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

VOLUME Ⅱ

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OVERSEAS TECHNICAL COOPERATION AGENCY
TOKYO, JAPAN

ABBREVIATIONS

σ_c (σ_{ca})	:	(allowable) stress for compression of concrete	(kg/cm^2)
σ_s (σ_{sa})	:	(allowable) stress for tension of reinforcement	(")
σ_i (σ_{ia})	:	(allowable) stress for compression of cast iron	(")
τ_c (τ_{ca})	:	(allowable) shearing stress of concrete	(")
σ_w (σ_{wa})	:	(allowable) stress for tension of wood	(")
M, M_B, M_C, M_d, M_1, Mmax, Mx and ΔMx	:	bending moment	(ton-m, kg-cm)
As, As', (Av)	:	amount of reinforcement (stirrup)	(cm^2)
Ai, Ac	:	supporting area of cast iron (concrete)	(")
R, R_A, R_B, R_C, R_d, R_1, H_A	:	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)
q_o	:	total dead load	(ton/m)
Pw, q'	:	water pressure	(")
We	:	live load	(")
P_f, Pr	:	front (reverse) portion of truck load	(ton)
T	:	Truck load (live load)	(")
Pc	:	curb load	(ton/m)
Ho, (Po, No)	:	horizontal (normal) force	(")
γ, γ_1, γ_2, γ_c, γ_r, γ_w, ω	:	unit weight	(ton/m^3)
K_A	:	coefficient of earth pressure	
F	:	safety factor	
P	:	ratio of As and area of section	
m	:	number of reinforcement bars	
d	:	effective thickness	(m, cm)
t	:	thickness of the upper part of T-beam	
(b'), b, B	:	(effective) width	(m, cm)
x, (1, 1')	:	distance (of span)	(m)
H, h	:	depth	(")
h', hs_1, hs_2, hs_3, hs_4, hs, hw, hw_1	:	conversion height of load	(")
s	:	distance form the top to the Neutral Axis of T-beam	(m)
V	:	total vertical load	(t)
I	:	geometrical moment of inertia	(cm^4)
e	:	distance of eccentricity	(m)
n	:	ratio of coefficient of elasticity	

DESIGN REPORT OF
PARALKOTE IRRIGATION CANAL SYSTEM SURVEY

GENERAL CONTENTS

VOLUME II

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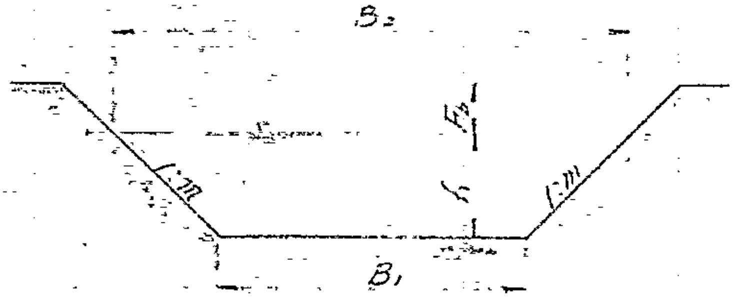
CHAPTER I HYDRAULIC CALCULATION

1-1 Hydraulic Calculation for Typical Section of Open Canal



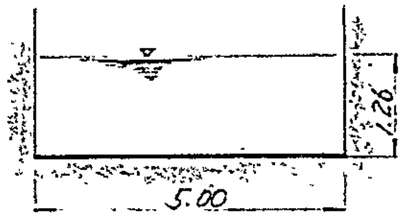
TYPE	B_1 (m)	h (m)	m	B_2 (m)	F_b (m)	A	P (m)	R
A	3.00	1.95	0.3	4.17	0.55	6.91	7.00	0.987
B	10.00	1.30	1	12.60	0.50	14.69	13.68	1.074
C	3.00	1.89	2		0.61	12.81	11.46	1.118
D	1.00	1.23	2		0.47	4.26	6.50	0.655
E	3.00	0.86	2			4.059	6.846	0.593
F	3.00	0.47	2			[illegible]	5.102	0.363
G	1.00	0.70	2			1.686	4.130	0.407

TYPE	$R^{2/3}$	n	I	$I^{1/2}$	V	$V^2/2g$	Q	REMARKS
A	0.991	0.0225	1/1,000	0.03162	1.393	0.099	9.74	> 9.63
B	1.049	"	1/5,000	0.01414	0.659	0.022	9.68	> 9.63
C	1.077	"	"	"	0.671	0.023	8.67	> 8.65
D	0.754	"	"	"	0.474	0.011	2.02	> 2.00
E	0.706	0.020	"	"	0.498	0.013	2.02	> 2.00
F	0.509	0.0225	"	"	0.318	0.005	0.589	> 0.569
G	0.549	"	"	"	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 aqueduct



Discharge	$Q = 9.63\ m^3/s$
Canal slope.	$I = 1/5\,000$
Coefficient of roughness	$n = 0.015$
Canal width	$B = 5.00$
Water depth	$H = 1.26$

Flow area $A_1 = 5.00 \times 1.26 = 6.30\ m^2$

Wetted perimeter $P_1 = 5.00 + 1.26 \times 2 = 7.52\ m$

Hydraulic radius $R_1 = A/P = 6.30/7.52 = 0.838\ m$

$R_1^{2/3} = 0.889$

Velocity $v_1 = \frac{1}{n} R_1^{2/3} I^{1/2} = \frac{1}{0.015} \times 0.889 \times (1/1{,}500)^{1/2}$
$= 1.531\ m/s$

$Q_1 = A_1 v_1 = 6.30 \times 1.531 = 9.65\ m^3/s \doteq 9.63\ m^3/s$ OK

Velocity head $h_{v_1} = v_1^2/2g = 1.531^2/2 \times 9.8 = 0.120\ m$

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

Discharge	$Q_2 = 9.63\ m^3/sec$
Canal slope	$I_2 = 1/5{,}000$
Flow area.	$A_2 = 14.69\ m^2$
Velocity	$v_2 = 0.659\ m/s$
Velocity head	$h_{v_2} = v_2^2/2g = 0.022\ m$

1-2-1-3 Loss head of upper transition ; h_{e1}

Transition length $L = 18^{m}.00$

(1) Loss head of gradual contraction.

$$h_{ed} = f_{gc}(h_{v1} - h_{v2})$$
$$= 0.2 \cdot (0.120 - 0.022)$$
$$= 0^{m}.020$$

h_{ed} : loss head of gradual contraction
f_{gc} ; coefficient of gradual contraction

(2) Loss head of friction

$$h_{el} = \frac{1}{2}(I_1 + I_2) \cdot L$$
$$= \frac{1}{2}\left(\frac{1}{1,500} + \frac{1}{5,000}\right) \times 18.00$$
$$= 0^{m}.008$$

h_{el} : loss head of friction

(3) Total loss head of upper transition.

$$h_{e1} = h_{ed} + h_{el} = 0.020 + 0.008 = 0^{m}.028$$

1-2-1-4 Loss head of lower transition ; h_{e2}

Transition length $L = 18^{m}.00$

(1) Loss head of gradual enlargement

$$h_{ei} = f_{ge}(h_{v1} - h_{v2})$$
$$= 0.3 \,(0.120 - 0.022)$$
$$= 0^{m}.029$$

h_{ei} ; loss head of gradual enlargement
f_{ge} ; coefficient of gradual enlargement

(2) Loss head of friction

$$h_{el} = \frac{1}{2}(I_1 + I_2) l = \frac{1}{2}\left(\frac{1}{1,500} + \frac{1}{5,000}\right) \times 18.00$$
$$= 0^{m}.008$$

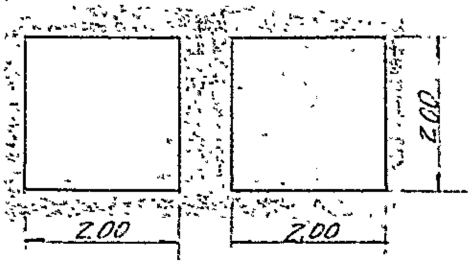
h_{el} ; loss head of friction

(3) Total loss head of lower transition

$$h_{l2} = h_{i2} + h_{o2} = 0.029 + 0.008 = 0.037^{m}$$

1-2-2 No.1 Irrigation Syphon.

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



Discharge $Q_1 = \frac{1}{2} Q = \frac{1}{2} \times 963 = 4.815 \ m^3/s$

Flow area $A_1 = BH = 2 \times 2 = 4000 \ m^2$

Coefficient of roughness $n = 0\ 020$

Velocity $V_1 = Q/A = 4\ 815 / 400 = 1204 \ m/s$

Velocity head $h_{v_1} = V_1^2/2g = 1204^2/2\times9.8 = 0.074 \ m$

Wetted perimeter $P = 4 \times 2 = 8\ 00 \ m$

Hydraulic radius $R = A/P = 400/8.00 = 1\ 500$

$R^{2/3} = 0\ 630$

Hydraulic gradient $I = \left(\frac{n \cdot V}{R^{2/3}}\right)^2 = \left(\frac{0\ 020 \times 1.204}{0\ 630}\right)^2 = 0.00146$

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

Discharge $Q_2 = 963 \ m^3/s$

Canal slope $I_2 = 1/5000$

Flow area $A_2 = 14.69 \ m^2$

Velocity $V_2 = 0.659 \ m/s$

Velocity head $h_{v_2} = 0.022$

1-2-2-3 Loss head of upper transition ; h_{e1}

Transition length $L = 8.00$

(1) Loss head of entrance

$h_e = f_i (h_{v1} - h_{v2})$

$= 0.4 (0.074 - 0.022) = 0.021$ f_i : coefficient of entrance

(2) Loss head of friction.

$h_f = \frac{1}{2} (I_1 + I_2) l$

$= \frac{1}{2} \times (0.00146 + 0.00020) \times 8.0 = 0.007^m$

(3) Total loss head of upper transition.

$h_{e1} = h_e + h_f = 0.021 + 0.007 = 0^m.028$

1-2-2-4 Loss head of syphon

culvert length $L \doteq 24.0^m$

$h_f = \frac{2 g n^2}{R^{4/3}} \cdot L \cdot h_v$ h_f : loss head of friction

$= \frac{2 \times 9.8 \times 0.012^2}{0.397} \times 24.0 \times 0.074 = 0^m.035$

Taking 10 percent allowance.

$h_f = 0.035 \times 1.1 = 0^m.038$

1-2-2-5 Loss head of lower transition ; h_{e2}

Transition length $L = 8.00^m$

(1) Loss head of exit.

$h_o = f_o (h_{v1} - h_{v2})$ f_o : coefficient of exit.

$= 0.6 (0.074 - 0.022) = 0^m.031$

(2) Loss head of friction

$h_f = \frac{1}{2} (I_1 + I_2) l$

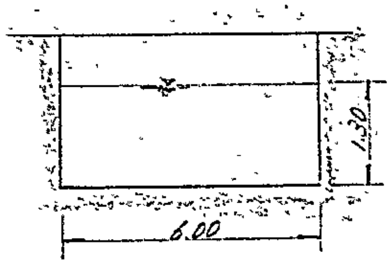
$= \frac{1}{2} \times (0.00146 + 0.00020) \times 8.0 = 0.007$

(3) Total loss head of lower transition h_{e2}

$$h_{e2} = h_0 - h_f$$
$$= 0.031 + 0.007 = 0.038^{m}$$

1-2-3. No.1 Brigde

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



Discharge $Q = 9.63\ m^3/s$
Coefficient of roughness $n = 0.0225$

Flow area. $A = 6.00 \times 1.30 = 7.80\ m^2$
Wetted perimeter. $P = 6.00 + 1.30 \times 2 = 8.60\ m$
Hydraulic radius $R = A/P = 7.80/8.60 = 0.907$
$R^{2/3} = 0.937$
Velocity $v = Q/A = 9.63/7.80 = 1.235\ m/s$
Hydraulic gradient $I = \left(\frac{n \cdot v}{R^{2/3}}\right)^2 = \left(\frac{0.0225 \times 1.235}{0.937}\right)^2 = 0.00088$

Velocity head. $h_v = v^2/2g = 1.235^2/2 \times 9.8 = 0.078$

1-2-3-2 Loss head of irrigation canal under brigde

Brigde width. $B = 4.25$

(1) Loss head due to sudden contraction

$h_{sc} = f_{sc}\, h_v$
$= 0.29 \times 0.078$
$= 0.023\ m$

$A_2/A_1 = 7.80/14.69 \doteqdot 0.5$
$\therefore f_{sc} = 0.29$
f_{sc}: coefficient of sudden contraction

(2) Loss head due to sudden enlargement

$h_{se} = f_{se}\, h_v$
$= 0.25 \times 0.078$
$= 0.020\ m$

$A_2/A_1 \doteqdot 0.5$
$f_{se} = 0.25$
f_{se}: coefficient of sudden enlargement

(3) Loss head of friction

$h_f = I \cdot L = 0.00008 \times 425 = 0.004^m$

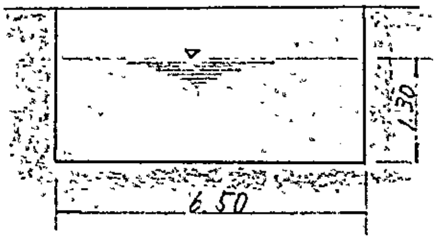
(4) Total loss head of irrigation canal under bridge.

$h_T = h_{ge} + h_{sc} + h_f$

$= 0.023 + 0.020 + 0.004 = 0.047^m$

1-2-4 No2 and No4 Bridge

1-2-4-1 Typical section of irrigation canal under Brigde



Discharge $Q = 9.63^{m^3/s}$
Coefficient of roughness $n = 0.0225$

Flow area $A = 6.50 \times 1.30 = 8.45^{m^2}$
Wetted perimeter $P_1 = 6.50 + 1.30 \times 2 = 9.10$ m
Hydraulic radius $R = 8.45/9.10 = 0.929$
$R^{2/3} = 0.952$
Velocity $v = Q/A = 9.63/8.45 = 1.140^{m/sec}$
Velocity head $h_v = v^2/2g = 1.140^2/2 \times 9.8 = 0.066^m$
Hydraulic gradient $I = \left(\frac{n \cdot v}{R^{2/3}}\right)^2 = \left(\frac{0.0225 \times 1.140}{0.952}\right)^2 = 0.00073$

1-2-4-2 Loss head of irrigation canal under brigde.
Brigde width $B = 5.00^m$

(1) Loss head due to sudden contraction

$h_{gc} = f_{gc} \cdot h_v$
$= 0.21 \times 0.066$
$= 0.014^m$

$A_2/A_1 = 8.45/14.69 \doteq 0.60$
$f_{gc} = 0.21$

(2) Loss head due to sudden enlargement.

$h_{se} = f_{se} \cdot h_v$
$= 0.25 \times 0.078$
$= 0.020^m$

$A_2/A_1 = 0.60$
$f_{se} = 0.25$

(3) Loss head of friction

$h_e = I.l = 0.00073 \times 5.00 = 0.^m004$

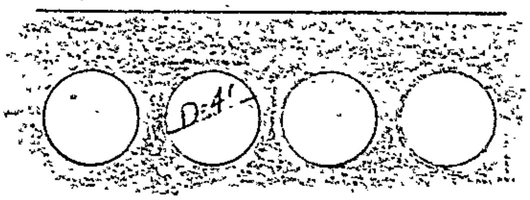
(4) Total loss head of irrigation canal under bridge

$h_\Sigma = h_{ge} + h_{se} + h_e$

$= 0.014 + 0.011 + 0.004 = 0.^m029$

1-2-5 No3 Bridge

1-2-5-1 Typical section of irrigation syphon under bridge



$Q_1 = 9.63 \, m^3/s$

$n = 0.015$

$A_1 = \frac{1}{4}\pi D^2 \times 4 = \frac{1}{4} \times 3.14 \times 1.202^2 \times 4$

$= 4.537 \, m^2$

Velocity $v_1 = Q/A = 9.63/4.537 = 2.123$ $\quad h_{v1} = 0.230^m$

Wetted perimeter $P = \pi D \times 4 = 3.14 \times 1.202 \times 4 = 15.097$

Hydraulic radius $R_1 = A/P = 4.537/15.097 = 0.301$

$R_1^{2/3} = 0.448$

Hydraulic gradient $I_1 = \left(\frac{n \cdot v}{R^{2/3}}\right)^2 = \left(\frac{0.015 \times 2.123}{0.448}\right)^2 = 0.00505$

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

Discharge	$Q_2 = 9.63 \, m^3/s$
Canal slope	$I_2 = 1/5.000$
Flow area	$A_2 = 14.69 \, m^2$
	$v_2 = 0.659 \, m/s$
	$h_{v2} = 0.022$

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

(1) Loss head of entrance

$h_e = f(h_{v1} - h_{v2})$

$= 0.5(0.230 - 0.022) = 0.104^m$

(2) Loss head of exit

$h_o = f(h_{v_2} - h_{v})$

$= 0.7(0.230 - 0.022)$

$= 0.146^m$

(3) Loss head of friction

$h_\ell = 124.5\, n^2 / D^{4/3}\, h_{v_2}\, \ell$

$= 124.5 \times 0.015^2 / 1.278 \times 5.10 \times 0.230$

$= 0.026^m$

Taking 10 percent allowance.

$h_\ell = 0.026 \times 1.1 = 0.029^m$

(4) Total loss head of irrigation syphon under bridge

$h_I = h_e + h_o + h_\ell$

$= 0.104 + 0.146 + 0.029 = 0.279^m$

1-2-6 No. 1 Transition (RD 0+317 ~ 383)

Transition length $L = 20.0^m$

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

Discharge $Q_1 = 9.63\ m^3/s$
Canal slope $I_1 = 1/1{,}000$
Flow area $A_1 = 6.99\ m^2$
Velocity $v_1 = 1.393\ m/s$
Velocity head $h_{v_1} = 0.099^m$

1-2-6-2 Hydraulic elements of B type open canal

Discharge $Q_2 = 9.63\ m^3/s$
Canal slope $I_2 = 1/5000$
Flow area $A_2 = 14.69\ m^2$
Velocity $v_2 = 0.659\ m/s$
Velocity head $h_{v_2} = 0.022^m$

1-2-6-3 Loss head of transition

(1) Loss head of gradual enlargement

$$h_{ge} = f_g(h_{v_1} - h_{v_2})$$
$$= 0.3(0.099 - 0.022) = 0.023^m$$

(2) Loss head of friction

$$h_f = \frac{1}{2}(I_1 + I_2)\,l$$
$$= \frac{1}{2}\left(\frac{1}{1000} + \frac{1}{5000}\right) \times 20 = 0.012^m$$

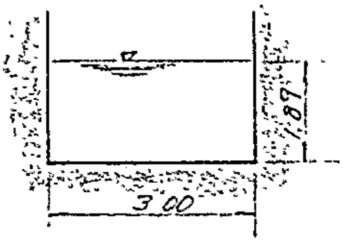
(3) Total loss head of transition

$h_T = h_{je} + h_e = 0.023 + 0.012 = 0.035$ m.

1-3 Hydraulic Calculation for Main Structure of B-Route

1-3-1 No.3 Irrigation Aqueduct.

1-3-1-1 Typical section of irrigation aqueduct



Discharge $Q = 8.65\ m^3/s$
Canal slope $I = 1/1{,}500$
Coefficient of roughness $n = 0.015$
Canal width $B = 3.00\ m$
Water depth $H = 1.89$

Flow area $A_1 = 3.00 \times 1.89 = 5.670\ m^2$
Wetted perimeter $P_1 = 3.00 + 1.89 \times 2 = 6.780\ m$
Hydraulic radius $R_1 = A/P_1 = 5.67/6.78 = 0.836$
$R_1^{2/3} = 0.887$
Velocity $v_1 = 1/n \cdot R_1^{2/3} I_1^{1/2} = 1/0.015 \times 0.887 \times 0.0258$
$= 1.526\ m/s$
Velocity head $h_{v1} = v_1^2/2g = 1.526^2/2 \times 9.8 = 0.119\ m$

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

Discharge $Q_2 = 8.65\ m^3/s$
Canal slope $I_2 = 1/5{,}000$
Flow area $A_2 = 12.81\ m^2$
Velocity $v_2 = 0.677\ m/s$
Velocity head $h_{v2} = 0.023\ m$

1-3-1-3 Loss head of upper transition ; h_{e1}

Transition length $L = 18.00^m$

(1) Loss head of gradual contraction

$$h_{ed} = f_{gc}(h_{v1} - h_{v2})$$
$$= 0.2(0.119 - 0.023) = 0^m.019$$

(2) Loss head of friction

$$h_{el} = \frac{1}{2}(I_1 + I_2)l$$
$$= \frac{1}{2}(1/1{,}500 + 1/5{,}000) \times 18.0 = 0^m.008$$

(3) Total loss head of upper transition

$$h_{e1} = h_{ed} + h_{el} = 0.019 + 0.008 = 0^m.027$$

1-3-1-4 Loss head of lower transition ; h_{e2}

Transition length $L = 18.00^m$

(1) Loss head of gradual enlargement

$$h_{ei} = f_{ge}(h_{v1} - h_{v2})$$
$$= 0.3(0.119 - 0.023) = 0^m.029$$

(2) Loss head of friction

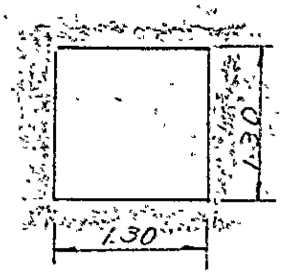
$$h_{el} = \frac{1}{2}(I_1 + I_2)l$$
$$= \frac{1}{2}(1/1{,}500 - 1/5{,}000) \times 18.0 = 0^m.008$$

(3) Total loss head of lower transition

$$h_{e2} = h_{ei} + h_{el} = 0.029 + 0.008 = 0^m.037$$

1-3-2 No.2 Irrigation Syphon

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



Discharge $Q_1 = 2.00\ m^3/s$
Coef. of roughness $n = 0.020$
Canal width $B_1 = 1.30\ m$
Water depth $H = 1.30\ m$

Flow area $A_1 = B\,H = 1.30 \times 1.30 = 1.69\ m^2$
Velocity $V_1 = Q_1/A_1 = 2.0/1.69 = 1.183\ m/s$
Velocity head $h_{v1} = V_1^2/2g = 1.183^2/2\times9.8 = 0.071$
Wetted perimeter $P_1 = 1.3 \times 4 = 5.2$
Hydraulic radius $R_1 = A_1/P_1 = 1.69/5.2 = 0.325$
$R_1^{2/3} = 0.473$
Hydraulic gradient $I = \left(\frac{n_1 V_1}{R^{2/3}}\right)^2 = \left(\frac{0.02 \times 1.183}{0.473}\right)^2 = 0.00250$

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

Discharge $Q_2 = 2.00\ m^3/s$
Canal slope $I_2 = 1/5{,}000$
Flow area $A_2 = 4.059\ m^2$
Velocity $V_2 = 0.498\ m/s$
Velocity head $h_{v2} = 0.013$

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

Discharge $Q_3 = 2.00\ m^3/s$
Canal slope $I_3 = 1/5{,}000$
Flow area $A_3 = 4.260\ m^2$
Velocity $V_3 = 0.474\ m/s$
Velocity head $h_{v3} = 0.011$

1-3-2-4 Loss head of upper transition ; h_{e1}

Transition length $L = 5.^{m}00$

(1) Loss head of entrance

$h_e = f_i(h_{v1} - h_{v2})$

$= 0.5(0.071 - 0.013) = 0.^{m}029$

(2) Loss head of friction

$h_f = \frac{1}{2}(I_1 + I_2)l$

$= \frac{1}{2}(0.00020 + 0.00250) \times 5 = 0.^{m}007$

(3) Total loss head of upper transition.

$h_{e1} = h_e - h_f = 0.029 + 0.007 = 0.^{m}036$

1-3-2-5 Loss head of lower transition ; h_{e2}

Transition length $L = 5.^{m}00$

(1) Loss head of exit

$h_o = f_o(h_{v1} - h_{v2})$

$= 0.7(0.071 - 0.011) = 0.^{m}042$

(2) Loss head of friction

$h_f = \frac{1}{2}(I_1 + I_2)l$

$= \frac{1}{2}(0.0002 + 0.00250) \times 50 = 0.^{m}007$

(3) Total loss of lower transition :

$h_{e2} = h_o + h_f = 0.042 + 0.007 = 0.^{m}049$

1-3-2-6 Loss head of syphon

culvert length $L = 50^{m}$

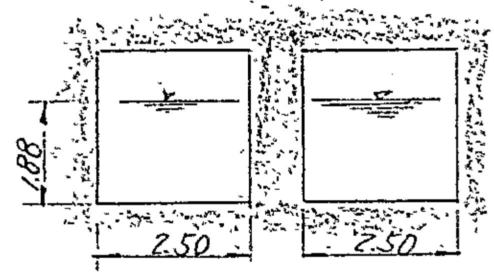
$h_f = Il = 0.0025 \times 50 = 0.125^{m}$

Taking 10 percent allowance.

$h_f = 0.125 \times 1.10 = 0.^{m}138$

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

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



Discharge $Q = 8.65/2 = 4.325$ m³/s
Canal slope $I = 1/2.000$
Coef. of roughness $n = 0.020$
Canal width $B = 2.50$ m
Water depth $H = 1.88$ m

Flow area $A_1 = 2.50 \times 1.88 = 4.70$ m²
Wetted perimeter $P_1 = 2.50 + 1.88 \times 2 = 6.26$
Hydraulic radius $R_1 = 4.70/6.26 = 0.751$ $R^{2/3} = 0.826$
Velocity $V_1 = \frac{1}{0.020} \times 0.826 \times 0.02236 = 0.923$ m/s
$Q_1 = A_1 V_1 = 4.70 \times 0.923 = \quad > 4.325$ OK
Velocity head $h_{v1} = V_1^2/2g = 0.923^2/2 \times 9.8 = 0.043$

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

Discharge $Q_1 = 8.65$ m³/s
Canal slope $I_2 = 1/5.000$
Flow area $A_2 = 12.81$ m²
Velocity $V_2 = 0.677$
Velocity head $h_{v2} = 0.023$

1-3-3-3 Loss head of upper transition ; he1
Transition length $L = 10.00$ m
(1) Loss head of gradual contraction
$h_{ed} = f_{gc}(h_{v1} - h_{v2})$
$= 0.2(0.043 - 0.023) = 0.004$ m
(2) Loss head of friction

$h_{ef} = \frac{1}{2} \times (I_1 - I_2) l$

$= \frac{1}{2} \times (1/2000 - 1/5000) \times 10 = 0.004^m$

(3) Total loss head of upper transition

$h_{e1} = h_{ed} + h_{ef} = 0.004 + 0.004 = 0.008^m$

1-3-3-4 Loss head of lower transition ; h_{e2}

Transition length $L = 10.00^m$

(1). Loss head of gradual enlargement

$h_{oe} = f_{go}(h_{v1} - h_{v2})$

$= 0.3(0.043 - 0.023) = 0.006^m$

(2) Loss head of friction

$h_{ol} = \frac{1}{2}(I_1 + I_2) \cdot l$

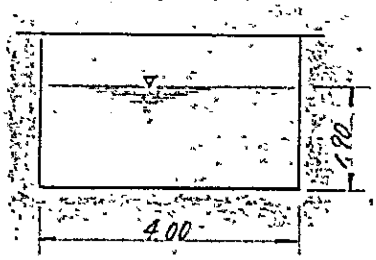
$= \frac{1}{2}(\frac{1}{2000} + \frac{1}{5000}) \times 10 = 0.004^m$

(3) Total loss head of lower transition

$h_{e2} = h_{oe} + h_{ol} = 0.006 + 0.004 = 0.010^m$

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

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



Discharge $Q = 8.65\ m^3/s$
Coef. of roughness $n = 0.0225$
Canal width $B = 4.00$
Water depth $H = 1.90$

Flow area $A = 1.90 \times 4.00 = 7.60\ m^2$
Wetted perimeter $P = 4.0 + 1.90 \times 2 = 7.80$
Hydraulic radius $R = A/P = 7.60/7.80 = 0.974$
$R^{2/3} = 0.983$
Velocity $v = Q/A = 8.65/7.60 = 1.138$
Velocity head $h_v = v^2/2g = 1.138^2/2 \times 9.8 = 0.066$
Hydraulic gradient $I = \left(\frac{n \cdot v}{R^{2/3}}\right)^2 = \left(\frac{0.0225 \times 1.138}{0.983}\right)^2 = 0.00068$

1-3-4-2 Loss head of irrigation canal under brigde

Brigde width $B = 5.00\ m$

(1) Loss head due to sudden contraction

$h_{ge} = f_{ge} \cdot h_v$ $\qquad A_2/A_1 = 7.60/12.81 \doteqdot 0.60$
$= 0.21 \times 0.066 = 0.014\ m$ $\qquad f_{ge} = 0.21$

(2) Loss head due to sudden enlargement

$h_{se} = f_{se} \cdot h_v$ $\qquad A_2/A_1 \doteqdot 0.6$
$= 0.16 \times 0.066 = 0.011\ m$ $\qquad f_{se} \doteqdot 0.16$

(3) Loss head of friction

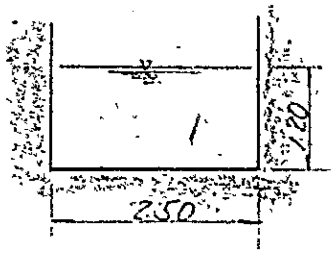
$$h_e = I.l = 0.00068 \times 5.00 = 0.003^{m}$$

(4) Total loss head of irrigation canal under bridge

$$h_T = h_{ge} + h_{se} + h_f$$
$$= 0.014 + 0.011 + 0.003 = 0.028^{m}$$

1-3-5 No.9 Bridge (Type C)

1-3-5-1 Typical section of irrigation canal under bridge



Discharge	Q	=	$2.00\ m^3/s$
Coef. of roughness	n	=	0.0225
Canal width	B	=	$2.50\ m$
Water depth	H	=	$1.20\ m$

Flow area $A = 2.50 \times 1.20 = 3.00\ m^2$

Wetted perimeter $P = 2.50 + 1.20 \times 2 = 4.90\ m$

Hydraulic radius $R = 3.00/4.90 = 0.612$

$R^{2/3} = 0.721$

Velocity $v = 2.00/3.00 = 0.667\ m/s$

Velocity head $hv = 0.023\ m$

Hydraulic gradient $I = \left(\frac{0.0225 \times 0.667}{0.721}\right)^2 = 0.00043$

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

Bridge width $B = 5.00\ m$

(1) Loss head due to sudden contraction

$h_{gc} = f_{gc} \cdot h_v$ $\qquad A/A_1 = 300/426 = 0.7$

$= 0.13 \times 0.023 = 0.003\ m$ $\qquad f_{gc} = 0.13$

(2) Loss head due to sudden enlargement

$h_{se} = f_{se} \cdot h_v$ $\qquad f_{se} = 0.09$

$= 0.09 \times 0.023 = 0.002\ m$

(3) Loss head of friction

$h_l = I \cdot l = 0.00043 \times 5 = 0.002\ m$

(4) Total loss head of irrigation canal under bridge

$h_e = h_{gc} + h_{se} + h_l = 0.002 + 0.003 + 0.002 = 0.007\ m$

1-3-6 No. 2 Transition (B type ~ C type)

Transition length $L = 15.0^{m}$

1-3-6-1 Hydraulic elements of B type open canal

Discharge $Q_1 = 9.63\ m^3/s$
Canal slope $I_1 = 1/5{,}000$.
Flow area $A_1 = 14.69\ m^2$
Velocity $v_1 = 0.659\ m/s$
Velocity head $h_{v_1} = 0.022^{m}$

1-3-6-2 Hydraulic elements of C type open canal

Discharge $Q_2 = 8.65\ m^3/s$
Canal slope $I_2 = 1/5{,}000$
Flow area $A_2 = 12.81\ m^2$
Velocity $V_2 = 0.677\ m/s$
Velocity head $h_{v_2} = 0.023^{m}$

1-3-6-3 Loss head of Transition

(1) Loss head of gradual enlargement

$$h_{ge} = f_g(h_{v_2} - h_{v_1})$$
$$= 1.3 \times (0.023 - 0.022) \doteqdot 0$$

(2) Loss head of friction

$$h_f = \frac{1}{2}(I_1 + I_2)\,l$$
$$= \frac{1}{2}\left(\frac{1}{5{,}000} + \frac{1}{5{,}000}\right) \times 15.0 = 0.003^{m}$$

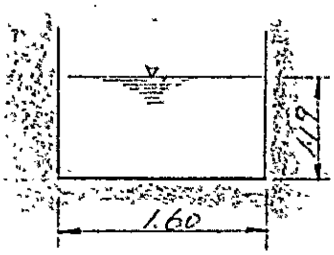
(3) Total loss head of transition

$h_t = 0.043$ m

1-4 Hydraulic Calculation for Main Structure of C-Route.

