ŌYA BRIDGE

OVER

CHIKUMA-GAWA AT ŌYA IN KAMIKAWA-MURA

NAGANO-KEN

Designed by A. Gotō. August 1926

Traced by W. Takahashi

Assumed loading of bridge

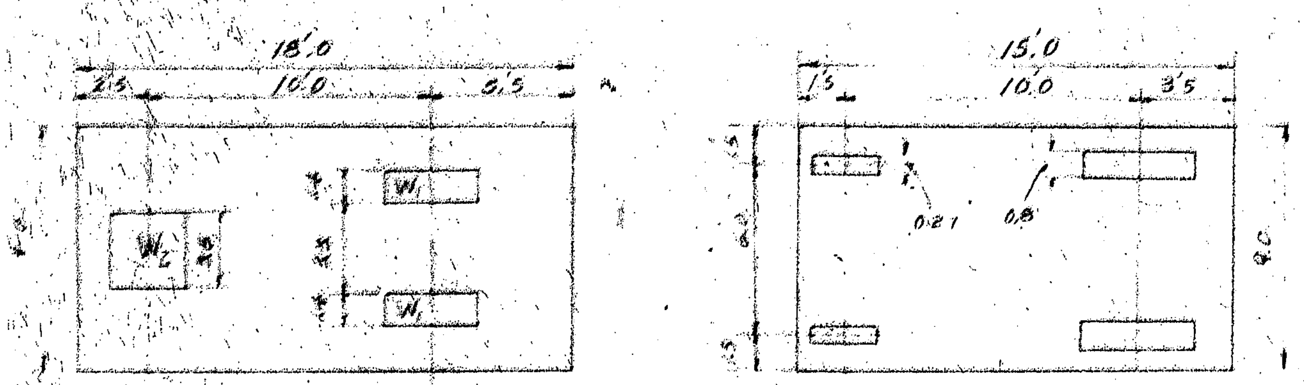
1. uniform live load

$$q^{kg}/m^2 = \frac{100,000}{170+l} \geqq 500^{kg}/m^2$$

where l = span length in meter.

2. Road Roller (17640#)
3. Motor truck (13280#)

Distributions of wheel concentrations both road roller and motor truck are shown as in sketches below.



Assnmed occupied area

Road Roller — Motor Truck.

Road Roller: $W_1 = 5290^{\#}$, $W_2 = 7060^{\#}$
Total wt. $= 2W_1 + W_2 = 17640^{\#}$

Motor Truck: $W_1 = 1660^{\#}$, $W_2 = 4980^{\#}$
Total wt. $= 2(W_1 + W_2) = 13280^{\#}$

Notes: One road roller on span is assumed without impact. When the trucks are running side by side the road way is filled with the uniform by distributed load and the impact is taken as specified below

$$\text{Impact}(i) = \frac{20^m}{60+l} \geqq 0.3$$

where l = loaded length in meter.

Assumed constants and unit stresses.

1. Censtants

Temperature change ± 30C

Expansion coefficient par centi-grade = 0,000012

Modulas of Elasticity

$E_e = 2100000\ ^{kg}/_{cm^2} \doteqdot 30000000\ ^{\#}/_{\square''}$ for steel.

$E_b = 140000\ ^{kg}/_{cm^2} \doteqdot 2000000\ ^{\#}/_{\square''}$ for concrete

$n = E_e \div E_b = 15$

Weights.

$2400\ ^{kg}/_{m^3} \doteqdot 150\ ^{\#}/_{ft^3}$ for reinforced concrete.

7850 „ $\doteqdot 490\ ^{\#}/_{ft^3}$ for steel.

2. Working unit stresses of concrete (1:2:4) and steel.

(a.) Conerete

Bearing	$45\ ^{kg}/_{cm^2} = 640\ ^{\#}/_{\square''}$
Direct compression	$35\ ^{kg}/_{cm^2} = 500\ ^{\#}/_{\square''}$
Compression from bending	$45\ ^{kg}/_{cm^2} = 640\ ^{\#}/_{\square''}$
Compression due to moment and direct comp. eombined	for strut $35\ ^{kg}/_{cm^2} = 500\ ^{\#}/_{\square''}$; for arch $45\ ^{kg}/_{cm^2} = 640\ ^{\#}/_{\square''}$
Punching shear	$9\ ^{kg}/_{cm^2} = 128\ ^{\#}/_{\square''}$
Shearing stress	$4\ ^{kg}/_{cm^2} = 57\ ^{\#}/_{\square''}$
Bond stress (Plain bar)	$6\ ^{kg}/_{cm^2} = 85\ ^{\#}/_{\square''}$

(b) Reinforcing bar

Tension	$1200\ ^{kg}/_{cm^2} = 17000\ ^{\#}/_{\square''}$
Compression	$1200\ ^{kg}/_{cm^2} = 17000\ ^{\#}/_{\square''}$
Sheoring	$900\ ^{kg}/_{cm^2} = 12800\ ^{\#}/_{\square''}$

(C) Structural steel

Tension. Axial tension on net section $1200\ ^{kg}/_{cm^2} = 17000\ ^{\#}/_{\square''}$

Compressiom $\left\{ \begin{array}{l} \text{Axial compression on cross section of columns} \\ = 1500\left(1 - 0{,}0055\,\frac{l}{r}\right)\ ^{kg}/_{cm^2} \leqq 1000\ ^{kg}/_{cm^2} = 14200\ ^{\#}/_{\square''} \end{array} \right.$

where "l" is the length of the member in cm.

and "r" the least radius of gyration in cm.

Direct compression on steel castings $17000^{\#}/_{\square''}$

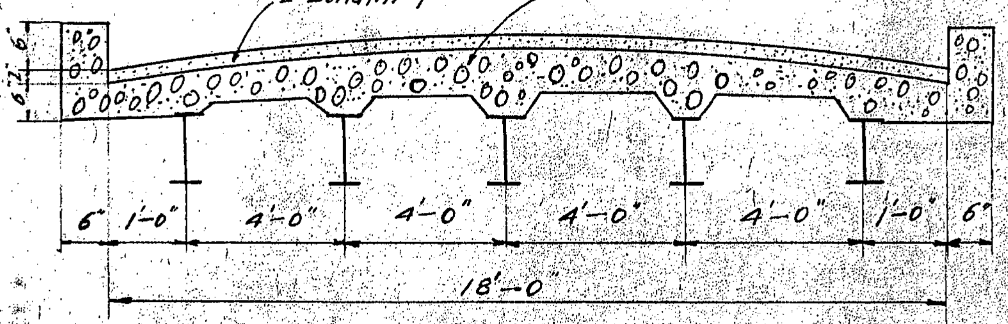
Bending. Bending on extreme fiber of rolled shapes built sections, girders and steel castings; net section $1200^{kg}/_{cm^2} = 17000^{\#}/_{\square''}$
On extreme fiber of pins. $1800^{kg}/_{cm^2} = 25600^{\#}/_{\square''}$

Shearing. Shearing on plate and pins $900^{kg}/_{cm^2} = 12800^{\#}/_{\square''}$
Shearing on shop driven rivets $850^{kg}/_{cm^2} = 12000^{\#}/_{\square''}$
Shearing on field driven rivets and turned bolts $750^{kg}/_{cm^2} = 10600^{\#}/_{\square''}$

Bearing Pins $1800^{kg}/_{cm^2} = 25600^{\#}/_{\square''}$
Shop driven rivets $1700^{kg}/_{cm^2} = 24100^{\#}/_{\square''}$
Field driven rivets and Turned bolts $1500^{kg}/_{cm^2} = 21300^{\#}/_{\square''}$
Expansion rollers per cm. $45d^{kg} = 640d^{\#}/_{inch}$
where "d" is the diameter of the roller in cm.
On masonry $45^{kg}/_{cm^2} = 640^{\#}/_{\square''}$

Wind load — $\begin{cases} 135^{\#} \text{ per ft run for upper chord.} \\ 270^{\#} \text{ per ft. run for lower chord.} \end{cases}$

Floor Slab



Assumed dead load

6" Reinforced concrete slab $75^{\#}/\square''$

2" Soliditit $24^{\#}/\square''$

$99^{\#}/\square''$

Curb $\frac{6 \times 14}{144} \times 150 = 87.5\ ^{\#}/ft.$

Overhanging arm of slab

Dead load moment road way $99 \times 0.5 = 49.5'^{\#}$

curb $87.5 \times 1.25 = 109.4'^{\#}$

$R_b = 186.5^{\#}$ $M_D = 158.9'^{\#}$

Live load moment. weight of one wheel $= 4980^{\#}$

$4980 \times (1+0.3) \times 0.5 = 3237'^{\#}$

$R_L = 6474^{\#}$ $M_L = 3237'^{\#}$

Load distribution $= \frac{2}{3}(l+b)+a = \frac{2}{3}(1.5+1.2)+1.1 = 2.9'$

1^{ft} strip moment $= 3237 \div 2.9 = 1116.2'^{\#}$

Dead and live load mt. $= 158.9 + 1116.2 = 1275.1'^{\#}$

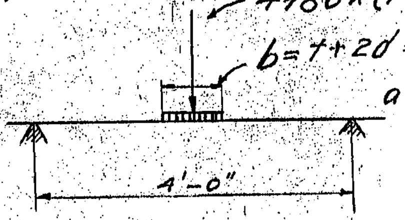
Intermediate slab

Dead load moment of slab $= \frac{wl^2}{10} = \frac{99 \times 4^2}{10} = 1584'^{\#}$

Live load moment $4980 \times (1+0.3) = 6474^{\#}$

$b = t + 2d = 0.8 + 0.4 = 1.2^{ft}$

$a = 0.7 + 0.4 = 1.1$



Moment at center $= \frac{6474}{2} \times 2 - \frac{6474}{2} \times \frac{1.2}{4} = 5503'^{\#}$

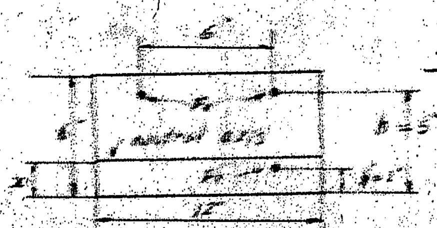
Considering continuity $M_c = 5503 \times \frac{8}{10} = 4402.4'^{\#}$

Load distribution $= \frac{2}{3}(l+b) + a = \frac{2}{3}(4+1.2)+1.1 = 4.6^{ft.}$

$1^{\#}$ strip moment $= 4402.4 \div 4.6 = 957^{1\#}$

Dead and live load moment of slab $= 158.4 + 957 = 1115.4^{1\#}$

Check the section of overhanging arm moment $= 1275.1^{1\#}$



Sectional area of reinforcement

$Fe' = 1 - \frac{1}{2}"\phi$ bar $= 0.1963$

$Fe = 2 - \frac{1}{2}\phi$ bar $= 0.3926$

$$x = -\frac{n(Fe+Fe')}{b} + \sqrt{\left\{\frac{n(Fe+Fe')}{b}\right\}^2 + \frac{2n}{b}(hFe + h'Fe')} = 1.7''$$

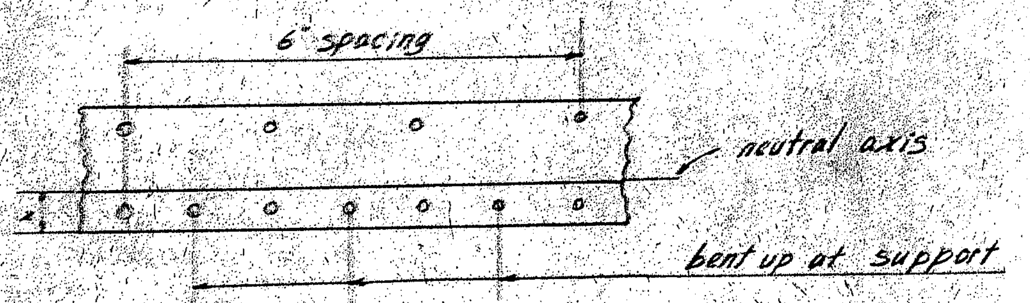
$$J = \frac{bx^3}{3} + nFe'(x-h')^2 + nFe(h-x)^2 = 85.23^{(in)^4}$$

$$\sigma_b = \frac{Mx}{J} = \frac{1275.1 \times 12 \times 1.7}{85.23} = 305^{\#}/\square''$$

$$\sigma_e' = \frac{n\sigma_b(x-h')}{x} = \frac{15 \times 305 \times 0.7}{1.7} = 1884^{\#}/\square''$$

$$\sigma_e = \frac{n\sigma_b(h-x)}{x} = \frac{15 \times 305 \times 3.3}{1.7} = 8881^{\#}/\square''$$

Check the section of intermediate span moment $= 1115.4^{1\#}$



Consider the one foot strip

Sectional area of reinforcement.

compression side $1 - \frac{1}{2}"\phi$ bar $= 0.1963$

Tension side $2 - \frac{1}{2}\phi$ bar $= 0.3926$

$$x = -\frac{n(Fe+Fe')}{b} + \sqrt{\left\{\frac{n(Fe+Fe')}{b}\right\}^2 + \frac{2n}{b}(hFe + h'Fe')} = 1.7''$$

$$J = \frac{bx^3}{3} + nFe'(x-h')^2 + nFe(h-x)^2 = 85.23^{(in)^4}$$

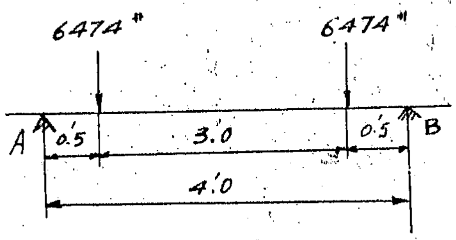
$$\sigma_b = \frac{Mx}{J} = \frac{1115.4 \times 12 \times 1.7}{85.23} = 267^{\#}/\square''$$

$$\sigma_e' = \frac{n\sigma_b(x-h')}{x} = \frac{15\times267\times(1.7-1)}{1.7} = 1649\ \#/\square''$$

$$\sigma_c = \frac{n\sigma_b(h-x)}{x} = \frac{15\times267\times(5-1.7)}{1.7} = 7774\ \#/\square''$$

Shear for intermediate span

Dead load shear $= \frac{6}{10}\,wl = \frac{6}{10}\times99\times4 = 238^{\#}$



$R_A = R_B = 6474^{\#}$

Considering continuity

Shear due to live load $= \frac{6474\times5}{4} = 8092^{\#}.5$

1^{ft} strip shear $= 8092^{\#}.5 \div 4.6^{ft} = 1759^{\#}$

Total shear $= 238 + 1759 = 1997^{\#}$

Section modulas about neutral axis (S)

$$= bx\,\frac{x}{2} + n\,F_e'(x-h')$$

$$= 12\,\frac{1.7^2}{2} + 15\times0.1963\times(1.7-1) = 19.4\ (in)^3$$

Moment of inertia $(J) = 85.23\ (in)^4$

Unit shear $(P_0) = \frac{QS}{bJ} = \frac{1997\times19.4}{12\times85.23} = 37.9\ \#/\square'' < 57\ \#/\square''$

Bond stress $(P_h) = \frac{QS}{JU} = \frac{1997\times19.4}{85.23\times1.5708\times3} = 96\ \#/\square'' < 85\times(1+0.5)$

Shear for overhanging arm

Dead load shear $= 99 + 87.5 = 186.5^{\#}$

1^{ft} strip shear (live load) $= \frac{6474}{2.9} = 2232.4^{\#}$

Total shear $= 186.5 + 2232.4 = 2418.9^{\#}$

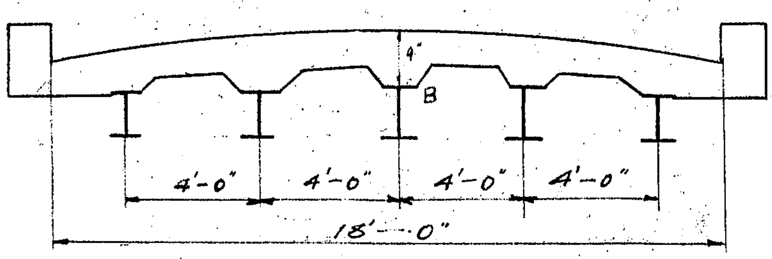
Unit shear $(P_0) = \frac{QS}{bJ} = \frac{2418.9\times19.4}{12\times85.23} = 46\ \#/\square'' < 57\ \#/\square''$

Bond stress $(P_h) = \frac{QS}{JU} = \frac{2418.9\times19.4}{85.23\times3\times1.5708} = 117\ \#/\square'' < 85(1+0.5)$

Stringers

Intermediate stringers

span length = 12'–0"   spaced 4'–0"



Dead load reaction at B $= \frac{11}{10} wl = \frac{11}{10} \times 99 \times 4 = 435.6$

$\fallingdotseq 436^{\#}$

Concrete fillet say $= 24^{\#}$

Stringer assumed $= 35^{\#}$

Total dead load $= 495^{\#}$ Per ft.

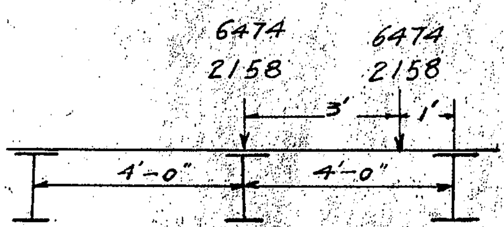
Dead load moment $= \frac{1}{8} \times 495 \times 12^2 = 8910^{1\#}$

Dead load shear $= \frac{1}{2} \times 495 \times 12 = 2970^{1\#}$

Live load

Motor truck with 30% impact

wheel	4980	1660
30% impact	1494	498
	$6474^{\#}$	$2158^{\#}$



Concentrated wheel load $6474 \times (1 + \frac{1}{4}) = 8093^{\#}$

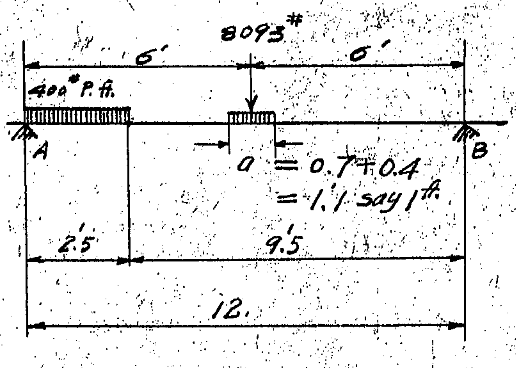
$2158 \times (1 + \frac{1}{4}) = 2698^{\#}$

Live load moment

$R_A = \frac{400 \times 2.5 \times 10.75 + 8093 \times 6}{12}$

$= \frac{1000 \times 10.75 + 8093 \times 6}{12}$

$= 4942^{\#}$



Live load moment at center $= R_A \times 6 - 1000 \times 4.75 - \frac{8093}{2} \times \frac{1.0}{4}$

$$= 4942 \times 6 - 1000 \times 4.75 - 4046.5 \times 0.25$$

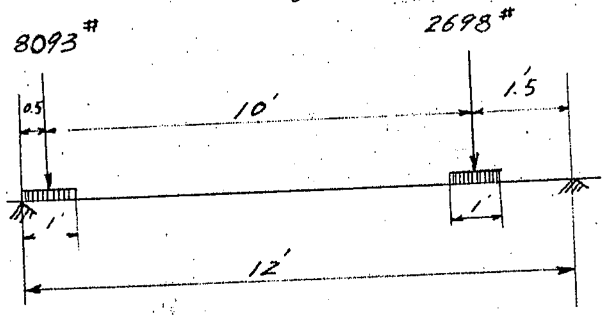
$$= 23890^{\prime\#} = \frac{Pl^2}{8}$$

$$P = 1327^{\#}$$

Summory of dead and live load moments

$$= 8910 + 23789 = 32699^{\prime\#} = \frac{ql^2}{8}$$

$$q = 1817^{\#}$$



Live load shear $= \dfrac{8093 \times 11.5 + 2698 \times 1.5}{12}$

$$= 7419^{\#}$$

Section modulus required $= \dfrac{32699 \times 12}{15600} = 25.2^{(in)^3}$

Section modulus of $12'' \times 5'' \times 31.99^{\#}$ I Beam $= 36.69^{(in)^3}$.

Deflection of Beam

$$\frac{\delta}{l} = \frac{5}{24}\left(\frac{\sigma}{E}\right)\left(\frac{P}{q}\right)\left(\frac{l}{h}\right)$$

$$= \frac{5}{24} \times \frac{15600}{30000000} \times \frac{1327}{1817} \times \frac{12 \times 12}{12} = \frac{1}{1053} < \frac{1}{800}$$

Actual stress $= \dfrac{32699 \times 12}{36.69} = 10695^{\#}/\square''$

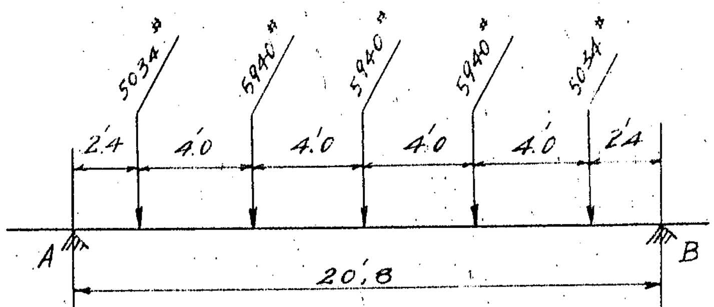
Allowed stress $= 1200\left(1 - 0.012\,\dfrac{l}{b}\right)$

$$= 1200\left(1 - 0.012 \times \frac{9}{5}\right) = 1100^{kg}/\square cm.$$

$$\doteqdot 15600^{\#}/\square''.$$

Floor Beam. Intermediate floor beam
span length 20,8 spaced 12',

Dead load
Concentration from intermediate stringer
$495 \times 12 = 5940^{\#}$
Concentration from end stringer
$\{(186.5^{\#} + \frac{1}{2} \times 99 \times 4) + 35^{\#}\} \times 12 = 419.5 \times 12 = 5034^{\#}$

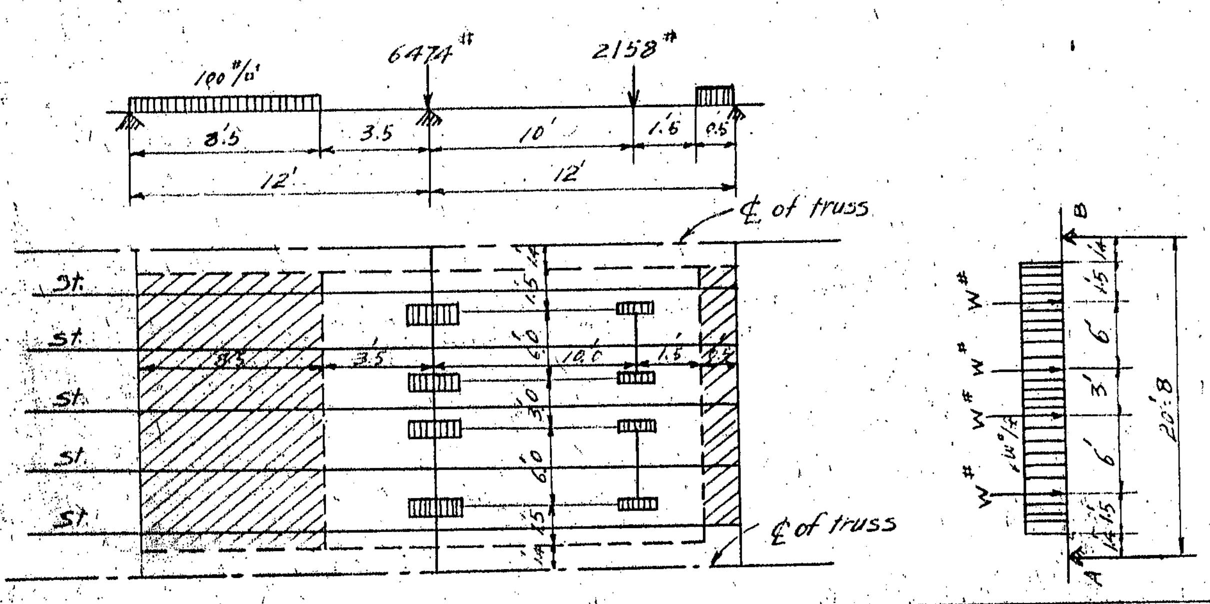


$R_A = R_B = 5034 + 1.5 \times 5940 = 13944^{\#}$

Moment at center due to concentration
$= 13944 \times 10.4 - 5034 \times 8 - 5940 \times 4 = 80986'^{\#}$
Floor beam assumed 20" x 85# I Beam
Moment due to own weight $= \frac{1}{8} \times 85 \times 20.8^2 = 4597'^{\#}$

Summary of dead load moment $= 80986 + 4597 = 85583'^{\#}$

Live load moment.
Motor truck with impact plus uniform load.



