梓　　橋

設　計　計　算　書

長野縣土木課道路改工	
路線名	縣道 松本糸魚川線
圖　名	設計々算書
技　師	鈴木和彦
設　計	北村清美　昭和　年　月　日完了
製　圖	仝　昭和　年　月　日完了
寫　圖	仝　昭和3年9月14日完了
照　査	昭和　年　月　日完了
全拾七枚ノ内其	

Assumed live loading

As shown in General drawing (Sheet No. 1)

Assumed Allowable Strengths

Concrete 1:2:4 mixture

Direct compression	35 kg. per sq. cm.
Fiber stress in concrete	45 kg/□cm for Positive moment
Fiber Stress in concrete	50 kg/□cm. for Negative moment
Punching shear	9 kg/□cm.
Shear (Plain concrete)	4 kg/□cm.
Shear (Reinforced)	9 kg/□cm.
Bearing stress	45 kg./□cm.
Bond stress	6 kg./□cm.

Reinforcing Bars

Tension and Compression	1 200 kg./□cm.
Shear	900 kg./□cm.

Structural Steel

Tension and Compression 1 200 kg/□cm.

Compression member for gross area $1\,500.(1-0.0055\ l/r)$ kg/□cm $\leqq 1{,}000$ kg/□cm

where l = Unsupported length of member in cm.

r = Least radius of gyration in cm.

Tension on Plate girder flange 1 200 kg/□cm.

Compression on Plate girder flange $1\,200.(1-0.012\ l/b)$ kg/□cm. $\leqq 1{,}100$ kg/□cm.

where l = Unsupported length of Plate girder flange in cm.

b = Width of Flange in cm.

Fiber stress of Pin		1 800 kg./□cm.
Shear	Plate	900 kg./□cm.
〃	Pin	900 kg./□cm.
〃	Shop rivets	850 kg/□cm.
〃	Field rivets	750 kg/□cm.
Bearing	Pin	1 800 kg/□cm.
〃	Shop rivets	1 700 kg./□cm.
〃	Field rivets	1 500 kg/□cm.
〃	Rollers per cm.	45 d kg. where d = Diam. of Roller in cm.

Miscellaneous data :-

Ratio between moduli of elasticities of Steel and Concrete $\frac{2{,}100{,}000}{140{,}000} = 15$ (n.)

Expansion coefficient of Steel = 0.0000 12 per 1° of C.

~~Tenpreture~~ Temperature change for Steel structure = ±30°C

Design of Superstructure

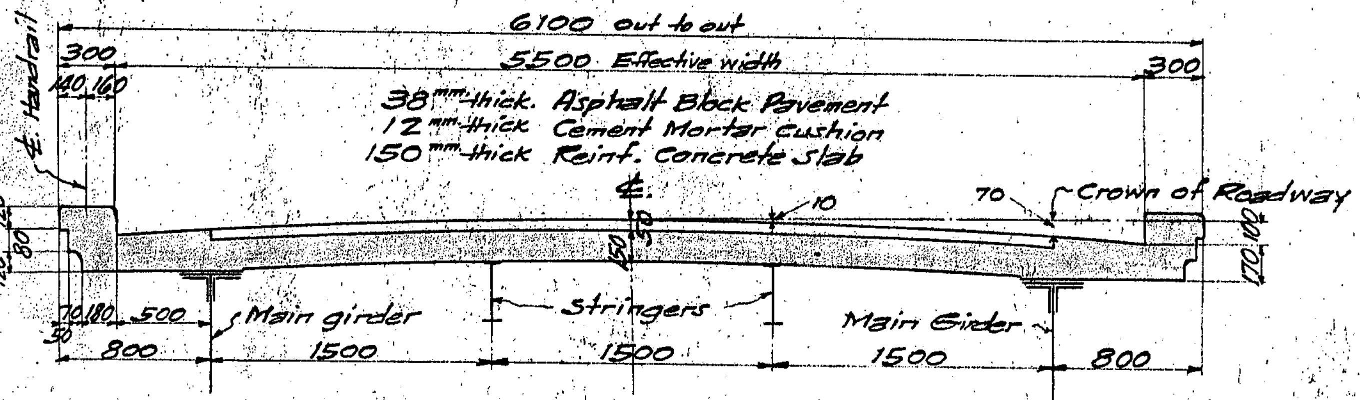
Cantaliver Deck Plate Girders 11 @ 24.4 m., Cantiliver length 4.7 m in 24.4 m span

Reinforced concret floor slab with Asphalt Block Pavement

1. Concrete Floor Slab

Span length of slab 1500 mm., Cross beam spacing about 5000 m.m.

Cross section of slab as shown on sketch below



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Dead load

Pavement assumed 100 kg. per sq. meter

Slab 15^{cm} assumed $\underline{360}$

460 kg. per sq. meter

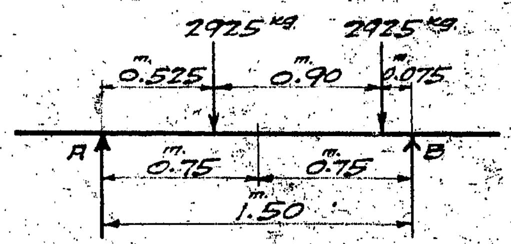
∴ Dead load Moment $= 0.1 \times 460 \times 1.5^2 = 103.5$ m. kg. $= 10{,}350$ cm. kg.

Dead load Shear $= 0.5 \times 460 \times 1.5 = 345$ kg.

Live load

Motor truck loading

	Rear Wheel	Front Wheel
Wheel concentration	2,250 kg.	750 kg.
30 % Impact	675	225
	2,925 kg.	975 kg.



Reaction $= 2925 \times 1.05 \div 1.50 = 2{,}047.5$ kg. at Support A.

Moment $= 2047.5 \times 0.525 = 1{,}075$ m kg

For continuity of Slab $M = 0.8 \times 1{,}075 = 860$ m-kg.

Distribution of wheel concentration on slab assumed as follows

Effective width $= \frac{2}{3} \times 1.5 + 0.3 = 1.30$ m.

Moment per meter strip $= 860 \div 1.3 = 661.5$ m-kg $= 66{,}150$ cm-kg.

Live load Shear neglecting continuity of slab and figured as simple beam. 1st concentrated load on support. Reaction $= 2{,}925 \times (1 + \frac{0.6}{1.5}) = 4{,}680$ kg.

∴ Shear per meter strip $= 4680 \div 1.3 = 3{,}600$ kg.

Summary of Moment and Shear

	Moments	Shears
Dead load	10,350 cm-kg.	345 kg.
Live load	66,150	3,600
For per meter strip	76,500 cm-kg.	3,945 kg.
For per 30 cm. strip	22,950 cm-kg. (M)	1,180 kg. (Q)

Check of the section at Intermediate span

Considered 30 cm. Strip ($b = 30$ cm.)

Compression side steel area (F_e') 1-1.3 cm. φ bar $= 1.33$ □cm.

Tension side steel area (F_e) 3-1.3 cm. φ bars $= \underline{3.99}$

Total steel area ($F_e' + F_e$) 4-1.3 cm. φ bars ····· 5.32 □cm.

Distance from the neutral axis to compression side edge

$$x = -\frac{n(F_e'+F_e)}{b} + \sqrt{\left\{\frac{n(F_e'+F_e)}{b}\right\}^2 + \frac{2n}{b}(F_e' h' + F_e h)} = -\frac{15 \times 5.32}{30} + \sqrt{\left\{\frac{15 \times 5.32}{30}\right\}^2 + \frac{2 \times 15}{30} \times (1.33 \times 2.5 + 3.99 \times 12.5)} = 5.1 \text{ cm.}$$

Moment of inertia about the neutral axis

$$J = \frac{bx^3}{3} + n F_e'(x-h')^2 + n F_e (h-x)^2 = \frac{30 \times 5.1^3}{3} + 15 \times 1.33 \times (5.1-2.5)^2 + 15 \times 3.99 \times (12.5-5.1)^2 = 4{,}740 \text{ cm}^4$$

Geometrical moment about the neutral axis

$$S = n F_e (h-x) = 15 \times 3.99 \times 7.4 = 443 \text{ cm}^3$$

Geometrical moment for upper strip

$$S' = bh'\left(x - \frac{h'}{2}\right) + n F_e'(x-h') = 30 \times 2.5 \times 3.85 + 15 \times 1.33 \times 2.6 = 341 \text{ cm}^3$$

Actual Stresses

Compression of Concrete $= \sigma_b = \frac{M \cdot x}{J} = \frac{22950 \times 5.1}{4740} = 24.7$ kg/□cm

Compression of Steel at Fe' $= \sigma_e' = n \sigma_b \frac{(x-h')}{x} = 15 \times 24.7 \times \frac{2.6}{5.1} = 190$ kg/□cm.

Tension of Steel at Fe $= \sigma_e = n \sigma_b \frac{(h-x)}{x} = 15 \times 24.7 \times \frac{7.4}{5.1} = 540$ kg/□cm.

Unit shear at Fe' $= \tau_0' = \frac{Q \cdot S'}{b \cdot J} = \frac{1180 \times 341}{30 \times 4740} = 2.8$ kg/□cm.

Unit shear at Fe $= \tau_0 = \frac{Q \cdot S}{b \cdot J} =$ [illegible] kg/□cm.

Unit Bond Stress at Fe = $\tau_h = \frac{Q.S}{U.J} = \frac{1180 \times 443}{3 \times 4.08 \times 4740} = 9.0\ kg/\square^{cm}$. Where U = Circumf. of bars $= 3 \cdot 4.08^{cm}$

2-bars as shown in sketch bent up at support and all bars to hooked at both ends to take care of adhesive force

Overhanging Slab carrying handrail

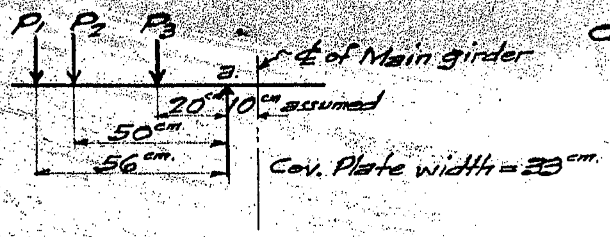
Dead load

Handrails, actual weight 146^{kg}/meter of one side call this $150^{kg}/m.$ (P_1)

Curbs $0.0787^{sq.m.} @ 2400^{kg} = 190^{kg}/m.$ (P_2)

Overhanging Slab for assumed span length $180^{kg}/m.$ (P_3)

Dead load concentration assumed as shown in sketch below



Considered moment at a

Dead load moment $= 150 \times 56 + 190 \times 50 + 180 \times 20 = 21,500^{cm\text{-}kg}$

Dead load Shear $= \Sigma P = 520^{kg}$

Live load

One wheel concentration near curb line. 28^{cm} distance from assuming point a

Live load Moment $= 2925 \times 28 = 81,900^{cm\text{-}kg}$

Live load Shear $= 2925^{kg}$

Summary of Moment and Shear	Moments	Shears
Dead load	$21,500^{cm\text{-}kg}$	520^{kg}
Live load	81,900	2925
For per meter strip	$103,400^{cm\text{-}kg}$	3445^{kg}
For 30^{cm} strip	$31,020^{cm\text{-}kg}$	$1,030^{kg}$

checked by End Section of intermediate span. Actual stresses as showing below

$\sigma_b = \frac{31020 \times 5.1}{4740} = 33.4\ kg/\square^{cm}$, $\sigma_e = 15 \times 33.4 \times \frac{7.4}{5.1} = 730\ kg/\square^{cm}$

$\tau_o = \frac{1030 \times 443}{30 \times 4740} = 3.2\ kg/\square^{cm}$, $\tau_h = \frac{1030 \times 443}{3 \times 4.08 \times 4740} = 7.9\ kg/\square^{cm}$

2. Stringers

Span length 5 meters spaced 1.5 meters

Neglect continuity of slabs and take reaction as simple beam

Dead load

Pavement & slab $460^{kg} \times 1.5 = 690^{kg}/m.$

Stringer assumed $= 50$

$740^{kg}/m.$

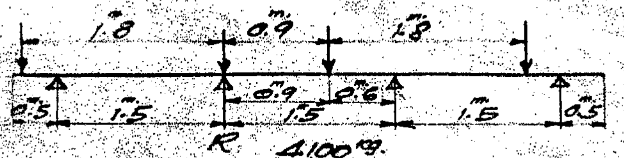
Moment $= \frac{1}{8} \times 740 \times 5^2 = 2,313^{m\text{-}kg}$

Shear $= \frac{1}{2} \times 740 \times 5 = 1,850^{kg}$

Live load

Motor truck loadings. distribution of wheel concentration on stringer assumed 0.6^{m} width

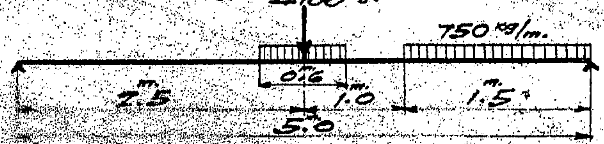
Spacing of wheel concentration as shown on sketch below and unoccupied space of roadway to be filled by Uniform load of 500^{kg}/sq. meter



R;- Rear wheel $2,925^{kg} \times 1.4 = 4,100^{kg}$

Front wheel $975^{kg} \times 1.4 = 1,370^{kg}$

Unifor load $500^{kg} \times 1.5 = 750^{kg}/m.$



For Moment

Moment Wheel concentration $\frac{1}{4} \times 4100 \times 5 - \frac{1}{8} \times 4100 = 4407^{m\text{-}kg}$

Uniform load $\frac{1}{4} \times 750 \times 1.5^2 = 423$

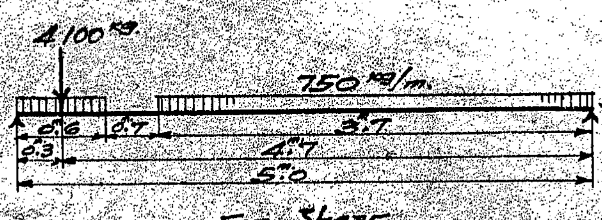
$4,830^{m\text{-}kg}$

Equivalent uniform load $= 4,830 \times 8 \div 5^2 = 1550^{kg}$ (p)

Shear Wheel concentration $4100 \times \frac{4.7}{5.0} = 3,854^{kg}$

Uniform load $\frac{1}{5} \times 750 \times \frac{1.5^2}{2} = 169$

$4,023^{kg}$



For Shear

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Summary of Moment and Shear

	Moments	Shears
Dead load	231,300 cm-kg.	1,850 kg.
Live load	483,000	~~4,020~~ 4,884
Summary	714,300 cm-kg.	~~5,870~~ kg. 6,464

Equivalent uniform load = 7,143 × 8 ÷ 5² = 2,290 kg. (q)

Section modulus required = 714,300 ÷ 1200 = 595.25 cm³.

