DESIGN REPORT OF PARALKOTE IRRIGATION CANAL SYSTEM SURVEY PARALKOTE ZONE, DANDAKARANYA PROJECT IN INDIA

VOLUME I



AUGUST 1971

国際協力等	多紫团
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OVERSEAS TECHNICAL COOPERATION AGENCY TOKYO, JAPAN

ABBREVIATIONS

```
: (allowable) stress for compression of concrete
  σc (σca)
                                                                           (kg/cm^2)
  os (osa)
                  : (allowable) stress for tension of reinforcement
  \sigma_i (\sigma_{ia})
                  : (allowable) stress for compression of cast iron
  τ<sub>c</sub> (τ<sub>ca</sub>)
                  : (allowable) shearing stress of concrete
                  : (allowable) stress for tension of wood
\sigma_{k'} \sigma_{k'a}
  M, MB, MC, Md, M1, Mmax, Mx and AMx: bending moment
                                                                    (ton-m, kg-cm)
  As, As, (Av) : amount of reinforcement (stirrup)
                                                                           (cm<sup>2</sup>
                 : supporting area of cast iron (concrete)
                                                                           ("
  R, RA, RB, RC, Rd, R1, HA: reaction force
                                                                           ( ton
  S, Smax, Sx
                : shearing force
  q_1, q_2, q_3, q_4, q_5, q_6, q_7, W, W', W : dead load
                                                                           (ton/m)
  W_1, W_2, W_3, P_1, P_2, P_1', P_2', P_3', P_4': dead load
                                                                           (ton)
                  : total dead load
  q_o
                                                                           (ton/m)
  Pw, q'
                  : water pressure
  We
                  : live load
                                                                           ( "
  Pf, Pr
                  : front (reverse) portion of truck load
                                                                           (ton)
  Т
                  : Truck load (live load)
                                                                           ( "
  Pc
                  : curb load
                                                                           (ton/r)
 Ho, (Po, No)
                : horizontal (normal) force
                                                                           (")
  \gamma, \gamma_1, \gamma_2, \gamma_c, \gamma_r, \gamma_W, \omega: unit weight
                                                                           (ton/m^3)
                  : coefficient of earth pressure
  F
                  : safety factor
                  : ratio of As and area of section
                     number of reinforcement bars
                     effective thickness
                                                                           ( m, cm)
                    thickness of the upper part of T-beam
  (b'), b, B
                  : (effective) width
                                                                           ( m, cm)
  x, (1, 1^1)
                     distance (of span)
                     depth
 h', h_{S_1}, h_{S_2}, h_{S_3}, h_{S_4}, h_S, h_W, h_{W_1}: conversion height of load
                  : distance form the top to the Neutral Axis of T-beam
 S
                                                                           ( m
 V
                  : total vertical load
 I
                     geometrical moment of inertia
                                                                           ( cm<sup>4</sup>
 e
                  : distance of eccentricity
                                                                           ( m
                  : ratio of coefficient of elasticity
 n
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DESIGN REPORT OF PARALKOTE IRRIGATION CANAL SYSTEM SURVEY

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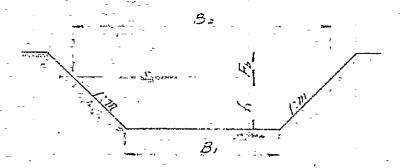
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CHAPTER I HYDROULIC CALCULATION

Plan Hydraulics Calculation for Typical Section

of Open Canal



TKPE	Bi	fr.	777	₿2	, FA	A	P	R
A	3.50	m 1. IS	6.3	4.17	6.55	6.97	7.50	0.98.7
B.	10.00	1.30	/	12.60.	0.56	14.69	1368	1.074
C,	3.00	1.89.	2	- -	1.61	12.81	1146	1.118
\tilde{D}	1.00	1.23	2		1.47	1.26	£ 5%	0455
L E	3.00	c. 86	2		•	4 059	6846	0.593
in A	3.70	0.47	2			182	t 102	0363
<u>C</u> 7	1.00	270	_ &			1686	4.130	0407

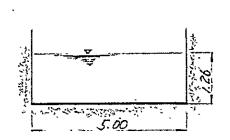
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1 7	0.991	00225	11.000	203/62	1.393	0.099	7.74.	> 9.63
5	1.019		1/5,000	0.01111	0.659	0022	9.60	79.63
144	1.677		٠,	"	6.677	2.023	867	7 2.65
A - 1	C750	,	3.	U	6.274	0.011	2.02	7 2.00
接	0,706	0.020	de .		6.498	0.0/3	·*.2.02	7 2.00
	ો ' ‡ં ક્રાંક્સ તે	0.0225	Land .	****	0.318.	0.005	0.5.89	7.0.569.
$\frac{1}{1}$	×0.549	, t	*	*,	0.344	0.006	0.578	> 0.569

1-2 Hydraulic Calculation for Main Structure.

of A Route.

- 1-2-1 No 1 and No. 2 Irrigation Aqueduct.

1-2-1-1 Typical section of irrigation aguiduct



Eischarge $Q = 9.63^{m/8}$ Laral Lopie. I = 1/5000Coefficient of roughness $\Lambda = 0.015$ Canal weath B = 5.00Water depth H = 126

Filow area $A_1 = 5.00 \times 1.26 = 630$ Welled perioritor $P_1 = 5.00 + 126 \times 2 = 7.752$ Hydraulic radius $R_1 = A/P = 6.30/7.52 = 0.838$ $R_1^{7/5} = 0.889$ Velocity $v_1 = \frac{1}{\pi} R_1^{3/5} I_2^{1/2} = \frac{1}{0.015} \times 0.889 \times (11.500)^2$ $= 1.571 \text{ M/s}_1$ $Q_1 = A_1 V_1 = 630 \times 1531 = 9.65 \text{ m/s}_5 = 9.63 \text{ m/s}_5$ Velocity $R_2 = 0.120$

1-2-1-2 Hydraulic elements of open canal

Discharge $Q_2 = 9.63$ m/sec Lamal slope $I_2 = 15.000$ Filow area. At = 14.69 Velocity $v_2 = 659$ m/s Velocity head $hv_2 = \frac{9}{2} = \frac{9}{2} = 0.022$

SAME CALL MAL A PROPERTY

Transition length L=1800

hed besthead of gradual contraction :

fge coefficient of gradual contraction

12) Loss head of fraction

 $ReL = \frac{1}{2}(I_1 + I_2) \cdot L$ $= \frac{1}{2}(\frac{1}{1.500} + \frac{1}{5.000}) \times 16.00$ $= 6^{14} COE$

her; los head of frections

(3) Total law hard of upper transition.

her=hed + hel = 0020 + 0.000 = 0028

1-2-1-4 Loss head of lower bransetion; her

Transition length L= 1800 (1) Loss head of gradual enlargement

> Rai = 172 (Rug - Rug) = 6.3 (0.120-0.022) = 0.029

. Aprilos hand of gradual enlargement hgs coefficient of gradual enlargement

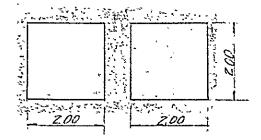
:(2) Loss head of frection:

Rel = \frac{1}{2} (\frac{1}{2}, \tau I_2) l = \frac{1}{2} (\frac{1}{1,500} - \frac{1}{5,000}) \times 18.00

(3) Total loss hand of Lower transition her = her + hor = 0 029 + 0.000 = 0.037

1-2-2 No.1. Imigation Syphon.

1-2-2-1 Typical section of irrigation syphone.



Dividing $Q_1 = \frac{1}{2}Q = \frac{1}{2} \cdot 963 = 4.615 \,^{m/s}$ Flow area $A_1 = BH = 2 \times 2 = 4000 \,^{m}$ Loofficient of rough sex p(=0.020)Velocity $V_1 = Q_1 = 4.615 / 400 = 1.204 \,^{m/s}$ Velocity head $hv_1 = \frac{15}{2} = \frac{1.204}{2 \times 4.6} = 0.074$ Wellow perimeter $p = 4 \times 2 = 8.00 \,^{m}$ Hydraulic radius $R = \frac{A}{p} = \frac{4.00}{6.00} = 1.500$ $R^{7/5} = 0.630$ Hydraulic gradient $I = \left(\frac{7.1^{1/5}}{R^{7/5}}\right) = \left(\frac{0.020 \times 1.204}{0.630}\right) = 0.00146$

1-2-2-2 Hydraulic dements of open cannot

Les charge Q = 963 m/sCanal States. I = 75000Flow area $A = 14.69 \text{ m}^2$ Velocity V = 0.659 m/sVelocity head $hv_3 = 0.022$

1-2-3. Loss hand of upper transition; her

Transition length L= 8.00.

115 Lind head of entrance her full (hvi - hva) = 0.4 (0.074-0,022) = 0.021

for coefficient of entrance

12) Low head of friction. $h_0 = \frac{1}{2} (I_1 + I_2) \mathcal{L}$ $= \frac{1}{2} \times (0.00146 + 0.00020) \times 80 = 0.007$

(3) Total loss head of upper transition.

hel = ho + he = 0.021 + 0.007 = 8028

1-2-2-4 Loss head of sypthen culvert length L=24.0

ly = 29x2. L. hv

hy, less head of priction

= 2x98x11212 x 24.0 x 0.074 = 0.035

Taking 10 percent allowance. Aj = 0.035 x 1.1 = 1.038

1-2-2-5 Loss head of kower transition; her

Transition length

11) Loss head of exit. ho = fo (hvi - kvz)

= 0.6 (0.094-0.022) = 0031

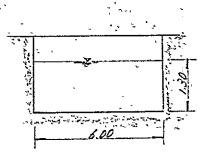
12) Loss head of friction

to z(InIz)l

(3) Total Low head of Lower transition hez

hez = ho - hig = 003/+0.007 = 0.038 1-2-3 No.1 Brigde

1-2-3-1 Typical section of irrigation land under brigde



Lescharge $Q = 963^{-m3/5}$ lostficient of reughness K = 0.0225

Flow area. $A = 609 \times 1.30 = 760^{m^2}$ Wetted perimeter. $P = 606 + 130 \times 2 = 600^{m}$ Hydraulic radius R = A/p = 7.60/660 = 0.907 $R^{7/3} = 0.937$ Value city $V = Q/A = 963/760 = 1235^{m/3}$ Hydraulic gradient $I = \left(\frac{A \cdot V}{R^{7/3}}\right)^2 = 0.00058$

Volacity hand. The = N/2g = 1.735 2/2x98 = 0078 : 1-2-3-2 Loss head of irrigation canal under designed Bright width. B=425

11) Loss hard due to sudden contraction

Age = fge AN = 0.29 × 0.678 = 0.7063

A=/A, = 7.80/14.69 = 65

for coefficient of pudden contraction

12) Loss head due to sudden enlargement.

hse = fsahn = 0.25 × 0.078.

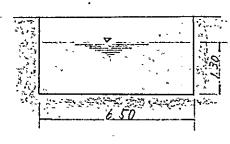
for coefficient of sudden enlargement

(4) Total loss head of irrigation canal under bright.

LI = hgc.+ hsc - he = 0.004 - 0.047

1-2-4 NOZ and No.4 Drudge.

1-2-4-1 Typical section of wrightion coint under Bright



Discharge Q=9.63 TVs Coefficient of northness 11=00225

Flow area . A = 6.50 x 1.30 = 8.45 212 Wetled perimeter P = 6.50 + 130 × 2 = 9.10 mm Hydraulic radius R = 8.45/910 = 0.929

R\$5 = 0.952

Velocity $V = Q/A = 9.63/845 = 1.140^{m/sec}$ Velocity head $f_{0} = V/8.9 = 1.140^{2}/2.498 = 0.066^{20}$ Hydraulic gradient $I = \left(\frac{11-1}{R^{\frac{1}{12}}}\right)^{2} = 0.00073$

1-Z-4-Z Loss head of irrigation canal under wright. B = 5.00

- (1) Los head due to sudden contraction

 Rgc = fgc . ho
 = 0.21 x 0.066

 = 0.014

 Al/A = 845/14.69 = 0.60

 + 2c = 0.21
- (2) Loss head due to sudden enlargement.

Asc = fise Ru.

= 0.25 × 0.078.

= 0.020

(3) Loss head of friction

he = I. L = 000073 x 5.00 = 0.004

(4) Total loss head of irrigation canal under aredge

hs = hgc + hse + hs = 0.014 + 0.011 + 0.004 = 0.029

1-2-5- / Typical section of impation sylon under bright

0.015

A, = \$ TED x 4 = \$ \$ 314 x 1.202 x 4

Velocity V= 9/A = 963/4.537 = 2.123 Rus 6.230ml

Wetled perimeter P = TED x 4 = 3.14 x 1202 x 4 = 15.097

Hydraulic R,= A/P = 4.537/15097 = 0.301 R13 - 0.448

Hydraulic I, = (M.V) = (0015x2123) = 0.00505

1-2-5-2 Hydraulic elements of open canal

Dweharge.

Q= 915 77/2

land slope

I= = 15.000

Felow area

Az = 14.69 2

V2 = 0 659 m/s

Kyz = 0.022

1-2-5-3 Loss head of irrigation sylon under bridge

(1) Lias head of entrance

he filher-hui)

12) Loss hard of exect

ho=f(kv;-hn) = 0.7(6.230-0.072) = 7.746

(3) Loss head of friction

he = 124.5 m²/043 hu, l = 124.5 × 0.015/1278 × 5.10 × 0.230 = 0.026 Taking 10 percent allowance. he = 0.025 × 1.1 = 0.029

14) Total loss head of irrigation syphon under bright

 $\hat{h}E = \hat{h}e - \hat{f}_{10} + \hat{h}l$ = 0104 + 0.146 + 0029 = 0279

1-2-6 No. 1 Transition (RD0+317~383)

Transition length L- 200

1-2-6-1 Hydraulic eliminate of A type open canal

. Sischary Q'= 29.63 m3/s

Carel aloyer 1, = . 11,000

Flow area. A, = 6,99 mi

Velocity 1, = 1.393 m/s

Velocity head kir = 0.0992

1-2-6-2 Hydronluc dements of Blype open caral

Sischery Q== 963 m/s

Canel sloper Iz = 15000

Flow area. Az = 14.69 Mi

Velocity V2 = 0.6597/s

Velocity head hos = 0.022m

1-2-6-3 hoss head of transition

11) Love head of gradual enlargement

 $k_{j}(-f_{j}(k_{v},-h_{v_{z}})$ = 0.3 (0.099-0.022) = 0.023

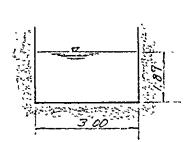
12) Less had of friction

A+ = = (I,+I.) L

Z (1000 + Faco) x 20 = 0012

1-3. Hydraulie Caluculation for Main Structure.
of 5-Route.
1-3-1 No.3 Irrigation Aqueduct.

1-3-1-1 Typical section of irrigation agreeofuct



Discharge Q = 8.65 m/s

Lanal slope I = V1.500

Lorfficient of rough new N = 0015

Land wedth B = 5.750

Hoter Lapth H=189

Flow area. A: = 3.00 × 189 = 5670

Wetted periode ? = 3.00 + 1.89 × 2 = 6780

Hydraulic radius R: = N/P; = 567/6.75 = 0836

R; = 6837

Velocity v; = /m · R; = 1/6 = 1/6 × 0837 × 06258

= 1.526 m/s

Velocity feed hv; = 28/2.9 = 1526/2×98 = 6119 m.

1-3-1-2 Hydraulic elements of open canal

Suchange $Q_z = 865^{m/s}$ Canal slope $I_z = 15.000$ Filow area $A_z = 12.81^{mz}$ Velocity $V_z = 0.677^{m/s}$ Velocity head $V_z = 0.023$ 1-3-1-3 Loss head of uppen transition; he,

Transition length L=18.00

Led = fgc (hvg - hvz) = 02 (0.119 - 0023) = 0019

12) Loss head of friction

flet = /2 (I, + I,) & = 1/2 (1.500 + 15,000) × 18.0 = 0.008

(3) Total loss head of upper transition hor = hod - hod = 0019 + 0008 = 0027

1-3-1-4 Loss hend of lower transclion; her

Transition length L= 1800

hoi = fgc (hv; -hv.) -= 03 (0119-0023) = 0029

(2) Loss head of frection

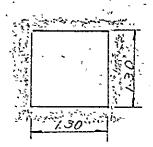
hal = \frac{1}{2}(I, \tau I)\land = 0.008

(3) Total loss head of lower transition

her = hor + hel = 0.029 +0008 = 0.007

1-3-2 No. 2 . Irrejation Syphon.

1-3-2-1 Typical section of irregation syphon



Qischarge $Q_1 = 2.00^{mV_5}$ Cerf. of nonytimess $\pi K = 0.020$ (and width $B_1 = 1.30$ Water depth H = 1.30

Flow area A, = 13 H = 1.30 - 1.30 = 1.69 = 1

1-3-2-2 Hydraulic elements of i type open canal

Descharge Q = Z.00 m/s

(and slope Iz = 15.000

Filow area Az . 4.059 m²

Velocity Vz = 0 498 m/s

Velocity hand hv = 0.013

1-3-2-3 Hydraulie elements of D type open canal Dicharge Qs = 200 % (Canal slape 23 /5.000.

Telow area A3 4.260 M

Velocity V3 0.474 M/S

Velocity hand his = 0.01/

1-3-2-4 Loss head of representation : her Transition length L-500 (1) Loss herd of entrance

- (1). Loss head of exclasico Re=fi(hv,-hvz) = 0.5(0.07/-0.0/3) = 0.029
- (2) Lass head of friction $A_{i} = \frac{1}{2}(I_{i} + I_{z})l$ $= \frac{1}{2}(0-00020 + 0.00250) \times 5 = 0.007$
- (3) Total loss hand of vyyez transition. he; = he As = 0029 + 0007 = 0136

1-3-2-5 Loss head of bours transition; her ... Transition length L=500

- 11) Lose land of exit

 hom for Chin Riv.)

 = 0.7 (0.07/-0.011) = 0.042
- (2) Long head of friction $\begin{aligned}
 \mathcal{L} &= \frac{1}{2} (L_1 + \Sigma_2) \mathcal{L} \\
 &= \frac{1}{2} (00002 + 000250) \times 50 = 0007
 \end{aligned}$
- (3) Total loss of Lower transition:

 1.62 = ho-he = 0.042 + 0.007 = 0.049

1-3-26 Loss head of syntion culvert length L=50mm ... Rf=Il = 0.0025×50=0.125mm

Taking 10 president allowance.

1-3-3 No. 1 and No. 2. Parigation Culvert.

1-3-3-1 Typical section of irrigation culvert.

Coscharge
$$Q = 865/z = 432$$
 S

Canal slope $I = 1/2.000$

Coof of noughness $n(=0.020)$

Canal width $B = 250^{-6}$

1 When depth $H = 188^{-10}$

Flow aren
$$A_1 = 250 \times 188 = 470^{22}$$
.

We that permeter $P_1 = 2.50 + 188 \times 2 = 6.26$

Hydraulic radius $R_1 = 470/6.26 = 0.751$

R\$5 = 0.826

Velocity $V_1 = \frac{1}{0.020} \times 0.626 \times 0.02236 = 6.923^{27/5}$
 $Q_1 = A_1V_1 = 4.70 \times 0.923 = 7.4325$

Valuidy hand the $V_1 = V_2 = 0.923^2/219.8 = 0.043$

1-3-3-2 Hydraulic elements of open canal

Liescharge $Q_1 = 865^{m}$ %:

Land slope $I_2 = 1/5.000$ Flow area $A_2 = 12.81^{m^2}$ Velocity $V_2 = 0.677$ Velocity head $h_1b_1 = 0.023$

1-3-3- Loss head of upper transition sher Transition Length L=10.00 (1) Loss head of gradual contraction hod = fgc Chu-hve) =02 (0.043-0.023) =0.004 hit = \frac{1}{2} \times (I, -I_2) \land (2000 - 15000) \times 10 = 6004

a) Total loss head of wyper transaction her = hed - hel = 0004 - 0004 = 0.008

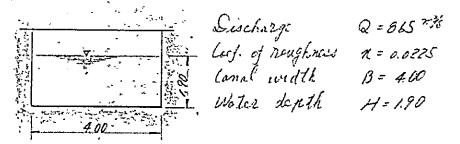
1-3-3-4 Loss bend of Lower transition; her Transition length L= 1000 (1). Loss head of gradual enlargement hos = fgo(hvi-hvs) = 03(2.003-0023)=0006

- (2) Loss heid of frection

 find = $\frac{1}{2}(2_1 + 2_2) \cdot l$ = $\frac{1}{2}(\frac{1}{2000} \frac{1}{5000}) \times 10 = \frac{300}{200} \neq \frac{1}{200}$
- (3) Total loss head of Lower transition her has + has = 0.006 0.004 = 0.010

1-3-4- No. 5 n No. 8 Bridge. (Type.B)

1-3-4-1 Typical section of irrigation canal under Exige



Flow area $A = 1.90 \times 400 = 760^{12}$ Wetter perimeter P = 4.0. + 1.9012 = 780Hydraul nodius $R = ^{1}/P = 7.60/780 = 0.974$ $R^{1/2} = 0.983$ Velocity $V = ^{1}/^{2}/^{2} = 0.138$ Velocity find $V = ^{1}/^{2}/^{2} = 1.138/^{2}/^{2}/^{2} = 0.00068$ Hydraulic $I = (\frac{11.5}{R^{1/3}})^{2} = (\frac{0.0225 \times 1138}{0.983})^{2} = 0.00068$

1-3-4-2 Loss head of wrighton conclunder bright

Bright width B=5.00.

$$flge = fgc \cdot hu$$

$$A = \frac{7.66}{12.81} = 0.60$$

$$= 0.21 \times 0.066 = 0.014$$

$$fgc = 0.21$$

12) Loss head due to sudden enlargement.

