A STUDY INTO MATERIALS COST IN CONNECTION WITH THE CONSTRUCTION OF A CROSS-UNDER GRADE SEPARATION ON M 39, W. SAGINAW AT GRAND TRUNK R. R.

Thesis for the Degree of B. S.

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A STUDY INTO MATERIALS COST IN CONNECTION WITH

THE

CONSTRUCTION OF A CROSS-UNDER GRADE SEPARATION

ON

M39, W. SAGINAW at GRAND TRUNK R.R.

A THESIS SUBMITTED THE FACULTY

of

MICHIGAN STATE COLLEGE

BY

R. C. TIMMICK R. A. NOWLIN

CANDIDATES FOR THE DEGREE OF

BATCHELOR of SCIENCE

JUNE 1929

THESIS

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EXPLANATION OF DESIGN OF

GRADE CROSSING ON M-39 at

W. SAGINAW at City Limits; LANSING, MICH.

With GRAND BELT RAILROAD

APPROACHES:

From the west the grade of the approach rises toward the track; also from the east, but with a lesser slope. The rise from the west is around eight feet, thus making the possibility of a cross-over inadvisable, both from a construction standpoint as well as initial cost. From the west the cut is begun 600' from the intersection and from the east 700' and the two grades are run as vertical curves, meeting under the intersection.

From the "Standard Road and Bridge Specifications" of the Michigan State Highway Department, a minimum clearance from bottom of girder to crown of roadway should not be less than 14'. In this design the clearance is 15' thus conforming to the specifications.

An allowance has been made for future growth of the intersecting railroad by designing the abutments for two tracks instead of the one that now crosses the highway.

The tracks of the railroad cross the highway at an angle of 90 degrees.

RAILROAD BRIDGE:

The type of bridge selected is a deck plate girder type 35' in length with a width center to center of girder of 10' according to the specifications above mentioned. Floorbeams are to be spaced 2' from each other to reduce the effects of swaying that would occur under heavy loads with the supports at greater distances apart. The space between the floorbeams is to be filled in with concrete to the depth indicated in the specification and above this will be placed a layer of crushed stone to be supplied by the railroad company. The standard ties are to be placed 10" center to center and embedded in the crushed stone.

The loadings under which the bridge is stressed is a combination of concentrated and uniformily distributed loads. As there is no means of determining in many cases as to what loadings are encountered (as in the distribution of the train loading through the crushed stone) a system of possible loadings must be calculated, using that loading which will give the maximum amount of stress in the individual members. The bridge was designed for a Cooper E-60 loading throughout.

RETAINING WALLS:

The retaining walls that were adopted in this design are the cantilever-counterfort type. This type was adopted because of the saving that can be obtained in required concrete. It is to be reinforced with deformed steel bars throughout. The horizontal thrust due to the approaching train was found to be equal to a surcharge of 8.9'. The safety against overturning and sliding was found to be over 2.0 which is satisfactory.

The counterforts act as cantilevers and are securely tied to both the vertical wall and the footing. The clear span of horizontal thrust is the distance between the outside of two counterforts. An equivalent liquid pressure was used in determining the stresses in the counterforts. The reinforcing bars are to be extended into both the vertical wall and the footing according to the formula for sufficient bond.

The projecting toe of the footing is a cantilever with the moments taken about the end next the outside of the vertical wall. The loading is determined by the earth pressure acting vertically upon the toe. The toe is to be reinforced with deformed steel bars and the bars are to be extended into the other side of the footing to give the proper bond for safety.

The inner portion of the footing is a slab supported by the counterforts. It is subjected to both horizontal and vertical forces and must be designed accordingly. The reinforcing rods are extended into the toe of the footing to give sufficient bond.

The vertical wall acts as a cantilever beam supported at intervals by the counterforts. The moments are taken about that section at the base of the wall next to the footing and is designed according to the procedure used in "Reinforced Concrete Construction", Volume 2 by Prof. Hool.