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Reaction from motor truck

$$W = 6474 + 2158 \times \frac{2.0}{12} = 6834^{\#}$$

Reaction due to uniform load (front and rear)

$$W = 100 \times \left(8.5 \times \frac{\frac{8.5}{2}}{12} + 0.5 \times \frac{\frac{0.5}{2}}{12}\right) = 302^{\#}$$

$$R_A = R_B = 6834 \times 2 + 302 \times 9 = 16386^{\#}$$

Live load moment

$$= 16386 \times 10.4 - 6834 \times (7.5 + 1.5) - 302 \times 9 \times 4.5$$
$$= 96677^{\prime\#}$$

Summary of dead and live load moments

$$= 85583 + 96677 = 182260^{\prime\#}$$

Shear due to dead load

Reaction from stringers	13944
Dead load of floor beam $= \frac{1}{2} \times 85 \times 20.8 =$	884
Total dead load shear	$= 14828^{\#}$

Shear due to live load $= 16386^{\#}$

Summary of dead and live load shears

$$= 14828 + 16386 = 31214^{\#}$$

$$\text{Section modulus required} = \frac{182260 \times 12}{15600} = 140 \ (in)^3$$

Section modulus of 20" × 85 # I Beam = 150.2 $(in)^3$

$$\text{Actual stress} = \frac{182260 \times 12}{150.2} = 14580 \ \#/\square''$$

$$\text{Allowed stress} = 1200\left(1 - 0.012 \times \frac{4 \times 12}{7.05}\right)$$
$$\fallingdotseq 1200 \times 91 \fallingdotseq 1100 \ kg/\square cm = 15600 \ \#/\square''$$

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End Floor Beam.

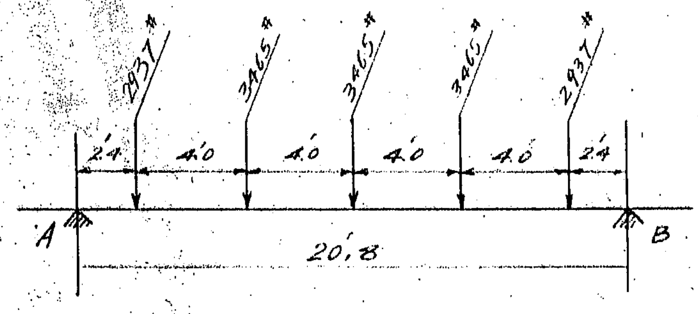
Span length 20.'8 , projection of stringer 1'

Dead load

Concentration from intermediate stringer and cantilever bracket.

$495 \times (\frac{1}{2} \times 12 + 1) = 3465^{\#}$

$419.5 \times (\frac{1}{2} \times 12 + 1) = 2937^{\#}$



$R_A = R_B = 2937 + 1.5 \times 3465 = 8135^{\#}$

Moment at center due to concentration

$= 8135 \times 10.4 - 2937 \times 8 - 3465 \times 4 = 47248'^{\#}$

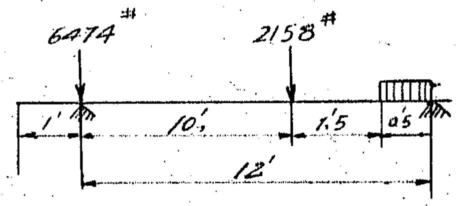
Floor beam assumed 20" x 75# I Beam

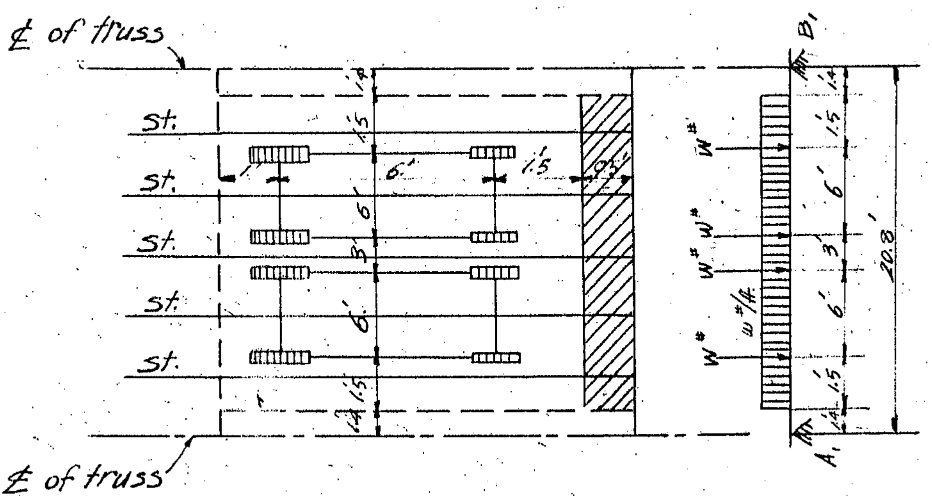
Moment due to own weight $= \frac{1}{8} \times 75 \times \overline{20.8}^2 = 4056'^{\#}$

Summary of dead load moment $= 47248 + 4056 = 51304'^{\#}$

Live load moment.

Motor truck with impact plus uniform load.





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Reaction from motor truck

$$W = 6474 + 2158 \times \frac{2}{12} = 6834$$

Reaction due to uniform load

$$W = 100 \times 0.5 \times \frac{\frac{0.5}{2}}{12} = 1^{\#}$$

$$R_A = R_B = 6834 \times 2 + 1 \times 9 = 13677^{\#}$$

Live load moment

$$= 13677 \times 10.4 - 6834 \times (7.5 + 1.5) - 1 \times 9 \times 4.5$$
$$= 80695'^{\#}$$

Summory of dead and live load moments

$$= 51304 + 80695 = 131999'^{\#}$$

Shear due to dead load

Reaction from stringers $= 8135^{\#}$

Dead load of floor beam $= \frac{1}{2} \times 75 \times 20.8 = 780^{\#}$

Total dead load shear $= 8915^{\#}$

Shear due to live load $= 13677^{\#}$

Summary of dead and live load shears

$$= 8915 + 13677 = 22592^{\#}$$

Section modulus required $= \dfrac{131999 \times 12}{15490} = 102.3\ (in)^3$

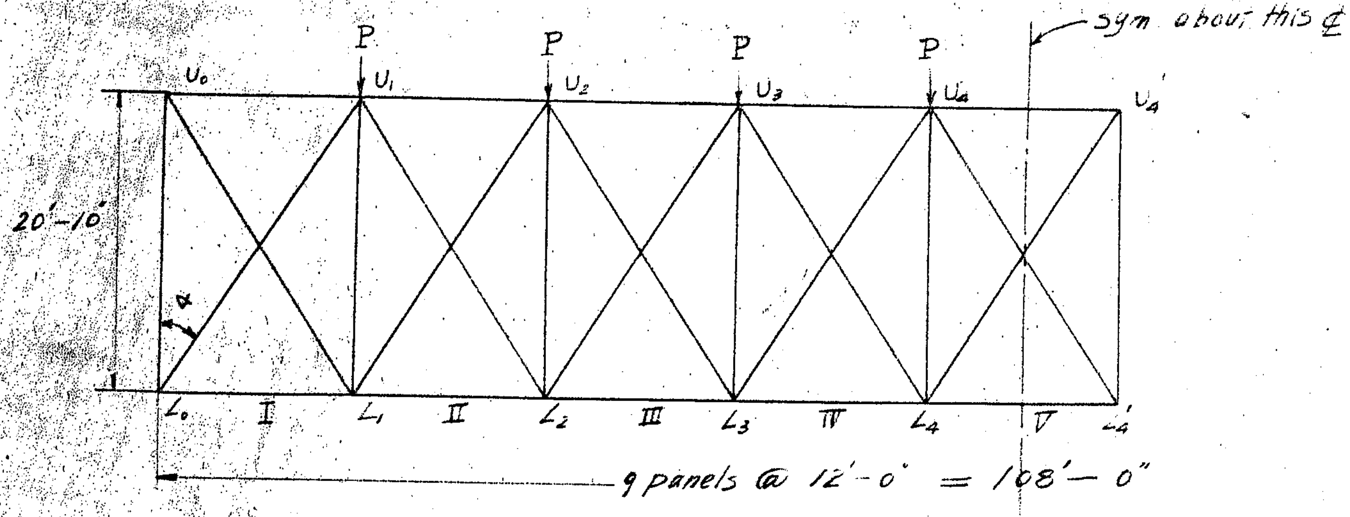
Section modulus of $20'' \times 65.4^{\#}$ I Beam $= 116.9\ (in)^3$

Allowable stress $= 1200\left(1 - 0.012 \times \frac{4 \times 12}{6.25}\right)$

$$= 1089\ \mathrm{kg/cm^2} = 15490\ \#/\square''$$

Actual stress $= \dfrac{131999 \times 12}{\cancel{126.3}\ 116.9} = \cancel{12500}\ 13550\ \#/\square''$

Lower laterals



Length of diagonals $= \sqrt{20{,}833^2 + 12^2} = 24.04$

$\sec\alpha = \dfrac{24.04}{20{,}83} = 1.154,\qquad \tan\alpha = \dfrac{12}{20{,}83} = 0.576$

Static wind load

loaded chord	270#/ft run
unloaded chord	135#/ft run
Total load	405#/ft run

panel concentration = 405 x 12 = 4860# = P

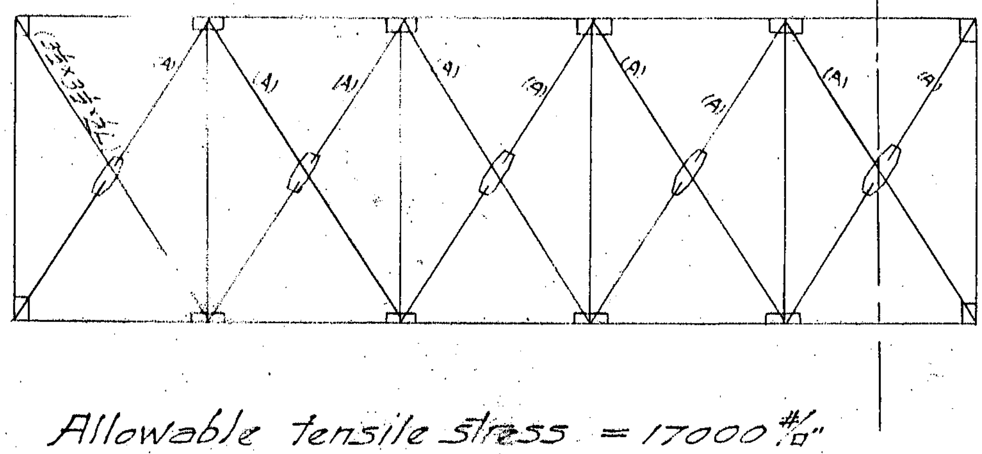
Panels	I	II	III	IV	V
Stresses in diagonals	L_0U_1 and U_0L_1 +4 P secα = +22434#	L_1U_2 and U_1L_2 + 3 P secα = 16825#	L_2U_3 and U_2L_3 + 2 P secα = +11217#	L_3U_4 and U_3L_4 + P secα = +5608#	L_4U_4' and U_4L_4' 0
Stresses in chords	L_0L_1 and L_1L_2 +4P tanα = +11197#	L_2L_3 +7P tanα = +19596#	L_3L_4 +9P tanα = +25194#	L_4L_4' +10P tanα = +27994#	

Stresses in chords are within 25 % of the sum of dead and live load stresses.

Section of lower lateral.

Use 1-L $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16} = 2.09$ gross area

$2.09 - \frac{5}{16} \times (\frac{7}{8} + \frac{1}{8}) = 1.7775$ net area

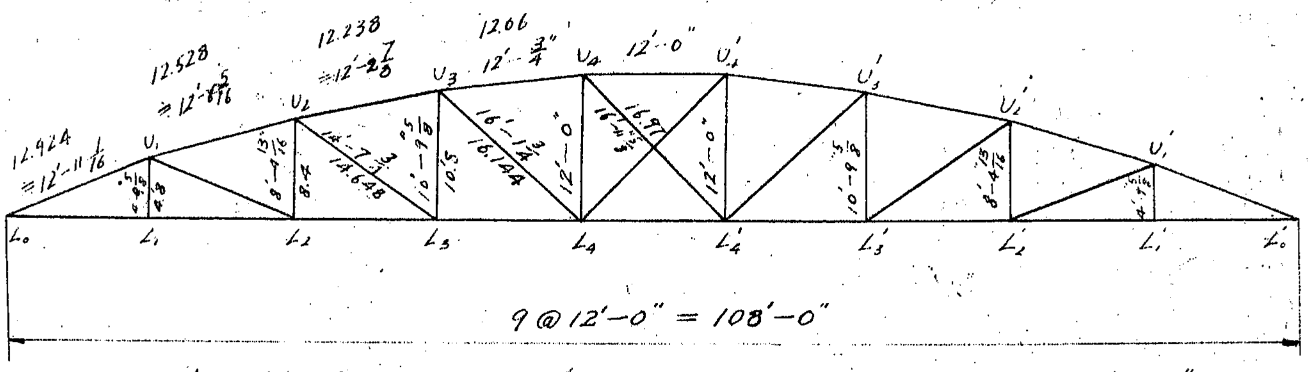


Allowable tensile stress $= 17000 \#/\square''$

Actual stress $= 22434 \div 1.7775 = 12620 \#/\square''$ Section I

The sections of panel II III IV and V are the same as in panel I.

Truss



Height of truss = 12' C to C of truss = 20'-10"

Dead load

Slab and pavement	= 99 x 18 =	1782 #/ft.
Concrete fillet	= 140 x 0.48 =	67 #/ft.
Curb	= 87.5 x 2 =	175 #/ft.
2 lines of handrails	2 @ 35# =	70 "
Stringers	35. x 5 =	175 "
Floor beams		148 "
Lateral bracing		30 "
Two trusses +	α	850 "
Total dead load		3297 #/ft.

Dead load per linear ft. per truss

= 3297 ÷ 2 = 1649# ≒ 1650# = W

Live load

Weight of crowd (w) = $\dfrac{100000}{170 + L}$ = 493 kg. per sq. m ≒ 100 #/□'

Live load per linear ft. per truss

= $100 \times \dfrac{18}{2}$ = 900#

Height of apex of parabola = 12ft.15 = f

Lower chord stress due to dead load

$$= \frac{WL^2}{8f} = \frac{1650 \times 108^2}{8 \times 12.15} = +198000\#$$

Do. due to live load = $\dfrac{900 \times 108^2}{8 \times 12.15}$ = +108000#

Total stress = 198000 + 108000 = +306000#

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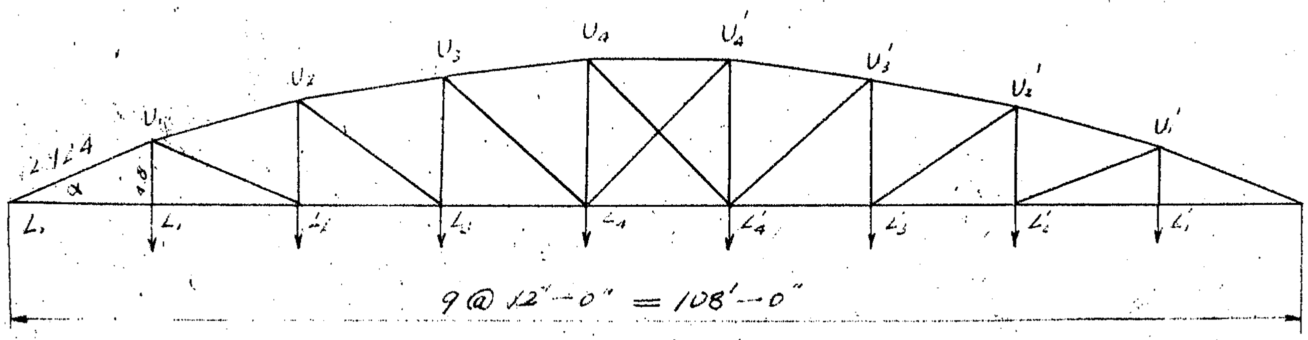
Loading for upper chord



Upper chord stress due to dead load

panel dead load $= 1650 \times 12 = 19800^{\#} = P_d$

Reaction $= 19800 \times 4 = 79200^{\#} = R$

Stress $L_0U_1 = R \operatorname{cosec}\alpha = 79200 \times \frac{12.924}{4.8} = -213246^{\#}$

Stress $U_1U_2 = \sqrt{(\text{stress } L_2L_3)^2 + (R-P_d)^2} = \sqrt{198000^2 + (79200-19800)^2}$

$\fallingdotseq -206700^{\#}$

Stress $U_2U_3 = \sqrt{198000^2 + (79200 - 2\times19800)^2} = -201900^{\#}$

Stress $U_3U_4 = \sqrt{198000^2 + (79200 - 3\times19800)^2} = -199000^{\#}$

Stress U_4U_4 $= -198000^{\#}$

Upper chord stress due to live load

panel live load $= 900 \times 12 = 10800^{\#} = P_l$

Reaction $= 10800 \times 4 = 43200^{\#} = R$

Stress $L_0U_1 = R \operatorname{cosec}\alpha = 43200 \times \frac{12.924}{4.8} = -116316^{\#}$

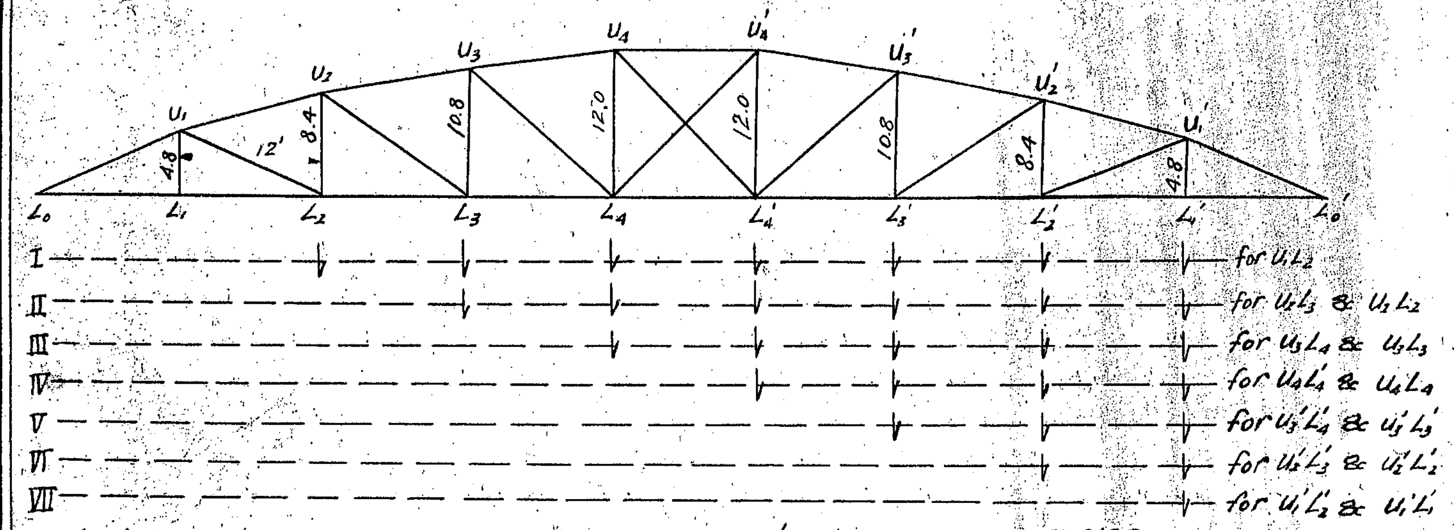
Stress $U_1U_2 = \sqrt{(\text{stress } L_2L_3)^2 + (R-P_l)^2} = \sqrt{108000^2 + (43200-10800)^2}$

$\fallingdotseq -112750^{\#}$

Stress $U_2U_3 = \sqrt{10800^2 + (43200 - 2\times10800)^2} \fallingdotseq -110140^{\#}$

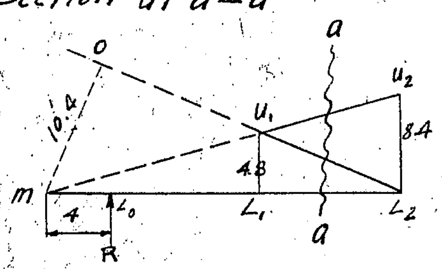
Stress $U_3U_4 = \sqrt{10800^2 + (43200 - 3\times10800)^2} \fallingdotseq -108540^{\#}$

