DESIGN

of a

REINFORCED CONCRETE GIRDER BRIDGE

with

PILE FOUNDATIONS

By

Karam Jabbur

May, 1951

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"This thesis submitted to the Civil Engineering Faculty in Partial fulfillment of the requirements for the degree of Bachelor of Science in Civil Engineering"

A.U.B.

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Karam Jabbur

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INTRODUCTION

Long span highway girder bridges have never been constructed in this country. Most highway bridges are masonry arches and short span girders. There are very few reinforced concrete arch bridges.

Pile foundation is rarely resorted to here due to the lack of proper equipment . The occasional small jobs do not warrant the purchase of expensive equipments. For large constructions on the sandy and swampy suburbs of Beirut and other regions in this country, pile foundations will be most economical and in certain cases a necessity.

Information about the design of piles is very much limited. For, the ultimate dimensions and bearing capacity of a pile is determined by test rather then by theoretical design. The available design formulas are imperical with rational basis.

Choice of type of bridge

There are three usual types of reinforced concrete bridges:

1. Slab bridge
2. Through bridge
3. T-Beam or Deck Girder Bridge

The first type is adapted for small spans up to 20 ft. . The second type is most economical when the width of the bridge is 20 or 25 ft. It consists of a deck slab supported on cross beams which rest on two main longitudenal girders. Some objections to this type of bridge are:

a. The whole of the load would be carried twice before it reaches the vertical suppot.

b. No continuity in the transverse beams, with consequent loss of economy and difficulty of obtaining satisfactory support for these members.

c. Concentration of the load from the bridge at the abutment under the bridge.

Moreovere a through bridge is bulkky and the appearence of heaviness should be avoided in concrete highway bridges. The architectural treatment of a bridge is constantly apparent , while its virtues or faults are likely to pass unnoticed.

The third type, T-beam or deck girder, is the most economical for this particular case. The width is 45 ft. and sufficient head room is available.

Specifications

Loading - French System of Loading for Highway Bridges*

Impact = $\frac{50}{L + 125}$ L = Span in ft.

f_s = 18,000 psi.

f'_c = 2,500 psi.

f_c = $0.4 f'_c$

n = 12

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* See Appendix. B

CHAPTER I

Design of the Bridge

Deck Slab.

The roadway is 10 meters or 33 ft., since the existing road leading to the bridge is 10 m. wide. The sidewalks are 1.8 m. or 6 ft. each. The total clear roadway is, therefore,45 ft.

The lateral spacing of longitudenal beams should lie between 6 - 10 ft. being fixed by the economical thickness of the slab which is less then 10 in.. On the other hand, the beams should be spaced as far apart as possible since the maximum wheel loads to be carried by each is the same no mater how close they are. The beams in this case are spaced 7.5 ft. which is adequate.

Since the parapet beams are heavy in comparison with the floor slab and since all beams and slabs will be poured monolithically and reinforced by diaphrams, the inclination under bending of the top of the slab across the width of the beam will be constant. Hence the span of the slab could be taken as the clear distance between suporting beams.

Assume the width of the beams to be 18 in., the clear span of the slab is 7.5 - 1.5 = 6 ft. Moreover due to continuity in the slab the maximum moment is reduced by 20 % according to the American Association of State Highway Officials specifications.

Thickness of road metal is 4 in.

Assume thickness of concrete 6 in.

Weight of road metal	4/12 x 110 -----------	36.70
Weight of concrete	6/12 x 150 -----------	75.00
		111.70 psf.

Dead load moment

$1/8 \times 112 \times 6^2 \times 12 \times 0.8 = 4850$ in. lbs.

Live load moment (uniform load)

$P = (824 - 4 \times 20) \times 2.2/10.75 = 153$ psf.

$M = 1/8 \times 153 \times 6^2 \times 12 \times 0.8 = 6600$ in. lbs.

Live load moment (concentrated load)

The distribution of wheel loads is according to the specification given on page 478 of the Design of Concrete Structure book by Urquhart and O'Rourke. $E = 0.7\ (2D + T)$

where E = effective width in feet for one wheel.

D = distance in feet from the center of the near support to the center of the wheel.

T = width of wheel or tire in feet.

$E = 0.7\ (2 \times 7.5/2 + 1.67) = 6.3$ ft.

$13200/6.3 = 2100$ lbs. on one ft. strip.

$M = 0.8\ (2100/2 \times 6/2 \times 12) = 30{,}300$ in.lbs.

$$\text{Impact} = \frac{50}{7.5 + 125} = 0.38 \text{ or } 40\ \%$$

Total moment

$30{,}300 + 12{,}100 + 4{,}850 = \underline{47{,}250}$ in.lbs.

Concentrated load moment governs.

$$d = \sqrt{\frac{47{,}250}{12 \times 173}} = 4.76 \text{ or } 5 \text{ in.}$$

$$A_s = \frac{47{,}250}{18{,}000 \times 0.87 \times 5} = 0.605 \text{ sq. in.}$$

Use 5/8 in. round bars $A_s = 0.305$ sq.in. $12/0.605/0.305 = 6"$ spacing..

Therefore use 5/8" round bars 5" spacing .

The total depth of the slab is 5" plus 1" insulation below the center of the bars equals 6" as assumed.

Check for shear and bond.

Dead load shear .... 112 x 6/2 = 336

Live load shear ....13200/6.3 = 2100

Impact ....2100 x 0.40 = 840

Total shear ..... 3276 lbs.

$$v = V/bjd = \frac{3280}{12 \times 0.87 \times 5} = 73 \text{ psi.}$$

Allowable 75 lbs./sq.in.

$$U = V/E_0 jd = \frac{3280 \times 5/12}{1.964 \times 0.87 \times 5} = 155 \text{ psi.}$$

Allowable 100psi.

The bond stress is high, but as the bars run straight through over the support in the top of the slab and the high bond stress occurs only for a short distance along the bar, and additional bars are provided in the top of the slab, it may be considered safe.

Slab Carrying the Foot-way.

The effective span of the footway is 0.75 ft. more than the deck slab span i.e. 6.75 ft.. But the bending moment to be resisted is much less. For a uniform load of 150 psf. the total bending moment is 13,350 in. lbs.. This slab will have the same depth and reinforcement as the deck slab for two main reasons: (1) Uniformity, the same bars run transversly and are anchored to the parapet beams. (2) Possibility of a truck or vehicle mounting the curb. Every other bar is bent up over the support and additional 5/8 in. round bars 10 in. center to center are placed in the top of the slab from one parapet beam to the other to take care of the negative bending moment. All the bars are anchored to the parapet beams by a standard hook.

Temperature and distribution stresses in the direction of the span are provided for by 2 rows of 1/2 in. round bars 12 in. c. to c. in the top and bottom of the slab. In each slab panel 5 bars are placed under the top bars and 5 over the bottom bars.

Roadway Beams

These are T-beams with flange width of 7.5 ft., the distance c. to c. of beams. The bridge seat is assumed to be 2 ft. wide and the effective span is 65.5 + 2 = 67.5 ft.

Dead load moment

Slab .... 6/12 x 150 = 75 psf.

Wearing surface ... 4/12 x 110 = 37 "

112 x 7.5 = 840 lbs./ft.

Assume the depth of the beam

below the slab 52 in. 1.5x(52/12)x150=975

1815 lbs./ft.

Maximum moment

$M_{max.} = 1/8 \times 1815 \times 67.5^2 \times 12 = 12,400,000$ in. lbs.

Moment at 10 ft. from the support.

$M_{10} = 1815 \times 67.5/2 \times 10 - 1815 \times 10^2/2 = 6,800,000$ in. lbs.

Moment at 25 ft. from the support

$M_{25} = (1815 \times 67.5/2 \times 25 - 1815 \times 25^2/2)\ 12 = 11,550,000$ in.lbs.

Live load moment (Uniform load)

153 x 7.5 = 1150 lbs./ft.

$M_{max} = 1/8 \times 1150 \times 67.5^2 \times 12 = 7,900,000$ in.lbs.

$M_{10} = (1150 \times 67.5/2 \times 10 - 1150 \times 10^2/2)\ 12 = 3,970,000$ in.lbs.

$M_{25} = (1150 \times 67.5/2 \times 25 - 1150 \times 25^2/2)\ 12 = 7,310,000$ in.lbs.

Impact moment

$$\frac{50}{67.5 - 125} = 0.26 \text{ or } 26\%$$

M_{max} = 7,900,000 x 0.26 = 2,060,000 in.lbs.

M_{10} = 3,970,000 x 0.26 = 1,030,000 "

M_{25} = 7,310,000 x 0.26 = 1,930,000 "

Live load moment (concentrated load)

The distributinn of wheel loads is according to the specification given on page 478 of the Design of Concrete Structure book by Urquhart and O'Rourke. The interior beams sustain S/4.5 wheel loads. (S = spacing of beams)

7.5/4.5 = 1.665 wheel loads

Rear wheel .... 13200 x 1.665 = 22,000 lbs.

Front wheel .... 4400 x 1.665 = 7,340 "

Position of maximum moment (Fig. 1)

22,000 x 13.1 + 7340 x 33,1 + 22,000 x 46.2 = 1,551,000 in.lbs.

1,551,000/(22,000+7,340)2 = 26.5 ft.