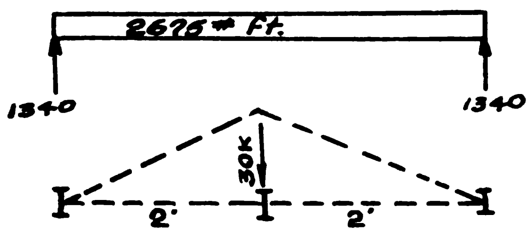
COMPUTATIONS

for

DESIGN OF DECK GIRDER BRIDGE

Ties 8'x6"x8" Spaced 10" apart
Wt. of tie 8x$\frac{1}{2}$x$\frac{3}{4}$x50 = 150# per tie
Space ties 10" apart
Wt. of tie per ft. = 10/12x150 = 125# per ft.
Rail & fastenings = 150# per ft. of track
Concrete plus gravel = 1275+1125=2400
Total dead load on floor beam=2675#

DESIGN OF FLOOR BEAMS



Live shear	30 Kips
Impact	30 "
Total	60 "

Maximum dead moment

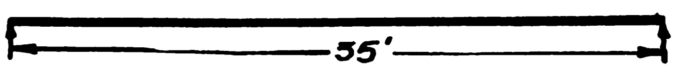
$2675 \times 2 \times 5 + \frac{500(2)^2}{8} = 27{,}000$ ft.#

30+30+2.675=8675# per linear ft of bridge

From Carnegie use 15"--45# I beam
1 Linear foot of floorbeam 225#

Total dead load + floorbeam = 3,000# per lin. ft.

DESIGN OF GIRDER



Live Shear at end

Load 1

$\frac{2460+3 \times 154.5}{35} = \frac{2924}{35} = 83.6$ Kips

Load 2

$\frac{4276.5-43 \times 15}{35} = \frac{3631.5}{35} = 101$ Kips

Load 3

$\frac{5244-48 \times 15-40 \times 30}{35} = 95.0$ Kips

Load 4

$\frac{5244+5 \times 213-53 \times 15-(45\ 40)30}{35} = 57.2$ Kips

$I = 101 \times \frac{300}{300\ \frac{35^2}{100}} = 105.2$ Kips Total=105.2+101.0 = 206.2 Kips

Load 2 gives maximum shear.

Dead shear at end = 52,500 #

Maximum dead Moment at center

3000x17.5x8.75 = 459,375 ft. lb.

Maximum Live Moment at center

Load 2

$\frac{1245+2.5\times135}{35} - (120) = 671.5$ Kip ft.

Load 3

$\frac{(1245+7.5\times135)17.5}{35} - 345 = 783.75$ Kip ft.

Load 4

$\frac{2460+3.5\times154.5}{2} - 720 = 780$ Kip ft.

Load 3 gives Maximum moment at center

<u>Height of web</u>

$h = \frac{35\times12}{10} = 42"$

$t = \frac{258,700}{42\times10,000} = .616"$ Use 5/8" web plate

$t = \frac{1}{20}\sqrt{30.5} = .276"$

Design of Flanges of Girder

$A = \frac{1,243,125\times12}{16,000\times41} - \frac{42\times5/8}{8} = 22.7 - 3.28 = 19.42$ sq. in. reqd.

<u>Trial section</u>

		Gross		Net
2 angs.	@ 6x6x½	11.50	_4x½x1	9.50
2 Pls.	@ 13x½	13.00	_4x½x1	11.00
1 Pl.	@ 13x3/8	4.88	_2x3/8	4.13
		29.38		24.13

<u>Check on trial design</u>

$z = \frac{17.88(1.68+0.65)}{29.38} = 1.43"$

$h = 42.5 - 2(1.68 - 1.43) = 42.0"$

$s = 16000 - 150\frac{420}{13} = 11,600$

Net area $= \frac{1,243,125\times12}{16,000\times42} - 3.28 = 18.92$ □"

Gross area $= \frac{1,243,125\times12}{11,600\times42} - 3.28 = 28.5$ □"

<u>Design of Bed Plates</u>

206,200+52,500+5,057 = 263,758# Total gross reaction

Area of Bed Plates $= \dfrac{263,758}{24,000} = 11$ sq. in.

Area of bottom of bed plate

$$= \frac{263,758}{600} = 440 \text{ sq. in. or } 21 \text{ in. sq.}$$

$$\frac{440}{18} = 25''\text{x}18''$$

Pitch of flange rivets

$P = \dfrac{13,120\text{x}42}{258,700} = 2.13$ Space them in two staggering rows with a pitch of $2\frac{1}{4}''$ all the way across girder.

No. of rivets in end stiffeners and their size.