1-4-1 No.4 Irrigation Aqueduct

1-4-1-1 Typical section of irrigation aqueduct



Discharge $Q = 2.00\ m^3/s$
Canal slope $I = 1/1{,}500$
Coef. of roughness $n = 0.013$
Canal width $B = 1.600\ m$
Water depth $H = 1.190\ m$

Flow area $A_1 = 1.60 \times 1.19 = 1.904$
Wetted perimeter $P_1 = 1.60 + 1.19 \times 2 = 3.980$
Hydraulic radius $R_1 = 1.904 / 3.980 = 0.478$
$R_1^{2/3} = 0.611$
Velocity $v_1 = \frac{1}{0.013} \times 0.611 \times 0.0258 = 1.051\ m/s$
Velocity head $h_{v1} = 0.056$
$Q = 1.904 \times 1.051 = 2.001 > 2.00\ m^3/s$ OK.

1-4-1-2 Hydraulic elements of D type open canal

Discharge $Q = 2.00\ m^3/s$
Canal slope $I_2 = 1/5{,}000$
Flow area $A_2 = 4.26\ m^2$
Velocity $v_2 = 0.470\ m/s$
Velocity head $h_{v2} = 0.011\ m$

1-4-1-3 Loss head of upper transition ; h_{c1}

Transition length $L = 10.00\ m$

(1) Loss head of gradual contraction

$h_{c1} = f_{gc}(h_{v1} - h_{v2})$
$= 0.2(0.056 - 0.011) = 0.009\ m$

(2) Loss head of friction

$h_l = \frac{1}{2} \times (I_1 - I_2) l$

$= 0.5 \times (\frac{1}{5000} + \frac{1}{1500}) \times 10 = 0.004^m$

(3) Total loss head of upper transition

$h_{e1} = h_{el} + h_l = 0.009 + 0.004 = 0.013^m$

1-4-1-4 Loss head of lower transition ; h_{e2}

Transition length $L = 10.00^m$

(1) Loss head of gradual enlargement

$h_{ol} = f_{se} (h_{v1} - h_{v2})$

$= 0.3 (0.056 - 0.011) = 0.014^m$

(2) Loss head of friction

$h_l = \frac{1}{2} (I_1 + I_2) l$

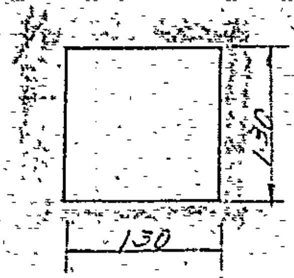
$= 0.5 (\frac{1}{5000} + \frac{1}{1500}) \times 10 = 0.004$

(3) Total loss head of lower transition

$h_{e2} = h_{ol} + h_l = 0.014 + 0.004 = 0.018^m$

1-4-2 No.3 Irrigation Syphon.

1-4-2-1 Typical section of irrigation syphon



Discharge $Q = 2.00 m^3/s$
Coef of roughness $n = 0.020$
Canal width $B = 1.30 m$
Water depth $H = 1.30 m$

Flow area $A_1 = BH = 1.30 \times 1.30 = 1.69 m^2$
Velocity $V_1 = Q_1/A_1 = 2.0/1.69 = 1.183 m/s$
Velocity head $h_{v1} = V_1^2/2g = 0.071$
Wetted perimeter $P_1 = 52$
Hydraulic radius $R_1 = A_1/P_1 = 1.69/5.2 = 0.325$
$R^{2/3} = 0.473$
Hydraulic gradient $I_1 = \left(\frac{0.02 \times 1.183}{0.473}\right)^2 = 0.00250$

1-4-2-2 Hydraulic elements of D type open canal.
Discharge $Q_2 = 2.00 m^3/s$
Canal slope $I_2 = 1/5.000$
Flow area $A_2 = 4.26 m^2$
Velocity $V_2 = 0.474 m/s$
Velocity head $h_{v2} = 0.011 m$

1-4-2-3 Hydraulic elements of irrigation culvert.
Discharge $Q_3 = 2.00 m^3/s$
Canal slope $I_3 = 1/2.000$
Flow area $A_3 = 2.620 m^2$
Velocity $V_3 = 0.766 m/s$
Velocity head $h_{v3} = 0.030 m$

1-4-2-4 Loss head of upper transition ; h_{e1}

Transition length $L = 5.0^m$

(1) Loss head of entrance.

$h_e = f_i(h_{v_2} - h_{v_1})$

$= 0.5(0.071 - 0.011) = 0.030^m$

(2) Loss head of friction

$h_f = \frac{1}{2}(I_1 + I_2)l$

$= \frac{1}{2}(0.00026 + 0.0025) \times 5.00 = 0.007^m$

(3) Total loss head of upper transition.

$h_{e1} = h_e + h_f = 0.030 + 0.007 = 0.037^m$

1-4-2-5 Loss head of lower transition ; h_{e2}

Transition length $L = 5^m$

(1) Loss head of exit

$h_o = f_o(h_{v_1} - h_{v_2})$

$= 0.7(0.071 - 0.030) = 0.029^m$

(2) Loss head of friction

$h_f = \frac{1}{2}(I_1 + I_2)l$

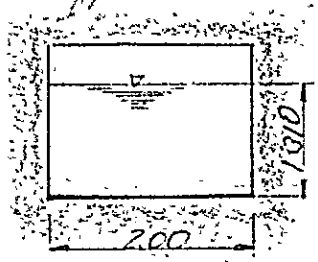
$= 0.5(0.0025 + 0.0005) \times 5.0 = 0.008^m$

(3) Total loss head of lower transition

$h_{e2} = h_o + h_f = 0.029 + 0.008 = 0.037^m$

1-4-3 No.4n No 11 Irrigation Culvert

1-4-3-1 Typical section of irrigation culvert



Discharge $Q = 2.00\ m^3/s$

Canal slope $I = 1/2{,}000$

Coef. of roughness $n = 0.020$

Canal width $B = 2.00\ m$

Water depth $H = 1.310\ m$

Flow area $A_1 = 2.00 \times 1.31 = 2.620\ m^2$

Wetted perimeter $P_1 = 2.00 + 1.31 \times 2 = 4.620$

Hydraulic radius $R_1 = 2.62 / 4.62 = 0.567$

$R_1^{2/3} = 0.685$

Velocity : $v_1 = \frac{1}{0.020} \times 0.685 \times 0.02236 = 0.766$

Velocity head $h_{v_1} = 0.030$

$Q_2 = 2.62 \times 0.765 = 2.007 > 2.00\ m^3/s$ OK

1-4-3-2 Hydraulic elements of D Type open canal.

Discharge $Q_2 = 2.00\ m^3/s$

Canal slope $I_2 = 1/5{,}000$

Flow area $A_2 = 4.26\ m^2$

Velocity $V_2 = 0.474\ m/s$

Velocity head $h_{v_2} = 0.011\ m$

1-4-3-3 Loss head of upper transition ; h_{e1}

Transition length $L = 10.0\ m$

(1) Loss head of gradual contraction

$h_{cd} = f_{gc}(h_{v_1} - h_{v_2})$

$= 0.2 \times (0.030 - 0.011) = 0.004\ m$

(2) Loss head of friction

$h_{el} = \frac{1}{2}(I_1 - I_2) \cdot l$

$= \frac{1}{2}(\frac{1}{5{,}000} - \frac{1}{2{,}000}) \times 10 = 0^m.004$

(3) Total loss head of upper transition

$h_{e1} = h_{ed} + h_{el} = 0.004 + 0.004 = 0^m.008$

1-4-3-4 Loss head of lower transition ; h_{e2}

Transition length $L = 10^m.00$

(1) Loss head of gradual enlargement

$h_{ex} = f_{ge}(h_{v1} - h_{v2})$

$= 0.3(0.030 - 0.011) = 0^m.006$

(2) Loss head of friction

$h_{el} = \frac{1}{2} \times (\frac{1}{5{,}000} - \frac{1}{2{,}000}) \times 10.0 = 0^m.004$

(3) Total loss head of lower transition

$h_{e2} = h_{ex} + h_{el} = 0.006 + 0.004 = 0^m.010$

1-5 Hydraulic calculation
for Elevation of Water Surface and Canal Bottom

Hydraulic Calculation for A Route

Station (Item / Dimension)	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El m	Velocity head (hv) m	Elevation of water surface El m	Hydraulic depth m	Elevation of canal bottom El m	Remarks
(251) RD0 + 218							336.709		336.610		334.660	
	9.630	30.00	Type A open Canal	1.393	1/1000	0.030		0.099		1.950		
+ 317							336.679		336.580		334.630	
	9.630	20.00	No.1 Transition			0.035						
+ 383							336.644		336.622		335.322	
	9.630	444.50	Type B open Canal	0.659	1/5000	0.089		0.022		1.300		
RD1 + 856							336.555		336.533		335.233	
	9.630	4.25	No.1 Bridge			0.047						
+ 870							336.508		336.486		335.186	
	9.630	368.75	Type B open Canal	0.659	1/5000	0.074		0.022		1.300		
RD4 + 144							336.434		336.412		335.112	
	9.630	18.00	Upper transition of No.1 Aqueduct			0.028						
+ 203							336.406		336.286		335.026	
	9.630	41.00	Typical section of No.1 Aqueduct	1.531	1/1500	0.027		0.120		1.260		
+ 338							336.379		336.259		334.999	
	9.630	18.00	Lower transition of No.1 Aqueduct			0.037						
+ 397							336.342		336.320		335.020	
	9.630	2211.00	Type B open Canal	0.659	1/5000	0.442		0.022		1.300		
RD11 + 1000							335.900		335.878		334.578	
	9.630	18.00	Upper transition of No.2 Aqueduct			0.028						
RD12 + 53							335.872		335.752		334.492	
	9.630	31.00	Typical Section of No.2 Aqueduct	1.531	1/1500	0.021		0.120		1.260		
+ 155							335.851		335.731		334.471	

Notes

Hydraulic Calculation for A-Route

Item / Station	Discharge (m^3/sec)	Distance (m)	Name of Canal	Velocity (m/sec)	Slope of Canal	Energy loss (m)	Elevation of energy line (El-m)	Velocity head (hv) (m)	Elevation of water surface (El-m)	Hydraulic depth (m)	Elevation of canal bottom (El-m)	Remarks
RD12 + 155							335.851		335.731		334.471	
	9.630	18.00	Lower transition of No.2 Aqueduct			0.037						
+ 214							335.814		335.792		334.492	
	9.630	470.00	Type B open Canal	0.659	1/5000	0.094		0.022		1.300		No.1 Outlet (RD12+775) Q=0.027 cum/sec
RD13 + 788							335.720		335.675		334.375	
	9.630	5.00	No.2 Bridge			0.029						
+ 804							335.691		335.667		334.369	No.2 Outlet (RD15+300) Q=0.179 cum/sec
	9.630	1,269.00	Type B open Canal	0.659	1/5000	0.354				1.300		No.3 Outlet (RD17+50) Q=1.361 cum/sec
RD19 + 443							335.337		335.315		334.015	
	9.630	8.00	Upper transition of No.2 Syphon			0.028						
+ 469							335.309					
	9.630	(24.00) 21.00	No.2 Syphon			0.038						
+ 535							335.271					
	9.630	8.00	Lower transition of No.2 Syphon			0.038						
+ 564							335.233		335.211		333.911	
	9.630	133.00	Type B open Canal		1/5000	0.027		0.022		1.300		
+ 1001							335.206		335.184		333.884	
	9.630	5.1	No.3 Bridge			0.279						
+ 1013							334.927		334.905		333.605	
	9.630	360.50	Type B open Canal	0.659	1/5000	0.172		0.022		1.300		
RD22 + 750							334.755		334.733		333.433	
	9.630	5.0	No.4 Bridge			0.029						
+ 766							334.726		334.704		333.404	
	9.630	715.10	Type B open Canal	0.659	1/5000	0.143		0.022		1.300		No.4 Outlet (RD24+710). Q=0.418 cum/sec
RD24 + 914							334.583		334.561		333.261	
	9.630	15.00	No.2 Transition			0.003						

Notes

Hydraulic Calculation for A-Route

/ Sheet of 3

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Station \ Item / Dimension	Discharge (m³/sec)	Distance (m)	Name of Canal	Velocity (m/sec)	Slope of Canal	Energy lo. (m)	Elevation of energy line (El-m)	Velocity head (hv) (m)	Elevation of water surface (El-m)	Hydraulic depth (m)	Elevation of canal bottom (El-m)	Remarks
RD2A+963							334.580		334.557		332.667	
			Type C open Canal	0.677	1/5000			0.023		1.890		

Notes

Hydraulic Calculation for 13-Raude

Item / Dimension / Station	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El m	Velocity head (hv) m	Elevation of water surface El m	Hydraulic depth m	Elevation of canal bottom El m	Remarks
RD 24 + 963							334.580		334.557		332.667	
	8.650	1,444.55	Type C open canal	0.677	1/5,000	1.289		0.023		1.890		No. 5 Outlet (RD25+725) Q = 0.017 cum/sec No. 6 Outlet (RD26+560) Q = 0.034 No. 7 Outlet (RD27+850) Q = 0.004 No. 8 Outlet (RD28) Q = 0.257
RD 28 + 678							334.291		334.268		332.378	
		5.00	No. 5 Brigde			0.028						
+ 694							334.263		334.240		332.350	
	8.150	1,881.00	Type C open canal	0.677	1/5,000	0.376		0.023		1.890		No. 9 Outlet (RD 32+150) Q = 0.157 cum/sec No. 10 Outlet (RD 32+350) Q = 0.012 No. 11 Outlet (RD 34+150) Q = 0.016
RD 34 + 850							333.887		333.864		331.974	
		5.00	No. 6 Brigde			0.028						
+ 866							333.859		333.836		331.946	
	8.150	399.60	Type C open canal	0.677	1/5,000	0.080		0.023		1.890		
RD 35 + 850							333.779		333.756		331.866	
		5.00	No. 7 Brigde			0.028						
+ 866							333.751		333.728		331.838	
	8.650	688.00	Type C open canal	0.677	1/5,000	0.138		0.023		1.890		No. 12 Outlet (RD 36+400) Q = 0.016 cum/sec
RD 38 + 474							333.613		333.590		331.700	
		18.00	Upper transition of No. 3 Aqueduct			0.027						
+ 533							333.586		333.467		331.577	
	8.650	31.00	Typical section of No. 3 Aqueduct	1.526	1/2,500	0.021		0.119		1.890		
+ 635							333.565		333.446		331.556	
		18.00	Lower transition of No. 3 Aqueduct			0.037						
+ 694							333.528		333.505		331.615	
	8.650	605.50	Type C open canal	0.677	1/5,000	0.121		0.023		1.890		
RD 40 + 600							333.407		333.384		331.494	
		5.00	No. 8 Brigde			0.028						
+ 616							333.379		333.356		331.466	

Notes

Hydraulic Calculation for *B-Route*

Item / Dimension / Station	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El-m	Velocity head (hv) m	Elevation of water surface El m	Hydraulic depth m	Elevation of canal bottom El-m	Remarks
RD 40+616							333.379		333.356		331.466	
	8.65	1,155.35	Type C open Canal	0.677	1/5,000	0.231		0.023		1.890		No.13 Outlet (RD 43+300) Q = 0.004 cum/sec
RD 44+400							333.148		333.125		331.235	No.14 Outlet (RD 44+400) Q = 7.585 cum/sec
	8.65	397.35	Type C open Canal	0.677	1/5,000	0.079		0.023		1.890		
RD 45+569							333.069		333.046		331.156	
		10.00	Upper transition of No.1 Culvert			0.008						
+602							333.061		333.018		331.138	
		30.00	Typical section of No.1 Culvert		1/2,000	0.015		0.043		1.880		
+700							333.046		333.003		331.123	
		10.00	Lower transition of No.1 Culvert			0.010						
+733							333.036		333.013		331.123	
		426.25	Type C open Canal	0.677	1/5,000	0.085		0.023		1.890		
RD 47+365							332.951		332.928		331.038	
		10.00	Upper transition of No.2 Culvert			0.008						
+398							332.943		332.900		331.020	
		80.00	Typical section of No.2 Culvert		1/2,000	0.040		0.043		1.880		
+660							332.903		332.860		330.980	
		10.00	Lower transition of No.2 Culvert			0.010						
+693							332.893		332.870		330.980	
		55.60	Type C open Canal		1/5,000	0.011		0.023		1.890		
+874							332.882		332.859		330.969	
		10.00	Upper transition of No.3 Culvert			0.008						
+907							332.874		332.851		330.971	
		26.00	Typical section of No.3 Culvert		1/2,000	0.013		0.023		1.880		
+992							332.861		332.838		330.958	

Notes

Hydraulic Calculation for B-Route

Station \ Item / Dimension	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El-m	Velocity head(hv) m	Elevation of water surface El-m	Hydraulic depth m	Elevation of canal bottom El-m	Remarks
RD47+992							332.861		332.838		330.958	
		10.00	Lower transition of No3 Culvert			0.010						
RD48+25							332.851		332.828		330.938	
	8.65	345.25	Type C open Canal	0.318	1/5,000	0.069		0.023 (0.013)		1.890 (0.860)		
RD49+41							332.782 (331.742)		332.759 (331.729)		330.869	
		5.00	Upper transition of No2 Syphon			0.036						
+57							331.706					
	2.00	50.00	No2 Syphon			0.138						
+221							331.568					
		5.00	Lower transition of No2 Syphon			0.049						
+237							331.519		331.508		330.278	
	2.00	1805.80	Type D open canal	0.474	1/5,000	0.361		0.011		1.230		
RD54+772							331.158		331.147		329.917	
		5.00	No.9 Bridge			0.007						
+788							331.151		331.140		329.910	
		1108.73	Type D open canal	0.474		0.222		0.011		1.230		
RD.58+588							330.929		330.918		329.688	

Notes

Hydraulic Calculation for C - Route

Item / Dimension / Station	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El-m	Velocity head (hv) m	Elevation of water surface El m	Hydraulic depth m	Elevation of canal bottom El-m	Remarks
NO 6 + 33.91							330.929		330.918		329.688	
	2.00	159.41	Type D open Canal		1/5,000	0.032		0.011		1.23		
NO 3 + 3.50							330.897		330.886		329.656	
		10.00	Upper transition of No4 Culvert			0.008						
NO 3 + 13.50							330.889		330.859		329.549	
	2.00	28.00	Typical section of No.4 Culvert		1/2,000	0.014		0.030		1.31		
NO 4 + 1.50							330.875		330.845		329.535	
		10.00	Lower transition of No.4 Culvert			0.010						
+11.50							330.865		330.854		329.624	
	2.00	855.78	Type D open Canal		1/5,000	0.171		0.011		1.23		
NO 25 + 3.50							330.694		330.683		329.453	
		10.00	Upper transition of No5 Culvert			0.009						
+13.50							330.696		330.656		329.346	
	2.00	20.00	Typical section of No.5 Culvert		1/2,000	0.010		0.030		1.31		
+33.50							330.676		330.646		329.336	
		10.00	Lower transition of No 5 Culvert			0.010						
NO 26 + 3.50							330.666		330.655		329.425	
	2.00	1119.68	Type D open Canal		1/5,000	0.224		0.011		1.23		
NO 53 + 25.00							330.442		330.431		329.201	
		10.00	Upper transition of No.6 Culvert			0.008						
+35.00							330.434		330.404		329.094	
	2.00	15.00	Typical section of No.6 Culvert		1/2,000	0.008		0.030		1.31		
NO 54 + 10.00							330.426		330.396		329.086	
		10.00	Lower transition of No.6 Culvert			0.010						
+20.00							330.416		330.405		329.175	
								0.011		1.23		

Notes

Hydraulic Calculation for C-Route

Station / Item / Dimension	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El-m	Velocity head (hv) m	Elevation of water surface El m	Hydraulic depth m	Elevation of canal bottom El-m		Remarks
NO.54+20.00							330.416		330.405		329.175		
		1346.36	Type D open Canal		1/5,000	0.269		0.011		1.23			
NO.88+35.00							330.147		330.136		328.906		
		10.00	Upper transition of No.4 Aqueduct			0.013							
NO.89+5.00							330.134		330.078		328.888		
		40.00	Typical section of No.4 Aqueduct		1/5,000	0.027		0.056		1.19			
+45.00							330.107		330.051		328.861		
		10.00	Lower transition of No.4 Aqueduct			0.018							
+55.00							330.089		330.078		328.848		
		171.96	Type D open Canal		1/5,000	0.034		0.011		1.23			
NO.93+30.00							330.055		330.044		328.814		
		10.00	Upper transition of No.7 Culvert			0.008							
NO.94							330.047		330.017		328.707		
		652.06	Typical section of No.7 Culvert		1/2,000	0.326		0.030		1.31			
NO.110							329.721		329.691		328.381		
		10.00	Lower transition of No.7 Culvert			0.010							
+10.00							329.711		329.700		328.470		
		125.00	Type D open Canal		1/5,000	0.025		0.011		1.23			No 15 Culvert (No 113) Q =
NO.113+15.00							329.686		329.675		328.445		
		10.00	Upper transition of No.8 Culvert			0.008							
+25.00							329.678		329.648		328.338		
		30.00	Typical section of No.8 Culvert		1/2,000	0.015		0.030		1.31			
NO.114+15.00							329.663		329.633		328.323		
		10.00	Lower transition of No.8 Culvert			0.010							
+25.00							329.653		329.642		328.412		
								0.011		1.23			

Notes

Hydraulic Calculation for C-Route

Item / Dimension / Station	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El-m	Velocity head (hv) m	Elevation of water surface El-m	Hydraulic depth m	Elevation of canal bottom El-m	Remarks
NO 114+26.00							329.653		329.642		328.412	
		230.00	Type D open Canal		1/5,000	0.046		0.011		1.23		
NO 120+15.00							329.607		329.596		328.366	
		5.00	Upper transition of No.3 Syphon			0.037						
+20.00							329.570					
		30.00	No.3 Syphon	0.025×30×11		0.083						
NO 121+10.00							329.487					
		5.00	Lower Transition of No.3 Syphon	0.007		0.037						
+15.00							329.450		329.420		328.110	
		310.26	Typical section of No 9 Culvert		1/2,000	0.155		0.030		1.31		
EC 21							329.295		329.265		327.955	
		10.00	Lower transition of No.9 Culvert			0.010						
NO.129+22.78							329.285		329.274		328.044	
		157.22	Type D open Canal		1/5,000	0.031		0.011		1.23		
NO.133+20.00							329.254		329.243		328.013	
		10.00	Upper transition of No.10 Culvert			0.008						
+30.00							329.246		329.216		327.906	
		20.00	Typical section of No.10 Culvert		1/2,000	0.010		0.030		1.31		
NO.134+10.00							329.236		329.206		327.896	
		10.00	Lower transition of No.10 Culvert			0.010						
+20.00							329.226		329.215		327.985	
		170.00	Type D open Canal		1/5,000	0.034		0.011		1.23		
NO.138+30.00							329.192		329.181		327.951	
		10.00	Upper transition of No.11 Culvert			0.008						
NO.139							329.184		329.154		327.944	
								0.030		1.31		

Notes

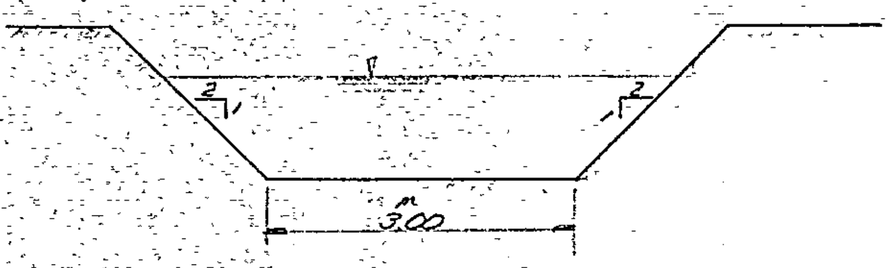
Hydraulic Calculation for C-Route

Item / Dimension / Station	Discharge m³/sec	Distance m	Name of Canal	Velocity m/sec	Slope of Canal	Energy loss m	Elevation of energy line El-m	Velocity head(hv) m	Elevation of water surface El-m	Hydraulic depth m	Elevation of canal bottom El-m	Remarks
NO.139							329.184		329.154		327.844	
		80.00	Typical section of NO.11 Culvert		1/2,000	0.040		0.030		1.31		
NO.141							329.144		329.114		327.804	
		10.00	Lower transition of NO.11 Culvert			0.010						
+10.00							329.134		329.123		327.893	
		210.10	Type D open Canal		1/5,000	0.042		0.011		1.23		
EP							329.092		329.081		327.851	

Notes

1-6 H-Q Curve for Banda - Sanjam Branch

(1) Typical Section



(2) Hydraulic calculation

Canal width $B = 3.00$ m
Water depth H (m)
Canal slope $I = 1/5000$
Cofficient of roughness $n = 0.0225$.
Water area A (m²)
Water perimeter P (m)
Hydraulic radius R (m)
Velocity $V = \frac{1}{n} R^{2/3} I^{1/2}$ (m/sec)
Discharge $Q = A V$ (m³/sec)

H (m)	A (m)	P (m)	R (m)	$R^{2/3}$	$\frac{1}{n} \cdot I^{1/2}$	V m/sec	Q m³/sec
0.20	0.68	3.89	0.175	0.313	0.629	0.197	0.134
0.40	1.52	4.79	0.317	0.465	"	0.292	0.444
0.60	2.52	5.68	0.444	0.582	"	0.366	0.922
0.80	3.68	6.58	0.559	0.679	"	0.427	1.571
1.00	5.00	7.47	0.669	0.765	"	0.481	2.405
1.20	6.48	8.37	0.774	0.843	"	0.530	3.434
1.40	8.12	9.26	0.877	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 Rational formula.

$$Q = 0.2778 \cdot f \cdot r\ A$$

Where Q; Maximum discharge in m^3/sec
f: Coefficient : 0.65
r: Critical intensity of rainfall per hour in the Catchment (mm/hr)
A; Catchment area in Km^2

$$r = \frac{R_{24}}{24}\left(\frac{24}{T}\right)^{2/3}$$

Where R_{24}; Rainfall per day is 8.5" = 215.9 mm/day
T: Critical time of flow for critical point in catchment to the discharge point. (hr)

$$T = L/W$$

Where L; Length for the critical point in Catchment to the discharge point (km)
W; Velocity of maximum flow. km/hr

$$W = 72\,(H/L)^{0.6}$$

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

(a) The maximum flood for A and B route

A	H	L	H/L	W	T	r	0.2778 f	Q
(km²)	(m)	(km)		km/hr	(hr)	(mm/hr)		(m³/sec)
0.061	6.4	0.38	0.01667	6.17	0.062	281.97	0.1806	3.1
0.121	5.0	0.38	0.01310	5.34	0.072	269.86	"	5.9
0.769	16.5	1.65	0.01000	4.54	0.36	146.90	"	20.4
4.047	9.0	3.00	0.00300	2.20	1.36	60.98	"	44.6
15.02	15.2	6.44	0.00236	1.91	3.37	33.31		90.4
120.18	33.5	12.87	0.00260	2.03	6.34	21.85	"	474.2

(b) The maximum flood for C route

A	H	L	H/L	W	T	r	0.2778 f	Q
(km²)	(m)	(km)		km/hr	hr	mm/hr		m³/sec
0.07	0.24	0.8	0.0003	0.55	1.455	58.30	0.1806	0.73
0.23	0.39	1.3	0.0003	0.55	2.364	42.18	"	1.75
0.28	0.42	1.4	0.0003	0.55	2.545	40.15	"	2.03
0.43	0.54	1.8	0.0003	0.55	3.273	33.95	"	2.64

(2) The maximum flood is computed by Dicken's formula.

$$Q = CA^{3/4}$$

where Q; Maximum discharge in ft³/sec
C; Cofficient 1400
A; Catchment area in miles

(a) The maximum flood for A and B route

A_1	A_2	$A_2^{3/4}$	Q_1	Q_2	REMARKS
(km²)	(miles²)		(ft³/sec)	(m³/sec)	1 km² = 0.3862 miles²
0.061	0.023	0.059	82.60	2.3	1 ft³ = 0.02832 m³
0.121	0.047	0.173	242.20	6.9	
0.769	0.297	0.402	562.80	15.9	
4.047	1.563	1.398	1957.20	55.4	
15.02	5.799	3.737	5231.80	148.2	
120.18	46.40	17.778	24889.20	704.9	

(b) The maximum flood for C route

A_1 (Km²)	A_2 (mile²)	$A_2^{3/4}$	Q_1 (ft³/sec)	Q_2 (m³/sec)	Remarks
0.07	0.027	0.0669	93.7	2.7	1 Km² = 0.3862 miles²
0.23	0.089	0.1629	228.1	6.5	1 ft³ = 0.02832 m³
0.28	0.108	0.1884	263.8	7.5	
0.43	0.166	0.2601	364.1	10.3	

Therefore maximum flood is decided as follow

A and B route

A (Km²)	Q (m³/sec)
0.061	3.1
0.121	5.9
0.769	20.4
4.047	55.4
15.02	148.2
120.18	704.8

C route

A (Km²)	Q (m³/sec)
0.07	0.7
0.23	1.8
0.28	2.0
0.43	2.6

CHAPTER 2 STRUCTURAL CALCULATION

2-1 Design Criteria

2-1-1 Allowable Stress of Reinforced Concrete

Allowable stress for compression of concrete	$\sigma_{ca} = 40$ kg/sq cm
Allowable stress for tension of reinforcement	$\sigma_{sa} = 1400$ kg/sq cm
Allowable shearing stress reinforcement	$\tau_{ca} = 4.5$ kg/sq cm
Safety factor	$F_s = 3.5$

2-1-2 Unit Weight of the Materials

Reinforced concrete	2.4 ton/cum
Plain concrete	2.3 ton/cum
Wet masonry	2.5 ton/cum
Saturated soil	2.0 ton/cum
Wet soil	1.8 ton/cum
Water	1.0 ton/cum

2-1-3 Coefficient of Earth Pressure

Using Rankine's formula earth pressure is expressed as; $P = K_A \gamma \cdot H$

Where
- P: earth pressure at given depth (ton/sq.m)
- K_A: coefficient of earth pressure
- γ: unit weight of soil (ton/cum)
- H: given depth (m)

A coefficient of earth pressure, K, is given in the term of internal friction angle, ϕ, as

$$K_A = \frac{1 - \sin\phi}{1 + \sin\phi}$$

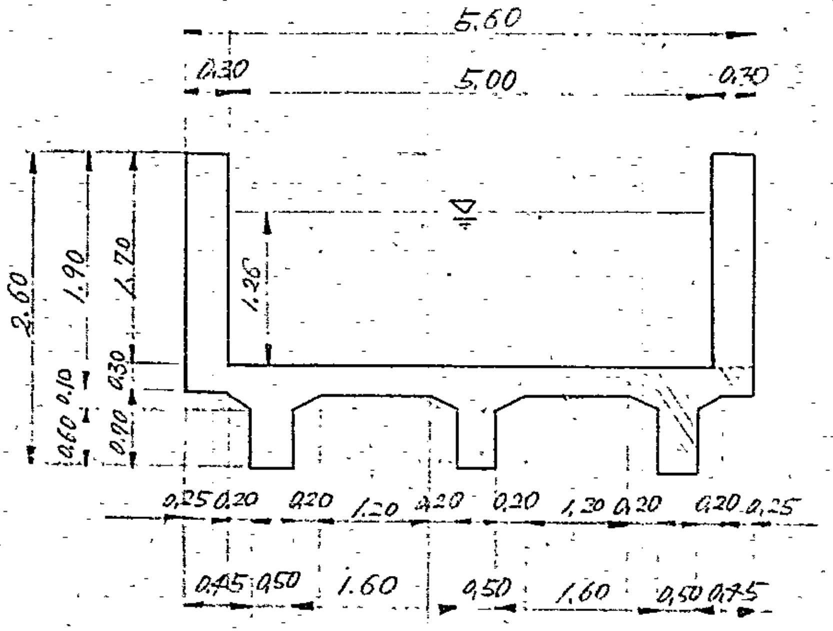
Here, assuming $\phi = 30°$, K_A is then given as follows.

$$K_A = \frac{1 - \sin 30°}{1 - \sin 30°} = \frac{1}{3} = 0.333$$

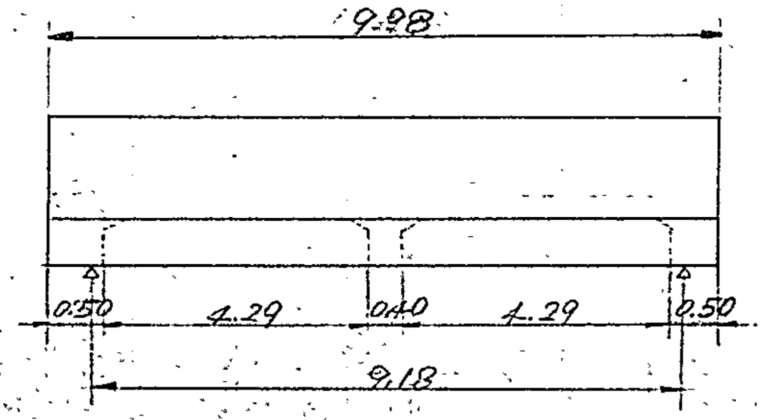
2-2. Irrigation Aqueduct.