Asc = fre ho Ash; = 0.6 = 0.16 0.066 = 8.011 fre = 0.16

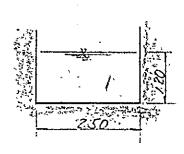
- (3) Loss wood of friction

 he = I.l = 000068 × 5.00 = 2003
- 4) Total Less head of irrigation canal under bright

 hs hgc + hic bis
 = 0 014 + 0011 0003 5000

1=3-5 Nã.9 Bright (Typo C)

1-3-5-1 Typical section of war got or carel under bridge



Sischarge Q = 2.00 % (orf. or noughness of = 0.0225 Canal wedth 13 = 250 m whiter depth H = 1.20 m

Filew are $A = 250 \times 1.20 = 300^{m^2}$ Without periantes $P = 250 + 120 \cdot 2 = 490^{m^2}$ Phydraulic nadius R = 3.00/490 = 0.612 $R^{3/3} = 0.721$ Velocity $V = 2.00/3.00 = 0.667^{m/8}$ Velocity head $hv = 0.023^{m}$ Hydraulic gradient $I = (0.0225 \times 0.667)^2 = 0.00043$

1-3-5-2: Loss head of irrigation comme under brigde

Bridge wirth 13=500

(1) Loss head due to sudden contraction

2) Low hard due to sudden enlargement hse = fso hv fse = 0.09 = 009.20.023 = 0.002

(4) Total loss head of irrigation canal under lingdo

he = hg here he = 0.002 = 0.00 3 + 0.002 = 0.007

1-3-6 HOLZ Transition (Bryps - Czyne)

Transition length L=150

1-3-6-1 Hydroulie elements of B type open conal

Awakaye Q1 = 9.63 7/6

Canal slope I, = 15.00.

Flow area A, = 14,69 m2

Velocity vr = 0.659m/s

Velocity had her = 0.022 m

1-3-6-2 Hydraulic elements of Ctype open comil

Sixtage Q1 = 865 7/8

Canal slegge I: : 15.000

Flow area. Az = 125/m2

Velocity Vi : 1.67.7 m/s

Velocity hand his = 0 023"

1-3-6-3 Loss head of transition

11) Loss head of gradual enlargement

hys = fg (hie, -hrz) = 0

12) Loss hard of friction

hy titing)

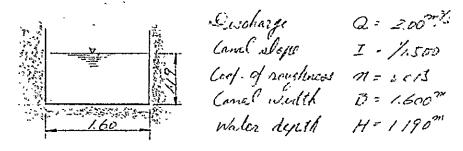
= = (Jon + Joos) + 15.0 = 0.003

. (3) Total loss head of transition

hi = m

1-4 Hydraudie Calculation for Man Stracture of C-Route

1-4-1 No.4 Insigntion Aquicalent
1-4-1-1 Typical section of irregation associated



Islaw area A = 1.60 × 1.19 = 1904

Welled perioreter P = 160 + 1.19 x 2 = 3980

Lydraules R = 1.96 + 1 = 950 = 0478

R = 061;

Valuedy V = 0615 × 0.611 × 0.258 = 1.05/1/2

Velocity head hv = 0656

Q = 1904 × 1.051 = 2001 > 200 = 250 o K

1-4-1-2 Hydrockie clements of D type spen canal Suchenge Q== 2.00 m/s Canal slape I== 1/5.000 Felon area A== 426 m= Velocity. V== 0.470 m/s Velocity hand hv== 0.011 m

1-4-1-3 Loss head of upper transaction 3 her Transaction length L= 10.00 11) Loss head of gradual contraction had = fgc (fiv. - hvs) = 0.2 (0056-0011) = 0.009

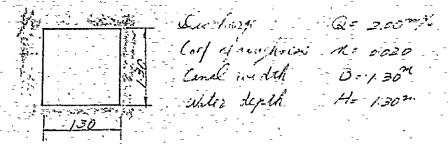
- (2) Laso head of friction $R = \frac{1}{2} \times (I_1 I_2) \mathcal{L}.$ $= a5 \times (\frac{1}{500} + \frac{1}{1500}) \times 10 = \frac{2}{6} \times 64$
- (3) Total loss head of upper transition he, = hed + he = 0.00 / + 0.04 = 0.013

1-4-1-4 Low hard of lower transition 3 her Transition langth L=1000 (1) Low hard of gradual injurgement hot= fs. (hv.-hv=) = 13 (2256-2011)= 0014

- (2) Lass head of fruction $f(I = \frac{1}{2}(I, +I_2)l$ $= 0.5(\frac{1}{5.00} + \frac{1}{1.500}) \times 10 = 0.004$
- (3) Total loss hand of lower transition her = has he = 0016 + 0.004 = 0018

1-4-2. No. 3 Invigation Syphon.

1-42-1 Typical section of irregation synthem



Filou area A1 = BH = 130 - 130 = 169 = 183 7/2

Velocity V1 = Q1/A1 = 20/189 = 1183 7/2

Velocity head hu W/2 = 207/

Wetled perimeter A 5 Z

Hydraulic nadius R = A1/F = 1169/50 = 0325

R 7 = 0473

Hydraulic I1 + 000 + 1183 = 000 > 60

Hydraulie I (002 x 1.183) - 0 00250
gradient 0473

1-4-2-2 Hydraulie skements of Dtype open canal. Lischarzi Q = 2.00 mg/s Canal slope I = 15.000

Flour area A = 426 200

Velocity V= 0.474 7/5

Velocity head hus 0.011 m

1-4-2-3 Hydraulia elements of irrigation culvert.

Discharge Q3 = 2.00 %

Cond alaper Is 12 000

Filow area. A3 = 2620 %

Velocity 3 V3 = 076676

Velocity had his 0030 %

1-4-24 Land hand of upper transition shell Inancition length L=535

(1) Low head of entrance.

to fi (hu, - Au)

= o \$(0.071-oc11) - 1030

(2) Love head of fraction

M: \$\frac{1}{2}(1-\frac{1}{2})\land \tag{2} \tag{2}

13) Total for hand of upper transition. how he - he - he = 0.030 - 0.007 = 2037

1-4-2-5 Loss had of Source Transition 3 hos Transition length 2=5%

(1) Loss food of excl.

ho=fo(hu-hu=)

= 0.7(0071-0030) = 0.29.

- (2) Loss had of frection : - les = = (I - I =) 9 - ot (00021 + 0005) x5.0 = 0.008
- (3) Total franches of lower transition her = ho + he = 0.029 -0.008 = 2007

1-4-3 Hayn No 11 Insignation Culwert

1-4-3-1 Typical section of exception culwert

Concharge Q = 200 m/s

Conf. of roughness 11 - 0020

Conf. of roughness 11 - 0020

Conal width 13 = 200 m/s

whater depth H 1 310 m/s

1-4-3-2 Hydraulia elements of DType open carel.

Lischarge $Oz = 2.00^{m/3}$ Canal slage Iz = 1/5.000Filew area. $Az = 4.26^{m/2}$ Velocity $Vz = 0.474^{m/3}$ Velocity head $f_{NS} = 0.017^{m/3}$

1-4-3-3 Loss head of upper transition ther.

Transition largeth L=10.76

11) Loss head of gradual contraction.

Red = fjc(hv-hvz)

=0.2 x(0000-0011) = 0.004

- 12) Loss had of friction $\hat{R}d = \frac{1}{2}(I I)\hat{C}$ $\frac{1}{2\pi i}(\frac{1}{2\pi i}) \times 10 = \frac{1}{2\pi i}$
- 13) Total Loss head of egyet Transition. Res = hed - hel = 2,000 - 2 004 = 3008.

1-4-3-4 Loss had of Lower bransition ; here

Transaction length 1= 10.00

(1). Loss had of gradual enlargement.

hox = foc(hu,-hor)

= 0.3(0030-000) = 0.006

- 12) Loso had of free time

 fiel = = = (500 - 000 = 0004
- (1) Total lass hard of lower transition for = har his = 0.006-0.004-0010

P.33

-						 		F A-		,` 				
It	em	Discharge	Distance	Name of Canal	Velocity	Slope of Canal	E lergy loss	Elevation of Cenergy line	Velocity head(hv)	Elevation of water surface	Hydraulic depth	Elevation of canal bottom -		Remarks
Station	iension į	ni/sec	n n		m/sec -	} } 1	. m _	Elmi	m _	Ľ! m	m —	El-m		
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	·	2.630	44450	Type Bopen Conal	0.659	-1/5000	0089		J 6.022		1.500] - !	de la Maddin et s. de la destination annual service.
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12 - 155	<u> </u>		Lover transition	· · · · · · · · · · · · · · · · · · ·] 	٤	- 85/	*	335.731		334.47/	L was to the same of the same
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·	2.630	1.269.00	Type B open Canal	0.659	1500	0.354				,	1.300		NO3 (d let 5017 - 58) Q=1361 Contrac
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57-11

Hydraulic Calculation for A - Route

/ Sheet of 3

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Item	Discharge	Distance	Name of Canal :	Velocity	Stope of Canal	Briergy lo.	La Eledati La epçini	es br	Verocity (headthy)	Lievation of water surface	Hydraulic depth	Elevation of canal bottom		
Dimension	m' sec	m		m/sec		i m	in the second	n di	/***	EI-m	m	El-m	.*	Remarks .
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,		20	Type C openan!		1/2	· • · · · · · · · · · · · · · · · · · ·		1-			. 332 300	,	Ordlet (Ft 32-15)	
		7.037.00	Type C often came	- B677	La Pier	4.5/6		:	engagere de da	2070	The second section is also	NO.10	Outlet (50.52.535) Outlet (50.38-150	2) R=0016 3
D34 + 850							333,282		333814	- 	231,974			i di Li iliani dang kanggangang panggang manggang man
	ر از داده از از پیشندسیا	5.00	No.6 Bujde		-				: 				No. minima of	
7866						. †	393.8 c?	1.	333,835		_331.946	1		- N 44 V W
- 5	7150	399 Ar	Type c openional	1177	Veren	1-180-		1		: 1.89a	-			
			i productional	- 1 m	7.3.000		1	}_		Company of the party	,	-,		ever en <u>Proper</u> — eue un
035_t.850_			ا المان الما			٠ ين ـــ	333.779	1	.000,75 E		_331.966			
		5.00	110,0 - Brigde		, <u> </u>	1. (28)			w.a					
		-	1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 -		.i .am-marsa -	· · · · · · · · · · · · · · · · · · ·	23752	1	333,725		33/238	m e e e e e e		
	8.650	688.00	Type C open Cand	. 1.67Z _,	15,00	1.138		L 2Z3		1 890		110 12	Cullet (KD 21 - trai	Q = 0. oll cury
P38+474		' '	,	1 3	į	· - ,	227.619		333,580					
	_	19.00	Upper transition of 40 3 Agridad			1.027.		17		<u>-</u>			•	
		70,00	Ly pro S A gestude	ست مستنسبستون هو د	ojas summit sa S	1. 1.6-7		i i			•	• •		m. mås
r 533		- ~ }	Tipical station			The same was	137.15K	-	333,467		331.577	j		
	2.150	31.00	Tipical station of NO. 3 Agranded	1.526	11500	0 621	٠ و المار	0.119	- •	. 1 290		j		
7600	-						- जुरु स्था <u>र</u>		. 333,446		331.55K	-		
	7	18.00	Lower transition placed and A questuel		<u></u>	•							,	
- 194	•				\$		333,528.	i				1		
·#=	0.00	,		ا مدسود در داری	1/	· •	' ' " 2					,		THE THE PERSON AND THE PERSON AND THE
		205,50	Type Coupen com	-,0 £17	15,000	1141				1.890			<u>, , , , , , , , , , , , , , , , , , , </u>	nantan gajir y — wa w desima.
10 T 600		-		-	1		15.407	1 .	333, F8.4		331494			and graphy. We have the state of the public resource in any angular
	•	5.00	No. 8- Brigde		- {}	0.028	المراجعة ا المراجعة المراجعة ال	;		·	4 3 -4			
7 616	-				ŧ	1	333 378	•	333356		331466	.+		
,		1			. ,			u → c 1		- •	,			

Dimension.		Distance .	Name of Carol							*		·	!
	<u> </u>		Name of Canal	Velocity	Slope of Canal	Ene gy loss	Elevation of energy line	Velocity head(hv)	Elevation of water surface	Hydraulic depth	Elevation of canal bottom	1	D o
	nt/sec	m	· · · · · · · · · · · · · · · · · · ·	m/sec	:	·	The state of the s			, m	Ēl-m	; ;	Remarks
040-616					1		333.377	2	333.356	T	331466	.,	
			,	_	-			3		5		1013	Outed (RD 43-30) Q= 0.004-Carpe
D.44+400	·	• <u> </u>							i			, .	Dutlet (contract & = 2.585 cm/see
	8.65	39735	Type Copier Canal	0.677	1/5,000	0.079		0.023	},	1890	; <u> </u>	1 1	
2045-549		; 	Upples transition of No.1 Culvert	THE THE PROPERTY OF THE PROPER	} - ,- ,-	, !	333 069		333.046		331.156		
' 		10.00	of No.1 Culvert			0.008		-	,	to the state of th	- - -		
+602		,	,		7 *** ***	ţ, ·	333.601		333.018				·
		30,00	Typical section of Ho. 1; Contract		1/2,000	0.015						-	- A. Al. of the minimum of annumber of annumber of
+700			Lover transition		· ·			= ;	333 093	' <u></u>	<i>33/./23</i>		
		10.00	Lover transition of No.1. Culvert.		-	0.010	, et (- 1) 2 I	†	1		-		
<u>+733.</u>			ر. المست المستند بدر الأمار الأمار المراجع		,	i .	333,036	\$		•	33/./SE		
		426.25	Type C open Canal.	0.677	1/5,000			1				,	
D47+365			Opple transition of NO 2. Culvert	-	; ; == - == * }-		332 957		332 9 <u>2</u> 8		331038	_	
	*	10.00	of No 2. Colvert	a care	·	0.008			1			-	
+378	· · ·	00.00	Typical section	•	1/		393/43		332.900	, I	331.020		
+660		80.00	. r.s.No.Zneutverl		7=2000.	0,040		,		1.880		, , ,	, , , , , , , , , , , , , , , , , , , ,
r' [,	10.00	Lower transition	and the first and the first section.	1		332903	4	1.332.860			1	
- 493	** * **	70.00	of NO 2 Culvert		'		333 V9 3	-	 970		 	` !	unin na sana n
1		15560	Twee Court Court	•	1/1000	,	1995	0.023	332,07	1.290	J. 0 / 0.0	ļ., !	
+874		. 00, 0	Type a careful state some		, , , , , , , , ,	1	335882		23.2	· · · · ·	-730 767	; :	
,		10.00	Upper transition	- Note: I P	•	0.008	3		The second secon	,	, , ,	1	
+907		4	9			1	33 2 874		332.857	,	330 971		
		26 00	Typical section of NO3 Culoust.		1/2,000	0.013		0.023		1.880			, a
+992	44. t.	- - - - 1		,		í •	332864	.,i	332 838	, -	330.958		
lotes			•										·

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		•			,	- 7. '				٨.	a			<i>p38</i>
Item	Discharge	Distance :	Name of Canal	Velocity	Slope of Canal	Energy loss	Eleva	stion folk graline	Velocity head(hv)	Elevation of water surface	Hydraulic depth	Elevation of canal bottom		Remarks
Dimension tation	<i>ni√</i> sec	m	ي ويندي ممسايي د	m/sec		m		lim F	i in	El-m	m	El-m	3 3 4	-
1047-1982		·		 	* !)		861	<u> </u>	3 <i>3,23,2</i> 38 د وج _{دیا}		-330.958	7 7	
ale en especie de la		10.00	Lower transition		} !	0.010				i i	al and a Principle of Principle (A).) 	
1048 + 25	lļ	- 1				; 	الجريوني ا	801	6.023	332.828	1890	330 938	9 3	-
2018 - 11	2.6S_	345.26	Type C gren Caral	0.318	15,000	6.069	دُرُد ا	182		*332 759	(6 8,00)	_	(g. 4.9) 	
D49 + 41	, , ,	·	Upra transition		<u>}</u>	0.036	1 - 4 - 3 - 5	742	N.	(33). 729) !)	330.869	į	
+57		_3.00	A Company	Turan I bulan		1	33/	7.6	1	· · · · · · · · · · · · · · · · · · ·		,		alem no 14 m 14 de n. m. 19 m
	*	50.00	202 Synhon.		,	0.138				***		<u> </u>		ж
7221	;				1		33/	£68_	<u>.</u>				· s	
) 	5.00	Lower transition . of NO2 Syphon.	4	3 0.	6.049			- %-	-		•)	
+ 237	}					· -	33/	519	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	33/508	. •	330,278		** * - ** - ** - * -
2054+772		1,803.80	Type Dopen carse	0.474	1. 15.000	0,36/	1		0.011		/230 N		,	
<u>D-7-112</u>	3	5.00	No. 9 Budge		_ 	0.007	ر الدين المارية المارية المارية المارية المارية	1. 1.5.0	- A-	331.147.	· *	329.917		*
+788		1			:	p } Sank Sant 2 , p project	133			331.140	-	3=9.910	•	
-		1.108.73	Type Dopum carel	0.474		0,222	i singre.		10.011		1230		1	
20.58 +588		; !;			ı		اج ج	r.729.	-	330.918	Χr	327.688		·
	3			, ,		•			1	-		tr .•	·	
Wheeler w a na		1 - 1			}	1			1				1	-
an the district and an including and an in-		 ¥			! }				-	-	←			• • • • • •
			· · · · · · · · · · · · · · · ·		; ;		American in the second		,	-				<u> </u>
		s B g management age anglesser s	no because of the same of	·] ·					.*
pr	·		And the second determinence of the second second		•	•							· · · · · · · · · · · · · · · · · · ·	· · · · · · · · · · · · · · · · · · ·
		, ' }	**************************************	Name of the same o			Tarana Marian Tarana Marian Tarana						-	
Notes		! , !	The state of the s				• ^5	u u u u.						
	,						F -					٠		•
		, 7a.										¶,		4.4

							i					P.39
Rem Discharge	Distance	Name of Capat	Velceity	Slope of Canal	Forgs loss	Elevation of a	Velocity head(by)	Elevation of water surface	Hydraul'c depth	Elevation of canal bottom	,	Remarks
Dimension m' sec	m		m/sec	1 1 2 2	, m	Æi-m,	m un	Elm	, w	g El-m		
NO 6-3391		· · · · · · · · · · · · · · · · · · ·	u Brown, and a state of		<u> </u>	1 1	1	330 918				
r		Type D. open Canal										
NO3+550		Upper transition	And ran weeks and recom-		0.008	330.877	- <u> </u>	330.786		329.656		
		· · · · · · · · · · · · · · · · · · ·				1 4 * 5 * 4 * 4	, server	. 330 . 859		329,549	- ·	
46.3-13.50		of No. 4 Culvert.		1/2,000								
NC 4+ 1.50?		Lower Transition	ganna hayba — aa assam — aa		1	336875	e de deservición	330.845		32853 <u>5</u>	-	ين به سد به سد به د
	1	And Cucueu.				339,865	,	30.854		329.624	-	
3 1	[در	Type Dopen Canal		1,000	•		,	•	123.			
NO 25 + 3.50;		Upper transition		l		330 677		330,683	- -	329413 	-	
		of 405 Calvert		·	6,009	330696		330,456		329.5 4 6		
> /3.50	20.00	Typical section Lef No.5 Culvert	•	1/2,000	6.010		10030					
33. <u>S</u> E		Lower transition			1	330.676	<u> </u>	930.646		329.336		- ANT
40.26.73 D	10,00	of No 5 Culved.	4 V		0.0/0	330 666		330.655				
2.00	1119.08	Type Depen Canal	A. A. P.M.	1/2000	0,324		. 0611		1-23			
NO 53-25,00°		Oppor transitions	. -	, , -	' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' '	336 343		330 431		329.201	~	
+30.00		U	* *	-	0.008		·	220 40 4		33.C a.C.1	-	
	15.00	Tyrical section of No. 6 Colvert.	4 м. тар 	1/2,000	0.008	3301.434	•	350,404	1.31	329.094 .		
NO 54+10.00	1	Lower transition		·	· 	330 426		330 <i>39</i> 6		3-9.086		
ji	10.00	. g. No 6 - Calvert.	e de contra sua	•	0.010				`		••	
+20 00		v <u></u> v			·	530416		330 405	<i>j.⊇3</i>	329.175	-	
Notes												
												•

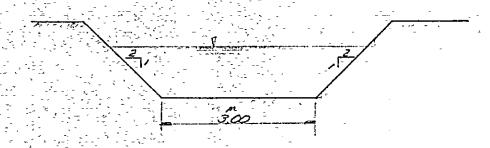
***************************************			*		· · · ·	and and the second				*** *** *** ***			and the same of th	P40
,	Discharge	Distance !	Name of Canal	Velocity	Slepe of Canal	Energy loss	Elevai Energ	ion of,	Velocity, head(hv)	Elevation of, water surface	Hydraulic depth	Lievation of canal bottom	Remark	· c
Dimension Station	m/sec	I m	e e e e e e e e e e e e e e e e e e e	i m/src	*	i ni j		l-m		i Elm	tn.	Ľl-m	Romar.	
NO 54+20.00		· 			} } }	() () () () () () () () () ()	330	416	-	330,20 f		327.175	ganganana way wa manana sa manana yan	and the second of the second o
- Maringaning sam was a services		1346.36	Type Dipun Conal	And the state of t	1/5,000	0.269			0.011	, wn	1.23		9	
NO. 88 + 35.00		ا ہے۔ از عماد مسیدات از	Upper bandier		, <u></u>	} 		727 (***)		, 330,136		322906	,	
Ne 99+5 60	· · · · · · · · · · · · · · · · · · ·	10.00	of NO.4 Agricatual	to the second of		0.0/3			; .					