Use 300 × 150 @ 48.34 kg. I-Beam (日本標準規格型). Section modulus = 633.20 cm³

Deflection of Beam

$$\frac{\delta}{l} = \frac{5}{24}\left(\frac{\sigma}{E}\right)\left(\frac{p}{q}\right)\left(\frac{l}{h}\right) = \frac{5}{24}\times\frac{1,200}{2,100,000}\times\frac{1,550}{2,290}\times\frac{5}{0.3} = \frac{1}{745}$$

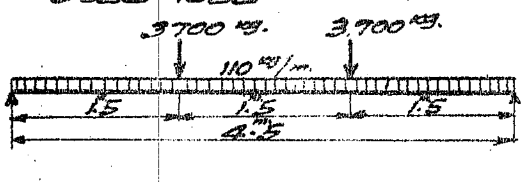
Stringers span length 4.892 m., 4.825 m., 4.700 m. and 4.525 m.

Used section same as in span length 5.000 m.

3. Cross Beam

Span length = 4.5 m. c. to c. Main Girders, Spacing assumed = 5.0 m. c. to c. Cross Beams

Dead load



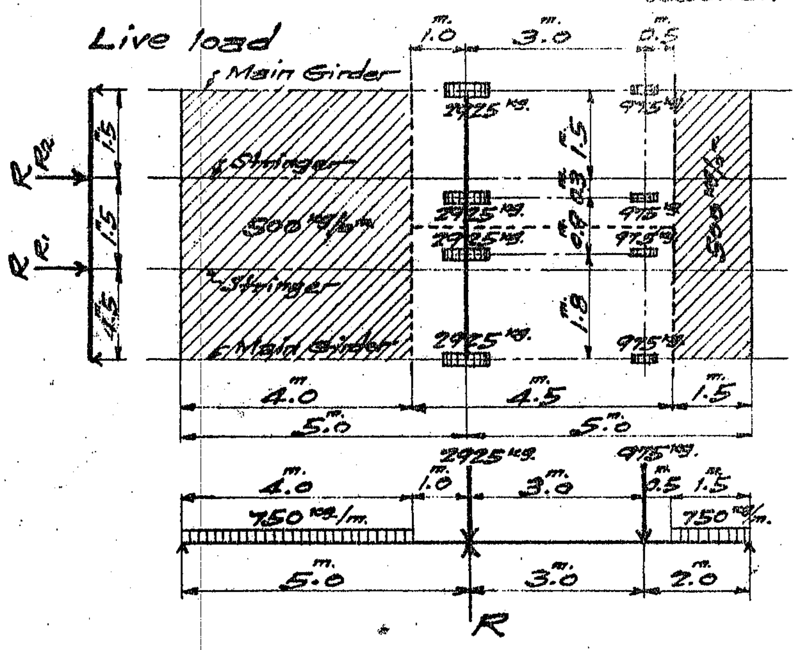
Concentrated load due to Stringers reaction = 740 kg. × 5.0 m. = 3,700 kg.

Cross beam its own weight assumed = 110 kg/m.

Moment = 3,700 × 1.5 m. + ⅛ × 110 × 4.5² = 5,828 m-kg.

Reaction or Shear = 3,700 + 2.25 × 110 = 3,948 kg.

Live load



R :-

Wheel concentration = 2,925 + 975 × 0.4 = 3,315 kg.

Uniform load = $\frac{750}{2\times5}$ × (4.0² + 1.5²) = 1,369

4,684 kg.

Moment = 4,684 × 1.5 m. = 7,026 m-kg.

Shear = 4,684 kg.

For other Motor truck arrangement

R_1 = 3,315 × 1.4 + 1,369 = 6,010 kg.

R_2 = 3,315 × 0.8 + 1,369 = 4,021 kg.

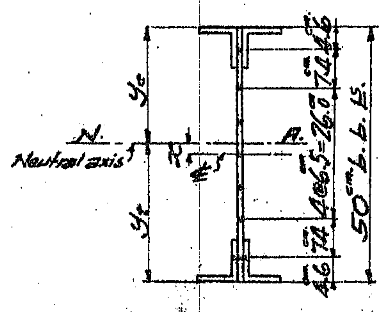
Moment = 5,347 × 1.5 = 8,021 m-kg.

Reaction or Shear = $\frac{1}{4.5}$ × (6,010 × 3.0 + 4,021 × 1.5) = 5,347 kg.

Summary of Maximum moment & Shear

	Moments	Shears
Dead load	582,800 cm-kg.	3,948 kg.
Live load	802,100	5,347
Summary	1,384,900 cm-kg.	9,295 kg.

Check of the section   used 19 mm. diameter rivets



Sectional Area

2-Flg. L. 80 mm. × 80 mm. × 9 mm.	= 27.18 □cm.		= 27.18 □cm.
1-Web Pl. 500 × 8	= 40.00	− 7 × 0.8 cm. × 2.2 cm.	= 27.68
2-Flg. L. 80 × 80 × 9	= 27.18	− 2 × 0.9 cm. × 2.2 cm.	= 23.22
	94.36 □cm. gross		78.08 □cm. net

Eccentricity x = 1.03 cm., y_t = 26.03 cm., y_c = 23.97 cm.

Moment of inertia about the neutral axis I_{N-A} = 32,770 cm⁴

Actual fibe Stresses   Allowable fiber stresses

For Tension $= \frac{1384900\times26.03}{32770} = 1,100$ kg/□cm.   1200 kg/□cm.

For Compre. $= \frac{1384900\times23.97}{32770} = 1,013$ kg/□cm. $1200\times\left(1-0.012\times\frac{1500}{168}\right) = 1,071$ kg/□cm.

Connection rivet of Cross beam and Main Girder

Shearing value of 19 mm. φ one field rivet = 2,130 kg.

Req'd no. of rivets = 9,295 ÷ 2130 = Say 5

Rivet connection of Web and Flange

$P = \frac{H \cdot I}{S \cdot Q} = \frac{2580 \times 32770}{9295 \times 590} = 15.4^{cm.}$ Used rivet spacing $8.0^{cm.}$

Where P = Rivet pitch in cm.

H = Horizontal force for one rivet in kg. $= 1700^{kg.} \times 1.9^{cm.} \times 0.8^{cm.} = 2{,}580^{kg.}$

I = Moment of inertia about the neutral axis in $cm^4 = 32{,}770^{cm^4}$

S = Vertical shear in kg. = 9,295 kg.

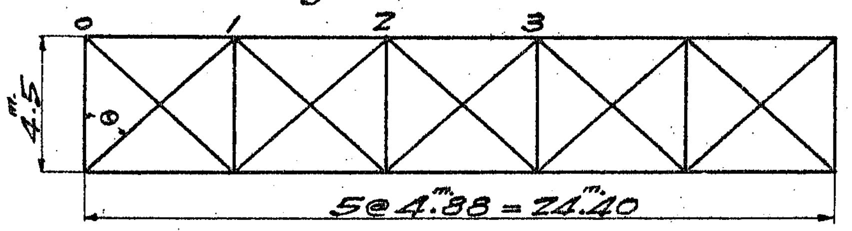
Q = Geometrical moment of one flange in $cm^3 = 27.18^{cm^2} \times (22.72^{cm.} - 1.03^{cm.}) = 590^{cm^3}$

Shearing of Web Plate

Unit Shear $= \frac{9.295 \times 590}{0.8 \times 32770} = 210$ kg/□cm.

4. Lateral Bracing for Main Girders

Lateral bracing assumed as shown in sketch below and diagonals act in tension and compression



Length of Diagonals = 6.64 m.

Assumed supported length of diagonals c. to c. rivets = 588 cm. or 277 cm.

Sec θ = 1.476 , tan θ = 1.084

Assumed Moving load = 400 kg/m + 200 kg/m = 600 kg/m.

Panel concentration = W = 600 × 4.88 = 2,928 kg.

Stresses as follows

	Panel	0 ~ 1	1 ~ 2	2 ~ 3
Diagonals	Coefficient = C	2.00	1.12	0.50
	Stress = $C \cdot \frac{W}{2} \cdot \sec\theta$ =	± 4322 kg.	± 2,420 kg.	± 1,080 kg.
Chords	Average coeff. for ajoining panel points C =	1.00	2.50	3.00
	Stress = $C \cdot W \cdot \tan\theta$ =	± 3 174 kg.	± 7,935 kg.	± 9,522 kg.

Stress in End cross beam $= \frac{1}{2} \times 24.4^{m.} \times 600^{kg} = -7{,}320^{kg.}$

Check of the section (This section used to all Panels)

Used 1-L 125 mm × 90 mm × 10 mm = 20.50 □cm. gross − 2.5 = 18.00 □cm. net area

Radii of gyration $r_x = 3.96^{cm.}$, $r_y = 2.07^{cm.}$

Req'd length $150r = 5.94^{m.}$ or 3.10 actual length = 5.88 m. or 2.77

Actual Stresses — Allowable stresses

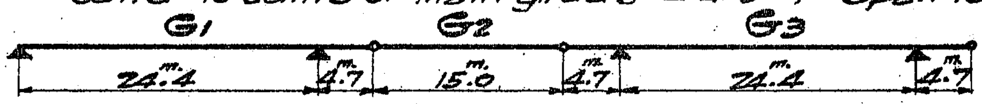
For tension $4{,}322 \div 18.00 = 240$ kg/□cm. — 1200 kg/□cm.

For compression $4{,}322 \div 20.50 = 210$ kg/□cm. — $1500 \times (1 - 0.0055 \times \frac{588}{3.96}) = 270$ kg/□cm.

Details:- 3-22 mm dia. rivets on one angle and used 10 m.m. thick gusset plate

5. Main Girder

Center to centre of Main girders = 4.5 m. , Span length as shows in sketch below



Dead load : Assuming uniformly distributed to main girders

Pavement	450 kg./meter of bridge	
Slab	2 500	
Handrail	300	
		3 250 kg/m. of bridge
Main girder	820	
Stringers	113	
Cross beam	96	
Laterals	50	
Shoes	21	
		1 100

4 350 kg/m. ÷ 2 = 2,175 kg./m. of one side girder

Call this 2,200 kg/meter of one girder

Live load ;

Live load on slab considering direct action to Main girder

Impact of Wheel concentration and load arragement as follows

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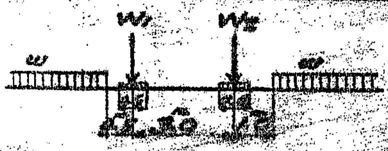


2-Wheel concentration with impact Rear W_2 Front W_1

For Girders G1 & G3 $2{,}250 \times 1.237 \times 2 = 5580^{kg}$ $750 \times 1.237 \times 2 = 1860^{kg}$

For Girder G2 $2{,}250 \times 1.267 \times 2 = 5700^{kg}$ $750 \times 1.267 \times 2 = 1900^{kg}$

Where l for impact coefficient $l = 24.^m4$ for G1 & G3, $l = 15.^m0$ for G2

Uniform live load $500\ ^{kg}/_{\square^m}$ $w = \frac{1}{2} \times 5.^m5 \times 500 = 1{,}375\ ^{kg}/$meter of one girder

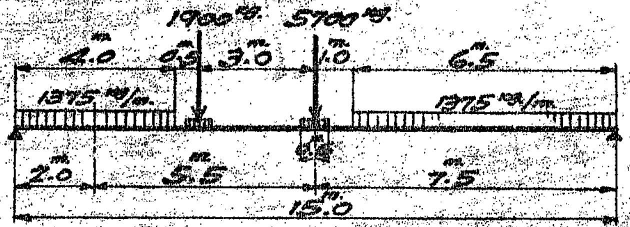
Distribution of wheel concentration assumed similar for slab $= 0.^m6$ width

Moment, Shear and Reaction due to dead and live load

For G2 Simple supported span length 15 Meter c. to c. bearings

Dead load moment $= 0.125 \times 2200 \times 15.0^2 \times 100 = 6{,}187{,}500^{cm\text{-}kg}$

Dead load reaction or shear $= 0.5 \times 2200 \times 15 = 16{,}500^{kg}$



Live load moment at centre of span

concentration	$\frac{1}{4} \times 5700 \times 15.0$	$= +21{,}375^{m\text{-}kg}$
do.	$\frac{1}{2} \times 1900 \times 10.5$	$= +9{,}975$
Uniform load	$\frac{1}{2} \times 1375 \times (3.25 \times 6.5 + 4.0 \times 13.0)$	$= +50{,}273$
less	1900×3.0	$= -5{,}700$
〃	$\frac{1}{8} \times 5700 \times 0.6$	$= -428$
〃	$1375 \times 4.0 \times 5.5$	$= -30{,}250$
		$45{,}245^{m\text{-}kg}$
		$4{,}524{,}500^{cm\text{-}kg}$

For Moment

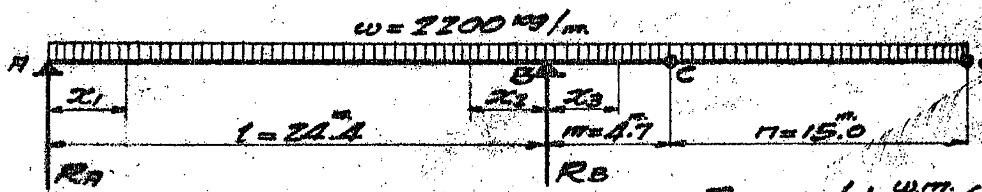
Live load Reaction or Shear, Rear wheel W_2 on Support

$= 5700^{kg} + \frac{1}{15} \times 1375 \times 14.^m0 \times 7.^m0 = 14{,}680^{kg}$

Summary of Maximum moment and shear	Moments $^{cm\text{-}kg}$	Shears ^{kg}
Dead load	6,187,500	16,500
Live load	4,524,500	14,680
Summary	$10{,}712{,}000^{cm\text{-}kg}$	$31{,}180^{kg}$

For G1

Dead load uniformly $2200\ ^{kg}/$meter



$R_A = \frac{wl}{2} - \frac{w.m}{2l}(n+m)$

$= \frac{2200 \times 24.4}{2} - \frac{2200 \times 4.7}{2 \times 24.4} \times (4.7 + 15.0)$

$= 26{,}840^{kg} - 4{,}180^{kg} = 22{,}660^{kg}$

$R_B = wl + \frac{w.m}{2.l}(n+m) = 53680^{kg} + 4180^{kg} = 57.860^{kg}$

Maximum Shears

Between A-B $V_{max} = R_A = 22{,}660^{kg}$

Between B-A $V_{max} = R_B - \frac{1}{2}wl = 57{,}860 - 26{,}840 = 31{,}020^{kg}$

Between B-C $V_{max} = \frac{1}{2}w.l = 26{,}840^{kg}$

Moment at any points and these maximum

Between A-B $M_x = R_A x_1 - \frac{1}{2} w x_1^2$

Positive $M_{max} = \frac{R_A^2}{2w}$ occurs when $x_1 = \frac{R_A}{w}$

$= 11{,}676{,}000^{cm\text{-}kg}$ $x_1 = 10.^m30266$

Negative $M_{max} = \frac{1}{2}w.m.(n+m)$ occurs when $x_1 = l$

$= 1100 \times 4.7 \times 19.7 = 101{,}849^{m\text{-}kg} = 10{,}184{,}900^{cm\text{-}kg}$ at B

Point of contraflexure $x_0 = \frac{2R_A}{w} = 20.^m6053$ from A

Between B-C $M_x = \frac{w}{2}(m - x_3)\{(m - x_3) + n\}$

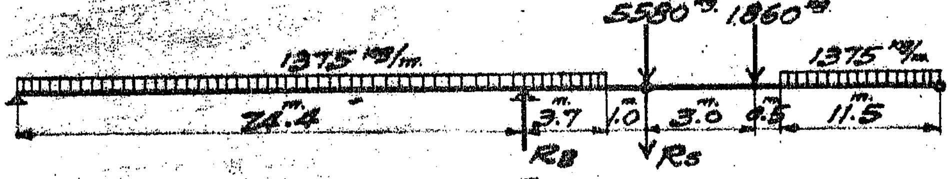
Negative $M_{max} = \frac{w}{2} m.(m+n) = 10{,}184{,}900^{cm\text{-}kg}$ at B

Live load

Maximum Reaction

For R_A.