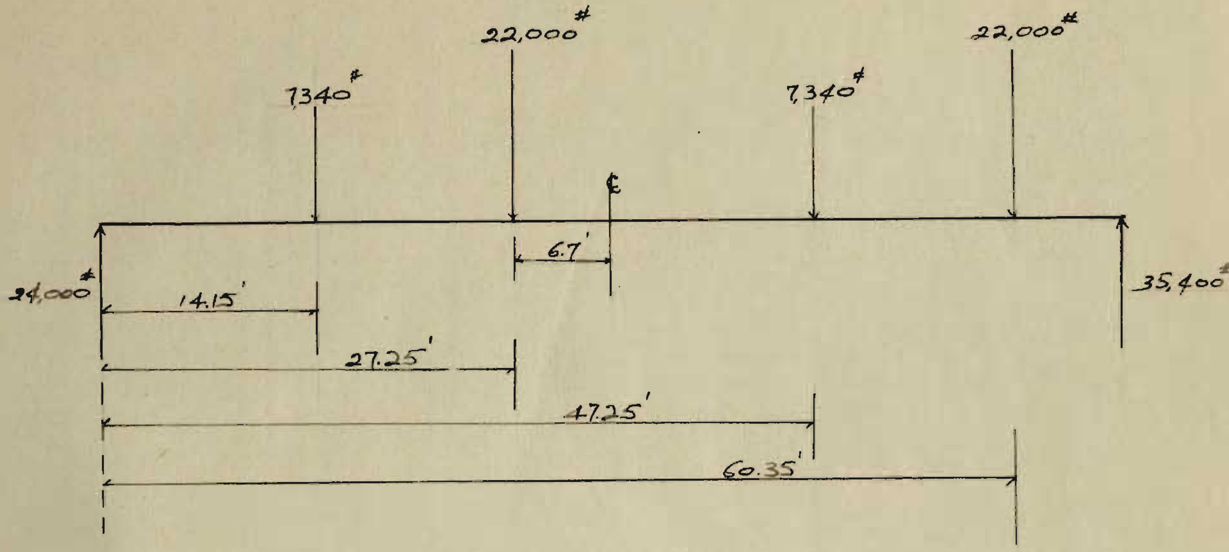
$\frac{26.5-13.1}{2}$ = 6.7 ft. distance, from the center, of the load which produces maximum moment.

$M_{max.}$ =(24,000 x 27.25 - 7340 x 13.1)12 = 6,700,000 in.lbs.

Moment at 10 ft. from the support occurs when the big wheel is at that point. (Fig. 2)

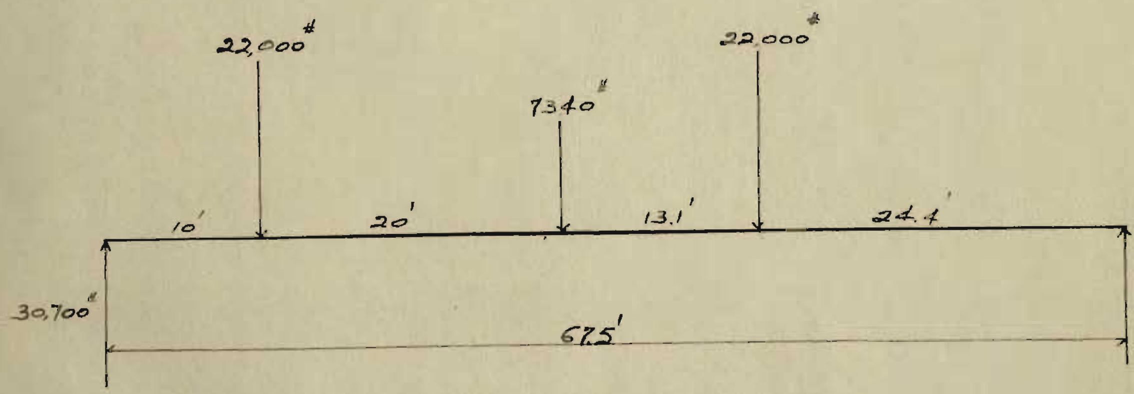
M_{10} = 30,700 x 10 x 12 = 3,680,000 in.lbs.

The uniform load moment governs.



Position of Max. Moment

Fig. 1



Max. Moment at 10' from the support.

Fig. 2

Total moment

	D.L.		L.L.		Impact		
$M_{max.}$ =	12,400,000	+	7,900,000	+	2,060,000	=	22,360,000 in.lbs.
M_{10} =	6,800,000	+	3,970,000	+	1,030,000	=	11,800,000 "
M_{25} =	11,550,000	+	7,310,000	+	1,930,000	=	20,790,000 "

Dead load shear

Maximum shear .... 1815 x 67.5/2 = 61,250 lbs.

Shear at 10' from the support 61,250 - 1815 x 10 = 43,100 lbs.

Shear at 25' from the support 61,250 - 1815 x 25 = 15,850 lbs.

Live load shear (uniform load)

Maximum shear occurs near the support when all the beam is loaded

1150 x 67.5/2 = 38,800 lbs.

Shear at 10' from the support occurs when the beam is loaded up to that point.

$$\frac{1150 \times 57.5^2}{67.5 \times 2} = 28{,}000 \text{ lbs.}$$

Shear at 25' from the support occurs when the beam is loaded up to that point.

$$\frac{1150 \times 42.5^2}{67.5 \times 2} = 15{,}350 \text{ lbs,}$$

Maximum shear at the center occurs when the beam is loaded up to the center.

$$\frac{1150 \times 33.75^2}{67.5 \times 2} = 9{,}700 \text{ lbs.}$$

Live load shear (concentrated load)

Consider 2 trucks moving side by side (Fig. 3)

Wheel loads coming on the beam 1 + 5/7.5 + 1.18/7.5 = 1.86

Wheel loads: rear wheel 13200 x 1.86 = 24,600 lbs.

front wheel 4400 x 1.86 = 8,200 "

Maximum shear at the support occurs when the big wheel is at the support. (Fig. 4)

shear = 46,300 lbs.

Maximum shear at 10' from the support occurs when the big wheel is at the section. (Fig. 5)

shear = 36,600 lbs.

Maximum shear at 25' from the support occurs when the big wheel is at the section. (Fig. 6)

shear = 22,500 lbs.

Maximum shear at the center occurs when the big wheel is at the center. (Fig. 7)

shear = 15,050 lbs.

Concentrated load governs.

Impact shear

Max. 46,300 x 0.26 = 12,050 lbs.

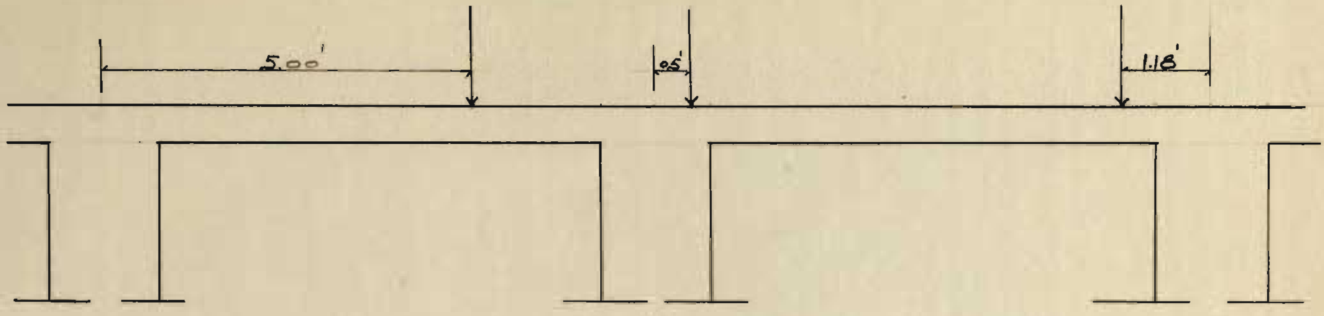
at 10' 36,600 x 0.26 = 9,500 lbs.

at 25' 22,500 x 0.26 = 5,850 "

at the center 15,050 x 0.26 = 3,910 lbs.

Total shear

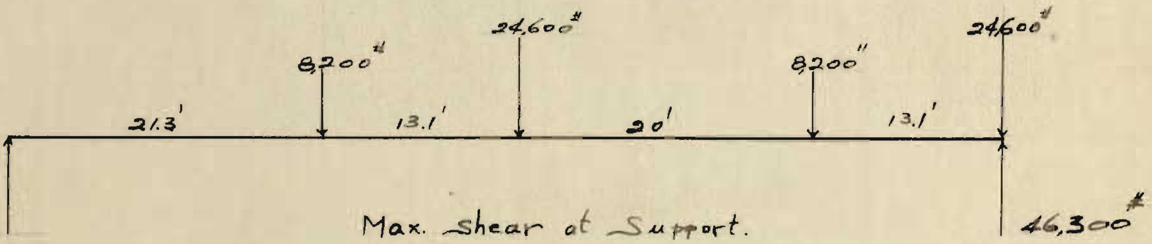
	D.L.		L.L.		Imp.		
Max.	61,250	+	46,300	+	12,050	=	119,600 lbs.
at 10'	43,100	+	36,600	+	9,500	=	89,200 "
at 25'	15,850	+	22,500	+	5,850	=	44,200 "
at center	-	+	15,050	+	3,910	=	18,960 "



Wheel Loads coming on beam.