Stress U_4U_4' $= -108000^{\#}$



+ = tension, - = compression

(I) Section at a–a



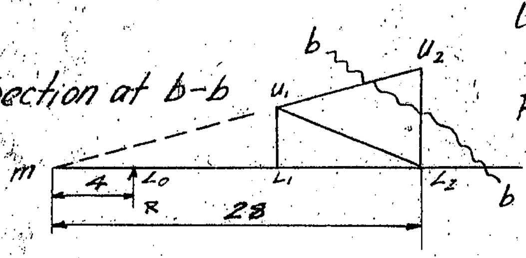
$$R = \frac{10800 \times 12\,(1+2+3+4+5+6+7)}{108}$$

$$= 33600^{\#}$$

$$\text{Moment at } m = 33600 \times 4 - U_1L_2 \times 10.4$$

$$U_1L_2 = \frac{33600 \times 4}{10.4} = +12923^{\#}$$

(II) Section at b-b

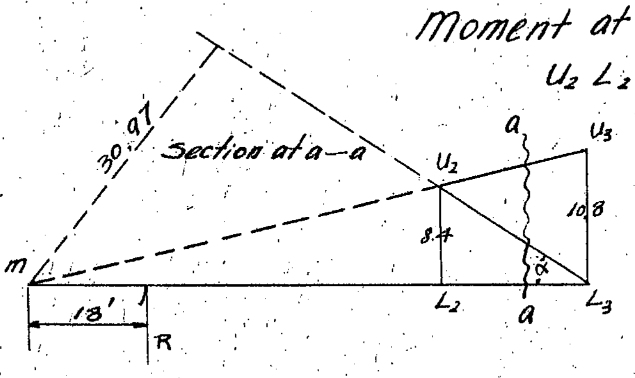


$$R = \frac{10800 \times 12\,(1+2+3+4+5+6)}{108}$$

$$= 25200^{\#}$$

$$\text{Moment at } m = 25200 \times 4 - U_2L_2 \times 28$$

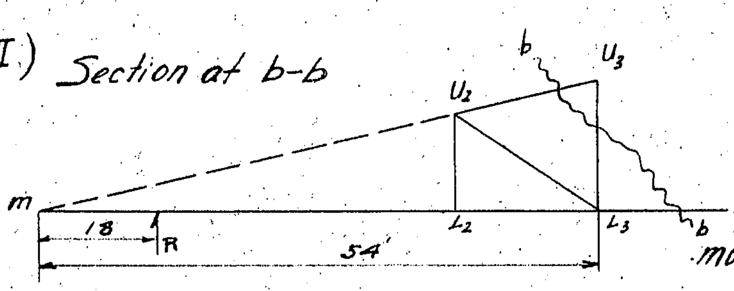
$$U_2L_2 = \frac{25200 \times 4}{28} = -3600^{\#}$$



$$\text{moment at } m = 25200 \times 18 - U_2L_3 \times 30.97$$

$$U_2L_3 = \frac{25200 \times 18}{30.97} = +14646^{\#}$$

(III) Section at b-b

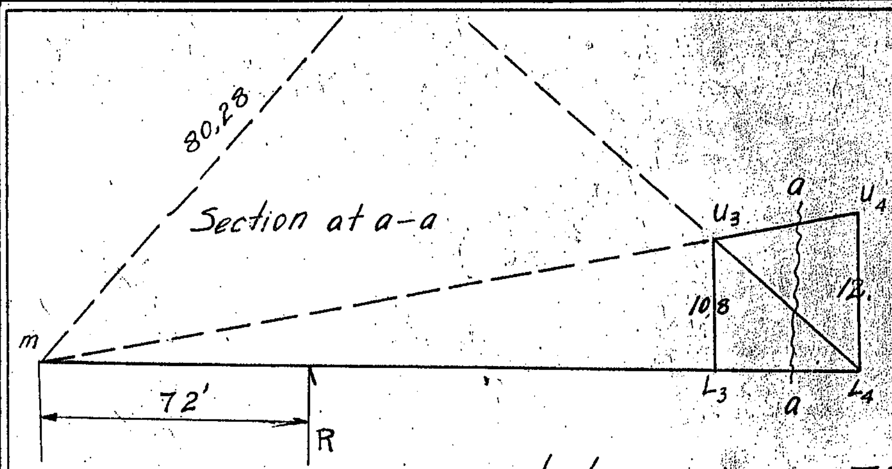


$$R = \frac{10800 \times 12\,(1+2+3+4+5)}{108}$$

$$= -18000^{\#}$$

$$\text{moment at } m = 18000 \times 18 - U_3L_3 \times 54$$

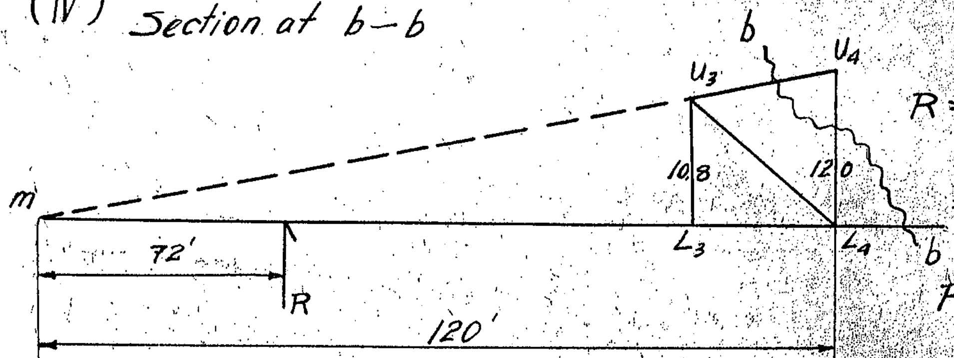
$$U_3L_3 = \frac{18000 \times 18}{54} = -6000^{\#}$$



$$\text{moment at } m = 18000 \times 72 - U_3L_4 \times 80{,}28$$

$$U_3L_4 = \frac{18000 \times 72}{80{,}28} = +16143^{\#}$$

(IV) Section at b−b



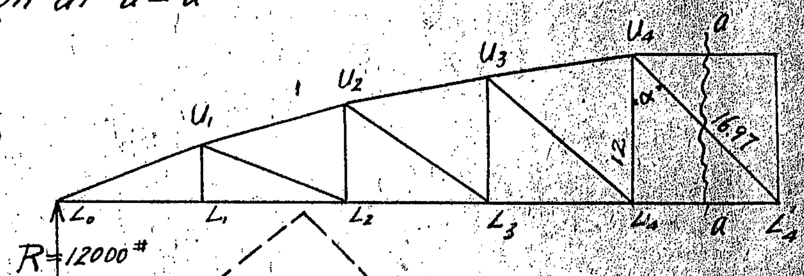
$$R = \frac{10800 \times 12(1+2+3+4)}{108} = 12000^{\#}$$

$$R' = 10800 \times 4 - 12000 = 31200^{\#}$$

$$\text{moment at } m = 12000 \times 72 - U_4L_4 \times 120$$

$$U_4L_4 = \frac{12000 \times 72}{120} = -7200^{\#}$$

Section at a−a

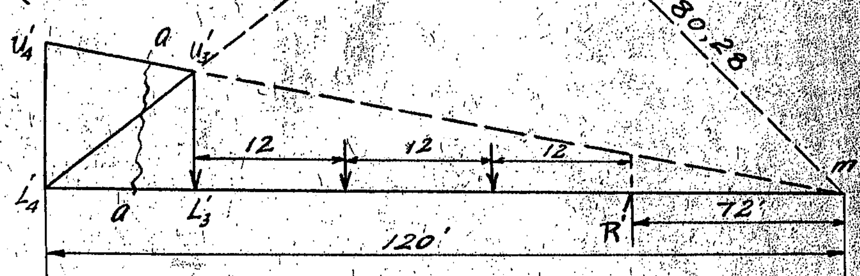


$$\Sigma V = 0$$

$$R - U_4L_4' \cos\alpha = 0$$

$$U_4L_4' = R \sec\alpha = 12000 \times \frac{16.97}{12} = +16970$$

(V)



$$R = \frac{10800 \times 12(1+2+3)}{108} = 7200^{\#}$$

$$R' = 10800 \times 3 - 7200 = 25200^{\#}$$

Section at a−a

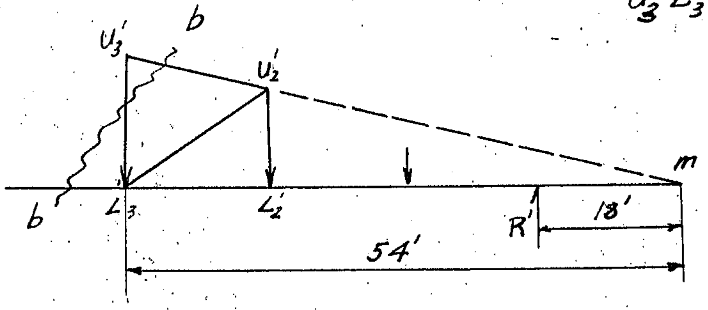
$$\text{moment at } m = 25200 \times 72 - 10800 \times (108+96+84) + U_3'L_4' \times 80{,}28 = 0$$

$$U_3'L_4' = \frac{-1814400 + 3110400}{80.28} = -16143$$

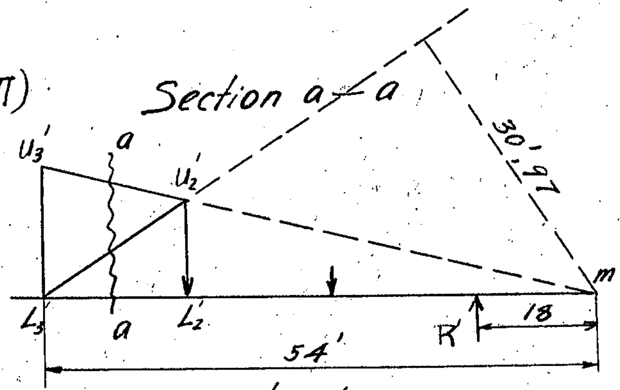
Section b-b

$$\text{Moment } m = 25200 \times 18 - 10800 \times (54+42+30) + U_3'L_3' \times 54$$

$$U_3'L_3' = \frac{-453600 + 1360800}{54} = +16800^{\#}$$



(VI) Section a-a

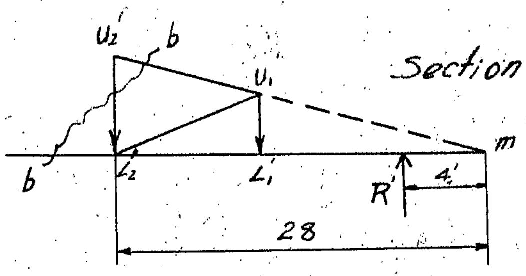


$$R = \frac{10800 \times 12(1+2)}{108} = 3600$$

$$R' = 10800 \times 2 - 3600 = 18000$$

$$\text{Moment at } m = 18000 \times 18 - 10800 \times (42+30) + U_2'L_3' \times 30.97$$

$$U_2'L_3' = \frac{-324000 + 777600}{30.97} = -14646^{\#}$$

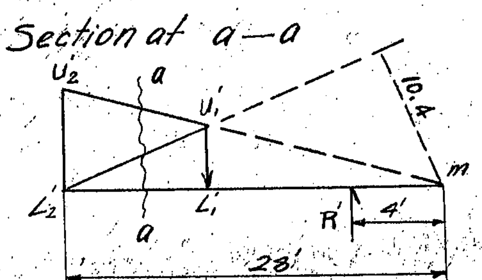


Section b-b

$$\text{moment at } m = 18000 \times 4 - 10800 \times (28+16) + U_2'L_2' \times 28$$

$$U_2'L_2' = \frac{-72000 + 475200}{28} = +14400^{\#}$$

(VII) Section at a-a



$$R = \frac{10800 \times 12}{108} = 1200^{\#}$$

$$R' = 10800 - 1200 = 9600^{\#}$$

$$\text{moment at } m = 9600 \times 4 - 10800 \times 16 + U_1'L_2' \times 10.4$$

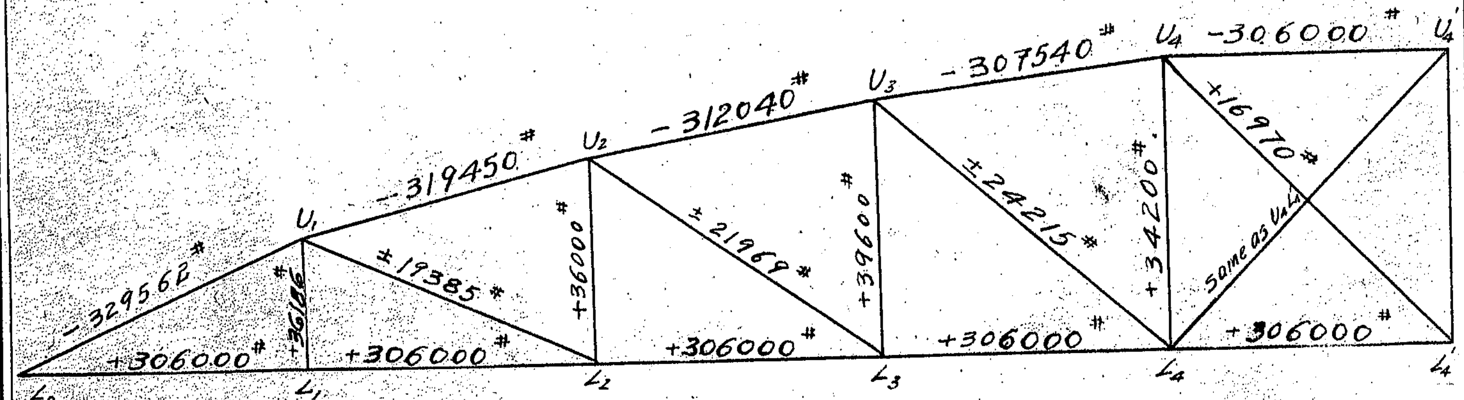
$$U_1'L_2' = \frac{-38400 + 172800}{10.4} = -12923^{\#}$$

chord members

members	Dead load stresses	Live load stresses	Total Stresses
$L_0 U_1$	−213246	−116316	−329562
$U_1 U_2$	−206700	−112750	−319450
$U_2 U_3$	−201900	−110140	−312040
$U_3 U_4$	−199000	−108540	−307540
$U_4 U_4'$	−198000	−108000	−306000
Lower chord	+198000	+108000	+306000

Web members

members	Dead load stresses	Live load stresses (full loading)	Live load stresses (partial loading)		Total Stresses
			main stresses	reverse stresses	
$U_1 L_1$	+19800#	+10800#	Reaction of Floor Beam = 16386#		+36186#
$U_2 L_2$	+19800#	+10800#	− 3600#	+14400#	$19800+(14400+3600\times\frac{50}{100})$ = +36000#
$U_3 L_3$	+19800#	+10800#	− 6000#	+16800#	$19800+(16800+6000\times\frac{50}{100})$ = + 39600#
$U_4 L_4$	+19800#	+10800#	− 7200#		$19800+(10800+7200\times\frac{50}{100})$ = + 34200#
$U_1 L_2$			+12923#	−12923#	$12923+(12923\times\frac{50}{100})$ = ± 19385
$U_2 L_3$			+14646#	−14646#	$14646+(14646\times\frac{50}{100})$ = ± 21969#
$U_3 L_4$			+16143#	−16143#	$16143+(16143\times\frac{50}{100})$ = ± 24215#
$U_4 L_4'$			+16970#		+16970#



Chord compression members,

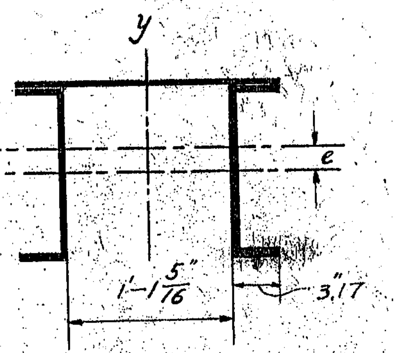
$L_0 U_1$

$S = -329562^{\#}$

Use 1-cov. pl. $20 \times \frac{7}{16} = 8.75^{\square''}$

2-[S 12" @ 30# $= 17.58$

$26.33^{\square''}$ gross area



$e = \frac{8.75 \times 6.22}{26.33} = 2.07''$

$I_x = 161.2 \times 2 + 8.75 \times \overline{6.22}^2$

$= 660.9^{(in)^4}$

$I_\xi = 660.9 - 26.33 \times \overline{2.07}^2$

$= 548.1^{(in)^4}$

$I_y = \frac{1}{12} \times \frac{7}{16} \times \overline{20}^3 + (5.2 + 8.79 \times \overline{7.34}^2) \times 2$

$= 1249.2^{(in)^4}$

$\rho_\xi = \sqrt{\frac{548.1}{26.33}} = 4.56$ $\qquad \rho_\xi < \rho_y \ \& \ \rho_x$

$l = 12.924' = 393.9^{cm.}$

$r = \rho_\xi = 4.56'' = 11.6^{cm}$

Allowable compressive stress

$= 1500\,(1 - 0.0055 \times \frac{l}{r}) \leqq 1000^{kg}/_{\square cm}$

$= 1500\,(1 - 0.0055 \times \frac{393.9}{11.6}) = 1220^{kg}/_{\square cm} > 1000^{kg}/_{\square cm}$

$\therefore$ Take $1000^{kg}/_{\square cm} = 14200^{\#}/_{\square''}$

Actual stress $= \frac{329562}{26.33} = 12517^{\#}/_{\square''}$

U_1U_2, U_2U_3, U_3U_4, and U_4U_4'

Use the same sections as U_0U_1

Chord tension members

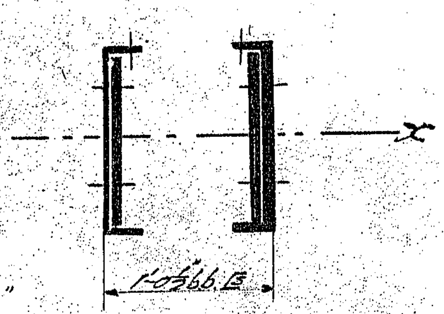
L_0L_1, L_1L_2, L_2L_3, L_3L_4, and L_4L_4'

$S = +306000^{\#}$

Requied net area $= \frac{306000}{17000} = 18.80''$

Use 2-⊏ 12" @ 30#

$= 17.58^{\#} - 8 \times \frac{1}{2} \times (\frac{7}{8} + \frac{1}{8}) = 13.58^{\square''}$



2-Pls. $9'' \times \frac{3}{8}''$

$= 6.75 - 4 \times \frac{3}{8} \times (\frac{7}{8} + \frac{1}{8}) = 5.25^{\square''}$

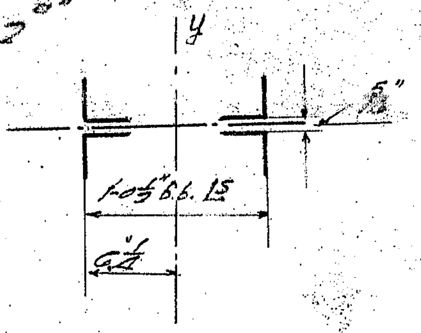
gr. area = 24.33　　net area $= 18.83^{\square''}$

Vertical members

U_4L_4

$S = +34200^{\#}$

Required net area $= \frac{34200}{17000} = 2.02^{\square''}$



Use 4Ls $4'' \times 3'' \times \frac{5}{16}''$

[illegible] $- 8 \times \frac{5}{16} \times (\frac{7}{8} + \frac{1}{8})$

$= 5.86^{\square''}$ net.

Actual direct stress

$= \frac{34200}{5.86} = 5836^{\#}/_{\square''}$

Upper panel concentration of wind load $= (135 \times 12)^{\#}$

Bending moment at L_4 due to windload

$= 135 \times 12 = 1940^{\#}$

$I_y = 1.7 \times 4 + (2 \times 2.09) \times (\frac{12.5}{2} - 0.76)^2 \times 2 = 258.77^{(in)^4}$

Actual stress due to bending

$= \frac{19440 \times 12 \times 6.25}{258.77} = 5634^{\#}/_{\square''}$

Total actual stress $= 5836 + 5634 = 11470^{\#}/_{\square''} < 17000^{\#}/_{\square''}$

U_3L_3, U_2L_2, and U_1L_1

Use the same sections as U_4L_4.

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Test U_3L_3

$S = 39600^{\#}$

Actual direct stress $= \frac{39600}{5.86} = 6758^{\#}$

Bending moment at L_3 due to Wind load.

$= 135 \times 12 \times 10.8 = 17496'^{\#}$

$I_y = 258.77^{(in)^4}$

Actual stress due to bending

$= \frac{17496 \times 12 \times 6.25}{258.77} = 5071 \,{}^{\#}/_{\square''}$

Total actual stress $= 6758 + 5071 = 11829 \,{}^{\#}/_{\square''} < 17000 \,{}^{\#}/_{\square''}$

U_2L_2 $S = 36000^{\#}$

Actual direct stress $= \frac{36000}{5.86} = 6143 \,{}^{\#}/_{\square''}$

Bending moment at L_2 due to Wind load

$= 135 \times 12 \times 8.4 = 13608^{\#}$

$I_y = 258.77^{(in)^4}$

Actual stress due to bending

$= \frac{13608 \times 12 \times 6.25}{258.77} = 3944 \,{}^{\#}/_{\square''}$

Total actual stress $= 6143 + 3944$

$= 10089 \,{}^{\#}/_{\square''} < 17000 \,{}^{\#}/_{\square''}$

U_1L_1

$S = 36186^{\#}$

Actual direcc stress $= \frac{36186}{5.86} = 6175 \,{}^{\#}/_{\square''}$

Bending moment at L_2 due to Wind load

$= 135 \times 12 \times 4.8 = 6576^{\#}$

$I_y = 258.77^{(in)^4}$

Actual stress due to bending

$= \frac{6576 \times 12 \times 6.25}{258.77} = 1906 \,{}^{\#}/_{\square''}$

Total actual stress $= 7832 + 2325$

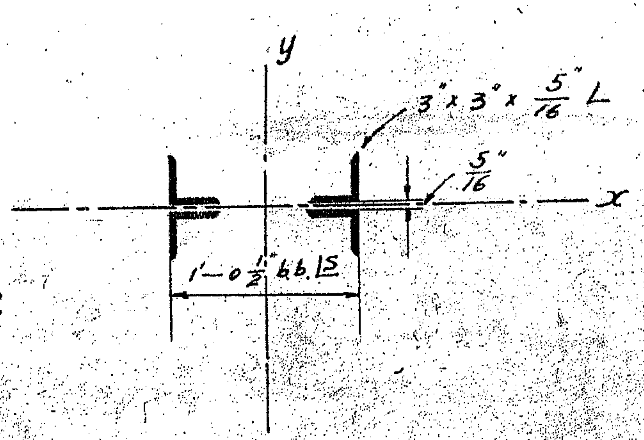
$= 10157 \,{}^{\#}/_{\square''} < 17000 \,{}^{\#}/_{\square''}$

Diagonal members.

