 \qquad M = 4,380,000 \qquad y = 25.25$

$f_c = \dfrac{4,380,000 \times 25.25}{817,750} = 136.5 \#$ Safe.

$f'_s = \dfrac{15 \times 4,380,000 \times 22.25}{817,750} = 1800 \#$ Safe.

$f_s = \dfrac{15 \times 4,380,000 \times 121.75}{817,750} = 9800 \#$ Safe.

Section 44'

$I = 1,032,750 \qquad M = 5,350,000 \qquad y = 25.25$

$f_c = \dfrac{5,350,000 \times 25.25}{1,032,750} = 131 \#$ Safe.

$f'_s = \dfrac{15 \times 5,350,000 \times 22.25}{1,032,750} = 1730 \#$ Safe.

$f_s = \dfrac{15 \times 5,350,000 \times 140.75}{1,032,750} = 10,900 \#$ Safe.

The above analysis is for one cantilever ,which is all that is required since each cantilever is a part of the preceding one.

In the above computations the values that are greater than those given in the specifications are enclosed in red. Some ,however, are of little importance since they are only slightly greater than the specified values. In case of steel a moderately high can safely exist because it is a dependable product. This is not the case with concrete and the stress should not be allowed to become larger than the value specified. In the girders the stress in the concrete is greatly in excess of that stated in the specifications. For the analysis see pages 18 & 19.