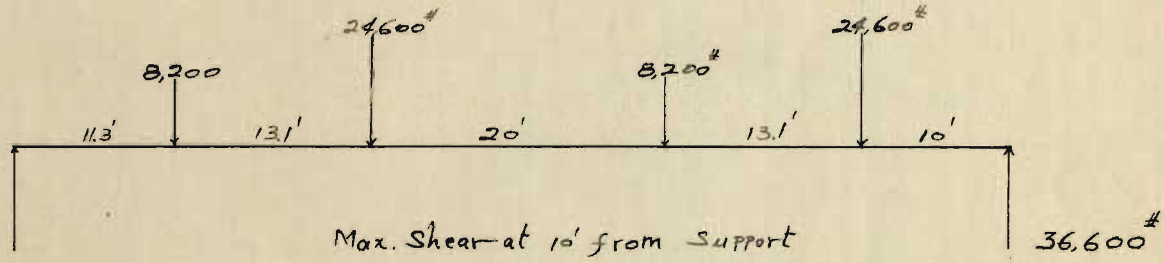
Fig. 3.

Fig. 4



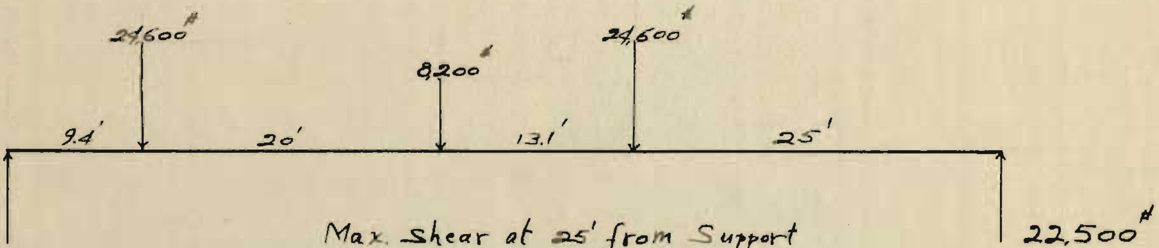
Max. shear at Support.

Fig. 5



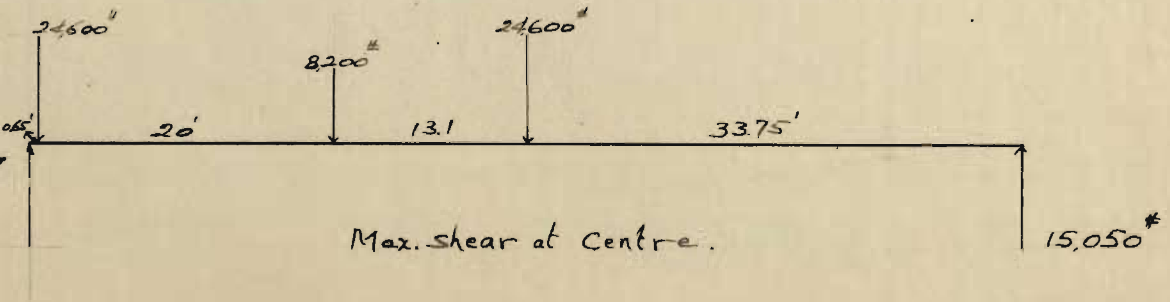
Max. Shear at 10' from Support

Fig. 6



Max. shear at 25' from Support

Fig. 7



Max. shear at Centre.

Determination of cross-section

The area required to sustain maximum shear

$$bd = \frac{119,600}{0.87 \times 150} = 918 \text{ sq.in.}$$

$$d = 918/18 = 51 \text{ in.}$$

Suppose 3 rows of steel are used 3in. c.to c. and 3 in. insulation from the center of the lower row, the depth of the beam below the slab is: 51 + 6 - 6 = 51 in.

$$A_s = \frac{22,360,000}{18,000\ (51 - 6/2)} = 26 \text{ sq. in.}$$

$$f_c = \frac{2M}{bt\ (d - 0.5t)} =$$

$$f_c = \frac{2 \times 22,360,000}{7.5 \times 12 \times 6(48)} = 1720 \text{ psi.}$$

The beam should be reinforced for compression. The concrete sustains 1000 psi. There is a stress of 1720 - 1000 = 720 psi. to be resisted by steel under compression.

Moment resisted by concrete

$$M = 1/2\ f_c\ bt(d - 0.5t)$$

$$M = 1/2 \times 1000 \times 6 \times 7.5 \times 12\ (51 - 3) = 12,950,000 \text{ in.lbs.}$$

Moment to be resisted by compressive steel

22,360,000 - 12,950,000 = 9,410,000 in.lbs.

Suppose 2 rows of steel are used 2.5 in. c. to c. and 2.5 in. insulation, d = 51 - 3.75 = 47.25 in.

$$A_s' = M/f_s\ d = \frac{9,410,000}{16,000 \times 47.25} = 12.45 \text{ sq.in.}$$

For tenssion use 15 round bars 1.5 in. A_s = 15 x 1.765 = 26.5 sq.

For compression use 10 round bars 1.25 in. A_s = 12.30 sq. in.

Web reinforcement

Shear carried by concrete

$V = vbjd = 75 \times 18 \times 0.87 \times 50 = 58{,}500$ lbs. (1)

Stirups spacing

$$S = \frac{A_v F_v jd}{V - V_e}$$ Use 3/8 in. round bars for stirups

At the support $S = \dfrac{10 \times 0{,}11 \times 18{,}000 \times 0.87 \times 51}{119{,}600 - 58{,}500} = 15$ in.

At 10' $S = \dfrac{10 \times 0{,}11 \times 18{,}000 \times 0.87 \times 51}{89{,}200 - 58{,}500} = 30$ in.

Bond

$$\Sigma_o = \frac{V}{ujd} = \frac{119{,}600}{100 \times 0.87 \times 51} = 27 \text{ in.} \quad (2)$$

$\dfrac{27}{1.5 \times 3.14} =$ 7 bars 1.5 in. are required for bond

Therefore, 7 bars are bent up leaving 15 - 7 = 8 bars to take care of bond stresses.

Camber

The camber to form the crown will be 4 in., 2 in. are formed by varying the depth of the beams below the slab. Thus the depth of the middle beam (0) is made 52 in. , the depth of beam (1) 51 in. and the depth of beam (2) 50 in. The other 2 in. are formed by varying the thickness of the wearing surface.

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(1) v = 75 & (2) u = 100 psi. Working stresses recommanded by the Joint Code.
Page 515, Design of Concrete Structures by Urquhart & O'Rourke.

Parapet Beam

Dead load moment

Load from the slab . . . . 6/12 x 150 = 75

Sidewalk . . . . 8/12 x 110 = 73

148 x 7.5/2 = 555 lbs./ft.

Parapet . . . . 8/12 x 4 x 150 = 400 "

Beam . . . . 66/12 x 20/12x 150 = 1375

2330 lbs./ft.

$M_{max.} = 1/8 \times 2330 \times 67.5^2 \times 12 = 16,000,000$ in.lbs.

$M_{10} = (2330 \times 67.5/2 \times 10 - 2330 \times 10^2/2)12 = 8,040,000$ in.lbs.

$M_{25} = (2330 \times 67.5/2 \times 25 - 2330 \times 25^2/2)12 = 14,900,000$ in.lbs.

Live load moment

The live load on the footway is considered to be the same as that on the deck slab to take care of the possibility of a vehicle mounting the curb.

153 x 7.5/2 = 575 lbs/ft.

$M_{max.} = 1/8 \times 575 \times 67.5^2 \times 12 = 3,930,000$ in.lbs.

$M_{10} = (575 \times 67.5/2 \times 10 - 575 \times 10^2/2)12 = 1,980,000$ in.lbs.

$M_{25} = (575 \times 67.5/2 \times 25 - 575 \times 25^2/2)12 = 3,660,000$ in.lbs.

Impact moment

$M_{max.} = 3,930,000 \times 0.26 = 1,015,000$ in.lbs.

$M_{10} = 1,980,000 \times 0.26 = 515,000$ in.lbs.

$M_{25} = 3,660,000 \times 0.26 = 950,000$ in.lbs.

Total moment

$M_{max.} = 16,000,000 + 3,930,000 + 1,015,000 = 20,945,000$ in.lbs.

$M_{10} = 8,040,000 + 1,980,000 + 515,000 = 10,535,000$ in.lbs.

$M_{25} = 14,900,000 + 3,660,000 + 950,000 = 19,510,000$ in.lbs.

Dead load shear

Maximum .... 2330 x 67.5/2 = 78,500 lbs.

At 10' from the support 78,500 - 2330 x 10 = 55,500 lbs.

At 25' from the support 78,500 - 2330 x 25 = 20,500 lbs.

Live load shear

Maximum .... 575 x 67.5/2 = 19,400 lbs.

At 10' from the support 575 x 57.5^2/(67.5x2) = 14,100 lbs.

At 25' from the support 575 x 42.5^2/(67.5x2) = 7,800 lbs.

At the center .... 575 x 33.75^2/(67.5x2) = 4,860 lbs.

Impact shear

Maximum .... 19,400 x 0.26 = 5,050 lbs.

At 10' from the support 14,000 x 0.26 = 3,660 "

At 25' from the support 7,800 x 0.26 = 2,030 "

At the center .... 4,860 x 0.26 = 1,260 "

Total shear

Maximum 78,500 + 19,400 + 5,050 = 102,950 lbs.

At 10' 55,500 + 14,100 + 3,660 = 73,260 lbs.

At 25' 20,500 + 7,800 + 2,030 = 30,330 lbs.

At center - 4,860 + 1,260 = 6,120 lbs.

This beam is designed as a rectangular beam since it projects above the deck slab. The total depth of the beam is 66 in.