New Y	uan are u u ga u	10.00	Typical scation		1/1 000	0.027	332	1334.	0.00	: 330.078		328,888	·	
+45.00		<u> </u>		1	/							328.861	4	4 44 44 44 44 44 44 44 44 44 44 44 44 4
		10.00	Lower transition of NO.4 Aguadust		, -	0.018.								
*+ 55,00	مهد مددمها معدد				} }	1	338	-0.29	; ** ; ** *	330.078	અ	328.848	; ▼ T	
		171.96	Type Dogen Comil	e u La managana managan ungan kangan La managana managan kangan	15,000	0 034			0.011		1.23	-		
NE93+30.00	***		Exper transition of NOT Culvert	-			337	25	-	330,044		328.814		
10.94	and the same of the same	10.00	-of NO Culvert	for the same of th	,	0.008	1.0				٠.		,	
DC9, / 4		652.06	Typical section of No.7 Cularit	dans note to the second of the	1/2,000	0326	330.	ca:/.		33C.017		338, <u>20</u> 2		
NO.110 -				1		!				329.691	1.31	328,381		u u
in the man supplied and supplied and		10.00	Lower transition of NO7. Culiert:	. :		0.010		in the second of	and the second	-	,	1 -		
+10.00	• •						329	74.	.* .¥	329.700		32 8.47n	; ,	- -
grammings for grown		12.6.00	Type Dopen Care	l.	15,000	0.025			0.011		. <i>1. 23</i> .	,	NO 13 Cutled (NO 113)	2 =
<u>Nº.</u> [13-]5, 60	مين ويوني بيناوس ميس		Upper transition					186	1	329.675°	•	3=8.445		
+25,60	a Managaga ga ga	, , , ,	ELINOS CULTURA	The state of the second		0.008		628_	•	329,648	•	328.338	<u> </u>	
)	-	3000	Typical section of NO. B. Lulvert	Salar	1/2.000	0.015			2.030		1.31	2-0.770	<u> </u>	
NO.114-1500	ما المانية الم		Tanga to and the	1 1	and the second s	" () 	ري پرچق	:663		329,633		328,323		
	ا وستنده مسلمدر ۱	10,00	Lower Transition 7 No. 8 Culner			2.010	The second secon		2	•		-		
T25.00		· · · · · · · · · · · · · · · · · · ·	The second secon		gand states _		309	<u>253</u>	* ,	329642		328.4/2		
Notes	ì	<u>المنافقة</u>			······································	*			-0.011	,	<u>/, 23</u>	······································		
	•	*	· · .		•			A .						
		· · · · · · · · · · · · · · · · · · ·	اره					* - ** ·						

				•	٠, .	وم رياد مرياد مالا				· 		•		P41
·	Discharge	Distance	Name of Canal	Velocity	Slope of Canal	Energy loss	Elevat energ	ion of a	Velocity Thead(hv)	Elevation of water surface	Hydraulic depth	Elevation of canal bottom	Remar	le o
Dimension	nî/sec	m		m/src	· · · · · · · · · · · · · · · · · · ·	·m · -	i A Vei	m V	i in	El-m	m m	El-m	-, Kemar.	7.5
S'a'ion	4		يادي وياد ما ما ما ما	e Historian de en la marie En la marie		يُّ يَـ حَيِّهِهِ مِعْمِدِ أَ			ing Angeles in the limit In the limit in the	j r		v.a		•
NO 110+24.00)	·				<u></u>	, <u>3</u> 29	<u>(£2)</u>		329.642	<u>.</u>	-328.4/2		
	· 	230,00	Type Dopen Cond		1/5,00	0.006	وسفادت الراد		0.011		123	·		and the second of the second o
NO 120+15.00	,	l	i		}	·	329	617		329.596		328,364	, , ,	
•	۲ ۲	: <i>s00</i> 1	Upper transition	<u>.</u> !	; ;	0,037	1 45 K		1				1	
+20.00	}			,	1	· , • • • • • • • • • • • • • • • • •	1		·	: <u></u>			. 1444 - 44 144 144 144 144 144	
	• • •	1		and the second						1	~ •• •		units	· · · · · · · · · · · · · · · · · · ·
,,,	i gan gurannas mas i	30.00	103 Synthen	10.025 X 20X		1.0,08.3			Ţ	· ·	.,		1	
NG.131-18.00	·		The Transition				. 229	487					ili Litarana manana manana I	
	<u></u>	5.00	Lower Transalion	0.002	<u> </u>	0.037			; ,,					
-15.00	, ,,,			a *	1		329	150		329020	>	.328.110		
<u> </u>	, t	310.26	Typical section of No 9 Calvert.	}	1/200	0.155			0.030		/3/			
	1	!	- The Mark of the Contract of	eminima in approximate	, ,,,,,,,,	1		مانشان برا معد رود مردو	0.000				- ·	
EC 21_	+	·	Lover transition	<u></u>			328	575	1	329.265	-	327 955		
	4 <u></u> !	10.00	of No.9 Calvert	1	` **	0.010			1	****		•	Ap	Minus Minus
10.129-22.78	t 1		an an an anne a	1 	: 		3.35	285	1	329.274		328.044	2	
		157.22	Type Dopun Conac		1/5,000	0.031	3 3	ا د مشرسید داشد .	0.011		123	-		<u>.</u>
NO.133 - DC.CO	1	•		!	1	1	و المراجع الما	250	_	329243		308.0/3		
	i	1000	Upper transition	•		- 008	1 % 4.		i-			2 - 1		
		1	_of NO.10. Culvert	, , , , , , , , , , , , , , , , , , ,	i '	. E.008	A		•	٠				
30.00		·	Typical section		1		329	346	ł.		• · · •	327.806	-	*
	ì	20.00	of NO10 Culved.	*	- 12,000	0.010	1,000	1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1	0.030		_ 13/			· · · · · · · · · · · · · · · · · · ·
10.134-10.00	t	i	**************************************			,	35°7.	٤٠٠٤ .	:	329.206	-	, 327.896	!	
to the companion to the contract of the contra	Y	10.00	Lower transition of NO 10 Culverd_	}-]	0.010			<u> </u>					
720,00		!		1.	!	ì	3 3 4	227	ļ.	==9;=15		3>7585		_
		100 10.	Zaz 12 a Const		1/0 000	6,034			1			0-/24-	•	
		770,00	Type Dopen Caral	` w. ' w.m.	, 13,000	6,034		and the state of t	6.011		1, 2,3			
NO.138-30.00		, ,	Oppor transections			- 4-100	329	192.		3-9.181		327951		
	w.	10.00	gNo.11 . Culued		*- }	.0.008	2	ر مارون مارون برامورد مارون برامورد	4		-			#1# - " - A 60 TH N WARRANT
No.139		}		and the same of the same			327.	184	<u>į</u> .	329.154		227.844		
		: i <u>(</u>		· · · · · · · · · · · · · · · · · · ·	<u> </u>		107 MIN 107	7.7 4.	. c.e.30		1.31			
Notes			The state of the s				4 3 3 4 3							
								Carrier St.						
		4												

	-,		;			<u> </u>		10 65	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	· · · · · · · · · · · · · · · · · · ·			P
Item Dimension	Discharge	Distance	Name of Canal	Velocity	Slope of Canal.	Energy In-	E'evation energy	in of 3 tine 3	Velocity head(hv)	Elevation of water surface	Hydraulle depth	Elevation of canal bottom	Remarks
ation .	m, sec	m	/	m/sec	1,	in *	El-	n di	J . m	El-m	m	Ll-m	Remarks
10 139 _		- •• •		1			3-9.1	90		329.154	, * * * · * ·	327.849	· · ·
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1-6 H- Q Curve for Banda - Sangam Branch

(1) Typica Section



(2) Hydraulic culculation

Canal width

Water depth

Canal slope I = 1/5000

Cofficient of roughness n = 0.0225.
Water area A(m).

A (me);

Water perimeter PCM

Hydraulic radius R (m)

Velocita

O = AV (misser) Discharge

i ++>	.=	+ /~ * 1	<u>.</u>				
H	A.	P	R -	R ⁷ /3	1/2 5/2	u	, Q
(m) 0.20	(111)	(m) 3.89	(M) 0.1.75	0.3/3	0.629	0.197.	11/500 0.134
0.40	152	4.79	. 0.317	[> 8 ₂		0.292	0.444
0.60	252	5.68	0.444	0.582		0.366	0.922
0.80	3.68	6.58	0.559	0.679		0.427	1.571
100	5.00	7.47	0.669	0.765		0.481	2405
1.20	6.48	8.37	0.774	0.843		0530	3034
1.40	8.72.	9.26		0.916		0.576	4.677
1.60	9.92	-10.16	0.976	0.984		0.619	6.140
1.80	11.88	11.05	1.075	1.049		0.660	7.841

1-7 maximum Flood Computation

The maximum flood is computed by Rational formula and Dicken's formula.

(1) The maximum flood is computed by Ratural formula.

0 = 02778.f. r A

there O; mucemum discharge in Misec

f: Coefficient: 065

1: Critical intensity of ramfall por hour in the Catchment (months)

A; Catchment area in Km

 $\Gamma = \frac{R_{24}}{24} \left(\frac{24}{T}\right)^{3}$

Where Res; Rainfall per day is 85 = 2159 may

T; Critical time of flow for critical point in catchment to the discharge point. (tir)

T = 4W

where Li Length for the critical point on Catchment to the descharge point (KM)

W: Velocity of maximum floor. KM/TW

W = 72 (4/2)0.6

where H; Difference in elevation from critical point to the discharge point in meters.

las The maximum flood for A and B route

بتر	Н	2	4/2	W	T		02778 f	Q
(KIT) 0.061	(m) 6.4	(KM) 0.38	0.01667	617	(#f) 0.062	(7K#/4s) 281.97	0 1806	(Mysec) 3.1
0/21	5.0	0.38	0.01310	5.34	0072	26986		5.9
0.769	16.5	1.65	0.01000	4.54	0.36	146.90		204
4047	9.0	3.00	0.00300	2.20	1.36.	6098	*	446
15 02	15.2	6.44	000236	1.91	3.37	33.31		904
120.18	مح جي جي	12.87	000260	2.03	6.34	2/85	,	474.2

(b) The maximum flood for c route

A	Н	۷	7/2	W	T	r	027785	0
0.07	(nt) 0.24	48	0.0003	KM/48 0.55	Tir 1.455	1114,1 5830	0 1806	m3/sec 0.73
0.23	0.39	13	0 0003	055	2.364	4218	,	1.75
0 28	042	1.4	0.0003	055	2.545	4015	,	2.03
0.43	054	18	0 0003	0.55	. 3273	3395	,	2.64

12) The maximum flord is computed by Dickens formula.

where O; Maximum discharge in 51% sec C; Cofficient 1400 A; Catchment area in miles

Α,	Az	A,364	۵,	Q2	REHARKS
(K)K) 0.061	(miles*) 0.023	0.059	(50%m) 82.60	(niskee) 2.3	/ KM2 = 0.3862 miles2
0.121	0.047	0.173	242.20	6.9	/ H3 = 0.02832 H3
0.769	0.297	0 402	562.80	15.9	
4.047	1.563	1.398	1957.20	5,5.4	
15.02.	5,799	3.737	5231.80	1482	
120.18	46.40	17.778	24889:20	7049	. A Marie rate

	(Se) The m	axineur f	Lood for	C Route		
Aı	Az	A314.	Q,	Θ,	Romarko	,
(KM2)	miles). 0.027	0.0669	& 1/5-12 93.7	(m/sec) 2.7	1 Km2 = 0.3862 miles	
ي در	0.089	0.1629	228.1	65	151° = 0.02832 m3	•
0.28	0.108	0.1884	263.8	7.5		
0.43	0166	0 2601	364.1	10.3	<u>'</u>	

Therefore maximum flood is decided as follow

Aand Biroute						
A	Q					
0061	- m/sec 3. /					
0121	5.9					
0 769	204					
4.047	55.4					
15.02 .	1482					
120 18	7048					

c route					
A	9				
(KT') 007	(713/02:1 07				
0.23	18				
0.28	2.0				
043	26				

CHAPTER 2 STRUCTURAL CALCULATION

2-1-1 allowable Stress of Removered Concrete

2-1-1 allowable stress for compression of concrete Gra = 20 Typem

Allowable stress for tension of remforcement Tso = 1400 Syem

Allowable shearing stress remforcement Too = 45 Typem

Safety factor F=3.5

2-1-2 Unit Weight of the Malerials

Reinforced concrete 24 longoum

Blain concrete 23 longoum

Wat masonry 25 tongum

Saturated soil 20 tongum

Wet soil 18 tongum

Water 10 tongum

2-1-3 Coefficient of Earth Prossure

User; Rankined formula cortu pressure _v expressed as; p = KAT.H

Where P. earth pressure a given depth (trysgim)

Ki coefficient of earth pressure

i: unit weight of such (towaim)

H: given depth (m)

A coefficient of earth pressure, K. iv given in the term of internal friction angle & as

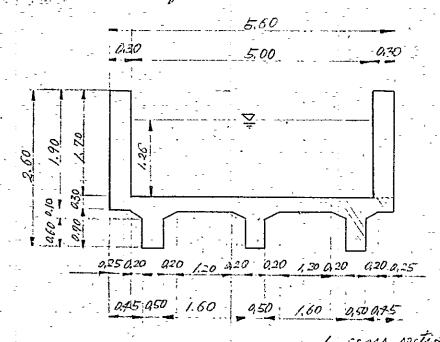
K1 = 1- sin \$

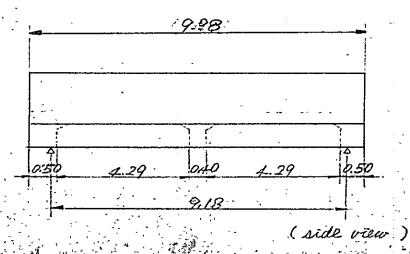
Here; assuming \$=30°. Kair-then given as follows.

2-2. Irrigation Aqueduct:

2-2-1. No.I. and No. 2. Irrigation Aqueduct

2-2-1-1. Joneral profile



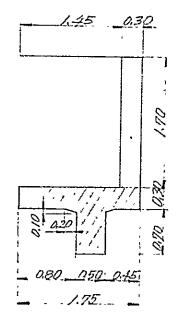


Note I Eigures are given in meters no consider

2-2-1-2

(1) Sept condition

Among three main beams, the side one is situated in the most riskful load condition, so the side one is examined;



(cross section of the main beam,)

(a) Read load

Water (8,).

8, = 1.70 × 1.45 × 1.00 = 2.47 t/m

Side wall. (82)

92 = 1.70 × 0.30 × 2.4 = 1.22 /m.

.. - Hunch . (85)

 $g_3 = 0.10 \times 0.20 \times \frac{1}{2} \times 2.4 \times 2 = 0.05 \stackrel{\cancel{\xi}}{=} m$

Flora (82)

84 = 0.30 × 1.75 × 2.4 = 1.26 =

Main beam (85)

85 = 0.70 × 0.50 × 2.4 = 0.84 = m

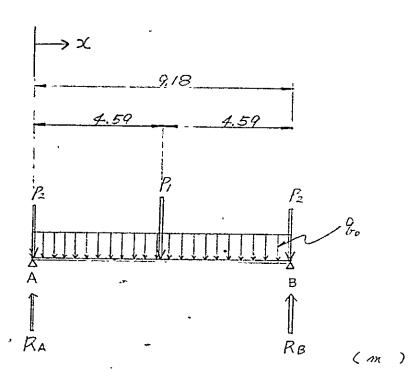
Center girder (P1)

 $P_{i} = 0.70 \times 0.40 \times 0.80 \times 2.4$ $+ 0.10 \times 0.20 \times \frac{1}{2} \times 0.80 \times 2.4 \times 2$ $= 0.58^{-t}$

End girder (P2)

 $P_{2} = 0.70 \times 0.50 \times 0.80 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 0.80 \times 2.4 \times 2 = 0.71$

· Total load: 80.



RA RB: reaction force

$$P_1 = 0.58^{\pm}$$
.
 $P_2 = 0.71^{\pm}$.
 $P_3 = 5.84^{\pm}$.

(2) Reaction: RA. RB.

$$R_{A} = R_{B} = \left(\frac{2}{9} \times \frac{9}{18} \times \frac{1}{2} + P_{1} \times \frac{4}{159} + P_{2} \times \frac{9}{18}\right) \times \frac{1}{9.18}$$

$$= \frac{1}{9.18} \left\{ \frac{5}{84} \times \frac{9}{18} \times \frac{1}{2} + 0.58 \times 4.59 + 0.77 \times 9.18 \right\}$$

$$= \frac{27.81}{4}$$

- (3) Bending moment and shearing force.

 1.3. Least in 15. ...
- (a) Bending moment;

0 \ \ \ \ \ \ \ 4.59;

$$M_{x} = R_{1} \times X - f_{2} \times X - f_{3} \times X \times X \times \frac{1}{2}$$

$$= 27.81X - 0.71X - \frac{5.84}{2}X^{2}$$

$$= \frac{5.84}{2}X^{2} + 27.10X \cdot (t-m)$$

$$\chi = 1.00:$$

$$M_{\chi = 1.00} = -\frac{5.84 \times 1.00^2 + 27.10 \times 1.00}{2}$$

$$= 24.18 \text{ t-m}$$

$$x = 2.00;$$

$$M_{x=200} = -\frac{5.84}{2} \times 2.00^{2} + 27.10 \times 2.00$$

$$= 42.52^{t-m}$$

$$\chi = 3.00;$$

$$M_{x=5.00} = -\frac{5.84}{2} \times 3.00^{2} + 27./0 \times 3.00$$

$$= 55.02 \stackrel{\text{f-m}}{=}$$

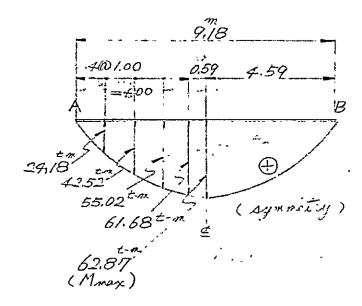
$$x = 4.00 :$$

$$M_{z=4.00} = -\frac{5.84}{2} \times 4.00^{2} + 27.10 \times 4.00$$

$$= 61.68 \pm -m$$

$$x = 4.59$$
;
 $M_{x=4.59} = M_{max}$
 $= -\frac{5.84}{2} \times 4.59^2 + 27.10 \times 4.59$
 $= 62.87^{t-m}$

Diagram of Bending moment.



(b) Shearing force

ķ.

$$S_{x} = \frac{dM_{x}}{dx}$$

$$= -5.84 \times + 27.10 \cdot (0 \le x \le 4.59)$$

x=4.59;

x=00.

Diegram of shearing force.

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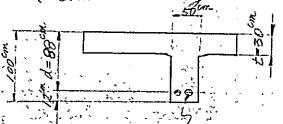
1 -27.10

1 -27.10

$$As = \frac{M_{max}}{\sigma_{sa} \left(d - \frac{t}{2}\right)}$$

where. $M_{max} = 62.87 \text{ t-m}$ = 6.287.000 lg-cm $S_{SQ} = 1.400 \text{ lg/cm}^2$ $d = 1.00 - 0.06 \times 2$ = 88 cm $d = 0.30^m = 30^{cm}$

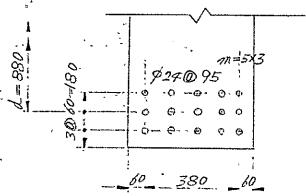
(concrete covering thickness: 6 cm)



Reinforcement (As cm²) section of T- beam)

Designed rainforcement,

$$\phi 25 = 0.95 \quad m = 5 \times 3 \quad (= 67.86^{cm^2})$$



(mm)

· Click;

$$p = \frac{A_S}{6d} = \frac{67.86}{125 \times 88} = 0.0044$$

= 0,5/

therefore :

·= 0.31-x 88

= 72728 CM < 30 CM = L

نام شريع بياريم.

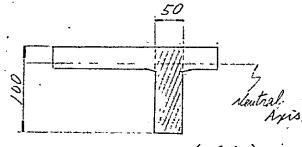
Neutral Axis

While I destruction with the the

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Required reinforcement amount.

rectangular section. 100 x 50 cm



Assuming d = 88 cm.

$$A_{sS} = \frac{M_{max}}{\sigma_{sa} \times \frac{2}{8} \times d}$$

$$= \frac{6.287.000}{1.400 \times \frac{7}{8} \times 88}$$

= 58,32 cm2

$$($924095, n=5x3.)$$

$$k = \sqrt{(np)^2 + 2np} - np$$

$$= \sqrt{(15 \times 0.015)^2 + 2 \times 15 \times 0.015} - 15 \times 0.015.$$

$$= 0.4.8.$$

$$\dot{j} = 1 - \frac{k}{3} = 1 - \frac{0.48}{5}$$

$$=1-\frac{0.48}{5}$$

$$\therefore \sqrt[n]{s} = \frac{M_{may}}{p.j.b.d^2}$$

$$=\frac{6.287.000}{0.015 \times 0.83 \times 50 \times 88^2}$$

$$= 1.287 \frac{k_9}{cm^2} < 1.400 \frac{k_9}{cm^2} = 0.5a$$

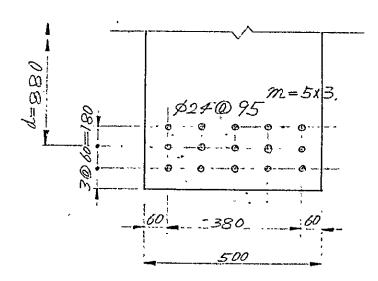
$$\sigma_{c} = \frac{2 M_{max}}{k_{j} B' d^{2}}$$

where: b'=175 < 45x30+20+45=2000m

$$\mathcal{G}_{c} = \frac{2 \times 6.287.000}{0.48 \times 0.84 \times 175 \times 88^{2}}$$

$$= 23.01 \frac{19}{6002} < 40 \frac{183}{6002} = 6 \text{ ca.}$$

Distribution of reinforcement.



(m m

 $j = 1 - \frac{k}{3}$ $= 1 - \frac{0.48}{3}$

= 0.84.