$R_A = 5580 + \frac{1}{24.4} \times 1375 \times \frac{23.4^2}{2} = 21{,}010^{kg}$



For R_B

$R_S = 5580 + \frac{1}{15} \times (1860 \times 12 + \frac{1}{2} \times 1375 \times 11.5^2) = 13130$

R_B: $1375 \times (12.2 + 3.7) + 13{,}130 = 34{,}990^{kg}$

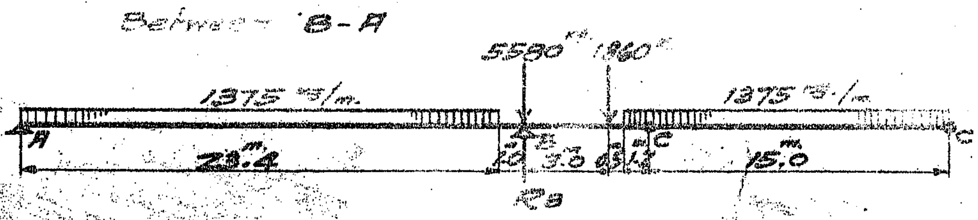
uplift $\frac{1}{24.4} \times (1375 \times 3.7 \times 1.85 + 13{,}130 \times 4.7) = \underline{2{,}920}$

$37{,}910^{kg}$

Maximum Shear

Between A-B $V_{max.} = R_A = 21,010^{kg.}$

Between B-A

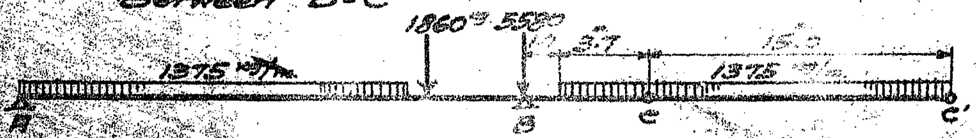


$R_A = 21,010^{kg}$

$\frac{1}{23.4}\times(1375\times7.5\times4.7+1375\times12\times2.1+1860\times3) = 2,490$

$V_{max.} = 23,500^{kg}$

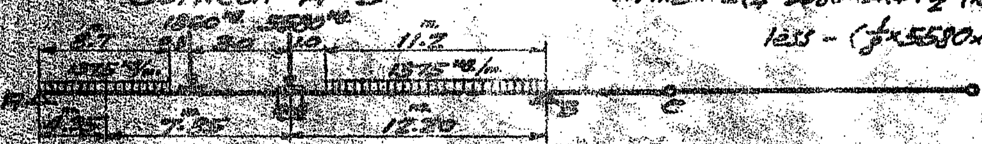
Between B-C



$V_{max.} = 5,580+1375\times(7.5+3.7) = 20,570^{kg.}$

Moment at any point and these maximum

Between A-B

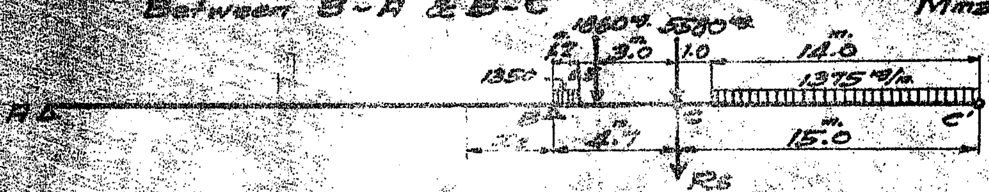


$M_{max} = [\frac{1}{2}\times5580\cdot21.4+\frac{1}{2}\times\{1860\times15.2+1375\times(11.2\times5.6+8.7\times20.05)\}]\times100 = +21,121,800^{cm\text{-}kg}$

less $-(\frac{1}{2}\times5580\times0.6+1860\times3+1375\times8.7\times7.85)\times100 = -9,990,400$

Positive $M_{max.}$ $11,131,400^{cm\text{-}kg}$

Between B-A & B-C $M_{max.}$



$R_S = 5580+\frac{1}{15}\times7\times14\times1375 = 14,560^{kg.}$

$M_{max.} = (14,560\times4.7+1860\times1.7+1375\times\frac{1.7^2}{2}) = 72,584^{m\text{-}kg}$

$= 7,258,400^{cm\text{-}kg.}$ Negative

$M_{x2} = M_{max.}(1-\frac{x}{l}) = 7,258,400\times(1-\frac{x}{l})$ zero at point A.

$= 7,258,400 - 297,500\,x$ where x = unit of Meter

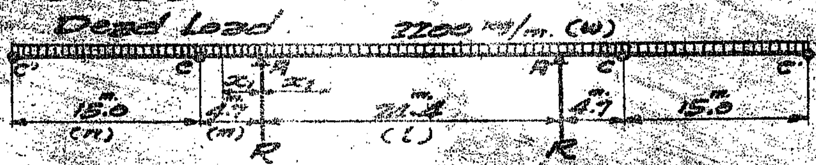
Summary of Maximum moment, shear and reaction

	Moments (cm.kg.)		Shears (kg.)			Reaction (kg.)	
	Positive	Negative	A-B	B-A	B-C	RA	RB
Dead load	11,676,000	10,184,900	22,660	31,020	26,840	22,660	57,860
Live load	11,131,400	7,258,400	21,010	23,500	20,570	21,010	37,910
Summary	22,807,400	17,443,300	43,670	54,520	47,410	43,670	95,770

Sum of Dead and Live load Maximum Positive moment $22,500,000^{cm\text{-}kg}$ from Moment diagram

For G3

Dead Load



Reaction $R = w.l = 53,680^{kg.}$

Shears

Between C-A $V_{max.} = \frac{1}{2}wl = 26,840^{kg}$ at A

Between A-A $V_{max.} = \frac{1}{2}R = 26,840^{kg}$ at A

Moment at any point and these maximum

Between C A $M_x = \frac{1}{2}w(m-x_1)\{(m-x_1)+n\}$

Negative $M_{max.} = \frac{1}{2}w\,m.(m+n)$ occurs when $x_1=0$ at A. $= 10,184,900^{cm\text{-}kg.}$

Between A-A $M_x = \frac{1}{2}w\{x_2(l-x_2)-m(m+n)\}$

Negative $M_{max.} = \frac{1}{2}w\,m\,(m+n)$ occurs when $x_2=0$ at A $= 10,184,900^{cm\text{-}kg.}$

Positive $M_{max.} = \frac{1}{2}w(\frac{l}{2}-m)^2 = \frac{1}{8}wn^2$ occurs at $x_2=\frac{l}{2}$ $= 6,187,500^{cm\text{-}kg.}$

Point of contraflexure $x_0 = m$ or $m+n$ $= 4.7^m$ or 19.7^m

Live Load

Maximum Reaction $R = 37,910^{kg.}$ same as in RB of Girder G1

Maximum Shear

Between C-A $V_{max.} = 20,570^{kg.}$ similar as shown in between B-C for girder G1

Between A-A $V_{max.} = 23,500^{kg.}$ 〃 〃 〃 〃 B-A 〃 〃

Moment at any point and these Maximum.

Between C-A & A-A $M_{max.} = 7,258,400^{cm\text{-}kg}$ Negative, as shown in girder G1

Any point at between A-A

$R_u = \frac{1375}{21.4}\times4.7\times(15.0+4.7) = 2,609^{kg}$

$M_x = 7,258,400-(2975-2,609)x\times100$

$= 7,258,400 - 36,600x$

Where x = Unit of Meter

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Between A-A Rear wheel concentration at center

Max. Positive moment $M_{max.} = 11,131,400^{cm.kg.}$ same as shown in between A-B of G-1

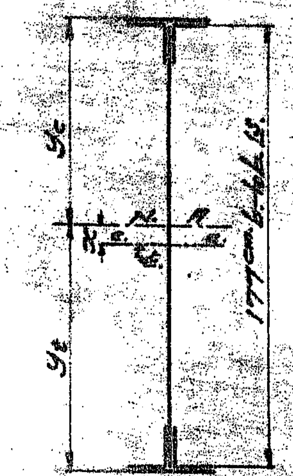
Summary of Max. Moment. Shear and Reaction

	Moments (cm. kg.) Positive	Moments (cm. kg.) Negative	Shears (kg.) C-A	Shears (kg.) A-A	Reaction (kg.)
Dead load	6,187,500	10,184,900	26,840	26,840	53,680
Live load	11,131,400	7,258,400	20,570	23,500	37,950
Summary	17,318,900	17,443,300	47,410	50,340	91,630

Where sum of Positive moment + Alternate moments $= 17,600,000^{cm.kg.}$ See moment diagram

Check of the girder section Use $22^{m}\phi$ rivet and out hole $25^{m}\phi$

For Negative moment $M_{max} = 17,443,300^{cm.kg.}$



1-Cov. Pl. 330×10 = $33.00^{\square cm.}$ = $33.00^{\square cm.}$

2-Flg. L. 150×150×11 = 63.58 = 63.58

1-Web Pl. 1760×10 = 176.00 − 20×1.0×2.5 = 126.00

2-Flg. L. 150×150×11 = 63.58 − 4×1.1×2.5 = 52.58

1-Cov. Pl. 330×10 = 33.00 − 2×1.0×2.5 = 28.00

369.16 gross 303.16 net

Eccentricity $x = 4.53$, $y_t = 94.03$, $y_c = 84.97^{cm}$

Moment of inertia $I_{c\text{-}c} = 1,$[illegible]

for neutral axis $I_{N\text{-}A} = I_{c\text{-}c} - 303.16 \times 4.53^2 = 1,651,425^{cm^4}$

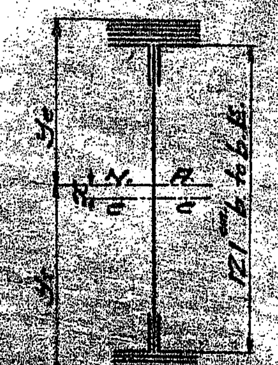
Geometrical moment of one flange about the neutral axis $Q = 7,870^{cm^3}$

Actual stresses

For compression $= \dfrac{17443300 \times 84.97}{1651204} = 800\ ^{kg}/_{\square cm}$

For tension $= \dfrac{17443300 \times 94.03}{1651204} = 990\ ^{kg}/_{\square cm}$

For Positive moment M_{max} G1 $= 22,500,000^{cm.kg}$, G3 $= 17,600,000^{cm.kg}$, G2 $= 10,712,000^{cm.kg}$



4-Cov. Pls 330×11 = $145.20^{\square cm}$ = $145.20^{\square cm}$

1-Cov. Pl. 330×10 = 33.00 = 33.00

2-Flg. L. 150×150×11 = 63.58 = 63.58

1-Web Pl. 1200×10 = 120.00 − 13×1.0×2.5 = 87.50

2-Flg. L. 150×150×11 = 63.58 − 4×1.1×2.5 = 52.58

1-Cov. Pl. 330×10 = 33.00 − 2×1.0×2.5 = 28.00

4-Cov. Pl. 330×11 = 145.20 − 8×1.1×2.5 = 123.20

603.56 gross 533.06 net

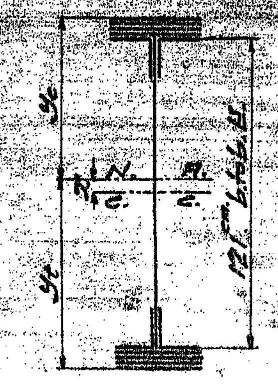
Eccentricity $x = 4.37^{cm}$, $y_t = 70.27^{cm}$, $y_c = 61.53$

Moment of inertia about the neutral axis $I_{N\text{-}A} = 1,790,237^{cm^4}$

Actual stresses for Girder G1

For tension $= \dfrac{22,500,000 \times 70.27}{1790,237} = 880\ ^{kg}/_{\square cm}$

For compression $= \dfrac{22,500,000 \times 61.53}{1790,237} = 770\ ^{kg}/_{\square cm}$



3-Cov. Pl. 330×11 = $108.90^{\square cm}$ = $108.90^{\square cm}$

1-Cov. Pl. 330×10 = 33.00 = 33.00 $x = 4.22^{cm.}$

2-Flg. L. 150×150×11 = 63.58 = 63.58 $y_t = 69.02^{cm.}$

1-Web Pl. 1200×10 = 120.00 − 13×1.0×2.5 = 87.50 $y_c = 60.58^{cm}$

2-Flg. L. 150×150×11 = 63.58 − 4×1.1×2.5 = 52.58 $I_{N\text{-}A} = 1,505,948^{cm^4}$

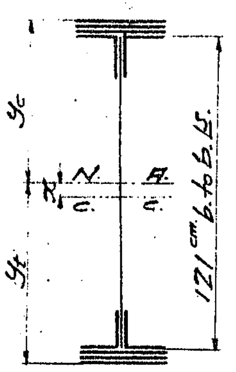
1-Cov. Pl. 330×10 = 33.00 − 2×1.0×2.5 = 28.00

3-Cov. Pl. 330×11 = 108.90 − 6×1.1×2.5 = 92.40

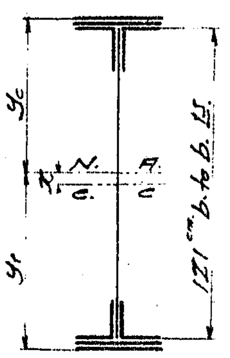
$530.96^{\square cm}$ gross $465.96^{\square cm}$ net

Actual Stresses for Girder G3

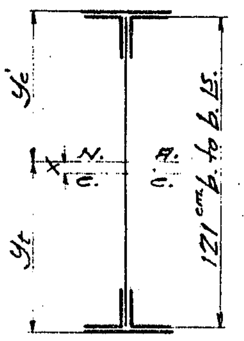
For tension $= \dfrac{17600,000 \times 69.02}{1,505,948} = 810\ ^{kg}/_{\square cm}$, For Compression $= \dfrac{17,600,000 \times 60.58}{1,505,948} = 710\ ^{kg}/_{\square cm}$



Sectional gross area = $458.36^{\square cm}$
Sectional net area = $398.86^{\square cm}$
Eccentricity $x = 4.05^{cm}$
$y_t = 67.75^{cm}$
$y_c = 59.65^{cm}$
$I_{N-A} = 1,230,942^{cm^4}$



Sectional gross area = $385.76^{\square cm}$
Sectional net area = $331.76^{\square cm}$
Eccentricity $x = 3.82^{cm}$
$y_t = 66.42^{cm}$
$y_c = 58.78^{cm}$
$I_{N-A} = 965,166^{cm^4}$



1-Cov. Pl. 330×10	= $33.00^{\square cm}$		= $33.00^{\square cm}$
2-Flg. ∟s. 150×150×11	= 63.58		= 63.58
1-Web Pl. 1200×10	= 120.00	− 13×1.0×2.5	= 87.50
2-Flg. ∟s. 150×150×11	= 63.58	− 4×1.1×2.5	= 52.58
1-Cov. Pl. 330×10	= 33.00	− 2×1.0×2.5	= 28.00
	$313.16^{\square cm}$ gross area		$264.66^{\square cm}$ net area

Eccentricity of neutral axis due to out rivet hole $x = 3.50^{cm}$
Distance from the neutral axis to extream fiber $y_t = 65.00^{cm}$, $y_c = 58.00^{cm}$

Moment of inertia.
For about the center axis $I_{c\text{-}c}$.