U_1L_2

$S = \pm 19385$

Use $4\angle s\ 3'' \times 3'' \times \frac{5''}{16} = 7.12^{\square''}$



$I_x = 1.5 \times 4 + (2 \times 1.78) \times (87 + \frac{5}{16 \times 2})^2 \times 2$

$= 13,495\ (in)^4$

$\rho_x = \sqrt{\frac{13,495}{7.12}} = 1.38''$

$I_y = 212,084\ (in)^4$

$\rho_y = \sqrt{\frac{212,084}{7.12}} = 5.46$

$r = \rho_x = 1.38\,in = 3.43\,cm$ $\qquad l = 12,924 = 393.9\,c.m.$

Allowable compressive stress

$= 1500\ (1 - 0,0055 \times \frac{l}{r}) \geqq 1000\ {}^{Kg}/_{\square cm}$

$= 1500\ (1 - 0,0055 \times \frac{393.9}{3.43}) = 552.6\ {}^{Kg}/_{\square cm}$

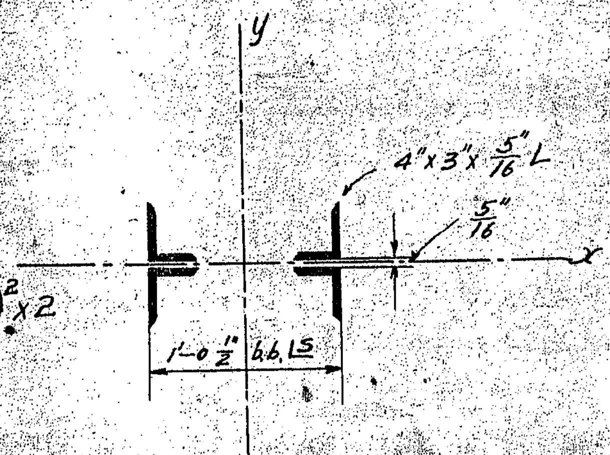
$= 7860\ {}^{\#}/_{\square''}$

Actual stress $= \frac{19385}{7.12} = 2723\ {}^{\#}/_{\square''}$

U_3L_4

$S = \pm 24215^{\#}$

Use $4\text{-}\angle s\ 4'' \times 3'' \times \frac{5''}{16} = 8.36^{\square''}$



$I_x = 3.4 \times 4 + (2 \times 2,09) \times (1.26 + \frac{5}{16 \times 2})^2 \times 2$

$= 30,36\ (in)^4$

$\rho_x = \sqrt{\frac{30,36}{8,36}} = 1.91\,in$

$I_y = 1.7 \times 4 + (2 \times 2.09) \times (\frac{12.5}{2} - 0.76)^2 \times 2 = 258.77\ (in)^4$

$\rho_y = \sqrt{\frac{258,77}{8,36}} = 5,57\,in$

$r = \rho_x = 1.91\,in = 4.85\,cm.$ $\qquad l = 16,144 = 492.1\,cm.$

Allowable compressive stress

$= 1500(1 - 0,0055 \times \frac{l}{r}) \geqq 1000\ {}^{Kg}/_{\square cm}$

$= 1500(1 - 0,0055 \times \frac{492.1}{4.85}) = 663\ {}^{Kg}/_{\square cm} = 9430\ {}^{\#}/_{\square''}$

Actual stress $= \dfrac{24215}{8{,}36} = 2896\ ^{\#}/_{\square''}$

U_2L_3

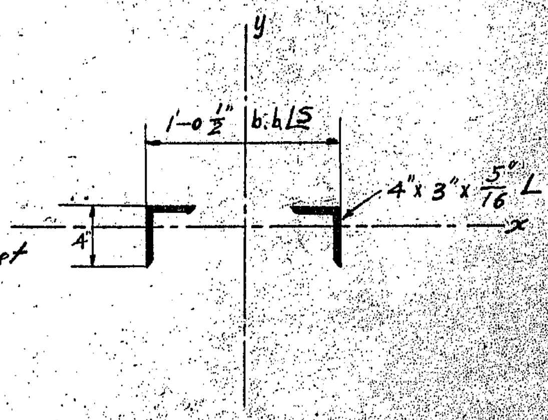
Use the same section as U_3L_4

U_4L_4', $U_4'L_4$

$S = +16970^{\#}$

Use 2-$\angle$s $4''\times3''\times\frac{5''}{16} = 4{,}18^{\square''}$ gross

$4{,}18 - 4\times\frac{5''}{16}\times\left(\frac{7''}{8}+\frac{1}{8}\right) = 2{,}93^{\square''}$ net



$I_x = 3.4\times2 = 6{,}8\ ^{(in)^4}$

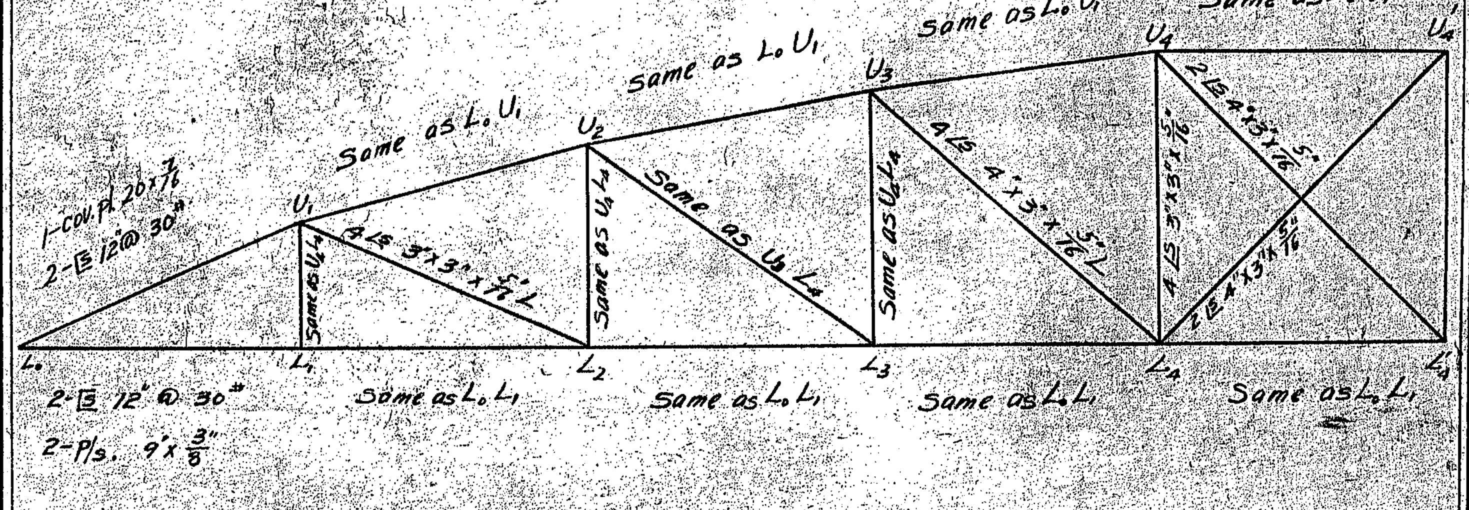
$\rho_x = \sqrt{\dfrac{6.8}{4.18}} = 1.27^{\,in.}$

$I_y = 1.7\times2 + 2{,}09\times\left(\dfrac{12{,}5}{2} - 0{,}76\right)^2\times2 = 129{,}39$

$\rho_y = \sqrt{\dfrac{129{,}39}{4.18}} = 5{,}57^{\,in.}$

Actual stress $= \dfrac{16970}{2{,}93} = 5792\ ^{\#}/_{\square''}$

Section sheet



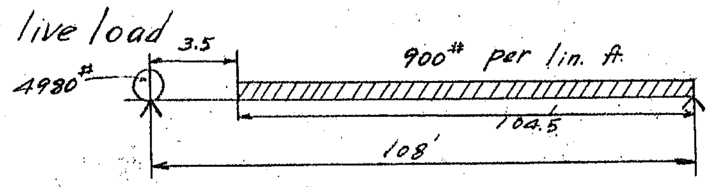
End Bearing

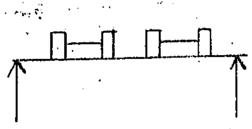
End Reaction.

for dead load $= 19800 \times 4.5$ + Expansion shoe.

$= 89100 + 1165 = 90265^{\#}$

for live load





impact coefficient $= \dfrac{20^m}{60 + l^m} = 22\%$

$4980 \times .22 = 1096^{\#}$

$(4980 + 1096) \times 2 = 12152^{\#}$

Reaction $= 12152 + 104.5 \times 900 \times 52.25 \times \dfrac{1}{108}$

$= 12152 + 45500 = 57652^{\#}$

End Reaction for dead and live load

$= 90265 + 57652 = 147917^{\#}$

Bearing area

Use $25'' \times 20''$ pl. $= 500^{\square''}$

Bearing stress $= 147917 \div 500 = 296^{\#}/_{\square''}$

Roller

Use $4''\phi$ rollers

Allowed stress $= 640 \times 4 = 2560^{\#}/\text{in.}$

Length $= 147917 \div 2560 = 58''$

4 - rollers $58 \div 4 = 14.5''$ net.

Use 4-rollers $4''\phi$ $1'\text{-}11\frac{1}{4}''$ shoulder to shoulder.

Pin

Assuming the thickness of pin plate $= \frac{5}{8}''$

Bearing thickness $= \frac{5}{8} + \frac{1}{2}$(gusset pl.) $= 1\frac{1}{8}''$

$\dfrac{147917}{2} = 73959^{\#} \doteqdot 25600 \times d \times 1\frac{1}{8}''$

$d = \dfrac{73959}{25600 \times 1.125} = 2.6''$

Use $4\frac{1}{2}''\phi$ pin $= 15.904^{\square''}$ gr.

Resisting mt. $= (4.5)^3 \times 0.098175 \times 25600 = 229000''^{\#}$

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Max moment arm $= 229000 \div 73959 = 3.''1$

Resisting shear $= 12800 \times 15.904 = 203571^{\#}$

No. of rivets $= 73959^{\#} \div 7220 = 11$

Use shop rivets 13

field rivets 6 20 % less in field rivets

Length of pin inside space $= 11''\frac{1}{16}$

pin plates $\frac{5}{8} \times 2 = 1\frac{1}{4}''$

gusset plates $\frac{1}{2} \times 2 = 1''$

space $= \frac{3}{8}''$

Web of shoe $\frac{1}{2} \times 2 + \frac{5}{8} \times 2 = 2\frac{1}{4}''$

add to grip $= \frac{1}{2}''$

$1' - 4''\frac{7}{16}$

Anchor bolt

Use $1\frac{1}{4}''\phi$

1'-6" set in masonry

4 per one shoe

Bolt Holes $= 1\frac{1}{2}''\phi$

ŌYA BRIDGE

Pier

Design of substructures.

Piers

Weight of superstructure

$$\text{metals} = \begin{cases} \text{2 lines of hand rails } 2 @ 35^{\#} = 70^{lbs} \text{ per lin. ft.} \\ \text{stringers} \quad 175 \quad " \\ \text{Floor beams} \quad 148 \quad " \\ \text{Lower laterals} + 2 \text{ trusses} + \alpha = 880 \quad " \\ \text{2 shoes} \quad 2 @ 35^{\#} = \underline{70} \quad " \end{cases}$$

1343 lbs per lin. ft.

Weight of road way

Road way $99^{\#} @ 18' = 1782^{\#}$ per lin. ft.

concrete fillet $67^{\#}$

curb, $87.5 \times 2 = \underline{175}^{\#}$

$2024^{\#}$ "

Sum of dead load $= (1343 + 2024) \times 108 = 363636^{\#}$

live load per lin. ft. $= 100 \times 18 = 1800^{\#}$ p lin. ft.

Total live load $= 1800 \times 108 = 194400^{\#}$

Total dead and live load $= 363636 + 194400 = \underline{558036}^{\#}$

Determinating the datum height of the top of pier.

Crown	3"
pavment	2"
Slab	6"
Floor beam	20"
Total height	31" $= 2'.583 = 2.598^{R}$

Thickness of gusset plate at bottom of floor beam $= \frac{5}{16}''$

Thickness of rivet head $= \frac{39}{64}''$

$$\frac{5}{16} + \frac{39}{64} = \frac{20+39}{64} = \frac{59''}{64} = \frac{59 \times 1.0058219}{64 \times 12} = 0.077 \text{ shaku}$$

Height of Crown. = 1582 shaku

Lowest point of floor beam = 1582 − 2,598 = 1579,402

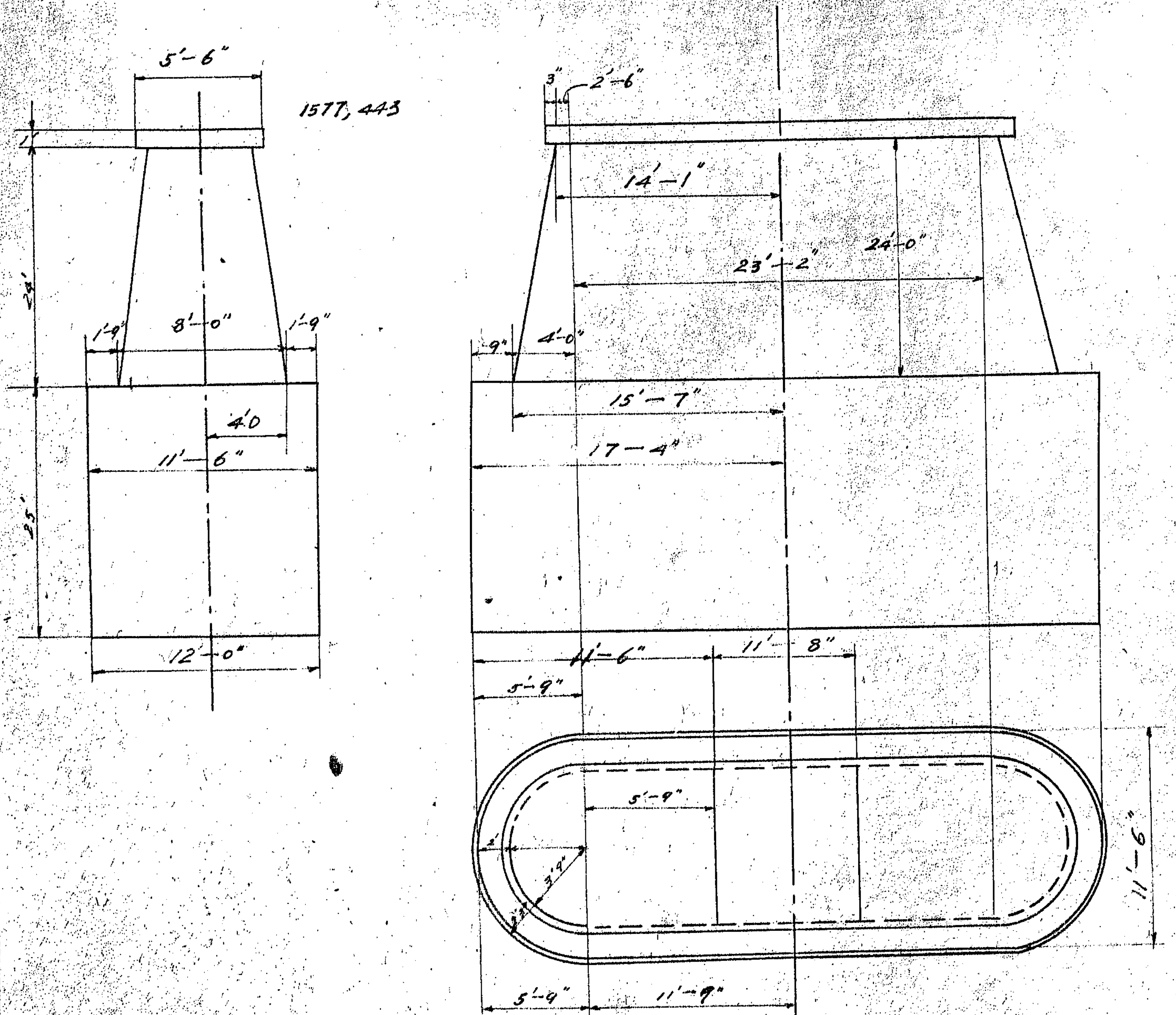
Lowest point of trusses = 1579,402 − 0,077 = 1579,325

Height of H.W.L. = 1570 shaku

Height between under side of floor beam and top of pier
= $1'-11\frac{3}{8}''$ = 1.9479 = 1.959

Datum height of the top of pier
1579,402 − 1.959 = 1577,443 shaku

Dimension of pier



Thickness of coping 1.0

Height of Shaft $\underline{24.0}$

$25.0 = 25.146^{shaku}$

Depth of well $25' = 25.146^{shaku}$

1. Coping $(5.5 \times 23'2'' + \pi\,\overline{2.75}^2) \times 1.0$

$= (5.5 \times 23.167 + 3.1416 \times \overline{2.75}^2) \times 1.0 = (127.4185 + 23.7584) \times 1 = 151^{cub. ft.}$

Weight $= 151. \times 140 = \underline{21140}^{\#}$

2. Shaft.

Top area $5 \times 23.167 + \pi \times \overline{2.5}^2 = (115.835 + 19.635) = 135.5^{sq. ft.}$

Bottom area $8 \times 23.167 + \pi\,4^2 = (185.336 + 50.266) = 235.6^{sq. ft.}$

$\therefore$ Volume $= 24 \times 23.167 \times \left(\frac{5+8}{2}\right) + \frac{24}{3} \times \{19.64 + 50.27 + \sqrt{19.64 \times 50.27} = \underline{4425}^{cub. ft.}$

Weight $= 4425 \times 140 = \underline{619500}^{\#}$

3. Well foundation

Area $23.167 \times 11.5 + \pi \times \overline{5.75}^2 = 370.3^{\square'}$

Height $= 25'$

Volume $= 370.3 \times 25 = 9258.^{cub. ft.}$

Weight $= 9258 \times 140 = \underline{1296120}^{\#}$

Total weight of pier $= 21140 + 619500 + 1296120^{\#}$

$= 1936760^{\#}$

Total dead and live load of superstructure $= 558036^{\#}$

Total weight $= 1936760 + 558036 = 2494796$

Unit compressive stress at the base of foundation

$= \frac{2494796}{370.3} = 6736^{\#}/_{\square'} = 3^{ton}/_{\square'}$

Horizontal action

Wind pressure

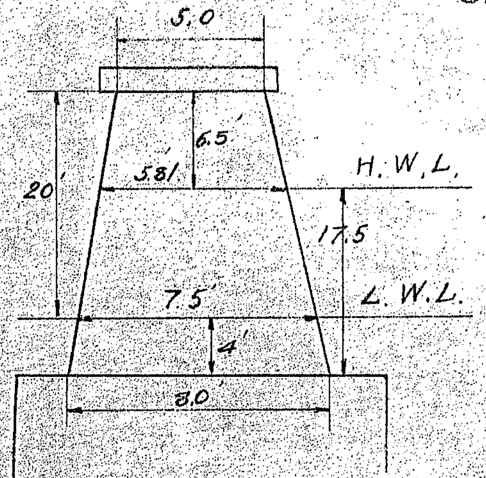
on bridge body assuming the static wind load = 405 p.l.ft.

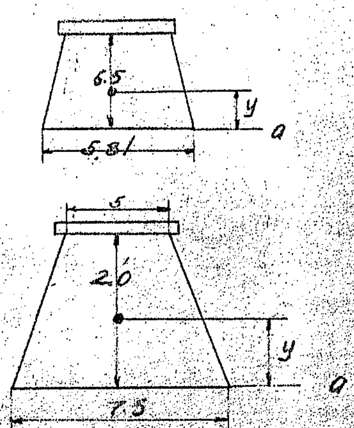
Total wind load = 405 × 108 = 43740 #

on shaft body

Expesed area = 5.5×1 + $\frac{1}{2}$(5 + 7.5)×20 = 131 □' when L.W.L

5.5×1 + $\frac{1}{2}$(5 + 5.81)×6.5 = 41 □' when H.W.L.





y = 3.7

y = 9.8

Wind pressure = 50 × 131 = 6550 # when L.W.L

50 × 41 = 2050 # when H.W.L.

Assuming the center of gravity of static wind pressure of trusses is at about 3 ft above the lower chord center

Height between lower chord center and the top of pier = 2.'4479 ≒ 2.'5

There-fore the center of gravity of super-out-posed load is at about 5.5 ft above the top of pier.

Bending mt. at the bottom of shaft

43740 × (25 + 5.5) = 1334070 '#

6550 × (4 + 9.8) = 90390 '# when L.W.L

2050 × (17.5 + 3.7) = 43460 '# 〃 H.W.L.

Summary

1424460 when L.W.L

1377530 〃 H.W.L.

Current pressure

Formula $p = K.W. \frac{v^2}{2g}$

where p = the pressure in pounds per sq. ft. of Vertical projection.

K = constant.

v = max. velocity of current in ft. per second.

W = the weight of a cub ft. of water.

g = gravity acceleration = 32,2 ft. per sec. per sec.

and Greiner gives us 0,75 for the value of $\frac{K.W}{2g}$

Assuming $v = 18$ ft./sec.

Then we have $p = 0{,}75 \times \overline{18}^2 = 243$ #/□'

Wetted area $= \frac{1}{2} \times (5{,}81 + 8{,}0) \times 17{,}5 = 120{,}8$ □'

Total current pressure $= 120{,}8 \times 243$

$= 29354{,}$ # when H. W. L.

Bending moment at the bottom of shaft

$29354 \times 17{,}5 \times 0{,}6 = 308217$ '#

Summary of moment due to wind and current pressure

1377530	for wind pressure
308217	" current "
1685747	summary

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(独立行政法人 土木研究所蔵)

Maximum stress at the bottom of shaft

Weight of superstructure (live load included)

= 558036 #

" " coping = 21140 #

" " shaft = 619500 #

summary = 1198676

$\therefore$ Unit compressive stress $= \dfrac{1198676}{235.6} = 5088$ #/□'

$= 35$ #/□"

When live load excluded

Weight of superstructure = 363636

" " coping = 21140

" " shaft = 619500

summary = 1004276

$\therefore$ Unit compressive stress $= \dfrac{1004276}{235.6} = 4263$ #/□'

$= 30$ #/□"

Bending stress :—

$$f = \frac{M.x_0}{I}$$

$$I = \frac{8 \times 23.17^3}{12} + \left\{0.007 \times 8^4 + 0.393 \times 8^2 \times (11.584 + 0.212 \times 8)^2\right\} \times 2$$

$$= 17222.(ft)^4$$

$x_0 = 15'-7" = 15.583'$

$$f = \frac{1685747 \times 15.583}{17222} = 1525 \text{ #/□'}$$

Combined stress = 5088 #/□' + 1525 #/□'

= 6613 #/□'

46 #/□"

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Destructive effect of earth quake

I = Moment of inertia of the area of fracture about its middle axis.

W = Weight of pier about the section of fracture + Weight of superstructure.

h = Height of the center of gravity of the whole structure above the section of fracture

x_0 = Half width of the section of fracture in direction parallel to earth quake motion

F = Tensile strength of concrete work (7200#/□' allowed) + compression of structure at the section of fracture

α = Seismic stability or the acceleration of earth quake motion capable of fracturing the structure

g = Gravity acceleration

M = Resisting moment of the section

Then we have

$$F = \frac{M}{\frac{I}{x_0}}, \qquad M = F_1 \frac{I}{x_0} = \frac{W}{g} \cdot \alpha \cdot h$$

$$\text{or} \quad \alpha = \frac{g.F.I}{x_0 W.h}$$

$x_0 = 4'$

$F = 7200^{\#/□'} + 5088^{\#/□'} = 12288^{\#/□'}$

$F = 7200^{\#/□'} + 4263^{\#/□'} = 11463^{\#/□'}$

$I = \frac{23.167 \times 8^3}{12} + \frac{\pi \times 8^4}{64} = 1190^{(ft)^4}$

$W = 1198676^{\#}$ (live load included)

$W = 1004276^{\#}$ (live load excluded)

h = Center of gravity from the bottom of shaft.

Superstructure	$W = 558036$	$d = 30'.5$
Coping	" $= 21140$	" $= 24.5$
Shaft	" $= 619500$	" $= 10'.8$
	1198676	

Then we have (when live load included)

$$h = \frac{558036 \times 30.5 + 21140 \times 24'.5 + 619500 \times 10.8}{1198676}$$

$$= 20'.2$$

$$\therefore \alpha = \frac{g.F.I}{x_0 W.h} = \frac{9800 \times 12288 \times 1190}{4 \times 1198676 \times 20.2} = 1480 \ ^{mm}/_{sec^2}$$

When live load excluded

$$h = \frac{363636 \times 30.5 + 21140 \times 24'.5 + 619500 \times 10.8}{1004276}$$

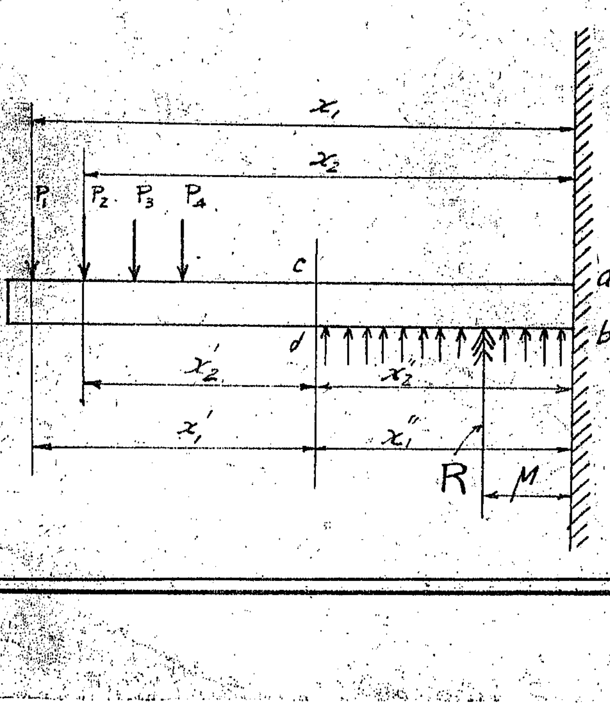
$$= 19.04$$

$$\therefore \alpha = \frac{g.F.I}{x_0 W.h} = \frac{9800 \times 11463 \times 1190}{4 \times 1004276 \times 19.04} = \underline{\underline{1748}} \ ^{mm}/_{sec^2}$$

Bending moment due to horizontal pressue at the section of well.

Bending moment due to horizontal pressure will be increased for longer arm, but the frictional resistance between well and surrounding earth will be grown larger for the depth increased. This amount will be different for the earth nature

However the assumption in this case is very rough We would treat this problem as follows.



$$M. \text{ at } a\text{-}b = \Sigma Px - R.M$$

$$= \Sigma P(x' + x'') - R.M$$

$$= \Sigma Px' + \Sigma Px'' - R.M$$

Assuming $\Sigma Px'' = R.M$

$$M. \text{ at } a\text{-}b = \Sigma Px' = M. \text{ at } c\text{-}d$$

We assume that the c–d section is at the point 6^{feet} below the top of well and every surface above this section is effected by scouring action

When H.W.L.