To Smax

= <u>27.100</u> 50x 0.84-'x 88

18/4=> 4.5 Kg/c= T ca

where;

Smax: shearing "fance at both sides. Tea: allowable shearing stress of concrete, Tea = 4.5 "Vom2.

Therefore.

it is necessary to use stirrup:

Reguired amount of sterrup

 $\dot{A}_{v} = \frac{\Delta M_{X}}{\Delta m_{z} Z},$

where: Av: stirrup amount.

Mx: difference of moment

Jsa: 1.400 19/cm:

る一方元

$$0 \le \chi \le 1.60$$

$$\Delta M_{\chi} = (M_{\chi-160}) - (M_{\chi-00})$$

$$= 35.88 \pm -m$$

$$= 3.588.000$$

$$A_{\chi} = \frac{3.588.000}{1400 \times 0.84 \times 88}$$

$$= 34.67 \text{ cm}^{2}$$

$$\Delta M_{\chi} = M_{max} - (M_{\chi-160})$$

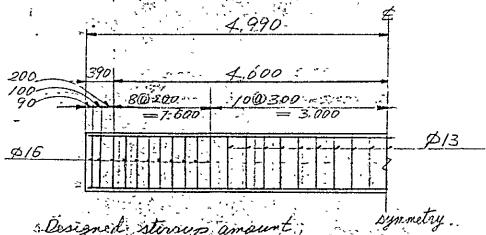
$$= 26.99 \pm -m$$

$$= 2.699.000 \frac{k_{g} - cm}{400}$$

$$= 26.99 \pm -m$$

$$= 2.699.000 \frac{k_{g} - cm}{400}$$

:: Av = 2.699.000: 1.400x084x88 20.08 313 -



Designed storsup amount;

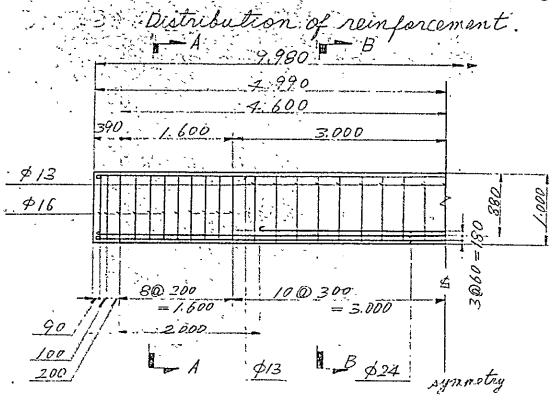
0 62 6 160 \$16@200 m= 9=

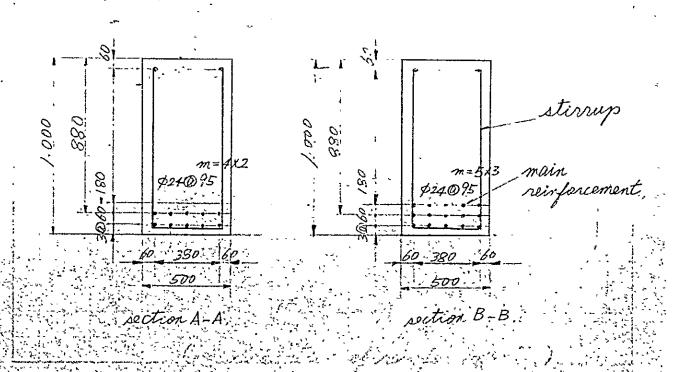
 $A.v = 2.011. \times .9$

1.6062 4.59

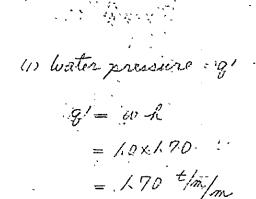
 $A_{v} = 10$ $A_{v} = 10$ $A_{v} = 1327 \times 10 \times 2 = 26.54$

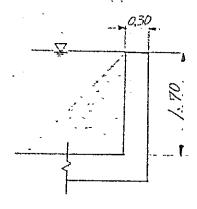
(7) Arrangement of neinforcement: At the point of x = 2.00 from the end of the beam $M_{\chi=2.00} = -\frac{5.8+}{2} \times 2.00^2 + 27.10 \times 2.00$ $= +2.52^{t-m}$ required reinforcement amount at this paint"; (As) = 200 = Mx=200 x As. = 42.52 62.87 × 67.86. Designed amount is, therefore, reduced for the section I = 0 to 2000 as follows, \$24.0 95, n=5x2. As = 67.86 x. = $= .45.89 = (As)_{z=2.00}$ CReck The $p = 0.015 \times \frac{12}{3} = 0.010$ 1= - (15 × 0.010)2+2×15×0.010 - 15×0.010, $j = 1 - \frac{1}{3} - 1 - \frac{0.42}{3} = 0.86$





2-2-1-3 Side wall.





(per unit width of wall)

where; w: unit weight of water (1.0 %m3)

h: water depth (1.70 m)

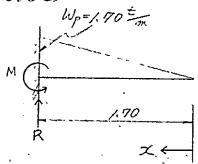
(2) Banding moment and shearing force.

calculation as cantilever.

$$M = M_{x=1.70} = \left| \frac{-2^{1} \chi^{3}}{6 \chi^{3}} \right|$$

$$= \frac{1.70 \times 1.70^{3}}{6 \times 1.70}$$

$$= 0.82 t^{-m}$$



 $S_{max} = R_{x=1/20} = \begin{vmatrix} -g/x^2 \\ \hline 2I \end{vmatrix}$ $= \frac{1}{2} \frac{1}{2}$

(3) Ranforiement amount

rectargle section; 30 cm x 100 cm assumming d = 24 cm

As = Mmax.

= 82000 = 1.400 x 7/8 x 24

2,79 cm2

Designed amount of reinforcement is;

Clock.

 $p = \frac{As}{bd} = \frac{(4.42)}{100 \times 24} = 0.0018$

 $k = \sqrt{(np)^2 + 2np'} - np,$

 $=\sqrt{(15\times0.0018)^2+2\times15\times0.0018}-15\times0.0018.$

= 0.21 $\dot{j} = 1 - \frac{fe}{3} = 1 - \frac{0.21}{3} = 0.93$

 $\nabla_{S} = \frac{M}{P + b d^{2}}$

850 Kg/on2 < 1.400 /cm2 = 0sd

$$\int_{C} = \frac{2M}{f_{2}j b d^{2}}$$

$$= \frac{2 \times 82.000}{0.21 \times 0.93 \times 100 \times 24^{2}}$$

$$= 14.58 \frac{K_{9}}{m^{2}} (40 \frac{K_{9}}{m^{2}} = 5co.$$
(0.K.)

2-2-1-4, Elosi (1)

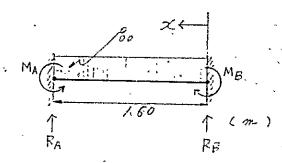
(1) Road condition.

Water: 8.

$$g_1 = 1.70 \times 1.00 \times 1.0$$

= 1.70 t/m

Which the was highly to a



Flooring 22 min 19

$$8_2 = 0.30 \times 1.00 \times 2.4$$

= 0.72. \pm /m

Total load 80
$$3. = 8. + 8. \\
= 2.42 \cdot \frac{t}{m}.$$

(2) Reaction and shearing force $RA = RB = \frac{80 \times 1.60}{2}$

$$S_A = |S_B| = |R_A| = |A/94| t/1$$

(3) Earding moment and shearing fience.

$$M_A = M_B = -\frac{30}{12} \times 1.60^2$$

$$= -\frac{2.42}{12} \times 1.60^2$$

$$= 0.52 \pm -m$$
 $M_{max} = M_A = 0.52 \pm -m$
 $S_{max} = S_A = 1.94 \pm$

(4) ; Reinforcement amount.

rectargle section; 30° × 100° m, assuming: d'= 24° m.

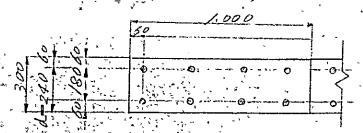
$$A_{5} = \frac{M_{\text{max}}}{\sigma_{\text{Sa}} \times 7/8 \times d}$$

$$= \frac{52,000}{1.400 \times 7/8 \times 24}$$

$$= 1.77^{0002}$$

Therefore, designed reinforcement: \$13 \$300.

 $A_S = 4.42 \, \text{sm}^2.$



$$P = \frac{As}{5d} = \frac{4.42}{100 \times 24} = 0.2018.$$

$$k = \sqrt{(\pi p)^2 + 2\pi p} - \pi p.$$

$$= \sqrt{(15 \times 0.0018)^2 + 2 \times 15 \times 0.0018} - 15 \times 0.0018.$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.21}{3} = 0.93$$

$$\sigma_{S} = \frac{M}{p j b d^{2}}$$

$$= 539 \frac{196m^2}{1.400} < 1.400 \frac{196m^2}{1.400} = \sqrt{54}$$

$$\sigma_{c} = \frac{2M}{l_{c}j bd^{2}}$$

$$= 9.25 \frac{kg}{cm^2} < 40 \frac{kg}{m^2} = 5_{cd}$$
(0.K.)

2-2-1-5; Flan (2)

(1) Load condition

· · · loater vision

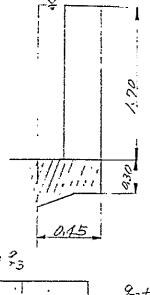
8, = 1.70 × 1.00 × 1.0 = 1.70 t/m (per unit largth)

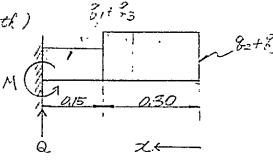
Side wall

82= 170×100×24

=4.08 t/m

(per unit longth)





Floor

83 = 0,30× 1,00×2.4

= 0.72 t/m (per unit length).

(2) Banding moment and shearing force.

calculation as cantilever.

0,30 & X & 0,45

 $M_{\chi} = (8 + 8_3) \times 0.30 \times (\chi - \frac{0.30}{2})$

+=(8,+83)(x-0,30)2x-1

$$M_{max} = M_{x=0.45}$$

$$= (4.08 + 0.02) \times 0.30 \times (0.45 - \frac{0.30}{2})$$

$$+ (170 + 0.02) \times (0.45 - 0.30)^{2} \times \frac{1}{2}$$

$$= 0.46 \quad t-m$$

$$S_{max} = (82+83) \times 0.30 + (81+83) \times 0.15$$

$$= (4.08+0.72) \times 0.36 + (1.70+0.72) \times 0.15$$

$$= 1.80^{\pm}.$$

(3) Reinfarce ment amount

assuming
$$d = 24^{cm}$$

 $A_{S} = \frac{M_{max}}{\sigma_{SA} \times \frac{1}{8} \times d}$
 $= \frac{46000}{1400 \times \frac{1}{8} \times 24}$
 $= \frac{1.56^{cm^{2}}}{1.56^{cm^{2}}}$

Therefore, designed amount of reinforcement. is; \$130 300 As = 4.42 cm²

$$p = \frac{As}{bd} = \frac{4 \cdot 4^{2}}{100 \times 24} = 0.0018$$

$$k = \sqrt{(\pi/r)^{2} + 2\pi p} - \pi p$$

$$= \sqrt{(15 \times 0.0018)^{2} + 2 \times 15 \times 0.0018} - 15 \times 0.0018.$$

$$= 0.21.$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.21}{3} = 0.93$$

$$\sigma_{s} = \frac{M}{PJbd^{2}} = \frac{46000}{0.0018 \times 0.73 \times 100 \times 24^{2}} \\
= 477 \frac{P_{cm}^{2}}{1.400} \times 1.400 \frac{P_{cm}^{2}}{1.400} = \sigma_{sa} \\
(o K.)$$

$$\sigma_{C} = \frac{2M}{h_{0}^{2} b_{0}^{2}} = \frac{2 \times 46000}{0.21 \times 0.93 \times 100 \times 24^{2}}$$

$$= 8.18 \frac{19}{h_{0}^{2}} = 40 \frac{19}{m^{2}} = 6ca.$$

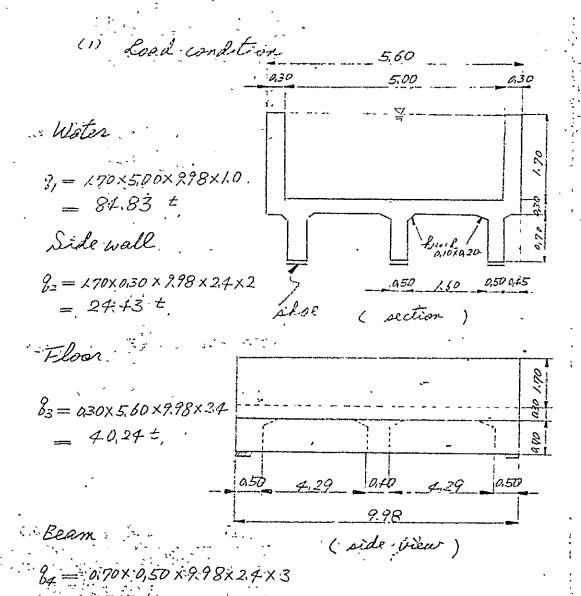
$$(0.4.)$$

$$T_{c} = \frac{\beta_{max}}{b_{d}d} = \frac{1800}{100 \times 0.93 \times 24}$$

$$= 0.81 \frac{19}{m^{2}} < 4.5 \frac{15}{6} = T_{ca}$$

$$= 0.81 \cdot (0.K.)$$

2-2-1-6, Shoe,



Guder

85= 0.70× x60 × 0.50×24×2 + 0.70× 1.60× 0.40× 2.4

Hunch

% = 0.16×0.20×298×2.4×6. = 2.87 ±

3y = 0,10 x 0,20 x x 60 x 2.4 x 4 x 2. = 0.6/ t,

Total load: %

.(2) Check

Löad per one skal: %

Number of skals: 6: $8_0'=181.900 \times \frac{1}{6}=30.317 \times \frac{1}{9}/\text{skal}$

allowable stress for compression of cast iron. (FC-15). Sia = 800 kg/cm².

allowable stress for compression of concrete, $\sigma_{co} = 40 \frac{kg}{cm^2}$

Supporting area of iron and concrete,

Cast inon:

$$A_i = 90 \times 420^{-nm^2}$$

 $= 9.0 \times 42.0^{-cm^2}$
 $= 378.0^{-cm^2}$
Concrete:
 $A_c = 450 \times 600^{-nm^2}$

 $= 45.0 \times 60.0^{\text{cm}^2}$ $= 2700.0^{\text{cm}^2}$

Corcrete.

check.

$$\sigma_{i} = \frac{\frac{80}{80}}{\frac{30,317}{378,0}} = \frac{30,317}{\frac{378,0}{600}} = \frac{80.20 \frac{149}{6002}}{\frac{1}{1000}}$$

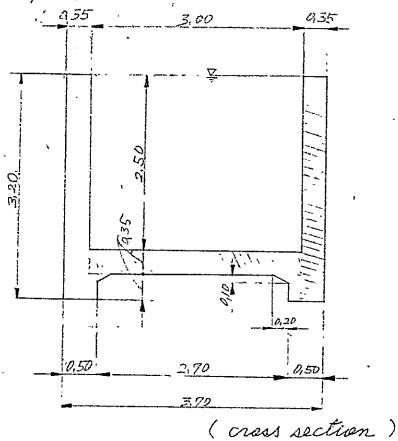
 $= 80.20 \frac{19}{6m^2} < 800 \frac{19}{5m^2} = 514$

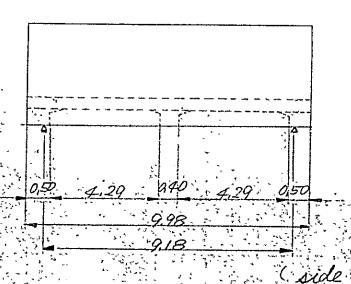
$$\sigma_{c} = \frac{g_{o}'}{Ac} \\
= \frac{30.317}{2.700.0} \\
= 11.23 \quad \frac{49}{m^{2}} < 40 \quad \frac{19}{m^{2}} = \sigma_{c4}. \\
(0, K,)$$

and the second of the second

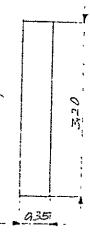
2-2-2, NO.3. Inigation Aqueduct.

2-2-2-1. Fiven diversion





2-2-2-2. Beam



(1) Load condition.

.: Water

81= 2.50 × 3.00 × 1.0 = 7.50 ₺m, · Side wall.

 $g_2 = (2.50 \times 0.35 \times 2.4 + 0.70 \times 0.50 \times 2.4) \times 2 = 5.88^{t/m}$ Hunch.

 $g_3 = 0.10 \times 0.20 \times \frac{1}{2} \times 2.4 \times 2 = 0.05 \frac{1}{2}$

Filear.

84 = 0,35 × 2,70× 2,4= 2,27 tm.

Center girder

 $P_{i} = (0.35 \times 0.40 \times 2.70 \times 2.4$

+ 9/0 × 0,20 × + × 2,70 × 2,4 × 2,)

= 1.04

Endigender

P2 = (0,35 × 0.50 × 2.70 × 2.4

+0,10×0,20×-1×2,70×2,4×2.)

=:/26

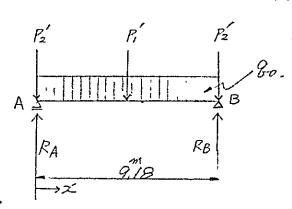
$$\frac{2}{80} = \frac{\sum 80/2}{1.85}$$

$$P_{i} = 1.07 \times \frac{1}{2}$$

= 0,52 t,

$$P_2' = 1.26 \times \frac{1}{2}$$

= 0.63 t,



.(3) Reaction; RA, RB,

$$R_{A} = R_{B}$$

$$= \frac{80}{2} \times 9.18 + \frac{P_{1}'}{2} + P_{2}'$$

$$= 36.92^{-6}$$

(4) Bonding moment and shearing force.

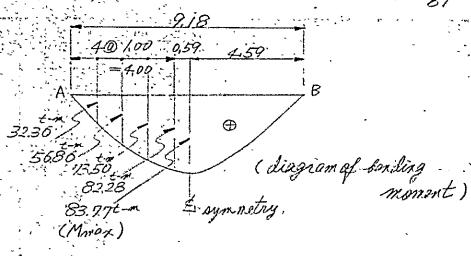
(a.) Bending moment.

$$M_{\chi} = R_{A}\chi - P_{z}\chi - \frac{Q_{o}\chi^{2}}{2}$$

$$:= -3.93 \times^2 + 36.29 \times$$

$$Z = 1.00$$
, $M_{X = 1.00} = 32.36$ tm

$$\chi = 2,000 \quad M_{\chi} = 2,000 = 56.86 \quad t-m$$



$$x = 4.59$$
, $S_{x=4.59} = 0.26^{t}$

$$x = 0.0$$
. $S_{x=0.0} = 36.29^{\pm}$

(5) Reinforcement amount.

Assuming $d = 3.10 = 310^{cm}$

lerefore designed amount:

$$\phi 19.0095 \quad m = 5 \times 2$$

$$\phi 19.095 \quad m = 5 \times 2$$

(6) Check

$$p = \frac{As}{b} = \frac{28.35}{35 \times 3/0} = 0.0026.$$

$$k = \sqrt{(np)^2 + 2np} - np = 0.24$$

$$j = 1 - \frac{f_c}{3} = 0.92.$$

$$O_{S} = \frac{M_{max}}{p \cdot j \cdot b \cdot d^2}$$

$$= 1.04/\frac{kg}{c_{1}\kappa^{2}} < 1.400.\frac{kg}{c_{1}\kappa^{2}} = 0.5a$$

$$= 0.64/\frac{kg}{c_{1}\kappa^{2}} < 1.400.\frac{kg}{c_{1}\kappa^{2}} = 0.5a$$

$$\sigma_{c} = \frac{2 \times M_{max}}{k \cdot j \cdot b \cdot d^{2}}$$

=
$$22.56 \frac{\text{Kg}}{\text{cm}^2} < 40 \frac{\text{Kg}}{\text{cm}^2} = 5\text{ca}$$

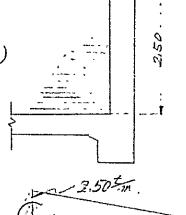
$$T_{c} = \frac{S_{max}}{b \cdot j \cdot d}$$

in the second second

$$= 3.6 \frac{\frac{1}{2}}{cm^2} = 4.5 \frac{\frac{1}{2}}{cm^2} = T_{cQ}$$

$$(0, K.)$$

2-2-2-3 Side wall



ス÷

calculation as cartilever.

$$M_{mox} = \left| -\frac{!9! \times 3.50^{2}}{6} \right|$$

$$= \frac{2.50 \times 2.50^{2}}{6!}$$

$$= 2.60. \pm -m$$

$$|\vec{S}_{max}| = \left| -\frac{9}{2} \times 250 \right|$$

$$= \frac{250}{2} \times 250$$

$$= 3/3$$

(3) Reinfärcement camount

Assuming
$$d = 29^{cm}$$

 $A_8 = \frac{26,000}{14.00 \times 7/8 \times 29} = 7.32^{cm^2}$

$$j = -1 - \frac{0.26}{3} = 0.91$$

$$\sigma_{c} = \frac{2 \times 262000}{0.26 \times 0.7/ \times 100 \times 29^{2}}$$

$$T_{c} = \frac{3.130}{100 \times 0.9/ \times 29}$$

$$= 1.2 \frac{19}{cn^2} < 4.5 \frac{19}{cn^2} = T_{ca}$$

(0K.)

(1) Lood condition

$$G_2 = 0.35 \times 1.00 \times 2.4$$

= 0.84 t/m.



(2) GB inding moment and shearing force.