Cover plate	$226,981^{cm^4}$
Flange angles	372,358
Web plate	112,355
	$711,694^{cm^4}$

For about the neutral axis $I_{N\text{-}A} = I_{c\text{-}c} - \text{Net area}\cdot x^2 = 708,452^{cm^4}$
Geometrical moment of one flange about the neutral axis $Q_{N\text{-}A} = 5,260^{cm^3}$
Actual stress of Girder G2 (15meter simple span)

For tension $= \dfrac{10,712,000 \times 65.0}{708,452} = 980^{kg/\square cm}$, For compression $= \dfrac{10,712,000 \times 58.0}{708,452} = 880^{kg/\square cm}$

Moment Diagram and required Cover Plate length as shown in next sheet, and unnoted effect of moment at any point.

Deflection of each spans (only maximum)
Approximate calculation for sum of dead and live load moment as follows considered to uniformly distributed load
Formula of the deflection $\delta = \dfrac{5}{48} \cdot \dfrac{M\,l^2}{E.I}$ for beam supported at ends

For Girder G2

$$\delta = \frac{5}{48} \times \frac{10,712,000 \times 1500^2}{2,100,000 \times 708,500} = 1.68^{cm} \qquad \frac{\delta}{l} = \frac{1}{888}$$

For Girder G3

For Dead load $\delta_d = \dfrac{5}{48} \times \dfrac{6,187,500 \times 1500^2}{2,100,000 \times 1,506,000} = 0.46^{cm}$ Where $l = 15.0^{m}$ = Distance between contraflexure points of moment

For Live load $\delta_l = \dfrac{5}{48} \times \dfrac{11,131,400 \times 2440^2}{2,100,000 \times 1,506,000} = \dfrac{2.18}{2.64^{cm}}$ $\qquad \dfrac{\delta}{l} = \dfrac{1}{924}$

For Girder G1 Where $l = 20.61^{m}$ = Distance between contraflexure points of bending moment in 24.4^{m} span

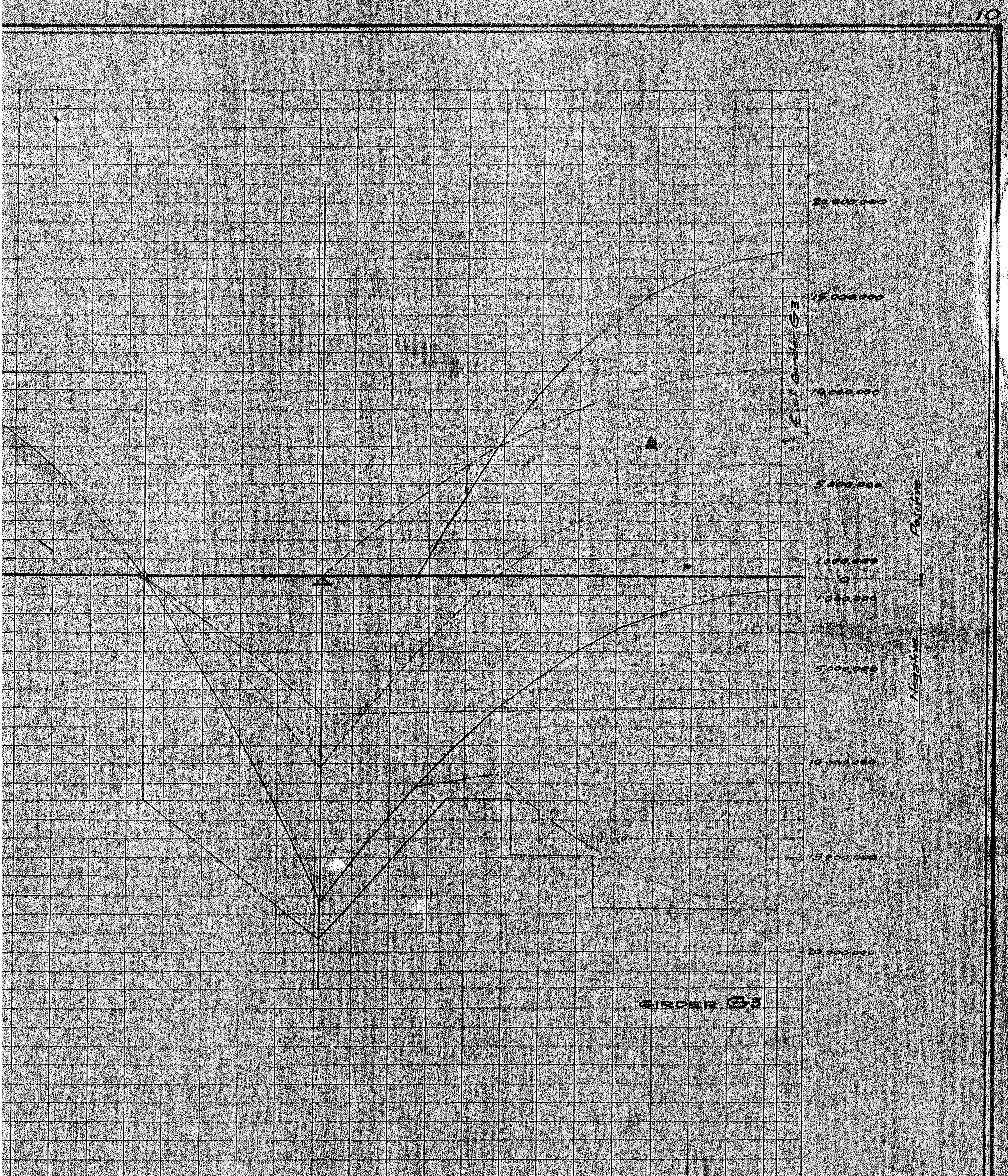
For Dead load $\delta_d = \dfrac{5}{48} \times \dfrac{11,676,000 \times 2061^2}{2,100,000 \times 1,790,000} = 1.37^{cm}$

For Live load $\delta_l = \dfrac{5}{48} \times \dfrac{11,131,400 \times 2440^2}{2,100,000 \times 1,790,000} = \dfrac{1.84}{3.21^{cm}}$ $\qquad \dfrac{\delta}{l} = \dfrac{1}{760}$

For Assumed Simple span 24.4^{m}
Dead load moment $= 0.125 \times 2200 \times 24.4^2 \times 100 = 16,372,400^{cm\text{-}kg}$

For G3 $\quad \delta = \dfrac{5}{48} \times \dfrac{16,372,400 \times 2440^2}{2,100,000 \times 1,506,000} = 3.22^{cm}$

For G1 $\quad \delta = \dfrac{5}{48} \times \dfrac{16,372,400 \times 2440^2}{2,100,000 \times 1,790,000} = 2.71^{cm}$



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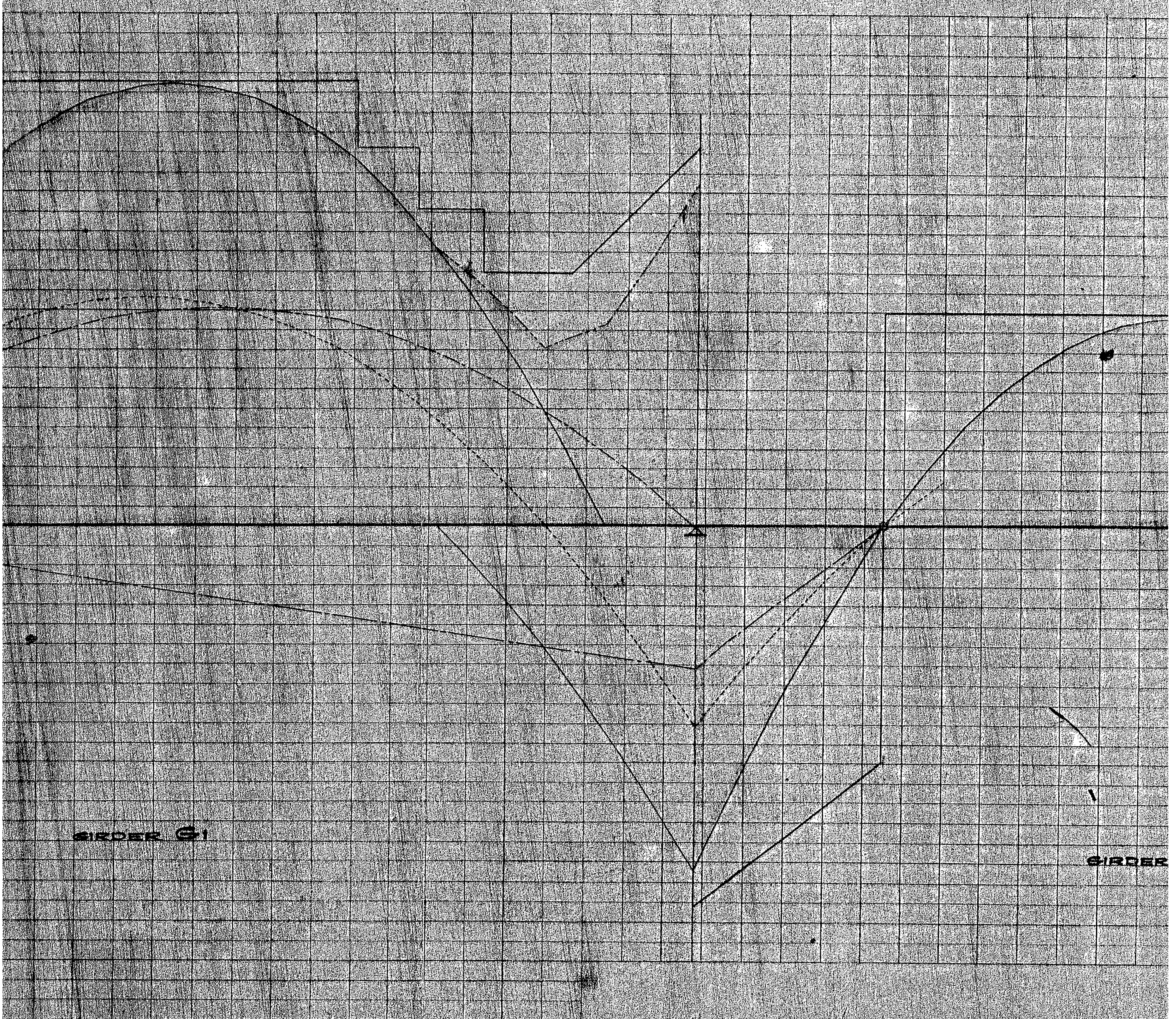
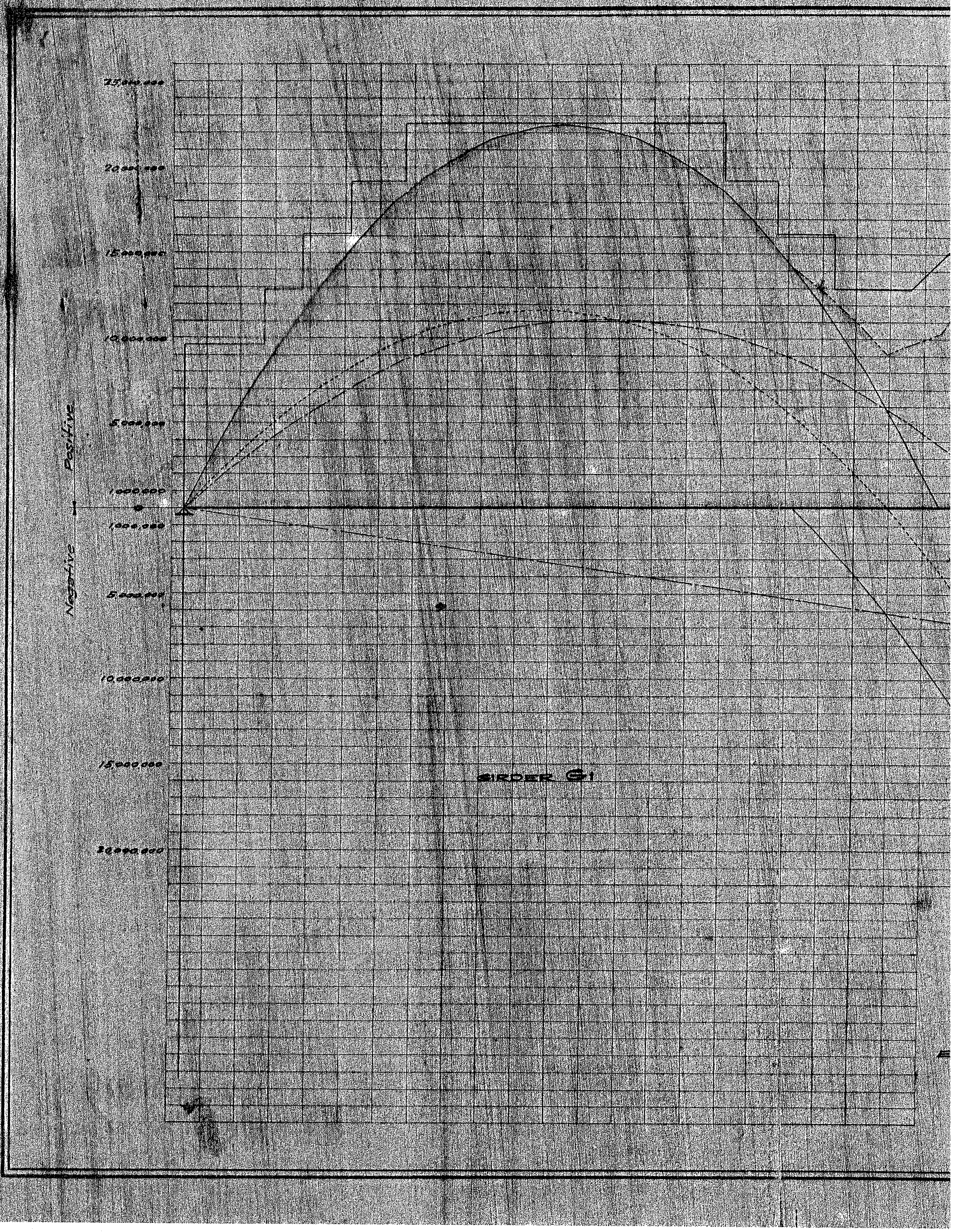


MOMENT DIAGRAM
&
LENGTH OF COVER PLATE

SCALE { VERT. 1 CM. = 2,000,000 CM-KG / HORZ. 1 CM. = 1 METER

EXPLANATION OF CURVE LINES
- Dead load moment
- Live load moment
- Sum of dead and live load moment
- Larger alternate moment plus 50% of smaller alter

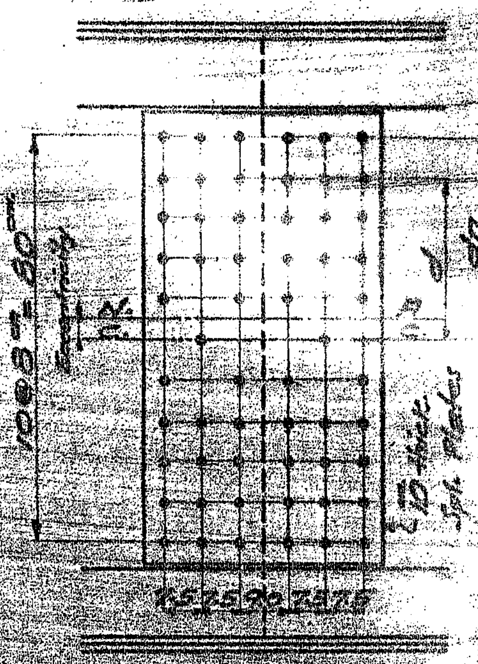

25,000,000
20,000,000
15,000,000
10,000,000
5,000,000
1,000,000
0
1,000,000
5,000,000
10,000,000
15,000,000
20,000,000
Positive
Negative
GIRDER G1

For Cantilever arm Formula $\delta = \frac{1}{4} \frac{M m^2}{E.I}$ where m = cantilever arm length

For Dead and live load $\delta = \frac{1}{4} \frac{17,443,300 \times 470^2}{[illegible] \times 165,14[illegible]} = 0.78^{cm}$ $\frac{\delta}{l} = \frac{1}{1600}$

Web Splice (Used rivets $22^{mm}\phi$)

Formula $r = \sqrt{\left(\frac{V}{n}\right)^2 + \left(\frac{I_w}{I} \cdot \frac{M \cdot d_n}{\Sigma d^2}\right)^2}$



Where r = [illegible] Strength in outermost rivet in kg.
V = [illegible] shear carried by web in kg.
M = Bending moment at splice part in cm-kg.
n = No. of rivets on one side of the splice
I_w = Moment of inertia of net section of the web plate cm^4
I = Moment of inertia of the net section in cm^4
d_n = Distance from neutral axis to outermost rivet in cm.
d = Distance from neutral axis to intermediate rivet in cm.