$= \dfrac{263,758}{13,120} = 20$plus Use 22 rivets

Size of stiffeners

Try 2 angs. @ 5x4x7/8

$$t = \frac{I}{A} = \frac{9.11}{4.37} = 1.43$$

$$s = 16,000 - 50\text{x}\frac{41.5}{1.43} = 14,550$$

$$A \text{ needed} = \frac{263,758}{14,550} = 18.1 \text{ sq. in.}$$

$$A \text{ available} = 17.5 \text{ sq. in.}$$

Spacing of intermediate stiffeners

$A = 72''$

$B = 42''$

c $\quad d = \dfrac{t}{40}(12,000 - S)$

$\quad d = \dfrac{5}{8\text{x}40}\left(12,000 - \dfrac{258,700}{5/8\text{x}42}\right) = 33.4''$

D $\quad \dfrac{V}{A} = 16,000 - 120\dfrac{d}{t}$

$\quad \dfrac{258,700}{42\text{x}5/8} = 16,000 - \dfrac{120d}{5/8}$

$\quad d = 34''$

No. of rivets to connect floor beam hitch angles to girder:

$\dfrac{61,500}{7,200} = 8.5$ and use rivets with (5) on each side, space $2\frac{1}{2}''$ apart and 1.2" from edges of hitch angles.

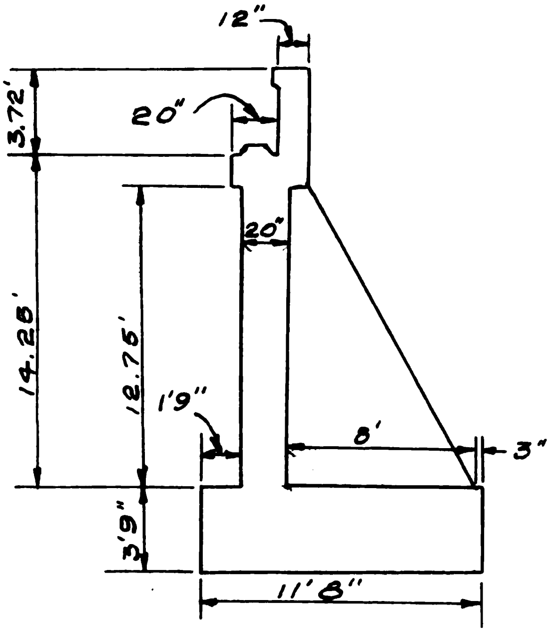
Use 5 rivets to fasten hitch angles to floor beam and use same spacing as above. Use 4"x4"x $\frac{1}{2}''$x12.4"long. Make intermediate stiffeners same size as end stiffeners; spaced alternate F-beams.

COMPUTATIONS

for

DESIGN OF

REINFORCED CONCRETE RETAINING WALLS

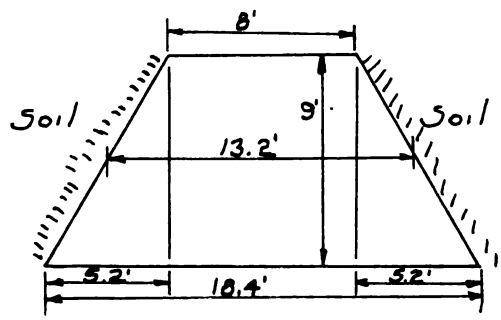


$P = (\frac{1}{2}wH^2 - \frac{1}{2}wh'^2)c = \left[\frac{1}{2}x100x(28.67)^2 - \frac{1}{2}x100x8.9^2\right]\frac{1}{4}$

$P = 9263\#$

$x = \frac{h^2 \ 3hh'}{3(h \ 2h')} = \frac{(17.97)^2 \ 3(17.97x8.9)}{3(17.97 \ 2x8.9)} = 7.3'$

Sketch showing way in which locomotive acts in distributing load on soil adjoining retaining wall



Average width of plane of weight distribution = 13.2'

Length of locomotive = 23.0'

23.0x13.2 = 304.0 sq. ft. over which the locomotive acts

$\frac{270,000}{304} = 890$ sq. ft.

$\frac{890}{100} = 8.9'$ height of surcharge.

$wh = 26.87 \times 25 \times 1 = 673$ lbs. per sq. ft.

$M = \frac{6.73 \times (10-1)^2}{10} = 5450$ ft. lbs. per ft. of width.

$d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{5450 \times 12}{94.4 \times 12}} = 7.6"$ Use 9" + 3" = 12" required.

$A_S = 0.0067 \times 12 \times 7.6 = 0.61$ sq. in. of steel per ft. of width

Use 5/8" round bars spaced 6" c-c, vertical steel in wall

next to fill and bent around in base

Short rods in wall back of counterforts shall be $53\frac{1}{2} \times 5/8 = 33.4"$

Total shear = $V = 673 \times 5 - 0.5 = 3020$ lbs.