$$M_{A} = M_{B} = -\frac{\frac{g_{o}}{12} \times 2.70^{2}}{\frac{3.34}{12} \times 2.70^{2}}$$

$$= 2.03 \pm -m.$$

$$R_{A} = R_{B} = \left| \frac{g_{o}}{2} \times 2.70 \right|$$

$$= \frac{3.54}{2} \times 2.70 = 4.51^{\pm}$$

$$\approx \frac{3.54}{2} \times 2.70 = 4.51^{\pm}$$

$$\approx \frac{3.54}{2} \times 2.70 = 4.51^{\pm}$$

The transfer of the proof of the proof of the transfer of the

(3) Painfarcement amount

Assuming $d = 29^{cm}$. $A_S = \frac{203,000}{1400 \times 1/8 \times 29} = 5.77^{cm^2}$. designed amount: $(5130150, A_S = 8.85^{cm^2})$

(4) Check

p = 0.0031.

 $f_2 = 0.26$.

 $\hat{j} = 0.91.$

 $\sigma_{S} = \frac{203,000}{0.0031 \times 0.91 \times 1000 \times 29^{2}}$ $= 856 \frac{18}{602} < 1.400 \frac{18}{602} = 50$

(D.K.)

 $\sigma_{c} = \frac{2 \times 203.000}{0.26 \times 0.91 \times 100 \times 29^{2}}$

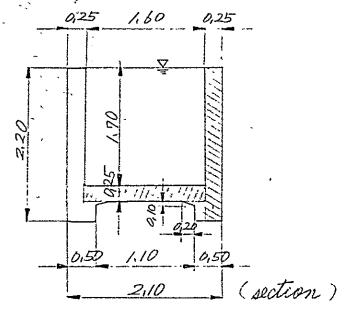
= 20.40 18/cm2 < 40 Febru = Jea

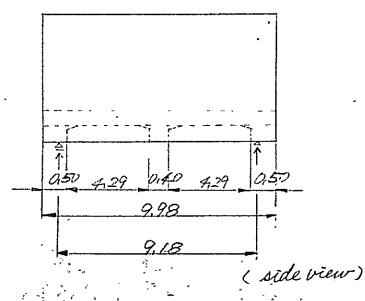
 $T\dot{c} = \frac{4.510}{100 \times 0.91 \times 29}$ (0.K.)

 $= 1.7 \frac{19}{1000} < 4.5 \frac{19}{1000} = Tca$ (0.K.)

2-2-3, No.4: Irrigation Aqueduct,

2-2-3-1 Given dimensions,





(,mi).

2-2-3-2! Beam

- (1) Lord condition.
- 5 water

8,=170×160×10= 2,92 5/m

Side walk

82= (170×0,25×34+0,50×0,50×34)

1×2= 3,24 t/m.

... Hunch

83 = 0,10 x 0,20 x ± x2,4 x2, = 0,05 t/m

- Floor.

9. = 0.25 × 110 × 2.4 = 0,66 t/m.

girder.

 $P_{l} = (0.25 \times 0.40 \times 1.10 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 0.31 + 0.10 \times 0.20 \times 0.20$

 $P_2 = (0.25 \times 0.50 \times 1.00 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 2.4 \times 2) = 0.39 \pm 0.39 \pm 0.00 \times 0$

(2) Total load : 30. ...

80=∑8i/2= 3,34-t/m.

 $P_1' = P_1/2 = 0.15 t$

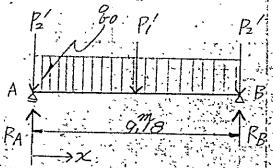
 $P_2 = P_2/2 = 0.20 t$

reaction RA RB

 $R_A = R_B$

= 80 x9/8+ P/+ P2

= 15.61



GABRES CONDUTORS RELIGIAN

(3) Earding moment and sleaving force

$$M_{X} = R_{4}X - P_{5}X - \frac{q_{5}X^{2}}{2}, \quad (0 \le X \le 4,59)$$

$$= -1.67 \times^{2} + 15.41 \times.$$

$$M_{max} = M_{x=4.59}$$
.
= -1.67.x4.59²+15.41x4.59
= 35.55 t-m.

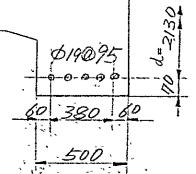
$$S_{max} = \left(\frac{dM_x}{dx}\right)_{x=0}$$

(4) Reinforcement amount

$$As = \frac{3.5.55.000}{1.400 \times 7/8 \times 213} = 13.62^{cm^2}$$

designed amount:

$$(\phi 19095, m=5)$$
 $(A_S=14.18^{cm^2})$



(O.K)

$$\begin{array}{lll}
(5) \cdot Clock \\
P &= \overline{As} &= 14.18 \\
P &= \overline{Dd} &= 25 \times 273 &= 0.0027 \\
k &= \sqrt{(np)^2 + 2\pi p} - np &= 0.24 \\
j &= 1 - \frac{k}{3} &= 0.72 \\
\hline
\sigma_S &= \frac{Mno_X}{P + b d^2} \\
&= \frac{3.555.000}{0.00^{-7} \times 0.92 \times 25 \times 213^{-1}} \\
&= 1.262 \quad | k_0 | |_{cm} &= 1.400 \quad | k_0 |_{cm} &= \sigma_{Sa} \\
\sigma_S &= \frac{2Mmx_X}{k_0 b d^2} \\
&= \frac{2 \times 3.5555.000}{0.24 \times 0.92 \times 25 \times 213^{-1}}
\end{array}$$
(0.K)

$$T_{C} = \frac{S_{M2X}}{b j d}$$

$$= \frac{15.710}{25.\times0.92\times213}$$

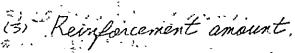
28,39 Fg/on2 < 40 Fg/on2 = 5ca.

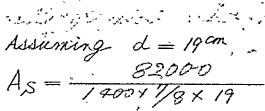
3.1 Kg/m= < 4.5 Kg/m2 = TCa

calculation as cantilever

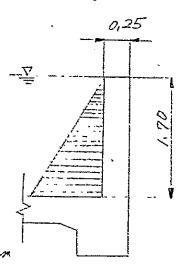
$$= 170 \times 1.70^{2} \times \frac{1}{6} = 0.82^{t-m}$$

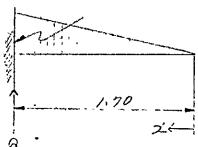
$$S_{max} = \frac{170^{2}}{2} = 14.5^{t}$$





$$A_{s} = 4.42 \, \text{cm}^2$$





1410 Check

$$p = \frac{As}{bd} = \frac{4.42}{100 \times 19} = 0.0023$$

$$k = \sqrt{(np)^2 + 2\pi p} - \pi p = 0.23$$

$$j = 1 - \frac{B}{3} = 0.92$$

$$\frac{ds = \frac{M\pi o \times 1}{P \int b d^2} \\
= \frac{82000}{0.0023 \times 0.72 \times 1.30 \times 19^2}$$

=
$$1.073$$
 $19/cm^2$ < 1.400 $19/cm^2 = 0.5a$ (0K)

$$\frac{2 M_{max}}{h_{j} b_{j} d^{2}}$$

$$= \frac{2 \times 82000}{0.23 \times 0.02 \times 100 \times 10^{2}}$$

$$= \frac{2 \times 82000}{0.23 \times 0.02 \times 100 \times 10^{2}}$$

$$= \frac{2 \times 82000}{0.23 \times 0.02 \times 100 \times 10^{2}}$$

$$= \frac{2 \times 82000}{0.23 \times 0.02 \times 100 \times 10^{2}}$$

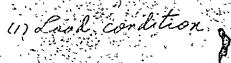
$$= \frac{2 \times 82000}{0.23 \times 0.02 \times 100 \times 10^{2}}$$

$$T_{c} = \frac{\beta_{max}}{b_{j}d}$$

$$= \frac{1.450}{100 \times 0.92 \times 19}$$

$$= 1.0 \frac{1}{3/m^{2}} < 4.5 \frac{1}{3/m^{2}} = T_{ca}$$

$$= 0.K.)$$





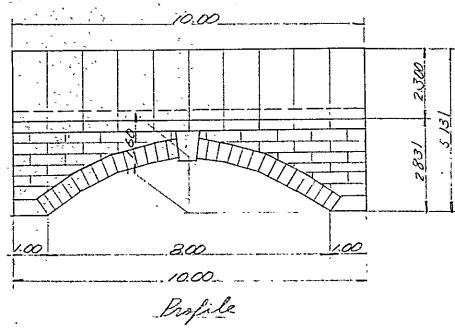
(2) Bonding monest and shearing force.

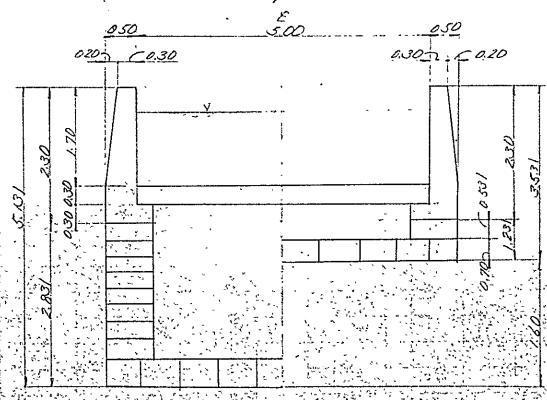
$$M_A = M_B = \left| \frac{2}{12} \times 1.10^2 \right| = 0.23 \text{ E-m}.$$

(3) Reinforcement amount.

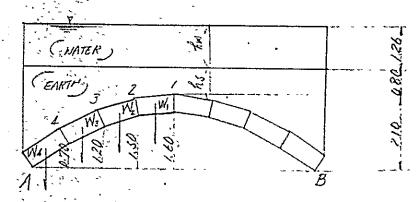
Assuming d-19 cm.

2-2-4 Arch type Irrigation Aqueduct (NO.1). 2-2-4-1 Given dimension.





2-2-4-2 Load condition



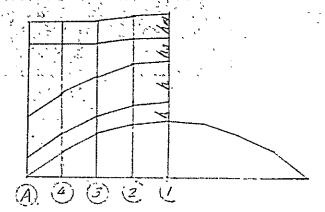
(1) Unit weight of the matericals

Wet masorry $r = 2.5 \text{ fm}^2$ Laturated soil r = 2.0Water r = 1.0

$$\frac{1}{1/r} = \frac{2.0}{1.5} = 0.000$$
 $\frac{1}{1/r} = \frac{1.0}{1.5} = 0.400$

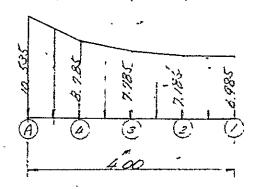
(2) Conversion height of load

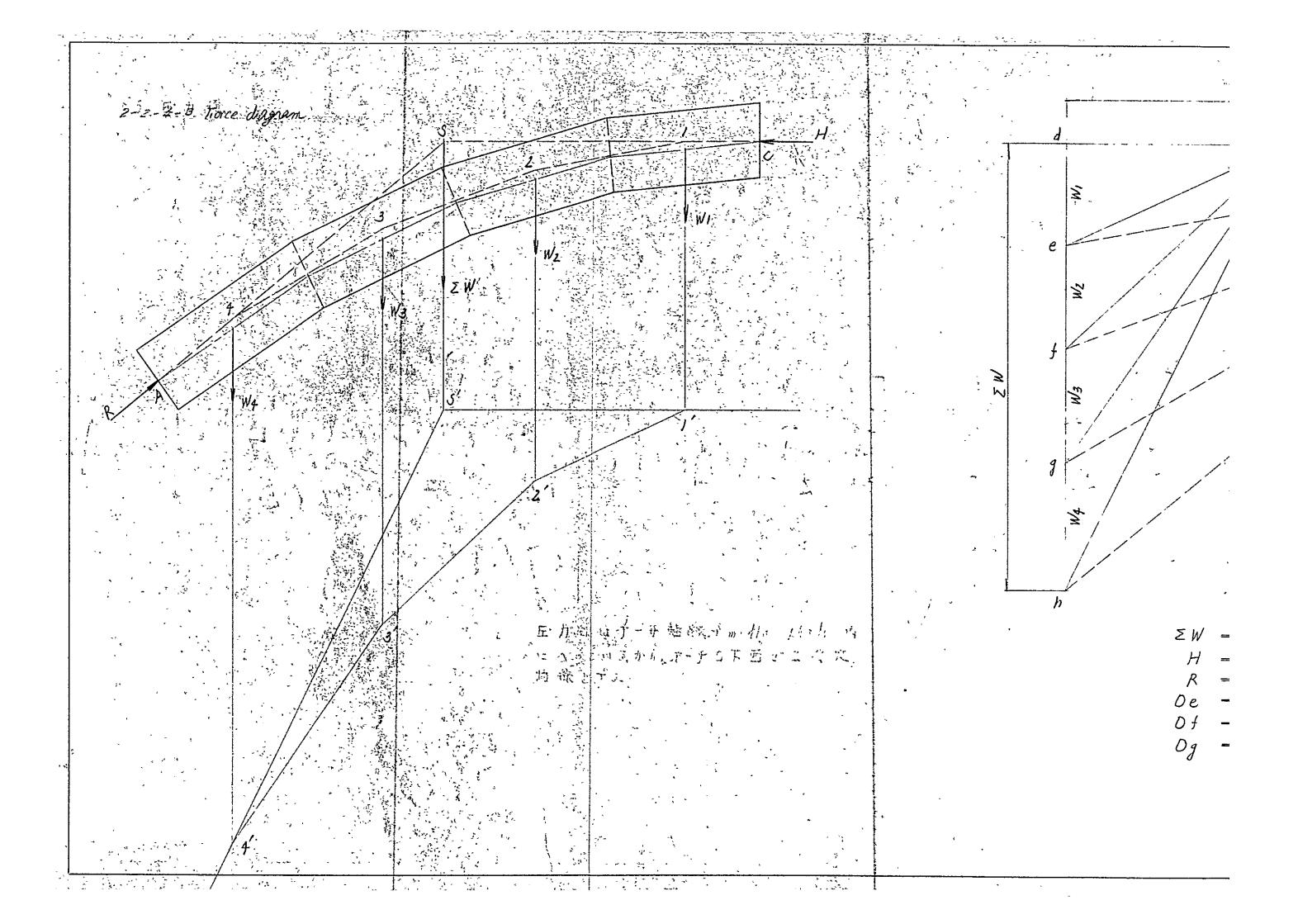
 $hsi = 0.80 - 0.80 = 0.640^{m}$ $hwi = 1.26 \times 0.40 = 0.504$ hsz = 0.90 + 0.80 = 0.720 $hsz = 1.20 \times 0.80 = 0.960$ hsd = 1.70 - 0.80 = 1.360 $h = 0.50 \times 2.50 = 1.150$

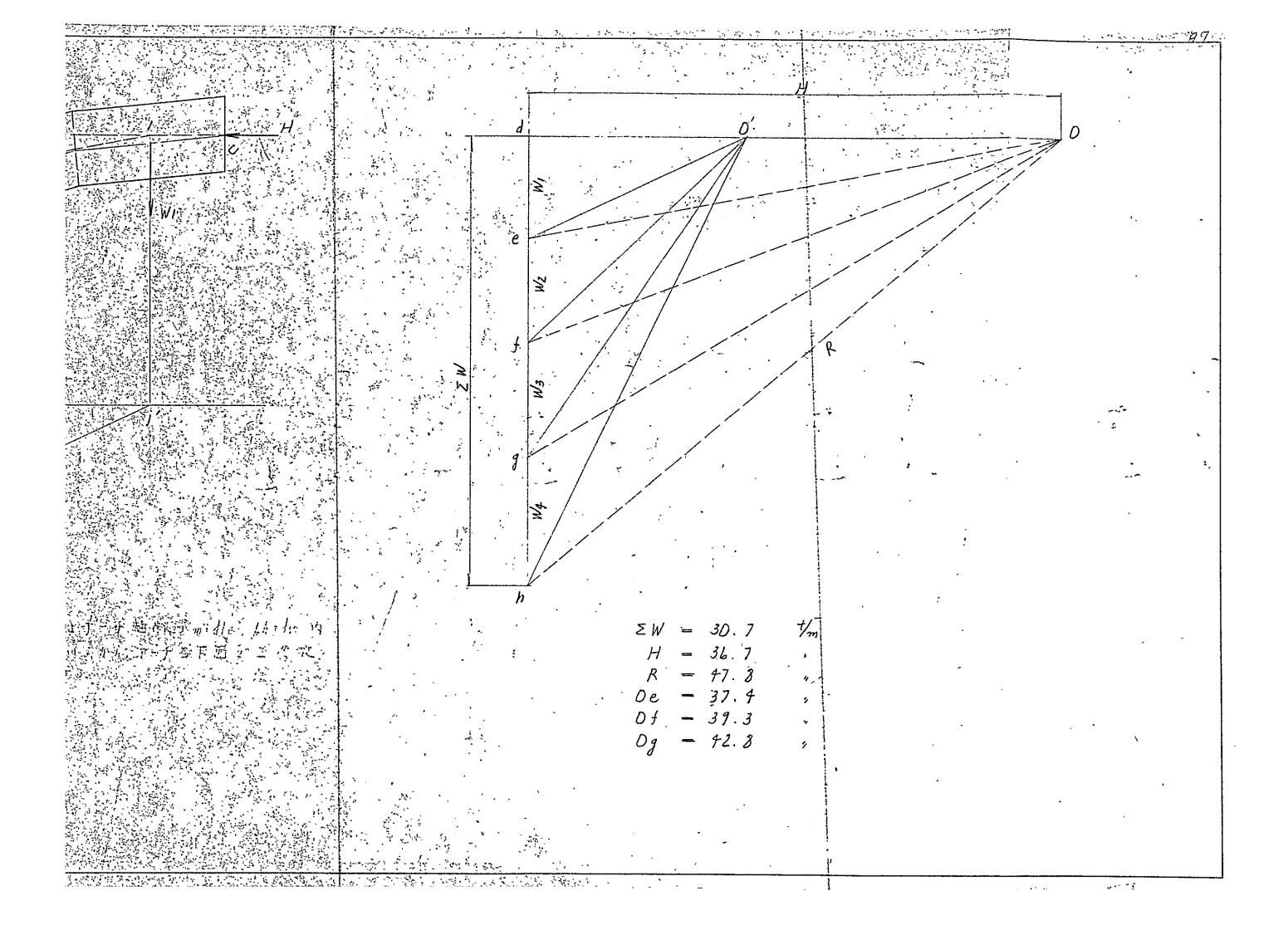


	Á	h.	hs	hw	Ξh	N
0	0.500	1.150	n 0540	0.504	2794	6985
2	,	•	0720	,	2.874	7185
(3)		•	0.960	,	3.114	7785
(4)		,	1.560	**	3514	8.785
A			2060	<u> </u>	4214	10535

(3) Lord diagram







2-2-4-4 Calculation of stress for compression.

(2)
$$P_0 = 37.4 \frac{1}{10}m$$
, $e = 10^{em}$
 $g = 5^*.30^\circ$
 $N_0 = P. \cos g = 37.4 + 605 \cdot 30^\circ = 37228^{em}$
 $H_0 = P. \sin g = 37.4 + 605 \cdot 30^\circ = 3.585^\circ$

$$\delta_{c} = \frac{100.0}{100.0} \left(1 - \frac{6e}{b} \right) \\
= \frac{37228}{100 \times 50} \times \left(1 \pm \frac{6 \times 10}{50} \right) \\
= \frac{18.3}{6.6} \times \frac{10}{50} \times \left(\frac{1}{50} + \frac{6}{50} + \frac{1}{50} \right)$$

MN6=06-37228 = 22.337 T > 3.585

$$-(3) P_0 = 393 \text{ fm} \qquad e = 2.0^{\text{cm}} \quad \mathcal{G} = 1.50$$

$$N_0 = 39.3 \times 000 1^{\circ}50 = 39.280 \text{ fm}$$

$$H_0^{\circ} = 39.3 \times 000 1^{\circ}50 = 1.25.7 \text{ fm}$$

$$G_c = \frac{39280}{100.50} \times \left(1 \pm \frac{6 \times 2.0}{50}\right)$$

MN= 0.6 - 39280 = 23.568 4 7 1257 7m

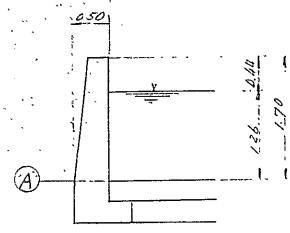
(4)
$$P_0 = 428 \text{ fm}$$
 $e = 20$ $9 = 5^{\circ}10^{\circ}$
 $N_0 = 428 \cdot 2005^{\circ}10^{\circ} = 12.620 \text{ fm}$
 $H_0 = 428 \cdot 2005^{\circ}10^{\circ} = 3.854 \text{ fm}$
 $C = \frac{12.620}{100 \cdot 50^{\circ}}(1 \pm \frac{6.20}{50^{\circ}})$

(5) Point A

R = 47.8 t

Oc = 700.50 = 9.6 Vine < 70 Mm

2-2-4-5 Side wall.



Section A-A.

Water, pressure

Bending moment.

The North Association

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	The state of the s
2-3 Ingiantic	n Syphon, to
2-3-1, NO. 7. In	rigation syphon,
والشرائح أزوالي ومعهومات فأكمه وأنهم ولالأزوالي أرائها والمائي	
2-3-1-1. Top- Elour	The state of the s
A District Control of the Control of	ower water surface
(1) general:	river water surface larth surface
	100
700	240 240
	A
	0
(Reinforced correcte)	
(Reinforted concrete).	