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

2-2-1-1. General profile.



(cross section)



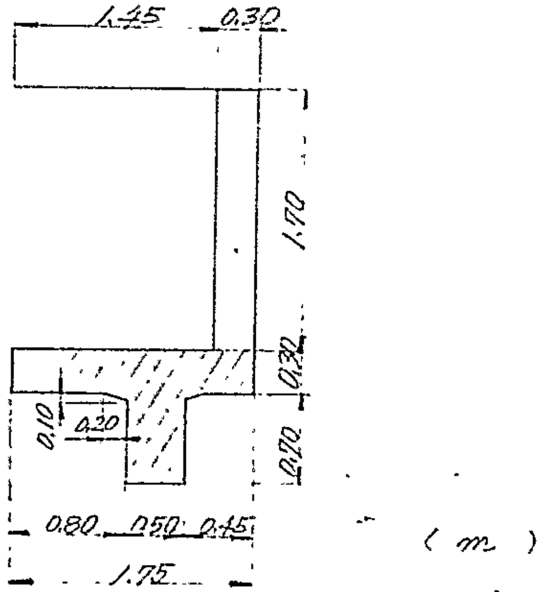
(side view)

Note) Figures are given in meters. (continue)

2-2-1-2 Beam

(1) Load 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) Dead load

Water (g_1).

$g_1 = 1.70 \times 1.45 \times 1.00 = 2.47$ t/m

Side wall (g_2)

$g_2 = 1.70 \times 0.30 \times 2.4 = 1.22$ t/m.

Hunch (q_3)

$$q_3 = 0.10 \times 0.20 \times \frac{1}{2} \times 2.4 \times 2 = 0.05 \ {}^{t}/_{m}$$

Floor (q_4)

$$q_4 = 0.30 \times 1.75 \times 2.4 = 1.26 \ {}^{t}/_{m}$$

Main beam (q_5)

$$q_5 = 0.70 \times 0.50 \times 2.4 = 0.84 \ {}^{t}/_{m}$$

Center girder (P_1)

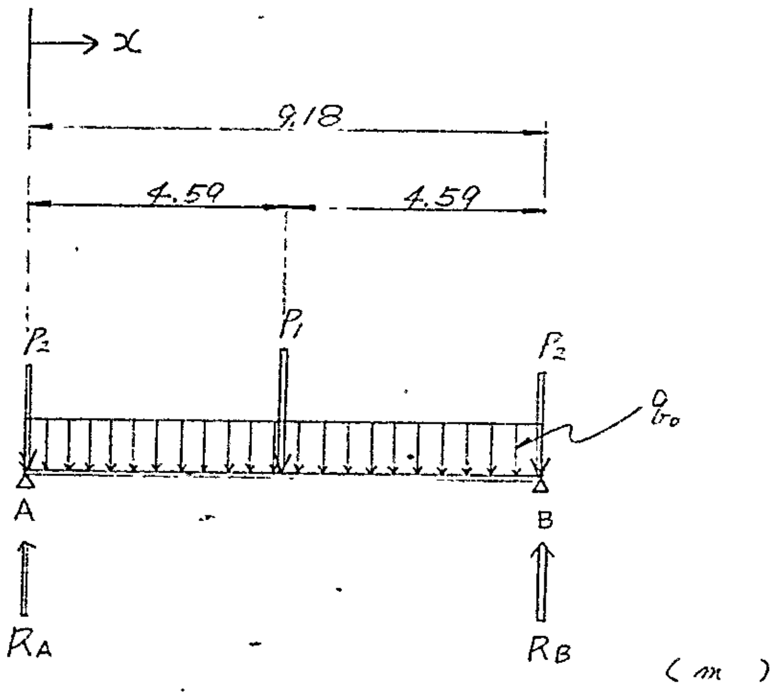
$$P_1 = 0.70 \times 0.40 \times 0.80 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 0.80 \times 2.4 \times 2 = 0.58^{\,t}$$

End girder (P_2)

$$P_2 = 0.70 \times 0.50 \times 0.80 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 0.80 \times 2.4 \times 2 \fallingdotseq 0.71^{\,t}$$

Total load : q_0

$$q_0 = \sum_{1}^{5} q_i$$
$$= (q_1 + q_2 + q_3 + q_4 + q_5)$$
$$= 2.47 + 1.22 + 0.05 + 1.26 + 0.84$$
$$= 5.84 \ t/m$$



R_A R_B : reaction force

$P_1 = 0.58 \ t$
$P_2 = 0.71 \ t$
$q_0 = 5.84 \ t/m$

(2) Reaction: R_A. R_B.

$$R_A = R_B = \left(g_0 \times 9.18 \times 9.18 \times \frac{1}{2} + P_1 \times 4.59 + P_2 \times 9.18\right) \times \frac{1}{9.18}$$

$$= \frac{1}{9.18}\left\{5.84 \times 9.18 \times 9.18 \times \frac{1}{2} + 0.58 \times 4.59 + 0.71 \times 9.18\right\}$$

$$= 27.81^{t}$$

(3) Bending moment and shearing force.

(a) Bending moment;

$0 \leqq x \leqq 4.59$;

$$M_x = R_A \times x - P_2 \times x - g_0 \times x \times x \times \frac{1}{2}$$

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

$$= -\frac{5.84}{2}x^2 + 27.10x \quad (t\text{-}m)$$

$x = 1.00^{m}$;

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

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

$x = 2.00^{m}$;

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

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

$x = 3.00^{m}$;

$$M_{x=3.00} = -\frac{5.84}{2} \times 3.00^2 + 27.10 \times 3.00$$

$$= 55.02^{\ t-m}$$

$x = 4.00^{m}$:

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

$$= 61.68 \ t-m$$

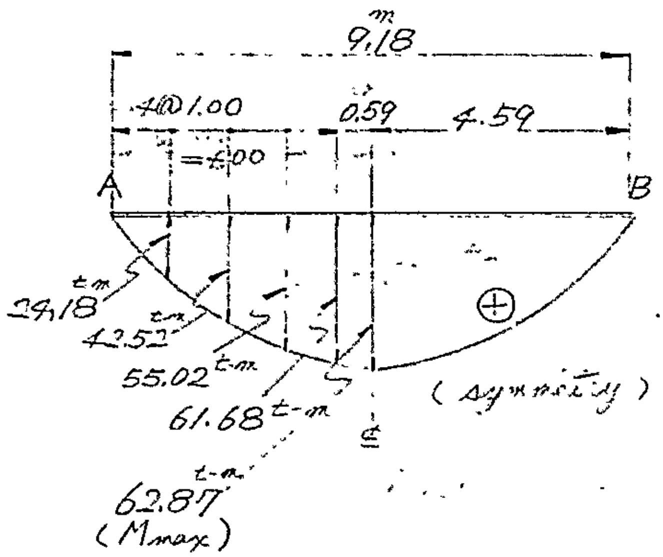
$x = 4.59^{m}$;

$$M_{x=4.59} = M_{max}$$

$$= -\frac{5.84}{2} \times 4.59^2 + 27.10 \times 4.59$$

$$= 62.87^{\ t-m}$$

Diagram of bending moment.



(b) Shearing force

$$S_x = \frac{dM_x}{dx}$$

$$\doteq -5.84x + 27.10 \qquad (0 \leqq x \leqq 4.59)$$

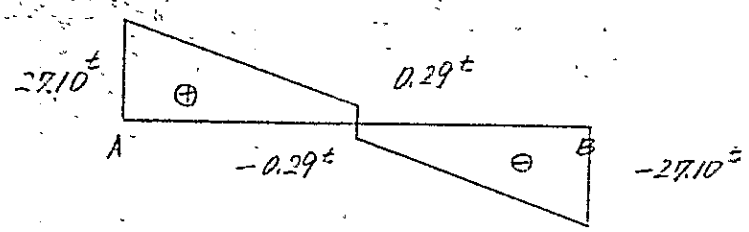
$x = 4.59$;

$$S_{x=4.59} = 0.29^{t}$$

$x = 0.0$

$$S_{x=0.0} = 27.10^{t}$$

Diagram of shearing force.



(4) Amount of reinforcement

(a) Applying T-beam formula.

Required reinforcement amount

$$A_s = \frac{M_{max}}{\sigma_{sa}\ (d - t/2)}$$

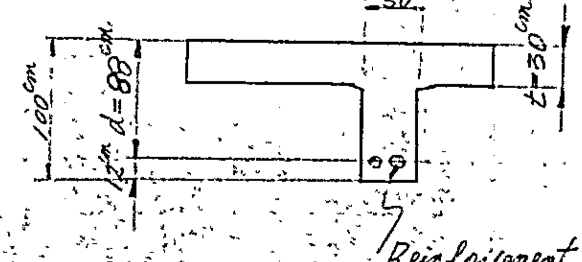
where. $M_{max} = 62.87^{\ t\text{-}m}$
$= 6287.000^{\ kg\text{-}cm}$

$\sigma_{sa} = 1.400\ ^{kg}/_{cm^2}$

$d = 1.00^{m} - 0.06^{m} \times 2$
$= 88^{\ cm}$

$t = 0.30^{m} = 30^{cm}$

(concrete covering thickness : 6^{cm})



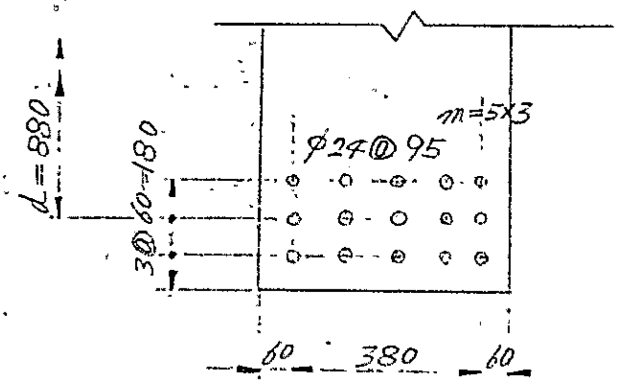
(cross section of T-beam)

$$\therefore A_s = \frac{6{,}287{,}000}{1400\,(88 - 30/2)}$$

$$= 61.52\ cm^2$$

Designed reinforcement,

$\phi 24 @ 95 \quad m = 5 \times 3 \quad (= 67.86\ cm^2)$



(mm)

Check;

$$p = \frac{A_s}{bd}$$

$$= \frac{67.86}{175 \times 88} = 0.0044,$$

$$k = \left[np + \frac{1}{2}(t/d)^2\right] \Big/ \left[np + (t/d)\right]$$

$$= \left[15 \times 0.0044 + \frac{1}{2}(30/88)^2\right] \Big/ \left[15 \times 0.0044 + (30/88)\right]$$

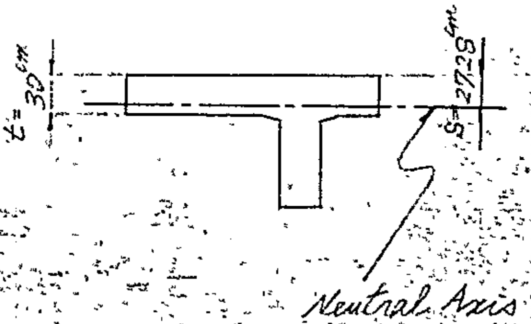
$$= 0.31,$$

therefore

$s = kd$

$= 0.31 \times 88$

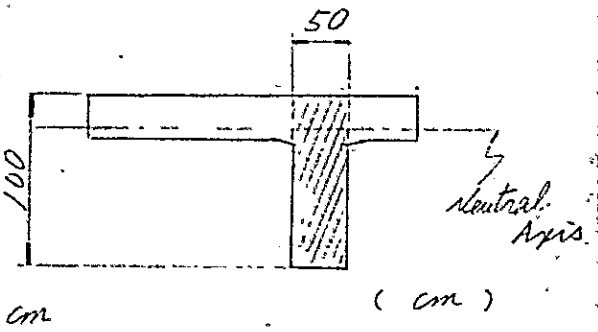
$= 27.28\ cm < 30\ cm = t$



(b) Applying rectangular formula

Required reinforcement amount.

rectangular section. $100^{cm} \times 50^{cm}$



Assuming $d = 88^{cm}$.

$$A_s = \frac{M_{max}}{\sigma_{sa} \times \frac{7}{8} \times d}$$

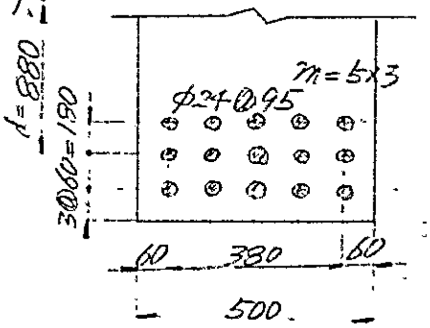
$$= \frac{6.287.000}{1.400 \times 7/8 \times 88}$$

$$= 58.32\ cm^2.$$

Designed reinforcement :

$A_s = 67.86 \text{ cm}^2$;

($\phi 24$ @ 95, $n = 5 \times 3$.)



Check :

$$p = \frac{A_s}{b\,d}$$

$$= \frac{67.86}{50 \times 88}$$

$$= 0.015$$

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

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

$$= 0.48.$$

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

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

$$= 0.84.$$

$$\therefore \sigma_s = \frac{M_{max}}{p \cdot j \cdot b \cdot d^2},$$

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

$$= 1.287 \text{ Kg/cm}^2 < 1.400 \text{ Kg/cm}^2 = \sigma_{sa}$$

(O.K.)

$$\sigma_c = \frac{2 M_{max}}{k j b'' d^2}$$

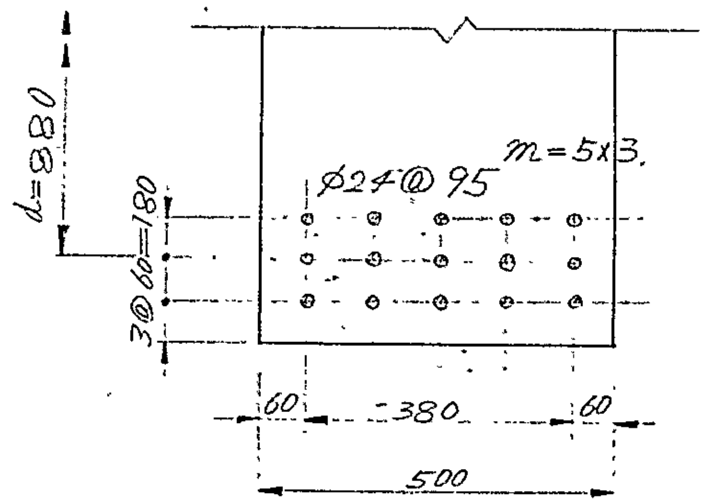
where: $b' = 175^{cm} < 4.5 \times 30 + 20 + 45 = 200^{cm}$

$$\therefore \sigma_c = \frac{2 \times 6.287.000}{0.48 \times 0.84 \times 175 \times 88^2}$$

$$= 23.01 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}.$$

(O.K.)

Distribution of reinforcement



(m.m)

Check;

$$j = 1 - \frac{k}{3}$$
$$= 1 - \frac{0.48}{3}$$
$$= 0.84.$$

$$\tau_c = \frac{S_{max}}{b\,j\,d}$$
$$= \frac{27.100}{50 \times 0.84 \times 88}$$
$$= 7.5 \text{ kg/cm}^2 > 4.5 \text{ kg/cm}^2 = \tau_{ca}$$

where;

S_{max} : shearing force at both sides.
τ_{ca} : allowable shearing stress of concrete.
$\tau_{ca} = 4.5$ kg/cm².

Therefore.
it is necessary to use stirrup:

Required amount of stirrup

$$A_v = \frac{\Delta M_x}{\sigma_{sa} Z},$$

where:
A_v : stirrup amount.
ΔM_x : difference of moment.
σ_{sa} : 1.400 kg/cm²
$Z = j\,d$

$0 \leq x \leq 1.60^{m}$;

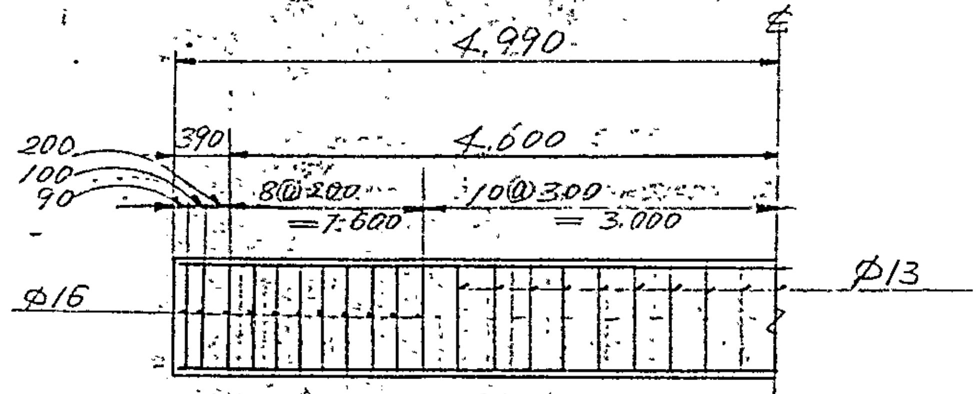
$$\Delta M_x = (M_{x=1.60}) - (M_{x=0.0})$$
$$= 35.88^{\ t\text{-}m}$$
$$= 3.588.000^{\ Kg\text{-}cm}$$

$$A_v = \frac{3.588.000}{1.400 \times 0.84 \times 88.}$$
$$= 34.67^{\ cm^2}$$

$1.60^{m} \leq x \leq 4.59^{m}$

$$\Delta M_x = M_{max} - (M_{x=1.60})$$
$$= 26.99^{\ t\text{-}m}$$
$$= 2.699.000^{\ Kg\text{-}cm.}$$

$$\therefore A_v = \frac{2.699.000}{1.400 \times 0.84 \times 88}$$
$$= 26.08^{\ cm^2}$$



Designed stirrup amount ;

$0 \leq x \leq 1.60^{m}$

$\phi 16 @ 200$. $m = 9$.

$$A_v = 2.011^{cm^2} \times 9 \times 2 = 36.20^{cm^2} > 34.67^{cm^2}$$
(O.K.)

$1.60^{m} \leq x \leq 4.59^{m}$

$\phi 13 @ 300$, $m = 10$.

$$A_v = 1.327^{cm} \times 10 \times 2 = 26.54^{cm^2} > 26.08^{cm^2}$$
(O.K.)

(7.) Arrangement of reinforcement.

At the point of $x = 2.^{m}00$ from the end of the beam.

$$M_{x=2.00} = -\frac{5.84}{2} \times 2.00^2 + 27.10 \times 2.00$$
$$= 42.52^{\ t\text{-}m}$$

∴ required reinforcement amount at this point ;

$$(A_s)_{x=2.00} = \frac{M_{x=2.00}}{M_{max}} \times A_s .$$
$$= \frac{42.52}{62.87} \times 67.86 .$$
$$= 45.89^{\ cm^2}$$

Designed amount is, therefore, reduced for the section $x = 0$ to 2.0^{m} as follows,

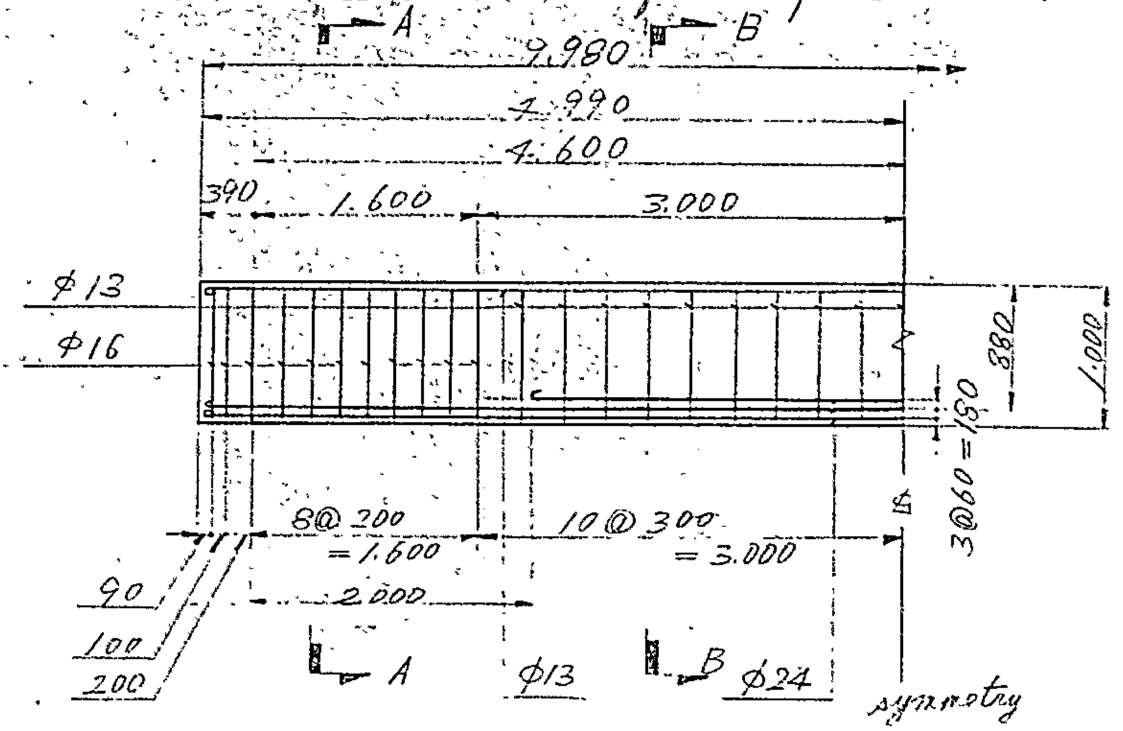
$\phi 24 @ 95$, $n = 5 \times 2$.

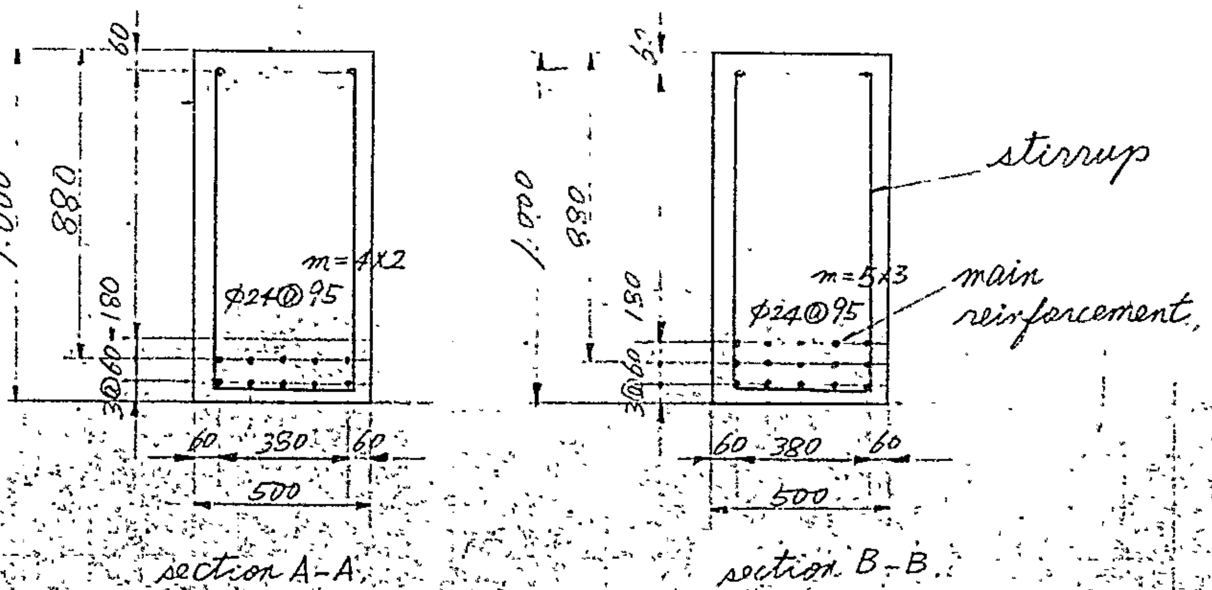
$$A_s = 67.86 \times \frac{2}{3}$$
$$= 45.24^{\ cm^2} \doteqdot 45.89^{\ cm^2} = (A_s)_{x=2.00}$$

Check,

$$p = 0.015 \times \frac{2}{3} = 0.010$$
$$k = \sqrt{(15 \times 0.010)^2 + 2 \times 15 \times 0.010} - 15 \times 0.010 .$$
$$= 0.42$$
$$j = 1 - \frac{k}{3} = 1 - \frac{0.42}{3} = 0.86 .$$
$$\therefore \sigma_s = \frac{M}{p \cdot j \cdot b \cdot d^2}$$
$$= \frac{4,252,000}{0.010 \times 0.86 \times 50 \times 88^2}$$
$$= 1,277 \ ^{kg}/_{cm^2} < 1,400 \ ^{kg}/_{cm^2} = \sigma_{sa} \quad (\text{O.K.})$$

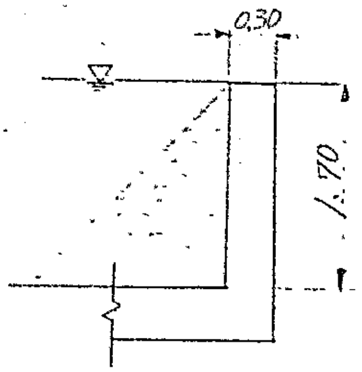
3) Distribution of reinforcement.





section A-A.   section B-B.

2-2-1-3 Side wall.



(1) water pressure q'

$q' = w \cdot h$

$= 1.0 \times 1.70$

$= 1.70 \ t/m^2/m$

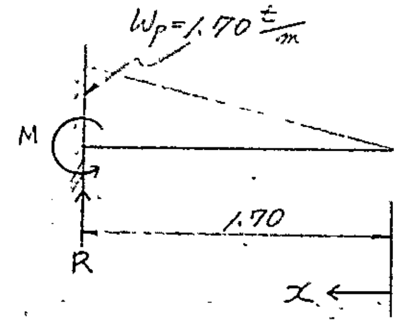
(per unit width of wall)

where; w: unit weight of water $(1.0 \ t/m^3)$

h: water depth (1.70^m)

(2) Bending moment and shearing force.

calculation as cantilever.



$M_{max} = M_{x=1.70} = \left| \dfrac{-q' x^3}{6 l} \right|$

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

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

$S_{max} = R_{x=1.70} = \left| \dfrac{-q' x^2}{2 l} \right|$

$= \dfrac{1.70 \times 1.70^2}{2 \times 1.70}$

$= 1.45^{\ t}$

(3) = Reinforcement amount

rectangle section ; $30^{cm} \times 100^{cm}$

assuming $d = 24^{cm}$

$$A_s = \frac{M_{max}}{\sigma_{sa} \times 7/8\, d}$$

$$= \frac{82000}{1400 \times 7/8 \times 24}$$

$$= 2.79 \ cm^2$$

Designed amount of reinforcement is ;

$\phi 13$ @ 300 $\quad A_s = 4.42^{cm}$

Check.

$$p = \frac{A_s}{bd} = \frac{4.42}{100 \times 24} = 0.0018$$

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

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

$$= 0.21$$

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

$$\therefore \sigma_s = \frac{M}{p \cdot j \cdot b\, d^2}$$

$$= \frac{82000}{0.0018 \times 0.93 \times 100 \times 24^2}$$

$$= 850 \ kg/cm^2 < 1400 \ kg/cm^2 = \sigma_{sa}$$

(O.K)

$$\sigma_c = \frac{2M}{k j b d^2}$$

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

$$= 14.58 \ \text{Kg/cm}^2 < 40 \ \text{Kg/cm}^2 = \sigma_{ca}.$$

(O.K.)

$$\tau_c = \frac{S_{max}}{b j d}$$

$$= \frac{1{,}450}{100 \times 0.93 \times 24}$$

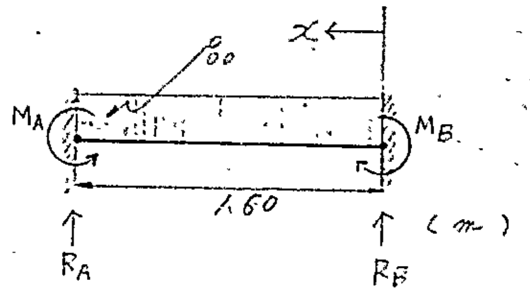
$$= 0.7 \ \text{Kg/cm}^2 < 4.5 \ \text{Kg/cm}^2 = \tau_{ca}$$

(OK)

2-2-1-4. Floor (1)

(1) Load condition.

Water: q_1



$$q_1 = 1.70 \times 1.00 \times 1.0 = 1.70 \ t/m$$

Floor: q_2

$$q_2 = 0.30 \times 1.00 \times 2.4 = 0.72 \ t/m$$

Total load q_0

$$q_0 = q_1 + q_2 = 2.42 \ t/m$$

(2) Reaction and shearing force

$$R_A = R_B = \frac{q_0 \times 1.60}{2} = 1.94 \ t$$

$$S_A = |S_B| = R_A = 1.94 \ t$$

(3) Bending moment and shearing force.

$$M_A = M_B = -\frac{g_0}{12} \times 1.60^2$$
$$= -\frac{2.42}{12} \times 1.60^2$$
$$= 0.52^{t-m}$$

$$M_{max} = M_A = 0.52^{t-m}$$

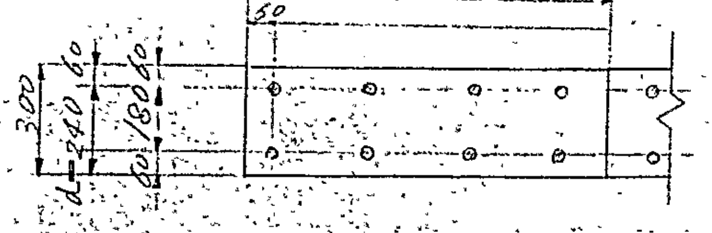
$$S_{max} = S_A = 1.94^{t}$$

(4) ; Reinforcement amount.

rectangle section ; $30^{cm} \times 100^{cm}$,
assuming : $d = 24^{cm}$.

$$A_S = \frac{M_{max}}{\sigma_{sa} \times 7/8 \times d}$$
$$= \frac{52{,}000}{1{,}400 \times 7/8 \times 24}$$
$$= 1.77^{cm^2}$$

Therefore, designed reinforcement :
$\phi 13$ @ 300.
$A_S = 4.42^{cm^2}$.



