THE DESIGN
OF A
REINFORCED CONCRETE BRIDGE
TO SPAN
THE BEIRUT RIVER
IN BEIRUT

June, 1948

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by

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B.S.C.E

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PREFACE

The traffic problem in Beirut has become an acute one because of the large increase in the number of vehicles, and the streets are entirely too narrow; The Lebanese Government began widening the roads where possible and realized that the traffic crossing the country from south to north and vice versa should not pass through the heart of the city. Wide Boulevards are to be constructed began by the Khaldé Boulevard, the southern approach to Beirut, and a similar one is to begin north of the city at Nahr-el-Mot.

This new Boulevard of fourty meters in width is to cross the Beirut River at a point where a span of about 100 meters will be necessary. The bridge is to be thirty meters wide, twenty meters of roadway (seven lanes) and a five meter sidewalk on each side.

The Public Works Ministry made a study of plans for a bridge that might be used at this location. It was to consist of seven spans of 14 meters each resting on piers using concrete piles for the main foundations.

When the pile driving was started, it was found that for the first four meters depth, the pile was forced down with difficulty.

It would be of great advantage to get rid of the beam and girder design and make a continuous slab resting on reinforced concrete walls six meters center to center (20 ft.) The waterway remaining the same (8 ft.) a height of two meters is gained, thus decreasing the approach's slope to a minimum (see longitudinal profile).

In this thesis, the author tries to solve the above mentioned problems. Reference is made to Principles of Reinforced Concrete Construction by Turneaure and Maurer, 1936 edition, for constants and formulas in order to make the design as easily read as possible.

I wish to thank Prof. J.R. Osborn for helping me with this Thesis

Mohammad Ali Itani

Beirut

May, 1948.

C O N T E N T S

Four sheets of Drawings are attached to this theses

Sheet 1 - The Lay-Out

Sheet 2 - The Longitudinal Profile

Sheet 3 - The Front View & The Transverse Cross-Section

Sheet 4 - The Longitudinal Cross-Section (Showing details of Reinforcement)

I N T R O D U C T I O N

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A successful solution for the construction of the Bridge to cross the Beirut River in Beirut might be of reinforced concrete beam and girder design, steel being expensive for its design of steel, foundations not dependable for an arch design.

? - grammatical constr.?

The present design procedure is divided into four parts. The bridge is composed of two panel frames 20' centre to centre of each span. An expansion joint is made at the center of the bridge:

$$\text{Exp.} = wlt^{\circ}$$

$$= 0.000006 \times 2 \times 8 \times 20 \times 12 \times 50$$

$$= 1.15 \text{ inches}$$

. . 2 inches are used for expansion joint which is filled with Bitumen.

Part I deals in finding the fixed end moments, the bridge structure being one ridged frame. The structure is statically indeterminate and the Hardy Cross method is used for finding these moments.

After finding the fixed end moments, Positive moments are easily calculated and the procedure for the design is found in Part II.

A.C.I. specifications for the design are used.

f_s = 18000 lbs/sq.inch

f'_c = 2500 lbs/sq.inch   n = 12

f_c = 800 lbs/sq.inch   (in slabs)

f_c = $0.22\, f'_c$ = 550 lbs/sq.inch (in columns)

Part III deals with the design of the anti-scouring structures namely the wing walls and the wood sheeting.

Quantities and cost are calculated in Part IV also to compare them with those of the design made by the Public Works Department, the cost amounting to 800,000 Leb.pounds while the cost of the present design is only 400,000 leb.pounds.

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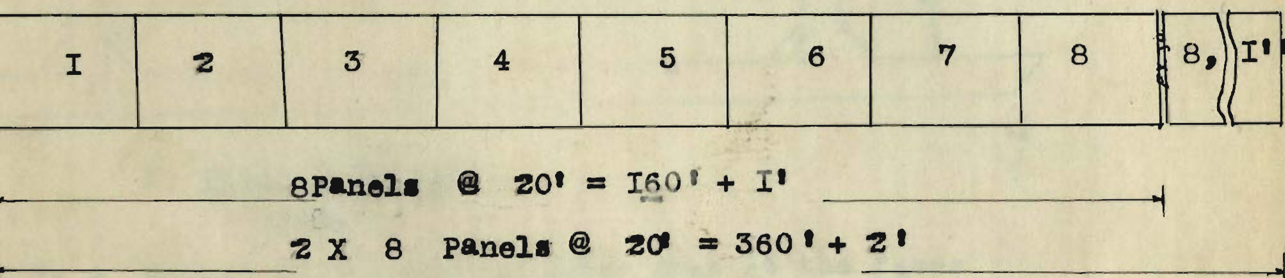
P A R T   O N E

T o   f i n d

THE FIXED END MOMENTS

Introduction

The total length of the bridge being 98 meters or 322 ft ; it is divided here into two eight panel frames of 20 ft center to center of supports for each span



The upper roadway deck is rigidly connected to vertical reinforced concrete walls built along the width of upper deck at right angles to the axis of the bridge. These walls are also rigidly connected to continuous reinforced concrete foundation denoted here as the lower deck.

The lower deck is assumed to be 10 inches thick; the vertical walls 12 inches; the upper deck 12 inches. Haunches 1' by 1' are used on the junctions to hold the members as rigid as possible.

C h a p t e r   I

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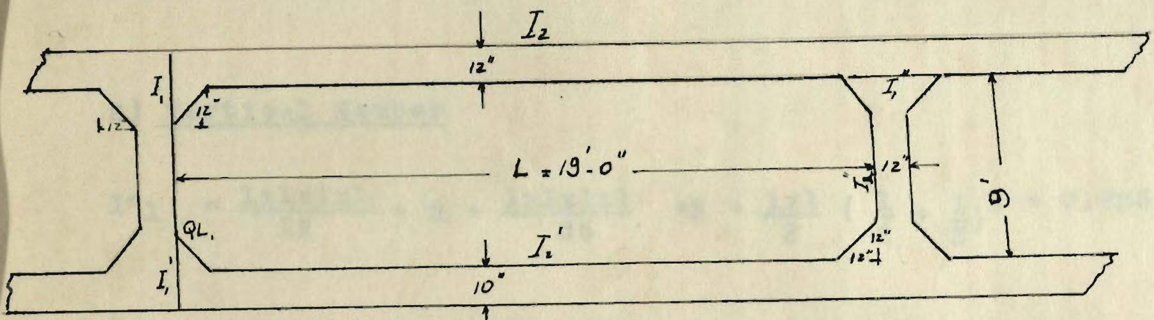
The Moment Distribution Factors

As the moment distribution factors depend upon the rigidities of the members in question, it is first required to find these rigidities.



1) **Upper Deck**

I_1 = Moment of inertia of upper slab at the faces of supports (point of haunch)

I_2 = Moment of inertia of upper slab between the haunches

$$I_1 = \frac{bd^3}{12} + \frac{b'd'^3}{36} + \frac{b'd'}{2}\left(\frac{1}{2} + \frac{1}{3}\right)^2$$

$$= \frac{1x1x1x1}{12} + \frac{1x1x1x1}{36} + \frac{1x1}{2}\left(\frac{1}{2} + \frac{1}{3}\right)^2 = 0.4583$$

$$I_2 = \frac{bd^3}{12} = \frac{1 \times 1 \times 1 \times 1}{12} = 0.0833$$

$$\frac{I_2}{I_1} = \frac{0.0833}{0.4583} = 0.I8I8$$

$$Q = \frac{1}{L} = \frac{1}{19} = 0.0526$$

from tables (p.289 Turneaure & Maurer) it is found that constants A.,B.,& C, are 2,53 , I.92 & I.II respectively

2) Vertical Member

$$I''_1 = \frac{1x1x1x1}{12} + 2 + \frac{1x1x1x1}{36} +2 + \frac{1x1}{2}\left(\frac{1}{2} + \frac{1}{3}\right)^2 = 0.833$$

$$I''_2 = \frac{1 \times 1 \times 1 \times 1}{12} = 0.0833$$

$$\frac{I''_2}{I_1} = \frac{0.0833}{0.8330} = 0.I0$$

$$Q = \frac{1}{9} = 0.III$$

from tables (p.289 T. & M.) it is found that constants A,B,& C are 3.0, I.85, & I.I3 respectively but the author prefers to use these constants used for unhaunched beams to be on safe side & A=2, B=2, C=1.

3) Lower Deck

$$I'_1 = \frac{1x10x10x10}{12x12x12x12} + \frac{1x1x1x1}{36} + \frac{1x1}{2}\left(\frac{4}{12} + \frac{5}{12}\right)^2 = 0.4397$$

$$I'_2 = \frac{1x10x10x10}{12x12x12x12} = 0.0485$$

$$\frac{I'_2}{I'_1} = \frac{0.0485}{0.4397} = 0.1103$$

$$Q = \frac{1}{19} = 0.0526$$

A = 2.61   B.= 1.195 & C = 1.115

K_1 = rigidity of upper member $\frac{EI}{L} = \frac{0.4583}{19} = 0.0241$

K_2 = " vertical member $\frac{EI}{L} = \frac{0.833}{9} = 0.0925$

K_3 = " lower " $\frac{EI}{L} = \frac{0.4397}{10} = 0.0231$

The value of E being the same in all the members it was taken equal unity since we are dealing with relative values. Therefore the corrected values of K's due to the haunches

K' = K_1 x A x B
 = 0.0241 x 2.53 x 1.92 = 0.117

K" = 0.0925 x 2 x 2 = 0.370

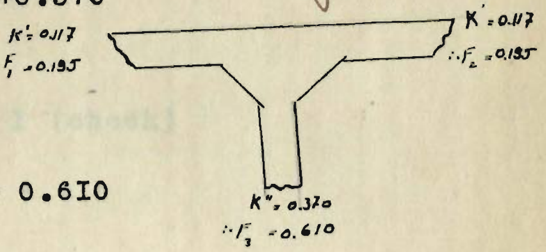
K" = 0.0231 x 2.61 x 1.915 = 0.115

Distribution factors of upper Members to verticals

A - Interior spans

$$F_1 = F_2 = \frac{K'}{K'+K'+K''} = \frac{0.117}{0.117+0.117+0.370}$$

$$= \underline{0.195}$$



$$F_3 = \frac{K''}{K'+K'+K''} = \frac{0.370}{0.117+0.117+0.370} = 0.610$$

$$F_1 + F_2 + F_3 = 1 = 0.195 + 0.195 + 0.610 = 1 \text{ (check)}$$

B - Exterior spans

$$F_1 = \frac{0.117}{0.117 + 0.370} = \underline{\underline{0.24}}$$

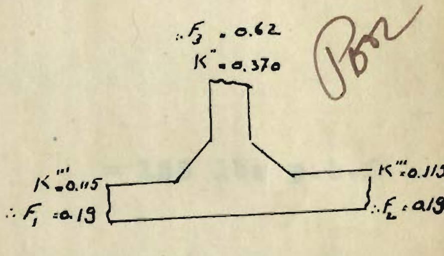
$$F_2 = \frac{0.370}{0.117+0.37} = \underline{0.76}$$

$$F_1 + F_2 = 1 = 0.24 + 0.76 = 1 \text{ (check)}$$

Distribution Factors of Lower Members to verticals

A - Interior spans

$$F_1 = F_2 = \frac{0.115}{0.115+0.115+0.370} = 0.190$$



$$F_3 = \frac{0.370}{0.115+0.115+0.370} = 0.620$$

$$F_1 + F_2 + F_3 = 1 = 0.190+0.190+0.620 = 1 \text{ (check)}$$

B - Exterior spans

$$F_2 = \frac{0.115}{0.115+0.370} = 0.240$$

$$F_3 = \frac{0.370}{0.370+0.115} = 0.760$$

$$F_1 + F_2 = 1 = 0.24 + 0.76 = 1 \text{ (check)}$$

CHAPTER II

Loads and Constants

In all the following calculations, only half of the bridge is dealt with as a frame composed of eight twenty feet panels. An expansion joint is left at the center of the bridge i.e. between the two frames forming the bridge.

$$\text{Impact Factor } I = \frac{L + 20}{6L + 20} = \frac{40}{140} = 0.285 \quad \text{(A.C.I.Spec.)}$$

I - Max. Pressure on soil

A - Lower deck D.L. $= \frac{10}{12} \times 150$ = 125 lbs p.s.f.

B - Verticals a)wall $= \frac{9 \times 1 \times 150}{20}$ = 67 " "

b)haunches= $4 \times \frac{1 \times 1}{2} \times \frac{150}{20}$ = 15 " "

(C.F.) 207

(B.F.) 207 p.s.f

c) Upper deck

D.L. a) concrete = 1 x I50 = I50

b) 3" bit.conc.= $\frac{3}{12}$ x I20 = 30 I80 " "

L.L. 2H 20 Lorries = 2 x 40000 = 80000 lbs

2H I5 " = 2 x 30000 = 60000 "

I40000 lbs

Width of lane = 9 ft.

Length of 1/2 bridge = I60 ft.

$\frac{140000}{9 \times 160}$ = I00 lbs p.s.f.

+ 0.285 impact= 28 = I28 " "

515 lbs psf

This value is much less than the allowable maximum pressure on soft clay of 2000 lbs p.s.f.

2) Effective Breadth due to wheel concentration

B = 0.7 L + W (p. 199 T. & M.)

Where
B = Effective breadth of slab carrying the wheel concentration.

L = Length of span

W = Width of the wheel

$= 0.7 \times 20 + 1.67 = 15.67$ ft. more than 7 ft the max.

For two overlapping wheels 3 ft center to center

$$B = \frac{7 + 3}{2} = \underline{\underline{5 \text{ ft}}}$$

3)- <u>D_1 and D_2 for haunched beams, concentrated loading.</u>

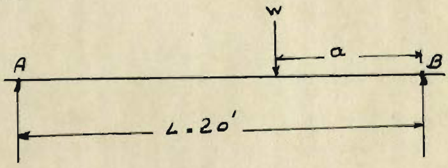
D_1 and D_2 are constants so as when multiplied by $\frac{WL}{B}$ give the fixed end moments of the span in question.

$$M = \frac{DWL}{B} \qquad \text{(P.287 T. \& M.)}$$

These constants are factors of the position of the concentrated load on the span at a distance (a') from right support B

D_1 = constant for moment on left support

D_2 = " " " on right support



<u>A</u> - a = 10 ft

$$\frac{a}{L} = \frac{10}{20} = 0.50$$

$$Q = \frac{1}{19} = 0.0526$$

$$\frac{I_2}{I_1} = 0.1818$$

Therefore from tables for D_1 & D_2 (p.290 T.& M.)