Wet masonry	
	(section)
Only the top floor is	calculated and examined
	, , , , , , , , , , , , , , , , , , , ,
as a continuous b	eam:
units weight	concrete : 24 m3
	earth in water: 20 m3
	water 1.0 tm3
beam longth:	1.80 ^m .
one span length	: 2,40m
Esa = 1.400	Kg/in2
	Kg /
Tow = 40	
4	Tom 2 and the second se
(2) Road condition	
Earth	
8/= Xoox Xo	
= 2.00 t/m	
And the state of t	
SANY CONSULT	MTS INTERNATIONAL TO THE STATE OF THE STATE
au terminanan mendalah sebesah di bermanan di bermanan di bermanan di di debah di 1976 MITA di 1976 (1976), di di di	and the resolution of the first

Water

$$8_2 = 1.00 \times 1.00 \times 1.0 = 1.00 \text{ t/m}$$
 $8_3 = 0.35 \times 1.00 \times 2.4 = 0.84 \text{ t/m}$

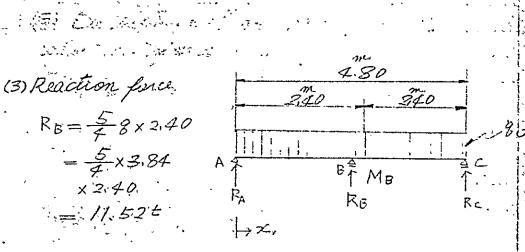
Total load

 $8_0 = 8_1 + 8_2 + 8_3$

= 9.84 t/m.

(3) Reaction fince.

$$R_{B} = \frac{5}{4} 8 \times 2.40$$
 $= \frac{5}{4} \times 3.84$
 $\times 2.40$
 $R_{A} = 11.52 \pm 11.52 \pm 12.40$



$$R_{A} = R_{c} = \frac{3}{8} \% \times 2.40$$

$$= \frac{3}{8} \times 3.84 \times 2.40$$

$$= 3.46 \pm 0.40$$

(4) Bending moment:

$$M_{B} = -\frac{8 \times \frac{2.40^{2}}{8}}{8}$$

$$= -3.84 \times \frac{2.40^{2}}{8}$$

$$= 2.76 t m$$

$$= 2.76 t m$$

$$= R_{A} \times \frac{2}{80} \times \frac{12}{2}$$

$$= -\frac{3.84}{2} \times 2 + 3.46 \times$$

$$\frac{dM_{2}}{dx} = -3.84 \times + 3.46.$$

$$\frac{MN_{2}}{J_{2}} = 0.90$$

$$= -\frac{3.94}{3.94} \times 0.90^{2} + 3.46 \times 0.90$$

$$= -\frac{3.94}{1.55} \times 0.90^{2} + 3.46 \times 0.90$$

$$= -\frac{3.94}{1.55} \times 2.40 + 3.46$$

$$= -\frac{3.94}{5.76} \times 2.40 + 3.46$$

$$= -\frac{3.94}{5.76$$

(E) in the case of
$$M=1.55^{\pm ...}$$
 $71.55.000$ kg on $M=1.55^{\pm ...}$ $71.55.000$ $1.55.000$ $1.400 \times 7/8 \times 29$ designed amount:

diagnet amount:
\$\phi 13.00 200 \quad (A_S=6.64^{cm^2})\$

(17) Chech

(a) in the case of
$$M = 2.76 = 2.96.000 \text{ kg-on}$$

$$p = \frac{As}{60} = \frac{10.06}{100 \times 29} = 0.0035$$

$$k = \sqrt{(np)^2 + 2np} - np$$

$$= \sqrt{(15 \times 0.0035)^2 + 2 \times 15 \times 0.0035} - 15 \times 0.0035$$

$$= 0.29$$

$$f = 1 - \frac{12}{3} = 1 - \frac{0.29}{3} = 0.90$$

$$s = \frac{M}{P_0 D D} = \frac{2.76.000}{0.0035 \times 0.90 \times 100 \times 29^2}$$

$$= 1.04.2 \frac{9}{6m^2} = 0.90$$

$$\frac{2 \times 276.000}{1.400} = 0.90 \times 100 \times 29^{2m}$$

$$= \frac{2M}{P_0 D D} = \frac{2 \times 276.000}{0.29 \times 0.90 \times 100 \times 29^{2m}}$$

$$= \frac{2M}{P_0 D D} = \frac{2 \times 276.000}{0.29 \times 0.90 \times 100 \times 29^{2m}}$$

$$= 25.75 \frac{\text{kg}_{\text{cm}^2}}{25.75} < 40 \frac{\text{kg}_{\text{cm}^2}}{2} = 5 \text{ca}_{\text{co}} (0.45)$$

$$\frac{C_{max}}{5.960} = \frac{5.960}{100 \times 0.90 \times 29}$$

$$= 2.2 \frac{69}{60} \le 4.5 \frac{69}{60} = 100$$

 $\frac{1}{2} \int C_{M^{2}} \left(\frac{1}{2} \right) \int C_{M} \left(\frac{1}{2} \right) \int C_{M}$

(6) in the case of
$$M = \frac{1.55}{1.55,000} = 1.55,000 \, fg^{-0}m$$

$$P = \frac{A_0^2}{6.6} = \frac{6.64}{100 \times 29} = 0.0023$$

$$E = \sqrt{(\pi P)^2 + 2\pi p} - \pi P$$

$$= \sqrt{(15 \times 0.0023)^2 + 2 \times 15 \times 0.0023} - 15 \times 0.0023$$

$$= 0.23$$

$$i = 1 - \frac{R}{3} = 1 - \frac{0.23}{3}$$

$$= 0.92$$

$$= \frac{1.55,000}{0.0023 \times 0.92 \times 100 \times 29^2}$$

$$= \frac{1.55,000}{0.0023 \times 0.92 \times 100 \times 29^2}$$

$$= \frac{1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{2M}{A_0^2 + 2 \times 1000 \times 29^2}$$

$$= \frac{2 \times 1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{2 \times 1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{5\pi a \times 1}{1000 \times 0.92 \times 29}$$

$$= 2.2 \, \frac{1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{5\pi a \times 1}{1000 \times 0.92 \times 29}$$

$$= 2.2 \, \frac{1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{5\pi a \times 1}{1000 \times 0.92 \times 29}$$

$$= 2.2 \, \frac{1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{5\pi a \times 1}{1000 \times 0.92 \times 29}$$

$$= 2.2 \, \frac{1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

$$= \frac{5\pi a \times 1}{1000 \times 0.92 \times 29}$$

$$= 2.2 \, \frac{1.400 \, \text{Kg/cm}^2}{0.0000} = \frac{5\pi a}{0.0000}$$

2-3-1-2. Examination of inside pressione; uppor water surface (1) Load condition top floor (Roinforced concrete) (section). let masonry. inside pressure, operating against the top floor from inside to upward, is desided by water head distance H=4.00m Pur= H. W. 1.00 = 4.00x 1.0x 1.00 = 4.00 /m Top floor 9 = 0.84 T/m. Total load: 8. 80= Ro-8 = 3.16 t/m inthis case, earth pressure over the top floor is reglected; (2) Roaction force. RB = 780×240= 7

$$RA = RC = \frac{3}{8} 2. \times 240$$

= $\frac{3}{8} \times 3.16 \times 240$

(3) Bending moment

$$M_{B} = -\frac{2}{3} \times \frac{240^{2}}{8}$$

$$= -3.16 \times \frac{240^{2}}{8} = 2.28 \cdot t - m$$

$$M_{x} = R_{A}x - 8. \times \frac{x^{2}}{2}$$

$$= \frac{3.16}{2}x^{2} + 2.28x.$$

$$\frac{M\alpha}{dx} = -3.16\chi + 2.28$$

$$\frac{M\alpha}{dx} = 0 \longrightarrow \chi = 0.72.$$

$$M_{max} = M_{s=0.72}$$

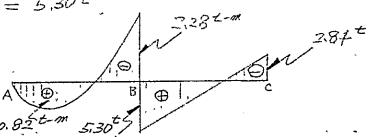
$$= -\frac{3.16}{2} \times 0.72^{2} + 2.28 \times 0.72^{1},$$

$$= 0.32 t-m.$$

(4) Shearing force,

$$S_{max} = |-3.16 \times 2.40 + 2.28|$$

= 5.30±



(5) Reinforcement amount.

now reinforcement amount of the upper part of the top floor A = 10.06 cm² (designed amount)

and, that of the lower one,

(designed amount)

(6) Chock

(05) in the case of
$$M = 228^{\pm 10} 228000$$
, (As=664)

$$\frac{As}{bd} = \frac{654}{100 \times 29} = 0.0023$$

$$\frac{A}{bd} = \frac{654}{100 \times 29} = 0.0023$$

$$\frac{A}{bd} = \frac{654}{100 \times 29} = 0.0023$$

$$\frac{A}{bd} = \frac{654}{100 \times 29} = 0.92$$

$$\frac{A}{bd} = \frac{0.23}{3} = 0.92$$

$$\frac{A}{bd} = \frac{228000}{0.0033 \times 0.92 \times 100 \times 292}$$

$$= 1.281 \frac{1.9}{100} = \frac{2.28000}{0.25 \times 0.92 \times 100 \times 292}$$

$$= 2.81 \frac{1.9}{100} = \frac{2.28000}{0.25 \times 0.92 \times 100 \times 292}$$

$$= 25.62 \frac{1.9}{100} = \frac{4.08}{100 \times 0.92 \times 29}$$

$$= 2.0 \frac{1.9}{100} = \frac{1.00 \times 0.92 \times 29}{1.000 \times 292}$$
(6) in the case of $M = 0.82 = 82000^{19} = 0.0035$

$$\frac{As}{bd} = \frac{1.006}{100 \times 29} = 0.0035$$

$$\frac{As}{bd} = \frac{82000}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 310 \frac{1.9}{100} = \frac{2.82000}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{2.82000}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{2.82000}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

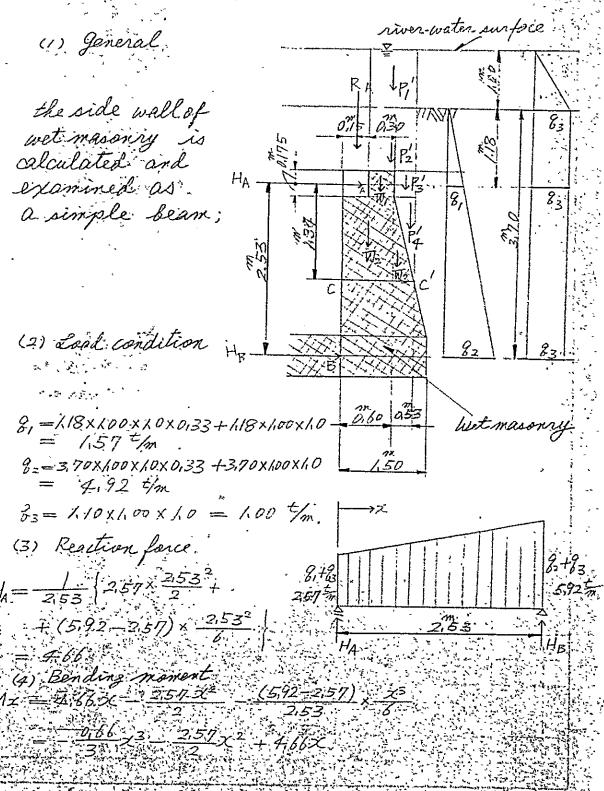
$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100 \times 292}$$

$$= 3.0 \frac{1.9}{100} = \frac{1.9}{0.0035 \times 0.90 \times 100}$$

$$= 3.0 \frac{1.9}{0.0$$

2-3-1-3 Safety colculation of side wall of let masonry



d.Mx = -0.6622-2.57x+4.66. $\therefore \chi = \frac{+2.57 \pm \sqrt{257^2 + 4 \times 0.66 \times 4.66}}{-2 \times 0.66}$ $= 1.34^{m}$ $\therefore M_{max} = M_{x=1.34}$ $= -\frac{0.66 \times 134^3 - 2.57 \times 134^2 + 4.66 \times 134 = 3.40}{3}$ (5) Check of stress on section C-C', that shows maximum moment; arm longth RA = 4,36. 0,15 W1 = 0,35 × 0,30 × 1,00 × 2,5 = 0,26 0.45 To = 1/7x 0,80x 1,00x 2,5 = 1,76. 0,30 $W_3 = 1.17 \times 0.53 \times \frac{1}{2} \times 1.00 \times 2.5 = 0.78$ 0.77 P' = 100x 0.83x 100 x10 = 0.83 0.71 P2 = 1.01 x 0.83 x 1.00 x 2.0 = 1.68 0.71 0.86 B' = 0,35 × 0,53 × 1,00 × 2,0 = 0,37 P4 = 1.17 x 0,53x = x1.00x2.0= 0,62. 0.95 Σπ=./0.66 t. Mc = RA x 0,15 + W, x A45 + W2 x A30 + W3 x A77

· + P(x0.71 + B'x0.71 + B'x0.86+ P4x0.95 + Mmax, = 799 t-m.

 $\overline{z} = \frac{M_c}{\overline{z} T_c} = \frac{7.99}{10.66} = 0.75$ $e = \left| \frac{B}{2} - \overline{z} \right| = \left| \frac{1/3}{2} - 0.75 \right| = 0.185,$

 $\frac{B}{6} = \frac{1.13}{6} = 0.188$

now strain stress is not occurred on section C= C! that shows maximum moment

23-2. No. 2 and No. 3. brigation Synton		
2-3-2-1. Top floor earth surface		
The state of the s		
(1) General; 2 130 130 120		
Reserve Server S		i.
150		
-up		
	er pce:	ĝig≃ ,the ,togle
concrete)		7 - 1 ·
	2)	
· · /	,	,
. Wet masonry . Crection	レノオー	3.
Only the top floor is calculated and	ا منابع منابع	
examined as a simple beam:	44	, .
unit weight concrete: 24 tm3	a mir china albir page d	
earthin water: 2:0 1/m3	:	•
beam length: 150m		~*
$\sigma_{sa} = 1.400 \cdot \frac{k_0}{cm^2}$,	
$\delta ca = 40 \frac{kg}{an^2}$	deringe, o	
$Tca = 4.5 \frac{kg}{cm^2}$		
water Read = 3.00 m		
	- "	
(2) Load condition		
Earth		3
$60 = 400 \times 100 \times 200 = 8000 \cdot 7000$		î,e
THE REPORT OF THE PROPERTY OF		

Top floor 32 1 00 × 2.4 = 0.72 1/m

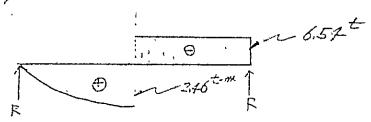
Total load: 80

3. 81+8= 8.72 1/m

(3) reaction faice. $R = 20 \times 1.50 \times \frac{1}{2} = 6.59^{\pm}$.

(4) bending-moment: $M_{mox} = R \times \frac{150}{2} - 8 \times \frac{150}{2} \times \frac{150}{2} \times \frac{1}{2}$ $= 6.54 \times \frac{1.50}{2} - 8.72 \times \frac{1.50^{2}}{8}$ = 2.46 t-m

(5) shearing force: (5) may = R = 6.54 t.



M-figure, S-figure,

(6) Roinforcement amount

Assuming
$$d = 24^{cm}$$
.

As = $\frac{M}{\sqrt{34 \times 7/3 \times d}}$

$$= \frac{246000}{14007 \frac{2}{8} \times 24}$$

$$= 8.37^{cm^2}$$

designed amount.
\$19.00 300 (As = 9.45 cm²)

(1) Chock,

$$p = \frac{As}{bd} = \frac{9.45}{100 \times 24} = 6.0059$$

$$k = \sqrt{(xp)^2 + 2np} - 7p = 0.29$$

$$j = 1 - \frac{k}{3} = 0.90.$$

$$\sigma_S = \frac{M}{PJbd^2} = \frac{24.6.000}{0.0039 \times 0.99 \times 100 \times 240}$$

$$= 1.217 \frac{13}{6n^2} = \frac{1.400 \frac{19}{6n^2}}{0.00} =$$

2-3-2-2, Examination of inside pressure,

top floor
(Reinforced: Concrete)

wet new cory.

(section).

inside pressure, operating against the top floor from inside to upward, is desided by water head distance $H=3.00^{m}$;

so that; $P_{W} = H \times W \times 1.00$ $= 3.00 \times 1.0 \times 1.00 = 3.00 \text{ fm}$

Tap floor $8 = 0.72 \frac{t}{m}$ Total closed. $30 = R_0 - 3_1 = 2.28 \frac{t}{m}$

In this case, learth pressure over the top floor is neglected;

(2) Reaction force
$$R$$
 $R = 3.28 \times 1.30 = 1.48$

(3) Bending monext: M
 $M_{max} = R \times \frac{1.30}{2} - 80 \times (\frac{1.50}{2})^2 \times \frac{1}{2}$
 $= 0.48 \text{ t-m}$

(4) Shearing force,

 $S_{max} = R = 1.48 \text{ t}$.

(5) Kein forcement omeint one into assuming, $d = 24 \text{ cm}$
 $A_S = \frac{M}{0.51 \times 1/9 \times d} = \frac{4.8000}{1.500 \times 7/9 \times 24}$.

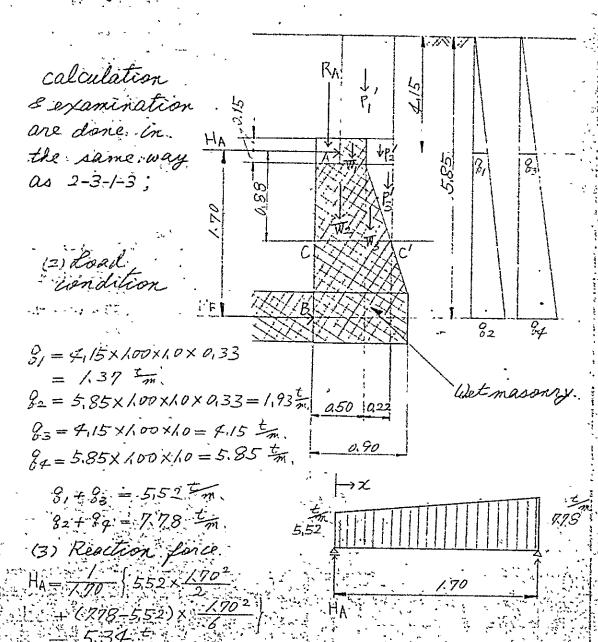
 $A_S = \frac{M}{0.51 \times 1/9 \times d} = \frac{4.8000}{1.500 \times 7/9 \times 24}$.

(6) Cleck

 $P = \frac{1.63 \text{ cm}^2}{500 \times 24} = 0.0018$
 $R = \sqrt{(np)} + 2np - np = 0.49$
 $J = 1 - \frac{2}{3} = 1 - \frac{0.49}{3} = 0.84$
 $G_S = \frac{M}{PIDd} = \frac{4.9000}{0.0018 \times 0.84 \times 1/00 \times 24^2}$
 $= 557 \frac{19}{100} = \frac{1.400 \text{ Win}}{0.918 \times 0.84 \times 1/00 \times 24^2} = \frac{1.8000}{1.9100 \times 0.9100} = \frac{1.8000}{0.9100 \times 0.9100} = \frac{1.9000}{0.9100 \times 0.9100} = \frac{1.9000$

2-3-2-3 Safety culculation of side wall of wet masonry.

(1) General



```
(4) Bording moment
  MX = -0.66x2-552x+5.34
                        \Rightarrow x = -5.52 \pm \sqrt{5.52^2 + 4 \times 0.66 \times 5.34}
   M_{MQX} = M_{X} = 0.88.
= -\frac{0.66}{3} \times 0.89^{3} - \frac{5.52}{2} \times 0.89^{2} + 5.34 \times 0.88
= 2.71 t \cdot m
     (5) Checker of stress on section C-C'.
           vertical load:
                                                               arm length
 RA= 6.54
                                                                      0,13
 W_1 = 0.30 \times 0.25 \times 1.00 \times 2.5 = 0.19
                                                                      0,38
  \overline{W}_2 = 0.73 \times 0.50 \times 1.00 \times 2.5 = 0.9/
                                                                      0,25
                                                                      0,57
  W_3 = 0.73 \times 0.22 \times \frac{1}{2} \times 100 \times 2.5 = 0.20
  P' = 400 × 0,47 × 1,00 × 2,0= 3,76
                                                                      0.48
  P'= 0,30x 0,22 x 1,00 x 2,0= 0,13
                                                                       0.61
   P_3' = 0.73 \times 0.22 \times \frac{1}{2} \times 1.00 \times 2.0 = 0.16
                                                                       0,67-
   IV= 11.89 t
 Mc = 6.54 x 0.15 + 0.19 x 0.38 + 0.9/x 0.25 + 0.20 x 0.57
      +3.76 x 0.48 + 0,13 x 0,61 + 0,16 x 0,64 + 2.41
      =5.65
\overline{z} = \frac{M_c}{z \, \overline{v}} = \frac{5.65}{1/.87} = 0.475
e = \frac{B}{2} - z = \frac{0.72}{2} - 0.475 = 0.115
  now strain stress is not occurred
```

on section C=C

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2-4-1, Na.1. and No.3. Insignifican Culvert.

2-4-1-1 Top floor

(1) General,

top floor

(Reinforced concrete)

Wet masony

(cross section)

only the top floor is calculated and cexamined as a continuous beam:

beam length; 6.00 m.

one span length; 3.00 m.

allowable stress for compression of concrete,

Sca = 40 kg/cm²

allowable stress for tension of rainforcement

Tsa = 1,400 kg/cm²

allowable shearing stress of concrete kg/

وُعَادَدُ مُنْ عُولَ إِنَّا

2) Load condition

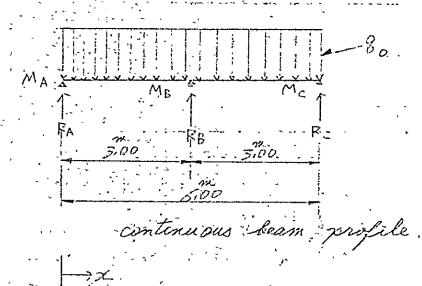
Water

3, = 5.15 x1.00 x1.8

Top flows 2= 0.55 × 100× 24 = 1.32 t/m

Total load

8= 8, + 9= = 10,59 t/m.



(3) Bending moment and shearing force.