$M_1 \equiv$ Moment at the bottom of well due to current pressure.

Wetted area $= 120\,{}^{\square'}\!,8 + 11.5 \times 6 = 189\,{}^{\square'}\!,8$

Current pressure $= 243 \times 189{,}8 = 46121^{\#}$

$M_1 = 46121 \times (17{,}5 + 6) \times 0{,}6 = 650306^{'\#}$

$M_2 \equiv$ Moment at the bottom of well due to wind pressure

$43740 \times (25 + 5{,}5 + 6) = 1596510^{'\#}$

$2050 \times (3.7 + 17{,}5 + 6) = \underline{55760}^{'\#}$

$M_2 = 1652270^{'\#}$

$M_1 + M_2 = 650306 + 1652270 = \underline{2302576}^{'\#}$

$$I = \frac{11{,}5 \times \overline{23{,}17}^3}{12} + \left\{0{,}007 \times \overline{11.5}^4 + 0{,}393 \times \overline{11{,}5}^2 \times (11{,}584 + 0{,}212 \times 11.5)^2\right\} \times 2$$

$$= 32602^{(ft)^4}$$

$x_0 = 17'\text{-}4'' = 17{,}33$

Bending stress $= \dfrac{M x_0}{I} = \dfrac{2302576 \times 17{,}33}{32602} = 1224\ {}^{\#}/_{\square'}$

Total comp. stress at the bottom of foundation

$= 6736 + 1224 = 7960\ {}^{\#}/_{\square'} \doteqdot 3{,}6\ {}^{ton}/_{\square'}$

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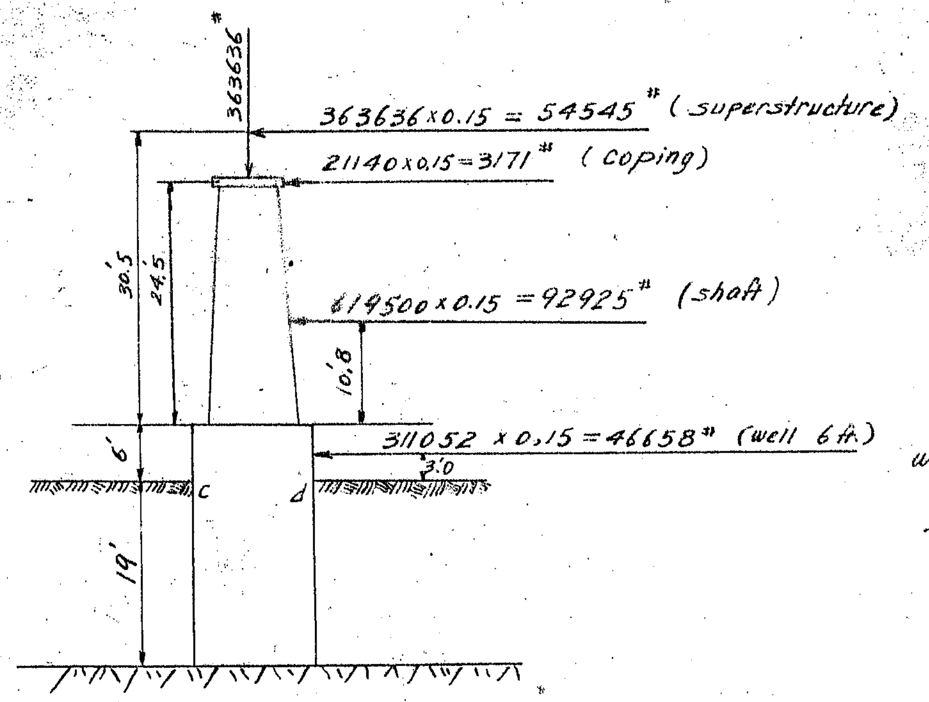
Bending moment due to Earth quake motion.



weight of well (6 ft.)

370.3 x 6 x 140 = 311052

$$M_{CD} = 54545 \times 36.5 + 3171 \times 30.5 + 92925 \times 16.8 + 46658 \times 3$$
$$= 1990893 + 96716 + 1561140 + 139974$$
$$= 3788723$$

$$I = \frac{23.17 \times \overline{11.5}^3}{12} + \frac{\pi \times \overline{11.5}^4}{64} = 2937 + 859 = 3796 \ (\text{ft.})^4$$

$$x_0 = 5.'75$$

$$\therefore \quad \text{stress} = \frac{Mx_0}{I} = \frac{3788723 \times 5.'75}{3796} = 5739 \ \#/\square'$$

Max. comp. stress at the bottom of foundation

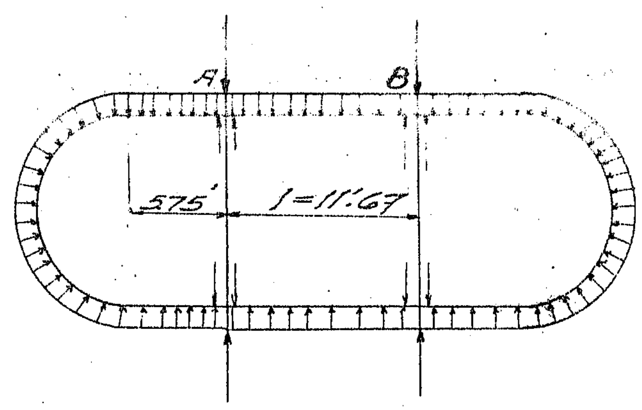
$$= 6736 + 5739 = 12475 \ \#/\square'$$
$$\fallingdotseq 5.6 \ \text{ton}/\square'$$

Limit of Foundation pressure of compact gravel.

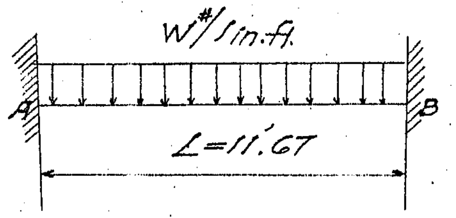
$$(Q) = 8 \ \text{ton}/\square'$$

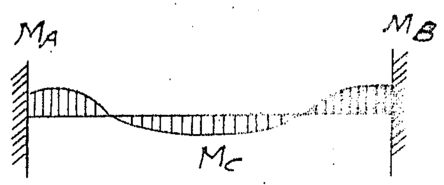
For Earth quake $Q = 8 \times \left(1 + \frac{60}{100}\right) = 12.8 \ \text{ton}/\square'$

$\therefore$ safe



Max bending moment and Max shear occurs at the Part A-B therefore we calculate at this part and use the same section all other part of the Well.



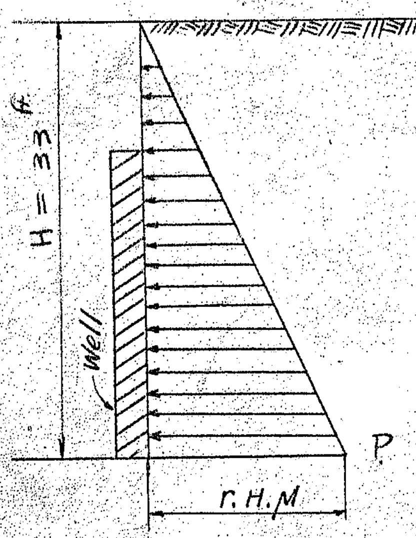


$$M_A = M_B = -\frac{WL^2}{12} = \frac{-W\times\overline{11.67}^2}{12} = -11.35\,W$$

$$M_c = \frac{WL^2}{24} = \frac{W\times\overline{11.67}^2}{24} = 5.68\,W$$

$$\text{Shear} = \frac{Wl}{2}$$

Pressure intensity



$$P = r.H.M \quad , \quad r = 120^{\#}/\text{cub. ft.} \qquad \phi = 30°$$

$$M = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1}{3}$$

$$r.M = 120 \times \frac{1}{3} = 40$$

$$H = 33^{ft.}$$

$$P = W = 1320^{\#}$$

Max. negative moment $= -11.35\,W = -11.35 \times 1320$

$= -14982'^{\#}$

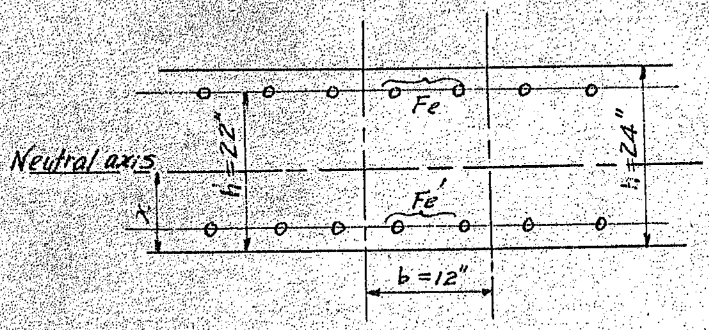
$= -179784''^{\#}$

Max positive moment $= 5.68W = 5.68 \times 1320$

$= 7497.6'^{\#}$

$= 89971''^{\#}$

Max. shearing force $= \dfrac{Wl}{2} = \dfrac{1320 \times 11.67}{2} = 7702^{\#}$



Section of well.

Tension side $\frac{5''}{8}\phi$ bar 6" spacing

$Fe = 2 - \frac{5''}{8}\phi$ bar

$= 2 \times 0.3068 = 0.6136$

Compression side $\frac{1''}{2}\phi$ bar 6" spacing

$Fe' = 2 - \frac{1}{2}\phi$ bar

$= 2 \times 0.1963 = 0.3926$

$$x = -\frac{n(Fe+Fe')}{b} + \sqrt{\left\{\frac{n(Fe+Fe')}{b}\right\}^2 + \frac{2n}{b}\left(hFe + h'F\right)}$$

$$= -\frac{15 \times 1.0062}{12} + \sqrt{\left\{\frac{15 \times 1.0062}{12}\right\} + \frac{2 \times 15}{12} \times \left(22 \times 0.6136 + 2 \times 0.3926\right)}$$

$$= \underline{4''.85}$$

$$J = \frac{bx^3}{3} + nF_e'(x-h')^2 + nF_e(h-x)^2$$

$$= \frac{12 \times 4.85^3}{3} + 15 \times 0.3926 \times (4.85-2)^2 + 15 \times 0.6136 \times (22-4.85)^2$$

$$= 456.3 + 47.8 + 2707.1$$

$$= \underline{3211}.$$

$$S = nF_e(h-x) = 15 \times 0.6136 \times (22-4.85)$$

$$= \underline{158}^{(in)^3}$$

For negative moment

$$\sigma_b = \frac{Mx}{J} = \frac{179784 \times 4.85}{3211} = \underline{272}\ ^{\#/\square''}$$

$$\sigma_e' = n\sigma_b \frac{x-h'}{x} = 15 \times 272 \times \frac{4.85-2}{4.85} = 2398\ ^{\#/\square''}$$

$$\sigma_e = n\sigma_b \frac{h-x}{x} = 15 \times 272 \times \frac{22-4.85}{4.85} = 14427\ ^{\#/\square''}$$

For shear

$$Q = 11283^{\#}$$

$$\tau_o = \frac{QS}{bJ} = \frac{\cancel{7702} \times 158}{12 \times 3211} = 32\ ^{\#/\square''} < 57\ ^{\#/\square''}$$

$\therefore$ safe

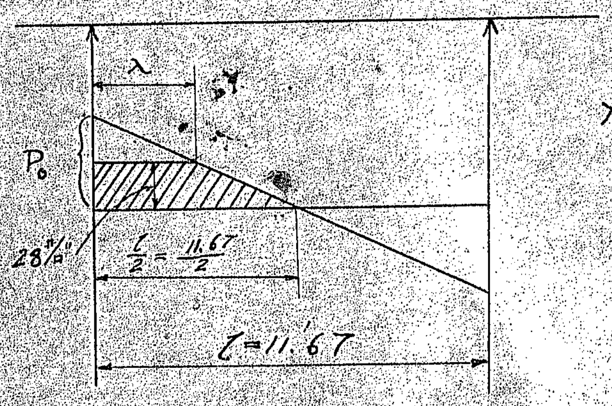
$$\tau_h = \frac{QS}{JU}$$

$$U = 1.9635 \times 2 = 3.927$$

$$\therefore \tau_h = \frac{\cancel{7702} \times 158}{3211 \times 3.927} = 96\ ^{\#/\square''} > 85\ ^{\#/\square''}$$

The value of τ_{max} corresponding to bond stress $85\ ^{\#/\square''}$

$$\therefore \tau_{max} = \frac{P_h U}{b} = \frac{85 \times 3.927}{12} = 28\ ^{\#/\square''}$$



$$\lambda = \frac{l(\tau_o - 28) \times 12}{2P_o}$$

$$= \frac{11.67(32-28) \times 12}{2 \times 46}$$

$$= \frac{11.67 \times 4 \times 12}{2 \times 46} = 6.1''$$

Required area of bent up bars (A)

$$= \frac{(\mathcal{Q}_0 - 28)\lambda \times b}{2\sqrt{2} \times \sigma}$$

where σ = allowed tensile stress of reinforcing bar #/□"

$$\therefore (A) = \frac{(46-28) \times 2.7 \times 12}{2 \times 1{,}4142 \times 17000} = 0{,}03\ □"$$

Bent up $\frac{5''}{8}\phi$ bars

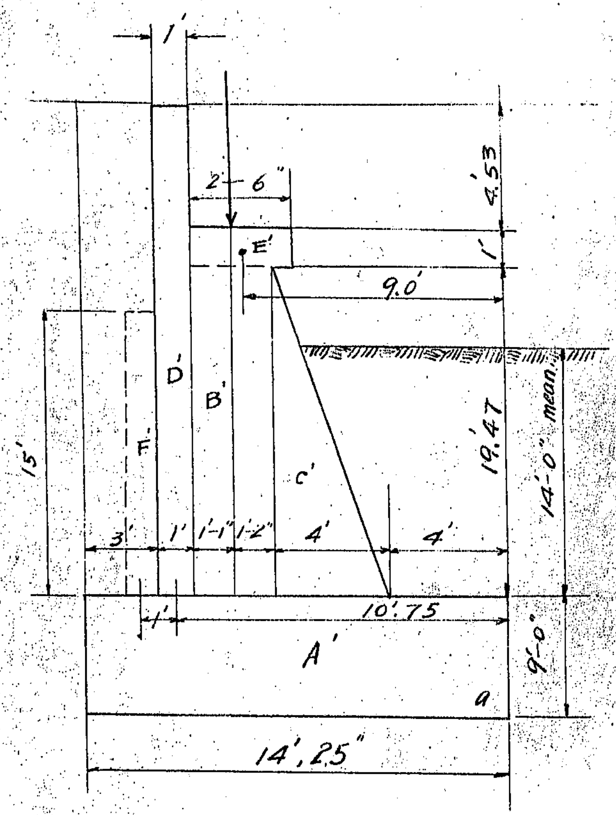
[illegible]

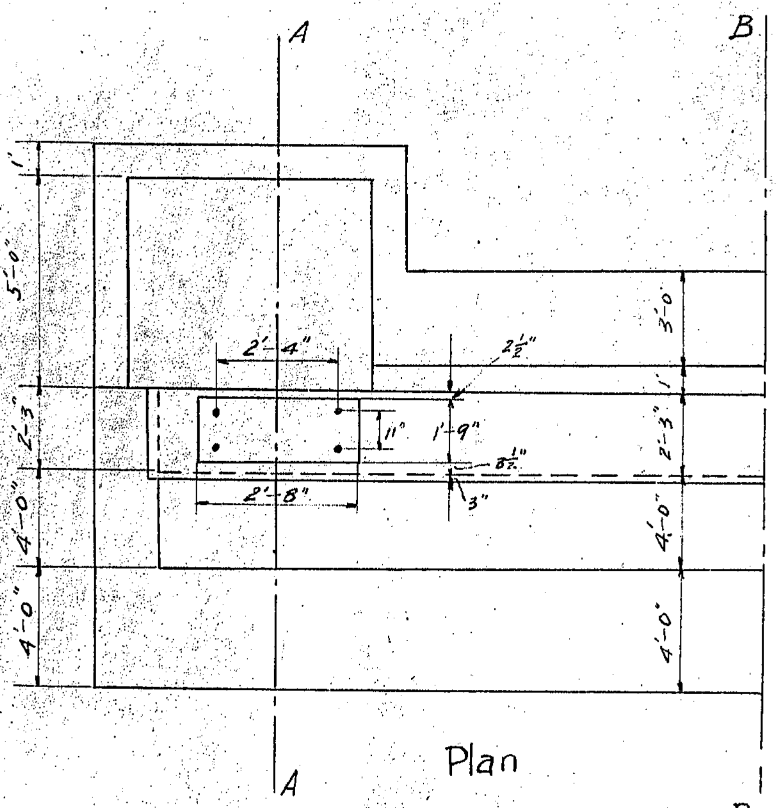
ŌYABASHI

Abutment of Right bank

(I) Abutment of Right bank

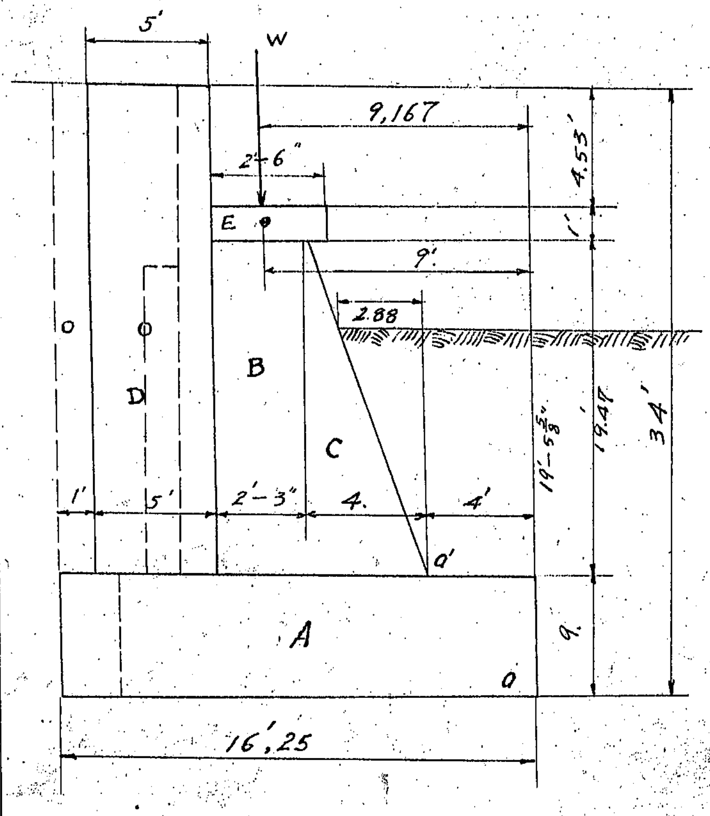
a) Dimension of abutment.

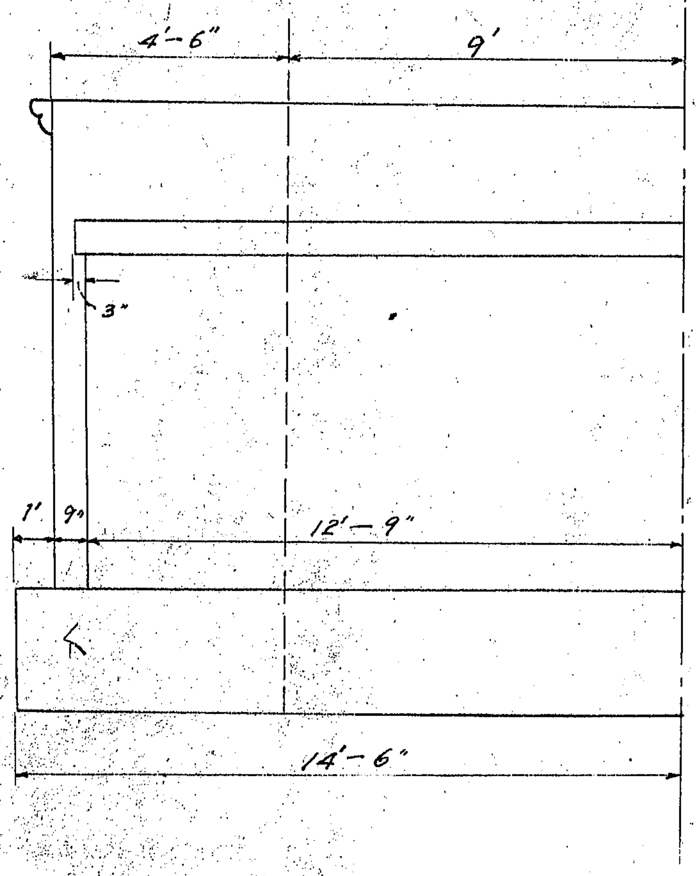




Plan

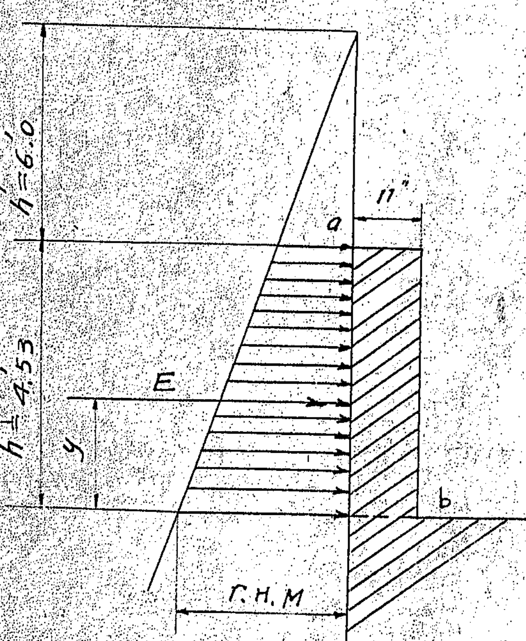
Section B—B





Section A—A

1. Parapet wall



$\gamma = 100^{\#}/\text{cub. ft.}$ $\qquad \mu = \frac{1}{3}$

Assumed surcharge for live load
= 6 feet of earth

pressue intensity at a $= \frac{1}{3} \times 6 \times 100$
$= 200\ ^{\#}/\square'$

" " at b $= \frac{1}{3} \times 10{,}53 \times 100$
$= 351\ ^{\#}/\square'$

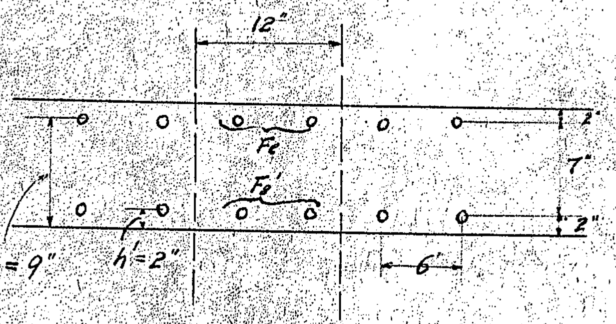
Resultant of earth pressue

$E = \frac{1}{2} \times (200 + 351) \times 4{,}53'$

$= 1248^{\#}$

$$y = \frac{h}{3} \times \frac{h + 3h'}{h + 2h'} = \frac{4.53}{3} \times \frac{4.53 + 3 \times 6}{4.53 + 2 \times 6} = 2{,}06$$

$$M.\text{at } b = 1248 \times 2{,}06 \times 12 = 30851''^{\#}$$



sectional area of bar

$Fe' = Fe = 2 - \frac{1}{2}"\phi$ bar

$= 0{,}1963 \times 2 = 0{,}3926^{\square''}$

$$x = -\frac{n(Fe + Fe')}{b} + \sqrt{\left\{\frac{n(Fe + Fe')}{b}\right\}^2 + \frac{2n}{b}\left(hFe + h'Fe'\right)}$$

$$= -\frac{15 \times 0{,}7852}{12} + \sqrt{\left\{\frac{15 \times 0{,}7852}{12}\right\}^2 + \frac{2 \times 15}{12}\left(9 \times 0{,}7852 + 2 \times 0{,}7852\right)}$$

$$= 3.8''$$

$$J = \frac{bx^3}{3} + nFe'(x - h')^2 + nFe(h - x)^2$$

$$= \frac{12 \times 3.8^3}{3} + 15 \times 0{,}3926 \times (3.8 - 2)^2 + 15 \times 0{,}3926 \times (9 - 3.8)^2$$

$$= 397.8\ (in)^4$$

$$\sigma_b = \frac{Mx}{J} = \frac{30851 \times 3.8}{397.8} = 295\ ^{\#}/\square''$$

$$\sigma_e' = \frac{n\sigma_b(x-h')}{x} = \frac{15\times 295/\square'' \times(3.8-2)}{3.8} = 2096\ ^{\#}/\square''$$

$$\sigma_e = \frac{n\sigma_b(h-x)}{x} = \frac{15\times 295\times(9-3.8)}{3.8} = 6055\ ^{\#}/\square''$$

2. Vertical loads for section A–A

Weight of concrete

$A = 16.25\times 9\times 140 = 20475\ ^{\#}/ft$

$B = 19.47\times 2.25\times 140 = 6133\ ^{\#}/ft$

$C = \frac{1}{2}\times 19.47\times 4\times 140 = 5452\ ^{\#}/ft$

$D = 5\times 25\times 140 = 17500\ ^{\#}/ft$

$E = 1\times 2.5\times 140 = 350\ ^{\#}/ft$

Weight of superstructure

metals =	2 lines of handrails	2 @ 35# =	70	lbs per lin. ft
	stringers		175	"
	Floor beams		148	"
	Lower laterals + ~~2-masses~~ 2-trusses + α	=	880	"
	2 shoes	2 @ 35# =	70	"
			1343	lbs per lin. ft.