Moment resisted by concrete $M = Kbd^2$

Suppose 3 rows of steel are used at 2.5" c. to c. and 2.5" insulation.

d = 66 - 5 = 61 b = 20 K = 173 (3)

$M = 173 \times 20 \times 61^2$ = 12,900,000 in.lbs.

(3) Table 6. Page 529 Design of Concrete Structures by Urquhart & O'Rourke

Moment to be resisted by compressive steel

$M_2 = 20,945,000 - 12,900,000 = 8,045,000$ in.lbs.

$$A_s = \frac{12,900,000}{18,000(61 \times 0.87)} = 13.50 \text{ sq.in.}$$

Additional tensile steel to resist moment M_2

$A_{s2} = M_2/(d-d')f_s$

Suppose 2 rows of compressive steel are used 2.5 in. c. to c. and 2.5 in. insulation. $d' = 2.5 + 1.25 = 3.75$ in.

$$A_{s2} = \frac{8,045,000}{18,000(61-3.75)} = 7.80 \text{ sq.in.}$$

Total tensile steel

$A_s = 7.80 + 13.50 = 21.30$ sq.in.

Use 15 round bars 1 & 3/8 in. $A_s = 22.20$ sq.in.

Compressive steel

$A'_s = A_{s2} \times (1-k)/(k-d'/d)$ $\quad k = 0.4$ (4)

$A'_s = 7.8 \times (1-0.4)/(0.4-3.75/61) = 13.80$ sq.in.

Use 10 round bars 1 & 3/8 in. $A_s = 14.85$ sq.in.

Web reinforcement

Shear carried by concrete

$V = 75 \times 20 \times 0.87 \times 61 = 80,000$ lbs.

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(4) Table 6. Page 529 Design of Concrete Structures by Urquhart & O'Rourke

Stirups spacing

$$s = \frac{A_v f_v jd}{V-V_c}$$

Use 3/8 in. round bars for stirups $A_s = 0.11$ sq.in.

$$s = \frac{10 \times 0.11 \times 18,000 \times 0.87 \times 61}{102,950 - 80,000} = 45 \text{ in.}$$

Bond

$$E_o = \frac{V}{ujd} = \frac{102,950}{100 \times 0.87 \times 61} = 19.4 \text{ in.}$$

$$\frac{19.4}{1.375 \times 3.14} = 5 \text{ bars required for bond at the bottom.}$$

Therefore, 7 - 1 & 3/8 in. round bars are bent leaving 8 bars at the bottom.

CHAPTER II

Design of the Abutment

The abutment will be of the mass type constructed with cyclopean concrete (1) and faced with stone masonry. Since the river is small and the scouring effect of the water is not excessive , the wings are tied to the body of the abutment and the whole structure is considered as one unit.

The surcharge on the approach to the abutment is the same as the uniform load on the bridge plus the weight of the road material. 153 + 40 = 193 or 200 psf.

The soil which the abutment is supposed to retain is sand and gravel.

Weight = 100 lbs. per cub. ft.

Angle of Repose = 33.5°

Friction Angle = 33.5°

Hight of abutment = 21 ft.

Length of abutment

Bridge	45.00
Parapet beams	3.50
Clearence	0.50
Side walls	2.00
	51.00 ft.

(1) Concrete in which rubble stones are immersed. The proportion is 70% concrete & 30% rubble stones.

The load on the abutment from the bridge.

Dead load and live load

$102,950 \times 2 + 119,600 \times 5 = 804,000$ lbs.

Dead load of the bridge only

$78,500 \times 2 + 61,250 \times 5 = 463,250$ lbs.

Assume the width of the base of the abutment to be 11 ft.

Weight of the abutment

$w = (11 \times 3 + 2.5 \times 13/2 + 5.5 \times 13 + 1 \times 5 + 2)140 \times 51 =$

$130.75 \times 140 \times 51 =$ 935,000

Wings $2(11 \times 5/2 \times 3 + 13 \times 4/2 \times 1/3 \times 13)140 =$ 54,000

989,600 lbs.

$x = 1/130.75(33 \times 5.5 + 16.25(2+2.5 \times 2/3) + 71.5 \times 7.5 + 5 \times 7.5 + 5(7.5-2/3) = 6.42$ ft.

$w' = 1 \times 18 \times 100 \times 51 = 92,000$ lbs.

$x' = 11 - 0.5 = 10.50$ ft.

$w'' = 2 \times 5/2 \times 100 \times 51 = 25,500$

$2 \times 2 \times 51 \times 100 = 20,400$

45,900 lbs.

$x'' = 11 - 1 - 2/3 = 9.35 - 0.1 = 9.25$ ft.

$P = 1/2\, wh(h+2h')k$ (2)

$P = 1/2 \times 100 \times 21(21 + 2 \times 2)0.29\ (51 + 5) = 425,000$ lbs.

$y = (h^2+3hh')/(3h+2h')$ (3)

$$y = \frac{21^2 + 3 \times 21 \times 2}{3(21 + 2 \times 2)} = 7.55 \text{ ft.}$$

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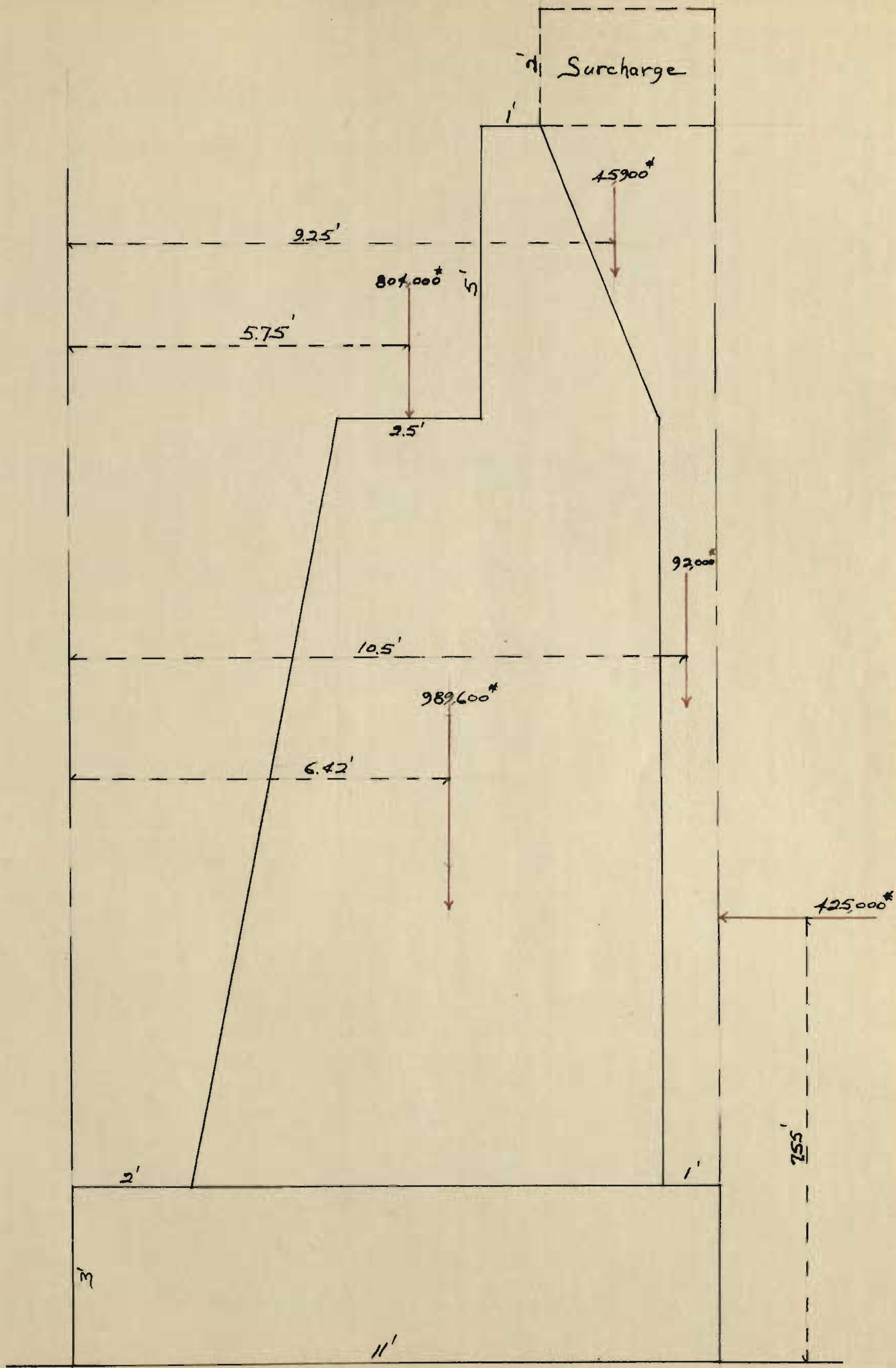
(2) & (3) Page 396, Design of Concrete Structures by Urquhart & O'Rourke



Surcharge
2'
1'
45900*
9.25'
807,000*
5'
5.75'
2.5'
92,000*
10.5'
989,600*
6.42'
425,000*
25.5'
2'
1'
3'
11'

Fig. 1

The stability of the abutment is investigated for three possible cases.

a) When there is no load from the bridge on the abutment.

b) When there is only the dead load of the bridge on the abutment.

c) When there is dead load and live load on the abutment.