 $M_A + 4M_B + M_c = -\frac{2}{6} \times 3,00^{\circ}$

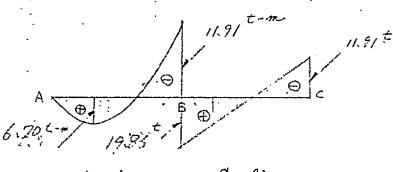
 $M_A = M_C = 0$ so that $M_B = -9_0 \times 3.00^2 \times \frac{1}{8} = -10.59 \times 3.00^2 \times \frac{1}{8}$ $= -11.91 \cdot t - m$

(a) Reaction force $R_{B} = \frac{5}{4} 80l = \frac{5}{4} \times 10.59 \times 3.00.$ $= 32.71 \pm \frac{3}{8} \times 10.59 \times 3.00.$ $= 11.91 \pm \frac{3}{8} \times 10.59 \times 3.00.$

(4) Bending moment $0 \le \pm \le 5.00$ $M_{\chi} = + R_{A} \times - \stackrel{?}{0} \times \frac{\chi^{2}}{2}$ $= + 11.91 \times - \frac{10.59}{2} \times$

 $M_{mox} = M_{x} = 1.12$ $= \frac{110.59}{2} \times 1.12^{2} + 11.91 \times 11.2$ = + 6.70 + 0.00

(c) Shearing force, 1 Smay = -10,59 x 3,00 + 11.91.



(4) Reinforcement amount

(a) in the case of $M = 11.91 \stackrel{t-m}{=} 1.191.000$

. Assuming. d = 0.49 = 49 m

 $A_{5} = \frac{M}{\sigma_{50} \times 7/8 \times d}$

= 19.84 cm²

· designed amount:

 $\phi_{2200150}$ $(1_6 = 25.34^{cm^2})$

in the case of M= 6.70 to m= 670,000 kg-cm

\$ 160 150

(a) Echoche.

(a) in the case of
$$M = 1/91^{\frac{1}{2}-n} = 1/191.000^{\frac{1}{2}}$$
 and

$$p = \frac{As}{bd} = \frac{25,34}{100 \times 49} = 0.0052$$

$$de = \sqrt{(\pi p)^2 + 2\pi p} - \pi p = 0.33$$

$$de = 1 - \frac{2}{3} = 1 - \frac{0.33}{3} = 0.99$$

$$de = \frac{M}{p d b d^2}$$

$$= \frac{1191000}{0.3952 \times 0.89 \times 100 \times 49^2}$$

$$= 1.092 \frac{\sqrt{3}}{6\pi^2} < 1.400 \frac{\sqrt{3}}{6\pi} = 0.52$$

$$(0 \text{ K.})$$

$$de = \frac{2M}{4 d b d^2}$$

$$= \frac{2 \times 1.191000}{0.35 \times 0.89 \times 100 \times 49^2}$$

$$= \frac{2 \times 1.191000}{0.35 \times 0.89 \times 100 \times 49^2}$$

$$= \frac{33.98}{6\pi^2} < 40 \frac{\sqrt{3}}{6\pi^2} = 0.22$$

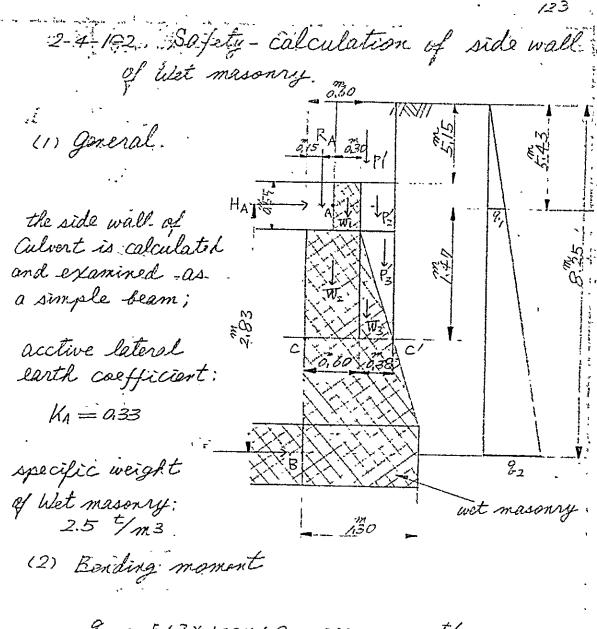
$$T_{c} = \frac{S_{max}}{b_{j} d}$$

$$= \frac{19860}{100 \times 0.87 \times 49}$$

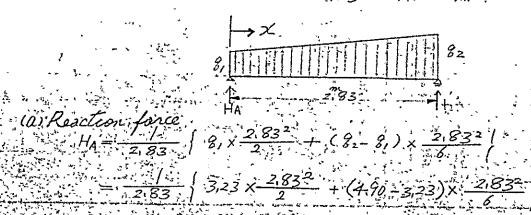
$$= 4.5 \frac{K_{g}}{cm^{2}} = 4.5 \frac{K_{g}}{cm^{2}} = T_{c}a$$

$$= (0.K.)$$

The second of th



3, = 5,+3 × 1,00 × 1.8 × 0,33 = 3,23 5/m. 82 = 8,25 × 1,00 × 1,8 × 0,33 = 4,90 t/m.



(6) Bending moment:
$$M_{\chi} = H_{A} \times - 3, \frac{\chi^{2}}{2} - (8 - 8,) \frac{1}{2.83} \times \frac{\chi^{3}}{6}$$

$$= -\frac{0.30}{3} \chi^{3} - \frac{3.23}{2} \chi^{2} + 5.36 \chi$$

$$\frac{dM_{\chi}}{d\chi} = -0.30 \chi^{2} - 3.23 \chi + 6.36$$

$$\frac{dM_{\chi}}{d\chi} = 0: \qquad \chi = \frac{3.23 \pm \sqrt{3.23^{2} + 4 \times 0.30 \times 5.36}}{-2 \times 0.30} = 1.4$$

$$\therefore M_{mex} = M_{X=1,47} \\
= -\frac{0.30}{3} \times 1.47^{3} - \frac{3.23}{2} \times 1.47^{2} + 5.36 \times 1.47 \\
= 4.09 t-m$$

(3)- Check of stress on section C-C'; this is done not to occure strain-stress on section of side wall of Wet masonry;

vertical load.	arm length
•	(from point C)
RA = 1/91 t	o,15
W. = 0,55×0,30×1.00×2.5 = 0,4/	0.45
W= (141-0,28) x0,60 x1,00 x 25=	= 1.78 . 0.30
W3 = (1.47-0.28) x 0.38 x 1/2 x 1.00 x 2	0.73 = 0.57
R'=(5,43-0,28) × 0,68×1,00 × 1,8=	1-0
P= 0.55 × 0.38 × 1.00 × 1.8 = 0.38	A 17(2)
P' = (147-0,28) x 0,38 x 1 x 1,00 x 1,8	
- 0 0 0 0	ŕ

 $\Sigma V = 21.76^{\frac{1}{2}}$. Bending moment:

 $M_{c} = R_{A} \times 0.15 + W_{1} \times 0.45 + W_{2} \times 0.30 + W_{3} \times 0.73$ $+ R_{2} \times 0.64 + R_{2} \times 0.79 + R_{3} \times 0.86 + M_{max},$ $= 11.91 \times 0.15 + 0.41 \times 0.45 + 1.78 \times 0.30 + 0.57 \times 0.73$ $+ 6.30 \times 0.64 + 0.38 \times 0.79 + 0.41 \times 0.86 + 4.07$ = 11.6.7 t-m

The state of the same of the same

$$\frac{Z}{Z} = \frac{M_c}{ZV} = \frac{111.67}{21.76} = 0.54$$

$$e = \left| \frac{B}{2} - Z \right| = \left| \frac{0.60 + 0.38}{2} - 0.54 \right| = 0.05,$$

$$\frac{B}{6} = \frac{0.60 + 0.38}{6} = 0.16$$

$$\therefore \frac{B}{6} > e \qquad (0.K.)$$

now. strain stress is not occured on section C-C', that shows maximum moment;

2-4-2-1, Top floor.

(1) General, earth surface

top floor
(Fairforced concrete)

(section) (m)

: Daly the top floor is calculated and examined as a continuous beam;

unit weight of correte: 24 5/m3.

unit weight of earth in water; 2.0 5/m3

Beam ilangth: 6.00 m,

one span length; 3.00 m.

 $\sigma_{c0} = 40 \text{ km}^2$ $\sigma_{s0} = 1400 \text{ kg/cm}^2$ $\sigma_{c0} = 4.5 \text{ kg/cm}^2$ (2) Kööd condition.

Earth

$$9/=1.95 \times 1.00 \times 3.0$$

= 3.90 t/m .

Top floor.

$$g_2 = 0.40 \times 1.00 \times 2.4$$

= $0.96 t/m$

Total load: 80

$$\hat{\beta}_{0} = \hat{\beta}_{1} + \hat{\beta}_{2}$$

$$= 4.86 \frac{t}{m}$$

10,	<u>:</u>			2
MA XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX	M _B R	M	ر اد د ۲	
TRA TO	Tre	1918.	TRC	
3,00	m 6.00	3,00		

profile of continuous bean.

 $\mapsto x_i$

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Marak 1 Add 44

اس ،

Bonding moment and shearing force,

by Method of three monents, calculation is done;

 $M_A + 4 M_B + M_C = -\frac{3}{60} \times \frac{1}{2}$. (Clapeyron eg.)

 $M_A = M_C = 0$.

so that.

$$M_{B} = -g_{0} \times 300^{2} \times \frac{1}{2} \times \frac{1}{4}$$

$$= -4.86 \times 300^{2} \times \frac{1}{2} \times \frac{1}{4}$$

$$= -5.47^{t-m}$$

(a) Reaction : force to - RA. Ro. Rc

$$R_{B} = \frac{5}{4} \% l$$

$$= \frac{5}{4} \times 4.86 \times 3.00$$

$$= 18.23 t$$

$$R_{A} = R_{C} = (3_{0} \times 6.00 - R_{B}) \times \frac{1}{2}$$

$$= (4.86 \times 6.00 - 18.23) \times \frac{1}{2}$$

$$= 5.49 t$$

(6) Banding moment.

$$M_{x} = +R_{A}x - 8x^{\frac{x^{2}}{2}}$$

$$= 5.49x - 43x^{\frac{x^{2}}{2}} - \frac{486}{2}x^{2} + 5.47x$$

$$\frac{dM_{2}}{dx} = -4.86x + 5.47.$$

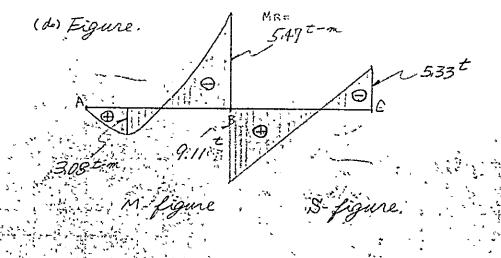
$$\frac{dM_{2}}{dx} = 0 : x = 1.13$$

$$M_{max} = M_{x=1/3}$$

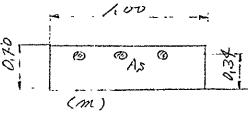
$$= -\frac{436}{2} \times 1/3^{2} + 5.47 \times 1/3$$

$$= +3.08$$

(C) Shearing force Smax = -4.35 × 3.00 + 5.47 . = 9.11 .



(4) Reinforcement amount.



(a) in the case of $M_B = 5.47^{t-m} = 547.000^{kg-cm}$.

Assuming $d = 0.34 = 34^{cm}$.

$$A_{s} = \frac{M}{\sigma_{sa} \cdot 7/s \cdot d}$$

= 545.000 1.400× 7/8×34

 $= 13.13 \text{ cm}^2$

designed amount $\phi/6 \otimes 150 \left(A_S = 13.41^{Cm^2}\right)$

(b) in the case of M=3.08 t-m 308,000 kg-om.

$$A_{\mathcal{S}} = \frac{M}{\sqrt{sa} \, \frac{\eta}{8} \, d}$$

_-<u>308.000</u> /400 x 1/8 x 34

= 739 cm

designed amount.

(As = 18,85 Cm2)

(5) Echeck.

(a) in the case of
$$M = 5.47^{t-m} = 547.000^{kg-int}$$
.

$$p = \frac{13.71}{8d.}$$

$$= \frac{13.41}{100 \times 34} = 0.0039.$$

$$k = \sqrt{(np)^2 + 2np} - np$$

$$= \sqrt{(15 \times 0.0039)^2 + 2 \times 15 \times 0.0039} - 15 \times 0.0039.$$

$$= 0.29$$

$$j = 1 - \frac{R}{3} = 1 - \frac{0.29}{3},$$

$$= 0.90.$$

$$\therefore \sigma_S = \frac{M}{Pj b d^2}.$$

$$= \frac{547.000}{0.0039 \times 0.90 \times 100 \times 34^2}.$$

$$= 1.348 \frac{kg}{cm^2} \left\langle 1.400 \frac{kg}{cm^2} = 5.52.\right\rangle$$

$$\sigma_C = \frac{2M}{kjbd^2}.$$

$$= \frac{2 \times 547.000}{0.29 \times 0.90 \times 100 \times 34^2}$$

$$= \frac{2 \times 547.000}{0.29 \times 0.90 \times 100 \times 34^2}$$

$$= \frac{18.13 \frac{kg}{cm^2} \left\langle 40 \frac{kg}{cm^2} = 5.52.\right\rangle$$

$$T_{c} = \frac{5 max}{b j d}$$

$$= \frac{9.110}{100 \times 0.90 \times 34}$$

$$= 3.0 \frac{k_{3}}{cm^{2}} < 4.5 \frac{k_{9}}{om^{2}} = T_{c}a.$$

$$\begin{array}{lll}
(b) & \text{in. the case of : } M = 3.08^{t-m} = 308.000^{t}8^{-cm} \\
p = \frac{As}{bd} = \frac{9.85}{100 \times 34} = 0.0026. \\
k = \sqrt{np}^2 + 2np - np \\
= \sqrt{(15 \times 0.0026)^2 + 2 \times 15 \times 0.0026} - 15 \times 0.0026. \\
= 0.24 \\
i = 1 - \frac{k}{3} = 1 - \frac{0.24}{3} = 0.92. \\
C_S = \frac{M}{Pdbd^2} \\
= \frac{308000}{0.0026 \times 0.92 \times 100 \times 34} = \frac{1114}{2} \times \frac{1400}{3} = \frac{15}{10} = 650. \\
C_S = \frac{2M}{4jbd^2} \\
= \frac{2 \times 308000}{0.24 \times 0.92 \times 100 \times 34} = \frac{2 \times 308000}{0.24 \times 0.92 \times 100 \times 34} = \frac{2}{100} = \frac{2}{100}$$

$$T_{c} = \frac{S_{max}}{b_{j}d}$$

$$\vdots \quad g_{j} \gamma_{j} o_{s}$$

$$100 \times 0.72 \times 34$$

2-4-2-2. Safety calculation of side wall of bet masonry.

(1) general.

calculation & examination are done in the same way as that before, (2) Lood condition. 8, = 0.50 × 1.00 × 1.8 × 0.33 = 0,30 5m wet masonry 0,60.0.51 82= 1.65×1.00×1.0×0.33 = 0.54 tm. 83= 4,35 x100 x10 x 0,33=1.44 == 84=165×100×10=165 5m. 85=4,35 x 1,00 x 1,0 = 4,35 tm. 8,+8=+84= 2.49 5m. 8, + 83 + 85 = 6.09 5m (3) Reaction force

0.15 (1x.) Pi'= 0,50 × 0.8/ × 1,00 × 1.8= 0.73 P=145x081x100x 20 = 2,35 0,70 0.85 P'= 0.40 × 0.51 × 1,00 × 2.0= 0.41 0,94 P/ = 1.24 x 0,5/x 1/2 ×1.00 × 2,0 =. 0,63

∑ v= /2.43 €

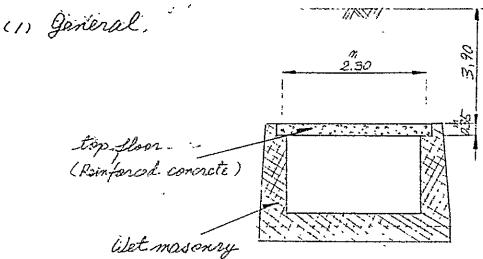
Bending monunt: Mc = 5,36 × 0,15 + 0,30 × 0,45 + 1.86 × 0,30 + 0,79 × 0,77 +0.73×0.70 + 2.35×0.70+ 0.41×0.85 +0.63×0.94 + 3:92 t-m. = 943

 $\bar{x} = \frac{M_c}{\Sigma T_0} = \frac{9.13}{12.43} = 0.73.$ $e = \frac{B}{2} + \frac{1}{2} - 0.73 = 0.18.$

- 11 - 10,19 W

stress is not occured on section

2-4-3. No.4. Irrigation Culvert, 2-4-3-1. Top floor



(section)

Only: the top floor is calculated and examined as a simple beam:

Unit Weight: | concrete:

2.4 5/m3

learth:

1.8 5/m3

. Seam length: 1.85 m

Osa = 1.400 19/cm2.

Oca = 40 19/cm2

Tca = 4.5 Kg/cm=.

(2) Load condition

Earth is so

8, = 3,90 × 1,00 × 1,8 - 7.02 5m.

Top floor

82 = 0.35 x 1.00 x 2.4 = 0.84 5m.

Total bad

8 = 18, +82 = 7.86 t/m

(3) Banding moment and shearing force.

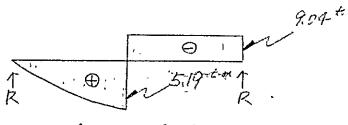
(a). Reaction face, $R = \frac{9}{0} \times 2.30 \times \frac{1}{2} = \frac{286 \times 2.30 \times \frac{1}{2}}{2}$ $= \frac{204}{5} \pm \frac{1}{2}$

(6) Bending moment.

 $M_{max} = \frac{190 \times 2.30^2}{80 \times 2.30^2}$ = $886 \times 2.30^2 \times \frac{1}{80}$ = 5.19^{-1} t-m

(c) Shearing force.

Smax = R = 9.04 t



M-figure Q-figure.

(4) Reinforcement amount.

Assuming $d = 29^{cm}$. $A_S = \frac{M}{\sigma_{SA} \times 7/8 \times d}$.

= 512,000:

14:00 × 7/8 x 29

= 14,6% cm2

designed amount

\$ 79 @ 150 (As = 18.90)

(5) Check

$$p = \frac{As}{bd} = \frac{1/8.90}{100 \times 29} = 0.0065$$

$$f_{2} = \sqrt{(np)^{2} + 2np^{2}} - np = 0.35.$$

$$j = 1 - \frac{40}{3} = 1 - \frac{0.35}{3} = 0.88$$

$$\ddot{\sigma}_{S} = \frac{M}{p \cdot \dot{j} \cdot b \cdot d^{2}}$$

$$= \frac{5/9.000}{0.0065 \times 0.83 \times /00 \times 29^{2}}$$

$$= 1.079 |_{00m^{2}} /_{0m^{2}} /_{1400} |_{0m^{2}} = \sigma_{Sa}$$

$$(0.K)$$

$$\sigma_{c} = \frac{2M}{\frac{2}{jbd^{2}}}$$

$$= \frac{1 \cdot 2 \times 5/9,000}{0,35 \times 0.88 \times 100 \times 29^{2}}$$

$$= 40 \frac{3}{m} \cdot \left(40 \cdot \frac{3}{m^{2}} = \sigma_{ca}\right)$$

$$(0.K.)$$

$$\begin{array}{rcl}
\text{$T_c = \frac{Smax}{b j d}$} \\
&= \frac{9.040}{100 \times 0.88 \times 29}
\end{array}$$

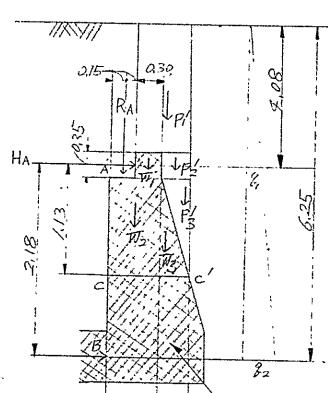
(m)

Wet masony:

2-4-3-2. Safety calculation of side wall of wet masonry.

(1) General.

calculation & . examination are done in the same way;



(2) Load condition

8, = 4.08 × 1.00 × 1.8 × 0.33 = 2.42 tm

·G2 = 6.25 x 1.00 x 1.8 x 0.33.

= 3,71 5m

±m → x 11 11 11 11 11 11 11 11 11 11 11 11 1		37/
3118	3	<u>.</u>
HAD		

(3) Reaction force. $H_A = \frac{218}{218} \left\{ 2.42 \times \frac{218^2}{2} + (3.71 - 2.42) \times \frac{218^2}{8} \right\}$

= 31/1 = 31/2 =

730 23 - 242 22 + 31/20

 $\frac{dM_{Z}}{dz} = -0.30z^{2} - 2.42z^{2} + 3.1/c$

 $\frac{dM_{X}}{dX} = 0 \implies \chi = \frac{-2.42 \pm \sqrt{2.42^2 + 4 \times 0.30 \times 3.11}}{2 \times 0.30}$

 $M_{may} = M_{\chi'=1/13}$ = $-\frac{0.30}{3} \times 1.13^{-3} - \frac{2.12}{2} \times 1.13^{-1} + 3.11 \times 1.13$ = 1.82 t-m

(5) Check of stress on section C-C'.