The allowable shear for deformed bars is 5%

5%x2000 = 100# Use ½" deformed bars 4" c to c.

$u = \frac{3020}{\frac{12 \times 1.57 \times .88 \times 9}{4}} = 81.4\#$ which is allowable

Spacing of bars = $\frac{3}{4} \times 26.87 = 20.2$ or $\frac{1}{4}$ way up

$wh = 20.2 \times 25 = 505\#$ per sq. ft.

$V = 505 \times 5 - 0.5 = 1770$ lbs. Allowable = 100#

Use ½" deformed bars 7" c to c.

$u = \frac{1770}{\frac{12 \times 1.57 \times 0.88 \times 9}{6}} = 84.0\#$ which is O.K.

Spacing of bars at ½ point and up to top

$wh = 13.43 \times 25 = 336\#$ per sq. ft.

$V = 336 \times 5 - 0.5 = 1510\#$ Allowable = 100#

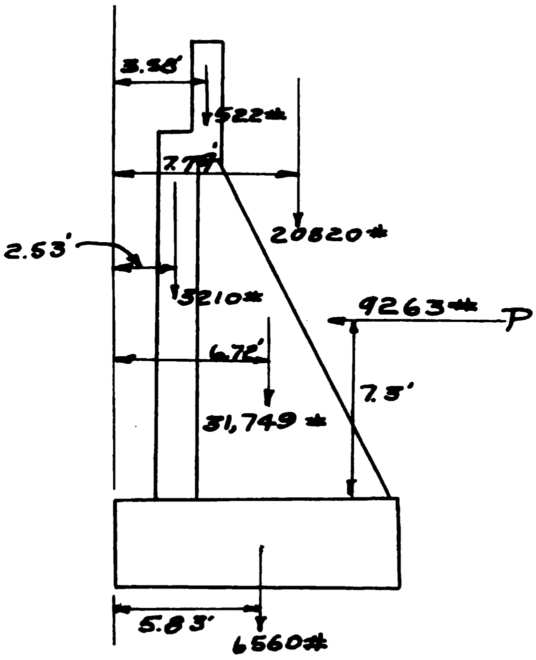
Use ½" cold rolled round bars; 9" c to c.

$u = \frac{1510}{\frac{12 \times 1.57 \times 0.88 \times 9}{9}} = 91\#$ and is O. K.

On outside wall place ½" round bars 24" c to c to take care of temperature stresses.

$$v = \frac{V}{bjd} = \frac{3020}{12x0.88x12} = 23.8$$ which is O.K. since 2% is the allowable or 40#

Design of footing



Factor of safety for overturning:

$$\frac{31,749x6.72}{9263x10.3} = 2.24 \quad \text{O.K.}$$

Factor of safety against sliding:

$$\frac{0.6x31,749}{9263} = 2.05 \quad \text{O.K.}$$

$$P = \frac{W}{t}\left(1 \pm \frac{6x_0}{t}\right), \quad x_0 = 1.41'$$

$$P_1 = \frac{31749}{11.67}\left(1 + \frac{6x1.24}{11.67}\right) = 4470\# \text{ per sq. ft.}$$

$$P_2 = \frac{31,749}{11.67}\left(1 - \frac{6x1.24}{11.67}\right) = 985\# \text{ per sq. in.}$$

Design of inner floor slab

$$\frac{985 + 1284}{2} = 1135\#$$

Earth pressure on 12" strip = 2687#
Wt. of concrete on 1' strip = 562#
Uniform load on 12" strip = 2687 + 562 - 1135 = 2114#

Max. bending moment in slab, considering it continuous:

$$M = \frac{wl^2}{10} = \frac{2114x(10-1)^2}{10} = 17,200\#$$

$$d = \frac{M}{Kd} = \frac{17200x12}{94.4x12} = 13.5" \quad \text{Total depth} = 17"$$

Depth of slab for shear

$$= 2114x(5-0.5) = 9,560\#$$

$$d = \frac{V}{bjd} = \frac{9,560}{12x0.88x24} = 38.2" \text{ Use } 45"$$

The thickness of slab is 45" for shear and not 17" as found for bending moment.

Place short rods under counterforts in top of slab.

$53\frac{1}{2}x1\text{-}1/8 = 60"$long, spaced 6" c to c.