(5) Check

$$p = \frac{A_s}{b\,d}$$
$$= \frac{4.42}{100 \times 24} \doteqdot 0.0018.$$

$$k = \sqrt{(np)^2 + 2np} - np.$$
$$= \sqrt{(15 \times 0.0018)^2 + 2 \times 15 \times 0.0018} - 15 \times 0.0018.$$
$$= 0.21.$$

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

$$\therefore \sigma_s = \frac{M}{p\,j\,b\,d^2}$$
$$= \frac{52000}{0.0018 \times 0.93 \times 100 \times 24^2}$$
$$\doteqdot 539\ \mathrm{kg/cm^2} < 1400\ \mathrm{kg/cm^2} = \sigma_{sd}$$

(O.K)

$$\sigma_c = \frac{2M}{k\,j\,b\,d^2}$$
$$= \frac{2 \times 52000}{0.21 \times 0.93 \times 100 \times 24^2}$$
$$= 9.25\ \mathrm{kg/cm^2} < 40\ \mathrm{kg/cm^2} = \sigma_{cd}$$

(O.K.)

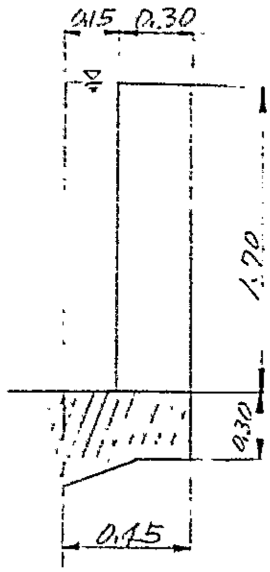
$$S_{max} = 1.94^{t} = 1940\ \mathrm{kg}.$$

$$\therefore \tau_c = \frac{S_{max}}{b\,j\,d}$$
$$= \frac{1940}{100 \times 0.93 \times 24} = 0.87\ \mathrm{kg/cm^2}$$
$$< 4.5\ \mathrm{kg/cm^2} = \tau_{ca}$$

(O.K.)

2-2-1-5, Floor (2)

(1) Load condition

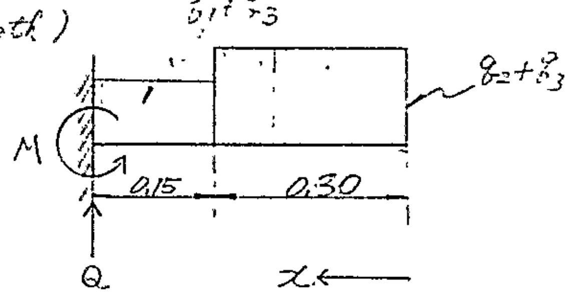


Water

$q_1 = 1.70 \times 1.00 \times 1.0$
$= 1.70$ t/m
(per unit length)

Side wall

$q_2 = 1.70 \times 1.00 \times 2.4$
$= 4.08$ t/m
(per unit length)



Floor

$q_3 = 0.30 \times 1.00 \times 2.4$
$= 0.72$ t/m (per unit length).

(2) Bending moment and shearing force.

calculation as cantilever.

$0.30 \leqq x \leqq 0.45$

$M_x = (q_2 + q_3) \times 0.30 \times (x - \frac{0.30}{2})$
$+ (q_1 + q_3)(x - 0.30)^2 \times \frac{1}{2}$

$$M_{max} = M_{x=0.45}$$
$$= (4.08+0.72)\times 0.30\times\left(0.45-\frac{0.30}{2}\right) + (1.70+0.72)\times(0.45-0.30)^2\times\frac{1}{2}$$
$$= 0.46 \ ^{t-m}$$

$$S_{max} = (q_2+q_3)\times 0.30 + (q_1+q_3)\times 0.15$$
$$= (4.08+0.72)\times 0.30 + (1.70+0.72)\times 0.15$$
$$= 1.80 \ ^{t}.$$

(3) Reinforcement amount

assuming $d = 24^{cm}$

$$A_s = \frac{M_{max}}{\sigma_{sa}\times 7/8\times d}$$
$$= \frac{46000}{1400\times 7/8\times 24}$$
$$= 1.56 \ ^{cm^2}.$$

Therefore, designed amount of reinforcement is;

$\phi 13 @ 300$ ..... $A_s = 4.42^{cm^2}$.

(4) Check

$$p = \frac{A_s}{bd} = \frac{4.42}{100 \times 24} = 0.0018$$

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

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

$$\fallingdotseq 0.21$$

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

$$\sigma_s = \frac{M}{pjbd^2} = \frac{46000}{0.0018 \times 0.93 \times 100 \times 24^2}$$

$$= 477 \ kg/cm^2 < 1.400 \ kg/cm^2 = \sigma_{sa}$$

(O.K.)

$$\sigma_c = \frac{2M}{kjbd^2} = \frac{2 \times 46000}{0.21 \times 0.93 \times 100 \times 24^2}$$

$$= 8.18 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$

(O.K.)

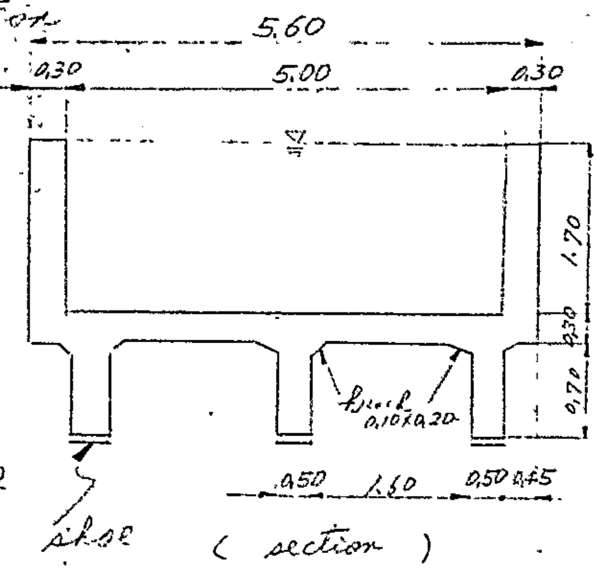
$$\tau_c = \frac{S_{max}}{bjd} = \frac{1800}{100 \times 0.93 \times 24}$$

$$= 0.81 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K.)

2-2-7-6, Shoe,

(1) Load condition



(section)

Water

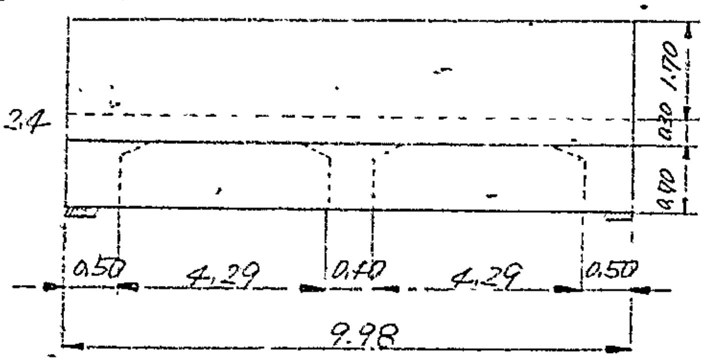
$q_1 = 1.70 \times 5.00 \times 9.98 \times 1.0$
$= 84.83 \ t$

Side wall

$q_2 = 1.70 \times 0.30 \times 9.98 \times 2.4 \times 2$
$= 24.43 \ t$

Floor

$q_3 = 0.30 \times 5.60 \times 9.98 \times 2.4$
$= 40.24 \ t$



(side view)

Beam

$q_4 = 0.70 \times 0.50 \times 9.98 \times 2.4 \times 3$
$= 25.15 \ t$

Girder

$q_5 = 0.70 \times 1.60 \times 0.50 \times 2.4 \times 2 + 0.70 \times 1.60 \times 0.40 \times 2.4$
$= 3.76 \ t$

Hunch

$$q_6 = 0.16 \times 0.20 \times 2.98 \times 2.4 \times 6$$
$$= 2.87^{t}$$

$$q_7 = 0.10 \times 0.20 \times 1.60 \times 2.4 \times 4 \times 2$$
$$= 0.61\ t$$

Total load : q_0

$$q_0 = \sum_1^7 q_i$$
$$= q_1 + q_2 + q_3 + q_4 + q_5 + q_6 + q_7$$
$$= 181.90\ t$$
$$181,900\ ^{kg}$$

(2) Check

Load per one shoe : q_0'

Number of shoes : 6

$$q_0' = 181,900 \times \frac{1}{6} = 30,317\ ^{kg}/shoe$$

allowable stress for compression of cast iron.
(FC-15) $\sigma_{ia} = 800\ ^{kg}/cm^2$.

allowable stress for compression of concrete.
$\sigma_{ca} = 40\ ^{kg}/cm^2$

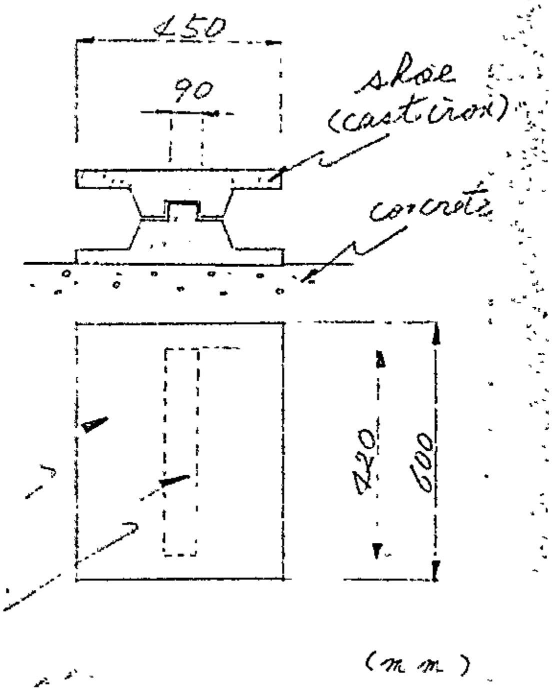
Supporting area of iron and concrete.

Cast iron:

$$A_i = 90 \times 420 \ mm^2$$
$$= 9.0 \times 42.0 \ cm^2$$
$$= 378.0 \ cm^2$$

Concrete:

$$A_c = 450 \times 600 \ mm^2$$
$$= 45.0 \times 60.0 \ cm^2$$
$$= 2{,}700.0 \ cm^2$$



check.

$$\sigma_i = \frac{g_0'}{A_i}$$
$$= \frac{30{,}317}{378.0}$$
$$= 80.20 \ kg/cm^2 < 800 \ kg/cm^2 = \sigma_{ia}.$$

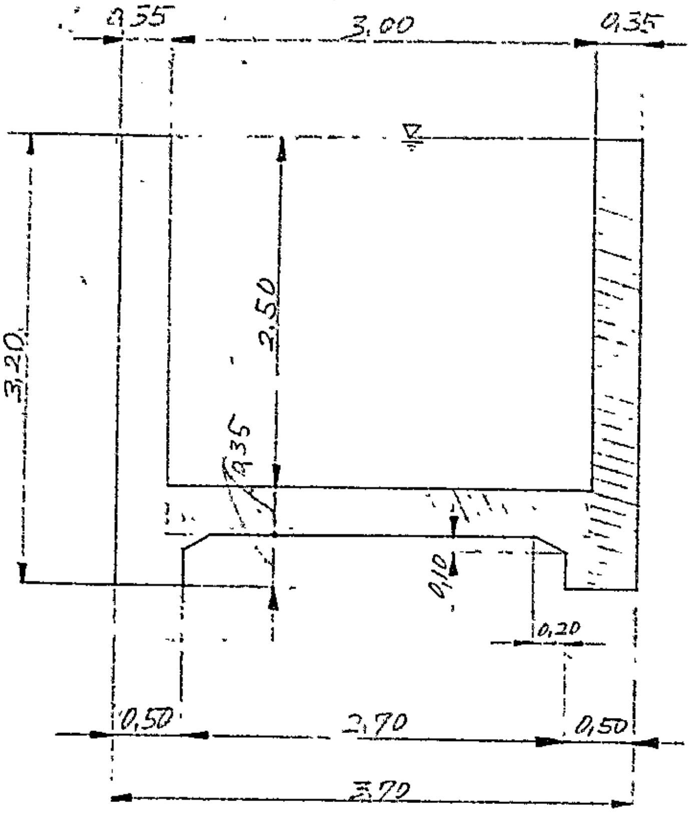
(O.K.)

$$\sigma_c = \frac{g_0'}{A_c}$$
$$= \frac{30{,}317}{2{,}700.0}$$
$$= 11.23 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}.$$

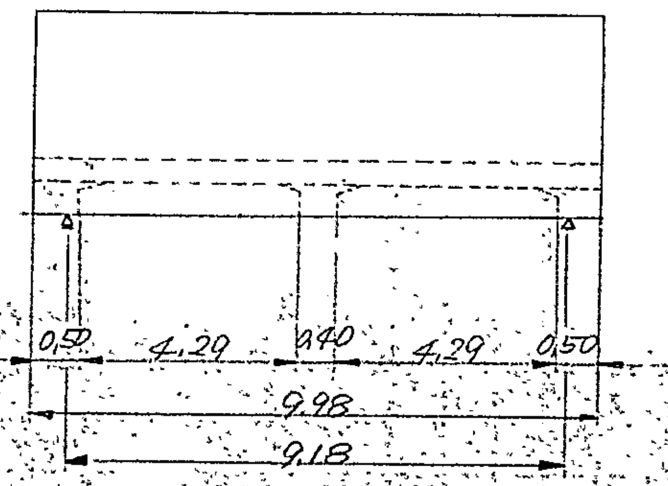
(O.K.)

2-2-2. NO.3. Irrigation Aqueduct.

2-2-2-1. Given dimension

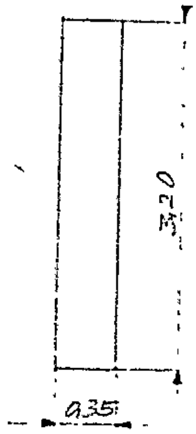


(cross section)



(side view)

2-2-2-2. Beam.



(1) Load condition.

Water

$q_1 = 2.50 \times 3.00 \times 1.0 = 7.50\ t/m$,

Side wall.

$q_2 = (2.50 \times 0.35 \times 2.4 + 0.70 \times 0.50 \times 2.4) \times 2 = 5.88\ t/m$

Hunch.

$q_3 = 0.10 \times 0.20 \times \frac{1}{2} \times 2.4 \times 2 = 0.05\ t/m$

Floor.

$q_4 = 0.35 \times 2.70 \times 2.4 = 2.27\ t/m$.

Center girder.

$P_1 = (0.35 \times 0.40 \times 2.70 \times 2.4$
$+ 0.10 \times 0.20 \times \frac{1}{2} \times 2.70 \times 2.4 \times 2.)$
$= 1.04\ t$

End girder.

$P_2 = (0.35 \times 0.50 \times 2.70 \times 2.4$
$+ 0.10 \times 0.20 \times \frac{1}{2} \times 2.70 \times 2.4 \times 2.)$
$= 1.26\ t$

(2) Total uniform load: q_0.

$q_0 = \Sigma q_i / 2$

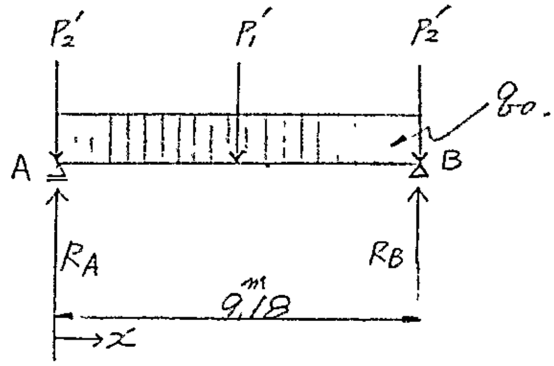
$= 7.85 \ t/m$.

$P_1' = 1.04 \times \frac{1}{2}$

$= 0.52^t$,

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

$= 0.63^t$,



(3) Reaction; R_A, R_B.

$R_A = R_B$

$= \frac{q_0}{2} \times 9.18 + \frac{P_1'}{2} + P_2'$

$= 36.92^t$

(4) Bending moment and shearing force.

(a.) Bending moment.

$0 \leqq x \leqq 4.59^m$

$M_x = R_A x - P_2 x - \frac{q_0 x^2}{2}$

$= -3.93x^2 + 36.29x$

$x = 1.00^m$, $M_{x=1.00} = 32.36^{t\text{-}m}$

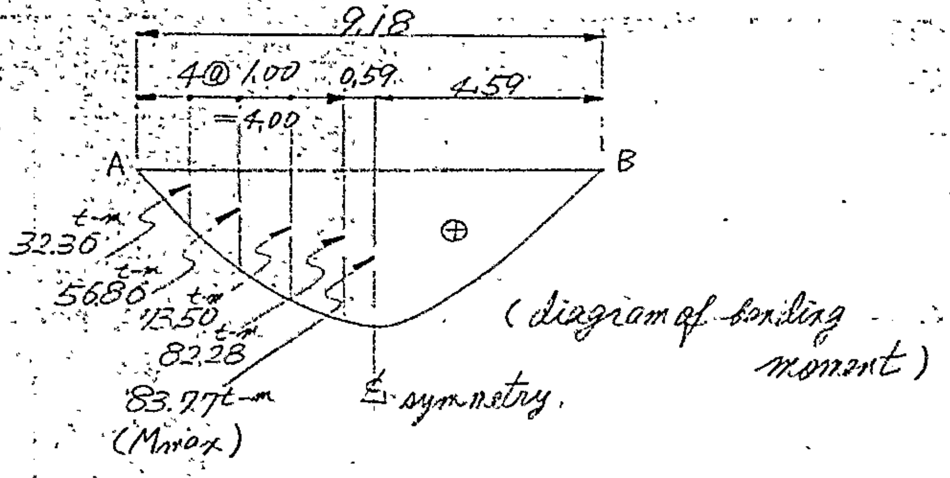
$x = 2.00^m$ $M_{x=2.00} = 56.86^{t\text{-}m}$

$x = 3.00^m$ $M_{x=3.00} = 73.50^{t\text{-}m}$

$x = 4.00^m$ $M_{x=4.00} = 82.28^{t\text{-}m}$.

$x = 4.59^m$ $M_{max} = M_{x=4.59}$

$= 83.77^{t\text{-}m}$.



(6) Shearing force.

$$S_x = \frac{dM_x}{dx} = -7.85x + 36.29$$

$x = 4.59^m$. $S_{x=4.59} = 0.26^t$

$x = 0.0$. $S_{x=0.0} = 36.29^t$

(5) Reinforcement amount.

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

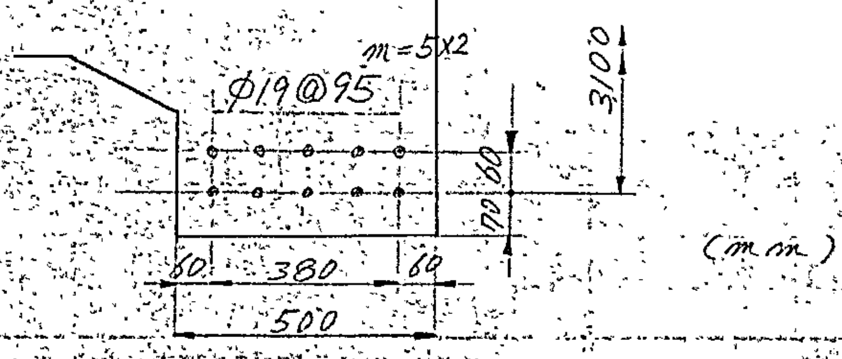
$$A_s = \frac{M_{max}}{\sigma_{sa} \times 7/8 \times d}$$

$$= \frac{8377000}{1400 \times 7/8 \times 310}$$

$$= 22.06^{cm^2}$$

Therefore designed amount:

(φ19 @ 95 m = 5 × 2)($A_s = 28.35^{cm^2}$)



(6.) Check

$$p = \frac{A_s}{b \cdot d} = \frac{28.35}{35 \times 310} = 0.0026.$$

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

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

$$\sigma_s = \frac{M_{max}}{p \cdot j \cdot b \cdot d^2}.$$

$$= \frac{8.377.000}{0.0026 \times 0.92 \times 35 \times 310^2}$$

$$= 1.041 \ Kg/cm^2 < 1.400. \ Kg/cm^2 = \sigma_{sa}$$

(O. K.)

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

$$= \frac{2 \times 8.377.000}{0.24 \times 0.92 \times 35 \times 310^2}$$

$$= 22.56 \ Kg/cm^2 < 40 \ Kg/cm^2 = \sigma_{ca}$$

(O. K.)

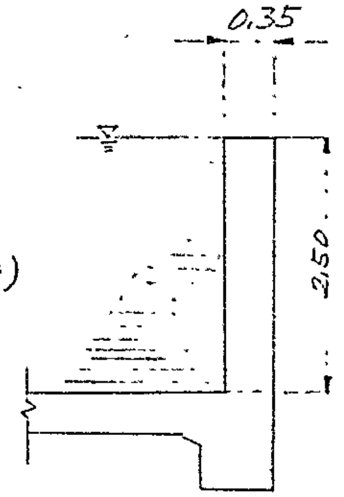
$$\tau_c = \frac{S_{max}}{b \cdot j \cdot d}$$

$$= \frac{36290}{35 \times 0.92 \times 310}$$

$$= 3.6 \ Kg/cm^2 = 4.5 \ Kg/cm^2 = \tau_{ca}$$

(O. K.)

2-2-2-3 Side wall.



(1) water pressure : q'

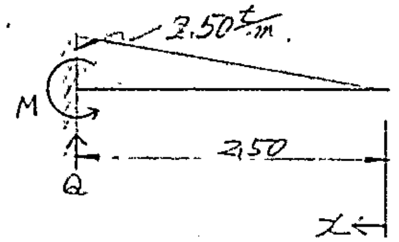
$q' = w \cdot h$

$= 1.0 \times 2.50$

$= 2.50$ t/m²/m (per unit width)

(2) Bending moment and shearing force.

calculation as cantilever.



$$M_{max} = \left| -\frac{q' \times 2.50^2}{6} \right|$$

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

$$= 2.60 \ \text{t-m}$$

$$S_{max} = \left| -\frac{q'}{2} \times 2.50 \right|$$

$$= \frac{2.50}{2} \times 2.50$$

$$= 3.13 \ \text{t}$$

(3) Reinforcement amount

Assuming $d = 29^{cm}$

$$A_s = \frac{26{,}000}{1400 \times 7/8 \times 29} = 7.32^{cm^2}$$

designed amount :

$\phi 13 @ 150 \quad A_s = 8.85^{cm^2}$

(4) Check

$$p = \frac{8.85}{100 \times 29} = 0.0031$$

$$k = \sqrt{(15 \times 0.0031)^2 + 2 \times 15 \times 0.0031} - 15 \times 0.0031$$
$$= 0.26$$

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

$$\sigma_s = \frac{260{,}000}{0.0031 \times 0.91 \times 100 \times 29^2}$$
$$= 1{,}096\ kg/cm^2 < 1{,}400\ kg/cm^2 = \sigma_{sa}$$
(O.K)

$$\sigma_c = \frac{2 \times 260{,}000}{0.26 \times 0.91 \times 100 \times 29^2}$$
$$= 26.13\ kg/cm^2 < 40\ kg/cm^2 = \sigma_{ca}$$
(O.K)

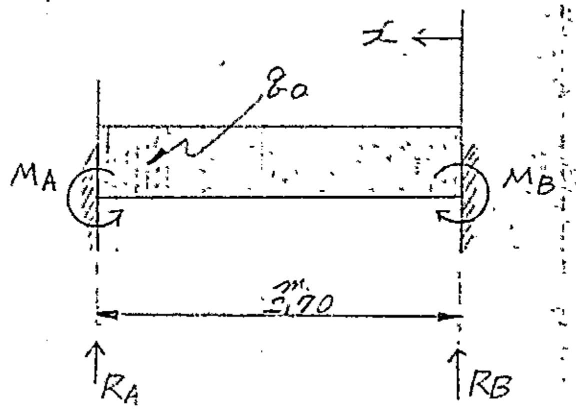
$$\tau_c = \frac{3{,}130}{100 \times 0.91 \times 29}$$
$$= 1.2\ kg/cm^2 < 4.5\ kg/cm^2 = \tau_{ca}$$
(O.K.)

2-2-2-4. Floor

(1) Load condition.

Water

$q_1 = 2.50 \times 1.00 \times 1.0$
$= 2.50\ ^{t}/_{m}$



Floor

$q_2 = 0.35 \times 1.00 \times 2.4$
$= 0.84\ ^{t}/_{m}$.

Total load.

$q_0 = q_1 + q_2$
$= 3.34\ ^{t}/_{m}$.

(2) Bending moment and shearing force.

$$M_A - M_B = -\frac{q_0}{12} \times 2.70^2$$
$$= -\frac{3.34}{12} \times 2.70^2$$
$$= 2.03\ ^{t-m}$$

$$R_A = R_B = \left| \frac{q_0}{2} \times 2.70 \right|$$
$$= \frac{3.34}{2} \times 2.70 = 4.51\ ^{t}$$

$$S_{max} = |R_A| = 4.51\ ^{t}$$

(3) Reinforcement amount

Assuming $d = 29^{cm}$.

$$A_s = \frac{203{,}000}{1{,}400 \times 7/8 \times 29} = 5.71^{cm^2}$$

designed amount:

($\phi 13 @ 150$, $A_s = 8.85^{cm^2}$)

(4) Check

$p \doteq 0.0031$.

$k = 0.26$.

$j = 0.91$.

$$\sigma_s = \frac{203{,}000}{0.0031 \times 0.91 \times 100 \times 29^2}$$

$$= 856 \ kg/cm^2 < 1{,}400 \ kg/cm^2 = \sigma_{sa}$$

(O.K.)

$$\sigma_c = \frac{2 \times 203{,}000}{0.26 \times 0.91 \times 100 \times 29^2}$$

$$= 20.40 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$

(O.K.)

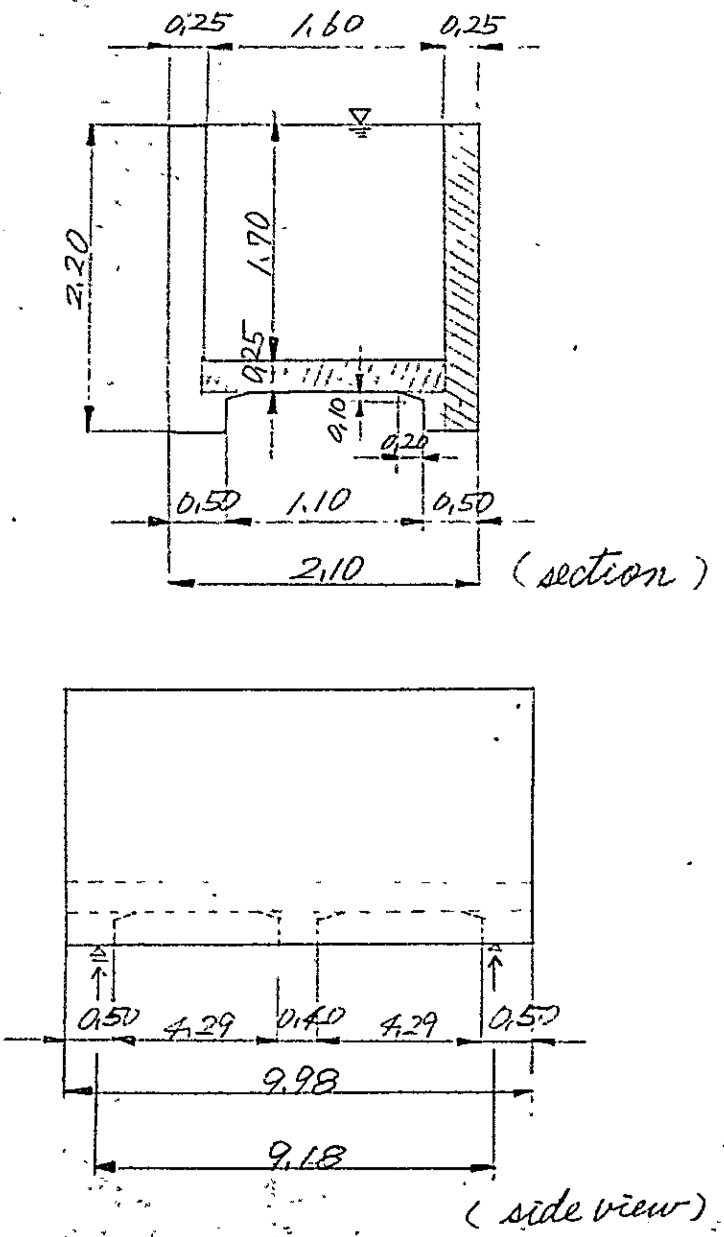
$$\tau_c = \frac{4{,}510}{100 \times 0.91 \times 29}$$

$$= 1.7 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K.)

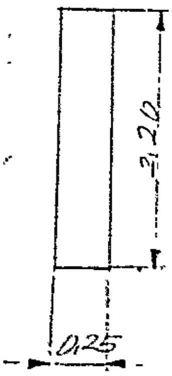
2-2-3. No.4. Irrigation Aqueduct.

2-2-3-1. Given dimensions.



(m)

2-2-3-2 Beam



(1) Load condition.

- water

$g_1 = 1.70 \times 1.60 \times 1.0 = 2.72\ t/m$

Side wall

$g_2 = (1.70 \times 0.25 \times 2.4 + 0.50 \times 0.50 \times 2.4) \times 2 = 3.24\ t/m.$

- Hunch

$g_3 = 0.10 \times 0.20 \times \frac{1}{2} \times 2.4 \times 2 = 0.05\ t/m.$

- Floor

$g_4 = 0.25 \times 1.10 \times 2.4 = 0.66\ t/m.$

girder.

$P_1 = (0.25 \times 0.40 \times 1.10 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 2.4 \times 2) = 0.31\ t.$

$P_2 = (0.25 \times 0.50 \times 1.10 \times 2.4 + 0.10 \times 0.20 \times \frac{1}{2} \times 1.10 \times 2.4 \times 2) = 0.39\ t.$

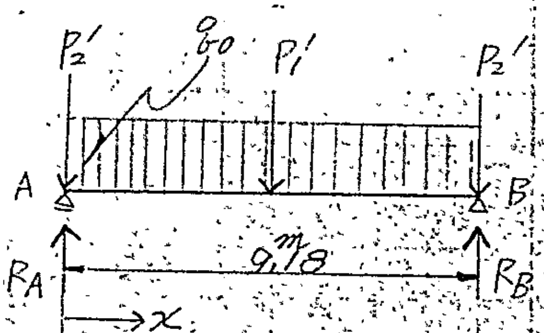
(2) Total load g_0.