$n = 29$, $I_w = 112,355^{cm^4}$, $d_n = 40^{cm}$,

$\Sigma d^2 = 2 \cdot ((40^2 + 32^2 + 24^2 + 16^2) \times 3 + 8^2 \cdot 2) = 20,992^{cm^2}$

$r = \sqrt{V^2 \, 0.0012 + \left(\frac{M}{I}\right)^2 45,835}$

Efect of web splice at each points as shown in table below

Parts		M (cm-kg)	V (kg)	I (cm⁴)	M/I	0.0012·V²	45835·(M/I)²	r (kg)
G1	9.2 m	21,800,000	14,000	1,790,237	12.18	235,200	6,799,700	2,650
〃	19.2	11,700,000	33,000	965,166	12.12	1,306,800	6,732,900	2,840
G2	Centre	10,712,000	1,800	708,450	15.12	3,900	10,478,500	3,240
G3	5.2 m	11,400,000	31,000	1,230,942	9.26	1,153,200	3,930,200	2,250
〃	Center	17,600,000	4,300	1,505,948	11.[illegible]	22,200	6,263,600	2,510

Allowable bearing strength of one rivet for 10 mm. thick. Web Plate = 3,300 kg. for field rivet.

Rivet Pitch for connection of Web and Flange at span of End

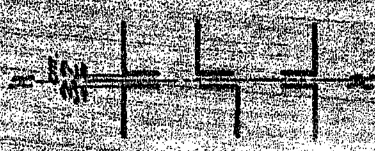
Working strength of one 22 mm dia. shop rivet = 3,740 kg.

For girder G1 :- at A Req'd Pitch = 11.5 cm and at B Req'd Pitch = 14.4 cm.

Use 8.5 cm Pitch at End in 1.5 distance. Girder G2 and G3 same as in G1

Stiffeners — used to similar for All spans.

End stiffeners considered column action, supported length of member assumed = $\frac{1}{2} \times 156 = 78^{cm}$



Maximum Reaction for all supports = 95,770 kg.

Section 6-L 125×90×10 = 123.0 cm^2 gross area

least radius of gyration about axis x-x = 6.8 cm.

Actual stress = 95,770 ÷ 123.0 = 780 kg/cm^2

Allowable compressive stress = $1500 \times \left(1 - 0.0055 \times \frac{78}{6.8}\right) = 1400 \ kg/cm^2 \geqq 1000 \ kg/cm^2$

Distance of intermediate stiffeners. Checked by Max. shear in part of shallow girders

Formula $d = 0.35\,t\left(950 - \frac{S.Q}{t.I}\right) = 0.35 \times 1.0 \times \left(950 - \frac{43,670 \times 5,260}{1.0 \times 708,450}\right) = 201.5^{cm}$ Req'd max. limit

Caststeel Shoes on Pier and Abutment (similar shoes for all supports)

Maximum Reaction = 95,770 kg.

Actual Bearing stress = 95,770 ÷ 56.46 = 37 kg/cm^2

Moment at center of base

$M = \frac{1}{8} \times 56 \times 95770 = 670,400$ cm-kg.

Section modulus about the axis y-y

$Q = \frac{1}{6} \times 46 \times 12^2 = 1,100^{cm^3}$

Actual fiber stress = 670,400 ÷ 1,100 = 610 kg/cm^2

Actual bearing of Roller surface = 95,770 ÷ 46 = 2,080 kg/cm Allow. = 45d = 13,500 kg.

Bearing for Girder G2

Maximum Reaction = 31,180 kg.

For Expansion end. used metal Bronze

Sliding surface dia. = 300 cm. (d) Rength req'd = 31,180 ÷ 45d = 2.4 cm. use 7.44 cm.

For Fixed end. Pin connection

Used diameter of Pin = 9.44 cm. sectional area = 70.0 □cm.

Resisting shearing strength = 900 × 70.0 = 63,000 kg.

Resisting bearing Value = 1800 × 9.44 . t = 17,000 . t where t = thick. of Plates

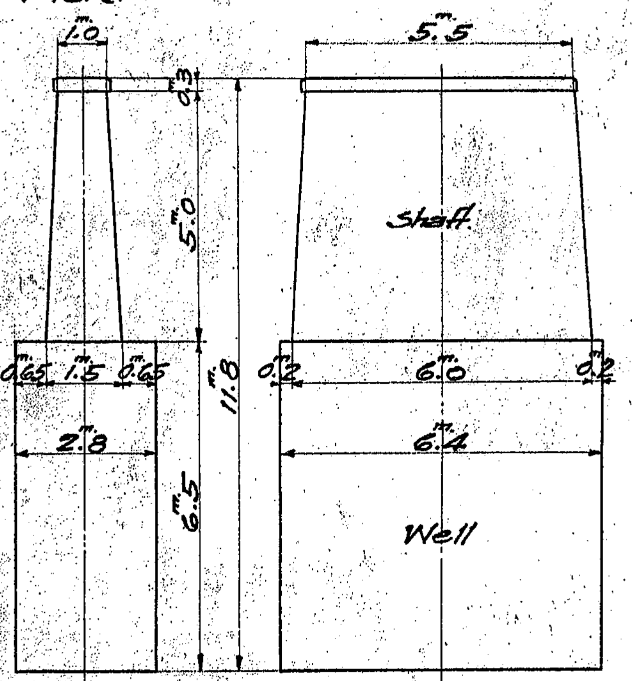
Resisting bending moment = $1800 \times 0.098 \times 9.44^3$ = 148,400 cm-kg.

for bearing used t = 2 - 1.1 cm. thick. Reinforcing Plates and 1.0 thick. Web Plate = 3.2 cm.

Actual bearing stress = 31,180 ÷ 9.44 × 3.2 = 1030 kg/□cm.

Design of Substructures

1. Piers



Dead load of Pier

Coping	1.86 cbm @ 2200 kg.	= 4,100 kg.		
Shaft	34.34 @ 2200	= 75,600		
			79,700 kg.	
Well body	51.53 m³ @ 2400 kg.	= 123,700		
〃 fill.	31.16 @ 2200	= 68,600		
〃 〃	24.13 @ 2100	= 50,700		
			243,000	
Summary of Weight of Pier			322,700 kg.	
Total dead and live load of superstructure			95,800 kg.	
			418,500 kg.	

Unit compressive stresses

For Base of Shaft area = 8.52 □m load = 175,500 kg.

Unit Stress = 175,500 ÷ 8.52 = 20,600 kg/□m. 2.06 kg/□cm

For Bottom of Well area = 17.54 □m.

Unit stress = 418,500 ÷ 17.54 = 23,860 kg/□m. 2.386 kg./□cm.

Unit compressive stresses due to excluded live load (37,900 kg.)

For bottom of shaft = 137,600 ÷ 8.52 = 16,200 kg/□m. 1.62 kg/□cm.

For bottom of Well = 380,600 ÷ 17.54 = 21,700 kg/□m. 2.17 kg/□cm.

Earthquake effect of structure

Assumed acceleration of earthquake motion = 2000 mm/sec²

Gravity acceleration = 9,800 mm/sec² $\frac{2000}{9800} = 0.204$

For bottom of Shaft

Moments at bottom of shaft due to dead load

Superstructure	57,900 kg × 7.30 m	= 422,700 m-kg.	
Coping	4,100 × 5.15	= 21,100	
Shaft	75,600 × 2.50	= 189,000	
		632,800	× 0.204 = 129,100 m-kg.

Section modulus of section at bottom of shaft. = $\frac{1}{6} \times 4.5 \times 1.5^2 + 0.098 \times 1.5^3 = 2.01$ m³

Maximum fiber stress = 129,100 ÷ 2.01 = 64,200 kg/□m 6.42 kg/□cm.

For bottom of Well

Moment ;- at bottom of Well.

Superstructure	57,900 kg. × 13.80 m.	= 799,000 m-kg.	
Coping	4,100 × 11.65	= 47,800	
Shaft	75,600 × 9.00	= 680,400	
Well	243,000 × 3.25	= 789,800	
		2,317,000	× 0.204 = 472,700 m-kg.

Section modulus of section at bottom of well = $\frac{1}{6} \times 3.6 \times 2.96^2 + 0.098 \times 2.96^3 = 7.80$ m³

Maximum fiber stress = 472,700 ÷ 7.80 = 60,600 kg/□m. 6.06 kg/□cm.

Sum of compressive stress

	Bottom of Shaft	Bottom of Well
Direct compressive stress due to dead load	2.06 kg/□cm	2.39 kg/□cm.
Comp. fiber stress due to earthquak motion	6.42	6.06
	8.48 kg/□cm.	8.45 kg/□cm.

Reinforced by 16^{mm} dia. steel bars arranged to 30^{cm} spacings for tensile strength due to horizontal forces (Earthquake motion, Wind Pressure and Current Pressure) and construction joint.

Checked of the well section

Used section as shown in sketch below and steel reinforcements $16^{mm}\phi$ bars 15^{cm} spacing

Considered 30^{cm} strip

Compression side steel area $2 - 1.6^{cm}\phi$ bars $= 4.02^{□cm}$

Tension side steel area $2 - 1.6^{cm}\phi$ bars $= 4.02$

Total steel area for 30^{cm} strip $8.04^{□cm}$

Distance $x = 10.7^{cm}$, Moment of inertia $J = 85,150^{cm^4}$, Geometrical moment $S = 2,070^{cm^3}$

Resisting moment for compression of concrete $= 358,100^{cm\text{-}kg}$ in 30^{cm} strip.

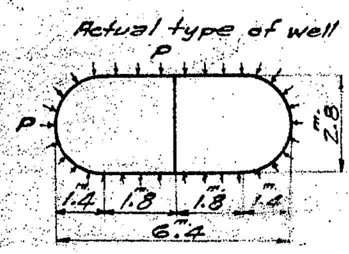
Resisting moment for tension of steel $= 198,600^{cm\text{-}kg}$ for 30^{cm} strip

$662,000^{cm\text{-}kg} = 6,620^{m\text{-}kg}$ for per one meter strip

Resisting shear $= 11,100^{kg}$ for per 30^{cm} strip. $= 37,000^{kg}$ for per meter strip

Assumed Pressure intensity at out side of well

$P.max.$ at bottom of well $= r \cdot H \cdot \mu = 2100^{kg} \times 7.5^{m} \times \frac{1}{3} = 5,250^{kg}/meter$



Now assuming try for 3.2 meter span (one half of long length of well) at both end fixed - distributed uniform load of $5,250^{kg}/m$.

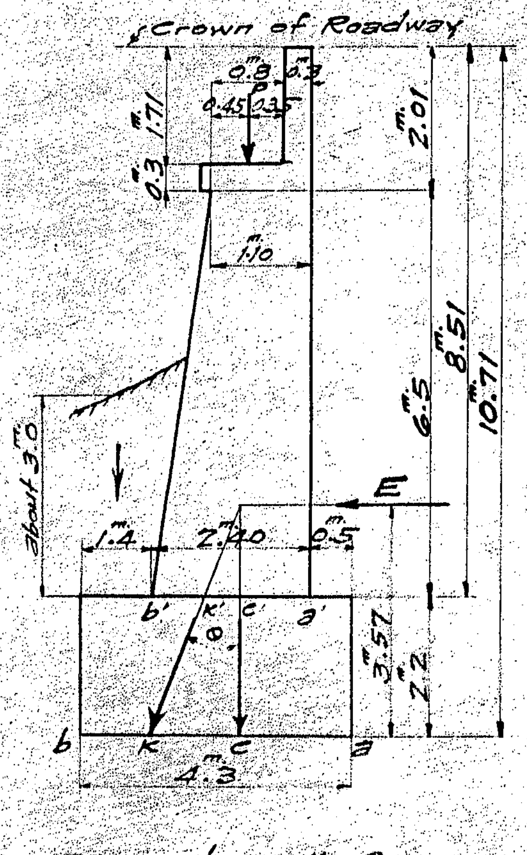
Maximum Negative moment $= 4,380^{m\text{-}kg}$.

Maximum Shear $= 8,400^{kg}$.

Actual section sufficient for these Maximum moment and shear. but moment and shear for actual calculation shall be less than assumed moment $4,380^{m\text{-}kg}$ and Shear 8400^{kg}.