$D_1 = D_2 = 0.138$

B - a = 8 ft.

$$\frac{a}{L} = \frac{8}{20} = 0.40$$

D_1 = 0.I0

D_2 = 0.I6

C a = 6 ft

$$\frac{a}{2} = \frac{6}{20} = 0.30$$

D_1 = 0.0422

D_2 = 0.I58

D - a = 4 ft

$$\frac{a}{L} = \frac{4}{20} = 0.2$$

D_1 = 0.020

D_2 = 0.I56

E - a = 2 ft

$$\frac{a}{L} = \frac{2}{20} = 0.1$$

D_1 = 0.0044

D_2 = 0.092

Table of constants D_1 + D_2 (Load a ft from right support)

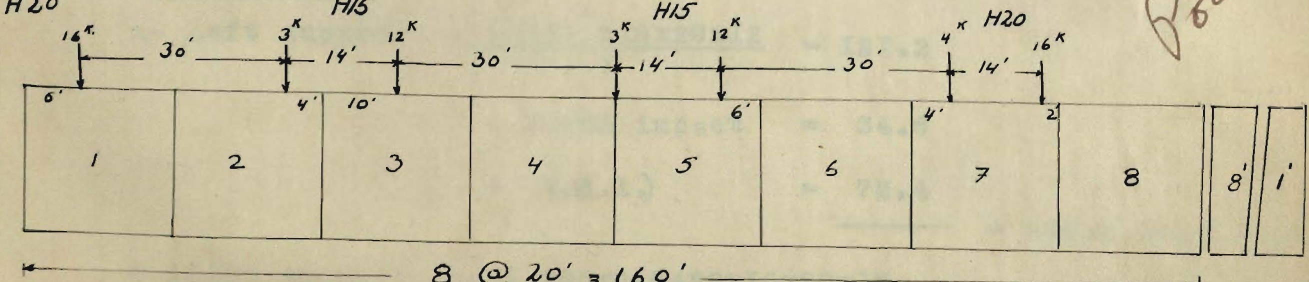
Value of a in feet	IO	8	6	4	2
D_1(for left support)	0.I38	0.IO	0.0422	0.020	0.044
D_2 " right support)	0.I38	0.I6	0.I58	0.I56	0.092

Chapter III

Maximum Negative Movements at the Exterior Supports

There are two exterior supports for every frame of half bridge, namely: the support at the biginning also exerted upon by earth pressur, and the exterior support of the frame adjacent to the expansion joint. Although the latter has no earth pressur on it, it is designed as the former for simplicity in construction with an error of about 1 % only on the safe side.

Poor



Position of load producing max.neg.mom.at the first Ext.support

Moments at supports are those due to Dead Load and moments due to live load plus the impact.

w of D.L. = I80 lbs p.s.f. (see chap.II art.1,c)

Constant C = 1.11 (see chap.I art.1)

M= Fixed end moment due to D.L. $= C \frac{wL^2}{12}$ at every support

$= \frac{1.II \times I80 \times 400}{I2} = 6040'$ lbs

$\times \frac{12}{1000} = 72.4$ inch kips (at every sup.)

Fixed End moments at supports due to Live Load according to previous loading follows:

$$M = \frac{DWL}{B}$$

where M = Fixed end moments in inch kips

D = Constant of position of load and haunches (found in Table of chapter II)

W = Concentrated load in kips

L = span in inches center to center of supports

B = Effective depth found out to be 5' (see chap.II art.2)

I - First span

a- Left support $M \frac{0.I58 \times I6 \times 20 \times I2}{5} = I2I.2$

+ 0.285 impact = 34.6

+ M.(d.1.) = 72.4

= 228.2 inch kips

b-Right support $M = \frac{I.285 \times 0.0422 \times I6 \times 20 \times I2}{5} + 72.4 = 76.6$ " "

2.- Second span

a) Left support, $M = I.285 \times \frac{0.002 \times 3 \times 20 \times I2}{5} + 72.40 = 76.I$ inch kips

b) Right support, $M = I.285 \times \frac{0.I56 \times 3 \times 20 \times I2}{5} + 72.4 = I0I.3$ " "

3.- Third span

a) Mom. at left support = Mom. at Right support

$= I.285 \times \frac{0.I38 \times I2 \times 20 \times I2}{5} + 72.4 = I74.5$ " "

4.- Fourth span

a) Mom. at left support = Mom. at Right sup. = M.(d.L.) = 72.4 " "

5.- Fifth span

a) Mom. at left support =

$= I.285 \times \frac{0.042 \times I2 \times 20 \times I2}{5} + 72.4 = I02.9$ " "

b) Mom. at right support =

$= I.285 \times \frac{0.I58 \times 12 \times 20 \times 12}{5} + 72.4 = I82.7$ " "

6.- Sixth span

Mom. at Left = Mom. at right = M.(d.l.) = 72.4 " "

7)- Seventh span

a) Mom.Left (4 kips)= $1.285 \times \frac{0.156 \times 4 \times 20 \times 12}{5}$ = 38.5

(12 kips)= $1.285 \times \frac{0.0044 \times 16 \times 20 \times 12}{5}$ = 4.3

+ M.(d.l.)= 72.4 = 115.2 inch

b) Mom.Right (4 kips)= $1.285 \times \frac{0.02 \times 4 \times 20 \times 12}{5}$ = 5.0

(16 kips)= $1.285 \times \frac{0.092 \times 16 \times 20 \times 12}{5}$ = 90.8

+ M.(d.l.) = 72.4 = 168.2 "

8)- Eighth span

Left M. = Right M. = M.(d.l.) = 72.4 "

9)- Fixed End moments at the Lower Deck

Total load on soil = 515 lbs p.s.q.f

Deduct wt. of Lower deck= 125 " "

Pressure exerting moment= 390 " "

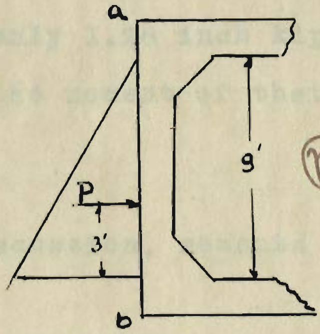
$M = \frac{Cwl^2}{12}$ c = 1.115 w= 390

$= \frac{1.115 \times 390 \times 20 \times 20 \times 12}{12 \times 1000}$ = 156 inch kips

This is the fixed end moment at the base of the vertical members to be taken off by the lower deck .

10)- Moment due to earth pressure on the Exterior wall

P= pressure concentrated at the lower thirs point.



$= \frac{Kwh^2}{2}$ (p.395 T.& M.)

when angle of repose $\theta = 30°$, $K=0.33$

$w= 125\ lbs/ft^3$ $h= 9'$

Therefore $P= \frac{0.33x\ 125x\ 9^2}{2} = 1.8$ kips - say 2 kips

$M_b = \frac{1}{10}\ pl = \frac{1}{10}\ x2x9x12 =$ 24 inch kips

$Ma = \frac{1}{15}\ pl = \frac{1}{15}\ x2x9x12 =$ 16 " "

11)-Moments Distribution

The fixed end moments duely found, it is necessary to distribute the unbalanced ones to the adjacent joints proportionally to these relative rigidities in order to develop equilibrium.

This process is found in Table I

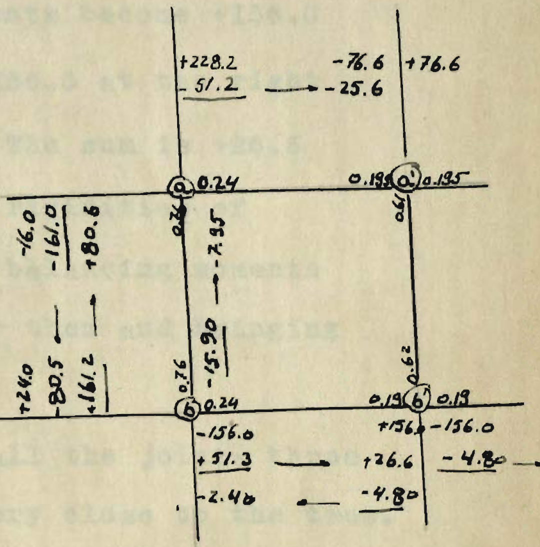
The calculated moments are shown at the top of the respective columns.

The relative rigidities are shown at the joints.

Taking joint by joint, the unbalanced moment is divided proportionally to the relative rigidities of the members and put under the original moment. To balance the joint it is necessary that half of the unbalanced moment be taken care of by the adjacent joint, leaving the former balanced, a dash is put under these to show that the joint is balanced.

This process is continued for the whole frame three rounds to minimize as much as possible the remaining unbalanced moments. This is found to be only I.28 inch kips at the second joint which is about 1 % of the moment of that joint of IOI.83 inch kips.

To illustrate the foregoing discussion, moments at the exterior wall joints is explained:



Taking joint (a) the unbalanced moment is 228.2- I6.0= 2I2.2 inch kips positive.
Multiplying this unbalanced moment by the relative rigidities for the respective members give:

M'_a = 2I2.Ix0.24 = 5I.2 inch kips

M''_a = 2I2.Ix0.76= I6I.0 " "

2I2.2

These are written under the previous moments opposite in sign to the unbalanced.

Half of them is brought to the adjacent joints, leaving the former balanced and a dash is drawn under it.

At joint (b) the moments are +24.0 and -80.5 on the foot of the vertical member and -I56.0 at the connection of the lower member. These algebric sum is 2I2.5 inch kips negative. The balancing moment should be 2I2.5 inch kips positive (opposite in sign). This is distributed proportionally to the relative rigidities of the member.

M'_b = 2I2.5 x 0.76 = I6I.2 (plus)

M''_b = 2I2.5 x 0.24 = 5I.3 "

2I2.5

These are written beneath their respective columns; balancing them bringing their half to the adjacent joint.

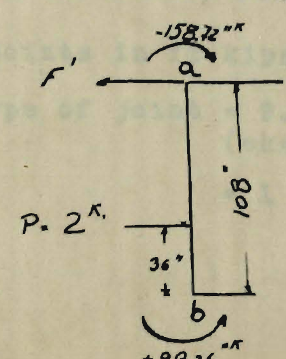
At joint b, the unbalanced moments become +I56.0 and +26.6 at the left of lower deck; -I56.0 at the right of lower deck, and 0 at the vertical. The sum is +26.6 divided proportionally to the relative rigidities of 0.I9, 0.I9 & 0.6I respectively. The balancing moments are written down, a dash is drawn under them and bringing their halfs to the adjacent joints.

This process is continued to all the joints three times bringing the fixed end moments very close to the true.

I2)— <u>Swaying of the Bridge due to Previous Loading:</u>

Due to the fixed end moments previously found, the bridge sways, thus producing new moments at the joints.

Considering first the exterior wall column on which earth exerts pressure, the face F' to hold the member rigid is calculated:



ΣM_b = 98.63 -I58.72-2x36+I08F'= 0

$F' = \frac{132.09}{108}$ = I.222 kips

The force F" to hold the remaining vertical interior vertical members is calculated:

F" x L + ΣM = 0

ΣM =	+ 22.77	- 14.38
	11.18	- 6.57
	66.39	51.84
	12.35	2.39
	69.74	31.45
	25.05	5.34
	52.37	37.09
	29.63	105.14
	+ 289.48	- 254.10 inch kips

$$F'' = -\frac{298.48-254.10}{108} = -\frac{35.38}{108} = \underline{\underline{-0.328}} \text{ kips}$$

$$F = F' + F'' = 1.222 - 0.328 = \underline{\underline{0.894}} \text{ kips}$$

13)- Moments due to displacement of 1 ft.

Herein are presented calculations for the fixed end moments produced due to a displacement of the bridge of one foot.

$$M = \frac{bIE\Delta}{l^2} \quad \text{where} \quad \frac{EI}{l} = K \quad \text{(p.268 T. \& M.)}$$

$$= \frac{bK\Delta}{l}$$

where
- M = Moments at upper & lower joints in ft kips
- K = constant depending upon type of joint = 9.25 (chap.1 art.3)
- Δ = Displacement in ft = 1
- l = length in feet = 9'

$$M = \frac{6 \times 9.25 \times 1}{9} = \underline{6.165} \text{ ft kips}$$

A moment distribution for the above moment is found in table II. The force F_1 which caused the fixed end moments shown in table II is:

$$F_1 = \frac{\Sigma M}{L} = \frac{39.842}{9} = \underline{\underline{4.427}} \text{ kips}$$

I4)- Actual Moments due to Swaying of previous Loading .

It was found in art.12 that $F = 0.894$ kips,

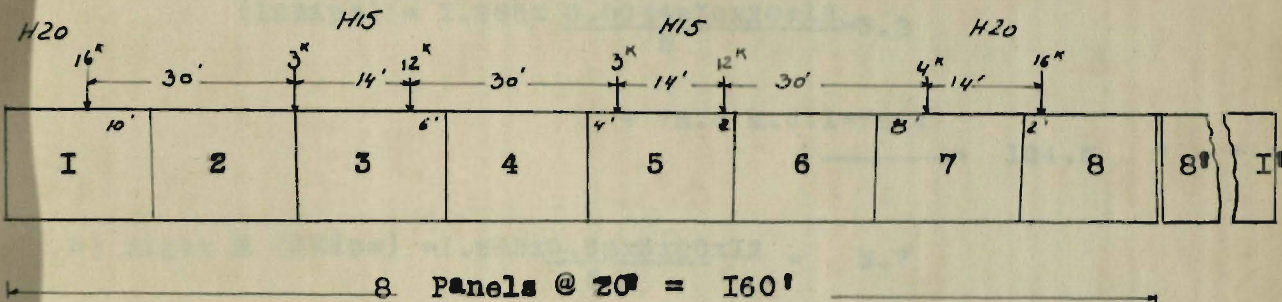
And from art.13, a force $F_1 = 4.427$ kips caused the moments shown in table II.

Therefore actual moments due to the force $F = 0.894$ kips is calculated by multiplying the moments due to F_1 by $\frac{0.894}{4.427} = 0.202$, multiplied by 12 give the moments in inch kips.

On table III are presented final moments due to moments found in table I added to the actual moments due to displacement, e.e. relative moments in table II multiplied by the factor 0.202 x I2 .

CHAPTER IV

Maximum Positive Moments at the Exterior spans



Position of Load Producing Max.Pos.Mom.at Ext.spans.

The reader is refered to the table at the end of chap.II for constants of loads.