$g_0 = \Sigma g_i / 2 = 3.34\ t/m.$

$P_1' = P_1/2 = 0.15\ t,$

$P_2' = P_2/2 = 0.20\ t$

(3) reaction R_A R_B



$R_A = R_B$

$= \frac{g_0}{2} \times 9.18 + \frac{P_1'}{2} + P_2'$

$= 15.61\ t$

(3) Bending moment and shearing force.

$$M_x = R_A x - P_3' x - \frac{q_0 x^2}{2}, \quad (0 \leq x \leq 4.59)$$
$$= -1.67 x^2 + 15.41 x.$$

$$M_{max} = M_{x=4.59}.$$
$$= -1.67 \times 4.59^2 + 15.41 \times 4.59$$
$$= 35.55 \ t\text{-}m.$$

$$S_{max} = \left(\frac{dM_x}{dx}\right)_{x=0.0}$$
$$= 15.41 \ t.$$

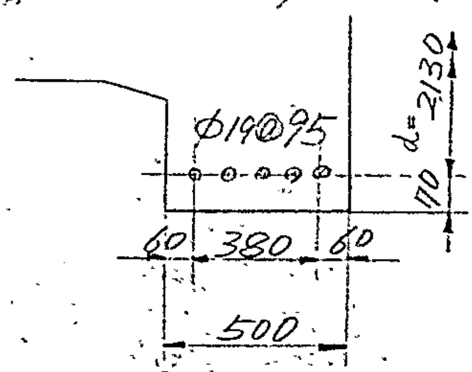
(4) Reinforcement amount

Assuming $d = 213$ cm

$$A_S = \frac{3{,}555{,}000}{1{,}400 \times 7/8 \times 213} = 13.62 \ cm^2.$$

designed amount:

$$(\phi 19 @ 95, \ m = 5.) \quad (A_S = 14.18 \ cm^2)$$



(5) Check.

$$p = \frac{As}{bd} = \frac{14.18}{25 \times 213} = 0.0027$$

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

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

$$\sigma_s = \frac{M_{max}}{pjbd^2}$$

$$= \frac{3.555.000}{0.0027 \times 0.92 \times 25 \times 213^2}$$

$$= 1.262 \ kg/cm^2 < 1.400 \ kg/cm^2 = \sigma_{sa}$$

(O.K)

$$\sigma_c = \frac{2M_{max}}{kjbd^2}$$

$$= \frac{2 \times 3.555.000}{0.24 \times 0.92 \times 25 \times 213^2}$$

$$= 28.39 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$

(O.K.)

$$\tau_c = \frac{S_{max}}{bjd}$$

$$= \frac{15.410}{25 \times 0.92 \times 213}$$

$$= 3.1 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K.)

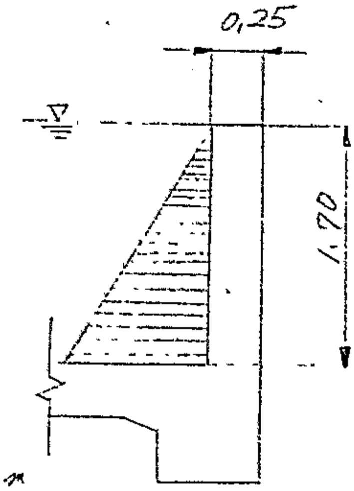
2-2-3-3. Side wall.

(1) Water pressure:

$q' = wh$

$= 1.0 \times 1.70 = 1.70\ t/m/m$

(per unit width)

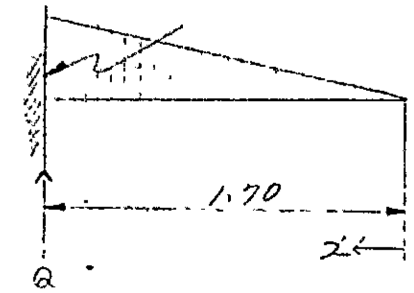


(2) Bending moment and shearing force calculation as cantilever

$M_{max} = \left| -\frac{8'}{6} \times 1.70^2 \right|$

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

$S_{max} = \frac{1.70^2}{2} = 1.45\ t$



(3) Reinforcement amount.

Assuming $d = 19^{cm}$,

$A_S = \frac{82000}{1400 \times 7/8 \times 19}$

$= 3.52^{cm^2}$

designed amount.

($\phi 13$ @ 300 $\quad A_S = 4.42^{cm^2}$)

(4) Check

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

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

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

$$\sigma_s = \frac{M_{max}}{pjbd^2}$$

$$= \frac{82000}{0.0023 \times 0.92 \times 100 \times 19^2}$$

$$= 1.073 \ kg/cm^2 < 1.400 \ kg/cm^2 = \sigma_{sa}$$

(O K.)

$$\sigma_c = \frac{2 M_{max}}{kjbd^2}$$

$$= \frac{2 \times 82000}{0.23 \times 0.92 \times 100 \times 19^2}$$

$$= 21.47 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca} \quad (O.K.)$$

$$\tau_c = \frac{S_{max}}{bjd}$$

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

$$= 1.0 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K.)

2-2-3-4 Floor

(1) Load condition

Water

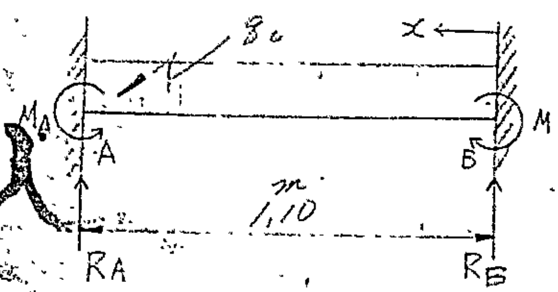
$q_1 = 1.70 \times 1.00 \times 1.0$

$= 1.70$

Floor

$q_2 = 0.25 \times 1.00 \times 2.4$

$= 0.60$ t/m



Total load

$q_0 = q_1 + q_2 = 2.$[illegible] t/m

(2) Bending moment and shearing force.

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

$R_A = R_B = \frac{q}{2} \times 1.10 = 1.27$ t

$S_{max} = |R_A| = 1.27$ t.

(3) Reinforcement amount.

Assuming $d = 19$ cm.

$A_s = \frac{23,000}{1,400 \times 7/8 \times 19} = 0.99$ cm²

designed amount:

($\phi 9$ @ 300 $A_s = 2.12$ cm²)

(4) Check

$p = \frac{2.12}{100 \times 19} = 0.0011$; $k = \sqrt{(np)^2 + 2np} - np = 0.17$, $j = 0.94$

$\sigma_s = \frac{23,000}{0.0011 \times 0.94 \times 100 \times 19^2} = 616$ kg/cm² $< 1,400$ kg/cm² $= \sigma_{sa}$

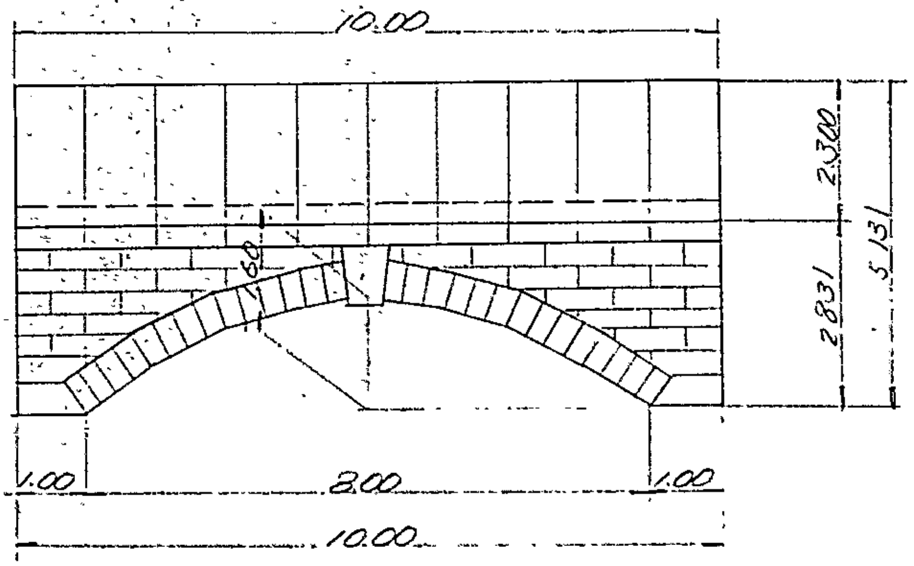
$\sigma_c = \frac{2 \times 23,000}{0.17 \times 0.94 \times 100 \times 19^2} = 7.98$ kg/cm² < 40 kg/cm² $= \sigma_{ca}$

$\tau_c = \frac{1,270}{100 \times 0.94 \times 19} = 0.7$ kg/cm² < 4.5 kg/cm² $= \tau_{ca}$

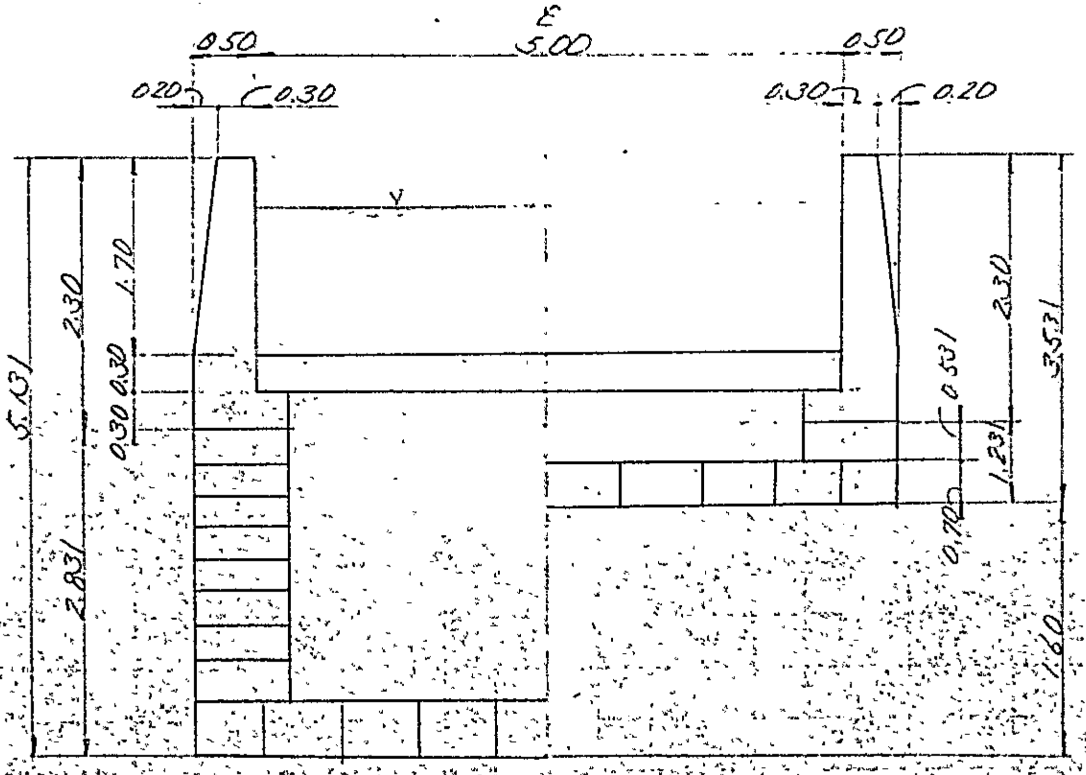
(O.K.)

2-2-4. Arch type Irrigation Aqueduct (No.1)

2-2-4-1. Given dimension

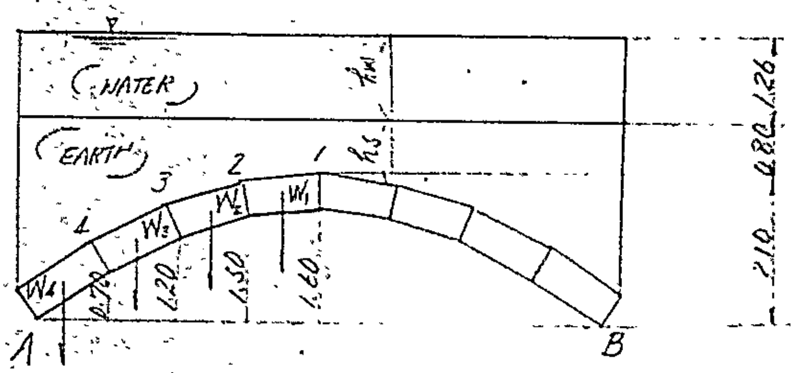


Profile



Cross section (m)

2-2-4-2 Load condition



(1) Unit weight of the materials

Wet masonry $\gamma = 2.5$ t/m³

Saturated soil $\gamma_1 = 2.0$

Water $\gamma_2 = 1.0$

$\gamma_1/\gamma = 2.0/2.5 = 0.800$

$\gamma_2/\gamma = 1.0/2.5 = 0.400$

(2) Conversion height of load

$h_{s1} = 0.80 \times 0.80 = 0.640^{m}$

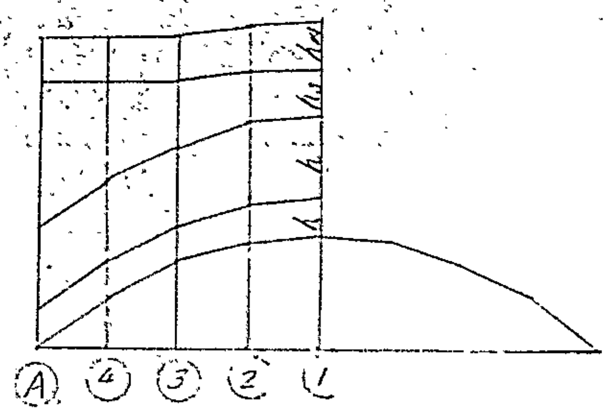
$h_{w1} = 1.26 \times 0.40 = 0.504$

$h_{s2} = 0.90 \times 0.80 = 0.720$

$h_{s3} = 1.20 \times 0.80 = 0.960$

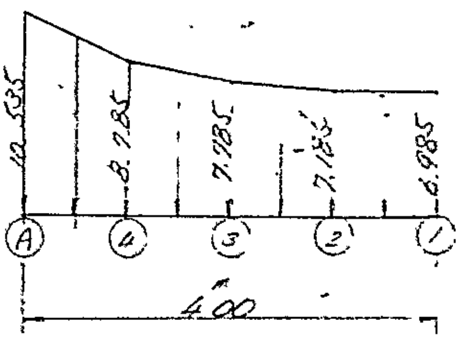
$h_{s4} = 1.70 \times 0.80 = 1.360$

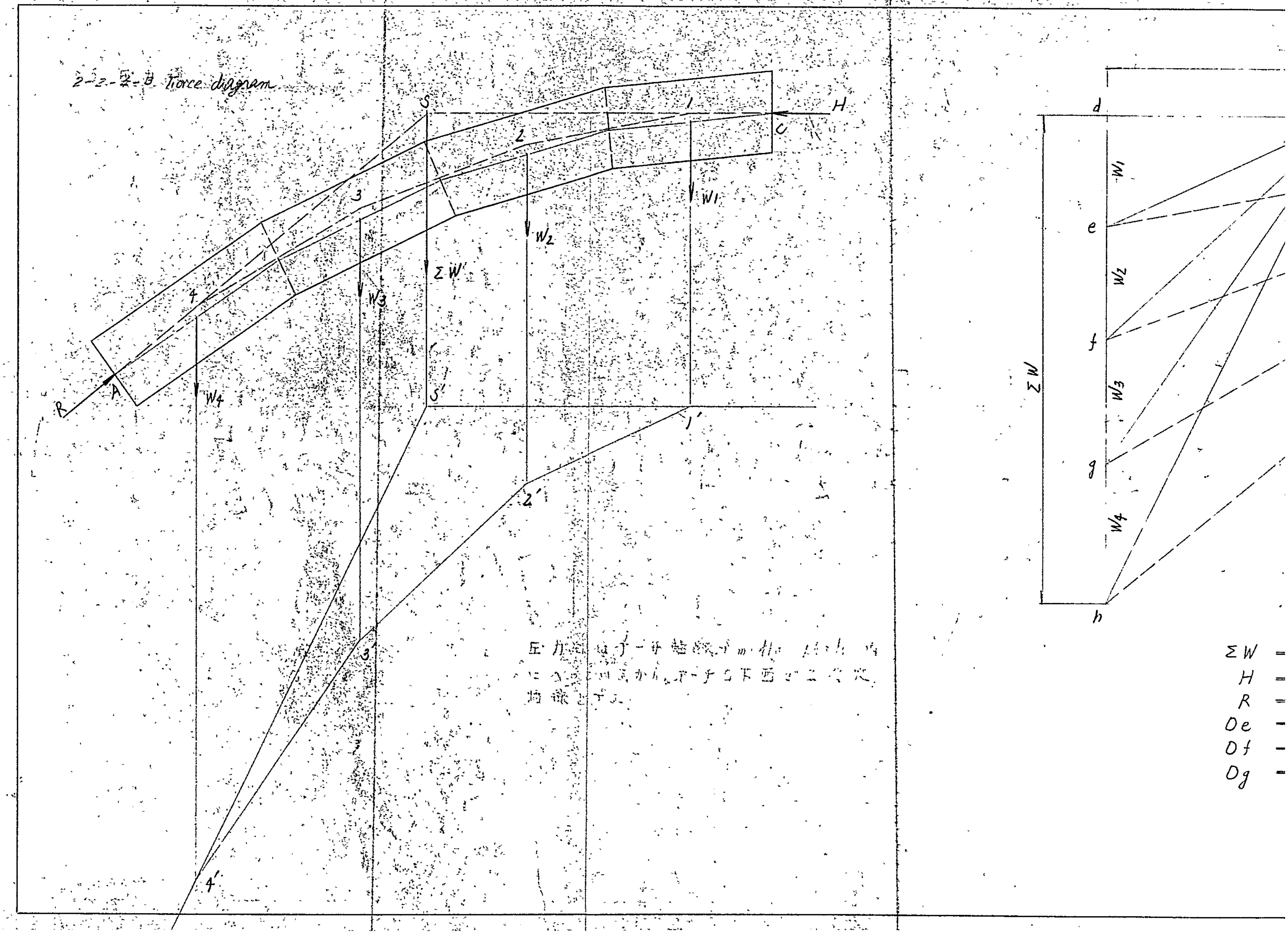
$h' = 0.50 \times 2.30 = 1.150$



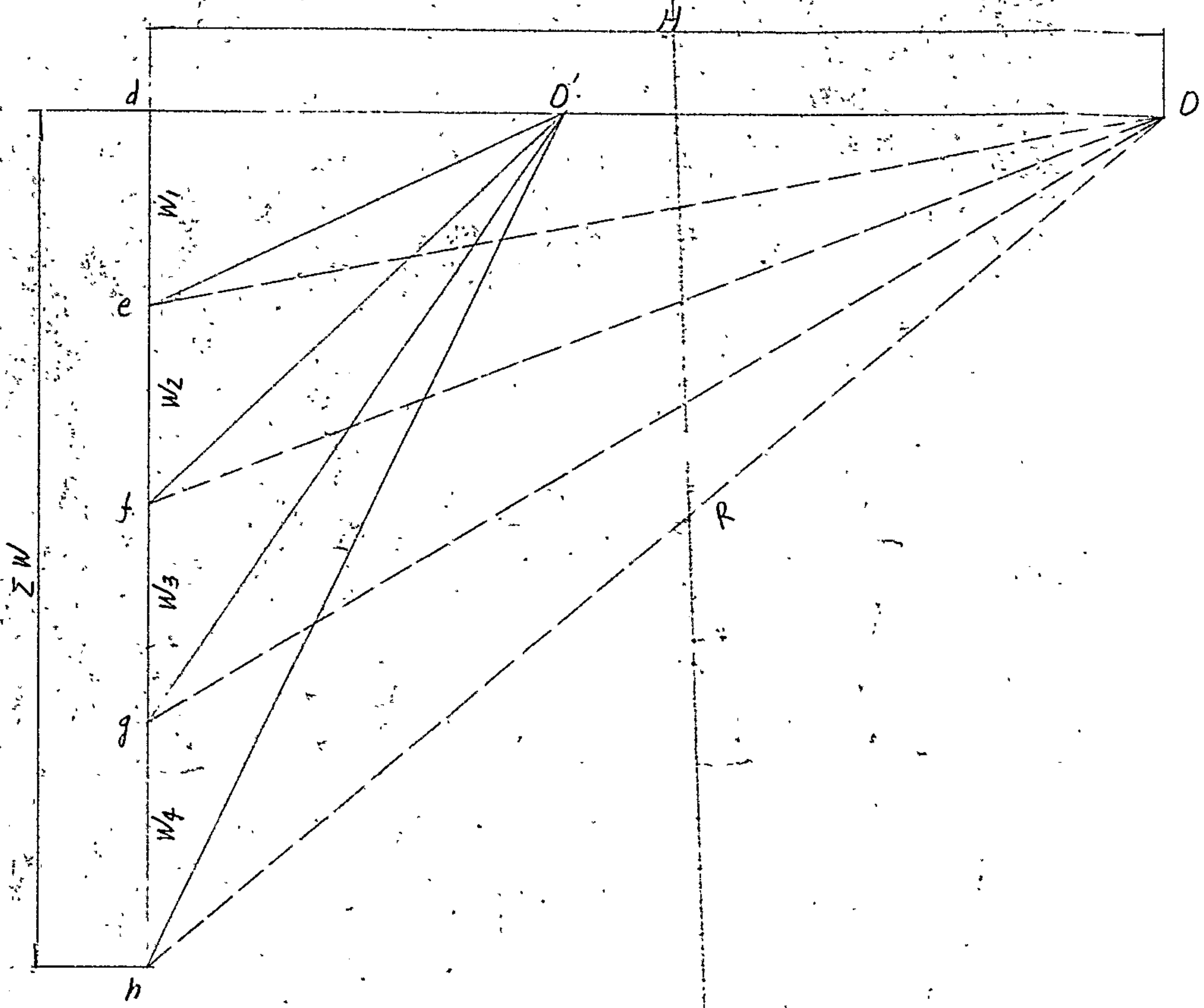
	λ	h'	hs	hw	Σh	W
①	m 0.500	m 1.150	m 0.640	m 0.504	m 2.794	6.985
②	˝	˝	0.720	˝	2.874	7.185
③	˝	˝	0.960	˝	3.114	7.785
④	˝	˝	1.560	˝	3.514	8.785
Ⓐ		˝	2.060	˝	4.214	10.535

(3) Load diagram





2-2-図-B Force diagram
S
1
2
3
4
H
C
A
R
W1
W2
W3
W4
ΣW'
S'
1'
2'
3
4'
d
e
f
g
h
ΣW
ΣW =
H =
R =
Oe -
Of -
Og -



$\Sigma W = 30.7$ t/m
$H = 36.7$ 〃
$R = 47.8$ 〃
$Oe = 37.4$ 〃
$Of = 39.3$ 〃
$Og = 42.8$ 〃

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

(1) $H_0 = 36.7\ t/m$

$\sigma_c = H/100 \cdot b = 36700/100 \times 50 = 7.3\ kg/cm^2 < \sigma_{ca} = 70\ kg/cm^2$

$\because \sigma_{ca} = 250/3.5 = 70\ kg/cm^2$

(2) $P_0 = 37.4\ t/m$, $e = 10\ cm$

$\varphi = 5°30'$

$N_0 = P \cdot \cos\varphi = 37.4 \times \cos 5°30' = 37.228\ t/m$

$H_0 = P \cdot \sin\varphi = 37.4 \times \sin 5°30' = 3.585$

$\sigma_c = \frac{N_0}{100 \cdot b}\left(1 \pm \frac{6e}{b}\right)$

$= \frac{37228}{100 \times 50} \times \left(1 \pm \frac{6 \times 10}{50}\right)$

$= \begin{cases} 18.3\ kg/cm^2 & < \sigma_{ca} = 70\ kg/cm^2 \\ 6.6 & < \text{"} \end{cases}$

$\mu N_0 = 0.6 \times 37228 = 22.337\ t > 3.585\ t$

(3) $P_0 = 39.3\ t/m$ $e = 2.0\ cm$ $\varphi = 1°50'$

$N_0 = 39.3 \times \cos 1°50' = 39.280\ t/m$

$H_0 = 39.3 \times \sin 1°50' = 1.257\ t/m$

$\sigma_c = \frac{39280}{100 \times 50} \times \left(1 \pm \frac{6 \times 2.0}{50}\right)$

$= \begin{cases} 9.7 & < \sigma_{ca} = 70\ kg/cm^2 \\ 6.0 & < \text{"} \end{cases}$

$\mu N_0 = 0.6 \times 39280 = 23.568\ t/m > 1.257\ t/m$

(4) $P_0 = 42.8\ t/m$ $e = 2.0$ $\varphi = 5°10'$

$N_0 = 42.8 \cdot \cos 5°10' = 42.620\ t/m$

$H_0 = 42.8 \times \sin 5°10' = 3.854\ t/m$

$\sigma_c = \frac{42620}{100 \times 50} \times \left(1 \pm \frac{6 \times 2.0}{50}\right)$

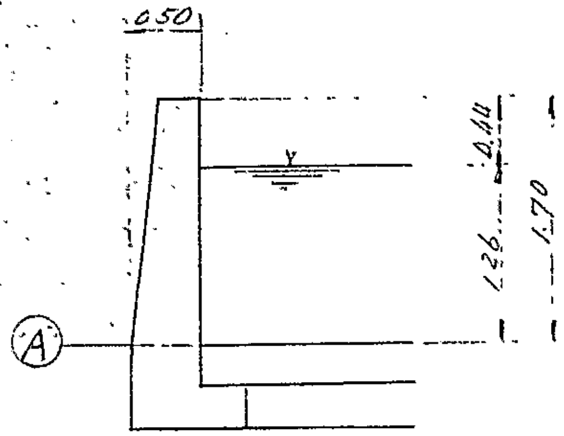
$= \begin{cases} 10.6\ kg/cm^2 & < 70\ kg/cm^2 \\ 6.5 & < \text{"} \end{cases}$

(5) Point A

$R = 47.8^{t}$

$\sigma_c = \frac{47800}{100 \times 50} = 9.6\ kg/cm^2 < 70\ kg/cm^2$

2-2-4-5 Side wall.



Section A~A.

Water pressure

$P_w = \frac{1}{2} \times 1.0 \times 1.260^2 = 0.794\ t/m$

$h = 1/3 \times 1.260 = 0.420^{m}$

Bending moment.

$M = 0.794 \times 0.420 = 0.333^{t-m}$

$I = \frac{1}{12} bh^3 = \frac{1}{12} \times 100 \times 50^3 = 1041667\ cm^4$

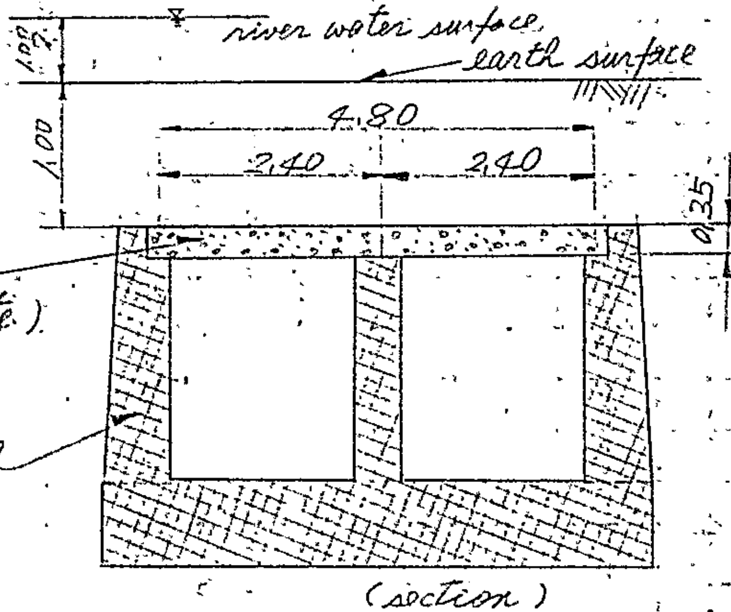
$\sigma_c = \frac{M}{I} \cdot \frac{h}{2} = \frac{33300}{1041667} \times 25 = 0.8\ kg/cm^2 < 70\ kg/cm^2$

2-3. Irrigation Syphon.

2-3-1. NO. 1 Irrigation Syphon.

2-3-1-1. Top Floor

(1) General.



Only the top floor is calculated and examined as a continuous beam.

unit weight
- concrete : 2.4 t/m^3
- earth in water : 2.0 t/m^3
- water : 1.0 t/m^3

beam length : 4.80 m

One span length : 2.40 m

$\sigma_{sa} = 1,400 \ kg/cm^2$

$\sigma_{ca} = 40 \ kg/cm^2$

$\tau_{ca} = 4.5 \ kg/cm^2$

(2) Load condition

Earth

$q_1 = 1.00 \times 1.00 \times 2.0$

$= 2.00 \ t/m$

Water

$q_2 = 1.00 \times 1.00 \times 1.0 = 1.00$ t/m

Top floor

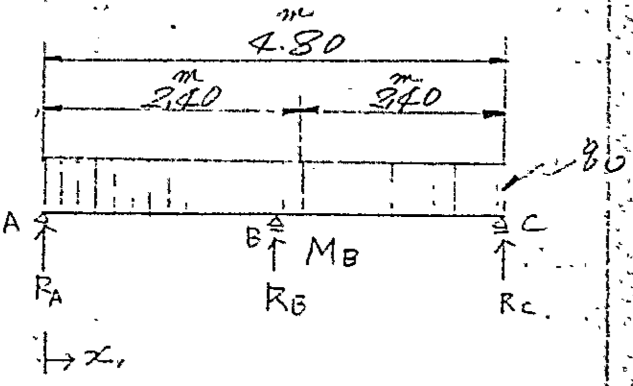
$q_3 = 0.35 \times 1.00 \times 2.4 = 0.84$ t/m

Total load

$q_0 = q_1 + q_2 + q_3$

$= 3.84$ t/m

(3) Reaction force



$R_B = \frac{5}{4} q \times 2.40$

$= \frac{5}{4} \times 3.84 \times 2.40$

$= 11.52$ t

$R_A = R_C = \frac{3}{8} q \times 2.40$

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

$= 3.46$ t

(4) Bending moment:

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

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

$= 2.76$ t-m

$M_x = R_A x - q_0 \times \frac{x^2}{2}$

$= -\frac{3.84}{2} x^2 + 3.46 x$

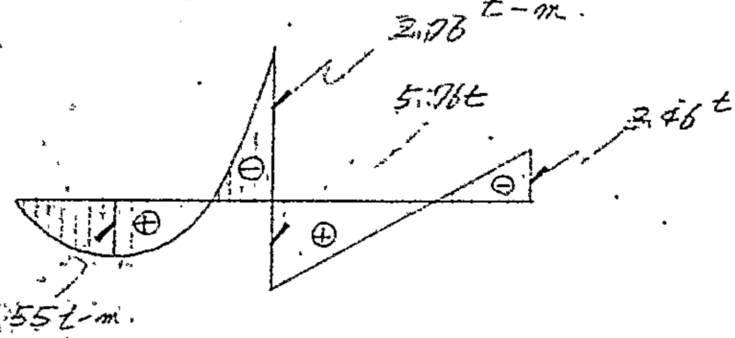
$$\frac{dM_x}{dx} = -3.84x + 3.46$$

$$\frac{dM_x}{dx} = 0 \longrightarrow x = 0.90$$

$$M_{max} = M_{x=0.90}$$
$$= -\frac{3.84}{2} \times 0.90^2 + 3.46 \times 0.90$$
$$= 1.55^{t\text{-}m}$$

(5) shearing force:

$$S_{max} = \left| -3.84 \times 2.40 + 3.46 \right|$$
$$= 5.76^t$$



M-figure. S-figure.

(6) Reinforcement amount.

Assuming $d = 0.29^m = 29^{cm}$.