Weight of road way

Road way	99# @ 18′ =	1782#	per lin. ft.
Concrete fillet		67#	
Curb.	87.5 × 2 =	175#	
		2024#	"

Sum of dead load = (1343 + 2024) × 108 = 363636#

Live load per lin. ft. = 100 × 18 = 1800# per lin. ft.

Total live load = 1800 × 108 = 194400#

Total dead and live load = 363636 + 194400 = 558036#

W = Weight of superstructure per unit length on abutment

= 558036 ÷ 25.5 × 2 = 10942#

Weight of earth on footing

$$\text{Rear side} = 1 \times 25 \times 100 = 2500^{\#}$$

$$\text{Front side} = \begin{cases} 14 \times 2.88 \times \frac{1}{2} \times 100 = 2016^{\#} \\ 14 \times 3 \times 100 = 5600^{\#} \end{cases}$$

Neglected the effect of live load surcharge for high wall

$$\text{Total vertical load per lin. ft.} = 20475 + 6133 + 5452$$
$$+ 17500 + 350 + 10942$$
$$+ 2500 + 2016 + 5600 = 70968$$

Center of gravity

$$M. \text{ at } a = 20475 \times \frac{16.25}{2} + 6133 \times 9.125 + 5452 \times 6.666$$
$$+ 17500 \times 12.75, + 350 \times 9.0$$
$$+ 10942 \times 9.167$$
$$+ 2500 \times 15.75 + 2016 \times 4.96 + 5600 \times 2$$
$$= 166359 + 55964 + 36343 + 223125$$
$$+ 3150 + 100305 + 39375 + 9999 + 11200$$
$$= \underline{645820}'^{\#}$$

Hor. dist. between the center of gravity and a.

$$= 645820 \div 70968 = 9.10$$

$$\text{Eccentricity} = 9.1 - \frac{16.25}{2} = 0.98$$

When live load excluded

$$194400 \div (25.5 \times 2) = 3812^{\#} \text{ per lin. ft. on abutment}$$

$$\therefore P = 70968 - 3812 = 67156^{\#}$$

$$\text{live load mt. at } a = 3812 \times 9.167 = 34945'^{\#}$$

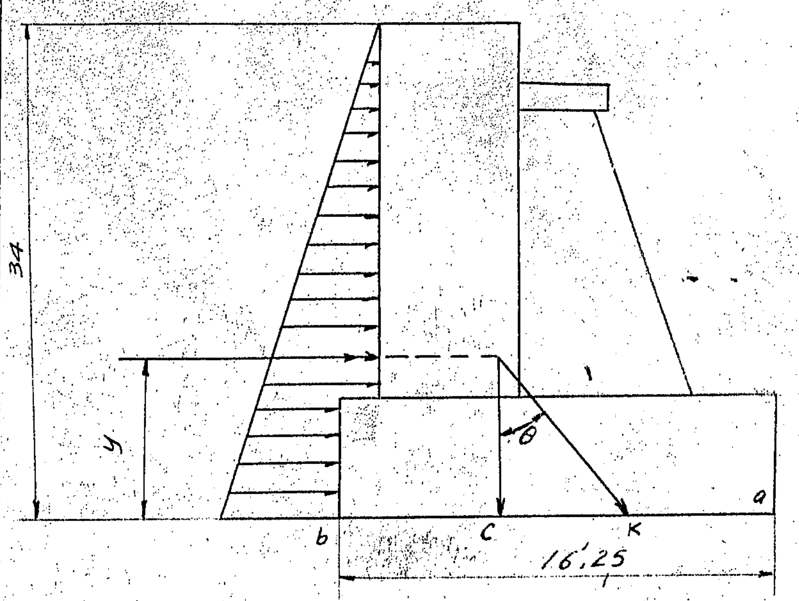
$$M. \text{ at } a = 645820 - 34945 = 610875'^{\#}$$

Hor. dist. between the center of gravity and a

$$610875 \div 67156 = 9.10$$

$$\text{Eccentricity} = 9.10 - \frac{16.25}{2} = 0.975 \doteqdot 0.98$$

Earth pressure



$$E = \frac{1}{2} r h^2 M$$

$$= \frac{100 \times \overline{34}^2}{2} \times \frac{1}{3}$$

$$= 19267^{\#}$$

$$y = \frac{34}{3} = 11.3$$

$$\tan\theta = \frac{19267}{70968} = 0.271 < 0.577$$

When live load excluded $\tan\theta = \frac{19267}{67156} = 0.287 < 0.577$

∴ safe for sliding

For overturning

$Kc = y \tan\theta = 11.3 \times 0.271 = 3.06$

or $Kc =$ " $=$ " $\times 0.287 = 3.24$

∴ $Ka = 9.10 - 3.06 = 6.04$

or $Ka = 9.10 - 3.24 = 5.86$

One 3rd of width of base $= \frac{16.25}{3} = 5.42$

∴ safe for overturning.

For bearing

Eccentricity of resultant.

$\frac{1}{2} \times 16.25 - 6.04 = 2.09$ or $\frac{1}{2} \times 16.25 - 5.86 = 2.27$

$\sigma = \frac{N}{A}\left(1 \pm \frac{6.e}{ab}\right)$

$= \frac{70968}{16.25}\left(1 \pm \frac{6 \times 2.09}{16.25}\right)$

$\sigma_1 = 4367 \times 1.772 = 7738^{\#}/_{\square'} = 3.5^{ton}/_{\square'}$

$\sigma_2 = 4367 \times 0.228 = 996^{\#}/_{\square'}$

$\sigma = \frac{N}{A}\left(1 \pm \frac{6.e}{ab}\right)$

$= \frac{67156}{16.25 \times 1}\left(1 \pm \frac{6 \times 2.27}{16.25}\right)$

$\sigma_1' = 4133 \times 1.838 = 7596^{\#}/_{\square'} \doteqdot 3.4^{ton}/_{\square'}$

$\sigma_2' = 4133 \times 0.162 = 670^{\#}/_{\square'}$

Vertical loads for section B-B

Weight of concrete

$A' = 14.25 \times 9 \times 140 = 17955$ #

$B' = 2.25 \times 19.47 \times 140 = 6133$ "

$C' = 4.00 \times 19.47 \times \frac{1}{2} \times 140 = 5452$ "

$E' = 1 \times 2.5 \times 140 = 350$ "

$D' = 1 \times 25 \times 140 = 3500$ "

$T' = 1 \times 15 \times (140-100) = 600$

Weight of earth on footing

Rear side $= 3 \times 25 \times 100 = 7500$ #

Front side $= \begin{cases} 14 \times 2.88 \times \frac{1}{2} \times 100 = 2016 \text{ "} \\ 14 \times 4 \times 100 = 5600 \text{ "} \end{cases}$

Weight of superstructure $= 10942$ "

Total load per lin. ft. $= 60048$ # $= P'$

Center of gravity

M. at a $= 17955 \times \frac{14.25}{2} + 6133 \times 9.125 + 5452 \times 6.666$
$+ 350 \times 9.00 + 3500 \times 10.75 + 7500 \times 12.75 + 600 \times 11.75$
$+ 2016 \times 4.96 + 5600 \times 2 + 10942 \times 9.167$
$= 127929 + 5594 + 36343 + 3150 + 37625$
$+ 95625 + 9999 + 11200 + 100305$
$= 478140$ '# $+ 7050 = 485190$

Hor. distance between the center of gravity and a

$485190 \div 60048 = 8.08$

Eccentricity $= 8.08 - \frac{1}{2} \times 14.25 = 0.955$

When live load excluded

$P = 60048 - 3812$ # $= 56236$

M. at a $= 485190 - 34945 = 450245$ '#

Hor. dist. between the center of gravity and a

$450245 \div 56236 = 8.01$

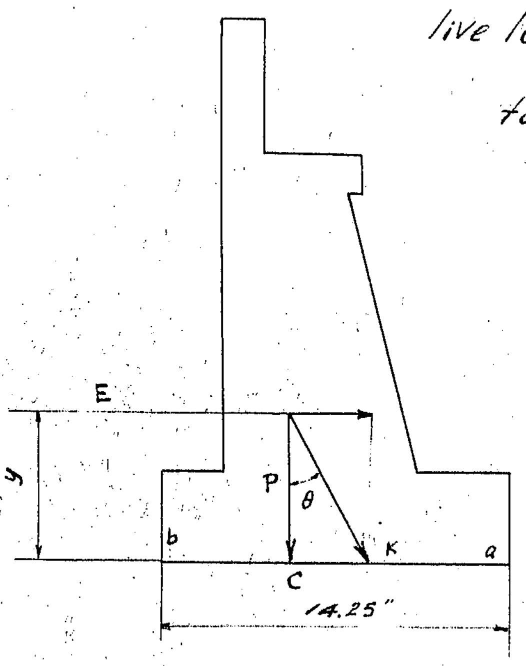
Eccentricity $= 8.01 - \frac{1}{2} \times 14.25 = 0.89$

$E = 19267$ #

live load included $\quad \tan\theta = \dfrac{19267}{60048} = 0.321 < 0.577$

live load excluded

$$\tan\theta = \frac{19267}{56236} = 0.343 < 0.577$$



$\therefore$ safe for sliding

$$y = 11.3$$

one third of width of base = 4.75

one 4th of width of base = 3.56

$$Kc = y \tan\theta = 11.3 \times 0.321 = 3.63$$

or $\quad Kc = \quad " \quad = 11.3 \times 0.343 = 3.88$

$$Ka = 8.08 - 3.63 = 4.45$$

or $\quad Ka = 8.01 - 3.88 = 4.13$

Eccentricity of resultant $= 7.125 - 4.45 = 2.68$

or $\quad$ " $\quad$ " $\quad$ " $\quad = 7.125 - 4.13 = 3.00$

$$\sigma = \frac{2V}{3c} = \frac{2 \times 60048}{3 \times 4.45} = 8996 \text{ \#/}_{\square'} \doteqdot 4.02 \text{ ton/}_{\square'}$$

$$\text{or } \sigma = \frac{2V}{3c} = \frac{2 \times 56236}{3 \times 4.13} = 9078 \text{ \#/}_{\square'} \doteqdot 4.05 \text{ ton/}_{\square'}$$

$F_1 = 100' \times 15 \times 140 = 2100$

$B' = 2.25 \times 19.47 \times 140 = 6133$

$C' = 4.00 \times 19.47 \times \frac{1}{2} \times 140 = 5452$

$D' = 1.00 \times 25 \times 140 = 3500$

$E' = 1.00 \times 2.5 \times 140 = 350$

$14 \times 2.88 \times \frac{1}{2} \times 100 = 2016$

weight of superstructure $= 10942$

$1 \times 10 \times 100 = 1000$

Total load $= \underline{\underline{31493}}$

M. at a' $= 2100 \times 7.75 + 6133 \times 5.125 + 5452 \times 2.666 + 3500 \times 6.75$
$+ 350 \times 5 + 2016 \times 0.96 + 10942 \times 5.167$
$+ 1000 \times 7.75$

$= 16275 + 31432 + 14535 + 23625 + 1750 + 1935 + 56537$
$+ 7750$

$= 153839$

Hor. distance between the center of gravity and a'

$153839 \div 31493 = 4.88$

Eccentricity $= 4.88 - \frac{8.25}{2} = 0.755$

When live load excluded

$P = 31493 - 3812 = 27681$

live load mt. at a'

$= 3812 \times 5.167 = 19697'^{\#}$

M. at a' $= 153839 - 19697 = 134142'^{\#}$

Hor. dist. between the center of gravity and a'

$134142 \div 27681 = 4.85$

Eccentricity $= 4.85 - \frac{8.25}{2} = 0.725$

Earth pressure $= \frac{1}{2} r h^2 M$

$= \frac{100 \times 25^2}{2} \times \frac{1}{3} = 10417^{\#}$

$y = \frac{25}{3} = 8.33$

live load included $\tan\theta = \frac{10417}{31493} = 0.331$

live load excluded $\tan\theta = \frac{10417}{27681} = 0.376$

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One third of width of base $= \frac{8.25}{3} = 2.75$

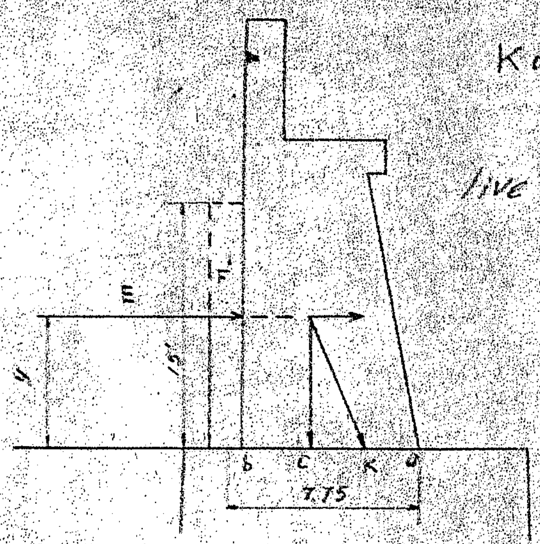
one fourth of width of base $= \frac{8.25}{4} \fallingdotseq 2.06$

one fifth of width of base $= \frac{8.25}{5} = 1.65$

live load included

$Kc = y \tan\theta = 8.33 \times 0.331 = 2.76$

$Ka = 4.88 - 3.06 = 1.82$



live load excluded

$Kc = y \tan\theta = 8.33 \times 0.376 = 3.13$

$Ka = 4.85 - 3.13 = 1.72$

Eccentricity of resultant $= \frac{8.25}{2} - 1.82 = 2.31$

or " " " $= \frac{8.25}{2} - 1.72 = 2.41$

live load inculuded

$\sigma = \frac{N}{A}\left(1 \pm \frac{6.e}{a.b}\right)$

$= \frac{31493}{8.25 \times 1}\left(1 \pm \frac{6 \times 2.31}{8.25}\right)$

$\sigma_b = 3817 \times 2.68 = 10230$
$= 4.6$ ton/□'

$\sigma_t = 3817 \times 0.68 = 3196$ #/□'

live load excluded

$\sigma = \frac{N}{A}\left(1 \pm \frac{6 \times 2.41}{8.25}\right)$

$= \frac{27681}{8.25 \times 1}\left(1 \pm \frac{6 \times 2.41}{8.25}\right)$

$\sigma_b = 3355 \times 2.75 = 9226$ #/□' $= 4.1$ ton/□'

$\sigma_t = 3355 \times 0.75 = 2516$ #/□'

Stability against Earth quake motion

a) Vertical loads for Section B ~ B

Weight of concrete

$A' = 14.25 \times 9 \times 140 = 17955^{\#}$

$B' = 2.25 \times 19.47 \times 140 = 6133''$

$C' = 4 \times 19.47 \times \frac{1}{2} \times 140 = 5452''$

$E' = 1 \times 2.5 \times 140 = 350''$

$D = 1 \times 25 \times 140 = 3500''$

$F' = 1 \times 15 \times 140 = 2100$

summation $= 35490$

Weight of earth on footing

Rear side $\begin{cases} 1 \times 10 \times 100 = 1000^{\#} \\ 2 \times 25 \times 100 = 5000 \end{cases}$

Front side $\begin{cases} 14 \times 2.88 \times \frac{1}{2} \times 100 = 2016 \\ 14 \times 4 \times 100 = 5600 \end{cases}$

summation $= 13616$

Weight of superstructure live load excluded per lin. ft.

$= 363636 \div (25.5 \times 2) = 7130^{\#}$

Total Weight of earth and concrete per lin. ft. of substructure

$= 35490 + 13616 = 49106^{\#}$

Earth quake motion $= 49106 \times 0.15 = 7366^{\#}$

Earth quake motion of superstructure live load excluded per lin. ft.

$= 7130 \times 0.15 \doteqdot \underline{1070}^{\#}$

Earth pressure in case of Earth quakes

$P = Wh \dfrac{1 - \sin\varphi_1}{1 + \sin\varphi_1}$ $\varphi = 30°$ $K = 0.15$

$\varphi_1 = \varphi - \tan K = 30° - 8° 32' = 21° 28'$

$\therefore P = Wh \dfrac{1 - \sin(21° 28')}{1 + \sin(21° 28')} = Wh \times 0.464$

$E = \dfrac{Wh^2}{2} \times 0.464 = \dfrac{100 \times 34^2 \times 0.464}{2} = 26819^{\#}$

$26819 \times \dfrac{3}{4} = \underline{20114}^{\#}$

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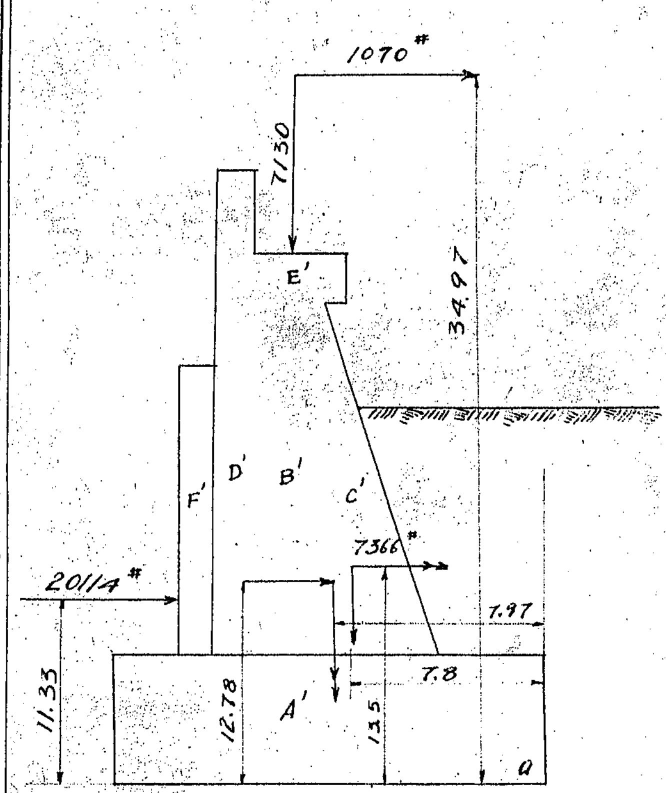

Total Vertical load
= 49106

Center of gravity

M. at a = 17955 × $\frac{14.25}{2}$ + 6133 × 9.125 + 5452 × 6.666
+ 350 × 9 + 3500 × 10.75 + 2100 × 11.75
+ 1000 × 11.75 + 5000 × 13.25 + 2016 × 4.96 + 5600 × 2

= 127929 + 55964 + 36343 + 3150 + 37625
+ 24675 + 11750 + 66250 + 9999 + 11200
= 384885'#

Hor. dist. between the center of gravity and a
= 384885 ÷ 49106 = 7.8'

M. at a = 17955 × 4.5 + 6133 × 18.74 + 5452 × 15.49 + 350 × 28.97
+ 3500 × 21.5 + 2100 × 16.5 + 1000 × 29 + 5000 × 21.5
+ 2016 × 18.33 + 5600 × 16
= 80798 + 114932 + 84451 + 10140 + 75250
+ 34650 + 29000 + 107500 + 36953 + 89600
= 663274

Ver. distance from a to C. G.
= 663274 ÷ 49106 = 13.5'

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Total Vertical load = 49106 + 7130 = 56236 #

Total horizontal load = 7366 + 1070 + 20114 = 28550 #

Mt. at a

= 49106 × 7.8 + 7130 × 9.167 = 448388

Hor. distance from a to Resultant

448388 ÷ 56236 = 7.97

Mt. at a

1070 × 34.97 + 7366 × 13.5 + 20114 × 11.33

= 37418 + 99441 + 227891

= 364750

Ver. distance from a to Resaltant

364750 ÷ 28550 = 12.78

$$\tan\theta = \frac{28550}{56236} = 0.508$$

12.78 × 0.508 = 6.49

7.97 − 6.49 = 1.49

$$P = \frac{2 \times 56236}{3 \times 1.49} = 25162^{\#/\square'} \fallingdotseq 11.2^{ton/\square'}$$

Limit of foundation pressure of firm coarse sand and gravel

(Q) = 8 ton/□'

For Earth quake $Q = 8 \times (1 + \frac{60}{100}) = 12.8$

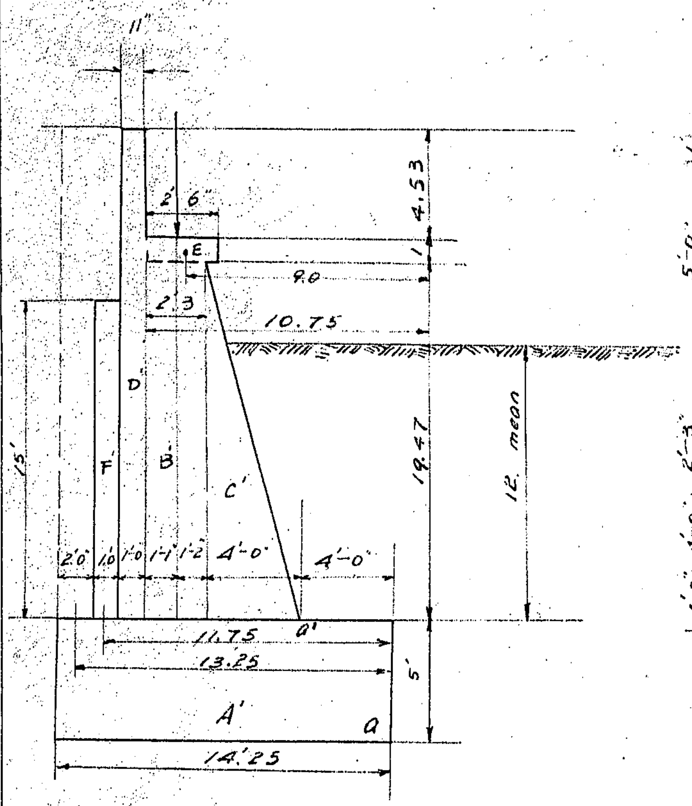
∴ safe

ŌYABASHI

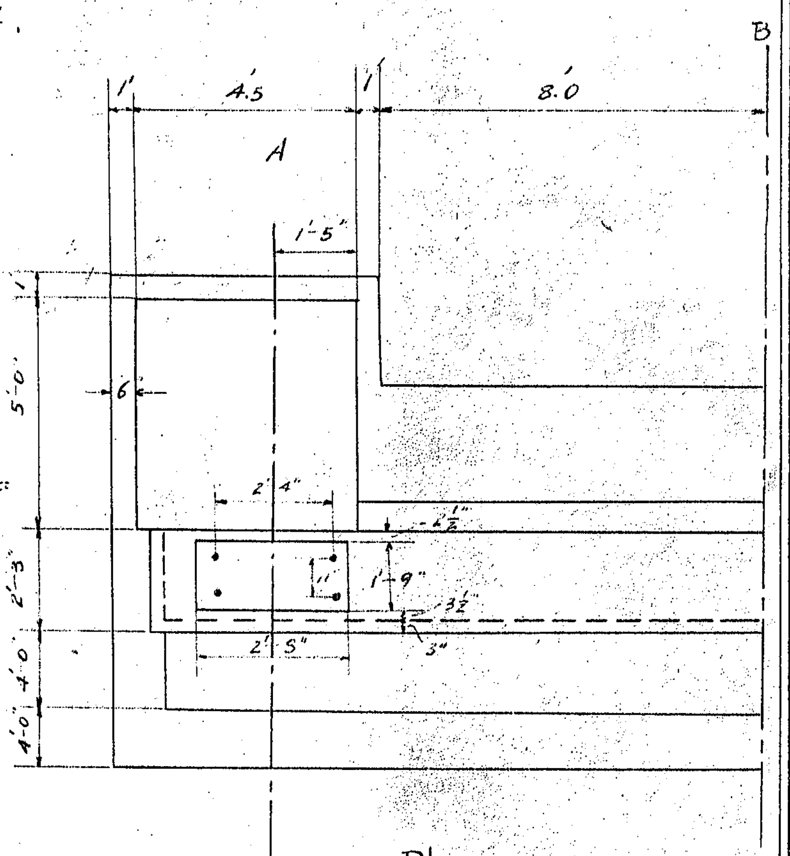
Abutment of Left bank

(I) Abutment of Left bank.