Case (a)

Overturning $$\frac{989,600 \times 6.42 + 92,000 \times 10.5 + 45,900 \times 9.25}{425,000 \times 7.55} =$$

$$\frac{7,740,000}{3,210,000} = 2.41 \quad \text{Safe}$$

Sliding $$\frac{989,600 + 92,000 + 45,900}{425,000} = 2.65$$

The coefficient of friction is rather high, since the steel bars from the head of the piles will project into the base of the abutment. Consider that the coefficient of friction $u = 0.6$

$2.65 \times 0.6 = 1.6$ Safe

Position of the resultant at the base

$$d = \frac{M_a}{R_v} = \frac{7,740,000 + 3,210,000}{1,127,500} = 4.02 \text{ ft.}$$

$$4.02 - 11/3 = 0.35 \text{ ft.}$$

The resultant falls in the middle third of the base.

Case (b)

Overturning $$\frac{7,740,000 + 463,250 \times 5.75}{425,000 \times 7.55} =$$

$$\frac{10,400,000}{3,210,000} = 3.23 \quad \text{Safe}$$

Sliding $\frac{1,127,500 + 463,250}{425,000} \times 0.6 = 2.25$ Safe

Position of the resultant at the base

$$d = \frac{10,400,000 - 3,210,000}{1,590,750} = 4.52 \text{ ft.}$$

$4.52 - 3.67 = 0.85$ ft.

The resultant falls in the middle third.

Case (c)

Overturning $\frac{7,740,000 + 804,000 \times 5.75}{3,210,000} =$

$$\frac{12,360,000}{3,210,000} = 3.87 \quad \text{Safe}$$

Sliding $\frac{1,127,500 + 804,000}{425,000} \times 0.6 = 2.7$ Safe

Position of the resultant at the base

$$d = \frac{12,360,000 - 3,210,000}{1,931,500} = 4.75 \text{ ft.}$$

$4.75 - 3.67 = 1.08$ ft.

The resultant falls in the middle third.

Eccentricity of the resultant from the center line of the base.

$5.50 - 4.75 = 0.75$ ft.

Pressure distribution at the base of the abutment.

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

$P = 1,931,500$ lbs.

$A = 51 \times 11 = 560$ sq.ft.

$M = 1,931,500 \times 0.75 = 1,450,000$ lbs.

$c = 11/2 = 5.5$ ft.

$I = 51 \times 11^3/12 = 5650$ ft.^{4}

$$S = \frac{1,931,5000}{560} \pm \frac{1,450,000 \times 5.5}{5,650}$$

$3,450 + 1,410 = 4,860$ psf.

$3,450 - 1,410 = 2,040$ psf.

Three rows of piles will be used. The pressure diagram is divided into three equal areas. (Fig. 2). Thus the load is distributed equally over the piles.

Area of the pressure diagram

$1/2 \times 11(4,860 + 2,040) = 38,000$ sq. units.

Each area should be $38,000/3 = 12,650$ sq. units

Let "h" be the hight of the trapezoidal area along the base of the abutment.

$A_1 = 1/2\ h(2,040 + 2,040 + 256h) = 12,650$

$h = 4.75$ ft.

$A_2 = 1/2\ h(2,040 + 256 \times 4.75 + 2,040 + 256h + 256 \times 4.75) = 12,650$

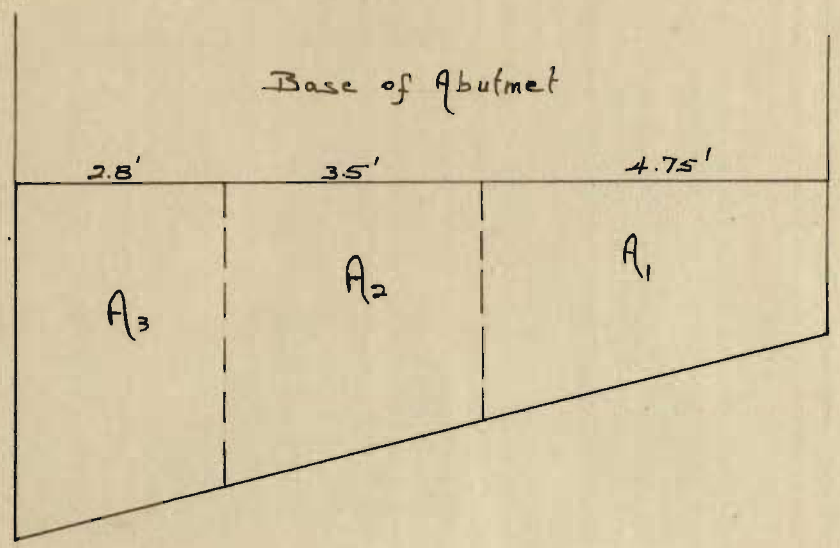
$h = 3.50$ ft.

$A_3 = 1/2\ h(4,860 + 4,860 - 256h) = 12,650$

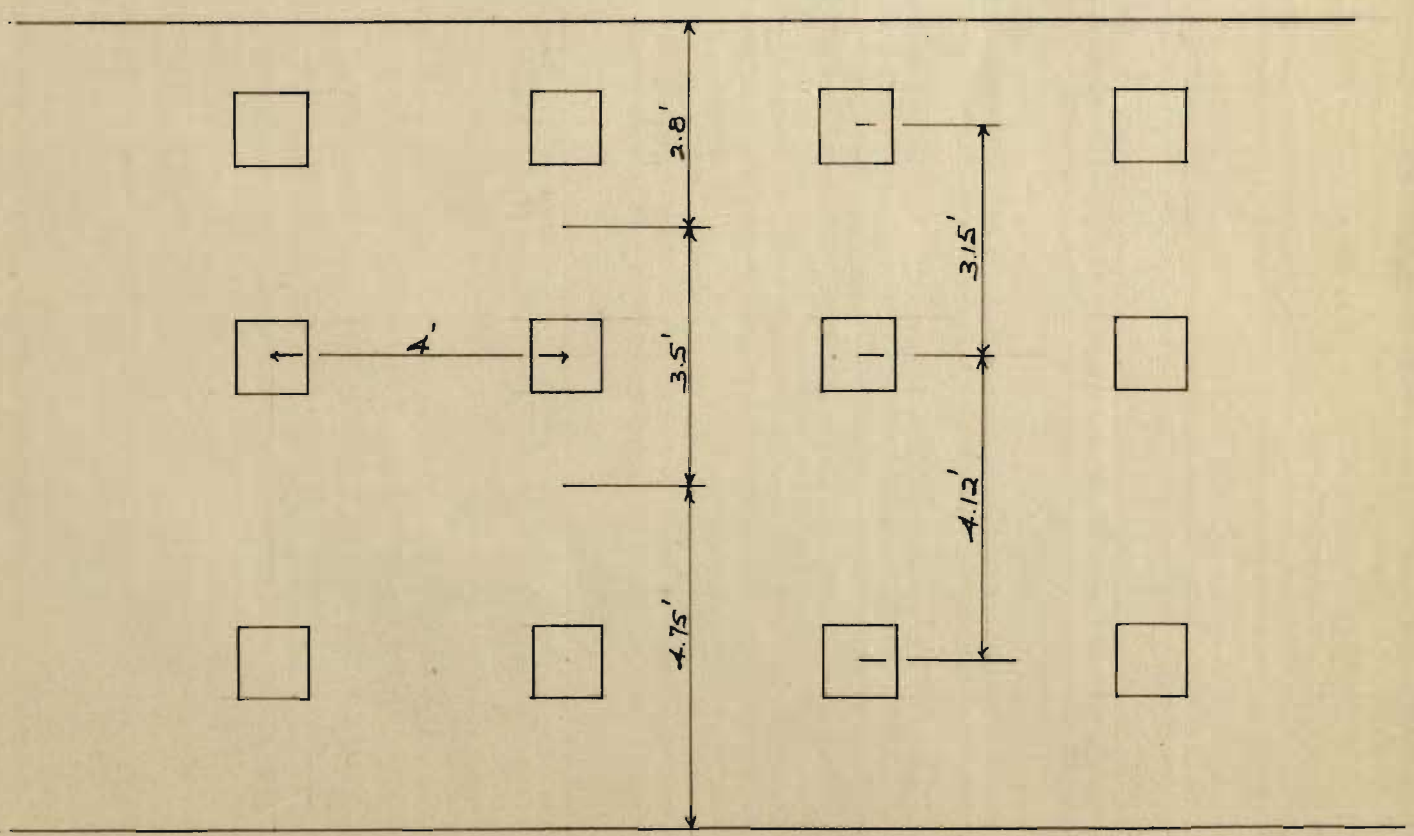
$h = 2.80$ ft.

The piles will be spaced 4ft. longitudenally (Fig.3).

Hence each pile carries $4 \times 12,650 =$ <u>50,600 lbs.</u>



Pressure Diagram
Fig. 2



Position of Piles
Fig. 3

Check for the backwall of the abutment (Fig.4)

$W = (1 \times 5 + 2 \times 5/2)140 \times 51 = 71{,}400$ lbs.

$x = 1/10\ (5 \times 0.5 + 5 \times 1.66) = 1.084$ ft.

$W' = 2 \times 5/2 \times 51 \times 100 = 25{,}500$

$2 \times 2 \times 51 \times 100 = \underline{20{,}400}$

$45{,}900$ lbs.

$x' = 1 + 2/3 \times 2 = 2.33 - 0.10 = 2.23$ ft.