1- First span

a) Left M= I.285 x $\frac{0.I38 \times I6\times20\times I2}{5}$ + 72.4 = 208.6 i.k.

b)Right M= " " = 208.6 " "

2)- Second span

a) Left M = Right M = M.(d.1.) = 72.4 " "

3- Third span

a) Left M = I.285 x $\frac{0.0422\times I2\times20\times I2}{5}$ + 72.4 = I02.9 " "

b)Right M = I.285 x $\frac{0.I58\times I2\times20\times I2}{5}$ + 72.4 = I87.2 " "

4) Fourth span

a) Left M = Right M = M.(d.l.) = 72.4 inch kips

5) Fifth span

a) Left M (3 kips) = $I.285 \times \frac{0.I56x3x20xI2}{5}$ = 28.9

(12 kips) = $I.285 \times \frac{0.0044xI2x20x12}{5}$ = 3.2

+ 72.4 M.d.1=72.4

= I04.5 " "

b) Right M (3kips) = $I.285 \times \frac{0.02x3x20xI2}{5}$ = 3.7

(12 kips) = $1.285 \times \frac{0.092x12x20x12}{5}$ = 68.3

+ M.d.l. = 72.4

= 144.4 " "

6- Sixth span

a) Left M = Right M = M.(d.l.) = 72.4 " "

7- Seventh span

Left M = $1.285 \times \frac{0.16x4x20}{5}$ xI2 + 72.4 = III.4 " "

Right M = $I.285 \times \frac{0.10x4x20}{5}$xI2 + 72.4 = 97.4 " "

8- Eigth span

Left M = $I.285 \times \frac{0.092xI6x20}{5}$xI2 + 72.4 = I63.3 " "

Right M = $I.285 \times \frac{0.0044xI6x20}{5}$xI2 + 72.4 = 76.4 " "

9) Moment distribution

Distribution of the above found moments proportionally to the relative rigidities of the members is shown in table IV.

Initial moments of the junction of the exterior wall is the same as that found in chapter III art.IO namely 24 inch kips & I6 inch kips at the bottom and top respectively.

Initial moments at the junction of the lower spans is the same as that found in chapter III art 9 namely I56 inch kips.

IO) Swaying of the Bridge due to Previous Loading

(Discussion presented in chapter III art.I2)

ΣM_b = I02.9- I54.2-2x36+I08F' = 0

$F' = \frac{123.3}{108}$ = I.207 kips

F" for interior members

-I08F" = 97.55+2.8-35.76-0.25+72.I+I2.75-3I.2

-2.70+46.5+8.0-22.3-5.7-42.0+6.83+45.84-I06.67

= + 45.79

$F'' = -\frac{45.79}{108}$ = - 0.424

$$F = F' + F'' = I.207 - 0.424 = \underline{\underline{0.784}} \text{ kips}$$

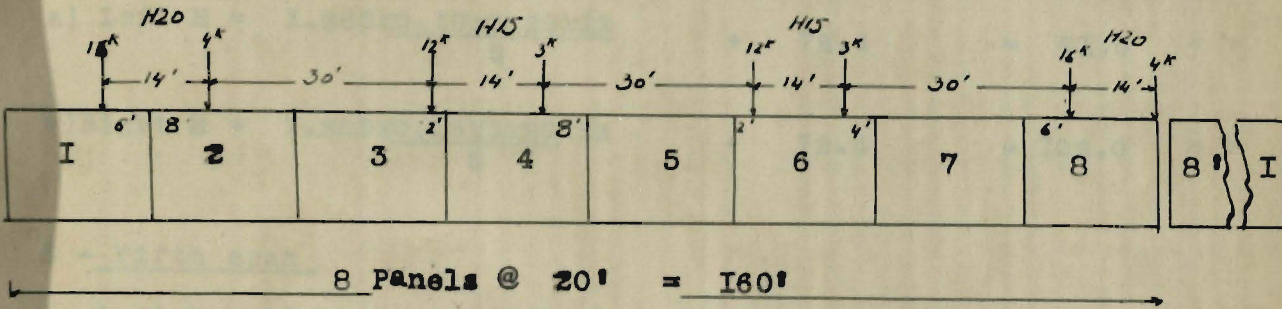
$$\frac{F}{F_1} = \frac{0.784}{4.424} = \underline{0.I77}$$ (where F_1 = force due to displacement of 1 ft.)

On table V are presented final moments due to moments found in table IV added to the actual moments due to displacement, i.e. relative moments in table II multiplied by the factor 0.I77x12.

CHAPTER V

Maximum Negative Moments
at the
First interior support

(used also for the other interior supports)



Position of Load producing Max.Neg.Mom.at firts Int.support

1- First span

a) Left M = $I.285x\frac{0.0422xI6x20xI2}{5}$ +72.4 = 76.6 i.k.

b)Right M = $I.285x\frac{0.I58xI6x20xI2}{5}$ +72.4 = 228.2 " "

2- Second span

a) Left M = $I.285x\frac{0.I6x\ 4x20}{5}x12$ +72.4 = III.9 " "

b)Right M = $I.285x\frac{0.I0x4x20}{5}xI2$ +72.4 = 97.I " "

3- Third span

a) Left $M = \frac{1.285 \times 0.00044 \times 12 \times 20 \times 12}{5} + 72.4 = 75.7$ 1.k.

b) Right $M = \frac{1.285 \times 0.092 \times 12 \times 20 \times 12}{5} + 72.4 = 140.5$ " "

4 - Fourth span

a) Left $M = \frac{1.285 \times 0.10 \times 3 \times 20 \times 12}{5} + 72.4 = 91.0$ " "

b) Right $M = \frac{1.285 \times 0.16 \times 3 \times 20 \times 12}{5} + 72.4 = 102.0$ " "

5 - Fifth span

Left M = Right M = M.(d.l.) = 72.4 " "

6 - Sixth span

a) Left M (12 k) $= \frac{1.285 \times 0.092 \times 12 \times 20 \times 12}{5} = 68.1$

(3 K) $= \frac{1.285 \times 0.02 \times 3 \times 20 \times 12}{5} = 3.7$

M.(d.l.) = 72.4 = 144.2 " "

b) Right M (12k) $= \frac{1.285 \times 0.0044 \times 12 \times 20 \times 12}{5} = 3.3$

(3 k) $= \frac{1.285 \times 0.156 \times 3 \times 20 \times 12}{5} = 28.9$

M.(d.L.) = 72.4 = 104.6 " "

7 - <u>Seventh span</u>

Left M = Right M = M.(d.l.) = 72.4 i.k.

8 - <u>Eighth span</u>

a) Left M. $= \frac{I.285x0.I58xI6x20xI2}{5} + 72.4$ = 228.5 " "

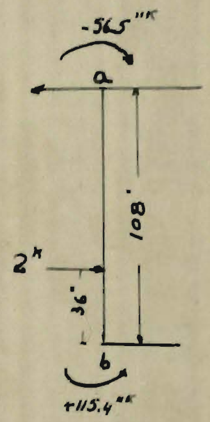
b) Right M $= \frac{I.285x0.0422xI6x20xI2}{5} + 72.4$ = 76.6 " "

The distribution of the above found fixed end moments is similar to the previous ones and is shown in table VI

9 - <u>Swaying of the Bridge due to Previous Loading</u>

ΣM_b = II5.4 - 56.5-2x36+I08F' = 0

F' = $\frac{I3.I}{I08}$ = 0.I2I kips



(F" for interior members)

- F" L = ΣM =	+ 72.5	- 45.9
	2.6	8.I
	I.5	I0I.77
	5.8	4.43
	26.3	I08.93
	6.7	
	19.1	- 269.I3
	3.9	
	34.23	
	3.72	
	48.I7	
	+224.52	

$$F'' = \frac{269.13 - 224.52}{108} = 0.415$$

$$F = F' + F'' = 0.536 \text{ kips}$$

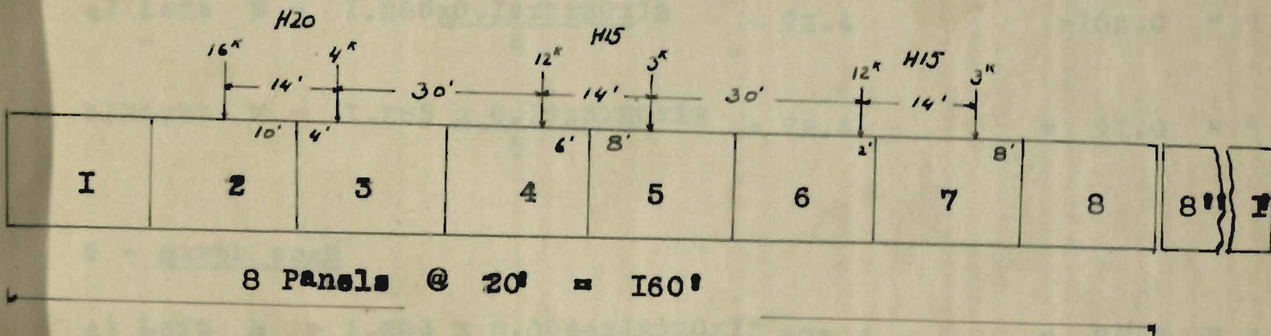
$$\frac{F}{F_1} = \frac{0.536}{4.427} = 0.1214$$

On table VII are presented final moments due to moments found in table VI, added to the actual moments due to displacement; i.e. relative moments in table II multiplied by the factor 0.1214x12.

C H A P T E R VI

Maximum Positive Moment in the Interior Spans

(also Max.Neg. in Ext. spans)



Position of Load Producing Max.Pos.Mom.in Int.spans

1) **First span**

a) Left M = Right M = M.(d.l.) = 72.4 1.k.

2) **Second span**

a) Left M = Right M = $I.285 \times \frac{0.I38 \times I6 \times 20}{5} \times I2 + 72.4$ = 208.6 " "

3) **Third span**

a) Left M = $I.285 \times \frac{0.I56 \times 4 \times 20}{5} \times I2 + 72.4$ = II0.9 " "

b) Right M = $I.285 \times \frac{0.02 \times 4 \times 20}{5} \times I2 + 72.4$ = 77.3 " "

4 - Fourth span

a) Left $M = 1.285 \times \frac{0.042 \times 12 \times 20 \times 12}{5} + 72.4 = 102.9$ i.k.

b) Right $M = 1.285 \times \frac{0.156 \times 12 \times 20 \times 12}{5} + 72.4 = 187.2$ " "

5 - Fifth span

a) Left $M = 1.285 \times \frac{0.16 \times 3 \times 20 \times 12}{5} + 72.4 = 102.0$ " "

b) Right $M = 1.285 \times \frac{0.10 \times 3 \times 20 \times 12}{5} + 72.4 = 91.0$ " "

6 - Sixth span

a) Left $M = 1.285 \times \frac{0.0044 \times 12 \times 20 \times 12}{5} + 72.4 = 75.5$ " "

b) Right $M = 1.285 \times \frac{0.092 \times 12 \times 20 \times 12}{5} + 72.4 = 140.5$ " "

7 - Seventh span

a) Left $M = 1.285 \times \frac{0.1 \times 3 \times 20 \times 12}{5} + 72.4 = 91.0$ " "

b) Right $M = 1.285 \times \frac{0.16 \times 3 \times 20 \times 12}{5} + 72.4 = 102.0$ " "

8 - Eighth span

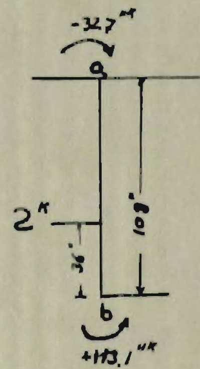
a) Left M = Right M m M.(d.l.) = 72.4 " "

The distribution of the above found fixed end moments is similar to the previous ones and is found on table VIII.

9 - <u>Swaying of the Bridge due to Previous Loading</u>

$\Sigma M_b = 113.1 - 32.7 - 2 \times 36 + 108F' = 0$

$F' = \frac{8.4}{108} = 0.0778$



(F" for interior members)

$-F''L$	= ΣM	=	+ 12.37	- 26.50
			9.50	1.60
			3.00	106.32
			4.97	79.30
			18.03	26.20
			65.40	-239.92
			48.70	
			1.20	
			27.73	
			7.45	
			35.73	
			+234.08	

$F'' = \frac{239.92 - 234.08}{108} = 0.054$

$F = F' + F'' = -0.078 + 0.054 = -0.014$ kips (a negligible value)

Therefore, Moments shown in table VIII will be taken as final moments producing Max. Pos. Mom. in the interior spans.