(a) in the case of $M = 2.76^{t\text{-}m} = 276{,}000^{kg\text{-}cm}$.

$$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d}$$
$$= \frac{276{,}000}{1400 \times 7/8 \times 29}$$
$$= 7.77^{cm^2}$$

designed amount

$\phi 16$ @ 200 ($A_s = 10.06^{cm^2}$)

(b) in the case of $M = 1.55^{t\text{-}m} \fallingdotseq 155.000^{kg\text{-}cm}$

$$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d}$$
$$= \frac{155.000}{1.400 \times 7/8 \times 29}$$
$$= 4.36\ cm^2$$

designed amount:

ϕ 13 @ 200 ($A_s = 6.64^{cm^2}$)

(IV) Check

(a) in the case of $M = 2.96^{t\text{-}m} = 296.000^{kg\text{-}cm}$

$$p = \frac{A_s}{bd} = \frac{10.06}{100 \times 29} = 0.0035$$

$$k = \sqrt{(np)^2 + 2np} - np$$
$$= \sqrt{(15 \times 0.0035)^2 + 2 \times 15 \times 0.0035} - 15 \times 0.0035$$
$$= 0.29$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.29}{3} = 0.90$$

$$\sigma_s = \frac{M}{pjbd^2} = \frac{276.000}{0.0035 \times 0.90 \times 100 \times 29^2}$$
$$= 1.042\ kg/cm^2 < 1.400\ kg/cm^2 = \sigma_{sa}\ (O.K.)$$

$$\sigma_c = \frac{2M}{kjbd^2} = \frac{2 \times 276.000}{0.29 \times 0.90 \times 100 \times 29^2}$$
$$= 25.15\ kg/cm^2 < 40\ kg/cm^2 = \sigma_{ca}\ (O.K.)$$

$$\tau_c = \frac{S_{max}}{bjd} = \frac{5.960}{100 \times 0.90 \times 29}$$
$$= 2.2\ kg/cm^2 < 4.5\ kg/cm^2 = \tau_{ca}$$
(O.K.)

(b) in the case of $M = 1.55^{t\text{-}m} = 155{,}000^{kg\text{-}cm}$

$$p = \frac{A_s}{bd} = \frac{6.64}{100 \times 29} = 0.0023$$

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

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

$$= 0.23$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.23}{3}$$

$$= 0.92$$

$$\therefore \sigma_s = \frac{M}{pjbd^2}$$

$$= \frac{155{,}000}{0.0023 \times 0.92 \times 100 \times 29^2}$$

$$= 871 \ kg/cm^2 < 1.400 \ kg/cm^2 = \sigma_{sa}$$

(O.K)

$$\sigma_c = \frac{2M}{kjbd^2}$$

$$= \frac{2 \times 155000}{0.23 \times 0.92 \times 100 \times 29^2}$$

$$= 17.42 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$

(O.K.)

$$\tau_c = \frac{S_{max}}{bjd}$$

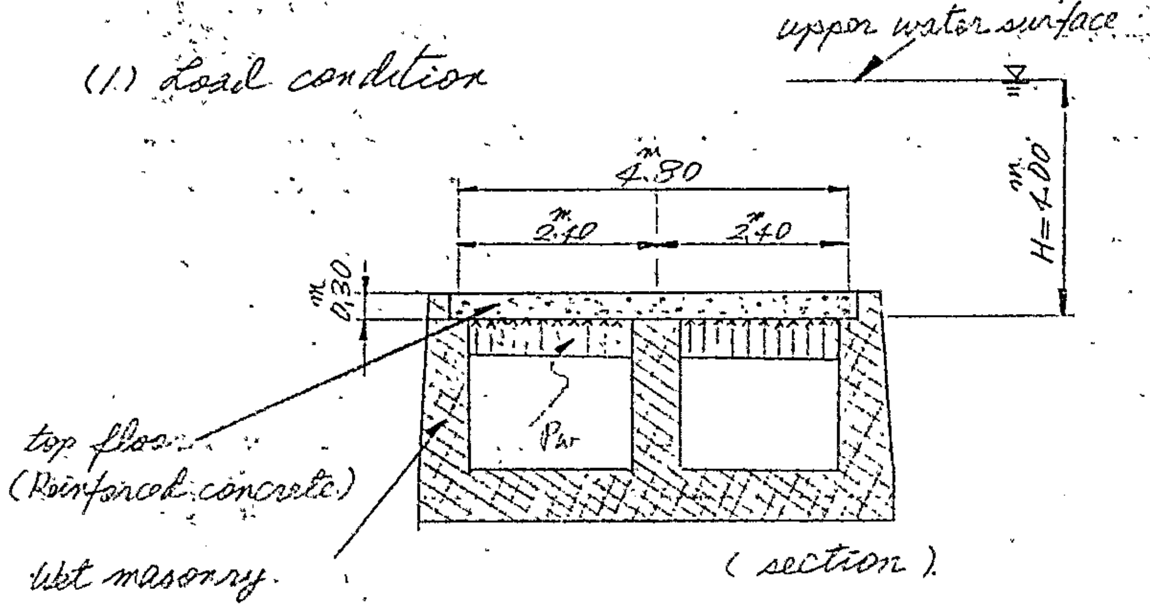
$$= \frac{5260}{100 \times 0.92 \times 29}$$

$$= 2.2 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K)

2-3-1-2. Examination of inside pressure.

(1) Load condition



inside pressure, operating against the top floor from inside to upward, is desided by water head distance $H=4.00^m$;

so that ;
$$P_w = H \cdot w \cdot 1.00 = 4.00 \times 1.0 \times 1.00 = 4.00\ t/m$$

Top floor

$$q = 0.84\ t/m$$

Total load q_0

$$q_0 = P_w - q = 3.16\ t/m$$

in this case, earth pressure over the top floor is neglected;

[illegible]

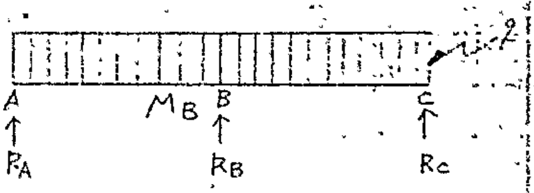
(2) Reaction force

$$R_B = \frac{5}{4} q_0 \times 2.40 = \frac{5}{4} \times 3.16 \times 2.40$$
$$= 9.48\ t$$

$R_A = R_C = \frac{3}{8} q_0 \times 2.40$

$= \frac{3}{8} \times 3.16 \times 2.40$

$= 2.84\ t$



(3) Bending moment,

$M_B = -q_0 \times \frac{2.40^2}{8}$

$= -3.16 \times \frac{2.40^2}{8} = 2.28\ t\text{-}m$

$M_x = R_A x - q_0 \times \frac{x^2}{2}$

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

$\frac{dM_x}{dx} = -3.16x + 2.28$

$\frac{dM_x}{dx} = 0. \longrightarrow x = 0.72.$

$M_{max} = M_{x=0.72}$

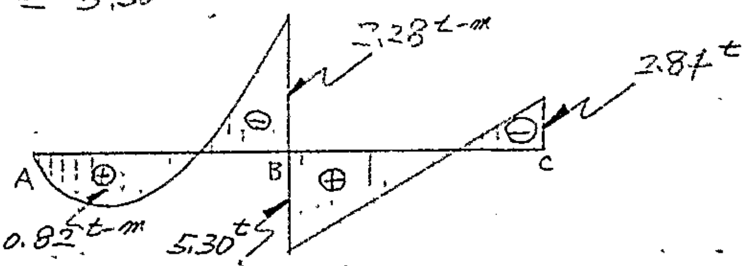
$= -\frac{3.16}{2} \times 0.72^2 + 2.28 \times 0.72,$

$= 0.82\ t\text{-}m.$

(4) Shearing force,

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

$= 5.30^t$



(5) Reinforcement amount.

now, reinforcement amount of the upper part of the top floor,

$\phi 16 @ 200 \quad A_s = 10.06\ cm^2$ (designed amount)

and, that of the lower one,

$\phi 13 @ 200 \quad A_s = 6.64\ cm^2$ (designed amount)

(6) Check

(a) in the case of $M = 2.28^{t\text{-}m} = 228.000^{kg\text{-}cm}$, ($A_s = 6.64^{cm^2}$)

$$p = \frac{A_s}{bd} = \frac{6.64}{100 \times 29} = 0.0023$$

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

$$j = 1 - \frac{k}{3} = 1 - \frac{0.23}{3} = 0.92$$

$$\sigma_s = \frac{M}{pjbd^2} = \frac{228000}{0.0023 \times 0.92 \times 100 \times 29^2}$$

$$= 1.281 \ kg/cm^2 < 1.400 \ kg/cm^2 = \sigma_{sa}$$ (O.K.)

$$\sigma_c = \frac{2M}{kjbd^2} = \frac{2 \times 228000}{0.23 \times 0.92 \times 100 \times 29^2}$$

$$= 25.62 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$ (O.K.)

$$\tau_c = \frac{S_{max}}{bjd} = \frac{5.300}{100 \times 0.92 \times 29}$$

$$= 2.0 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$ (O.K)

(b) in the case of $M = 0.82^{t\text{-}m} = 82.000^{kg\text{-}cm}$, ($A_s = 10.06^{cm^2}$)

$$p = \frac{A_s}{bd} = \frac{10.06}{100 \times 29} = 0.0035$$

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

$$j = 1 - \frac{k}{3} = 1 - \frac{0.29}{3} = 0.90$$

$$\sigma_s = \frac{M}{pjbd^2} = \frac{82000}{0.0035 \times 0.90 \times 100 \times 29^2}$$

$$= 310 \ kg/cm^2 < 1400 \ kg/cm^2 = \sigma_{sa}$$ (O.K)

$$\sigma_c = \frac{2M}{kjbd^2} = \frac{2 \times 82000}{0.29 \times 0.90 \times 100 \times 29^2}$$

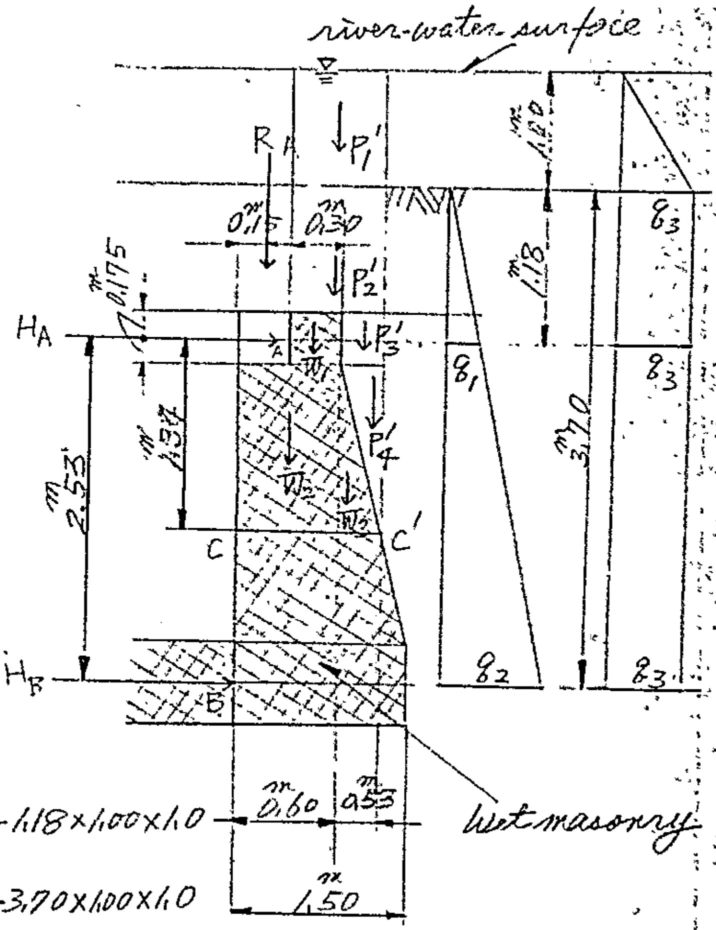
$$= 7.47 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$ (O.K)

$$\tau_c = \frac{S_{max}}{bjd} = \frac{5.300}{100 \times 0.90 \times 29}$$

$$= 2.0 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$ (O.K)

2-3-1-3. Safety calculation of side wall of wet masonry

(1) General



the side wall of wet masonry is calculated and examined as a simple beam;

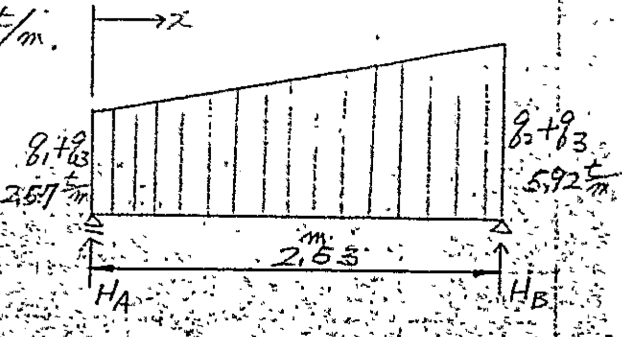
(2) Load condition

$q_1 = 1.18\times1.00\times1.0\times0.33 + 1.18\times1.00\times1.0$
$= 1.57$ t/m

$q_2 = 3.70\times1.00\times1.0\times0.33 + 3.70\times1.00\times1.0$
$= 4.92$ t/m

$q_3 = 1.10\times1.00\times1.0 = 1.00$ t/m.

(3) Reaction force



$$H_A = \frac{1}{2.53}\left\{2.57\times\frac{2.53^2}{2} + (5.92-2.57)\times\frac{2.53^2}{6}\right\}$$
$$= 4.66$$

(4) Bending moment

$$M_x = 4.66x - \frac{2.57x^2}{2} - \frac{(5.92-2.57)}{2.53}\times\frac{x^3}{6}$$
$$= -\frac{0.66}{3}x^3 - \frac{2.57}{2}x^2 + 4.66x$$

$$\frac{dM_x}{dx} = -0.66x^2 - 2.57x + 4.66.$$

$$\frac{dM_x}{dx} = 0. \quad \therefore x = \frac{+2.57 \pm \sqrt{2.57^2 + 4 \times 0.66 \times 4.66}}{-2 \times 0.66} = 1.34^m$$

$$\therefore M_{max} = M_{x=1.34}$$
$$= -\frac{0.66}{3} \times 1.34^3 - \frac{2.57}{2} \times 1.34^2 + 4.66 \times 1.34 = 3.40^{t\text{-}m}$$

(5) Check of stress on section C-C', that shows maximum moment;

vertical load (t)	arm length (m)
$R_A = 4.36$	0.15
$W_1 = 0.35 \times 0.30 \times 1.00 \times 2.5 = 0.26$	0.45
$W_2 = 1.17 \times 0.60 \times 1.00 \times 2.5 = 1.76$	0.30
$W_3 = 1.17 \times 0.53 \times \frac{1}{2} \times 1.00 \times 2.5 = 0.78$	0.77
$P_1' = 1.00 \times 0.83 \times 1.00 \times 1.0 = 0.83$	0.71
$P_2' = 1.01 \times 0.83 \times 1.00 \times 2.0 = 1.68$	0.71
$P_3' = 0.35 \times 0.53 \times 1.00 \times 2.0 = 0.37$	0.86
$P_4' = 1.17 \times 0.53 \times \frac{1}{2} \times 1.00 \times 2.0 = 0.62$	0.95
$\Sigma V = 10.66^t$	

$$M_C = R_A \times 0.15 + W_1 \times 0.45 + W_2 \times 0.30 + W_3 \times 0.77$$
$$+ P_1' \times 0.71 + P_2' \times 0.71 + P_3' \times 0.86 + P_4' \times 0.95 + M_{max}.$$
$$= 7.99^{t\text{-}m}.$$

$$\bar{x} = \frac{M_C}{\Sigma V} = \frac{7.99}{10.66} = 0.75.$$

$$e = \left| \frac{B}{2} - \bar{x} \right| = \left| \frac{1.13}{2} - 0.75 \right| = 0.185,$$

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

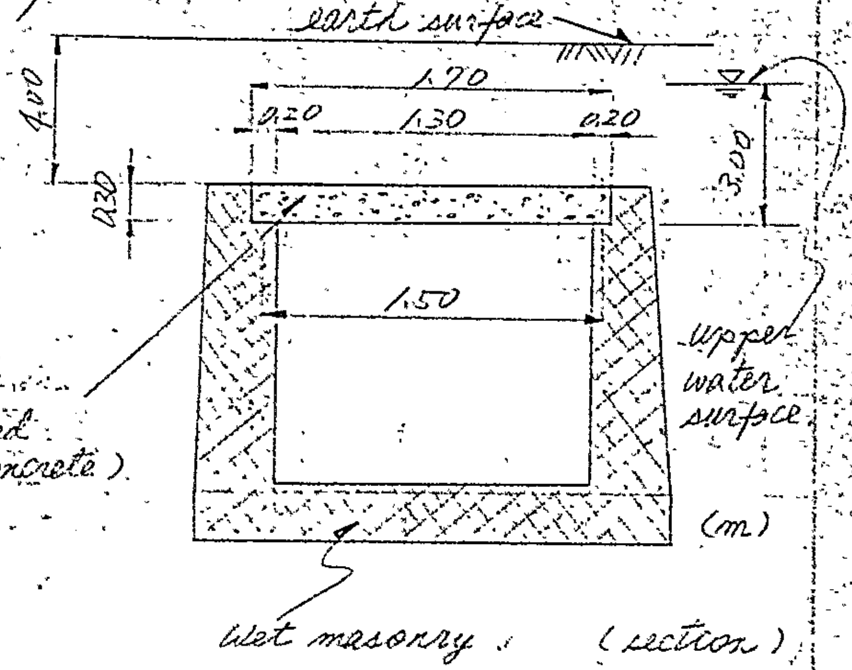
$$\frac{B}{6} \geqq e \quad \text{(O.K.)}$$

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

2-3-2. No.2 and No.3 Irrigation Syphon.

2-3-2-1. Top floor

(1) General.



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

unit weight { concrete: 2.4 t/m^3 ; earth in water: 2.0 t/m^3 }

beam length: 1.50 m

$\sigma_{sa} = 1.400 \ kg/cm^2$

$\sigma_{ca} = 40 \ kg/cm^2$

$\tau_{ca} = 4.5 \ kg/cm^2$

water head = 3.00 m

(2) Load condition

Earth

$q_1 = 4.00 \times 1.00 \times 2.0 = 8.00 \ t/m$

Top floor

$q_2 = 0.30 \times 1.00 \times 2.4 = 0.72\ ^{t}/_{m}$

Total load: q_0

$q_0 = q_1 + q_2 = 8.72\ ^{t}/_{m}$.

(3) reaction force.

$$R = q_0 \times 1.50 \times \frac{1}{2} = 6.54\ ^{t}.$$

(4) bending moment.

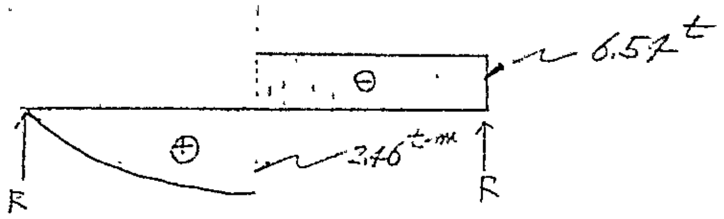
$$M_{max} = R \times \frac{1.50}{2} - q \times \frac{1.50}{2} \times \frac{1.50}{2} \times \frac{1}{2}$$

$$= 6.54 \times \frac{1.50}{2} - 8.72 \times \frac{1.50^2}{8}$$

$$= 2.46\ ^{t\text{-}m}$$

(5) shearing force

$$S_{max} = R = 6.54\ ^{t}.$$



M-figure, S-figure.

(6) Reinforcement amount

Assuming $d = 24^{cm}$.

$$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d}$$

$$= \frac{246.000}{1.400 \times 7/8 \times 24}$$

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

designed amount

<u>$\phi 19$ @ 300 $(A_s = 9.45^{cm^2})$</u>

(7) Check.

$$p = \frac{A_s}{bd} = \frac{9.45}{100 \times 24} = 0.0039$$

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

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

$$\sigma_s = \frac{M}{pjbd^2} = \frac{246.000}{0.0039 \times 0.90 \times 100 \times 24^2}$$

$$= 1.217 \text{ kg/cm}^2 < 1.400 \text{ kg/cm}^2 = \sigma_{sa}$$

(O.K)

$$\sigma_c = \frac{2M}{kjbd^2} = \frac{2 \times 246.000}{0.29 \times 0.90 \times 100 \times 24^2}$$

$$= 32.73 \text{ kg/cm}^2 < 40 \text{ kg/cm}^2 = \sigma_{ca}$$

(O.K.)

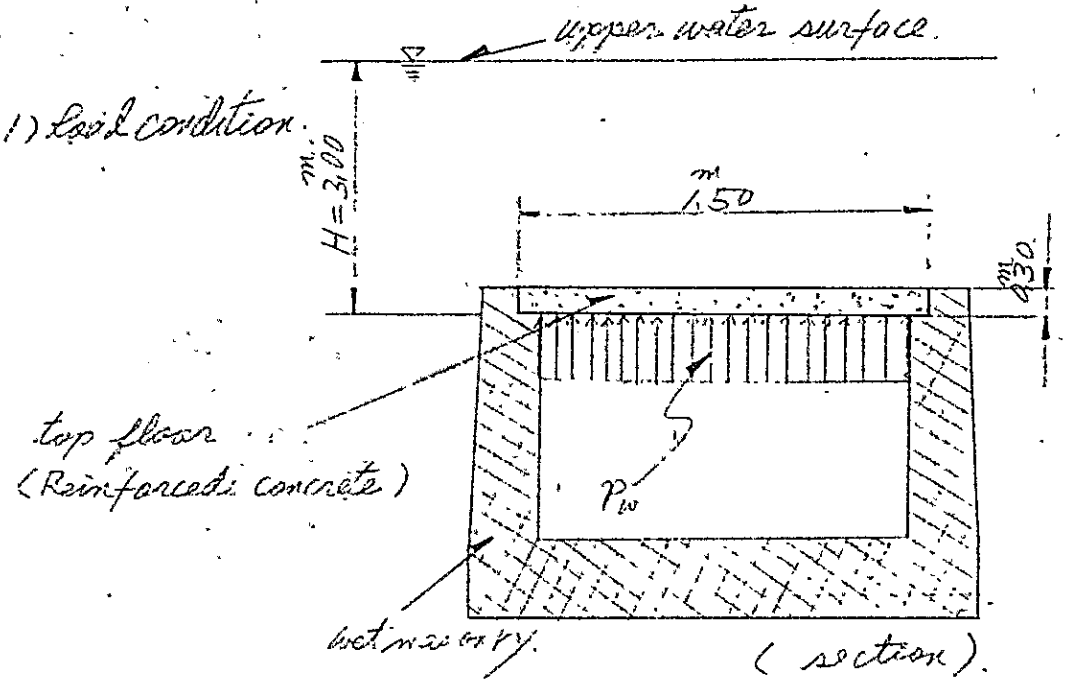
$$\tau_c = \frac{S_{max}}{bjd} = \frac{6.540}{100 \times 0.90 \times 24}$$

$$= 3.03 \text{ kg/cm}^2 < 4.5 \text{ kg/cm}^2 = \tau_{ca}$$

(O.K.)

2-3-2-2. Examination of inside pressure.

(1) load condition.



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

so that;

$$P_w = H \times w \times 1.00^m$$
$$= 3.00 \times 1.0 \times 1.00 = 3.00\ t/m$$

Top floor

$$g_1 = 0.72\ t/m$$

total load.

$$g_0 = P_w - g_1 = 2.28\ t/m$$

in this case, earth pressure over the top floor is neglected;

(2) Reaction force: R

$$R = \frac{2.28 \times 1.30}{2} = 1.48\ t$$

(3) Bending moment: M.

$$M_{max} = R \times \frac{1.30}{2} - 8_0 \times \left(\frac{1.30}{2}\right)^2 \times \frac{1}{2}$$
$$= 0.48\ t\text{-}m$$

(4) Shearing force.

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

(5) Reinforcement amount.

assuming $d = 24^{cm}$

$$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d} = \frac{48000}{1400 \times 7/8 \times 24}$$

$$= 1.63\ cm^2$$

designed amount:

ϕ 13 @ 300 $\quad (A_s = 4.42\ cm^2)$

(6) Check.

$$p = \frac{A_s}{bd} = \frac{4.42}{100 \times 24} = 0.0018$$

$$k = \sqrt{(np)^2 + 2np} - np = 0.49$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.49}{3} = 0.84$$

$$\sigma_s = \frac{M}{pjbd^2} = \frac{48.000}{0.0018 \times 0.84 \times 100 \times 24^2}$$
$$= 551\ kg/cm^2 < 1400\ kg/cm^2 = \sigma_{sa} \quad (O.K)$$

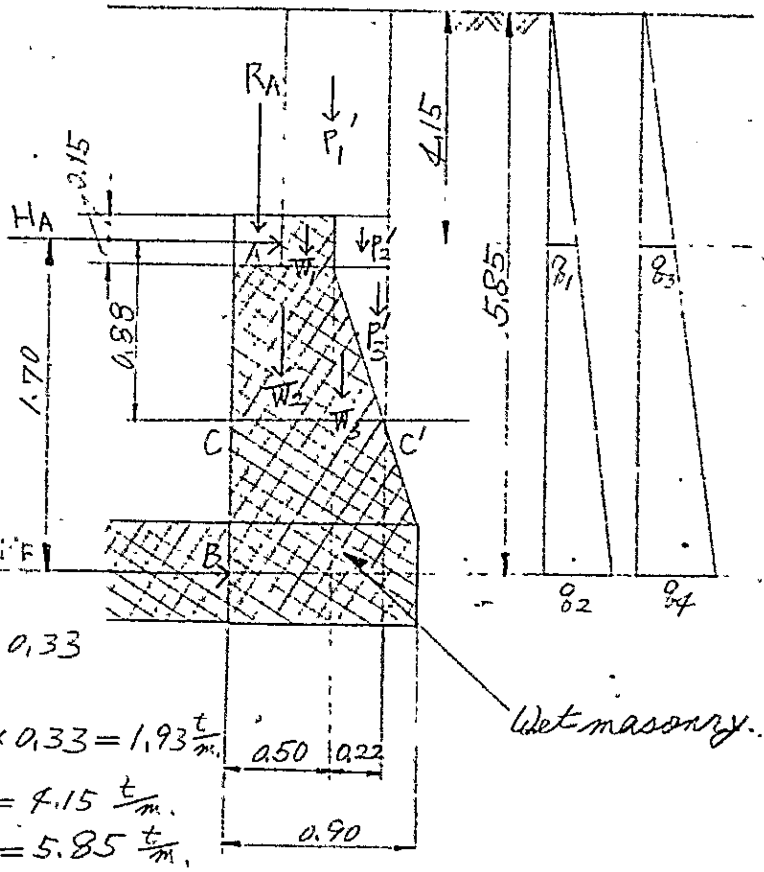
$$\sigma_c = \frac{2M}{kjbd^2} = \frac{48.000}{0.49 \times 0.84 \times 100 \times 24^2}$$
$$= 2.03\ kg/cm^2 < 40\ kg/cm^2 = \sigma_{ca} \quad (O.K.)$$

$$\tau_c = \frac{S_{max}}{bjd} = \frac{1480}{100 \times 0.84 \times 24}$$
$$= 0.7\ kg/cm^2 < 4.5\ kg/cm^2 = \tau_{ca} \quad (O.K.)$$

2-3-2-3 Safety calculation of side wall of Wet masonry.

(1) General

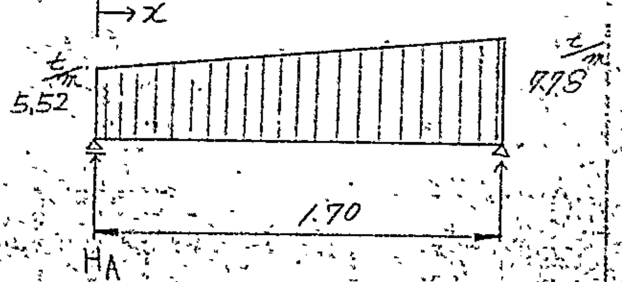
calculation & examination are done in the same way as 2-3-1-3;



(2) Load condition

$q_1 = 4.15 \times 1.00 \times 1.0 \times 0.33$
$= 1.37\ t/m.$
$q_2 = 5.85 \times 1.00 \times 1.0 \times 0.33 = 1.93\ t/m.$
$q_3 = 4.15 \times 1.00 \times 1.0 = 4.15\ t/m.$
$q_4 = 5.85 \times 1.00 \times 1.0 = 5.85\ t/m.$

$q_1 + q_3 = 5.52\ t/m.$
$q_2 + q_4 = 7.78\ t/m.$



(3) Reaction force.

$$H_A = \frac{1}{1.70}\left\{5.52 \times \frac{1.70^2}{2} + (7.78 - 5.52) \times \frac{1.70^2}{6}\right\}$$
$= 5.34\ t$

(4) Bending moment

$$M_x = 5.34x - \frac{5.52}{2}x^2 - \frac{(7.78-5.52)}{1.70} \times \frac{x^3}{6}$$
$$= -\frac{0.66}{3}x^3 - \frac{5.52}{2}x^2 + 5.34x.$$

$$\frac{dM_x}{dx} = -0.66x^2 - 5.52x + 5.34.$$

$$\frac{dM_x}{dx} = 0 \longrightarrow x = \frac{-5.52 \pm \sqrt{5.52^2 + 4 \times 0.66 \times 5.34}}{2 \times 0.66} = 0.88$$

$$M_{max} = M_{x=0.88}.$$
$$= -\frac{0.66}{3} \times 0.88^3 - \frac{5.52}{2} \times 0.88^2 + 5.34 \times 0.88$$
$$= 2.41^{t\text{-}m}$$

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

vertical load. arm length (m)

$R_A = 6.54^t$	0.13
$W_1 = 0.30 \times 0.25 \times 1.00 \times 2.5 = 0.19$	0.38
$W_2 = 0.73 \times 0.50 \times 1.00 \times 2.5 = 0.91$	0.25
$W_3 = 0.73 \times 0.22 \times \frac{1}{2} \times 1.00 \times 2.5 = 0.20$	0.57
$P_1' = 4.00 \times 0.47 \times 1.00 \times 2.0 = 3.76$	0.48
$P_2' = 0.30 \times 0.22 \times 1.00 \times 2.0 = 0.13.$	0.61
$P_3' = 0.73 \times 0.22 \times \frac{1}{2} \times 1.00 \times 2.0 = 0.16$	0.64

$$\Sigma V = 11.89^t.$$

$$M_c = 6.54 \times 0.13 + 0.19 \times 0.38 + 0.91 \times 0.25 + 0.20 \times 0.57$$
$$+ 3.76 \times 0.48 + 0.13 \times 0.61 + 0.16 \times 0.64 + 2.41$$
$$= 5.65.$$

$$\bar{x} = \frac{M_c}{\Sigma V} = \frac{5.65}{11.89} = 0.475$$

$$e = \left|\frac{B}{2} - \bar{x}\right| = \left|\frac{0.72}{2} - 0.475\right| = 0.115.$$

$$\frac{B}{6} = \frac{0.72}{6} = 0.12$$

$$\therefore \frac{B}{6} \geqq e$$

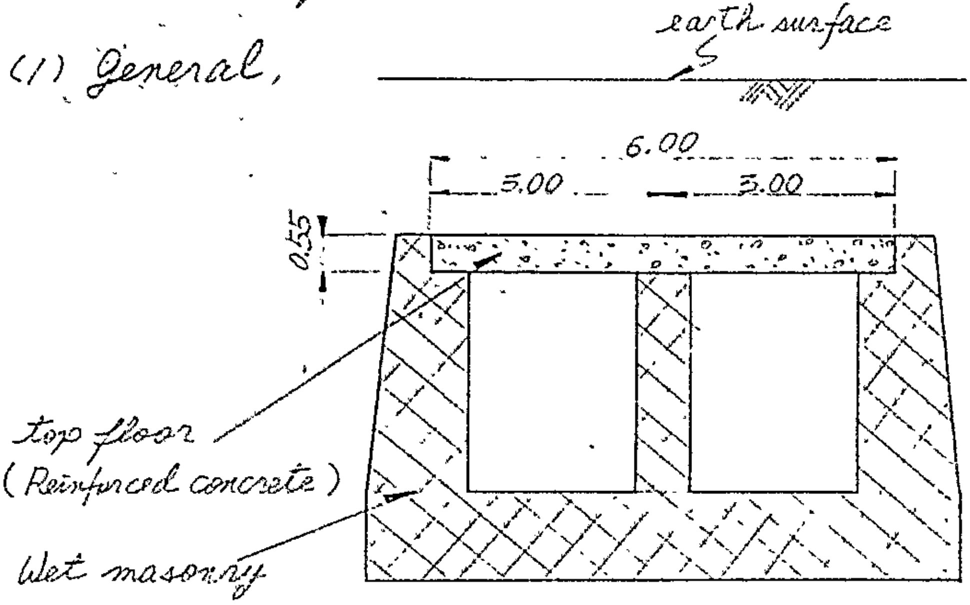
now, strain stress is not occurred on section C-C'.