In addition to Main bars arranged to $16^{mm}\phi$ longitudinal bars about 50^{cm} spacing and $6^{mm}\phi$ tie bars about 1 meter spacing

2. Abutment "A"



P = Reaction from Girder section

Dead load reaction $2 \times 22,660^{kg}$ $\div 6.0^{m} = 7,550^{kg}/m$

Live load reaction $2 \times 21,010$ $\div 6.0 = 7,000^{kg}/m$

Position of center of gravity for vertical load

	Volume	Unit wt.	Weight	Arm	Moment at a
Front Earth	$3.0^{m} \times 1.4^{m}$	@ 1600^{kg}	= 6720 kg	3.60 m	24,192 m-kg
Rear Earth	$0.5^{m} \times 8.51^{m}$	@ 1,600	= 6,808	0.25	1,702
Parapet wall	$0.3^{m} \times 1.71$	@ 2,400	= 1,231	0.65	800
Coping	0.3×1.18	@ 2400	= 850	1.09	927
Shaft	1.75×6.5	@ 2,200	= 25,025	1.42	35,536
Base	4.3×2.2	@ 2,200	= 20,812	2.15	44,746
			61,446 kg		107,903 m-kg
Dead load from girder			7,550	1.15	8,683
			68,996 kg		116,586 m-kg

Position of center of gravity $= 116,586 \div 68,996 = 1.69^{m}$ from a

Live load from girder	7,000	1.15	8,050
	75,996 kg		124,636 m-kg

Position of centre of gravity $= 124,636 \div 75,996 = 1.64^{m}$ from a

Assumed Earth Pressure $E = \frac{1}{2} \times 1,600 \times 10.71^2 \times \frac{1}{3} = 30,600^{kg}/m$ Point of action 3.57^{m}

For Sliding

Dead load only

$\tan\theta = \frac{30,600}{68,996} = 0.444 = 24°$ less than 30°

Live load included

$\tan\theta = \frac{30,600}{75,996} = 0.40$ less than 30°

For Overturn[illegible]

Dead load only	Live load included
$ac = [illegible]69^{m}$	$ac = 1.64^{m}$
$kc = 3.57 \times 0.444 = 1.59^{m}$	$kc = 3.57 \times 0.403 = 1.44^{m}$
$ak = ac + kc = 3.28^{m}$	$ak = ac + kc = 3.08^{m}$
$bk = 4.30^{m} - ak = 1.02^{m}$	$bk = 4.30 - ak = 1.22^{m}$

For Bearing

Dead load only

$\sigma = \frac{2}{3} \times \frac{68.996}{1.02} = 45,100\ kg/m^2 = 4.51\ kg/cm^2$

Live load included

$\sigma = \frac{2}{3} \times \frac{75.996}{1.22} = 41,500\ kg/m^2 = 4.15\ kg/cm^2$

Earthquake effect of structure

For base of Shaft

	Weight	Arm	Moment at bottom of shaft
Earth Pressure $\frac{1}{2} \times 1600 \times 8.51^2 \times \frac{1}{3}$	= 19,300 kg	2.84 m	54,812 m-kg
Parapet wall	= 1,231	7.66	9.430
Coping	= 850	6.65	5.653
Shaft	= 25,025	2.85	71.321
Dead load from girder	= 7,550	8.24	62.212
			203.428 × 0.204 = 41,500 m-kg

Section modulus of the section at the bottom of shaft $= \frac{1}{6} \times 2.4^{2} \times 1.0 = 0.96\ m^3$

Maximum fibe stress $= 41\,500 \div 0.96 = 43,200\ kg/m^2 = 4.32\ kg/cm^2$

For bottom of base

	Weight	Arm	Moment at bottom of base
Earth Pressure	= 30,600 kg.	3.57 m	109 242 m-kg
Parapet wall.	= 1,231	9.86	12.138
Coping	= 850	8.85	7.523
Shaft	= 25,025	5.05	126 376
Base	= 20,812	1.10	22,893
Dead load from girder	= 7.550	10.44	78.822
			356,994 × 0.204 = 72 800 m-kg.

Section modulus of the base $= \frac{1}{6} \times 4.3^{2} \times 1.0 = 3.08\ m^3$

Maximum fiber stress $= 72,800 \div 3.08 = 23,600\ kg/m^2 = 2.36\ kg/cm^2$

For bottom of Shaft (Check of direct load)

Dead load only

Weight	Moment
68,996 kg	116,586 m-kg.
− 20 812	− 44.746
48,184 kg.	71,840 m-kg.

Position of c.g. $= 71.840 \div 48.184 = 1.49^{m}$

$\tan\theta = 19.300 \div 48.184 = 0.4005 = 21°50'$

$a'c' = 1.49 - 0.5 = 0.99^{m}$

$c'k' = 2.84 \times 0.4005 = 1.14^{m}$

$b'k' = 2.40 - (1.14 + 0.99) = 0.27^{m}$

$\sigma = \frac{2}{3} \times \frac{48.184}{0.27} = 135,700\ kg/m^2 = 13.57\ kg/cm^2$

Live load included

75,996 kg.	124.636 m-kg.
− 20.812	− 44.746
55,184 kg.	79 890 m-kg.

Position of c.of g. $= 79.890 \div 55,184 = 1.45^{m}$

$\tan\theta = 19300 \div 55,184 = 0.3497 = 19°20'$

$a'c' = 1.45 - 0.5 = 0.95^{m}$

$c'k' = 2.84 \times 0.3497 = 0.99^{m}$

$b'k' = 2.40 - (0.99 + 0.95) = 0.46^{m}$

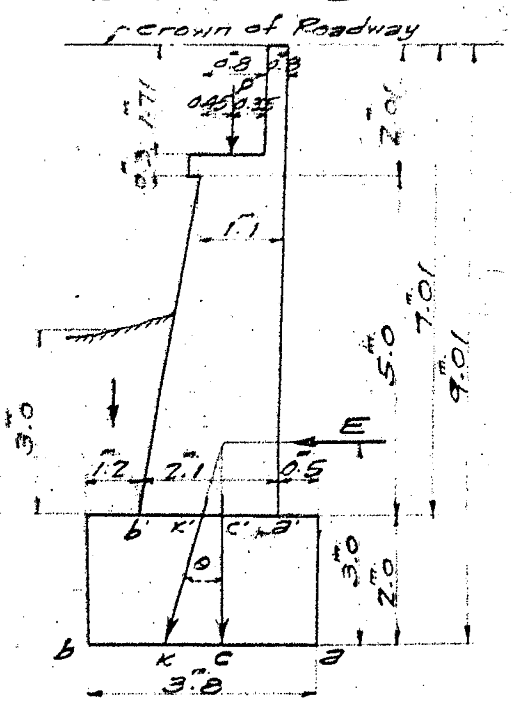
$\sigma = \frac{2}{3} \times \frac{55.184}{0.46} = 80,000\ kg/m^2 = 8.00\ kg/cm^2$

Summary of Bearing stresses (Combined stresses)

For bottom of shaft $= 13.57\ kg/cm^2 + 4.32\ kg/cm^2 = 17.89\ kg/cm^2$

For bottom of base $= 4.51\ kg/cm^2 + 2.36\ kg/cm^2 = 6.87\ kg/cm^2$

3 Abutment "B"



Cheked for bottom of Base

Position of centre of gravity for vertical loads

	Volume	Unit wt.	Weight	Arm.	Moment at a
Fron Earth	3.0 m × 1.20 m	@ 1,600 kg	= 5,760 kg	3.20 m	18,432 m-kg
Rear Earth	0.5 m × 7.01	@ 1,600	= 5,608	0.25	1,402
Parapet wall			= 1,231	0.65	800
Coping			= 850	1.09	927
Shaft	1.6 m × 5.00 m	@ 2,200	= 17,600	1.33	23,408
Base	2.0 m × 3.80	@ 2,200	= 16,720	1.90	31,768
			47,769 kg		76,737 m-kg
Dead load from girder			7,550	1.15	8,683
			55,319 kg		85,420 m-kg

Position of center of gravity = 85,420 ÷ 55,319 = 1.54 m form a.

Live load from girder	7,000	1.15	8,050
	62,319 kg		93,470 m-kg

Position of center of gravity = 93,470 ÷ 62,319 = 1.50 m from a.

Assumed Earth Pressure $E = \frac{1}{2} \times 1{,}600 \times 9.01^2 \times \frac{1}{3} = 21{,}650$ kg/m. Point of action $y = 3.00$ m

For Sliding

Dead load only

$\tan\theta = \frac{21{,}650}{55{,}319} = 0.391 = 21°\text{-}20' < 30°$

Live load included

$\tan\theta = \frac{21{,}650}{62{,}319} = 0.347 = 19°\text{-}10' < 30°$

For Overturning

Dead load only

$ac = 1.54$ m , $ck = 3.0$ m $\times 0.391 = 1.17$ m

$ak = 2.71$ m , $bk = 3.80 - 2.71 = 1.09$ m

Live load included

$ac = 1.50$ m , $ck = 3.0$ m $\times 0.347 = 1.04$ m

$ak = 2.54$ m , $bk = 3.80 - 2.54 = 1.26$ m

For Bearing

Dead load only

$\sigma = \frac{2}{3} \times \frac{55{,}319}{1.09} = 33{,}800$ kg/□m $= 3.38$ kg/□cm.

Live load included

$\sigma = \frac{2}{3} \times \frac{62{,}319}{1.26} = 33{,}000$ kg/□m $= 3.30$ kg/□cm.

Cheked for bottom of shaft

Position of center of gravity for vertical load (from point a)

For dead load only $= (85{,}420 - 31{,}768) \div (55{,}319 - 16{,}720) = 53{,}652 \div 38{,}599 = 1.39$ m

For live load included $= (93{,}470 - 31{,}768) \div (62{,}319 - 16{,}720) = 61{,}702 \div 45{,}599 = 1.35$

Assumed Earth Pressure $E = \frac{1}{2} \times 1600 \times 7.01^2 \times \frac{1}{3} = 13{,}100$ kg/m. Point of action $y = 2.34$

For sliding

Dead load only

$\tan\theta = \frac{13{,}100}{38{,}599} = 0.339 = 18°\text{-}45' < 30°$

Live load included

$\tan\theta = \frac{13{,}100}{45{,}599} = 0.287 = 16° < 30°$

For Overturning

Dead load only

$a'c' = 0.89$ m , $c'k' = 2.34$ m $\times 0.339 = 0.79$ m

$a'k' = 1.68$ m , $b'k' = 2.10 - 1.68 = 0.42$ m

Live load included

$a'c' = 0.85$ m , $c'k' = 2.34$ m $\times 0.287 = 0.67$ m

$a'k' = 1.52$ m , $b'k' = 2.10 - 1.52 = 0.58$

For Bearing

Dead load only

$\sigma = \frac{2}{3} \times \frac{38{,}599}{0.42} = 61{,}300$ kg/□m $= 6.13$ kg/□cm.

Live load included

$\sigma = \frac{2}{3} \times \frac{45{,}599}{0.58} = 52{,}400$ kg/□m $= 5.24$ kg/□cm.

Earthquake effect of structure

For bottom of shaft

Moment due to horizontal force

Earth Pressure	13,100 kg.	2.34 m.	30,654 m-kg.
Parapet wall	1,231	6.16	7,583
Coping	850	5.15	4,378
Shaft	17,600	2.24	39,424
Dead load from girder	7,550	6.74	50,887
			132,926 × 0.204 = 27,120 m-kg.

Section modulus of the section at the bottom of shaft $= \frac{1}{6} \times 2.1^{m\,2} \times 1.0^{m} = 0.735^{m^3}$

Maximum fiber stress $= 27,120 \div 0.735 = 36,900^{kg/\square m.} = 3.69^{kg/\square cm.}$

For bottom of Base

Moment due to horizontal force

Earth Pressure	21,650 kg.	3.00 m.	64,950 m-kg.
Parapet wall	1,231	8.16	10,045
Coping	850	7.15	6,078
Shaft	17,600	4.24	74,624
Base	16,720	1.00	16,720
Dead load from girder	7,550	8.74	65,987
			238,404 × 0.204 = 48,630 m-kg.

Section modulus of the base $= \frac{1}{6} \times 3.8^{m\,2} \times 1.0^{m} = 2.407^{m^3}$

Maximum fiber stress $= 48,630 \div 2.407 = 20,200^{kg/\square m.} = 2.02^{kg/\square cm.}$

Summary of Bearing stresses (Combined stresses)

For bottom of shaft $= 6.13 + 3.69 = 9.82^{kg/\square cm.}$

For bottom of base $= 3.38 + 2.02 = 5.40^{kg/\square cm.}$

梓　　橋

材 料 調 書

長野縣土木課道路改良	
路線名	縣道松本糸魚川線
圖名	材料調書
技師	鈴木邦彦
設計	北村清美　昭和3年8月15日完了
製圖	北村清美　昭和　年　月　日完了
寫圖	松木.加納　昭和3年9月3日完了
調査	昭和　年　月　日完了
全拾四枚之内貳	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension Size (mm)	Length (mm)	Weight Unit (kg/m)	Total Wt. (kg)	Remarks
		Girder G1				
4	Angle	150x150x11	9500	24.95	948	Flange Angles
4	〃	〃	10000	〃	998	〃 〃
2	〃	〃	9765	〃	487	〃 〃
2	〃	〃	10100	〃	504	〃 〃
2	Plate	330x10	9500	25.90	492	cover Plates
2	〃	〃	10000	〃	518	〃 〃
1	〃	〃	9765	〃	253	〃 〃
1	〃	〃	10100	〃	262	〃 〃
2	〃	330x11	6580	28.49	375	〃 〃
2	〃	〃	9280	〃	529	〃 〃
2	〃	〃	1980	〃	113	〃 〃
2	〃	〃	4790	〃	273	〃 〃
2	〃	〃	7900	〃	450	〃 〃
2	〃	〃	2690	〃	153	〃 〃
2	〃	〃	6500	〃	370	〃 〃
2	〃	〃	790	〃	45	〃 〃
2	〃	〃	4090	〃	233	〃 〃
1	〃	1200x10	9500	94.19	895	web plates
1	〃	〃	10000	〃	492	〃 〃
1	〃	1760x10	10010	138.15	1073	
8	Angle	150x150x11	715	24.95	143	splice Angles
4	plate	465x10	904	36.50	132	〃 plates
4	〃	330x11	720	28.49	82	〃 〃
2	〃	〃	2090	〃	119	〃 〃
2	〃	〃	2100	〃	120	〃 〃
2	〃	〃	1750	〃	100	〃 〃
2	〃	〃	3500	〃	199	〃 〃
2	〃	〃	4900	〃	279	〃 〃
2	〃	〃	6280	〃	358	〃 〃
12	Angle	125x90x10	1188	16.09	229	stiffeners
36	〃	〃	1200	〃	695	〃
4	〃	〃	1220	〃	79	〃
4	〃	〃	1300	〃	84	〃
4	〃	〃	1440	16.09	93	〃
4	〃	〃	1615	〃	104	〃
6	〃	〃	1748	〃	169	〃
1	〃	〃	235	〃	4	Lateral connection
1	〃	〃	150	〃	2	〃
2	〃	〃	160	〃	5	〃
2	plate	90x21	160	14.93	5	Fillers
1	〃	〃	70	〃	1	〃
1	〃	〃	75	〃	1	〃
6	〃	90x11	1464	7.77	68	〃
10	〃	90x11	904	〃	70	〃
2	〃	475x11	904	41.01	54	Reinf.
2	〃	220x22	462	37.99	35	〃
1	〃	160x11	330	13.81	4	Top
1	〃	297x11	330	25.64	7	Bottom
3732	Rivet head	22φ		0.0875	327	Shop rivets
1210	〃	〃		〃	106	Field rivets
					13137	
		4-Req'd			52548 kg	