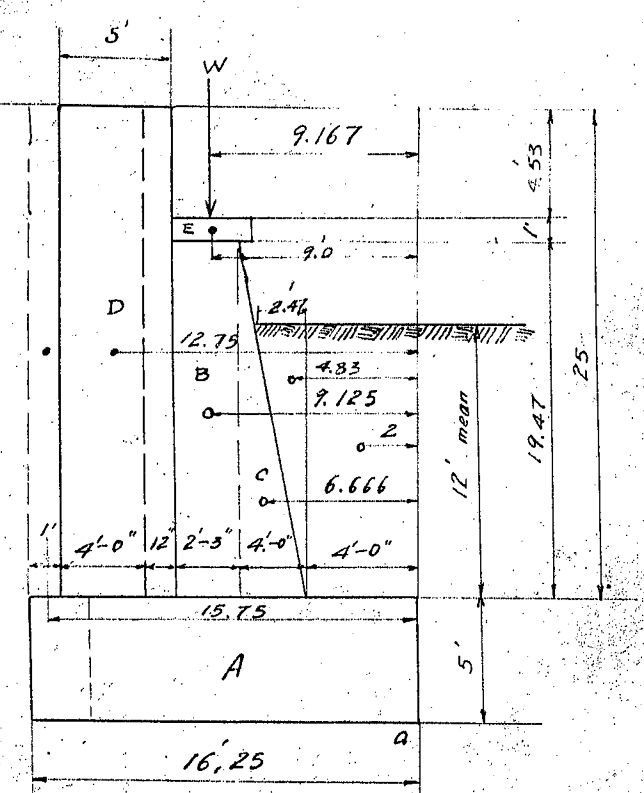
a) Dimension of abutment.



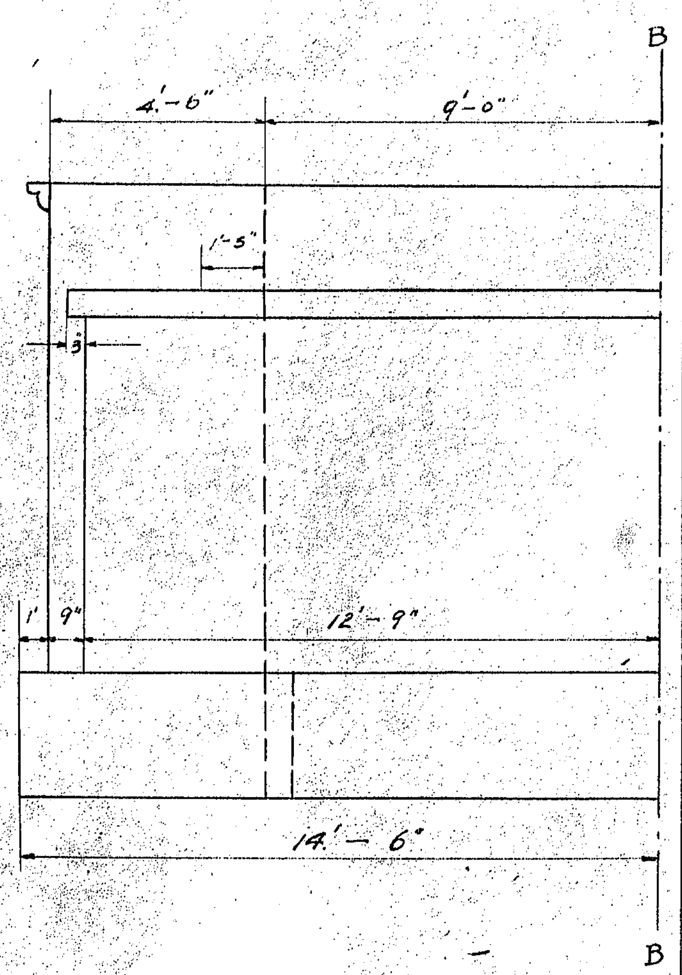
section B—B



Plan



section A—A



Front Elevation

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Vertical loads for section A–A.

Weight of concrete

$A = 16.25 \times 5 \times 140 = 11375^{\#}$

$B = 19.47 \times 2.25 \times 140 = 6133^{\#}$

$C = \frac{1}{2} \times 19.47 \times 4 \times 140 = 5452^{\#}$

$D = 5 \times 25 \times 140 = 17500^{\#}$

$E = 1 \times 2.5 \times 140 = 350^{\#}$

Weight of superstructure per unit length on abutment

$= 558036 \div 25.5 \times 2 = 10942^{\#}$

Weight of earth on footing

Rear side $= 1 \times 25 \times 100 = 2500^{\#}$

Front side $\begin{cases} 12 \times 2.47 \times \frac{1}{2} \times 100 = 1482 \\ 12 \times 4 \times 100 = 4800 \end{cases}$

Total load $= \underline{60534} = P$.

Neglected the effect of live load surcharge for high wall.

Center of gravity

$M.$ at $a = 11375 \times \frac{16.25}{2} + 6133 \times 9.125 + 5452 \times 6.666$
$+ 17500 \times 12.75 + 350 \times 9.0 + 10942 \times 9.167$
$+ 2500 \times 15.75 + 1482 \times 4.82 + 4800 \times 2$
$= 92422 + 55964 + 36343 + 223125$
$+ 3150 + 100305 + 39375 + 7143 + 9600$
$= \underline{567427}$

Hor. dist. between the center of gravity and a

$567427 \div 6.0534 = 9.37$

Eccenticity $= 9.37 - \frac{16.25}{2} = 1.245 \doteqdot 1.25$

when live load excluded

live load per lin. ft. $= 3812^{\#}$

$\therefore P = 60534 - 3812 = 56722$

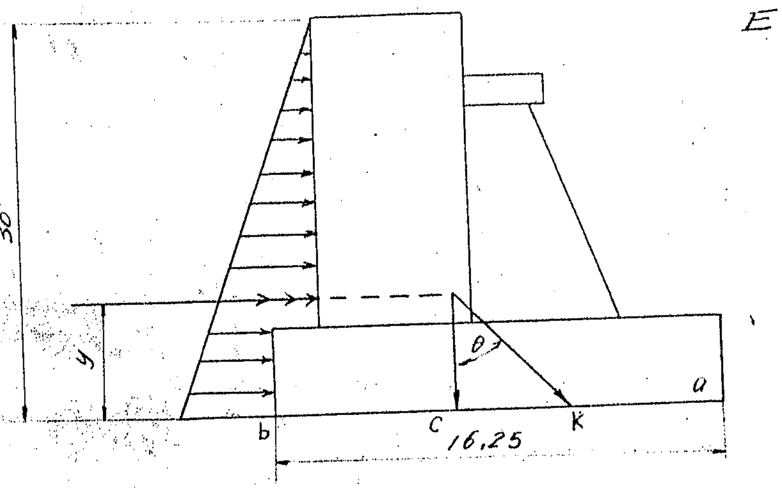
live load mt. at $a = 34945^{\#}$

$M.$ at $a = 567427 - 34945 = 532482^{\#}$

Hor. dist. bet. the center of gravity and a

$532482 \div 56722 = 9.39$

Eccentricity $= 9.39 - \frac{16.25}{2} = 1.265 \doteqdot 1.27$



$$E = \frac{1}{2} r h^2 M$$

$$= \frac{100 \times \overline{30}^2}{2} \times \frac{1}{3}$$

$$= 15000^{\#}$$

$$y = \frac{30}{3} = 10.'00$$

When live load included $\tan\theta = \frac{15000}{60534} = 0.248 < 0.577$

" " " excluded $\tan\theta = \frac{15000}{56722} = 0.264 < 0.577$

$\therefore$ safe for sliding

$K_c = y\tan\theta = 10 \times 0.248 = 2.48$

or $K_c =$ " $= 10 \times 0.264 = 2.64$

$K_a = 9.37 - 2.48 = \underline{6.89}$

or $K_a = 9.39 - 2.64 = \underline{6.75}$

one third of width of base $= \frac{16.25}{3} = \underline{5.'42}$

For bearing

Eccentricity of resultant

$\frac{1}{2}16.25 - 6.89 = 1.235 \doteqdot 1.23$ or $\frac{1}{2}16.25 - 6.75 = 1.375 = 1.38$

$\sigma = \frac{N}{A}\left(1 \pm \frac{\sigma e}{ab}\right)$

$\frac{60534}{16.25}\left(1 \pm \frac{6 \times 1.23}{16.25}\right)$

$\sigma_1 = 3725 \times 1.454 = 5416^{\#}/_{\square'} \doteqdot 2.4\ ^{ton}/_{\square'}$

$\sigma_2 = 3725 \times 0.546 = 2034^{\#}/_{\square'} \doteqdot 0.9\ ^{Ton}/_{\square'}$

$\sigma = \frac{N}{A}\left(1 \pm -\frac{6e}{ab}\right)$

$\frac{56722}{16.25}\left(1 \pm \frac{6 \times 1.38}{16.25}\right)$

$\sigma_1 = 3409 \times 1.51 = 5271^{\#}/_{\square'} \doteqdot 2.4\ ^{Ton}/_{\square'}$

$\sigma_2 = 3491 \times 0.49 = 1711^{\#}/_{\square'} \doteqdot 0.76\ ^{Ton}/_{\square'}$

Vertical loads for section B—B

Weight of concrete

$A' = 14.25 \times 5 \times 140 = 9975^{\#}$

$B' = 2.25 \times 19.47 \times 140 = 6133^{\#}$

$C' = 4.00 \times 19.47 \times \frac{1}{2} \times 140 = 5452^{\#}$

$E' = 1 \times 2.5 \times 140 = 350^{\#}$

$D' = 1 \times 25 \times 140 = 3500^{\#}$

$F = 1 \times 15 \times 140 = 2100^{\#}$

Weight of earth on footing

Rear side $\begin{cases} 2 \times 25 \times 100 = 5000 \\ 1 \times 10 \times 100 = 1000 \end{cases}$

Front side $\begin{cases} 12 \times 2.47 \times \frac{1}{2} \times 100 = 1482 \\ 12 \times 4 \times 100 = 4800 \end{cases}$

Weight of super structure $= 10942$

Total load per. lin. ft $= 50734$

Neglected the effect of live load surclarge for high wall

Center of gravity

$$M.\ at\ a = 9975 \times \frac{14.25}{2} + 6133 \times 9.125 + 5452 \times 6.666$$
$$+ 350 \times 9 + 3500 \times 10.75 + 2100 \times 11.75$$
$$+ 5000 \times 13.25 + 1000 \times 11.75 + 1482 \times 4.82$$
$$+ 4800 \times 2 + 10942 \times 9.167$$
$$= 71072 + 55964 + 36343 + 3150 + 37625$$
$$+ 24675 + 66250 + 11750 + 7143 + 9600$$
$$+ 100305$$
$$= 423877$$

Hor. distance between the center of gravity and a

$423877 \div 50734 = 8.35$

Eccentricity $= 835 - \frac{1}{2} \times 1425 = 1.225$

When live load excluded

$P = 50734 - 3812 = 46922$

$M.\ at\ a = 423877 - 34945 = 388932$

Hor. distance between the center of gravity and a

$388932 \div 46922 = 8.29$

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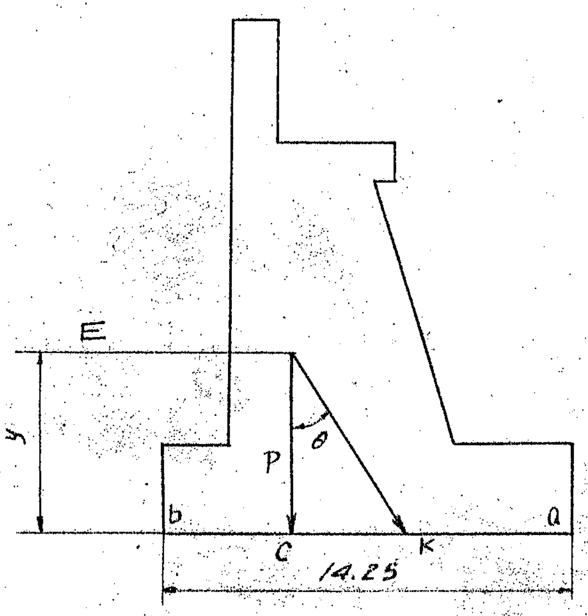
Eccentricity $= 8.29 - \frac{1}{2} \times 14.25 = 1.165 = 1.17$

Earth pressure $= 15000$

live load included $\tan\theta = \dfrac{15000}{50734} = 0.296 < 0.577$

live load excluded $\tan\theta = \dfrac{15000}{46922} = 0.320 < 0.577$

$\therefore$ safe for sliding



$y = \frac{30}{3} = 10'$

one third of width of base $= 4.75$

one fourth of width of base $= 3.56$

$Kc = y \tan\theta = 10 \times 0.296 = 2.96$

or $Kc =$ " $= 10 \times 0.320 = 3.20$

$Ka = 8.35 - 2.96 = 5.39$

or $Ka = 8.29 - 3.20 = 5.09$

Eccentricity of resultant $= 7.125 - 5.39 = 1.74$

or " $= 7.125 - 5.09 = 2.04$

live load included

$\sigma = \frac{N}{A}\left(1 \pm \frac{6 \cdot e}{a\,b}\right)$

$= \frac{50734}{14.25}\left(1 \pm \frac{6 \times 1.74}{14.25}\right)$

$= 3560(1 \pm 0.733)$

$\sigma_1 = 3560 \times 1.733 = 6169 \,\#/\square' = 2.8 \text{ ton}/\square'$

$\sigma_2 = 3560 \times 0.267 = 951 \,\#/\square' = 0.4 \text{ ton}/\square'$

live load excluded

$\sigma = \frac{N}{A}\left(1 \pm \frac{6 \cdot e}{a\,b}\right)$

$= \frac{46922}{14.25}\left(1 \pm \frac{6 \times 2.04}{14.25}\right)$

$= 3293 \times$

$\sigma_1 = 3293 \times 2.76 = 9089 \,\#/\square' = 4.1 \text{ ton}/\square'$

$\sigma_2 = 3293 \times 1.76 = 5796 = 2.6 \text{ ton}/\square'$

Stability against Earth quake motion

a) Vertical loads for section B ~ B

Weight of concrete

$A' = 14.25 \times 5 \times 140 = 9975$

$B' = 2.25 \times 19.47 \times 140 = 6133$

$C' = 4. \times 19.47 \times \frac{1}{2} \times 140 = 5452$

$E' = 1 \times 2.5 \times 140 = 350$

$D' = 1 \times 25 \times 140 = 3500$

$F' = 1 \times 15 \times 140 = 2100$

Summary $= 35490$

Weight of earth on footing

Rear side $\begin{cases} 1 \times 10 \times 100 = 1000 \\ 2 \times 25 \times 100 = 5000 \end{cases}$

Front side $\begin{cases} 12 \times 2.47 \times \frac{1}{2} \times 100 = 1482 \\ 12 \times 4 \times 100 = 4800 \end{cases}$

Summary $= 12282$

Weight of super structure live load excluded per lin. ft.

$= 363636 \div (25.5 \times 2) = 7130^{\#}$

Total weight of earth and concrete per lin. ft. of substructure

$= 35490 + 12282 = 47772^{\#}$

Earth quake motion $= 47772 \times 0.15 = 7166^{\#}$

Earth quake motion of superstructure live load excluded per lin. ft.

$= 7130 \times 0.15 = 1070^{\#}$

Earth pressure in case of Earth quakes

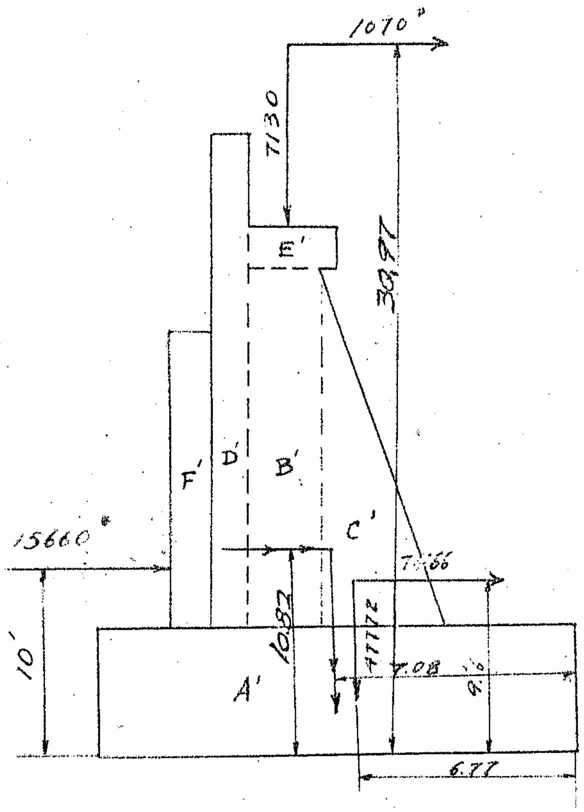
$$P = wh \frac{1 - \sin\varphi_1}{1 + \sin\varphi_1} \qquad \varphi = 30^\circ, \quad K = 0.15$$

$$\varphi_1 = \varphi - \tan K = 30^\circ - 8^\circ 32' = 21^\circ 28'$$

$$\therefore P = wh \frac{1 - \sin(21^\circ 28')}{1 + \sin(21^\circ 28')} = wh \times 0.464$$

$$E = \frac{wh^2}{2} \times 0.464 = \frac{100 \times \overline{30}^2 \times 0.464}{2} = 20880^{\#}$$

$$\frac{3}{4} \times 20880 = 15660^{\#}$$



Center of gravity

$$M. \text{ at } a = 9975 \times \frac{14.25}{2} + 6133 \times 9.125 + 5452 \times 6.666$$
$$+ 350 \times 9 + 3500 \times 10.75 + 2100 \times 11.75$$
$$+ 1000 \times 11.75 + 5000 \times 13.25 + 1482 \times 4.82$$
$$+ 4800 \times 2$$
$$= 71072 + 55964 + 36343 + 3150 + 37625$$
$$+ 24675 + 11750 + 66250 + 7143 + 9600$$
$$= 323572$$

Hor. distance between the center of gravity and a

$$= 323572 \div 47772 = 6.77$$

$$M. \text{ at } a = 9975 \times \frac{5}{2} + 6133 \times 14.74 + 5452 \times 11.49 + 350 \times 24.97$$
$$+ 3500 \times 17.5 + 2100 \times 12.5 + 1000 \times 25 + 5000 \times 17.5$$
$$+ 1482 \times 13 + 4800 \times 11$$
$$= 24938 + 90400 + 62643 + 8740 + 61250$$
$$+ 26250 + 25000 + 87500 + 19266 + 52800$$
$$= 458787$$

Ver. distance from a to C . G.

$$458787 \div 47772 = 9.6'$$

Total Vertical load $=47772+7130=54902$

Total horizontal load $=7166+1070+15660=23896$

M at a $=47772\times6.77+7130\times9.167=388777$

Hor. distance from a to Resultant

$=388777\div54902=7.08$

M. at a $7166\times9.6+1070\times30.97+15660\times10$

$=68794+33138+156600=258532$

Ver. distance from a to Resultant

$=258532\div23896=10.82$

$\tan\theta=\frac{23896}{54902}=0.44$

$10.82\times\tan\theta=10.82\times0.44=4.76$

$7.08-4.76=2.32$

$p=\frac{2\times54902}{3\times2.32}=15776^{\#}=7.04^{ton}/_{\square'}$

Limit of foundation pressure of firm coarse sand and gravel

$=8^{ton}/_{\square'}$

For Earthquake $Q=8\times(1+\frac{60}{100})=12.8$

$\therefore$ safe.