2-4, Irrigation Culvert.

2-4-1, No.1 and No.3 Irrigation Culvert.

2-4-1-1 Top floor

(1) General,



(cross section) (m)

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

beam length ; 6.00 m

one span length ; 3.00 m

allowable stress for compression of concrete,

$\sigma_{ca} = 40 \ \mathrm{Kg/cm^2}$

allowable stress for tension of reinforcement,

$\sigma_{sa} = 1400 \ \mathrm{Kg/cm^2}$

allowable shearing stress of concrete,

$\tau_{ca} = 4.5 \ \mathrm{Kg/cm^2}$

(2) Load condition

Water

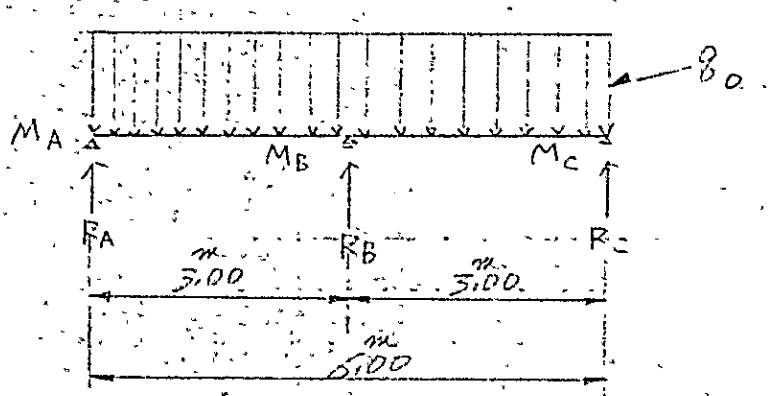
$q_1 = 5.15 \times 1.00 \times 1.8 = 9.27\ t/m$

Top floor

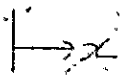
$q_2 = 0.55 \times 1.00 \times 2.4 = 1.32\ t/m$

Total load

$q_0 = q_1 + q_2 = 10.59\ t/m$



continuous beam profile



(3) Bending moment and shearing force

By Method of three moments calculation is done ;

$$M_A + 4M_B + M_C = -q_0 \times 3.00^2 \times \frac{1}{2}$$

(Clapeyron eq.)

$M_A = M_C = 0$

so that

$$M_B = -q_0 \times 3.00^2 \times \frac{1}{8} = -10.59 \times 3.00^2 \times \frac{1}{8}$$
$$= -11.91 \ t\text{-}m$$

(a) Reaction force

$$R_B = \frac{5}{4} q_0 l = \frac{5}{4} \times 10.59 \times 3.00$$
$$= 39.71 \ t$$

$$R_A = R_C = \frac{3}{8} q_0 l = \frac{3}{8} \times 10.59 \times 3.00$$
$$= 11.91 \ t$$

(b) Bending moment

$0 \leq x \leq 3.00^m$

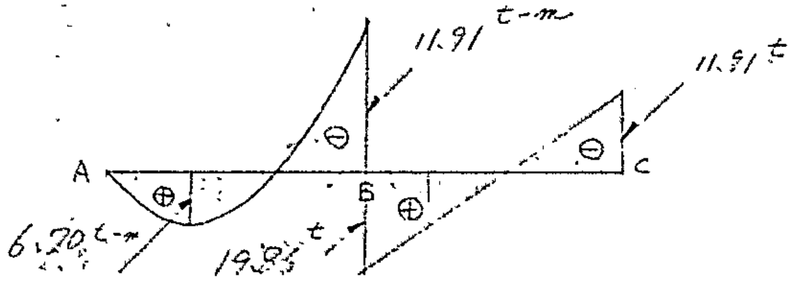
$$M_x = +R_A x - q_0 \times \frac{x^2}{2}$$
$$= +11.91 x - \frac{10.59}{2} x^2$$
$$= -\frac{10.59}{2} x^2 + 11.91 x$$

$$\therefore \frac{dM_x}{dx} = -10.59 x + 11.91$$
$$\therefore x = 1.12 \ m$$

$$M_{max} = M_{x=1.12}$$
$$= -\frac{10.59}{2} \times 1.12^2 + 11.91 \times 1.12$$
$$= +6.70 \ t\text{-}m$$

(c) Shearing force

$$S_{max} = \left| -10.59 \times 3.00 + 11.91 \right|$$
$$= 19.86 \ t$$



M-figure. S-figure.

(4) Reinforcement amount

(a) in the case of $M = 11.91^{t\text{-}m} = 1{,}191{,}000^{kg\text{-}cm}$

Assuming. $d = 0.49^{m} = 49^{cm}$

$$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d}$$

$$= \frac{1191{,}000}{1{,}400 \times 7/8 \times 49}$$

$$= 19.84^{cm^2}$$

designed amount:

$\phi 22 @ 150$ $(A_s = 25.34^{cm^2})$

(b) in the case of $M = 6.70^{t\text{-}m} = 670{,}000^{kg\text{-}cm}$

$$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d} = \frac{670{,}000}{1{,}400 \times 7/8 \times 49}$$

$$= 11.16^{cm^2}$$

designed amount

$\phi 16 @ 150$ $(A_s = 13.41^{cm^2})$

(5). Check.

(a). in the case of $M = 11.91$ t-m $= 1.191.000$ kg-cm

$$p = \frac{As}{bd} = \frac{25.34}{100 \times 49} = 0.0052$$

$$k = \sqrt{(np)^2 + 2np} - np = 0.33$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.33}{3} = 0.89$$

$$\sigma_s = \frac{M}{pjbd^2}$$

$$= \frac{1191000}{0.0052 \times 0.89 \times 100 \times 49^2}$$

$$= 1.072 \text{ kg/cm}^2 < 1.400 \text{ kg/cm}^2 = \sigma_{sa}$$

(OK.)

$$\sigma_c = \frac{2M}{kjbd^2}$$

$$= \frac{2 \times 1.191000}{0.33 \times 0.89 \times 100 \times 49^2}$$

$$= 33.78 \text{ kg/cm}^2 < 40 \text{ kg/cm}^2 = \sigma_{ca}$$

(O.K.)

$$\tau_c = \frac{S_{max}}{bjd}$$

$$= \frac{19860}{100 \times 0.89 \times 49}$$

$$= 4.5 \text{ kg/cm}^2 \fallingdotseq 4.5 \text{ kg/cm}^2 = \tau_{ca}$$

(o.k.)

(b) in the case of $M = 6.70^{t\text{-}m} = 670{,}000^{kg\text{-}cm}$

$$p = \frac{A_s}{bd} = \frac{15.41}{100 \times 49} = 0.0027$$

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

$$= \sqrt{(15 \times 0.0027)^2 + 2 \times 15 \times 0.0027} - 15 \times 0.0027$$

$$= 0.17$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.17}{3} = 0.94$$

$$\sigma_s = \frac{M}{pjbd^2} = \frac{670000}{0.0027 \times 0.94 \times 100 \times 49^2}$$

$$= 1099 \ kg/cm^2 < 1200 \ kg/cm^2 = \sigma_{sa}$$

(OK)

$$\sigma_c = \frac{2M}{kjbd^2} = \frac{2 \times 670{,}000}{0.17 \times 0.94 \times 100 \times 49^2}$$

$$= 34.92 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$

(OK)

$$\tau_c = \frac{S_{max}}{bjd}$$

$$= \frac{19.860}{100 \times 0.94 \times 49}$$

$$= 4.3 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K.)

2-4-1-2. Safety-calculation of side wall of Wet masonry.

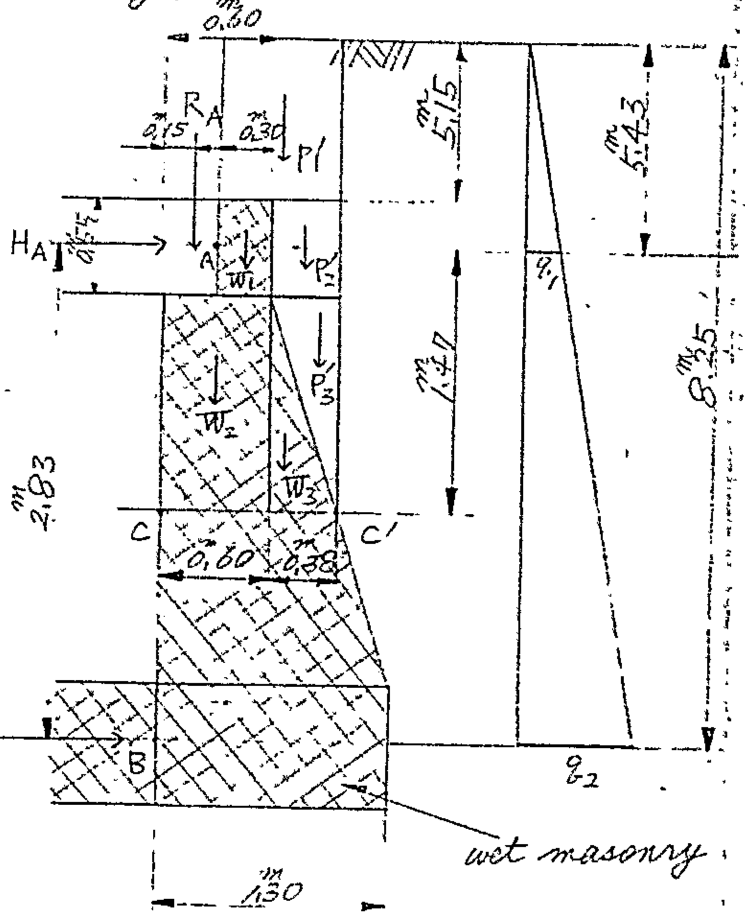
(1) General.

the side wall of Culvert is calculated and examined as a simple beam;

acctive lateral earth coefficient:

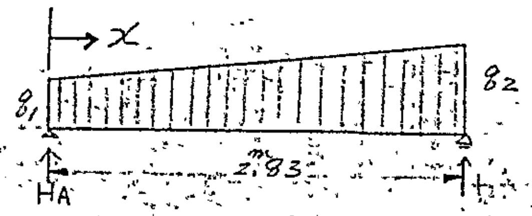
$K_A = 0.33$

specific weight of Wet masonry:
2.5 t/m3



(2) Bending moment

$$q_1 = 5.43 \times 1.00 \times 1.8 \times 0.33 = 3.23\ t/m$$
$$q_2 = 8.25 \times 1.00 \times 1.8 \times 0.33 = 4.90\ t/m$$



(a) Reaction force

$$H_A = \frac{1}{2.83}\left\{ q_1 \times \frac{2.83^2}{2} + (q_2 - q_1) \times \frac{2.83^2}{6} \right\}$$
$$= \frac{1}{2.83}\left\{ 3.23 \times \frac{2.83^2}{2} + (4.90 - 3.23) \times \frac{2.83^2}{6} \right\} = 5.36^t$$

(4) Bending moment:

$$M_x = H_A x - g_1 \frac{x^2}{2} - (g_2 - g_1)\frac{1}{2.83} \times \frac{x^3}{6}$$

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

$$\frac{dM_x}{dx} = -0.30x^2 - 3.23x + 5.36$$

$$\frac{dM_x}{dx} = 0: \qquad x = \frac{3.23 \pm \sqrt{3.23^2 + 4 \times 0.30 \times 5.36}}{-2 \times 0.30} = 1.47$$

$$(x \geqq 0)$$

$$\therefore \ M_{max} = M_{x=1.47}$$

$$= -\frac{0.30}{3} \times 1.47^3 - \frac{3.23}{2} \times 1.47^2 + 5.36 \times 1.47$$

$$= 4.07 \ t\text{-}m$$

(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)
	m
$R_A = 11.91$ t	0.15
$W_1 = 0.55 \times 0.30 \times 1.00 \times 2.5 = 0.41$	0.45
$W_2 = (1.47 - 0.28) \times 0.60 \times 1.00 \times 2.5 = 1.78$	0.30
$W_3 = (1.47 - 0.28) \times 0.38 \times \frac{1}{2} \times 1.00 \times 2.5 = 0.57$	0.73
$P_1' = (5.43 - 0.28) \times 0.68 \times 1.00 \times 1.8 = 6.30$	0.64
$P_2' = 0.55 \times 0.38 \times 1.00 \times 1.8 = 0.38$	0.79
$P_3' = (1.47 - 0.28) \times 0.38 \times \frac{1}{2} \times 1.00 \times 1.8 = 0.41$	0.86

$\Sigma V = 21.76$ t

Bending moment;

$$M_C = R_A \times 0.15 + W_1 \times 0.45 + W_2 \times 0.30 + W_3 \times 0.73 + P_1 \times 0.64 + P_2 \times 0.79 + P_3 \times 0.86 + M_{max},$$

$$= 11.91 \times 0.15 + 0.41 \times 0.45 + 1.78 \times 0.30 + 0.57 \times 0.73 + 6.30 \times 0.64 + 0.38 \times 0.79 + 0.41 \times 0.86 + 4.07$$

$$= 11.67 \ t\text{-}m$$

$$\bar{x} = \frac{M_c}{\Sigma V} = \frac{11.67}{21.76} = 0.54$$

$$e = \left| \frac{B}{2} - \bar{x} \right| = \left| \frac{0.60+0.38}{2} - 0.54 \right| = 0.05.$$

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

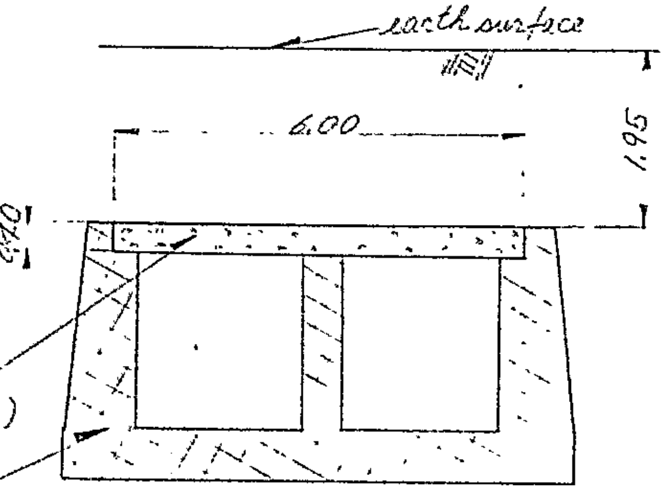
$$\therefore \quad \frac{B}{6} > e \qquad (\text{O.K.})$$

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

2-4-2. No.2. Irrigation Culvert.

2-4-2-1. Top floor.

(1) General.



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

unit weight of concrete : 2.4 t/m^3.
unit weight of earth in water ; 2.0 t/m^3
~~Beam length~~ : 6.00^m.
one span length ; 3.00^m.
σ_{ca} = 40 Kg/cm^2,
σ_{sa} = 1400 Kg/cm^2
τ_{ca} = 4.5 Kg/cm^2,

(2) Load condition.

Earth

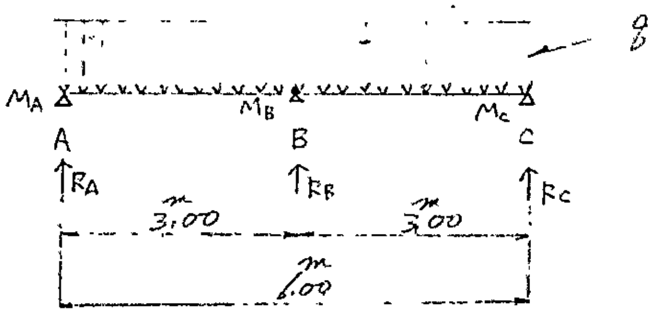
$q_1 = 1.95 \times 1.00 \times 2.0$
$= 3.90$ t/m.

Top floor.

$q_2 = 0.40 \times 1.00 \times 2.4$
$= 0.96$ t/m

Total load : q_0

$q_0 = q_1 + q_2$
$= 4.86$ t/m



profile of continuous beam.

(3) Bending moment and shearing force.

by Method of three moments, calculation is done;

$$M_A + 4M_B + M_C = -g_0 \times 3.00^2 \times \frac{1}{2}.$$

(Clapeyron eq.)

$M_A = M_C = 0.$

so that,

$$M_B = -g_0 \times 3.00^2 \times \frac{1}{2} \times \frac{1}{4}$$
$$= -4.86 \times 3.00^2 \times \frac{1}{2} \times \frac{1}{4}$$
$$= -5.47^{t\text{-}m}.$$

(a) Reaction forces R_A. R_B. R_C

$$R_B = \frac{5}{4} g_0 l$$
$$= \frac{5}{4} \times 4.86 \times 3.00$$
$$= 18.23^{t}$$

$$R_A = R_C = (g_0 \times 6.00 - R_B) \times \frac{1}{2}.$$
$$= (4.86 \times 6.00 - 18.23) \times \frac{1}{2}.$$
$$= 5.47^{t}$$

(b) Banding moment.

$0 \leq x \leq 3.00^{m}$:

$$M_x = +R_A x - q_0 \times \frac{x^2}{2}$$
$$= 5.47x - 4.86 \times \frac{x^2}{2}$$
$$= -\frac{4.86}{2}x^2 + 5.47x.$$

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

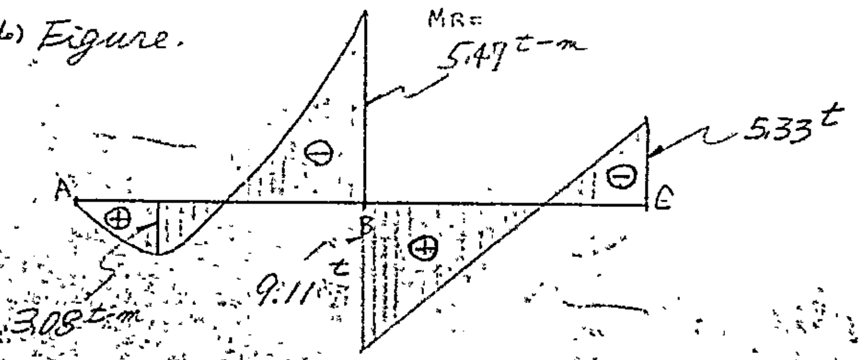
$$\frac{dM_x}{dx} = 0 \quad \therefore \quad x = 1.13^{m}.$$

$$M_{max} = M_{x=1.13}$$
$$= -\frac{4.86}{2} \times 1.13^2 + 5.47 \times 1.13$$
$$= +3.08$$

(c) Shearing force

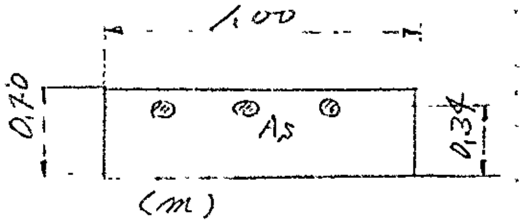
$$S_{max} = \left| -4.86 \times 3.00 + 5.47 \right|.$$
$$= 9.11^{t}$$

(d) Figure.



M-figure , S-figure.

(4) Reinforcement amount.



(a) in the case of $M_B = 54.7^{t\text{-}m} = 547{,}000^{kg\text{-}cm}$.

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

$$A_s = \frac{M}{\sigma_{sa} \cdot 7/8 \cdot d}$$

$$= \frac{547{,}000}{1{,}400 \times 7/8 \times 34}$$

$$= 13.13 \ cm^2$$

designed amount

$\phi 16$ @ 150 ($A_s = 13.41^{cm^2}$)

(b) in the case of $M = 30.8^{t\text{-}m} = 308{,}000^{kg\text{-}cm}$.

$$A_s = \frac{M}{\sigma_{sa} \cdot 7/8 \cdot d}$$

$$= \frac{308{,}000}{1{,}400 \times 7/8 \times 34}$$

$$= 7.39 \ cm^2$$

designed amount

$\phi 13$ @ 150 ($A_s = 8.85^{cm^2}$)

(5) Check.

(a) in the case of $M = 5.47^{t\text{-}m} = 547.000^{Kg\text{-}cm}$

$$p = \frac{13.41}{b \cdot d}$$

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

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

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

$$= 0.29$$

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

$$= 0.90.$$

$$\therefore \sigma_s = \frac{M}{pjbd^2}.$$

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

$$= 1.348 \ Kg/cm^2 < 1.400 \ Kg/cm^2 = \sigma_{sa}$$

(O.K.)

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

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

$$= 18.13 \ Kg/cm^2 < 40 \ Kg/cm^2 = \sigma_{ca}$$

(O.K.)

$$\tau_c = \frac{S_{max}}{bjd}$$

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

$$= 3.0 \ Kg/cm^2 < 4.5 \ Kg/cm^2 = \tau_{ca}$$

(O.K.)

(b) in the case of : $M = 3.08^{t\text{-}m} = 308.000^{kg\text{-}cm}$

$$p = \frac{As}{bd} = \frac{8.85}{100 \times 34} = 0.0026.$$

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

$$= \sqrt{(15 \times 0.0026)^2 + 2 \times 15 \times 0.0026} - 15 \times 0.0026$$

$$= 0.24$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.24}{3} = 0.92 ,$$

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

$$= \frac{308000}{0.0026 \times 0.92 \times 100 \times 34^2}$$

$$= 1.114 \ kg/cm^2 < 1400 \ kg/cm^2 = \sigma_{sa}$$

(O.K.)

$$\sigma_c = \frac{2M}{k j b d^2}$$

$$= \frac{2 \times 308000}{0.24 \times 0.92 \times 100 \times 34^2}$$

$$= 24.13 \ kg/cm^2 < 40 \ kg/cm^2 = \sigma_{ca}$$

(O. K.)

$$\tau_c = \frac{S_{max}}{b j d}$$

$$= \frac{9.110}{100 \times 0.92 \times 34}$$

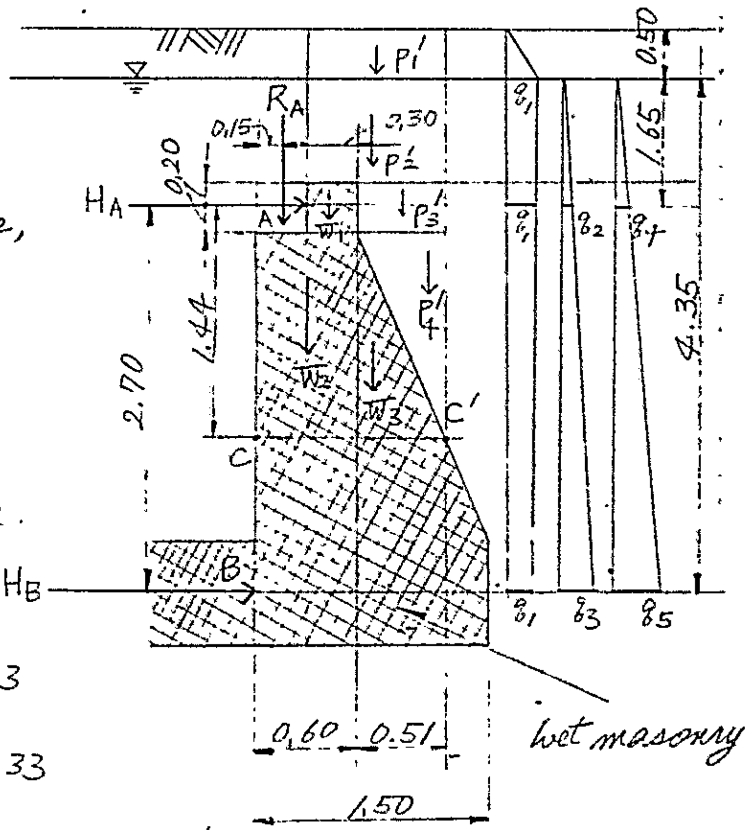
$$= 2.9 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}.$$

(O. K.)

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

(1) General.

calculation & examination are done in the same way as that before,



(2) Load condition.

$q_1 = 0.50 \times 1.00 \times 1.8 \times 0.33$
$= 0.30$ t/m

$q_2 = 1.65 \times 1.00 \times 1.0 \times 0.33$
$= 0.54$ t/m.

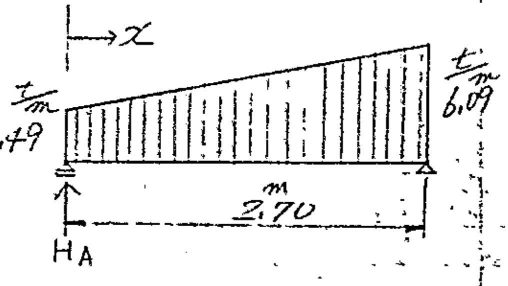
$q_3 = 4.35 \times 1.00 \times 1.0 \times 0.33 = 1.44$ t/m.

$q_4 = 1.65 \times 1.00 \times 1.0 = 1.65$ t/m.

$q_5 = 4.35 \times 1.00 \times 1.0 = 4.35$ t/m.

$q_1 + q_2 + q_4 = 2.49$ t/m.

$q_1 + q_3 + q_5 = 6.09$ t/m.



(3) Reaction force

$$H_A = \frac{1}{2.70}\left\{2.49 \times \frac{2.70^2}{2} + (6.09 - 2.49) \times \frac{2.70^2}{6}\right\}$$

$= 4.98^t$

(4) Bending moment.

$$M_x = 4.98x - \frac{2.49}{2}x^2 - \frac{(6.09-2.49)}{2.70} \times \frac{x^3}{6}$$
$$= -\frac{0.67}{3}x^3 - \frac{2.49}{2}x^2 + 4.98x$$

$$\frac{dM_x}{dx} = -0.67x^2 - 2.49x + 4.98$$

$$\frac{dM_x}{dx} = 0. \qquad x = \frac{-2.49 \pm \sqrt{2.49^2 + 4 \times 0.67 \times 4.98}}{2 \times 0.67}$$
$$= 1.44^{m}$$

$$M_{max} = M_{x=1.44}$$
$$= -\frac{0.67}{3} \times 1.44^3 - \frac{2.49}{2} \times 1.44^2 + 4.98 \times 1.44$$
$$= 3.92^{t-m}$$

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

vertical load. arm length (m.)

		arm length (m.)
$R_A = 5.36^t$		0.15
$W_1 = 0.40 \times 0.30 \times 1.00 \times 2.5 =$	0.30	0.45
$W_2 = 1.24 \times 0.60 \times 1.00 \times 2.5 =$	1.86	0.30
$W_3 = 1.24 \times 0.51 \times \frac{1}{2} \times 1.00 \times 2.5 =$	0.79	0.77
$P_1' = 0.50 \times 0.81 \times 1.00 \times 1.8 =$	0.73	0.70
$P_2' = 1.45 \times 0.81 \times 1.00 \times 2.0 =$	2.35	0.70
$P_3' = 0.40 \times 0.51 \times 1.00 \times 2.0 =$	0.41	0.85
$P_4' = 1.24 \times 0.51 \times \frac{1}{2} \times 1.00 \times 2.0 =$	0.63	0.94

$$\Sigma V = 12.43^t$$

Bending moment.

$$M_c = 5.36 \times 0.15 + 0.30 \times 0.45 + 1.86 \times 0.30 + 0.79 \times 0.77 + 0.73 \times 0.70 + 2.35 \times 0.70 + 0.41 \times 0.85 + 0.63 \times 0.94 + 3.92$$
$$= 9.13^{t-m}$$

$$\bar{x} = \frac{M_c}{\Sigma V} = \frac{9.13}{12.43} = 0.73$$

$$e = \left|\frac{B}{2} - \bar{x}\right| = \left|\frac{1.11}{2} - 0.73\right| = 0.18$$

$$\frac{B}{6} = \frac{1.11}{6} = 0.19$$

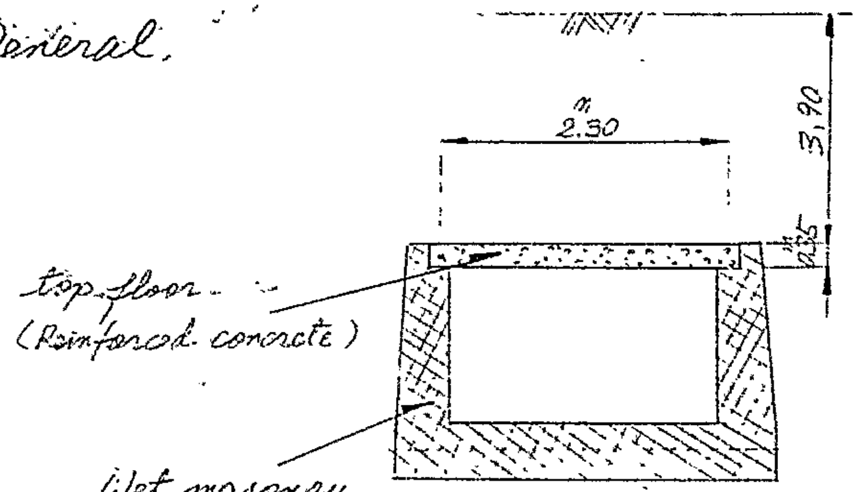
$$\frac{B}{6} > e, \qquad (O.K.)$$

now strain stress is not occured on section C-C'.

2-4-3. No. 4. Irrigation Culvert,

2-4-3-1. Top floor

(1) General.



(section)

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

Unit weight: concrete : 2.4 t/m^3
earth : 1.8 t/m^3

beam length : 1.85 m

$\sigma_{sa} = 1,400\ kg/cm^2$

$\sigma_{ca} = 40\ kg/cm^2$

$\tau_{ca} = 4.5\ kg/cm^2$

(2) Load condition

Earth

$q_1 = 3.90 \times 1.00 \times 1.8 = 7.02\ t/m$

Top floor

$q_2 = 0.35 \times 1.00 \times 2.4 = 0.84\ t/m$

Total load

$q_0 = q_1 + q_2 = 7.86\ t/m$

(3) Bending moment and shearing force.

(a) Reaction force.

$R = g_0 \times 2.30 \times \frac{1}{2} = 7.86 \times 2.30 \times \frac{1}{2}$

$= 9.04\ t$

(b) Bending moment.

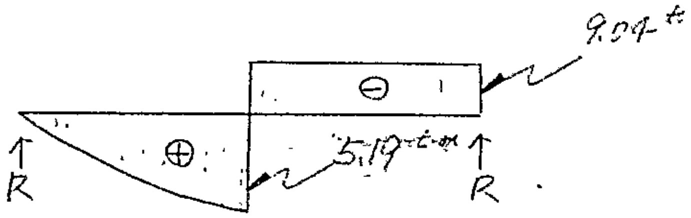
$M_{max} = \frac{1}{8} g_0 \times 2.30^2$

$= 7.86 \times 2.30^2 \times \frac{1}{8}$

$= 5.19^{t\text{-}m}$

(c) Shearing force.

$S_{max} = R = 9.04^{t}$



M-figure Q-figure.

(4) Reinforcement amount.

Assuming $d = 29^{cm}$.