TABLE I

TABLE of Max. Neg. Moments over the Exterior Supports due to Dead Loads & Live Loads

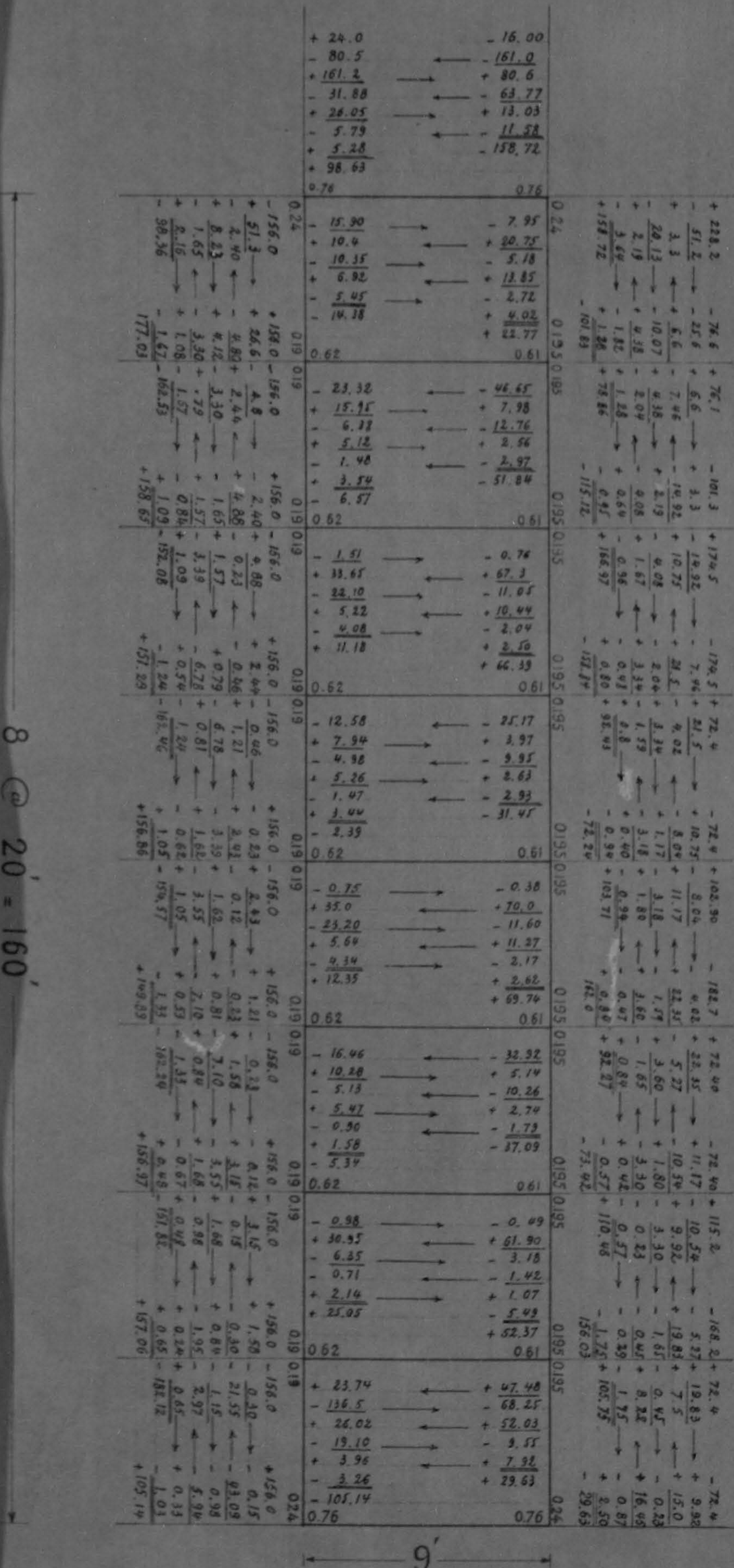


TABLE II

TABLE of Relative Moments Exerted on Members due to Assumed Sway

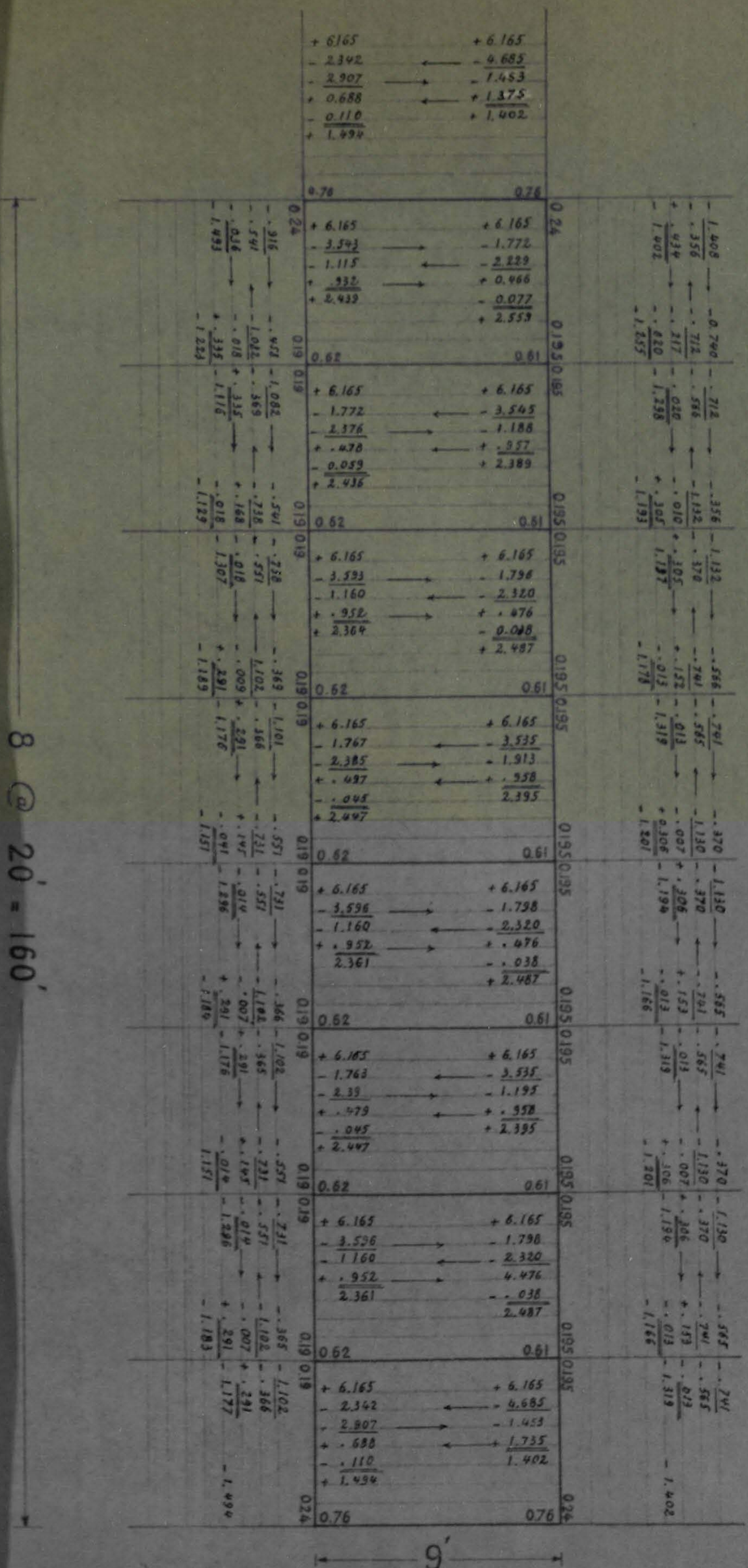


TABLE III

TABLE of Max. Neg. Moments over Exterior Supports

due to

D.L., L.L., and Swaying

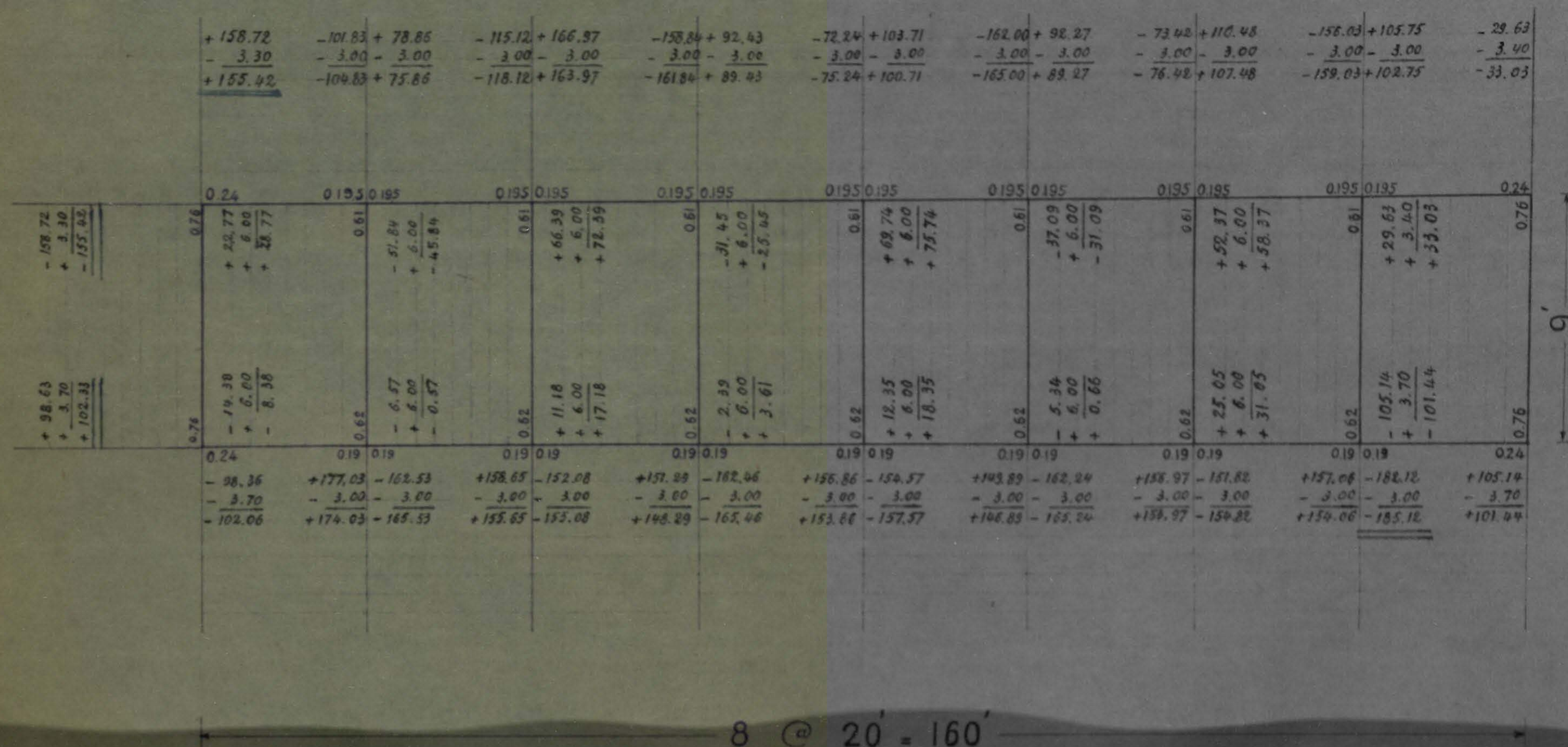


TABLE of Max.Pos. Moments for the Exterior Spans due to D.L.+L.L.

& Max.Neg. " " " Interior " " " "

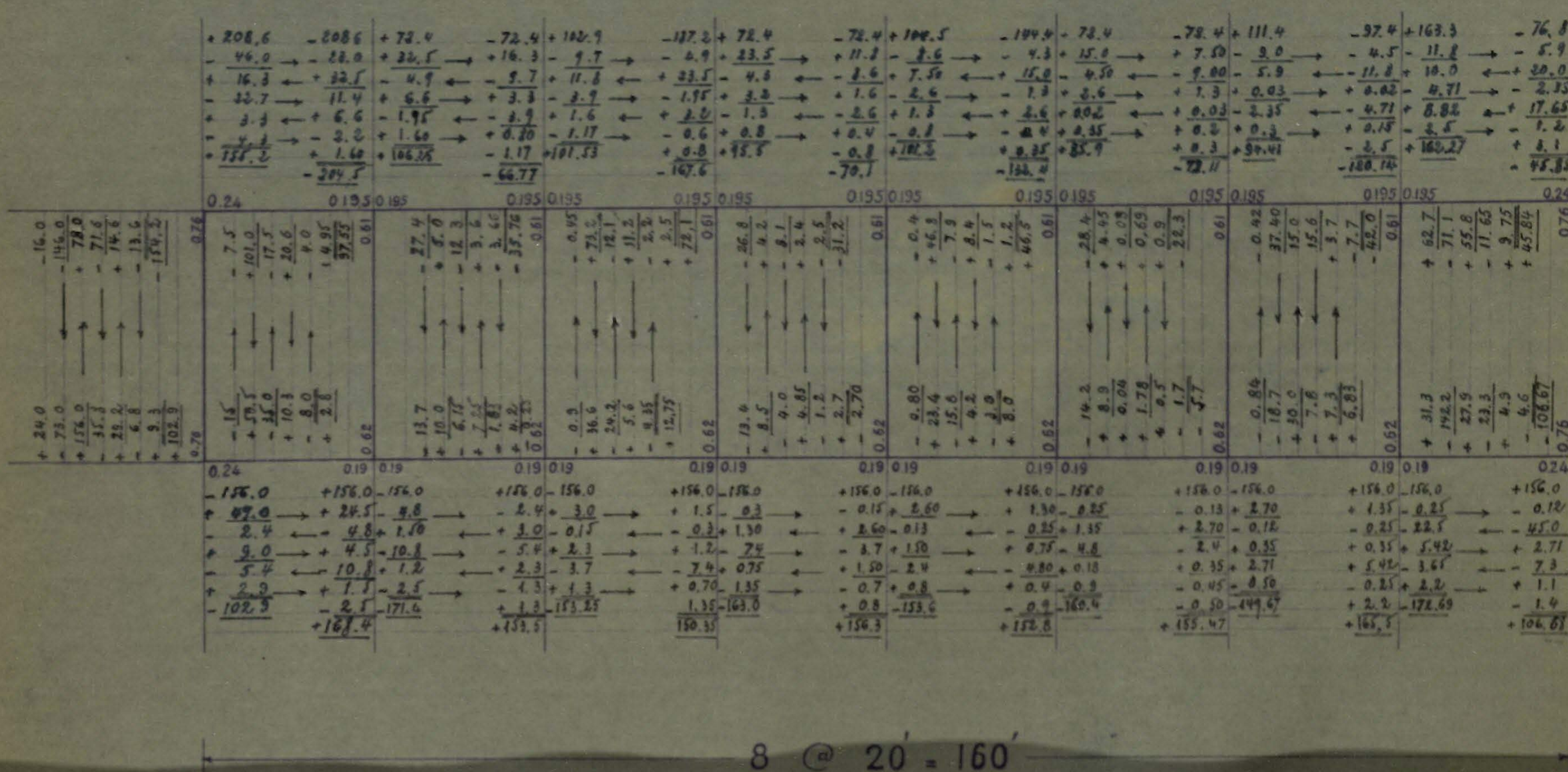


TABLE of Final Fixed End Moments due to D.L;, L.L., & Swaying which produces Max.Pos. Moments in exterior spans.