The size of bottom rods for moment:

$$K = \frac{17,200x12}{12x42} = 41$$

$$P = \frac{K}{f_s j} = \frac{41}{16,000x0.88} = 0.0039$$

$A_s = 0.0039x12x42 = 1.96$ Use 1 1/8" round rods spaced 6" c to c and running parallel to vertical wall.

$$u = \frac{V}{ojd} = \frac{9560}{\frac{12x3.53x0.88x42}{6}} = 36.7\# \quad \text{Allowable} = 80\#$$

$$s = \frac{uVpjd}{v} = \frac{36.7x12x3.53x0.88x42}{9560} = 6" \quad \text{O.K.}$$

$\frac{4470-985}{11.67} = 299\#$ unit load is decreased at 2' from end.

$\frac{7x2114}{2114-299} = 8"$ spacing of top rods parallel to vertical wall

Length of top rods

$$p = \frac{0.99}{8x42} = 0.00295$$

$$j = 0.934$$

$$f_s = \frac{M}{A_s jd} = \frac{M}{bd^2 pj}$$

$fs = \dfrac{17{,}200x12}{12x(42)^2x.0029x.934} = 3610\#$ per sq. in.

$\dfrac{fi}{4u} = \dfrac{3610x1\ 1/8}{4x36.7} = 27.7"$

$v = \dfrac{9560}{12x.918x42} = 20.6$ which is O.K.

Outer cantilever:

Center of gravity $= \dfrac{1.75}{3} - \dfrac{1.75x2(3950) + 4470}{3950 + 4470} = 0.91$ft.

Moment due to vertical wall:

$\dfrac{(4470 = 3950)}{2}x1.75x0.91-1.75x3.75x1.50x\dfrac{1.75}{2}$

$= 6550-860 = 5690$ft. lbs.

The minimum depth to steel should be:

$d = \dfrac{5690}{94.4} = 24.6"$ and use 25"

$A_s = 0.0067x12x24.6 = 1.98$ sq. in. Use 1-1/8" round bars spaced 6" c to c.

$V = \dfrac{4470 + 3950}{2}$ x1.75x1.0-1.75x3.75x150 = 6380#

$d = \dfrac{6380}{12x42x0.88} = 14.4"$ Total depth 18"

$u = \dfrac{V}{ojd} = \dfrac{6380}{\frac{12}{6}x3.53x0.88x42} = 24.4\#$ O.K.

$P = \dfrac{0.994}{42x6} = 0.0039$

$j = 1-1/3k$

$k = \sqrt{2pn + (pn)^2} - pn \quad \sqrt{2x.0039x15 + (.0039x15)^2} - .0039x15$

$k = .288$

$j = .904$

$u = \dfrac{6380}{\frac{12}{6}x3.53x0.90x42} = 24\#$ Allowable is 80# or 4%

The stress in the steel is:

$$\frac{5,690 x 12}{.0039 x .90 x 12 x (42)^2} = 921 \text{ \# per sq. in.}$$

Length of embeddment necessary $= \frac{921 x 1\text{-}1/8}{4 x 24.4} = 10.5"$ on each side

Counterfort:

$\frac{1}{2} w h^2 w = \frac{1}{2} x 25 x (26.87)^2 x 10 = 90,400\#$

Bending moment $= 90,400 x \frac{26.87}{3} x 12 = 9,720,000$ in. lbs.

$K = \frac{M}{bd^2} = \frac{9,720,000}{16 x (8.94)^2 (12)^2} = 51.0$ $\qquad$ $p = .0035$

$A_s = pbd = .0035 x 12 x 16 x 8.94 = 6.00$ sq. in.

Use $1\frac{1}{4}"$ square bars spaced $3\frac{1}{2}"$ from sides of counterfort

and 3" c.to c. at 12" from top

$P = \frac{1}{2} x 25 x (21.87)^2 10 = 59,600$

$M = 59600 x \frac{21.87}{3} x 12 = 5,220,000$

$K = \frac{5,220,000}{16 x (8.94)^2 (12)^2} = 28.4$ $\qquad$ $p = .0019$

Shear at top of footing:

$25 x 26.87 x 8.67 = 5,820\#$

$\frac{5,820}{.196 x 16,000} = 1.85$ number of bars in a foot of height

$\frac{12 x 2}{1.85} = 13"$ apart

Shear at 3' from bottom of wall:

$25 x 23.87 x 8.67 = 5,180\#$ $\qquad$ $\frac{5,180}{.196 x 16,000} = 1.65$

$\frac{12 x 2}{1.65} = 14.5"$ spacing 3' above the footing

Spacing of vertical rods in counterforts:

$2114 x 8.67 = 18,350\#$ on 12" of length

$\frac{4470 + 985}{11.67} x 8.67 = 4,670\#$ the tension is decreased on each 12"

STEEL BAR LIST

SIZE	LENGTH	No. BARS	LBS.
Steel in Bottom of Footing:			
1-1/8"Φ	14' 3"	12	578
"	15' 9"	20	1066
"	17' 6"	16	947
"	22' 6"	6	456
"	25' 9"	10	870
"	29' 6"	8	798
1" Φ	5' 0"	220	2934
1" Φ	11' 0"	34	999
Steel in Bottom of Footing:			
1-1/8"Φ	5' 0"	72	1218
"	7' 9"	18	471
"	7' 3"	20	490
¾" Φ	5' 6"	248	2050
Steel in Counterforts and Vertical Wall			
¾" Φ	3' 0"	20	91
"	4' 6"	20	135
"	6' 6"	20	195
"	9' 0"	20	271
"	11' 6"	20	346
"	15' 0"	72	1622
"	3' 6"	72	378
"	13' 6"	56	1135
"	11' 6"	32	553
"	6' 6"	32	317
"	8' 6"	20	255
"	8' 0"	20	240
"	7' 3"	20	219
"	6' 9"	20	205
"	6' 0"	20	180
"	5' 0"	20	150
"	4' 3"	20	129
"	3' 6"	20	108
"	2' 6"	20	78
½" Φ	18' 0"	88	1058
"	31' 0"	44	911
5/8" Φ	6' 0"	176	1100
"	9' 0"	44	413
½" Φ	28' 0"	16	299
"	18' 0"	40	480
1-¼"sq.	19' 0"	24	2421
"	27' 9"	24	3540

Total weight of steel in both abutments 29,702#

EXCAVATION QUANTITIES FOR APPROACHES

STATION			LENGTH	END AREA	CU. YDS.
-6	plus	00		0	
-5	"	50	50	22.5	20.8
-5	"	00	50	56.0	72.7
-4	"	50	50	107.0	151.0
-4	"	00	50	179.5	264.6
-3	"	50	50	240.5	389.0
-3	"	00	50	336.0	535.0
-2	"	50	50	408.0	689.0
-2	"	00	50	401.0	748.0
-1	"	50	50	602.5	930.0
-1	"	00	50	677.0	1185.0
-0	"	50	50	735.0	1308.0
-0	"	15	35	652.5	900.0
0	"	00	15	652.5	362.0
0	"	15	15	652.5	362.0
0	"	50	35	716.5	888.0
1	"	00	50	679.5	1292.0
1	"	50	50	610.0	1193.0
2	"	00	50	531.5	1056.0
2	"	50	50	422.0	883.0
3	"	00	50	353.0	717.0
3	"	50	50	252.5	561.0
4	"	00	50	179.0	400.0
4	"	50	50	110.0	268.0
5	"	00	50	61.5	159.5
5	"	50	50	28.5	83.4
6	"	00	50	0	26.3
Total Yardage in excavation:--					15444.3

ABUTMENT CONCRETE QUANTITIES

	CU. FT..
Footing Concrete	2388.80
Vertical Wall	1621.00
Vertical Wall above bridge seat	115.54
Center Counterfort	239.62
2 Counterforts perpendicular to road	147.76
2 Counterforts oblique to road	136.68
Total	4649.40
For both Abutments	9298.80
Amount in cubic yards	344.40
Cost @ $26.87 per yd. (class A)	$9244.03

QUANTITY AND COST SHEET

29,702# reinforcement steel@ $.053 per lb.-------	$1,574.21
700' of 8" vit. tile @ 15¢ per ft.	105.00
720' of 6" vit. tile @ 11¢ per ft.	79.20
344.4 cu. yds. concrete(abutment & wingwalls)@26.87	9,244.03
32799# plate girder steel @$.0635 per lb.	2,082.74
15,445 cu. yds excavation @ $.46	7,104.70
280.44 cu. yds concrete for girder @ $26.87 yd.	7,515.42
~~890~~ ~~cu.~~ 36000 sq. yds of paving concrete for road @ $1.95 sq. ~~ft.~~ yd.	7,800.00
6 catch basins @ $83.51	501.06
TOTAL COST	$36,006.36