$P = 1/2 \times 100 \times 5(5 + 2 \times 2)0.29 \times 51 = 33{,}200$ lbs.

$$y = \frac{5^2 + 3 \times 5 \times 2}{3(5 + 2 \times 2)} = 2.04 \text{ ft.}$$

Overturning $\dfrac{71{,}400 \times 1.084 + 45{,}900 \times 2.28}{33{,}200 \times 2.04} =$

$$\frac{185{,}500}{67{,}800} = 2.72 \quad \text{Safe}$$

Sliding $\dfrac{71{,}400 + 45{,}900}{33{,}200} \times 0.6 = 2.14$ Safe

Position of the resultant at the base

$$d = \frac{185{,}500 - 67{,}800}{117{,}300} = 1.002 \text{ ft.}$$

The resultant falls at the third of the base.

Crushing $S = \dfrac{P}{A} \pm \dfrac{Mc}{I}$

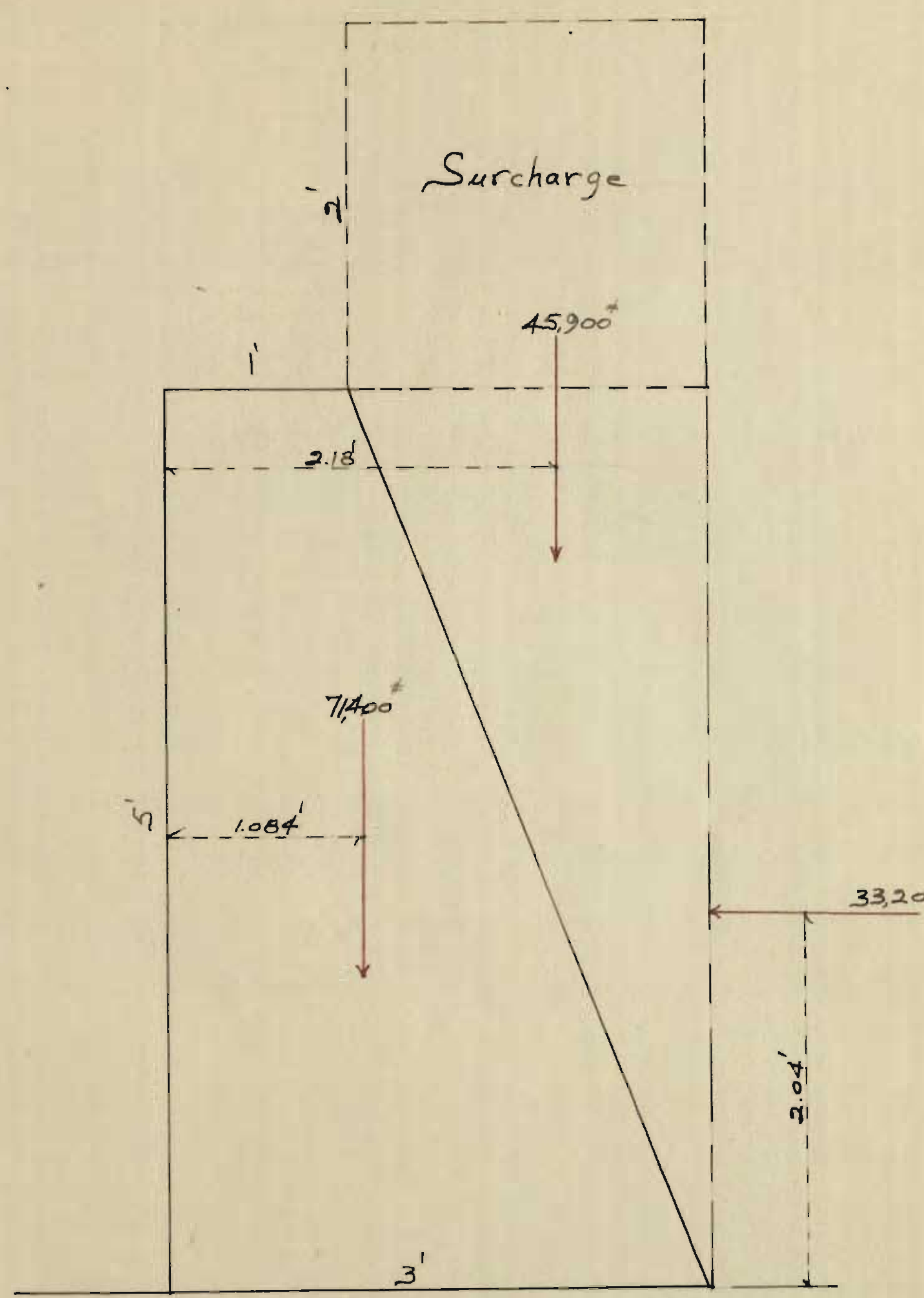
$P = 117{,}300$ $\qquad A = 3 \times 51 = 153$ sq. ft.

$M = 0.5 \times 117{,}300 = 58{,}700$ in. lbs $\qquad c = 1.5$

$I = 51 \times 3^3/12 = 104.5\ ft^4$.

$$S = \frac{117{,}300}{153} \pm \frac{58{,}700 \times 1.5}{104.5} =$$

$767 + 840 = \underline{1{,}607 \text{ psf}}$.



Back wall

Fig. 4

CHAPTER III

Pile Foundations

A pile is a compression member driven in the ground in order to increase its power to support the weight of a structure.

Piles have been used since immemorial. The oldest type of piles is the timber pile. Metal piles have been used since the middle of the 19th. century and from the beginning of the present century concrete piles have been widely used. Advantages of concrete piles over timber piles are: Concrete piles are more durable. Their life is independent of the ground water table. Timber piles are subject to decay under the action of the ground water. Their useful life, even when creosoted (1) or treated, is limited to 30 or 40 years. Moreover concrete piles have a greater bearing capacity due to their larger size. They may safely be used for loads up to 50 tons. Thus the number of piles required to support a given structure is materially reduced. On the other hand, timber piles are not loaded above 15 or 20 tons and it is difficult to find the required size.

Strict economy requires that adequate exploration of the soil be made to determine the proper length of the pile. For, the loss of time in cutting concrete piles is more serious then that of material and labor.

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(1) Coated with an oily antiseptic liquid distilled from coal-tar

Bearing power of piles.

At a distanc of about 30 meters from the position of the abutment there is a collection well which shows clearly that the sub-soil is composed of gravel and sand to a depth of 10 meters below which there is a hard stratum. Consequently, the bearing capacity of the piles will depend on two different factors,namely the frictional resistance acting along the sides of the piles and the point resistance, i.e. the resistance of the soil against being compressed or displaced by the piles. Hence the resistance acting while the pile is being driven closely resembles the resistance of the pile under static load. It follows that the pile formulas strictly apply in this case. On the other hand, piles driven in compressible soil, such as clay, the resistance of the pile when it is being driven bears little relation to its ultimate capacity under static load due to the change in the structure of the soil particles with time. Since the permeability of the clay is low, the water is not squized out while the pile is being driven and the particles do not assume a final position until later.

The load on every pile is distributed at the foot of the pile over a horizontal circular area, being maximum at the center and decreases gradually until it is zero at the circonference of the circle. Hence in a group of piles there is an overlap of stress zones which increase with decrease in pile spacing and with increase in pile length . In the first case the circles become closer and in the second case they become larger.

Consequently, the load carried by a group of piles is less then ~~the then~~ the product of the number of piles times the resistance of one pile. In this particular case, since the piles are bearing on a hard stratum in addition to the frictional resistance it is safe to consider that the bearing capacity of a group of piles is equal to the resistance of one pile times their number.

Therefore, supposing that the test gives a true value of the carrying capacity of a pile at the time of testing, some of the uncertainities for which allowence must be made are:

a. Effect of time on pile resistance.

b. Effect of group action as compared with a single pile.

c. Possible increase of live load.

Design of Piles

The roadway is 15 ft. above the surface of the well from which the information about the subsoil was known. The abut ment is 21 ft. high, and the pile should project at least one foot above the surface of the ground. Therefore, the length of the pile should be $33 + 15 - 21 + 1 = 28$ ft.

The pracitcal dimension of a square pile 28 ft. long is 14 in. The pile will have a constant cross-section all through and will be pointed at the lower end.

Concrete. According to the A.R.E.A. specifications the concrete should have a compressive strenght of 3500 psi, before the piles are handled. To attain this strength the mix should be 1 : 2.08 : 2.80 with water cement ratio of 0.65 and maximum size of coarse agregate 1 in. The piles are cast in a horizontal position on unyeilding base to prevent fluxural stresses in the concrete. The side forms may be removed in 24 to 48 hours. The pile is allowed to rest on the base for a week during which time it should be showered with water to permit complete chemical action for the setting of the cement.

Reinforcement. The reinforcement is required to resist the stresses due to:

a. Handling the piles.

b. Driving the piles.

c. Static load.

Every pile will have 2 points of suspension at a distance of $0.207L = 0.207 \times 28 = 5.80$ ft. from the head and the end of the pile (.1). The maximum bending moment in the pile, when it is lifted, occurs in the middle and is equal to $0.257WL$ in.lbs. (2) where W = weight of the pile in lbs. and L = length of the pile in ft.

$$W = \frac{14 \times 14}{144} \times 28 \times 150 = 5,730 \text{ lbs.}$$

$$L = 28 \text{ ft.}$$

$$M = 0.257 \times 5,730 \times 28 = 41,300 \text{ in. lbs.}$$

$$A_s = 41,300/18,000 \times 0.87 \times 12 = 0.22 \text{ sq.in.}$$

The area of concrete is always in excess of what is required to sustain the axial load. Therefore, the capacity of longitudenal steel is not considered effective in carrying the axial load. The A.R.E.A. specifies a minimum of longitudenal steel of 1% of the cross-sectional area of the pile and a maximum of 4%. In this case the steel used is 1.5% of the cross-sectional area of the pile.