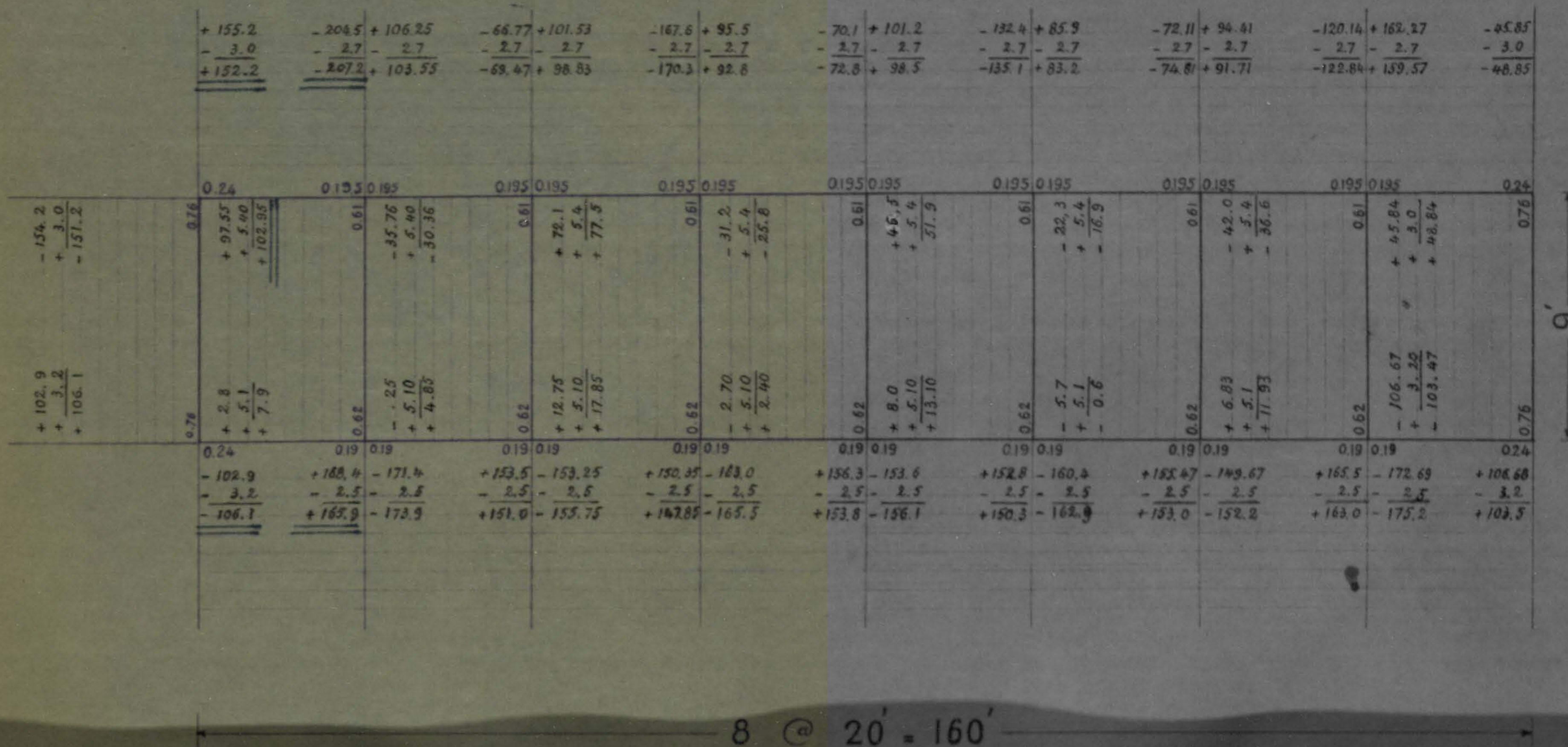


TABLE VI

TABLE of Max. Neg. Moments over First Interior Supports due to Dead Load & Live Load

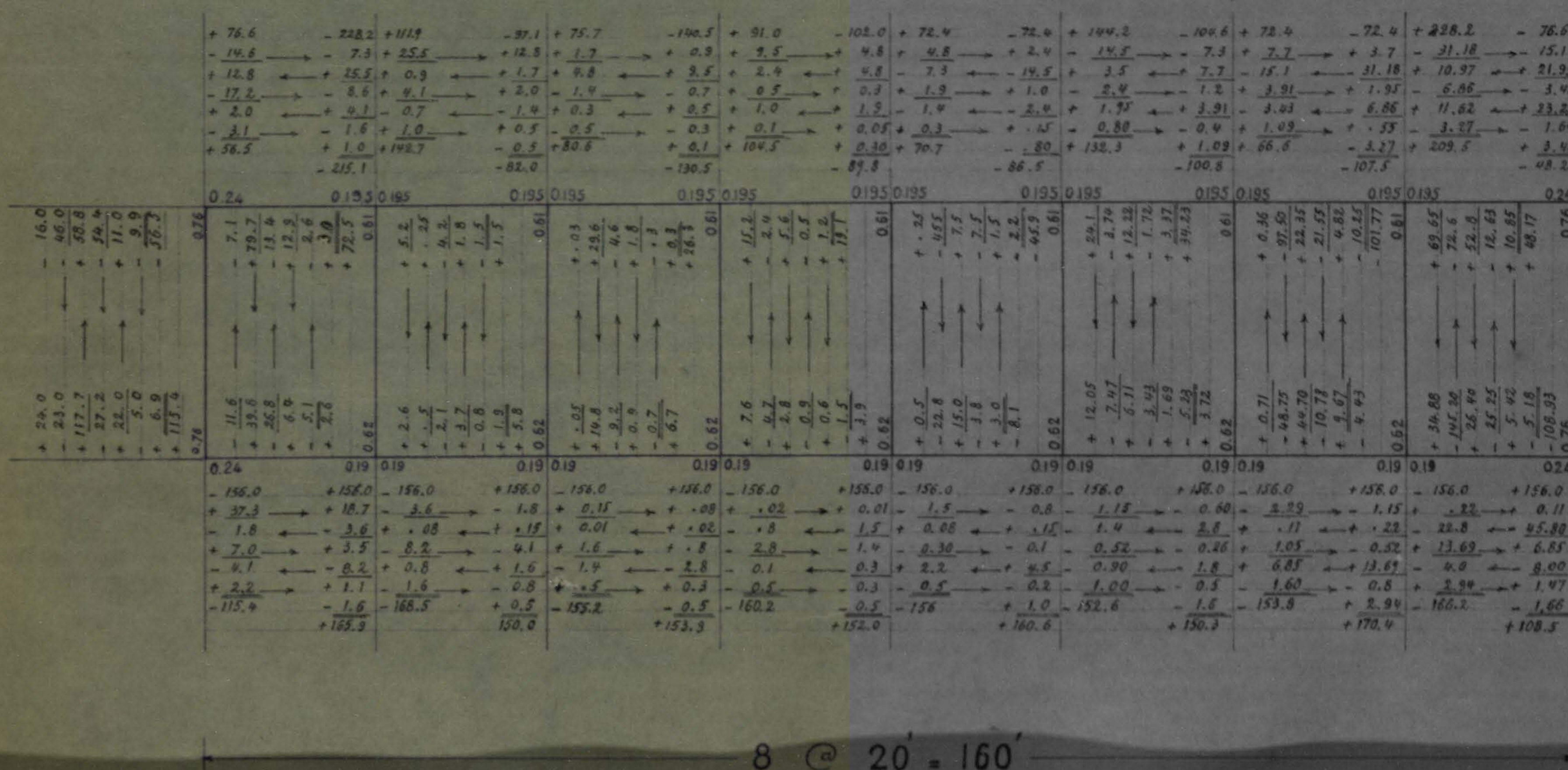


TABLE VII

TABLE of Final Fixed End Moments due to D.L., L.L.,& Swaying which produces Max.Neg.Moments over First Interior Supports

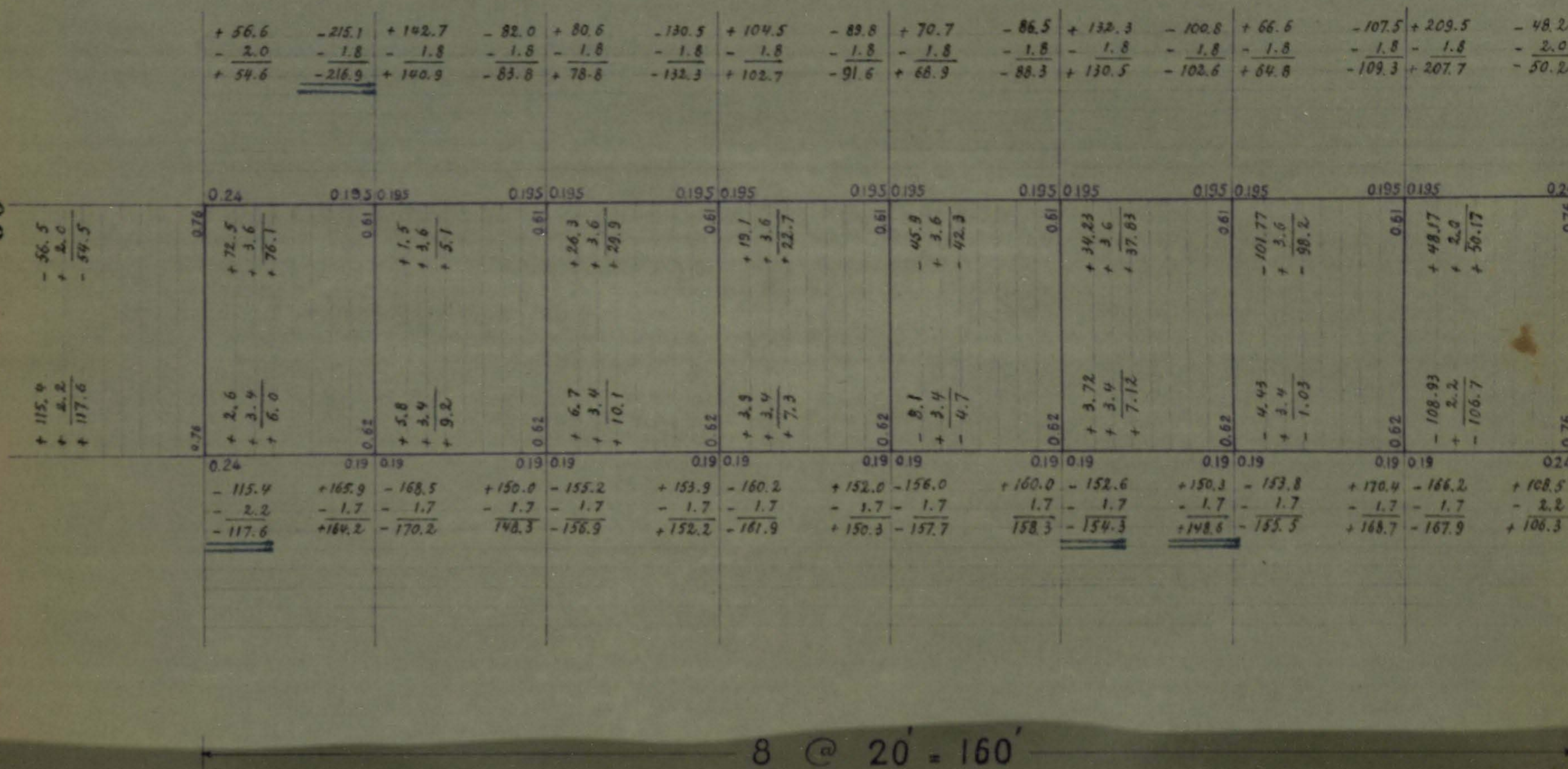
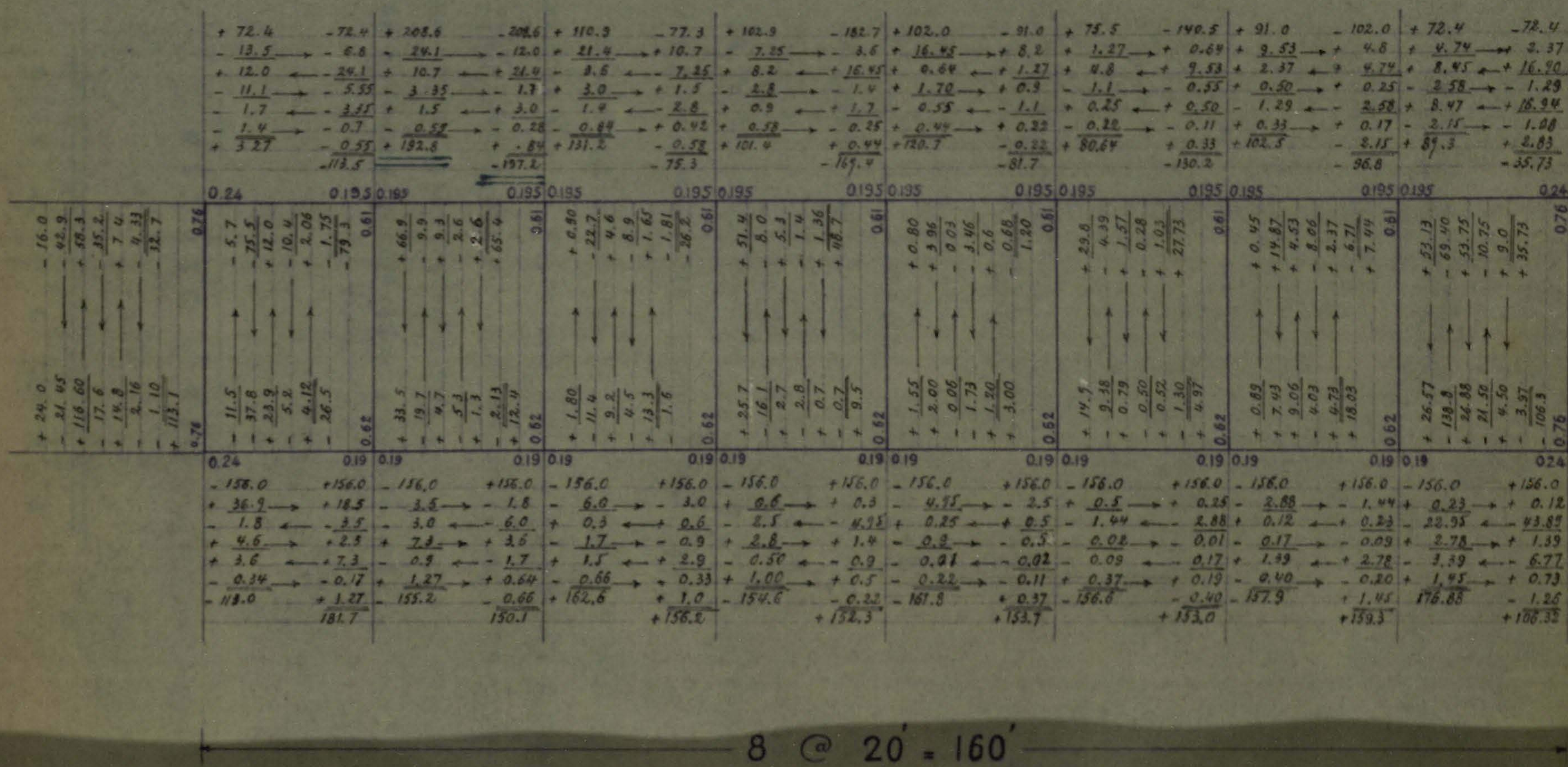


TABLE causing Max. Positive Moments in Interior Spans

and Max. Negative Moments for Exterior Spans

due to

D.L.,L.L., & Swaying



P A R T   T W O

T H E   D E S I G N

Introduction

Part one is essentially devoted to representation of the maximum fixed end moments occuring at critical sèctions which will be taken as true for the other similar sections being on the safe side, and owing to the impracticability of designing each slab or support differently.