Number	Description	Dimension Size (m.m.)	Dimension Length (m.m)	Weight Unit (kg/m)	Weight Total Wt. (kg.)	Remarks
			Girder G2			
4	Angle	150x150x11	7610	24.95	759	Flange Angles
4	〃	〃	7365	〃	735	〃 〃
2	Plate	330x10	7610	25.90	394	cover Plates
2	〃	〃	7365	〃	382	〃 〃
1	〃	1200x10	7610	94.19	705	
1	〃	〃	7610	〃	703	
4	Angle	150x150x11	715	24.95	71	splice Angles
2	Plate	465x10	904	36.50	66	〃 Plates
2	〃	330x10	720	25.90	37	〃 〃
26	Angle	125x90x10	1200	16.09	502	stiffeners
6	〃	〃	1188	〃	115	〃
2	Plate	90x11	904	7.77	14	Fillers
2	〃	297x11	330	25.64	14	Top
2	〃	160x11	330	13.81	8	Bottom
2	〃	475x11	904	41.01	54	
2	〃	220x22	460	37.99	35	
2	〃	220x11	570	19.00	22	
2	〃	〃	700	〃	25	
2	〃	475x11	904	41.01	54	
2190	Rivet head	22φ		0.0875	192	Shop rivets
328	〃	〃		〃	29	Field rivets
					4916	
		10-Req'd			49160kg	
			Girder G3			
4	Angle	150x150x11	9765	24.95	975	Flange Angles
4	〃	〃	10100	〃	1008	〃 〃
8	〃	〃	7000	〃	1397	〃 〃
2	Plate	330x10	9765	25.90	506	cover Plates
2	〃	〃	10100	〃	523	〃 〃
4	〃	〃	7000	〃	725	〃 〃
4	〃	330x11	1380	28.49	157	〃 〃
4	〃	〃	6280	〃	716	〃 〃
4	〃	〃	4900	〃	558	〃 〃
4	〃	〃	3500	〃	399	〃 〃
6	〃	〃	720	〃	123	splice Plates
4	〃	〃	2090	〃	238	〃 〃
2	〃	〃	2100	〃	120	〃 〃
4	〃	〃	1750	〃	199	〃 〃
2	〃	〃	3500	〃	199	〃 〃
2	〃	〃	4880	〃	278	〃 〃 web Pl.
1	〃	1760x10	10010	138.15	1075	
1	〃	〃	10010	〃	1073	〃
2	〃	1200x10	7000	94.19	1319	web Plates
12	Angle	150x150x11	715	24.95	214	splice Angles
6	Plate	465x10	904	36.50	198	〃 Plates
8	Angle	125x90x10	1188	16.09	153	stiffeners
28		〃	1200	〃	541	〃
8		〃	1220	〃	157	〃
8		〃	1300	〃	167	〃
8	〃	〃	1440	〃	185	〃
8	〃	〃	1615	〃	208	〃
12	〃	〃	1748	〃	338	〃
2		〃	235	〃	8	Lateral Shelf Angles

		Dimension		Weight		
Number	Description	Size (mm)	Length (mm)	Unit (kg/m)	Total Wt. (kg)	Remarks
2	Angle	125x90x10	150	16.09	5	Latral Shef Angles
4	〃	〃	160	〃	10	〃
4	Plate	90x21	160	14.93	10	Fillers
2	〃	90x21	70	〃	2	〃
2	〃	〃	75	〃	2	〃
12	〃	90x11	1464	7.77	137	〃
4	〃	〃	904	〃	28	〃
2	〃	475x11	904	41.01	54	
2	〃	220x11	380	19.00	23	
2	〃	475x11	904	41.01	54	
2	〃	220x22	462	37.99	35	
2	〃	160x11	330	13.81	8	Top
2	〃	297x11	330	25.64	14	Bottom
1434	Rivet head	22φ		0.0875	388	Shop rivets
1552	〃	〃		〃	136	Field rivets
					14663	
		6-Reqd			87978kg	
		Girder G4				
4	Angle	150x150x11	9765	24.95	975	Flange Angles
4	〃	〃	10100	〃	1008	〃 〃
8		〃	7000	〃	1397	〃 〃
2	Plate	300x10	9765	25.90	506	cover plates
2		〃	10100	〃	523	〃 〃
4	〃	〃	7000	〃	725	〃 〃
4	〃	330x11	1380	28.49	157	〃 〃
4	〃	〃	6280	〃	716	〃 〃
4	〃	〃	4900	〃	558	〃 〃
4	〃	〃	3500	〃	399	〃 〃
6	〃	〃	720	〃	123	splice Plates
4	〃	〃	2090	〃	238	〃 〃
2	〃	〃	2100	〃	120	〃 〃
4	〃	〃	1750	〃	199	〃 〃
2	〃	〃	3500	〃	199	〃 〃
2	〃	〃	4880	〃	278	〃 〃
2	〃	1760x10	10100	138.15	2151	Web Plates
2	〃	1200x10	7000	94.19	1319	〃 〃
12	Angle	150x150x11	715	24.95	214	splice Angles
6	Plate	465x10	904	36.50	198	〃 Plates
8	Angle	125x90x10	1188	16.09	153	stiffeners
28	〃	〃	1200	〃	541	〃
8	〃	〃	1220	〃	157	〃
8	〃	〃	1300	〃	167	〃
8	〃	〃	1440	〃	185	〃
8	〃	〃	1615	〃	208	〃
12	〃	〃	1748	〃	338	〃
2	〃	〃	235	〃	8	Shelf Angles for Lateral 1.8
2	〃	〃	150	〃	5	〃
4	〃	〃	160	〃	10	〃
4	plate	90x21	160	14.93	10	Fillers
2	〃	〃	70	〃	2	〃
2	〃	〃	75	〃	2	〃
12	〃	90x11	1464	7.77	137	〃 For Stiff
4	〃	〃	904	〃	28	〃 〃
4	〃	475x11	904	41.01	108	Reinf plates

		Dimension		Weight		
Number	Description	Size (mm)	Length (mm)	Unit (kg/m)	Total Wt.(kg)	Remarks
4	Plate	220x11	380	19.00	46	Reinf. Plates
2	〃	160x11	330	13.81	8	Gauge Plates
2	〃	297x11	330	25.64	14	〃 〃
4446	Rivets Head	22φ		0.0875	389	Shop rivets
1552	〃	〃		〃	136	Field rivets
					14655	
		2-Req'd			29310 kg	
		Cross Beams				
		For on Abutment & Intermediate				
2	Angle	80x80x9	4210	10.66	90	Flange Angles
2	〃	〃	4380	〃	93	〃 〃
1	Plate	500x8	4380	31.40	138	Web Plates
8	Angle	70x70x8	482	8.28	32	Stiffeners
4	Plate	140x9	334	9.89	13	Fillers
2	〃	170x10	295	13.34	8	Bed Pl. of String
2	Angle	80x80x9	267	10.66	6	Knee Bracings
2	〃	〃	352	〃	8	〃 〃
2	〃	〃	520	〃	11	〃 〃
2	〃	〃	670	〃	14	〃 〃
2	Plate	460x8	500	28.89	17	
368	Rivet Head	19φ		0.0563	21	Shop rivets
52	〃	〃		〃	3	Field 〃
					454	
		56-Req'd			25424 kg	
		For on Piers (Girder End)				
2	Angle	80x80x9	4210	10.66	90	Flange Angles
2	〃	〃	4380	〃	93	〃 〃
1	Plate	500x8	4380	31.40	138	Web Plates
8	Angle	70x70x8	482	8.28	32	Stiffeners
4	Plate	140x9	334	9.89	13	Fillers
2	〃	170x10	295	13.34	8	Bed Plates
2	Angle	80x80x9	267	10.66	6	Knee Bracings
2	〃	〃	352	〃	8	〃 〃
2	〃	〃	870	〃	19	〃 〃
2	〃	〃	1150	〃	25	〃 〃
2	Plate	460x8	1060	28.89	50	
388	Rivet Head	19φ		0.0563	22	Shop rivets
76	〃	〃		〃	4	Field rivets
					508	
		10-Req'd			5080 kg	
		Stringers				
4	I-Beam	300x150@48.34 kg	5050	48.34	976	
24	〃	〃	4990	〃	5789	
22	〃	〃	4885	〃	5195	
20	〃	〃	4690	〃	4534	
20	〃	〃	4615	〃	4462	
20	〃	〃	5160	〃	4989	
1056	Rivet Head	19φ		0.0563	59	Field rivets
		1-Req'd			26004 kg	

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Number	Description	Dimension Size (mm)	Dimension Length (mm)	Weight Unit (kg/m)	Weight Total Wt (kg)	Remarks
		Lateral Bracings				
12	Angle	125x90x10	5970	16.09	1153	Lateral B's
24	〃	〃	2930	〃	1131	〃
23	〃	〃	6185	〃	2289	〃
46	〃	〃	3035	〃	2246	〃
10	〃	〃	5845	〃	940	〃
20	〃	〃	2870	〃	924	〃
10	〃	〃	6060	〃	975	〃
20	〃	〃	2975	〃	957	〃
22	Plate	230x10	560	18.05	222	Splice Plate
23	〃	〃	565	〃	235	〃 〃
10	〃	〃	555	〃	100	〃 〃
24	〃	325x10	320	25.51	196	Gusset Pl.
24	〃	〃	750	〃	459	〃 〃
24	〃	315x10	760	24.72	451	〃 〃
20	〃	295x10	735	23.16	340	〃 〃
20	〃	330x10	315	25.90	163	〃 〃
20	〃	310x10	755	24.33	366	〃 〃
4	〃	125x11	150	10.79	6	Fillers
4	〃	〃	280	〃	12	〃
20	〃	〃	75	〃	16	〃
20	〃	〃	390	〃	84	〃
330	Rivet Head	22φ		0.0875	29	Shop rivets
1980	〃	〃		〃	173	Field rivets
		1-Req'd			13467 kg	
		Pin and Bolts				
10	Pin	94.4φ	160	11.70	117	With 2-nuts
20	Bolt	22φ	130	0.66	13	With Cotter
48	〃	38φ	600	6.18	297	Anchor bolts with washer
		1-Req'd			427 kg	
		Bronze Bearings				
10	Bronze	76x120	220	10.60	106	Top bearings
10	〃	74.4x130	220	10.80	108	Bottom 〃
		1-Req'd			214 kg	
		Sole Plates				
12	Plate	400x22	458	69.07	310	Slide
12	〃	〃	458	〃	329	Fixed
		1-Req'd			639 kg	
		Caststeel Shoes				
24	cast steel	560x120	760	207.0	4968	Shoes
48	〃	190x128	205	13.2	634	2-shape
		1-Req'd			5602 kg	

Number	Description	Dimension Size(mm)	Length(mm)	Weight Unit(kg/m)	Total Wt(kg)	Remarks
		Expansion Joints (Steel)				
	For on Abutment					
1	Flat	50x11	5502	4.32	24	on Parapet wall
1	Angle	125x75x10	5502	14.91	82	〃 〃
1	Plate	170x11	5502	14.68	81	cover Plate
1	〃	185x8	5502	11.62	64	web Plate
2	Angle	75x75x9	5502	9.96	110	Flange Angles
2	Plate	85x60	330	40.03	26	Fillers
57	Bar	13φ	400	1.04	24	End Hooked
175	Rivet Head	〃		0.0180	3	Shop rivets
					414	
		2-Req'd			828 kg	
	For Intermediate					
1	Flat	50x11	5502	4.32	24	
1	Plate	160x11	5502	13.81	76	cover
2	〃	185x8	5502	11.62	128	web
1	Angle	125x75x10	5502	14.91	82	Flange
3	〃	75x75x9	5502	9.96	164	〃
4	Plate	85x49	330	32.69	43	Fillers
58	Bars	13φ	400	1.04	25	End Hooked
270	Rivet Head	〃		0.0180	5	shop rivets
					547	
		10-Req'd			5470 kg.	
	Summary of Exp. Joints				6298 kg.	
		Handrail Panels (Struct. Steel)				
3	Bar	20 sq.	680	3.14	6.4	
4	Flat	20x10	170	1.57	1.1	
2	Plate	80x10	260	6.28	3.3	
					10.8	
		1052-Req'd	1052 x 10.8 kg =		11362 kg.	
		Castiron Drain Holes				
66	Castiron	200x200	180	14.16	935	Pipe
66	〃	200x24	200	4.43	292	cover net
		1-Req'd			1227 kg.	
		Expansion Joint of Slab (Lead)				
1	Sheet Lead	180x3	5500	6.11	34	
		6-Req'd			204 kg.	
		Well Curb Shoes				
2	Plate	440x6	3600	20.72	149	
2	〃	400x6	3600	18.84	136	
2	Angle	125x75x10	3600	14.91	107	
12	Plate	500x6	755	23.55	213	
12	〃	400x6	755	18.84	171	
12	Angle	125x75x10	755	14.91	135	
14	Plate	120x6	400	5.65	32	splice Plate
14	〃	〃	440	〃	35	〃 〃
80	〃	90x6	450	4.24	153	Tie Flat
40	Bar	16φ	1000	1.57	63	Anchorage
2736	Rivet Head	13φ		0.0180	49	shop or Field
					1243	
		10-Req'd			12430 kg.	

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			Dimension	Weight		
Number	Description	Size(mm)	Length(mm)	Unit(kg/m)	Total Wt(kg)	Remarks
		Summary				
	Girder	G1	52 548 kg			
	〃	G2	49 160			
	〃	G3	87 978			
	〃	G4	29 310			
	Cross Beams		25 424			
	Cross Beams		5 080			
	Stringers		26 004			
	Lateral Bracings		13 467			
	Pin and Bolts		427			
	Sole Plates		639			
			290 037 kg.			
	Bronze Bearings		214			
	Caststeel Shoes		5 602			
			295 853 kg.			