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(2) Page 171. Foundations of Bridges and Buildings by Jacoby & Davis

$A_s = 14 \times 14 \times 0.015 = 2.94$ sq. in.

Use 4 - 1 in. round bars $A_s = 3.14$ sq.in.

Lateral reinforcement is required to increase the resistance of the concrete to longitudenal compression, and also to resist diagonal tenssion. For lateral reinforcement use separate 3/8 in. round bars spaced 2 in, at the head and foot of the pile and the spacing is increased gradually to 6 in. at the center. Moreover 1/4 in. round bars are used to tie the main reinfocement diagonally. (Fig.3, Plate IV).

Pile Driving

Pile driving is forcing a pile into the ground without previous excavation. The operation in its simplest and most primitive form consists of raising a heavy weight and let it drop on the pile which is held vertically. This weight is called a drop hammer. At first men then animals and afterwardsthe steam engine were used to raise the hammer. Some steam-hammers are lifted a short distance by steam pressure and allowed to fall by gravity. Others are designed to reinforce the action of gravity. Another recent method of pile driving consists of using a water jet to aid in displacing the earth at the foot of the pile. This method is usually employed in conjuction with a drop hammer.

Pile driving in this country is done only by the use of a drop hammer. Since such jobs are small and rare, it is uneconomical to import the heavy and expansive equipment required by steam-hammer operations.

The energy of the hammer at the instant it strikes the pile is equal to the product of its weight and the free fall minus losses due to friction between hammer and leaders. $E = WH - F$ or if "e" equals the efficiency of the hammer $E = eWH$.

This energy is transformed in accordence with the laws of impact as follows.

a. A cértain portion goes into the pile.

b. A certain portion remains in the hammer in the form of kinetic energy.

c. And the remainder is lost in heating the head of the pile.

Let W_h = weight of drop hammer or ram of steam hammer.

W_p = wieght of pile and pile cap.

V_h = velocity of hammer prior to impact.

V_p = velocity of pile (=0) prior to impact.

V'_h = velocity of hammer following impact.

V'_p = velocity of pile following impact.

H = fall of hammer.

e = efficiency of hammer.

$r = W_h/W_p$

Energy of the hammer at the instant of impact is:

$$W_h \times V_h^2/2g = eW_hH$$

By the principle of conservation of momentum

$$W_hV_h + W_pV_p = W_hV'_h + W_pV'_p \quad \text{.........} \quad (a)$$

Coefficient of restitution is:

$$n = (V_h' - V_p')/(V_h - V_p) \ldots\ldots\ldots\ldots\ldots \text{(b)}$$

From equations (a) & (b)

$$V_p' = \frac{W_h V_h (1 + n)}{W_h + W_p}$$

$$\& \quad V_h' = \frac{V_h (W_h - W_p n)}{W_h + W_p}$$

a. Kinetic energy of the pile after impact is:

$$(W_p)\,(V_p')^2/2g$$

Substitute the value of V_p'

$$\frac{W_p}{2g} \times \frac{(W_h V_h (1+n))^2}{(W_h + W_p)^2} = \frac{W_p W_h^2 V_h^2 (1+n)^2}{2g(W_h + W_p)^2}$$

But $W_h V_h^2/2g = e W_h H$

Hence $e\, W_h H \times \dfrac{W_p W_h (1+n)^2}{(W_h + W_p)^2}$

Divide numerator and denomenator by W_p^2

$$e W_h H \; \frac{r(1+n)^2}{(r+1)^2}$$

or $\underline{e W_h H K_1}$ where $K_1 = \dfrac{r(1+n)^2}{(r+1)^2} \ldots\ldots\ldots\ldots \text{(a')}$

b. Kinetic energy left in the hammer after impact is:

$$W_h (V_h')^2/2g = \frac{W_h}{2g} \; \frac{V_h^2 (W_h - W_p n)^2}{(W_h + W_p)^2}$$ after substituting the value of V_h'

But $e W_h H = W_h V_h^2/2g$

Substitute in the previous equation

$$K.E. = eW_hH \frac{(W_h - W_p n)^2}{(W_h + W_p)^2}$$

Divide numerator and denominator by W_p^2

$$eW_hH \frac{(r-n)^2}{(r+1)^2}$$

or $\underline{eW_hHK_2}$ where $K_2 = \frac{(r-n)^2}{(r+1)^2}$ .... (b')

c. The kinetic energy lost in heating the head of the pile is:

$$K_3 = 1 - K_1 - K_2 =$$

$$1 - \frac{r(1+n)^2}{(r+1)^2} - \frac{(r-n)^2}{(r+1)^2}$$

and $K_3 = \frac{1-n^2}{r+1}$

$K.E. = \underline{eW_hHK_3}$ .... (c')

Values of K_1, K_2, and K_3 for several ratios of hammer weight to pile weight (r) and for 2 values of (n) are given in Table (1), Appendix (A).

Derivation of rational pile driving formulas.

Let R_d = ultimate resistance of a pile.

R_a = allowable load on the pile.

s = penetration of the pile per blow.

E = modulus of elasticity of pile.

A = mean cross-sectional area of pile.

L = length of pile.

C_1 = elastic compression of pile.

C_2 = rebound of pile due to elasticity of the soil.

C_3 = elastic compression of pile cap.

$C = C_1 + C_2 + C_3$

$1/2\ R_dR_dL/AE = 1/2\ R_dC_1$ = Energy lost in elastic compression of pile (3)

$1/2\ R_dC_2$ = Energy lost in elastic compression of soil.

$1/2\ R_dC_3$ = Energy lost in elastic compression of cap.

R_ds = Useful work done on pile.

Equating the kinetic energy of the pile (a') to the useful work plus losses;

$$eW_hHK_1 = 1/2\ R_dC + R_ds$$

$$R_d = \frac{eW_hH}{s+C/2} K_1 \quad \text{.................. (d')}$$

Where the residual kinetic energy of the hammer is added to the kinetic energy of the pile.

$$eW_hHK_1 + eW_hHK_2 = 1/2\ R_dC + R_ds$$

$$R_d = \frac{eW_hH}{s+C/2} (K_1+K_2) \quad \text{where} \quad (K_1+K_2) = \frac{r+n^2}{r+1} \quad \text{......(e')}$$

These formulas were first developed by Hily.

In using these formulas values of e,n and C must be evaluated.

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(3) Work $= 1/2\ R_d\Delta L$ but $\Delta L = Ls/E = L \frac{R_d/A}{E} = R_dL/EA = C_1$

Therefore, work $= 1/2\ R_d\ R_dL/EA$.

Values of (e), efficiency of hammer, are given in Table 2. The value of the coefficient of restitution (n) varies theoretically from 0 to 1. In Table 3 are given the values which are widely used. Values of $C = C_1 + C_2 + C_3$ are given in Table 4 in term of pile length and unit compressive stress R_d/A. (All tables are given in Appendix A)

Several pile driving formulas are in use. Most of them have rational basis with coefficients developed from tests. The results from these formulas are not totally dependable. They should be coordinated by tests.

A practical rational formula follows directly from (e'). Apply a factor of safety of 3 and multiply the numerator by 12 (since H is expressed in feet with all other dimensions in inches).

R_a = Allowable load on pile in lbs.

$$R_a = \frac{4eW_hH}{s+C/2} \times \frac{r+n^2}{r+1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(1)}$$

The most commonly used imperical formula is the Engineering News formula developed by A.M. Wellington.

$$R_a = \frac{2W_heH}{s+c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2)}$$

Where $c = 1$ for drop hammer

& $c = 0.1$ to 0.3 for steam hammer

Application of the formulas.

In the particular case under consideration a 5,000 lbs. drop hammer is used and the fall is 5 ft. The weight of the hammer should not be less then 1/2 the weight of the pile and fall should not exceed 8 ft.. Because with a light hammer and a high fall, a large portion of the energy is dissipated in distructive work.

R_a = 51,000 lbs. from page 20

W_h = 5,000 lbs.

W_p = 14x14/144 x 150 x 28 = 5,730 lbs.

r = 5,000/5,730 = 0.87

H = 5 ft.

e = 0.75 from Table 2

C = for R_d/A = 51,000 x 3/(14x14) = 785 lbs./sq.in. and L = 28 ft. interpolate in Table 4.

C = 0.42

n = 0.40 from Table 3

Using formula (1)

$$51,000 = \frac{4 \times 0.75 \times 5,000 \times 5}{s + 0.42/2} \times \frac{0.87 + 0.4^2}{0.87 + 1}$$

$$= 75.000/(s+0.21) \times 0.55$$

$$51,000s = 41,200 - 11,200 = 30,000$$

$$s = 30,000/51,000 = \underline{0.59 \text{ in.}}$$

Using formula (2)

$c = 1$ for drop hammer

$$51,000 = \frac{2 \times 5,000 \times 0.75 \times 5}{s + 1}$$

$$51,000s = 37,500 - 51,000$$

This formula, it seems,does not apply in this case. In developping this formula Mr. Wellington assumed that all of the hammer energy $12W_hHe$ in.lbs. went to overcome the driving resistance R_as. He used a factor of safety of 6 and equated the 2 quantities.

$$1/6 \times 12W_hHe = R_as$$

$$R_a = 2W_hHe/s$$

In applying this formula it was found that the results were absurd. To correct it a constent $c = 1$ was added to the denomenator for drop hammers and $c = 0.1$ to 0.3 for steam hammers since the interval between blows is very short.