Part Two presents the design of half of the bridge which is composed of a frame of 8 panels of 20 ft center to center each.

The frame is of reinforced concrete:

$f'c$ = 2500 lbs/s.i.

n = 12 fc = 800 lbs/s/i

f_s = 18000 lbs/s.i.

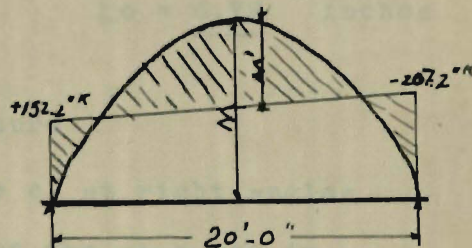
R = 123

J = 0.884

CHAPTER I

Design of The Upper Deck

1) The exterior spans



Refering to Table V,
Table of fixed end moments producing Max.Pos.Mom.in Ext.spans, it is shown that these moments are + I52.2 inch kips at the left support and - 207.2 inch kips at the right support.

Max. positive moment occurs under the wheel, i.e. at the center of the span.

M = Max.pos.mom. for a simply supported beam,

$= \frac{1}{8} wl^2 + I.285 \frac{Pl}{4B}$ (same notations as before) including impact

$= \frac{1}{8} x I80 x 20 x 20 x 12 + I.285 x \frac{I6000 x 20 x I2}{4 x 5}$

= I08000 + 246600 = 354600 inch pounds

M' = actual positive moment = M - average fixed end moments

$= 354600 - \frac{207200 + I52200}{2}$ = I74400 inch kips

$d = \sqrt{\frac{M}{bR}} = \sqrt{\frac{I74400}{I2 x I23}}$ = I0.8 inches

use total thickness D = I0.8 + I.2 = I2 inches

$$A_s = \frac{M}{f_s Jd} = \frac{174400}{18000 \times 0.884 \times 10.8} = \underline{\underline{1.03}} \text{ sq.inches}$$

use 5/8 inch Φ @ 3 1/2 inch c.to c. $A = \underline{1.05}$ sq.i.

$\Sigma o = \underline{6.73}$ inches

Reinforced for shrinkage and temperature

Use 1/2 inch Φ @ 14 inches c.to c. at right angles to axis of bridge at the lower face of the slab and in both ways at the upper face of the slab

2)- At the Exterior supports

Refering to tables of fixed end Moments, it is shown that table III give max.negative moment at the support equals to 155420 inch lbs.

$$d = \sqrt{\frac{155420}{12 \times 123}} = 10.25$$ use 10.8 inches

& d = 12 "

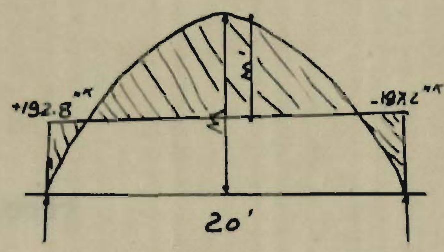
$$A_s = \frac{155420}{18000 \times 0.884 \times 10.8} = \underline{\underline{0.905}} \text{ sq.inches}$$

supplied by	5/8 inch	Φ	@ 7	inches	from adj. span	A=0.53	Σo=3.36
" "	1/2 "	Φ	@ 14	"	" Temp.Reinf.	A=0.17	Σo=1.35
add	5/8 "	Φ	@ 14	"	" over support	A=0.26	Σo=1.68
						0.96	6.39

3 - The Interior spans

Refering to TableVIII, Table of fixed end moments producing Max.pos.Mom. in Int. spans, it is shown that these moments are

+ I92.8 inch kips at the left support of the second span and - 197.2 inch kips at the right support.



M = 354600 inch lbs (see art.1)

$$M' = 35466 - \frac{192800 + 197200}{2} = 159600 \text{ inch lbs}$$

$$A_s = \frac{159600}{18000 \times 0.844 \times 10.8} = 0.94$$

use 5/8 inch Φ @ 3 1/2 inch A = I.05 Σo = 6.73

4 - At the Interior supports

Refering to tables of Max.fixed end moments, it is found that table VII has the max.negative moment at the right support of the first span and equals to 2I6900 inch lbs. All interior supports are designed the same

$$d = \sqrt{\frac{216900}{12 \times 123}} = 12.1 \text{ inches}$$

Therefore compressive reinforcement is required because our d is I0.8 inches only.

d' = distance from face of concrete to center of comp. reinf. = 1.1 inch

$$\frac{d'}{d} = 0.1$$

assume $J = 0.89$

$$\frac{f_s}{nf_c} = \frac{18000}{12+800} = 1.875$$

$$P = \frac{216900}{18000 \times 0.89 \times 10.8 \times 12 \times 10.8} = 0.0097$$

and $Pn = 0.0097 \times 12 = 0.116$

Therefore $P'n = 0.045$ (from graphs in reinf.concrete books where J is found 0.889 (check))

and $P' = \frac{0.045}{12} = 0.00375$

$$\times \frac{n}{n-1} = 0.0041$$

$A_s = 12 \times 10.8 \times 0.0097 = 1.26$ sq.inches

supplied by 5/8 inch φ @ 3 1/2 inch from adjacent spans

A = 1.05 Σo = 6.76

" 1/2 inch φ @ 7 inch from temp.Reinf. A = 0.34 Σo = 2.69

1.39 9.42

$A'_s = 12 \times 10.8 \times 0.0041 = 0.53$ sq.inches

supplied by 5/8 inch φ @ 3 1/2 inches from adjacent spans

A = 1.05 Σo = 6.76

CHAPTER II

DESIGN OF THE LOWER DECK

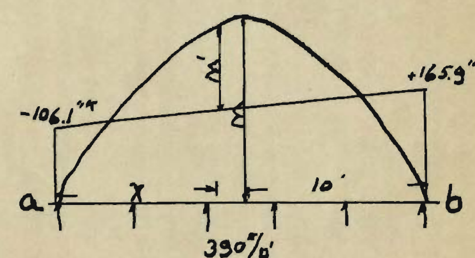
1 - The Exterior spans

Refering to tables of max.fixed end moments, it is seen that table V give minimum fixed end moments in the exterior spans of the lower deck, and thus maximum positive moment in these spans.

These fixed end moments are

- I06I00 inch lbs and

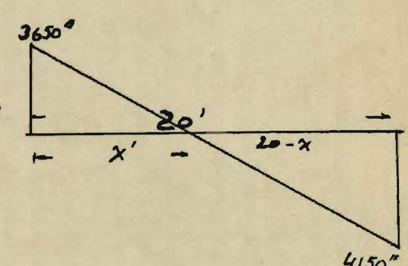
+ I56900 inch lbs.



M = moment due to resultant soil pressure

$= \frac{1}{8} wl^2$; w= soil pressure -w .of lower deck

= 5I2 - I25 = 390lbs/□'



$= \frac{1}{8}$ x 390x20x20xI2

= <u>234000</u> inch lbs

M'= Max.positive moment in the span. It occurs at the joint of zero shear.

X = Distance from left support of joint of zero shear.

$\Sigma M_a = 0 = 165900 - 106100 + 390 \times 20 \times 12 \times 12 - R_b \times 20 \times 12$

$R_b = 995800 = \underline{\underline{4150}}$ lbs

$\Sigma M_b = 0 = 165900 - 106100 - 390 \times 20 \times 10 \times 12 + 240\ R_a$

$R_a = \frac{876200}{240} = \underline{\underline{3650}}$ lbs

$R_a + R_b = 20 \times 390 = 7200 = 4150 + 3650$ (check)

$\frac{X}{3650} = \frac{20 - X}{4150}$

$4150\ X = 73000 - 3560\ X$

$X = 9.37$ ft.

$M_X = \frac{w}{2}\ (1X - X^2)\ 12$

$= \frac{390}{2}\ (20 \times 9.37 - 9.37^2) \times 12$

$= 195 \times 99.6 \times 12$

$= 233000$ inch lbs

$M' = 233000 - (106100 + \frac{165900 - 106100}{20} \times 9.37)$

$= 233000 - (106100 + 28000)$

$= \underline{\underline{98900}}$ inch lbs

$$d = \sqrt{\frac{98900}{12 \times 123}} = 8.2 \text{ inch} \quad \text{use} \quad d = 9 \text{ inches}$$

$$\text{and} \quad D = 9 + 1 = \underline{\underline{10}} \text{ inches}$$

$$A_s = \frac{98900}{18000 \times 0.884 \times 9} = \underline{\underline{0.69}} \text{ sq inch}$$

use 5/8 inch Φ @ 12 inches	A = 0.31	Σo = 1.96
and 3/4 " Φ @ 12 inches	A = 0.44	Σo = 2.36
	0.75	4.32

2 - <u>At the Exterior support</u>

Refering to tables of maximum fixed end moments, it is seen that table VII gives maximum negative moment at the exterior support of the lower deck equals to 117600 inch lbs

$$d = \sqrt{\frac{117600}{12 \times 123}} = 8.9 \text{ inches}$$

$$A_s = \frac{117600}{1800 \times 0.884 \times 9} = 0.82$$

supplied by 5/8" Φ @ 12 inches from adj.spans	A = 0.31	Σo=1.96
" " 1/2" Φ @ 12 " " Temp.Reinf.	A = 0.20	Σo=1.57
add 5/8" Φ @ 12 "	A = 0.31	Σo=1.96
	0.82	10.98

3 - The Interior spans

Refering to tables of max.fixed end moments it is seen that table VII, span 6, gives min.fixed end moments (at the interior supports I54.3 + I48.6 inch kips) and hence maximum positive moment for the interior spans.

$$M = wl^2 = 23400 \text{ inch lbs}$$

Let X = as before

$$\Sigma M_a = 0 = 148600 - 154300 + 390 \times 20 \times 10 \times 12 - 240 R_b$$

$$R_b = \frac{930300}{240} = 3880 \text{ lbs}$$

$$R_a = 7800 - 3880 = 3920 \text{ lbs}$$

$$\frac{X}{3920} = \frac{20-X}{3880}$$

$$3880\,X = 78400 - 3920\,X$$

X = I0.04 ft practically at the middle

$$M' = 23400 - \frac{(154300+148600)}{2}$$

$$= 23400 - 151950 = \underline{82050} \text{ inch lbs}$$

$$A_s = \frac{82050}{18000 \times 0.884 \times 9} = 0.574 \text{ sq inches}$$

use 5/8 inch φ @ 6 inches A= 0.6I Σo= 3.92

4 - At the Interior supports

Refering to tables of max.fixed end moments, it is seen that table III gives max.negative moment at the interior supports equals to I85I20 inch lbs.

$d = \frac{185120}{12 \times 123}$ = II.2 inches , Therefore comp.reinf.is required

b = 12 inches

$\frac{d'}{d}$ = 0.I

$\frac{f_s}{nf_c}$ = .875

assume J = 0.895

and $P = \frac{185120}{1800 \times 0.895 \times 9 \times 9 \times 12}$ = 0.0II83

Pn = 0.0II83xI2 = 0.I42

from tables P'n = 0.I22 & P' = 0.0I02 & J= 0.893(correct)

A_s = 0.0II83 x 9 x I2 = I.27

supplied by 5/8" φ @ 6 inches from adj.spans	A=0.6I	Σo= 3.92
" " I/2" φ @ 6 " " Temp.Reinf.	A=0.39	Σo= 3.I4
add 5/8 φ @ 12 "	A=0.3I	Σo= 1.96
	I.3I	9.02

$A'_s = 0.0I02 \times 9 \times I2 \times \frac{I2}{II} = I.20$ sq inch

supplied by	5/8" ϕ @ 6 inches from adj.spans	A= 0.6I	$\Sigma o = 3.92$
add	5/8" ϕ @ 6 "	A= 0.6I	$\Sigma o = 3.92$
		I.22	7.84

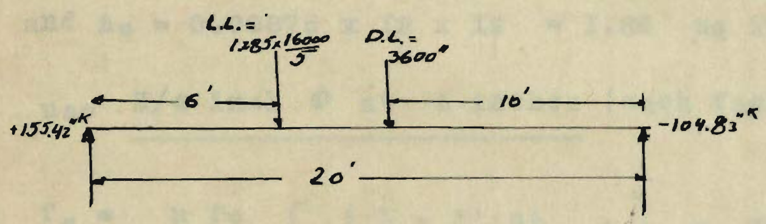
CHAPTER III

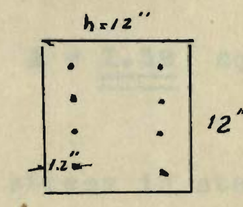
Design of the Vertical Walls

I - The Exterior walls

a) At the upper Part

Refering to tables of max.fixed end moments, it is seen that table III gives maximum fixed end moment at the top of the exterior supports equals to - I55420 inch lbs





$$M_b = 0 = I55420-I0483+I.285x \frac{I6000}{5} \times I4 \times I2+3600 \times I0 \times I2-20 \times I2\ R_a$$

$$R_a = \frac{I020640}{240} = 4255 \text{ lbs}$$

$$e = \text{eccentricity} = \frac{M}{R}$$

$$= \frac{I55420}{4255} = 36.54 \text{ inches} \qquad (\text{more than } \frac{12}{6})$$

This is case II where there is tension on one side -the external.

$$\frac{h}{e} = \frac{12}{36.54} = 0.329$$

f_c = 0.22 f'_c = 0.22 x 2500 = 550 lbs/sq.inch

= C $\frac{M}{bh^2}$ (where C is a constant of graphs in T. & M.)

$C = \frac{f_c bh^2}{M}$ = $\frac{550x12x12x12}{155420}$ = 6.I

$\frac{d'}{h} = \frac{1.2}{12}$ = 0.I

From Graphs, P_n and K are found to be: (T. & M.)

Pn = 0.I05 K = 0.33

P = $\frac{0.105}{12}$ = 0.00875

and A_s = 0.00875 x I2 x I2 = I.26 sq inches

use 3/4 inch Φ at 4 inches (each face) A = I.32 sq.i.