vertical load RA = 5.36 t	arm length 0.15 m
$R_A = 5.36$ t	0.15 m
$W_1 = 0.35 \times 0.30 \times 1.00 \times 2.5 = 0.26$	0.45
.W = 0.95 x 0,60 x 1,00 x 2,5 = 143	0.30
$W_3 = 0.95 \times 0.19 \times \frac{1}{2} \times 100 \times 2.5 = 0.20$	0.66
P1 = 3,90 × 0,49 × 100 × 1,8= 3,30	0,53
P' = 0,35 × 0,19 × 100 × 18= 0,11.	0,88
P1=0.95×0.19×±× 100×18=0.15.	0,72

ΣT= 10.81

 $M_{C} = 5.36 \times 0.15 + 0.26 \times 0.45 + 1.43 \times 0.30$ $+ 0.20 \times 0.66 + 3.30 \times 0.53 + 0.11 \times 0.68$ $+ 0.15 \times 0.72 + 1.82$ = 5.23 + 0.48 $\overline{x} = \frac{5.23}{10.81} = 0.48$ $e = \left| \frac{B}{2} \right|_{T} \overline{x} = 0.10$

Bi = 0.13

 $\frac{B}{\delta} > e$

now strain stress is not occured on section C-C!;

2-5-1 NO.4 Bridge

2-5-1-1 Design critéria

Type: Reinforced Concrete Bridge (Simple Stab)

Bridge Length , 7" 300

Width 5 mas

(Effective 4"520)

Span 6"900

Line Load Truck Load To 12 ton

Allowable Stress for compression For - 40 19/1002 of concrete

Adlowable stress for tension (so - 1-00" of reinforcement

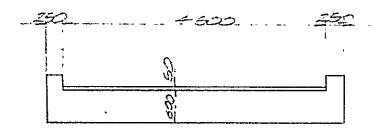
unit.weight | concrete = = 2.3 t/m3

Reinforced concrete &r = 2.4 tin3

2-3-7-2 fiven dimension,

Profile £20°. Plan center of road. 2-5-1-3 & Rood

11) Boad Load



(Per one meter of width)

Pavement (Concrete)
$$2.3 \cdot c.05 = 0.115$$

Slab (R. Conevete) $2.4 \cdot 0.00 = 1.440$
 $\Sigma W = 1.559 t/m$

12) Live load

Truck Load T = 12 tonInpact conficient $i = \frac{20}{50-1}$ B = 7000 (1 = Span) $P_{5} = 1.4 \text{ ton}$ $P_{7} = 5.6 \text{ ton}$

2-5-1-4 Bending moment

- (1) Barding market by dood load (Md)

 DEAD LOAD $W = 1.559 \, tm$ $Md = \frac{1}{8} WL^2 = \frac{1}{8} \times 1.559 \times 6.9^2$ $= 9.278 \, tm$
- The bending moment by live lood M. (M) The bending moment (M) at the center of bridge upon caused by the live load (including injunct land) can be obtained from the next formula, providing that the moment is expressed per one meter of bridge width and the bridge is of simple support slab with the span of less than 10 m and has no projecting side-arm portions and the sprive-way and-walk-way are not separated.

 ML = (1.8 L + 0.5): 0.7 (t.m)

in. [!= Span ---- 2" = [= 10"]

1=6.9m

: MR = (1.8 + 6.9 - 2.5) - 0.7 = 9.04 to

- (3) Total bending moment (Ex1)

ZM= Md-M1 = 9.278+9.0== = 18.322 +.m

2-5-1-5 Reaction force

· 11) Pead Load

Curb ised $Pc = 2.4.0.25 - 0.2.2 - \frac{1}{5.0}$

W'= 1.559 + 0.05 = 1.609 t/m

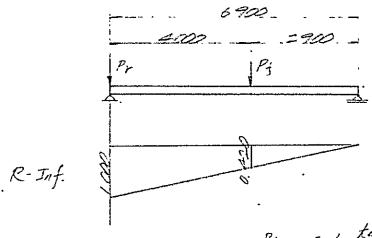
Rd = 1 WE = 1 × 1.609 x 7.2 = 5.873 ton

· (l'= Bridge length)

Per One meter of width

 $WL = \frac{RL}{B} = \frac{5873}{5.0} = 1.175$

(2) Live lood



Pr = 5.6 ton P= = 1.4 ton

 $RI = 2 (Pr \cdot 1.20 - P \cdot 0.220) \cdot (1 - 1)$ $= 2(5.6 \cdot 1.000 + 1.4 \cdot 0.420) \cdot (1 - 0.351)$ = 16.720 tonPer one metr of width $w_0 = 3.5 + 1/n$

(3) Total reaction force. $\Sigma R = RA + RA = 22.593 \text{ ton}$

Per one meter of midth

 $\frac{ER}{B} = \frac{22,593}{5,0} = 4,52 \, t/m$

2-5-1-6 Slib section and amount of main rainforcement; M=18.673 t.m = 1867300 Eg. an

$$\frac{g_{1}^{2}}{g_{2}^{2}} = 1 = 2.255 \text{ m}^{2}$$

As = 2.835, 100 = 28.85 cm2 AS = 2.855. 100 =1-.175 mt

$$\chi = -\frac{\kappa(As + As')}{b} - \sqrt{\frac{n(As + As')}{b}} \left(\frac{1}{b}, \frac{2n}{b}(dAs + d'As')\right)$$

 $= \frac{15(38.35-14.175)}{100} \cdot \sqrt{\frac{15(28.35-14.775)}{100}(-2.15)(54-28.35-6\times14.175)}$

16.514 CRU

$$K_{c} = \frac{6\pi}{2} (L - \frac{\pi}{3}) + nA_{s}' \frac{\pi - \lambda'}{\pi} (\lambda - \lambda')$$

$$= \frac{100 \cdot 16.544}{2} (54 - \frac{16.544}{3}) - 15 \times 14.175 \cdot \frac{16.544 - 6}{16.544} \times (54 - 6)$$

$$= 46.612 \text{ cm}^{3}$$

 $Ks = \frac{Ka}{n} \frac{R}{d-R} = \frac{46612}{15} \times \frac{16.541}{54-16.544} = 1372.5$

concrete $S_c = \frac{M}{K_c} = \frac{1832200}{46.612} = 39 \frac{49 \text{ m}^2}{40.612} < 40.612 \text{ m}^2$

Reinforcingt $S = \frac{M}{KS} = \frac{1832200}{13725} = 1.335 \% = 1400^{\circ}$

2-5-1-7. Distribution of reinforcement

" (1) tension side

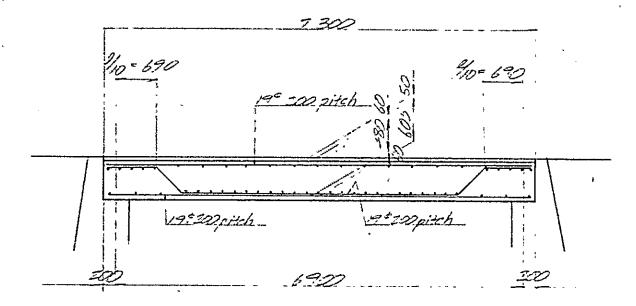
$$As = 28.35 cm^{2}$$

$$A = \frac{0.6}{\sqrt{\lambda}} \le 0.5 \qquad (1 \cdot S_{\text{rean}})$$

$$= \frac{0.6}{\sqrt{6.9}} = 0.228 < 0.5$$

(2) compression side.

(3) Runforcement



SWIT CONCERNED AND A PROPERTY OF

2-5-2-1 Deign criteria.

Runjorced Concrete Erity (Simple Sta Bridge Length : 4"200 Truck land Told ton Live bond

Allowable stress for compression Ta = 40 W/cm Allowable stress for tension of reinforcement,

unit weight

1 Concrete . Te = 2,3 · 1/m3

Reinforced concrete TV = 2.4. this

2-5-2-2 Given Dinansion

Profile Blan Center of road assis section

"SANTO CONSELTANTS OF LIGHTAN

2-5-2-3 Rood

(1) Pead load.

(Per one moter of width)

Pavement (Concrete) 2.3 · 0.05 = 0.115

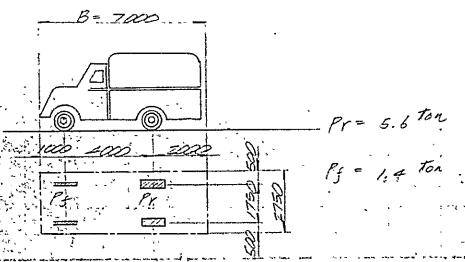
Slub (R-Concrete) 5.4 · 0.5 = 1.200

=W = 1.315

12) Rive load

Truck Load To 14 ton

Inpact coefficient $1 = \frac{20}{50 \pm 3.4} = 0.368$



CAMPO CORSULTAN & MIRHULT DAWN

2-5-2-4 Berding moment.

(1) Bording moment by dead load. (Md)

DEAD LOAD w = 1.515 Mm $Md = \frac{1}{8}wl^2 = \frac{1}{8} \cdot 1.315 \cdot 4.4 = 3.182 \text{ t.m}$

The bending moment (Me) at the Center of bridge span coused by the live land (including inpact load) can be obtained from the next formula, providing that the moment is expressed per one neter of bridge width and the bridge is of single emport shab with the span of less than 10 m and ha no projecting side-arm portions and the drive-way and walk-way are not interpreted.

Ml=, (1.81+0.5)x0.7 (time)

1= 4.4 m

16 Me = (18-14-0.5).07 = 5.89c

13) Total bending moment (ZM)

IM= Md-Ml = 3.182-5.894 = 9.076 tim

2-5-2-5 Reaction.

(1) Read load

Curo Load Pc = 2.4 + 0.25 - 0.2 x 2 x 5.0 = 0.048 t/m

W=1.315-0.048 = 1,363 the

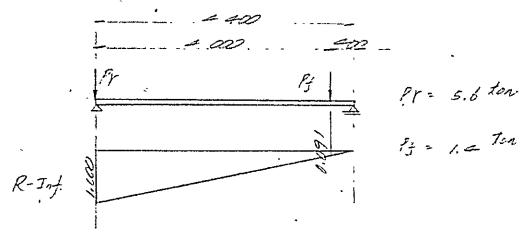
Rd = = 1W11' = 1 1363 + 48 = 3.271 ton

(l' = Bidge length)

Per one meter of width

 $Wd = \frac{Rd}{B} = \frac{3.271}{5.0} = 0.65 t/m$

(2) Live load.



$$Rl = 2(Pr \times 1.000 - P_{+} \cdot 0.091) \times (1+2)$$

$$= 2(5.6 \cdot 1.000 + 1.4 \times 0.091) \times (1+0.368)$$

$$= 15.670 \text{ ton}$$

Per one meter of width We = 15.870 = 3.13 t/m

(3) Total reaction for a $\Sigma R = Rd - Rl = 18.887^{-701}$

Per one meter of width

\[\frac{ZR}{B} = \frac{18.887}{500} = 3.797 \frac{t}{m}
\]

2-5-2-6 Slab section and amount of main sainforcement

M=9.076 tim = 907500 kg and

$$\frac{100}{160} \quad a = \pm 0.000$$

$$As = 2.011 \cdot \frac{100.5}{10.0} = 20.11 \text{ cm}^{2}$$

$$P = \frac{As}{bd} = \frac{20.11}{100 - 44.0} = 0.0046$$

$$k = \sqrt{2}np - (np) - np$$

$$= \sqrt{2} \cdot 15 \cdot 0.0046 + (15 \cdot 0.0046)^{2} - 15 \cdot 0.0046$$

$$i = 1 - \frac{k}{3} = 1 - \frac{0.309}{3} = 0.897$$

$$Ce = \frac{2M}{k_{3} k_{4}^{2}} = \frac{2 \times 9076.00}{0.309 \times 0.899 \times 100 \times 42^{2}}$$

$$= 33.8 \frac{k_{6}^{2}/cn^{2}}{0.309 \times 0.899 \times 100} \times 42^{2}$$

= 1143 - Kalen - < 1400 Plant OK.

2-5-2-1 Distribution of reinforcement.

(1) Tension side

a = Asix

$$As = 20.11 \text{ cm}^{2}$$

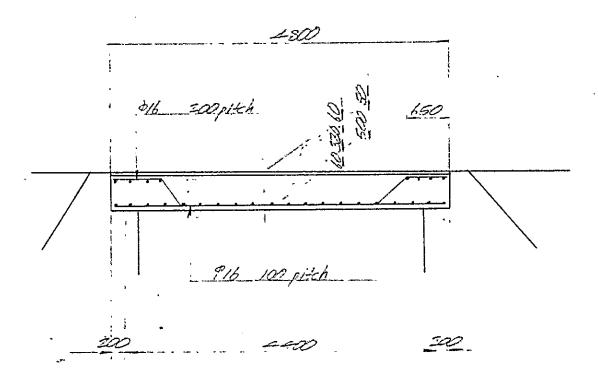
$$As = \frac{0.8}{\sqrt{2.5}} \le 0.5 \quad (1; Spm)$$

$$= \frac{0.6}{\sqrt{2.5}} = 0.286 \quad (0.5)$$

a = 20.11 + 0.286 = 5.81m2

 $\frac{18^{\circ}}{a'} = 2.011 \cdot \frac{100}{25} = 8.0 \text{ cm}^{2} > 5.8^{\circ}$

1. (2) Reinforcement



2-5-3 NO.9-Bridge

2-5-3-1 Design criteria

Type: Remforced Cororeta Bridge (Single Slab)

Bridge Lingth; 3" ax

Width 5 " ovo

(Effective 1 4"500)

Span : 2 900

Live Load Truck Load T= 14 tex

Allowable stress for compression on 40 49/cm² of concrete:

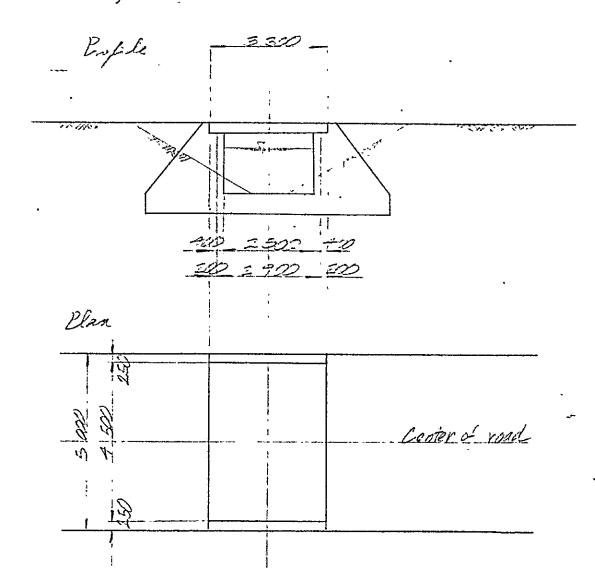
Allowable stress for tension of reinforcement;

Concrete 8c= 2.3 4/m=

unit weight.

Reinfoced concrete fr = 2.4 4/m3

2-5-3-2 Given dinersion.



Cross section 500 250

BARRY SOUTH SOUTH OF COST 124

2-5-3-3 Rood

11) Dead load

21 21 121

(per one meter of width)

Povement (concrete) 2.3 + 0.05 =

Slob (R- (occrete) =.4 - 0.40 = 0.960

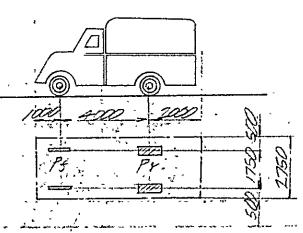
ZW = 1.075 t/m

(E) Live load

Truck Lood To 1st ton

Inport coefficient $i = \frac{20}{50 + 2.9} = 1378$

<u>5= 700</u> _



Pr = 5.6 ton

Pt = 1.4 ton

BALL IT THERE WITH THE FREE WATER

2-5-3-4 Bending moment

- : (1) Binding moment by dead load (Md)

 DEAD LOAD W = 1.075 t/m $M\ddot{a} = \frac{1}{8}WL^2 = \frac{1}{8} \cdot 1.075 \cdot 2.9^2 = 1.130 t t m$
- The bending moment by live lood (ML)

 The bending moment (ML) at the center of bridge apan caused by the live lood

 (including inpact load) can be obtained from the next formula, providing that the moment is expressed per one neter of bridge with and the bridge is of simple support shall with the span of less them 10 m and has no projecting side-orm poetions and the drive-vay and walk-way are not separated.

Me= (1.81+0.5) x0.7

(li span)

2=219. m

". ML = (1.8. =.9+0.5) ,0.7 = 4.004 !"

- (3) Total berding moment (IM).

IM= Md+ Mc = 1.130+ = w4 = 5,134 tim

2-5-3-5 Reaction force

(1) Dead load.

Curb Load

Pc = 2.4 x 0.25 x 0.2 x 2 x \frac{1}{5.0} = 0.048 t/m

N'=1.015+0.048-1.123 the

(l' = Bridge length)

Per one meter of width

Wd = Rd = 1.853 = 0.37/ t/n

:: . Pr= 5.6 ton

· Re= 5.6x2 · (1+0.378)

= 15.43+ ton

Per one meter of width We = 15.434 = 3.09 4m

(3) Total reaction force

IR = RA+RD = 17.287 ton

Per one meter of width

EP = 17.281 = 3.46 t/m

2-5-3-6 Slob-section and amount of main rainforcement

M= 5./34 tom = 5/3400 by an

150 2= 2.011.cot.

As = 2.011 x - 100.0 = 13.407 cm2

 $p = \frac{As}{bd} = \frac{13.207}{105 \cdot 34} = 0.0039$

 $k = \sqrt{2np - (np)^2 - np} = \sqrt{2.15.0.0039 - (15.0.0039)^2 - 15.0.0039}$ - 0.289

 $\dot{k} = 1 - \frac{\dot{k}}{3} = 1 - \frac{0.89}{3} = 0.904$

 $C = \frac{211}{k_b^2 b d^2} = \frac{2 \times 513400}{0.289 \times 0.904 \times 100 \times 3.5^2}$ $= 34 \cdot k_b^2 lem^2 < 40 \cdot k_b^2 lem^2$

(s= M. 513+00 Asigid. 13,40.7. 0.904 x 34.

= 1.24-6 Flan - <

<1400 Folon= OK

2-5-3-7 Distribution of reinforcement.

(1) Tension side.

1 = As. d

As= 13.407 cm=

 $\chi = \frac{06}{I_{5.9}} = 0.352$

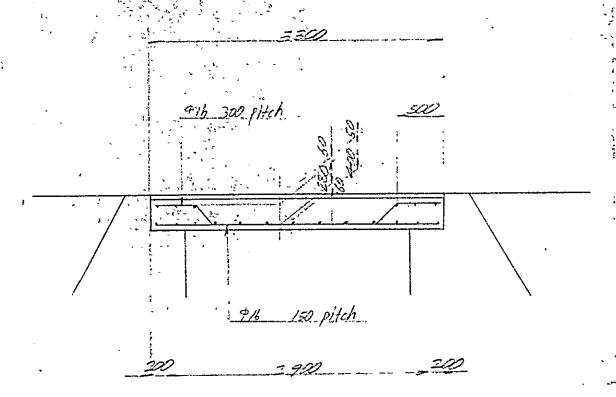
a=13,407x0.352 = 4.7 cm=

_16 th 300 pitch .

a = 2.011 Cx

9'=2.011. - 100 = 6.7 cm > 4.7 m2

(2) Reinforce ment



167:

2-5-4 Weeder Still

2-5-2-1 Design criteria

Type ; Wooden simple beam bridge

Bridge length; 7 "300

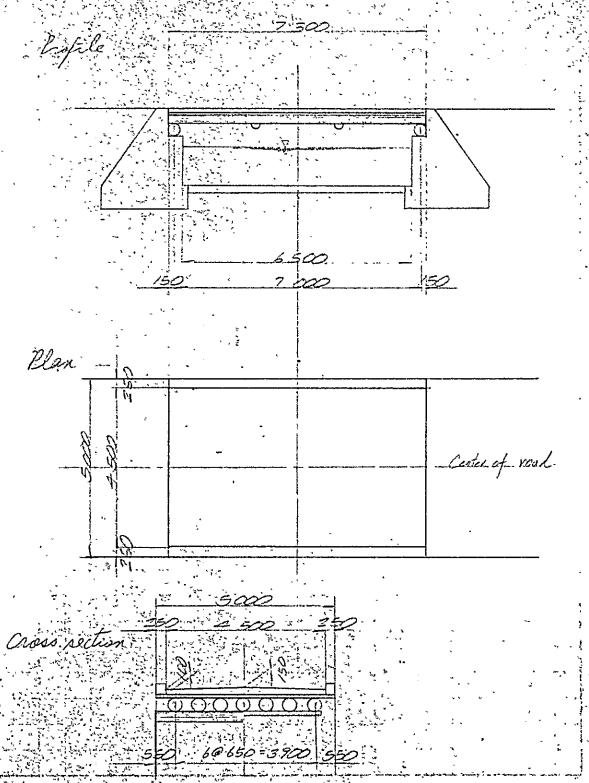
Width : 5" 000 (Effective = "500)

Span ; 7" 000

Live load; Truck load T = 14 ton

Allowable stress; Wood Two= 120 Kg/om² fortersion.

25-4-2 general view



esangue in oscil tause en expertue de

- -5-4-3 Road and bending moment
 - 1) Dead load.

(Per one meter of width)

Curb 0.8.0.25.0.25.2 /3.0 = 0.000

Wet soil 1.8.0.125.4.5 /3.0 = 0.003

Board 0.8.0.09.5.0 /5.0 = 0.072

Girder 0.8.7.027.7 /3.0 = 0.141

0.456

(2) Live lood.

Truck land. T= 14 ton

(P = 1. = ton

Pr = 5.5.

In part conficient $i = \frac{20}{50-7} = 0.351$

(3) Bending moment,

Md = 1 0.236-72 = 2.67/1.m

ML=(18,7+0.5)-0.7 = 9.170

ZM = 11.841 tim

2---- Section of main girder.

seven woods of \$150 are used for Bridge's width of 5.00M,

I= To 45 / 7 = 281.662 cm2

W I 1 = 1184100 + 15 = 945 Plor = < 120 /2

OK

The same of the sa