In this case an electric motor will be used to raise the hammer. It is possible to to attain an interval between blows of 1/2 a minute i.e. 2 blows per minute. Therefore take $c = 0.2$ and apply formula (2).

$$51,000 = \frac{2 \times 5,000 \times 0.75 \times 5}{s + 0.2}$$

$$51,000s = 37,500 - 10,200 = 27,300$$

$$s = 27,300/51,000 = \underline{0.54 \text{ in.}}$$

Therefore, the piles should be driven until a penetration of 0.5 in. under the last blow is attained. Anyhow, as stated before, these results should be combined with results of tests carried out on the site.

Pile Cap.

To protect the head of the pile against injury in driving, a cap is used to cushion the blows. Another function of the cap is to adapt the base of the hammer to the different shapes of piles.

The cap (Fig.2 Plate IV) consists of a cylendrical steel casting 20 in. in diameter recessed at the 2 ends. In the upper portion is fitted a wooden block (oak) on top of which thereis a cast steel plate which receives the blows. The lower portion contains 2 layers of rubber belting or ropes and 2 to 3 in. yellow pine planks. This side rests on the concrete pile. To prevent the head of the pile from spalling, a band of 3/8 in. steel plate is bolted around it. The cap has jaws on the sides which fit into the leads. Thus the pile is held vertically in position and guided while driving.

The cushion reduces the value of (n), coefficient of restitution. From the equations on page 28 it is abvious that the energy which goes into the pile decreases while the energy which remains in the hammer increases. To compensate for this loss the fall should be increased a little, say 0.5 ft.

Pile Shoe

The pile shoe is a conical steel casting 20 in. high and the point at the end is 1 in. in diameter. It is fixed to the pile when the concrete is poured. It serves as a form at the end of the pile. The function of the shoe is to protect the end of the pile and to increase the rate of penetration or to reduce the energy required for driving.

The end bearing power of the pile is not affected by the pointed shoe. The end is held tight from all sides and the sum of all the reactions on the tapered shoe from the earth is equivelant to the reaction on the horizontal projection of the cone.

Drop Pile Hammer

The drop Hammer consists of a solid casting with jaws on each side which fit into the pile driver leads. Some hammers have a projection in the back part which fits in between the pile driver leads. There is a pin at the top of the hammer for the attachment of the rope or the cable by which the hammer is raised. The base which strikes the pile is either flat or concave. The hammer is made as long as practicable in order to increase the bearing in the leads. The form is arranged in such a way as to have the center of gravity as low as possible. The weight of a drop hammer ranges from about 2,000 to 6,000 lbs. The light ones being used to drive timber piles and the heavy ones to drive concrete piles.

Steam pile hammers are more commonly used due to their high efficiency and compact form. The hammer is automatically raised and dropped a short distance by the action of a steam cylinder and piston supported in a frame which follows the pile. The weight of steam hammers ranges from about 1,500 to 6,000 lbs. and for specially large jobs 30,000 lbs. hammers have been used.

Pile Driver

Pile drivers are built of timber or of steel. Steel pile drivers are usually constructed of viriety of forms to serve different purposes. The common type is a crane and derrick mounted on a crawler or car. Timber pile drivers are not as strong and rigid as steel drivers. They are used in this country due to their availability and ease of construction. Their main characteristic are the leads, two upright parallel members supporting the sheaves used to hoist the hammer and piles. They also guide the hammer in its movement. Their inner faces are lined with steel plates to reduce friction and wear. The leads are held in position by three latteral supports two on the sides and one at right angle to them, used as a ladder. All the members are rigidly braced and rise in the form of a tripod tower. An elctric motor or a disel engine, located at the base of the tower, is used to raise the hammer.

Reinforced Concrete Capping

A reinforced concrete slab is poured over the head of the piles. Its main function is to distribute the load uniformly over the pilesand to tie them properly. This cap should be strong and rigid. It is made 36 in. deep and reinforced with 3 - 1 in. round bars in the top and bottom running in both directions over the head of the piles. Similarly 5/8in. round bars 10 in. c. to c. are placed in the top and bottom of the slab in both directions.

CHAPTER IV

Miscellaneous Details

Diaphragms

Transvers diaphragms, 30 in. deep and 12 in. wide reinforced with 4 - 7/8 in. round bars, will be built between the longitudenal beams at the two ends and at the third points of the span. Their main function is to support the beams laterally.

End Bearings

Safe working stress for concrete in direct compression is 600 psi.. Due to the defflection of the longitudenal beams, the unit pressure below the forward edge of the bearing is nearly doubled. Therefore the allowable concrete stress should be halved i.e. 300 psi.

Required bearing plate area

119,600/300 = 398 sq. in.

Use a plate 18 x 22 in.

Fixed bearing. One steel plate is fixed to the beam and another concave steel plate is fixed to the concrete in the abutment. Dowels project from the abutment into the beam passing through the plates in tapered holes in order to allow angular without horizontal movement. This arrangement is required to spread the load which tends to concentrate in the forward edge due to the inclination caused by defflection.

Expansion bearing. A similar arrangement is used for expansion bearing as the fixed bearing, but without dowels in order to allow horizontal as well as angular movement.

Drainage

Removal of surface water sidewise is accomplished by the crown. The slope to the two ends of the bridge is caused by raising the forms 3 in. at the center. Besides facilitating drainage this camber prevents the appearence of sag which would be evident if the beams were perfectly level throughout the span.

Wearing Surface

First grade idealite will be used for wearing surface. It consists of small chips 12 to 19 mm. in size, sand 3 mm. in size, light oil, bituman and powder lime. These ingredients are properly proportioned and mixed.

Sidewalk

The curb stones are layed at a distance of 6 ft. from the parapet. The space in between is filled with sand and gravel and tiled with cement tiles. The hight of the sidewalk at the parapet is 8 in. above the deck slab, and 7 in. at the curb stones. Thus 1 in. slope is allowed for drainage.

****

APPENDIX A

TABLE 1*

$r = \frac{W_h}{W_p}$	n	Proportion of total energy: Transferred to pile $K_1 = \frac{r(1+n)^2}{(r+1)^2}$	Remaining in hammer $K_2 = \frac{(r-n)^2}{(r+1)^2}$	Lost in heat $K_3 = \frac{(1-n^2)}{r+1}$
0.25	0.2	0.23	0,0.[illegible]	0.77
0.50	0.2	0.32	0.04	0.64
0.75	0.2	0.35	0.10	0.55
1.00	0.2	0.36	0.16	0.48
1.50	0.2	0.35	0.27	0.38
2.00	0.2	0.32	0.36	0.32
0.25	0.4	0.31	0.01	0.68
0.50	0.4	0.44	0.0	0.56
0.75	0.4	0.48	0.04	0.48
1.00	0.4	0.49	0.09	0.42
1.50	0.4	0.47	0.19	0.34
2.00	0.4	0.44	0.28	0.28

(*) Page 106. Foundations of Bridges and Buildings by Jacoby and Davis.

TABLE 2

Type of Hammer	Value of e
Drop-hammer, free fall	1.00
Drop-hammer, line attached	0.75
Single-acting steam-hammer	0.90
Double-acting steam-hammer	1.00

TABLE 3

	n, Coefficient of restitution
Cast iron on steel	0.55 - 0.60
Cast iron on concrete	0.40
Cast iron on wood	0.20 - 0.25

TABLE 4

Leng. of Pile ft.	R_d/A= 500 psi. Easy driving			R_d/A=1,000 psi. Medium driving			R_d/A=1,500 psi. Hard driving			R_d/A=2,000 psi Very hard driv		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
10	0.19	0.16	0.25	0.28	0.21	0.41	0.37	0.27	0.57	0.41	0.27	0.67
20	0.23	0.19	0.28	0.36	0.27	0.47	0.49	0.36	0.65	0.57	0.39	0.79
30	0.27	0.22	0.31	0.44	0.33	0.53	0.61	0.45	0.74	0.73	0.51	0.91
40	0.31	0.25	0.34	0.52	0.39	0.59	0.73	0.54	0.83	0.89	0.63	1.03
50	0.35	0.28	0.37	0.60	0.45	0.65	0.85	0.63	0.92	1.05	0.75	1.15
60	0.42	0.31	0.40	0.68	0.51	0.71	0.97	0.72	1.01	1.21	0.87	1.27

(1) For timber piles.
(2) For reinforced-concrete piles with 1 in. material on head.
(3) For reinforced-concrete piles fitted with effective driving cap.

APPENDIX B

French System of Loading for Highway Bridges

Two systems of loading will have to be considered:

I. The roadway is designed to carry a uniform live load of:

P = (820 - 4L) Kgs/m.sq.   L = Span

with a minimum of 500 Kgs/m.sq. for L greater then 80 m.

II. The roadway is then designed to carry a system composed of two trucks each having the following characteristics:

Total load	16 tons	=	35,200 lbs.
Rear axle load	12 tons	=	26,400 "
Front axle load	4 tons	=	8,800 "
Total length	10 met.	=	32.80 ft.
Total width	2.5 m.	=	8.20 ft.
Distance c. to c. of axles	4 met.	=	13.10 ft.
Distance c. to c. of wheels	1.7 m.	=	5.58 ft.

We will assume, travelling side by side and in the same direction as many of these systems as the width of the road permits.

The two systems of loading have to be considered and whichever gives the biggest results will govern the design.

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