$f_s = n f_c \left[\frac{(1 - d'/n)}{K} - 1 \right]$ = Tensile stress in steel

= I2 x 550 ($\frac{1 - 0.1}{0.33}$ - 1) = 6600 (2.725 - 1)

= II380 lbs/sq.i (less than I8000) O.K.

$f_s' = n f_c \left(1 - \frac{1}{k} \times \frac{d'}{h} \right)$ = compression stress in steel

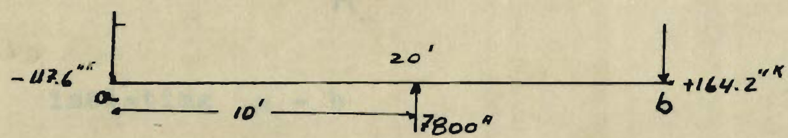
= I2 x 550 (1- $\frac{1}{0.33}$ x 0.I) = 6.600 (1-0.303)

= 4600 lbs/sq.i. (less than I6000) O.K.

b - <u>At Lower Part</u>

Using the same reinforcement of above for the lower part, the stresses in the concrete and steel should be investigated.

Refering to tables of max.fixed end moments, it is seen that table VII gives max. moment at the bottom of the first exterior wall equals to II7600 inch lbs



$\Sigma M_b = 0 = 164200 - 117600 - 7800 \times 10 \times 12 + 240\ R_a$

$R_a = \frac{889400}{240} = 3705$ lbs

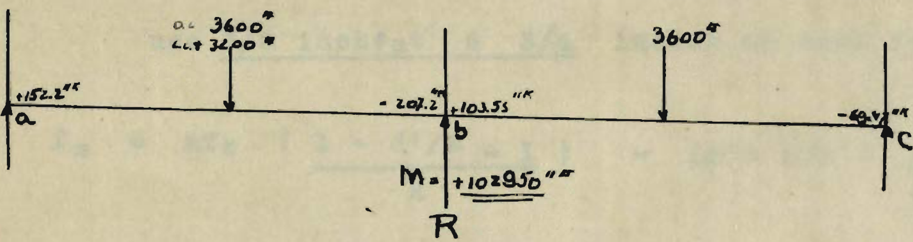
$e = \frac{M}{R} = \frac{117600}{3705} = 31.7$ inches

The value of e being less than that of the upper part of 36.54 , the stresses in the concrete and steel will obviously be less than those found above.

2 - The Internal walls

a) At the Upper Part

Refering to tables of max. fixed end moments, it is seen that table V gives max. fixed end moment in the interior walls equals to 102950 inch lbs.



Isolating a - b

$$\Sigma M_a = 0 = 152200 - 207200 - 6800 \times 120 + 240\ R'_b$$

$$R'_b = \frac{871000}{240} = 3630 \text{ lbs}$$

isolating b-c

$$\Sigma M_c = 0 - 103550 - 69470 + 3600 \times 120 - 240\ R''_b$$

$$R''_b = \frac{466080}{240} = 1940 \text{ lbs}$$

$$\text{total } R = R'_b + R''_b = 3930 + 1940 = \underline{\underline{5570}} \text{ lbs}$$

and M = 102950 inch lbs

$$\therefore e = \frac{102950}{5570} = 18.5 \text{ inches (more than } \frac{12}{6}) \because \text{ case II}$$

$$\frac{h}{e} = \frac{12}{18.5} = 0.65$$

$$C = \text{constant} = \frac{f_c b h^2}{M} = \frac{550 \times 12 \times 12 \times 12}{102950} = 9.24$$

Therefore from graphs.

$Pn = 0.046$ and $K = 0.283$

$P = \frac{0.046}{12} = 0.00383$

$A_s = 0.00383 \times 12 \times 12 = 0.55$ sq.i. (each face)

use 5/8 inch ϕ at 6 3/4 inches on each face $A = 0.55$ sq.i.

$f_s = nf_c \left(\frac{1 - d'/h}{k} - 1 \right) = 12 \times 550 \left(\frac{1 - 0.1}{0.283} - 1 \right)$

$= 6600 \ (3.18 - 1)$

$= 14400$ lbs/sq.i. (less than 18000) O.K.

$f'_s = nf_c \left(1 - \frac{1}{k} \times \frac{d'}{h} \right) = 12 \times 550 \left(1 - \frac{0.1}{0.283} \right)$

$= 6600 \ (1 - 0.353)$

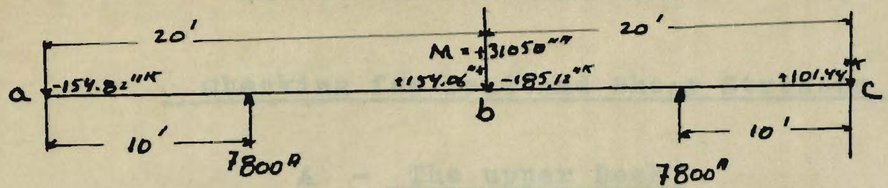
$= 4260$ lbs/sq.i. (less than 16000) O.K.

---

b) At Lower Part

Using the same reinforcement found above in the lower part, the stresses in the concrete and steel should be investigated.

Refering to tables of max. fixed end moments, it is shown that table III gives max. moment at the bottom of the interior walls equals to 31050 inch lbs.



$$\Sigma M_c = 101440 - 185120 - 7800 \times 120 + 240\, R'_b$$

$$R'_b = \frac{1019680}{240} = 4250 \text{ lbs.}$$

$$\Sigma M_a = 0 = 154060 - 154820 + 7800 \times 120 - 240\, R''_b$$

$$R''_b = \frac{935240}{240} = 3940 \text{ lbs}$$

$$R_b = 4250 + 3940 = \underline{\underline{8190}} \text{ lbs}$$

$$M = 31050 \text{ inch lbs}$$

$$e = \frac{M}{R} = \frac{31050}{8190} = 3.80 \text{ lbs}$$

This value of e being less than the previous value of 18.5, the stresses in the steel and concrete will obviously be less than those found above in (a).

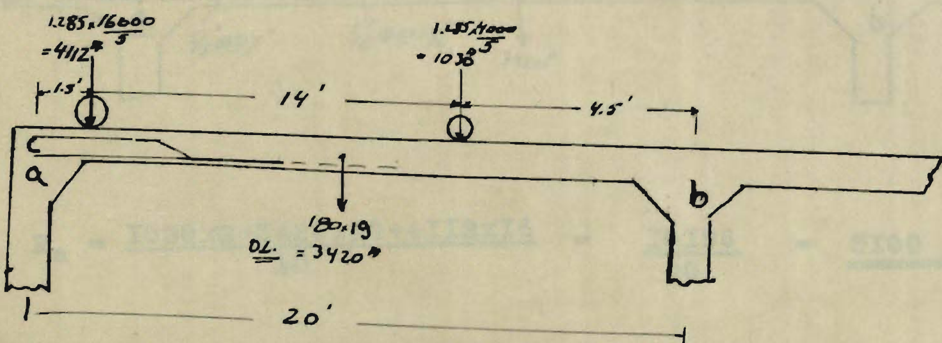
CHAPTER IV

Checking for Bond and Shear Stresses

A - The upper Deck

1) At the Exterior supports

The critical sections for bond and shear are at the end of the haunches and at the point of inflection. Wheel loads will be put on these points then bond and shear stresses will be examined.



$$R_a = \frac{1030x4.5+3420xI0+4II2xI8.5}{20} = \frac{II4940}{20} = 5730 \text{ lbs.}$$

V = shear at end of haunch (1.5 ft from center of support)

$= 5730 - I.5 \, xI80 = \underline{\underline{5460}}$ lbs

and $v = \frac{V}{bJd}$ b = 12 J = 0.884 d = I0.8

$$= \frac{5460}{I2x0.884xI0.8} = \frac{5460}{114.6} = \underline{\underline{47.7}} \text{ lbs p.s.i.}$$

(less than 50) O.K.

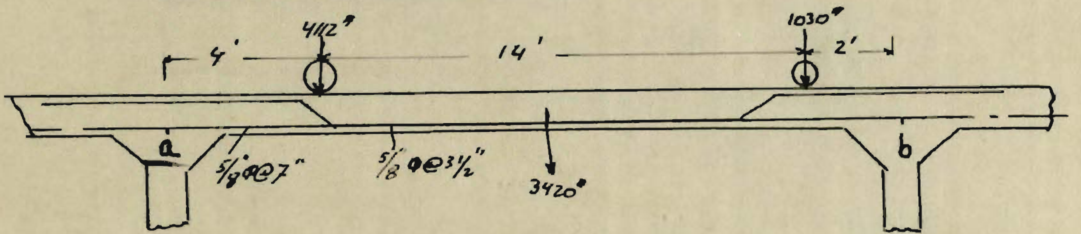
Refering to the design of upper deck at exterior support,

$\Sigma o = 6.39$

$$U = \frac{V}{\Sigma o\ Jd} = \frac{5460}{6.39x0.884xI0.8} = \frac{5460}{61} = \underline{\underline{89.5}} \text{ lbs p.s.i.}$$

(less than 100) **O.K.**

2) <u>At the Point of Inflection</u>



$$R_a = \frac{I030x2+3420xI0+4II2xI6}{20} = \frac{I0I96}{20} = \underline{5I00} \text{ lbs}$$

$$V = 5I00 - I80 \times 4 = 5I00 - 720 = 4380 \text{ lbs}$$

$$v = \frac{4380}{I2x0.884+I0.8} = \underline{38.2} \text{ lbs p.s.i.}$$

(less than 50) **O.K.**

$$\Sigma o = \frac{6.73}{2} = 3.36$$

$$U = \frac{4380}{3.36x0.884xI0.8} = \underline{\underline{I,36}} \text{ lbs p.s.i.}$$

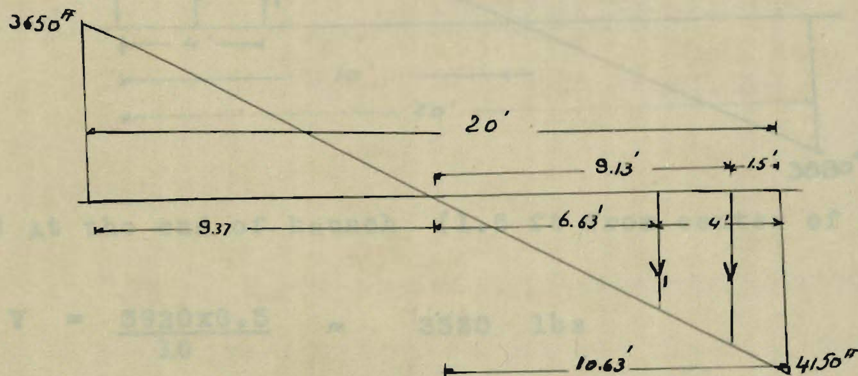
(special anchorage provided)

B - The Lower Deck

1) The exterior spans

a) at the end of haunches

Refering to chapter II, it is seen that the maximum shear in the exterior spans is 4150 lbs.



V = shear at end of haunch (1.5 ft from center of support)

$$= \frac{4150 \times 9.13}{10.63} = 3570 \text{ lbs}$$

$$v = \frac{3570}{12 \times 0.884 \times 9} = \underline{\underline{37.4}} \text{ lbs p.s.i.} \qquad \text{O.K.}$$

Σo = 9.02 inches

$$u = \frac{3570}{9.02 \times 0.893 \times 9} = \underline{\underline{49}} \text{ lbs.p.s.i.} \qquad \text{O.K.}$$

b - Point of inflection (4' for center of support)

$$V_1 = \frac{4150 \times 6.63}{10.63} = 2590$$

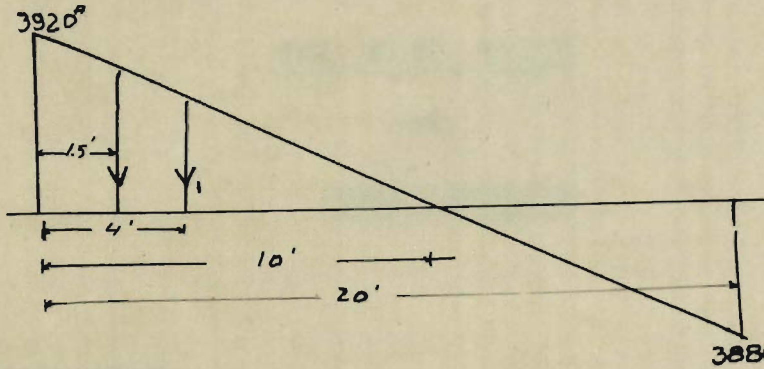
Σo = 2.36

$$u = \frac{2590}{2.36 \times 0.884 \times 9} = 138 \text{ lbs p.s.i.}$$

(special anchorage is provided)

2 - <u>The Interior spans</u>

Refering to chapter II for max. shear in the interior spans it is found to be 3920 lbs.



a) At the end of haunch (1.5 ft from center of support)

$$V = \frac{3920 \times 8.5}{10} = 3330 \text{ lbs}$$

$$v = \frac{3330}{12 \times 0.893 \times 9} = \underline{\underline{34.5}} \text{ lbs} \qquad \text{O.K.}$$

$$\Sigma o = 9.02$$

$$\mu = \frac{3330}{9.02 \times 0.893 \times 9} = \underline{\underline{46}} \text{ lbs} \qquad \text{O.K.}$$

b) <u>At point of inflection</u> (4 ft. from center of support)

$$V_1 = \frac{3920 \times 6}{10} = 2352 \text{ lbs}$$

$$\Sigma o = \frac{3.92}{2} = 1.96$$

$$\mu = \frac{2352}{1.96 \times 0.884 \times 9} = \underline{\underline{150}} \text{ lbs p.s.i.}$$

(special anchorage provided) O.K.

P A R T T H R E E

ANTI-SCOURING STRUCTURES

THE WING WALLS

and

WOOD SHEETING

Introduction:

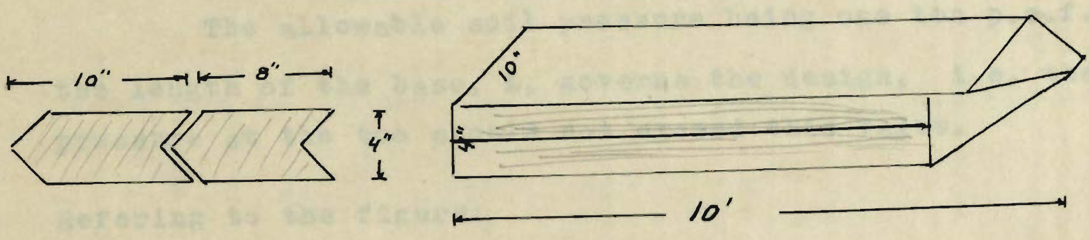
Wing walls at the approaches of the bridge on both sides are required to prevent the fill behind the approaches from falling into the river, and to prevent the river water from seeking behind the external walls and causing scouring.