Weight of Steel Reinforcements

Mark	Number	Diam. (m.m)	Length	Tot. length	Weight (kg) unit (kg/m)	Weight (kg) Total weight	Remarks.
				Floor Slab			
Slab Panel "A."							
S_1	43	13	6.480	278.640	1.04	290	Top
S_2	43	〃	6.880	295.840	〃	308	Bottom
S_3	82	〃	6.700	549.400	〃	571	Bent-up at Supports
L_1	44	〃	6.450	283.800	〃	295	Longitudinal
						1,464	
				2-Req'd		2,928	kg
Slab Panel "B"							
S_1	57	13	6.480	369.360	1.04	384	Top
S_2	57	〃	6.880	392.160	〃	408	Bottom
S_3	111	〃	6.700	743.700	〃	773	Bent-up at supports
L_2	66	〃	5850	386.100	〃	402	Longitudinal
						1,967	
				10-Req'd		19,670	kg
Slab Panel "C"							
S_1	51	13	6.480	330.480	1.04	344	Top
S_2	51	〃	6.880	350.880	〃	365	Bottom
S_3	102	〃	6.700	683.400	〃	711	Bent-up at supports
L_3	66	〃	5.350	353.100	〃	367	Longitudinal
						1,787	
				5-Req'd		8,935	kg
Summary						31,533	kg
				Handrails			
CP	96	13	2.120	203.520	1.04	212	Post C & B
CH	48	6	1.400	67.200	0.22	15	〃 Hoops
AP	676	13	2.040	1.379.040	1.04	1434	Post A
AH	676	6	720	486.720	0.22	107	〃 Hoops
BP	48	13	2.040	97.920	1.04	102	Post B
BH	48	6	500	24.000	0.22	5	〃 Hoops
T_1	40	13	6.450	258.000	1.04	268	Top & Bottom rail
T_2	300	〃	5.850	1.755.000	〃	1825	〃
T_3	150	〃	5.350	802.500	〃	835	〃
TH	1760	6	500	880.000	0.22	194	〃 Hoops
CP	16	13	2.120	33.920	1.04	35	Post on Abutment
CPH	8	6	900	7.200	0.22	2	〃 〃
AH	8	〃	720	5.760	〃	1	〃 〃
AP	20	13	2.040	40.800	1.04	42	〃 〃
APH	8	6	1.500	12.000	0.22	3	〃 〃
R_1	20	13	1.950	39.000	1.04	41	Railing 〃
R_2	20	〃	1.100	22.000	〃	23	〃 〃
TH	40	6	500	20.000	0.22	4	〃 〃
				1-Req'd		5,143	kg
				Piers P_1 ..... P_{10}			
WH_1	18	16	8.590	154.620	1.57	243	Well Horizontals
WH_2	36	〃	8.340	300.240	〃	471	〃
WH_3	54	〃	7.090	382.860	〃	601	〃
WH_4	50	〃	3.200	160.000	〃	251	〃
WV_1	32	〃	4.200	134.400	〃	211	〃 Verticals
WV_2	32	〃	4.300	137.600	〃	216	〃 〃
WV_3	6	〃	3.160	18.960	〃	30	〃 〃

Mark	Number	Diam. (m.m)	Length	Tot. length	Weight (kg) unit (kg/m)	Total weight	Remarks
WV4	70	16	2,960	207,200	1.57	325	Well verticals
WT1	32	6	1,200	38,400	0.22	8	〃 Ties
WT2	104	〃	1,000	104,000	〃	23	〃 〃
PV1	44	16	1,600	70,400	1.57	111	Shaft connetion
PV2	24	〃	2,500	60,000	〃	94	〃 Verticals
PV3	20	〃	5,000	100,000	〃	157	〃 〃
PH1	2	〃	7,010	14,020	〃	22	〃 Hoops
PH2	2	〃	6,890	13,780	〃	22	〃 〃
PH3	2	〃	6,700	13,400	〃	21	〃 〃
PH3	2	〃	5,640	11,280	〃	18	〃 〃
PH5	2	〃	4,390	8,780	〃	14	〃 〃
PC	4	〃	6,570	26,280	〃	41	Coping
PCH	18	13	2,700	48,600	1.04	51	〃
						2,930	10-Req'd = 29,300 kg.

Abutments A & B

Parapet Wall (Similar A & B)

W1	41	13	4,360	178,760	1.04	186	Bent to U-shape
W2	8	〃	7,000	56,000	〃	58	
F1	2	〃	6,000	12,000	〃	12	shelf Reinf
F2	4	〃	1,300	5,200	〃	5	〃 〃
F3	4	〃	2,500	10000	〃	10	〃 〃
F4	35	〃	1,140	39,900	〃	42	Bent to U-shape
				1-Req'd		313 kg	

Wing Wall for Abutment A

AW1	34	13	3,200	108,800	1.04	113	Bent to U-shape
AW2	20	〃	2,800	64,000	〃	67	
				1-Req'd		180 kg	

Wing Wall for Abutment B

BW1	30	13	3,200	96,000	1.04	100	Bent to U-shape
BW2	20	〃	2,800	56,000	〃	58	
				1-Req'd		158 kg	

Summary of Abutments

For Abutment A	493 kg
For Abutment B	471 kg
	964 kg

Summary of Steel Reinforcements

Slab	31,533 kg
Handrails	5,148
Piers	29,300
Abutments	964
	66,945 kg

CONCRETE VOLUME

[I] Floor Slab (1:2:4 Concrete)

sq.m.

Slab 0.15×4.5+1.0×0.185 = 0.8600

Curb 2×{0.32×0.3−(0.2×0.05+0.07×0.05+0.0038)} = 0.1574

Sectional area ---- 1.0174 sq.m

Length 2×12.495+10×16.76+5×15.22 = 268.69 m. — cub.m.

Volume 1.0174×268.69 ---------------- 273.36

Fillet 0.33×0.06×(4×5.0+10×34.4) ---------- 6.84

280.20 cub.m.

[II] Handrails on Girders (1:2:4)

cub.m.

Post A. 338×0.22×0.16×0.814 ---------- = 9.68

Post B. 12×0.21×0.16×0.814 ---------- = 0.33

Post C. 14×{0.60×0.24×0.82+(0.0226×0.4)+0.0064×0.2} -- = 1.80

Post D. 10×(0.11808+0.0226×0.24) ---------- = 1.24

Railing {0.16×(0.134+0.075)}{1.29×44+1.28×288+1.035×20} = 14.92

27.97 cub.m.

[III] Handrail on Abutments.

cub.m.

Post 0.34×0.24×0.82+0.00542+0.00064 ---------- 0.073

" 0.30×0.24×0.82+0.00271+0.00064+0.00336 ----- 0.066

" 0.6×0.16×0.814 ---------------- 0.078

Railings 0.03344×(0.51+1.28) ---------------- 0.060

0.277 cub.m.

4-Req'd 0.277×4 — 1.11 cub.m.

29.08

[IV] Light Pedestals (1:2:4 Concrete)

cub.m.

Bottom 0.2×(1.0²−4×0.1²) ---------- = + 0.192

Name Part 1.0×(0.9²−4×0.1²) ---------- = + 0.770

Upper 2.05×(0.6²+1.6×0.075)+0.5×0.05 --- = + 0.997

Less 0.02×0.16×0.54×4+0.00442×3.3 ----- = − 0.022

1.937 cub.m.

4-Req'd 1.937×4 = 7.75

[V] Piers

Coping 0.04×(1.08×4.5+0.916) = 0.23 cub.m.

0.26×(1.16×4.5+1.057) = 1.63

1.86 cub.m.

10-Req'd 18.6 cub.m. (1:2:4)

Shaft 1.25×5.0×4.5 + 1/3×5.0×(0.7854+1.7672+$\sqrt{0.7854\times1.7672}$) = 34.34

10-Req'd 343.4 cub.m. (1:3:6)

Well 0.5×5.92×(7.2+7.226) = 42.70

0.04×1.92×(7.2+8.96) = 1.24

0.25×0.5×(7.2+7.74) = 1.87

wall (0.5×1.8+2×0.4×0.1)×5.84 = 5.72

51.53 cub.m.

10-Req'd 515.3 cub.m. (1:2:4)

Well fill (3.6×1.8+2.5447−0.98)×3.5 = 28.16

0.29×0.58×(7.2+6.87)+0.98×0.65 = 3.00

31.16 cub.m.

10-Req'd 311.6 cub.m. (1:3:6)

Well fill (3.6×1.8+2.5447−0.98)×3.0 = 24.13 cub.m.

10-Req'd 241.3 cub.m. (Sand & Gravels)

Summary

1:2:4	cub.m.	1:3:6	cub.m.
Coping	18.6	Shaft	343.4
Well	515.3	Fill	311.6
	533.9 cub.m.		655.0 cub.m.

(IV) Abutments.

(a) Abutment A & B (Similar A & 3) cub.m.

Parapet wall 0.3 × 5.5 × 1.95 = 3.22

Wing 2×(2.3×1.12+0.3×0.98)×2.06−0.82×0.8×0.5) = 11.63

Shaft 0.3×0.2×2×(2.2+0.62+2.75) = 0.67

Coping 0.3×6.0×0.3+0.08×0.28×16.06 = 1.80

17.32 (1:2:4)

(b) Abutment A. cub.m.

Shaft ½×(1.1+2.4)×6.5×6.0+0.25×0.82×6.5 = 69.58

½×(1.12+2.42)×6.5×4.1 = 47.17

Wing 2×0.3×1.29×0.48 = 0.76

Base 2.2×(2.3×6.0+2.12×4.7+1.1×0.62) = 00.86

218.37 cub.m (1:3:6)

~~(c) Abutment B.~~

~~Shaft ½×(1.1+2.1)×5.0×6.0+0.25×0.82×5.0 = 49.03~~

~~, ½×(1.12+2.12)×5.0×4 = 33.21~~

~~Wing 2×0.3×0.89×0.48 = 0.52~~

~~Base 2.0×(3.8×6.0+3.62×4.7+1.1×0.62) = 80.99~~

~~163.75 (1:3:6)~~

218.37 cub.m − 0.76 cub.m + 0.52 cub.m = 218.13 cub.m (1:3:6)

FORM AREA (型枠)

(I) Floor slab sq.m

268.59×2×(0.05+0.15+0.41+0.515+1.26+0.675) = 1644.4

0.12×(20.0+344.0) = 43.7

1.017 sq.m × 18 = 18.3

1,706.4 sq.m

(II) Handrails. sq.m.

Post A&B 350×(0.44×0.82+0.32×0.605) = 194.04

Post C&D 24×(1.20×0.82+0.48×0.912)−0.03344 m² ×44 = 32.65

, Top 0.0678×48+0.1055×14 = 4.73

Rail 0.60×446.10 = 267.66

Post of Abutment 4×(0.32×0.605+1.2×0.82) = 4.71

, 4×(0.68×0.82+0.48×0.912−0.03344 m² +0.0678 m²) = 4.12

, 4×(0.6×0.82+0.24×0.912−0.03344 m² +0.1 m²) = 3.11

Rail 4×0.60×(1.28+0.51) = 4.30

515.32 sq.m.

(III) Light Pedestals

Bottom 4.0×0.2 = 0.80

Namp Part 3.6×1.0−4×0.18×0.56 = 3.20

Upper 3.6×2.05+2.0×0.05 = 7.48

11.48 sq.m.

4-Req'd 11.48 sq.m. ×4 = 45.92 sq.m.

(IV) Piers sq.m.

Coping 0.3×(9.0+3.644) = 3.79

Shaft 5.006×(3.927+9.0) = 64.71

68.50 sq.m.

Well sq.m.

4.5×(7.2+8.7965) = 71.98

1.6×(7.2+9.0478) = 26.00

5.92×(9.4492+5.6549) = 89.42

0.31×(7.2+6.283) = 4.18

1.90×0.6 = 1.14

.... 192.72
sq.m. 261.22

10-Req'd 2612.2

[V] Abutment A

Base	2.2 × 2 × (3.0 + 0.8 + 2.35 + 4.12 + 2.9 + 0.62 + 2.45)	=	71.46 sq.m.
Shaft	6.63 × (4.1 + 6.0) + 0.8 × 6.5 × 2 + (1.12 + 2.42) × 6.5	=	100.37
〃 back	6.5 × 2 × (2.3 + 0.82 + 2.75) − 0.3 × 1.29	=	75.92
Wall 〃 back	2.01 × 2 × (1.28 + 2.0 + 0.82 + 2.75) + 0.4 × 11.14	=	31.99
〃 front	2.01 × 2 × (3.0 + 0.8 + 2.05 + 2.1) + 0.17 × 16.06	=	34.69
			314.43 sq.m.

~~[VI] Abutment B~~

~~Base~~	~~2.0 × 2 × (3.0 + 0.8 + 2.35 + 3.62 + 2.9 + 0.62 + 2.45)~~	=	~~62.96~~ sq.m.
~~Shaft~~	~~5.1 × 10.1 + 1.6 × 5.0 + (1.12 + 2.12) × 5.0~~	=	~~75.71~~
~~〃 back~~	~~5.0 × 2 × (2.3 + 0.82 + 2.75) − 0.3 × 0.89~~	=	~~58.43~~
~~Wall 〃~~	~~2.01 × 2 × (1.28 + 2.0 + 0.82 + 2.75) + 0.4 × 11.14~~	=	~~31.99~~
~~〃 front~~	~~2.01 × 2 × (3.0 + 0.8 + 2.05 + 2.1) + 0.17 × 16.06~~	=	~~34.69~~
			~~263.78~~ sq.m.

人造洗出シ仕上 面積

[I] Handrails

Post A & B	350 × (1.8 × 0.22 + 0.32 × 0.605)	=	206.36 sq.m.
Post C & D	24 × (1.20 × 0.82 + 0.48 × 0.912) − 0.03344 × 44	=	32.65
〃 Top	48 × 0.0614 + 14 × 0.0927 + 24 × 0.06	=	5.69
Railing	446.1 × (0.29 + 0.60) − 1036 × 0.16 × 0.26	=	353.93
on Abutments Post	4 × (1.8 × 0.6 + 0.32 × 0.605)	=	5.09
〃	4 × (0.48 × 0.912 − 0.03344 + 0.68 × 0.82)	=	3.85
〃	4 × (0.6 × 0.82 + 0.24 × 0.912 − 0.03344)	=	2.71
〃 Top	4 × (3 × 0.0614 + 0.06 + 0.1056)	=	1.40
Rails	4 × 1.79 × (0.29 × 0.60) − 16 × 0.16 × 0.26	=	5.71
			617.39

[II] Light Pedestals

Bottom	4.0 × 0.2 + 0.05 × 3.8	=	0.99 sq.m.
Name Part	3.6 − 4 × 0.18 × 0.56 − 0.24 × 1.0 + 0.05 × 3.4 + 0.4 × 0.2	=	3.42
Upper	3.6 × 2.05 + 0.1 + 0.05 × (1.6 + 2.2 + 1.8)	=	7.76
			12.17 sq.m.

4-Req'd 12.17 sq.m. × 4 = 48.68 sq.m.

MORTAR FINISH

[I] Floor Slab

268.64 × 2 × (0.5 + 0.15 + 0.14 + 0.41) = 644.9 sq.m.

[II] Curb on Abutments.

2 × 0.15 × (1.12 + 1.2 + 0.82 + 0.8 + 2.1 + 2.3 + 2.1) .... = 3.13 sq.m.
2 × 0.8 × 0.82 = 1.31
4.44

2-Req'd 8.9 sq.m.

Excavation Volume

[I] Piers

Average stectional area = 3.6 × 3.5 + 9.62 = 22.22 sq.m.

Length = 8.02+8.10+7.31+8.55+8.72+8.92+8.35+8.61+8.90+9.32 = 84.80

Volume = 22.22 × 84.8 = 1884.3 cub.m. say

[II] Abutments

(a) Abutment A

6.5 × 7.0 × 12.0 = 546.0 cub.m.

(b) Abutment B

~~6.0 × 6.0 × 12.0 = 432.0~~ cub.m.

6.5 × 8.0 × 12.0 = 624.0

Pavement

Asphalt Block Pavement area

4.5 × {269.0 − (2 × 0.083 + 10 × 0.241)} = 1199.25 sq.m.

Cement Mortar cushion Volume

0.015 × 4.5 × 266.5 + 0.03 × 4.5 × (2 × 0.066 + 10 × 0.181) = 18.25 cub.m.

袖石垣面積

			sq.m.
右岸	9.0 × 6.0	-------	54.0
左岸	6.0 × 6.0	------	36.0
			90.0 sq.m.

Bridge Entrance (Both Ends)

(a) Asphalt Block Pavement area

2 × {2.0 × 0.86 + 5.5 × 1.68} = 21.92 sq.m.

(b) 花崗石 (Edge ston)

2 × 0.12 × 9.5 = 2.28 sq.m.

(c) Cement Mortar cushion Volume

2 × 0.015 × (2.0 × 0.98 + 5.5 × 1.8) = 0.35 cub.m.

(d) Concrete foundation

2 × {0.15 × 2.0 × 0.98 + 0.2 × 5.5 × 1.8} = 4.55 cub.m.