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（独立行政法人 土木研究所蔵）

大屋橋材料調書

長野縣

4 - End Post $L_0 - U_1$

Number	Description	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
2	Ls	12" @ 30#	11'-8 1/2"	326.95	654	
1	Pl.	20" × 7/16"	11'-8 1/2"	314.10	314	2'-10 1/2" Cover Pl.
2	Pls.	20" × 3/8"	1'-6"	38.25	77	Tie Pls.
6	Flat Bars	2 1/2" × 3/8"	2'-0 11/16"	6.56	39	Lacing Bars
1	Pl.	20" × 7/16"	4'-6 9/16"	135.27	135	Bent Pls.
2	Ls	4" × 3" × 3/8"	3'-10"	32.58	65	
2	Ls	3 1/2" × 3 1/2" × 3/8"	1'-4 1/2"	11.69	23	
2	Ls	4 1/2" × 3 1/2" × 3/8"	2'-6"	26.00	52	Cut from 5" × 3 1/2" × 3/8" Ls
1	Ls	3 1/2" × 3 1/2" × 3/8"	1'-1 1/2"	9.56	10	Connecting
4	Ls	4" × 3" × 5/16"	2'-3"	16.20	65	Diaphragm D
1	Pl.	10 1/2" × 5/16"	2'-3"	25.11	25	Diaphragm D
2	Pls.	3'-2" × 1/2"	5'-1/2"	246.77	494	Gusset Pls.
2	Pls.	1'-6 1/4" × 5/8"	1'-9 1/2"	63.60	127	Pin Pls.
3	Pls.	8 1/2" × 5/8"	1'-4 5/16"	29.55	49	Fillers.
2	Ls	4" × 4" × 3/8"	1'-0"	9.80	20	Hand rail Connecting Angles
1	Pl.	1'-0" × 5/16"	1'-4 1/2"	17.53	18	Hand Rail Connecting Angles
544	Rivet Heads	7/8" φ		0.1928	105	Shop Rivets
164	Rivet Heads	7/8" φ		0.1928	32	Field Rivets

Wt. = 2304#

4 × Wt. = 9216#

4 - Upper Chord $U_1 \sim U_2$

Number	Description	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
2	[s	12@30#	12'-7"	377.50	755	
1	Pl.	20×7/16"	12'-7"	374.35	374	Cover Pl.
2	Pls.	20"×3/8"	1'-6"	38.25	77	Tie Pls.
2	Pls.	2'-1"×3/8"	3'-3 1/2"	94.37	189	Gusset Pls.
2	Pls.	9 1/2"×3/8"	1'-7"	19.17	38	Splice Pls.
1	Pl.	20"×7/16"	1'-7"	47.10	47	Splice Pls.
2	Flat Bars	3"×3/8"	1'-6"	5.75	12	Splice Pls.
8	Flat Bars	2 1/2	2'-0 11/16"	6.56	53	Lacing Bar
308	Rivt Heads	7/8"φ		0.1928	59	Shop Rivet
144	Rivt Heads	7/8"φ			28	Field Rivet

Wt.=1632

4×Wt.=6528

4 - Upper Chord $U_2 \sim U_3$

2	[s	12"@30#	12'-3 9/16"	368.91	738	
1	Pl.	20"×7/16"	12'-3 9/16"	365.83	366	Cover Pl.
2	Pls.	20"×3/8"	1'-6"	38.25	77	Tie Pls.
2	Pls.	2'-1 1/2"×3/8"	3'-0 1/2"	91.11	182	Gusset Pls.
2	Pls.	9 1/2"×3/8"	1'-7"	19.17	38	Splice Pls.
1	Pl.	20"×7/16"	1'-7"	47.10	47	Splice Pls.
2	Flat Bars	3"×3/8"	1'-6"	5.75	12	Splice Pls.
8	Flat Bars	2 1/2"×3/8"	2'-0 11/16"	6.56	53	Lacing Bars
304	Rivet Heads	7/8"φ		0.1928	59	Shop Rivets
144	Rivet Heads	7/8"φ		0.1928	28	Field Rivets

Wt. =1600

4×Wt. =6400

4-Upper Chord U3~U4						
Number	Description	Dimension		Weight (#)		Remarks
		Size	Length	Unit	Total	
2	[s	12"@30#	12'-1 7/16"	363.59	727	
1	Pl	20"×7/16"	12'-1 7/16"	360.56	361	Cover Pls.
2	Pls.	20"×3/8"	1'-6"	38.25	77	Tie Pls.
2	Pls.	2'-2"×3/8"	3'-0"	90.27	181	Gusset pls.
2	Pls	9 1/2"×3/8"	1'-7"	19.17	38	Splice Pls.
1	Pl.	20"×7/16"	1'-7"	47.10	47	Splice Pls.
2	Flat Bars.	3"×3/8"	1'-6"	5.75	12	Splice Pls
8	Flat Bars	2 1/2"×3/8"	2'-0 11/16"	6.56	53	Lacing Bars
300	Rivet Heads	7/8"φ		0.1928	58	Shop Rivets
144	Rivet Heads	7/8"φ		0.1928	28	Field Rivets

Wt.=1582
4×Wt.=6328

2-Upper Chord U4~U4'						
2	[s	12"@30#	12'-0 3/4"	361.88	724	
1	Pl	20"×7/16"	12'-0 3/4"	358.86	359	Cover Pls.
2	Pls	20"×3/8"	1'-6"	38.25	77	Tie Pl.
4	Pls	2'-2"×3/8"	3'-0"	89.10	356	Gusset Pls.
4	Pls	9 1/2"×3/8"	1'-7"	19.17	77	Splice Pls.
2	Pls	20"×7/16"	1'-7"	47.10	94	Splice Pls.
4	Flat Bars	3"×3/8"	1'-6"	5.75	23	Splice Pls.
8	Flat Bars	2 1/2"×3/8"	2'-0 11/16"	6.56	53	Lacing Bars
312	Rivet Heads	7/8"φ		0.1928	60	Shop Rivets
144	Rivet Heads	7/8"φ		0.1928	28	Field Rivets

Wt.=1851
2×Wt.=3702

4-Lower Chord L_0-L_1-J_{L1}

Number	Description	Dimension: Size	Dimension: Length	Weight: Unit	Weight: Total	Remarks
2	[s	12"@30#	18'-10 3/8"	565.94	1132	
2	Pls.	9" x 3/8"	18'-10 3/8"	216.57	433	Reinforcing P's.
4	Ls	4x3x 5/16"	0'-9	5.40	22	Diaphragm
1	Pl.	9" x 5/16"	0'-10 1/2"	8.37	8	Diaphragm
2	Pls.	1'-8 1/2" x 3/8"	2'-2"	47.23	95	Gusset Pls.
1	Pl.	1'-2" x 5/16"	2'-2"	32.24	32	Gusset Pl.
1	Pl.	2'-3" x 5/16"	2'-3 5/16"	65.30	65	Gusset Pl.
10	Pls.	9" x 5/16"	1'-0"	9.56	96	Tie Pls.
308 128	Rivet Heads	7/8" φ 3/4" φ		0.1928 0.1242	59 16	Shop Rivets.
200 56	Rivet Heads	7/8" φ 3/4" φ		0.1928 0.1242	39 7	Field Rivets.

Wt.=2004

4xWt.=8016

4-Lower Chords J_{L1}-L_2-L_3-J_{L2}

Number	Description	Size	Length	Unit	Total	Remarks
2	[s	12"@30#	23'-11 3/4"	719.38	1439	
2	Pls.	9" x 3/8"	23'-11 3/4"	275.28	551	Reinforcing Pls.
8	Ls	4x3x 5/16	0'-9"	5.40	43	Diaphragm
2	Pls.	9" x 5/16"	0'-10 1/2"	8.37	17	Diaphragm
2	Pls.	2'-2" x 3/8"	4'-1"	117.00	234	Gusset Pls.
2	Pls.	2'-2" x 3/8"	3'-4 1/2"	89.18	178	Gusset Pls.
2	Pls.	2'-3" x 5/16"	2'-3 5/16	65.30	131	Gusset Pls.
10	Pls.	9" x 5/16"	1'-0	9.56	96	Tie Pls.
2	Pls.	12" x 1/2"	3'-0 1/4"	61.62	123	Splice Pls.
2	Pls.	12 x 3/8	2'-3 1/4"	34.74	70	Splice Pls.
2	Pls.	9" x 1/2"	1'-6 1/4"	23.27	47	Splice Pls.
556 184	Rivet Heads	7/8" φ 3/4" φ		0.1928 0.1242	107 23	Shop Rivets.
128 64	Rivet Heads	7/8" φ 3/4" φ		0.1928 0.1242	25 8	Field Rivets.

Wt.=3092

4xWt.=12,368

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2 - Lower Chords $JL_2 - L_4 - L_4'$ JL_2'

Number	Description	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
2	∠s	12" @ 30#	20'-11$\frac{3}{4}$"	62938	1259	
2	P/s.	9" × $\frac{3}{8}$"	20'-11$\frac{3}{4}$"	240.84	482	Reinforcng P/s.
8	∠s	4" × 3" × $\frac{5}{16}$"	0'-9"	5.40	43	Diaphragm
2	P/s.	9" × $\frac{5}{16}$"	0'-10$\frac{1}{2}$"	8.37	17	Diaphragm
4	P/s.	2'-2" × $\frac{3}{8}$"	3'-5$\frac{1}{2}$"	108.83	435	Gusset P/s.
2	P/s.	2'-3" × $\frac{5}{16}$"	2'-3$\frac{5}{16}$"	65.30	131	Gusset P/s.
6	P/s.	9" × $\frac{5}{16}$"	1'-0"	9.56	57	Tie P/s.
4	P/s.	12" × $\frac{1}{2}$"	3'-0$\frac{1}{4}$"	61.62	247	Splice P/s.
4	P/s.	12" × $\frac{3}{8}$"	2'-3$\frac{1}{4}$"	37.74	151	Splice P/s.
4	P/s.	9" × $\frac{1}{2}$"	1'-6$\frac{1}{4}$"	23.27	93	Splice P/s.
512 136	Rivet Heads	$\frac{7}{8}$"φ		0.1928 0.1242	99 17	Shop Rivet
96 64	Rivet Heads.	$\frac{7}{8}$"φ		0.1928 0.1242	19 8	Field Rivet

Wt = 3058

2 × Wt = 6116

4 - Vertcals $L_1 - U_1$

2	∠s	4" × 3" × $\frac{5}{16}$"	3'-5$\frac{15}{16}$"	25.16	50	
2	∠s	4" × 3" × $\frac{5}{16}$"	3'-4$\frac{7}{16}$"	24.26	49	
1	Pl.	20$\frac{3}{4}$" × $\frac{5}{16}$"	3'-2$\frac{15}{16}$"	51.86	52	Tie Plate (Handra[illegible] pl)
52	Rivet Heads	$\frac{3}{4}$"φ		0.1242	6	Shop Rivet
64	Rivet Heads	$\frac{7}{8}$"φ		0.1928	12	Field Rivet

Wt = 169

4 × Wt = 676

4 - Verticals $L_2 - U_2$

Number	Description	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
2	L^s	$4'' \times 3'' \times \frac{5}{16}''$	$7'-1\frac{7}{8}''$	51.53	103	
2	L^s	$4'' \times 3'' \times \frac{5}{16}''$	$7'-0\frac{3}{8}''$	50.63	101	
1	Pl.	$20\frac{3}{4}'' \times \frac{5}{16}''$	$2'-4\frac{3}{4}''$	40.59	41	Tie Plate (Handrail Conn. Pl.)
1	Pl.	$12'' \times \frac{5}{16}''$	$1'-0''$	12.75	13	Tie Pl.
4	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'-2\frac{3}{8}''$	3.19	13	Lacing Bars
68	Rivet Heads	$\frac{3}{4}''\phi$		0.1242	8	Shop Rivet
64	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	12	Field Rivet

Wt.= 291

4 × Wt.= 1164

4 - Vertical $L_3 - U_3$

Number	Description	Size	Length	Unit	Total	Remarks
4	L^s	$4'' \times 3'' \times \frac{5}{16}''$	$9'-5\frac{15}{16}''$	68.36	273	
1	Pl	$20\frac{3}{4}'' \times \frac{5}{16}''$	$2'-4\frac{3}{4}''$	40.59	41	Tie Plate (Handrail Conn Pl.)
1	Pl	$12'' \times \frac{5}{16}''$	$1'-3''$	15.94	16	Tie Pl
6	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'-2\frac{3}{8}''$	3.19	19	Lacing Bars
80	Rivet Heads	$\frac{3}{4}''\phi$		0.1242	10	Shop Rivet
64	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	12	Field Rivet

Wt.= 371

4 × Wt.= 1484

4 - Verticals $L_4 - U_4$

Number	Description	Dimension Size	Dimension Length	Weight Unit	Weight Total	Remarks
4	L^s	$4'' \times 3'' \times \frac{5}{16}''$	$10'$-$8\frac{13}{16}''$	77.29	309	
1	Pl.	$20\frac{3}{4}'' \times \frac{5}{16}''$	$2'$-$4\frac{3}{4}''$	40.59	41	Tie Plate (Handrail conn. Pl.)
1	Pl.	$12'' \times \frac{5}{16}''$	$1'$-$3''$	15.94	16	Tie Pl.
8	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'$-$2\frac{3}{8}''$	3.19	26	Lacing Bars
88	Rivet Heads	$\frac{3}{4}''\phi$		0.1242	11	Shop Rivets
64	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	12	Field Rivets

Wt. = 415
4 x Wt. = 1660

4 - Diagonals $U_1 - L_2$

Number	Description	Dimension Size	Dimension Length	Weight Unit	Weight Total	Remarks
4	L^s	$3'' \times 3'' \times \frac{5}{16}''$	$9'$-$9\frac{3}{8}''$	59.67	239	
2	Pls.	$12'' \times \frac{5}{16}''$	$1'$-$0''$	12.75	26	Tie Pls.
8	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'$-$2\frac{3}{8}''$	3.19	26	Lacing Bars
64	Rivet Heads	$\frac{3}{4}''\phi$		0.1242	8	Shop Rivets
64	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	12	Field Rivets

Wt. = 311
4 x Wt. = 1244

4 - Diagonals $U_2 - L_3$

Number	Description	Dimension Size	Dimension Length	Weight Unit	Weight Total	Remarks
4	L^s	$4'' \times 3'' \times \frac{5}{16}''$	$12'$-$2\frac{5}{8}''$	87.98	352	
2	Pls.	$12'' \times \frac{5}{16}''$	$1'$-$3''$	15.94	32	Tie Pls.
10	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'$-$2\frac{3}{8}''$	3.19	32	Lacing Bars
80	Rivet Heads	$\frac{3}{4}''\phi$		0.1242	10	Shop Rivets
64	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	12	Field Rivets

Wt. = 438
4 x Wt. = 1752

4 - Diagonals $U_3 - L_4$						
Number	Descrption	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
4	Ls	$4'' \times 3'' \times \frac{5}{16}''$	$13'-11\frac{13}{16}''$	100.69	403	
2	P/s.	$12'' \times \frac{5}{16}''$	$1'-3''$	15.94	32	Tie P/s.
12	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'-2\frac{3}{8}''$	3.19	38	Lacing Bars
88	Rivet Heads	$\frac{3}{4}''\phi$		0.1242	11	Shop Rivet
64	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	12	Field Rivet

Wt. = 496

Wt. × 4 = 1984

2-Diagonals $U_4 - L_4'$, $U_4 - L_4$						
2	Ls	$4'' \times 3'' \times \frac{5}{16}''$	$14'-9\frac{1}{2}''$	106.50	213	
2	Ls	$4'' \times 3'' \times \frac{5}{16}''$	$7'-3\frac{13}{16}''$	52.69	105	
2	Ls	$4'' \times 3'' \times \frac{5}{16}''$	$6'-11\frac{11}{16}''$	50.21	100	
2	P/s.	$12'' \times \frac{5}{16}''$	$1'-3''$	15.94	32	Tie P/s.
4	P/s	$12'' \times \frac{5}{16}''$	$1'-0''$	12.75	51	Tie P/s.
35	Flat Bars	$2\frac{1}{2}'' \times \frac{5}{16}''$	$1'-2\frac{3}{8}''$	3.19	112	Lacing Bars
2	P/s.	$12'' \times \frac{5}{16}''$	$2'-6''$	31.88	64	Splice P/s.
16 180	Rivet Heads	$\frac{7}{8}''\phi$ $\frac{3}{4}''\phi$		0.1928 0.1242	3 22	Shop Rivets
96	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	19	Field Rivets

Wt. = 721

2 × Wt. = 1442

9 - Lower Laterals

Number	Description	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
1	L	$3\frac{1}{2}" \times 3\frac{1}{2} \times \frac{5}{16}"$	$22'-1\frac{1}{2}$	15.930	159	Mark (A)
2	Ls	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$	$10'-8\frac{3}{4}"$	7.725	155	Mark (B)
1	Pl.	$1'-0" \times \frac{3}{8}"$	$2'-8"$	4.080	41	Splice Pl.
16	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	3	Shop Rivets
40	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	8	Field Rivets

Wt.= 366
9 x Wt.= 3294

2 - End Floor Beams

Number	Description	Size	Length	Unit	Total	Remarks
1	I	20" @ 75#	$19'-8\frac{11}{16}"$	147.93	1479	
10	Ls	$3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{5}{16}"$	$0'-8"$	4.8	48	Supporting Ls
4	Ls	$4" \times 4" \times \frac{7}{16}"$	$1'-4\frac{1}{2}"$	15.54	62	End Connection Ls
8	Pls.	$4\frac{3}{4}" \times \frac{5}{16}"$	$0'-9\frac{1}{2}"$	4.00	32	Stiffing Pls.
40	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	8	Shop Rivets
32	Rivet Heads	$\frac{1}{2}"\phi$		0.0398	1	Shop Rivets
60	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	12	Field Rivets
48	Rivet Heads	$\frac{1}{2}"\phi$		0.0398	2	Field Rivets

Wt.= 1644
2 x Wt.= 3288

8 - Intermediate Floor Beam

Number	Description	Size	Length	Unit	Total	Remarks
1	I	20" @ 85#	$19'-8\frac{3}{4}$	1676.98	1677	
10	Ls	$3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{5}{16}"$	$0'-8"$	4.80	48	Supporting Ls
4	Ls	$4" \times 4" \times \frac{7}{16}"$	$1'-4\frac{1}{2}"$	15.54	62	End Conn. Ls
8	Pls.	$4\frac{3}{4}" \times \frac{5}{16}"$	$0'-9\frac{1}{2}"$	4.00	32	Stiffining Pls.
40	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	8	Shop Rivets
32	Rivet Heads	$\frac{1}{2}"\phi$		0.0398	1	Shop Rivets
60	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	12	Field Rivets
8	Rivet Heads	$\frac{1}{2}"\phi$		0.0398	1	Fiveld Rivets

Wt.= 1841
8 x Wt.= 14728

Stringers (S)E (S)I & Bracket (B)

Number	Description	Dimension Size	Dimension Length	Weight Unit	Weight Total	Remarks
45	I	12"@35#	11'-10"	408.77	18395	(S)E (S)I
10	I	12"@35#	0'-10"	26.47	265	(B) Bracket
200	Ls	$6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	0'-9"	8.78	1756	Connection Angles
10	P/s	$5'' \times \frac{3}{8}''$	1'-10"	11.70	117	Splice P/s
1000	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	193	Shop Rivets
560	Rivet Heads	$\frac{1}{2}''\phi$		0.0398	22	Field Rivets

wt. = 20748

2 - Handrail

Number	Description	Size	Length	Unit	Total	Remarks
20	Ls	$3 \times 3 \times \frac{5}{16}$	$3'-10\frac{1}{16}''$	23.73	475	Mark (A)
4	Ls	$3\frac{1}{2} \times 3 \times \frac{1}{4}$	12'-11"	69.75	279	Mark (E)u & (E)L
14	Ls	$3\frac{1}{2} \times 3 \times \frac{1}{4}$	12'-0	64.80	907	Mark (I)u & (I)L
10	Ls	$4 \times 3 \times \frac{1}{4}$	$0'-6\frac{5}{16}''$	3.05	31	Mark (a)I
10	Flat Bars	$3'' \times \frac{5}{16}''$	1'-0"	3.19	32	Filler (F)
36	Flat Bars	$2'' \times \frac{1}{4}''$	$22'-6\frac{1}{2}''$	38.32	1380	Mark (B)
162	Flat Bars	$2'' \times \frac{1}{4}''$	$2'-0\frac{1}{2}''$	3.47	562	Mark (0)
8	Flat Bars	$3\frac{1}{2}'' \times \frac{5}{16}''$	$0'-6\frac{1}{2}''$	2.02	16	Mark (P)
2	P/s	$7\frac{27}{32} \times \frac{1}{4}$	$2'-5\frac{1}{2}''$	16.40	33	End P/s
36	Ls	$4'' \times 2\frac{11}{16}'' \times \frac{1}{4}''$	$0'-2\frac{11}{16}''$	1.30	47	Conn. Ls (cut from $4'' \times 3 \times \frac{1}{4}$ L)
80	Rivet Heads	$\frac{7}{8}''\phi$		0.1928	15	Shop Rivets
2180	" "	$\frac{1}{2}''\phi$		0.0398	87	Shop Rivets
60	" "	$\frac{7}{8}''\phi$		0.1928	12	Field Rivets
92	" "	$\frac{1}{2}''\phi$		0.0398	4	Field Rivets

wt. = 3880

2 x wt. = 7760

1 - Expansion Joints

Number	Description	Dimension		Weight		Remarks
		Size	Length	Unit	Total	
1	Pl	$8\frac{1}{2}" \times \frac{3}{8}"$	18'-0"	195.12	195	
1	Pl	$2" \times \frac{3}{8}"$	18'-0"	45.90	46	
1	Pl	$6\frac{1}{2}" \times \frac{3}{8}"$	18'-0"	149.22	149	
2	Ls	$4" \times 4" \times \frac{3}{8}$	18'-0"	176.40	353	
2	Bent Pls.	$9\frac{3}{8}" \times 6 \times \frac{3}{8}$	0'-5"	8.17	16	
4	〃	$8\frac{3}{4}" \times 6" \times \frac{3}{8}$	0'-5"	7.84	31	
4	〃	$7" \times 6" \times \frac{3}{8}$	0'-5"	6.91	28	
48	Bolts	$\frac{1}{2}"\phi$	0'-9"	0.63	30	Bolt & Nut
108	Rivet Heads	$\frac{1}{2}"\phi$		0.0398	4	Shop Rivets

Wt. = 852

2 - Expansion Shoes & Rollers

Number	Description	Size	Length	Unit	Total	Remarks
2	Pls.	$1'-3\frac{1}{4} \times \frac{1}{2}$	1'-9"	36.84	74	Rib Pls.
2	Pls.	$1'-3\frac{1}{4}" \times \frac{5}{8}"$	1'-9"	46.05	92	Rib Pls.
4	Ls	$6" \times 4" \times \frac{1}{2}"$	1'-9"	28.35	113	
2	Pls.	$1'-9" \times 1\frac{1}{4}"$	2'-8"	238.00	476	Base Pls. Planed to $1\frac{1}{8}"$
2	Flat Bars	$2 \times \frac{1}{4}"$	1'-9"	2.98	6	Spacer Bars
4	Rollers	$4"\phi$	$2'-0\frac{3}{4}"$	81.27	325	
2	Rods	$\frac{3}{4}"\phi$	$2'-2\frac{3}{8}"$	3.30	7	Grip b = $2'-0\frac{7}{8}"$
2	Pls.	$2\frac{1}{2}" \times \frac{3}{4}"$	$1'-9\frac{1}{4}"$	11.30	231	Side Pls.
2	Ls	$3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{3}{8}"$	1'-9"	14.88	30	
92	Rivet Heads	$\frac{7}{8}"\phi$		0.1928	18	Shop Rivets
3	Rivet Heads	$\frac{1}{2}"\phi$		0.0398	1	Shop Rivet

Wt. = 1165

2 × Wt. = 2330

2 - Fixed Shoes

Nomber	Description	Dimension: Size	Dimension: Length	Weight: Unit	Weight: Total	Remarks
2	Pls.	1'-3$\frac{1}{4}$"×$\frac{1}{2}$"	1'-9"	36.84	74	Rib Pls.
2	Pls.	1'-3$\frac{1}{4}$"×$\frac{5}{8}$"	1'-9"	46.05	92	Rib Pls.
4	∠s	6"×4"×$\frac{1}{2}$"	1'-9"	28.35	113	
2	Pls.	1'-9"×1$\frac{1}{8}$"	2'-8"	214.22	428	Base Pls.
6	[s	4 @7.25#	2'-8"	19.33	116	
2	[s	4 @7.25#	2'-0"	14.50	29	
84	Rivet Heads	$\frac{7}{8}$"φ		0.1928	16	Shop Rivets
92	Rivet Heads	$\frac{1}{2}$"φ		0.0398	4	Field Rivets

Wt. = 872

2 × Wt. = 1744

4 - Pins & Nuts

Number	Description	Size	Length	Unit	Total	Remarks
1	Pin	4$\frac{1}{2}$"φ	1'-8"	83.89	84	
2	Nut	5$\frac{3}{4}$"×1$\frac{3}{8}$"	0'-6$\frac{5}{8}$"	4.60	9	6$\frac{5}{8}$" 5$\frac{3}{4}$"

Wt. = 93

4 × Wt. = 372

16 - Anchor Bolts

Number	Description	Size	Length	Unit	Total	Remarks
1	Anchor Bolts	1$\frac{1}{4}$"φ	2'-3$\frac{1}{2}$"	9.56	10	Fixed & Expansion end
1	Nut	2$\frac{1}{16}$"×1$\frac{1}{4}$"		0.85	1	

Wt. = 11

16 × Wt. = 176

Total wt. of one span = 125372# = 55.9696 ton

Do. of five span = 626860# = 279.848 ton