These walls are built of reinforced concrete simultaneously with the bridge. Being ten feet high (the height of the bridge) they are to be ten feet long measured from the back of the exterior walls.

To prevent water from causing scouring under the foundations of the bridge (the lower deck), wood sheeting is driven on both sides of the foundation.

The railroad bridge constructed over the Beirut river near our bridge has been protected against scouring by a cut of wall 2.5 meters deep after it has been found that maximum scouring occured to a depth of 2 meters.

Wood sheets 4 inches thick, 10 inches wide, and 10 feet long are driven on both sides of the foundation. Graves have to be cut on the sides to prevent seepage of water as far as possible



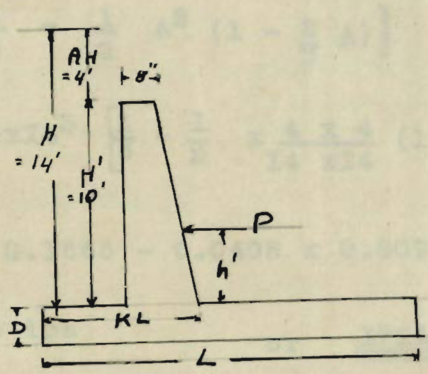
The bed of the river is levelled to receive the foundations. After the wood piles have been driven on both sides, excavation begins and soiling of eight inches thick is compactly placed. Then a layer of four inches of concrete is poured on top of which, the reinforced concrete foundation (the lower deck) is poured.

The wing walls'computations follow:

CHAPTER 1

The Concrete

1) The Dimensions



The allowable soil pressure being one ton p.s.f., the length of the base, L, governs the design, i.e. the pressure at the toe should not exceed this value.

Refering to the figure:

Use $KL = \frac{L}{2}$ $\quad K = \frac{1}{2}$ $\quad H' = 10$ ft

θ = angle of repose = 30° for gravel, sand and clayfill

C = 0.33 a constant of equation

w = 110 lbs p.cu.ft.

$$L = H\sqrt{\frac{c\cos\theta-}{3K(1-k)}}$$

$$= 14\sqrt{\frac{0.33x0.867}{1.5x0.5}} = I4\sqrt{0.3805}$$

$= \underline{8.6}$ ft $\qquad$ Use 8 ft - 6 inches

$$P = \frac{cwh^2}{2}$$

$$= \frac{0.33 \times 110 \times 14^2}{2} = 3558 \text{ lbs}$$

M = moment at the base of stern

$$= Cwh^3 \left[\frac{1}{6} - \frac{1}{2} A^2 (1 - \frac{2}{3} A)\right] \quad \text{where } A = \frac{4}{14}$$

$$= 0.33 \times 110 \times 14^3 \left[\frac{1}{6} - \frac{1}{2} \times \frac{4 \times 4}{14 \times 14} (1 - \frac{2}{3} \times \frac{4}{14})\right]$$

$$= 107840 \; (0.1666 - 0.0408 \times 0.809) = 107840 \times 0.1336$$

$$= \underline{\underline{14370}} \text{ ft lbs} \quad \text{or} \quad \underline{\underline{172440}} \text{ inch lbs}$$

h = distance of resultant soil pressure (P) from top of base

$$= \frac{M}{P} = \frac{14370}{3585} = 4.04 \text{ ft}$$

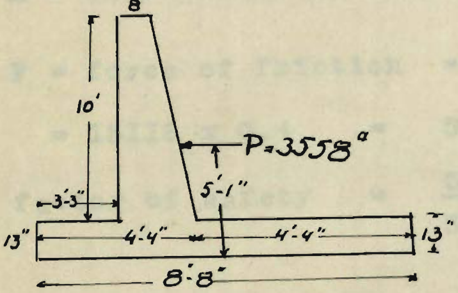
$f_c = 750$ $f_s = 18000$ $n = 12$ $R = 110$ $J = 0.889$

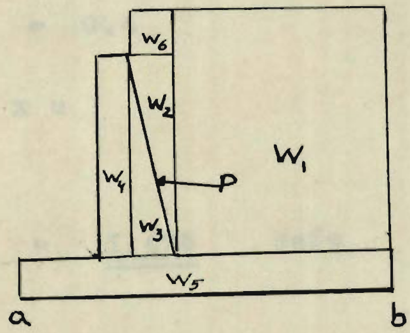
$$d = \sqrt{\frac{172440}{12 \times 111}} = \underline{\underline{11.3}} \text{ inches} \quad D = 13 \text{ inches}$$

use 13 inches thickness of base also.

---

2 - Test for overturning





Load	Weight in lbs		dist. of cent.	M_a
w1	$\frac{13}{3} \times 14 \times 110$	= 6680	78	520800
w2	$\frac{5}{12} \times \frac{14}{2} \times 110$	= 320.7	51.3	164600
w3	$\frac{5}{12} \times 10 \times 150$	= 625	48.7	30460
w4	$\frac{8}{12} \times 10 \times 150$	= 1000	43	43000
w5	$\frac{13}{12} \times 8 \times 67 \times 150$	= 1423	52	74000
w6	$\frac{5}{12} \times 4 \times 40$	= 183	49.2	9060
		13118		841920

$M_p = 3558 \times 61 = 217100$

Factor of safety for overturning

$= \frac{841920}{217100} = \underline{\underline{3.88}}$ safe

3 - Test for sliding

M = coefficient of friction = 0.4

F = force of friction = $\Sigma w \times u$

= I3II8 x 0.4 = 5247

factor of safety = $\frac{5247}{3558}$ = I.475 safe

4 - Test for soil Pressure

The position of the resultant from point (a)

$= \frac{\Sigma M_a - M_p}{\Sigma w}$

$= \frac{841920 - 217100}{13118}$ = 47.6 inches

e = eccentricity = 52 - 47.6 = 4.4 inches = 0.3665 ft.

s = soil pressure in lbs p.s.f.

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

where $c = \frac{L}{2}$ = 4.33 ft.

$M = \Sigma w \times e$ = I3II8 x0.3665

$I = \frac{bd^3}{12} = \frac{1.08 \times 8.67^3}{12}$ ft^4

S_t = soil pressure at toe

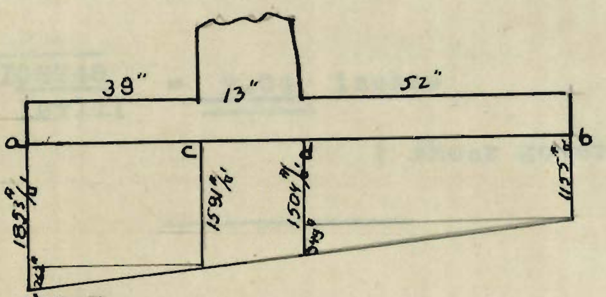
$= \frac{13188}{1 \times 8.67} + \frac{13118 \times 0.3665 \times 4.33 \times 12}{1.08 \times 8.67^3}$

= I504 + 349 = I853 lbs p.s.f. (less than 2200)

S_h = Soil pressure at heel

= 1504 - 349 = 1155 lbs p.s.f.

5 - Thickness of Toe



a) for shear

slope of pressure $= \frac{1853 - 1155}{104} = 6.71$ lbs p.i.

$P'_c = 1853 - 6.71 \times 39 = 1591$ lbs.

$P'_d = 1155 + 6.71 \times 52 = 1504$ lbs.

$P_c = \frac{1853 + 1591}{2} \times \frac{39}{12} = 5600$ lbs

V_e = P_c - Weight of toe

$= 5600 - \frac{39}{12} \times \frac{13}{12} \times 150 = 5600 - 530 = 5070$ lbs

$d = \frac{5070}{12 \times 0.889 \times 40} = \frac{5070}{426} = 11.9$ inches (less than 12)

b) for Moment

$$\Sigma M_e = 159 \times \frac{39}{12} \times \frac{39}{2} + \frac{262 \times 39}{12 \times 2} \times \frac{39 \times 2}{3} - \frac{530 \times 39}{2}$$

$$= 108000 + 11070 - 10330$$

$$= 108740 \text{ inch lbs.}$$

$$d = \sqrt{\frac{108740}{12 \times 111}} = \underline{\underline{9.04}} \text{ inches}$$

(shear governs)

---

6 - Thickness of Heel

a) for shear

$$V_e = w_1 + \text{wt. of heel} - \frac{(P_d + P_b)}{2} \times \frac{52}{12}$$

$$= 6680 + \frac{52}{12} \times \frac{13}{12} \times 150 - \frac{(1504 + 1155)}{2} \times \frac{52}{12}$$

$$= 6680 + 700 - 5755 = 1525 \text{ lbs}$$

$$d = \frac{1525}{12 \times 0.889 \times 40} = \underline{\underline{3.6}} \text{ inches} \quad \text{(less than 12)}$$

b) for Moment

$$\Sigma M_d = 7380 \times \frac{52}{2} - 1155 \times \frac{52}{12} \times \frac{52}{2} - \frac{349 \times 52}{2 \times 12} \times \frac{52}{3}$$

$$= 191900 - 13000 - 9450$$

$$= 55450$$

$$d = \sqrt{\frac{52450}{12 \times 111}} = \underline{\underline{6.26}} \text{ inches} \quad \text{(less than 12)}$$

CHAPTER II

The Reinforcements

1) For the stem

M = I72440 inch lbs & d = I3 - I.5 = II.5 inches

slope = 5 inches in I20 inches θ is less than 15° (no correction)

$$A_s = \frac{172440}{18000 \times 0.889 \times 11.5} = 0.935 \text{ s.i.}$$

use 5/8 inch ϕ at 4 inches $A = 0.92$ $\Sigma o = 5.89$

2) For the Toe

M = I08740 inches lbs d = I3 - I.5 = II.5 inches

$$A_s = \frac{108740}{18000 \times 0.889 \times 11.5} = 0.59 \text{ s.i.}$$

use 1/2 inch ϕ at 4 inches $A = 0.59$ $\Sigma o = 4.71$

3) For the Heel

M = 52450 inch lbs d = 11.5

$$A_s = \frac{52450}{18000 \times 0.889 \times 11.5} = 0.285$$

Use 3/8 inch ϕ at 4 1/2 inch $A = 0.29$ $\Sigma o = 3.14$

CHAPTER III

Test for Bond & Shear Stresses

1 - at the stem

V = P = 3558 lbs d = 11.5 inches

$$v = \frac{3558}{12 \times 0.889 \times 11.5} = 43.8 \text{ lbs p.s.i.}$$

(less than 50) O.K.

Σo = 5.89

$$u = \frac{3558}{5.89 \times 0.889 \times 11.5} = 89.4 \text{ lbs p.s.i.}$$

(less than 100) O. K.

---

2 - At the Toe

V = 5070 lbs d = 11.5

$$v = \frac{5070}{12 \times 0.889 \times 11.5} = 41.5 \text{ lbs p.s.i.}$$

O. K.

Σo = 4.71 inch

$$u = \frac{5070}{4.71 \times 0.889 \times 11.5} = 105 \text{ lbs p.s.i.}$$

O. K.

---

3 - At the Heel

V = 1525 lbs d = 11.5 inches

$$v = \frac{1525}{12 \times 0.889 \times 11.5} = 12.5 \text{ lbs p.s.i.}$$

O. K.

Σo = 3.14

$$v = \frac{1525}{3.14 \times 0.889 \times 11.5} = 47.4 \text{ lbs p.s.i.}$$

O. K.

P A R T   F O U R

QUANTITIES & COST

1 - Wood $\frac{320 \times 12}{8} \times 2$ = 960 piles number 1000

$960 \times 10 \times \frac{10'}{12} \times \frac{4'}{12}$ = 2667 cu. ft.

= 75 cubic meters 80

2 - Excavation in water

320 x 100 x 3 = 96000 cubic ft.

= 2800 cubic meters 3000

3 - Soling (Blocage)

$320 \times 100 \times \frac{8}{12}$ = 21333 cubic ft

= 620 cubic meters 650

4 - Poor concrete (200 kg/m3)

$320 \times 100 \times \frac{4}{12}$ = 10667 cubic ft

= 310 cubic meters 325

5 - Reinforced concrete

Lower deck : $320' \times 100' \times \frac{10}{12}$ = 26700 cubic ft.

Verticals: $2 \times 9 \times 9' \times \frac{12'}{12} \times 100'$ = 16200 " "

Upper deck : $320' \times 100 \times \frac{12'}{12}$ = 32000 " "

Haunches : $100' \times \frac{12}{12} \times \frac{12}{12} \times 2 \times 16$ = 1600 " "

Wing walls

Base : $10' \times 8.67' \times \frac{13'}{12}$ = 94

stem : $10' \times 10' \times \frac{8+13}{2 \times 12}$ = 87.5

181.5 x 4 = 7260 " "

83760 " "

= 2365 cub.m. 2375

6 - Steel for reinforcement 338 tons 340

7 - Tiles for sidewalls	2 x 4.5x100	= 900	s.m.	900
8 - Curb stresses	2 x 100	= 200	m.r.	200
9 - 3" Pipes	2 x 100	= 200	" "	200
10 - 1/2" plasters =(100+3x30)		= 3000	m^2	3000

C O S T

	Quantity	Unit cost LL.	Total cost LL.
1 - Wood piles	1000 n°	10.00	I0000
	80 m^3	200.00	I6000
2 - Excavation in water	3000 m^3	4.00	I2000
3 - Soling	650 m^3	5.00	3250
4 - Poor concrete	325 m^3	30.00	9750
5 - Reinforced concrete	2375 m^3	80.00	I90000
6 - Steel (round)	340 tons	600.00	I44000
7 - Tiles (laid)	900 m^2	4.00	3600
8 - Curb stones	200 m.r.	3.00	600
9 - 3" pipes	200 m.r.	8.00	I600
10 - 1/2 " plastering	3000 m^2	I.50	4500
			395300
		supervision	4700
			400000

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T H E   E N D

FOUR SHEETS OF DRAWINGS ARE ATTACHED TO THIS THESES

SHEET 1 - THE LAY-OUT

SHEET 2 - THE LONGITUDINAL PROFILE

SHEET 3 - THE FRONT VIEW
&
THE TRANSVERSE CROSS-SECTION

SHEET 4 - THE LONGITUDINAL CROSS-SECTION
(SHOWING DETAILS OF REINFORCEMENT)