$A_s = \frac{M}{\sigma_{sa} \times 7/8 \times d}$

$= \frac{519{,}000}{1400 \times 7/8 \times 29}$

$= 14.61\ cm^2$

designed amount

$\phi\ 19\ @\ 150 \quad (A_s = 18.90^{cm^2})$

(5) Check

$$p = \frac{A_s}{bd} = \frac{18.90}{100 \times 29} = 0.0065$$

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

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

$$\sigma_s = \frac{M}{p \cdot j \cdot b \cdot d^2}$$

$$= \frac{519.000}{0.0065 \times 0.88 \times 100 \times 29^2}$$

$$= 1.079 \ kg/cm^2 < 1.400 \ kg/cm^2 = \sigma_{sa}$$

(O.K)

$$\sigma_c = \frac{2M}{kjbd^2}$$

$$= \frac{2 \times 519.000}{0.35 \times 0.88 \times 100 \times 29^2}$$

$$= 40 \ kg/cm^2 \leqq 40 \ kg/cm^2 = \sigma_{ca}$$

(O.K.)

$$\tau_c = \frac{S_{max}}{bjd}$$

$$= \frac{9.040}{100 \times 0.88 \times 29}$$

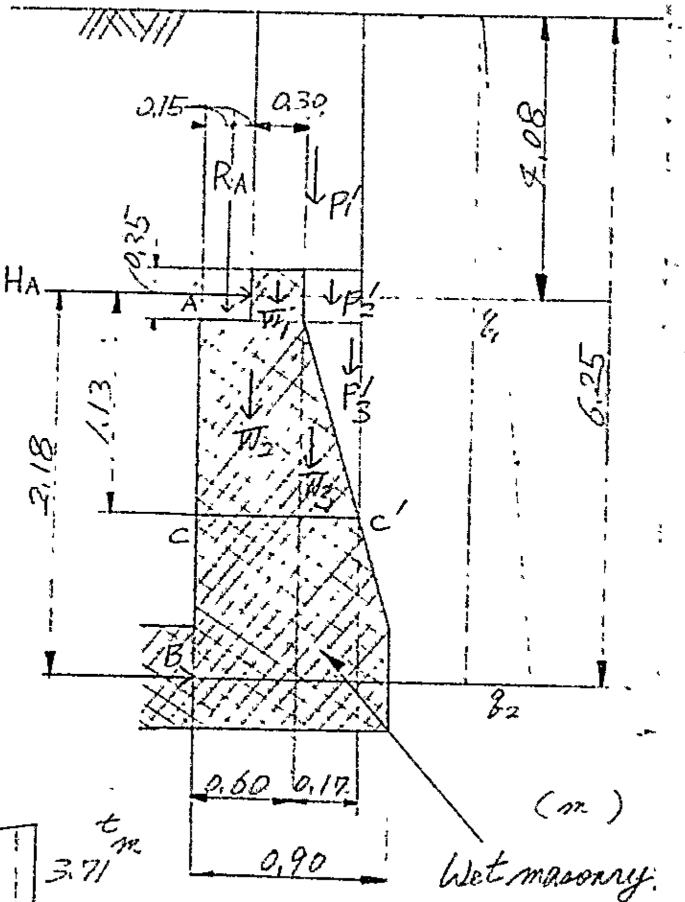
$$\doteq 3.5 \ kg/cm^2 < 4.5 \ kg/cm^2 = \tau_{ca}$$

(O.K.)

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

(1) General.

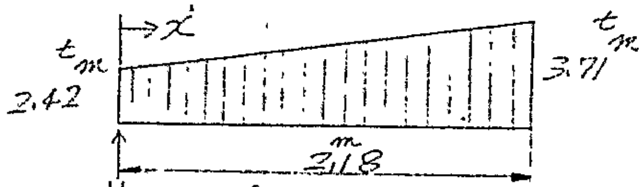
calculation & examination are done in the same way;



(2) Load condition

$$q_1 = 4.08 \times 1.00 \times 1.8 \times 0.33 = 2.42 \ t/m$$

$$q_2 = 6.25 \times 1.00 \times 1.8 \times 0.33 = 3.71 \ t/m$$



(3) Reaction force.

$$H_A = \frac{1}{2.18}\left\{2.42 \times \frac{2.18^2}{2} + (3.71 - 2.42) \times \frac{2.18^2}{6}\right\} = 3.11 \ t$$

(4) Bending moment.

$$M_x = 3.11x - \frac{2.42}{2}x^2 - \frac{(3.71-2.42)}{2.18} \times \frac{x^3}{6}$$

$$= -\frac{0.30}{3}x^3 - \frac{2.42}{2}x^2 + 3.11x$$

$$\frac{dM_x}{dx} = -0.30x^2 - 2.42x + 3.11$$

$$\frac{dM_x}{dx} = 0 \rightarrow x = \frac{-2.42 \pm \sqrt{2.42^2 + 4 \times 0.30 \times 3.11}}{2 \times 0.30} = 1.13^m$$

$M_{max} = M_{x=1.13}$

$= -\frac{0.30}{3} \times 1.13^3 - \frac{2.22}{2} \times 1.13^2 + 3.11 \times 1.13$

$= 1.82$ t-m

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

vertical load	arm length m
$R_A = 5.36$ t	0.15
$W_1 = 0.35 \times 0.30 \times 1.00 \times 2.5 = 0.26$	0.45
$W_2 = 0.95 \times 0.60 \times 1.00 \times 2.5 = 1.43$	0.30
$W_3 = 0.95 \times 0.17 \times \frac{1}{2} \times 1.00 \times 2.5 = 0.20$	0.66
$P_1' = 3.90 \times 0.47 \times 1.00 \times 1.8 = 3.30$	0.53
$P_2' = 0.35 \times 0.17 \times 1.00 \times 1.8 = 0.11$.	0.68
$P_3' = 0.95 \times 0.17 \times \frac{1}{2} \times 1.00 \times 1.8 = 0.15$.	0.72

$\Sigma V = 10.81$

$M_C = 5.36 \times 0.15 + 0.26 \times 0.45 + 1.43 \times 0.30$

$+ 0.20 \times 0.66 + 3.30 \times 0.53 + 0.11 \times 0.68$

$+ 0.15 \times 0.72 + 1.82$

$= 5.23$ t-m

$\bar{x} = \frac{5.23}{10.81} = 0.48$

$e = \left| \frac{B}{2} - \bar{x} \right| = 0.10$.

$\frac{B}{6} = 0.13$

$\therefore \frac{B}{6} > e$.

(O.K.)

now, strain stress is not occured on section C-C';

2-5 Bridge

2-5-1 NO.4 Bridge

2-5-1-1 Design criteria

Type : Reinforced Concrete Bridge (Simple slab)

Bridge Length , 7m300

Width 5m000
(Effective 4m500)

Span 6m900

Live Load Truck Load T=12 ton

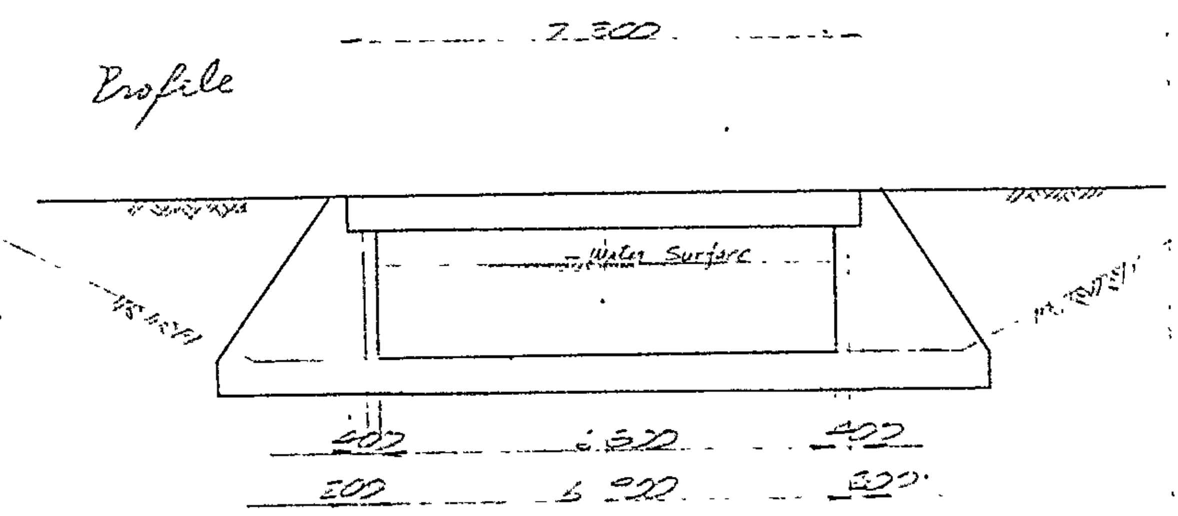
Allowable Stress for compression of concrete $\sigma_{ca} = 40$ kg/cm²

Allowable stress for tension of reinforcement $\sigma_{sa} = 1400$ "

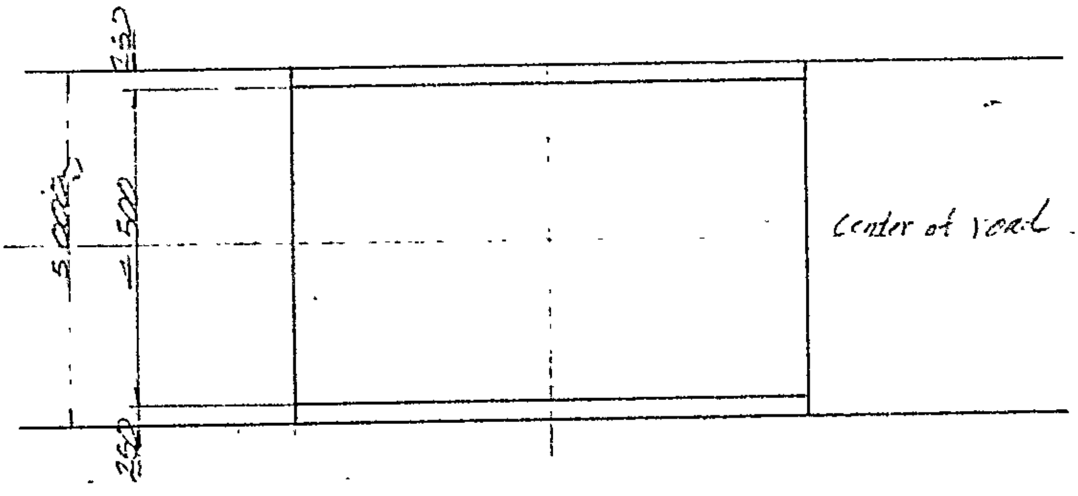
unit weight
- concrete $\gamma_c = 2.3$ t/m³
- Reinforced concrete $\gamma_r = 2.4$ t/m³

2-3-1-2 Given dimension.

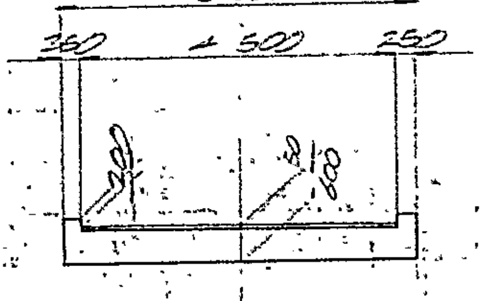
Profile



Plan

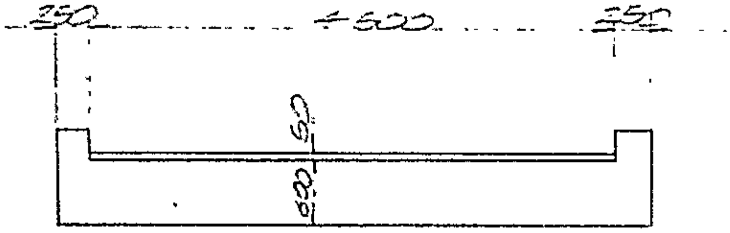


Crossection



2-5-1-3 Load

(1) Dead load



(Per one meter of width)

Pavement	(Concrete)	2.3 × 0.05 =	0.115
Slab	(R. Concrete)	2.4 × 0.60 =	1.440
		ΣW =	1.559 t/m

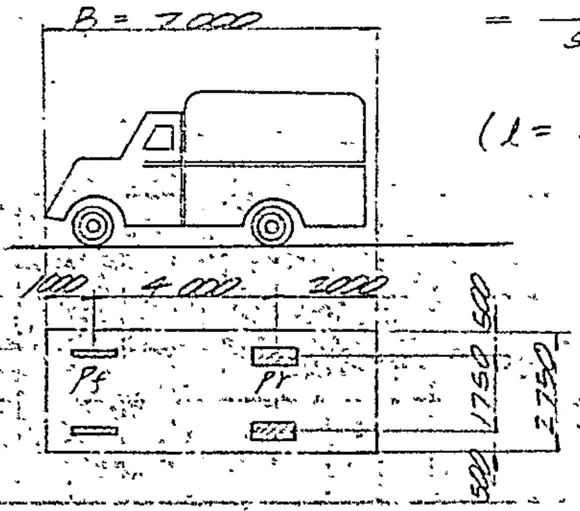
(2) Live load

Truck Load T = 14 ton

Impact coefficient $i = \frac{20}{50 - l}$

$= \frac{20}{50 - 6.9} = 0.351$

(l = Span)



Pf = 1.4 ton

Pr = 5.6 ton

2-5-1-4 Bending moment

(1) Bending moment by dead load (Md)

DEAD LOAD $w = 1.559$ t/m

$$Md = \frac{1}{8} w l^2 = \frac{1}{8} \times 1.559 \times 6.9^2$$

$$\doteq 9.278 \ t \cdot m$$

(2) Bending moment by live load (Ml)

The bending moment (Ml) at the center of bridge span caused by the live load (including impact load) 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 drive-way and walk-way are not separated.

$$Ml = (1.8 l + 0.5) \cdot 0.7 \quad (t \cdot m)$$

[l = Span ...... $2^m \leq l \leq 10^m$]

$l = 6.9m$

$\therefore M_l = (1.3 \times 6.9 - 0.5) \times 0.7$
$= 9.044$ t·m

(3) Total bending moment (ΣM)

$\Sigma M = M_d + M_l = 9.278 + 9.044$
$= 18.322$ t·m

2-5-1-5 Reactionforce

(1) Dead load

Curb load $P_c = 2.4 \times 0.25 \times 0.2 \times 2 \times \frac{1}{5.0}$
$= 0.05$ t/m

$w' = 1.559 + 0.05 = 1.609$ t/m

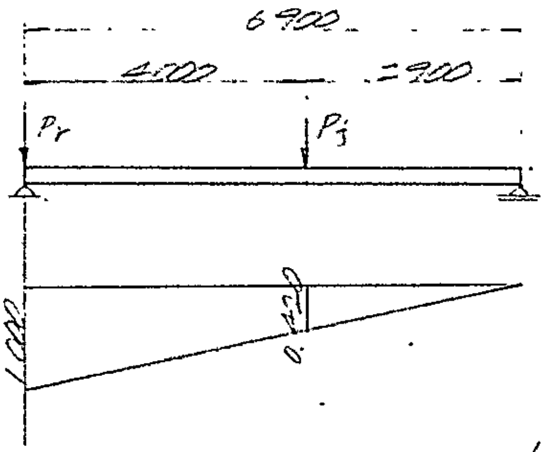
$R_d = \frac{1}{2} w' l' = \frac{1}{2} \times 1.609 \times 7.3 = 5.873^{ton}$

(l' = Bridge length)

Per One meter of width

$w_d = \frac{R_d}{B} = \frac{5.873}{5.0} = 1.175$ t/m

(2) Live load



R-Inf.

$P_r = 5.6$ ton
$P_t = 1.4$ ton

$R_l = 2(P_r \times 1.000 - P_t \times 0.420) \times (1+i)$
$= 2(5.6 \times 1.000 + 1.4 \times 0.420) \times (1+0.351)$
$= 16.730$ ton

Per one meter of width $w_l = 3.34$ t/m

(3) Total reaction force.

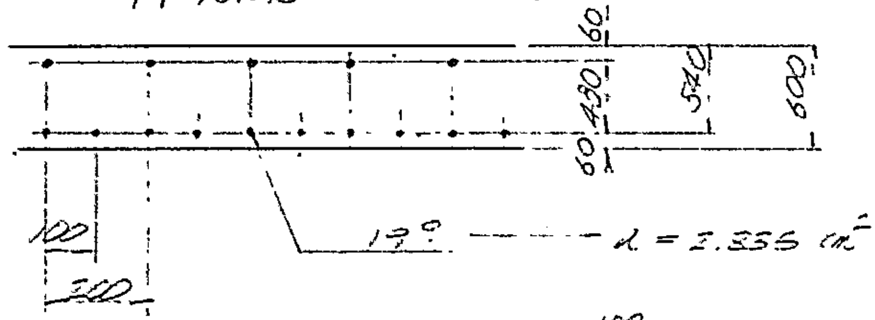
$\Sigma R = R_d + R_l = 22.593$ ton

Per one meter of width

$\frac{\Sigma R}{B} = \frac{22.593}{5.0} = 4.52$ t/m

2-5-1-6 Slab section and amount of main reinforcement;

$M = 18.673^{t\cdot m} = 1867300^{kg\cdot cm}$



$As = 2.835 \times \frac{100}{10.0} = 28.35 \ cm^2$

$As' = 2.835 \times \frac{100}{20.0} = 14.175 \ cm^2$

$$x = -\frac{n(As+As')}{b} + \sqrt{\left\{\frac{n(As+As')}{b}\right\}^2 + \frac{2n}{b}(dAs + d'As')}$$

$$= -\frac{15(28.35+14.175)}{100} + \sqrt{\left\{\frac{15(28.35+14.175)}{100}\right\}^2 + \frac{2\times 15}{100}(54\times 28.35 + 6\times 14.175)}$$

$$= 16.544 \ cm$$

$$Kc = \frac{bx}{2}\left(d - \frac{x}{3}\right) + nAs'\frac{x-d'}{x}(d-d')$$

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

$$= 46612 \ cm^3$$

$$Ks = \frac{Kc}{n}\cdot\frac{x}{d-x} = \frac{46612}{15}\times\frac{16.544}{54-16.544} = 1372.5 \ cm^3$$

stress | | allowable

concrete $\sigma_c = \frac{M}{Kc} = \frac{1832200}{46612} = 39 \ kg/cm^2 < 40 \ kg/cm^2$ OK.

Reinforcement $\sigma_s = \frac{M}{Ks} = \frac{1832200}{1372.5} = 1,335$ 〃 < 1400 〃 OK.

2-5-1-7. Distribution of reinforcement

(1) tension side

$$a = As \cdot \alpha$$

$$As = 38.35\ cm^2$$

$$\alpha = \frac{0.6}{\sqrt{l}} \leq 0.5 \quad (l \cdot \text{Span})$$

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

$$a = 38.35 \times 0.228 = 6.48\ cm^2$$

16φ, 200 pitch $\quad a = 2.011\ cm^2$

$$a' = 2.011 \times \frac{100}{20} = \underline{10.06}\ cm^2 > 6.48\ cm^2$$

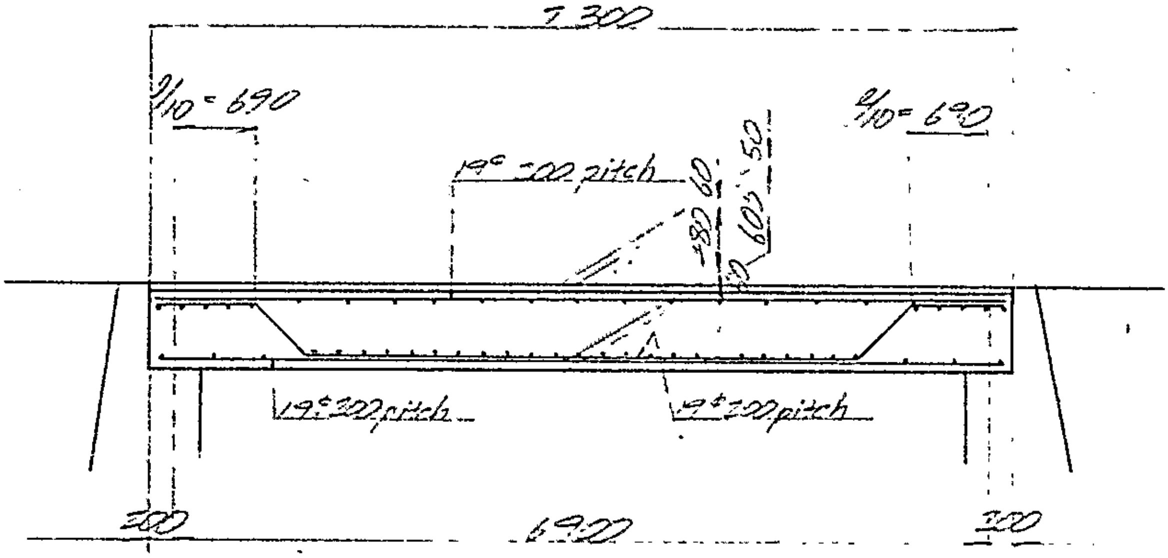
(2) compression side.

$$a = \frac{1}{3} As \cdot \alpha = \frac{1}{3} \times 6.48 = 2.16\ cm^2$$

16φ, 400 pitch $\quad a = 2.011\ cm^2$

$$a' = 2.011 \times \frac{100}{40} = 5.03\ cm^2 > 2.16\ cm^2$$

(3) Reinforcement



2-5-2 No.5 Bridge

2-5-2-1 Design criteria.

Type : Reinforced Concrete Bridge (Simple Slab)

Bridge Length : 4ᵐ800

Width : 5ᵐ000
(Effective 4ᵐ500)

Span : 4ᵐ400

Live Load : Truck Load T=14 ton

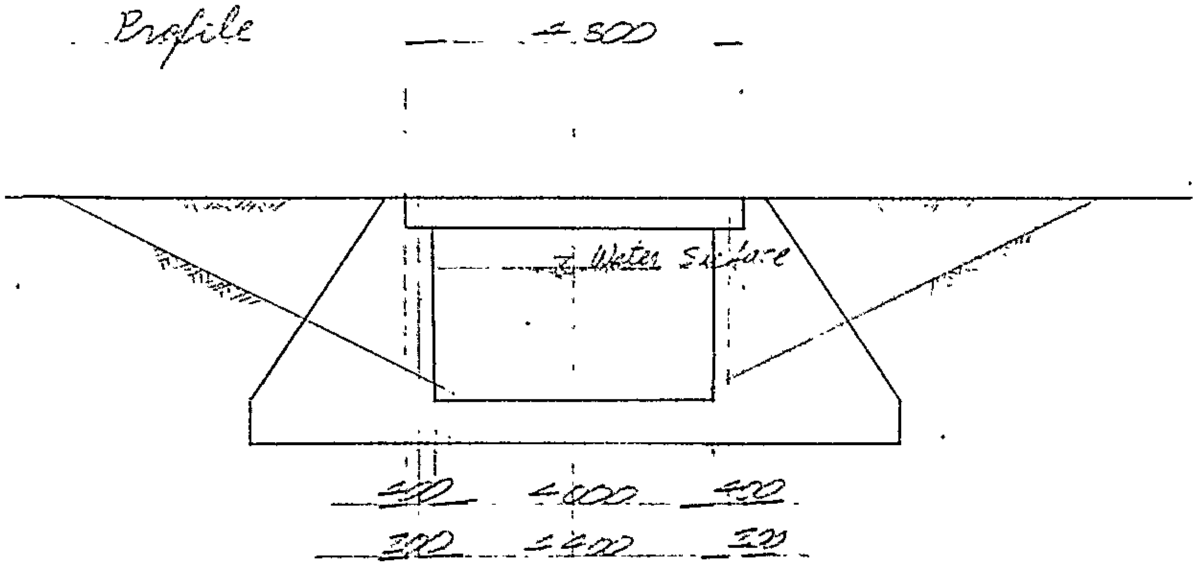
Allowable stress for compression of concrete, $\sigma_{ca} = 40$ kg/cm²

Allowable stress for tension of reinforcement, $\sigma_{sa} = 1400$ "

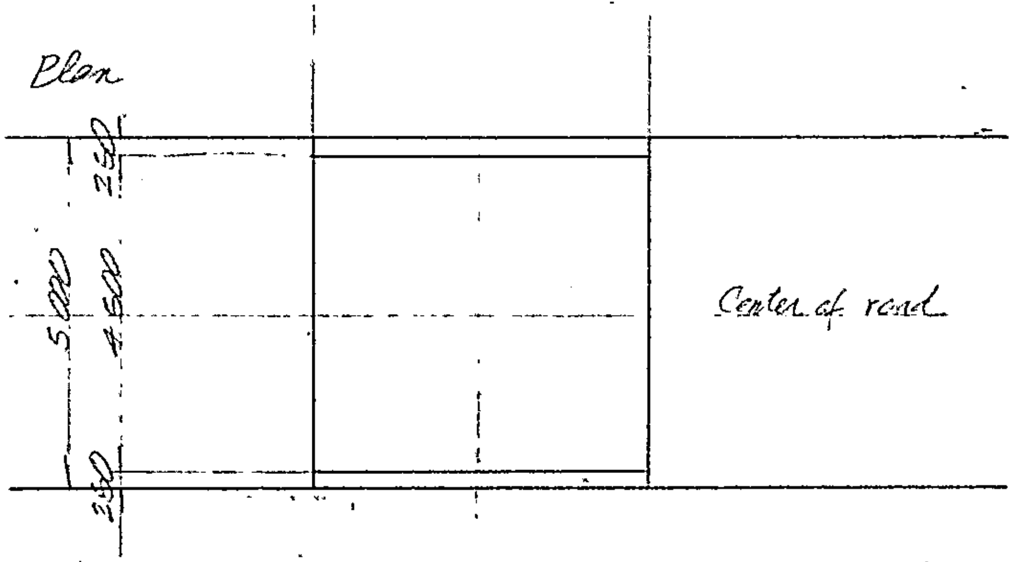
unit weight.
- Concrete. $\gamma_c = 2.3$ t/m³
- Reinforced concrete $\gamma_r = 2.4$ t/m³

2-5-2-2 Given dimension

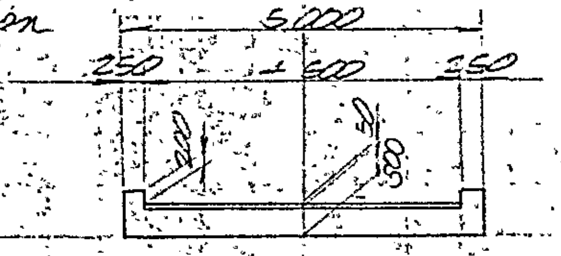
Profile



Plan

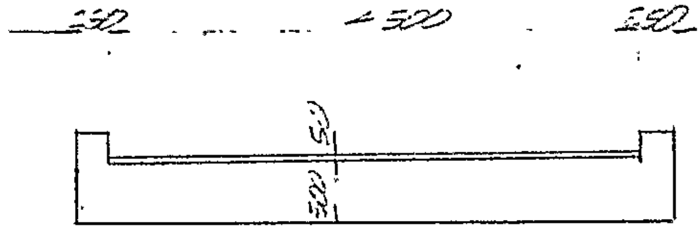


Cross section



2-5-2-3 Load

(1) Dead load



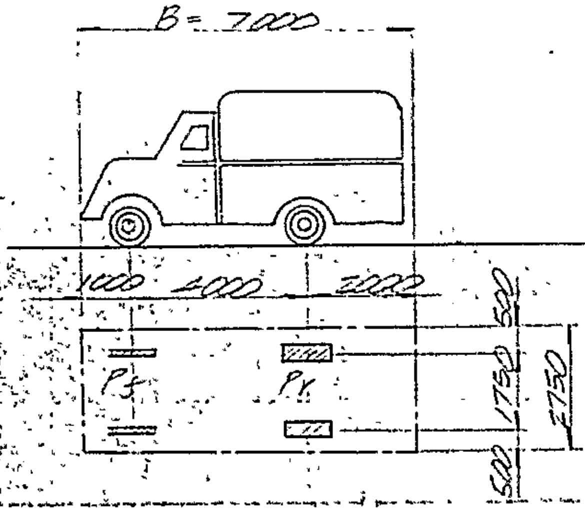
(Per one meter of width)

Pavement (Concrete)	2.3×0.05	$= 0.115$
Slab (R-Concrete)	2.4×0.5	$= 1.200$
	ΣW	$= 1.315$

(2) Live load

Truck Load $T = 14$ ton

Impact coefficient $i = \frac{20}{50 + 4.4} = 0.368$



$Pr = 5.6$ ton

$Pf = 1.4$ ton

2-5-2-4. Bending moment.

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

DEAD LOAD $w = 1.315 \ t/m$

$$Md = \frac{1}{8} w l^2 = \frac{1}{8} \times 1.315 \times 4.4^2 = 3.182 \ t \cdot m$$

(2) Bending moment by live load (Ml)

The bending moment (Ml) at the Center of bridge span caused by the live load (including impact load) 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 drive-way and walk-way are not separated.

$$Ml = (1.8 l + 0.5) \times 0.7 \quad (t \cdot m)$$

$$(l = \text{Span})$$

$l = 4.4\,m$

$Ml = (1.8 \times 4.4 - 0.5) \times 0.7 = 5.894$

(13) Total bending moment (ΣM)

$\Sigma M = Md + Ml = 3.182 + 5.894 = 9.076\ t \cdot m$

2-5-2-5 Reaction.

(1) Dead load

Curb Load $\quad Pc = 2.4 \times 0.25 \times 0.2 \times 2 \times \frac{1}{5.0}$
$= 0.048\ t/m$

$W' = 1.315 + 0.048 = 1.363\ t/m$

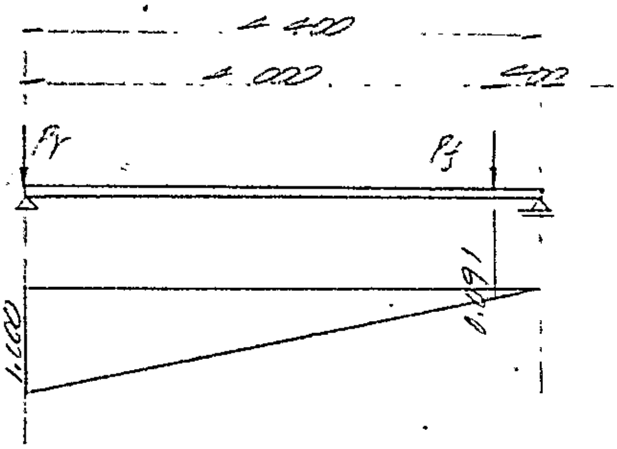
$Rd = \frac{1}{2} W' l' = \frac{1}{2} \times 1.363 \times 4.8 = 3.271\ ton$

(l' = Bridge length)

Per one meter of width

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

(2) Live load.



$Pr = 5.6$ ton

$Pt = 1.4$ ton

$$Rl = 2(Pr \times 1.000 + Pt \times 0.091) \times (1+i)$$
$$= 2(5.6 \times 1.000 + 1.4 \times 0.091) \times (1+0.368)$$
$$= 15.670 \text{ ton}$$

per one meter of width $Wl = \frac{15.670}{5.0} = 3.13$ t/m

(3) Total reaction force

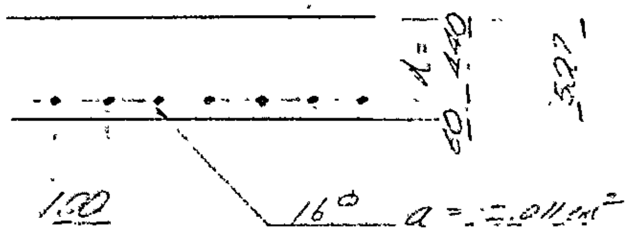
$$\Sigma R = Rd + Rl = 18.887 \text{ Ton}$$

Per one meter of width

$$\frac{\Sigma R}{B} = \frac{18.887}{5.0} = 3.777 \text{ t/m}$$

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

$$M = 9.076\ t\cdot m = 907600\ kg\cdot cm$$



$$A_s = 2.011 \times \frac{100.0}{10.0} = 20.11\ cm^2$$

$$p = \frac{A_s}{b\,d} = \frac{20.11}{100 \times 44.0} = 0.0046$$

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

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

$$= 0.309$$

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

$$\sigma_c = \frac{2M}{k\,j\,b\,d^2} = \frac{2 \times 907600}{0.309 \times 0.897 \times 100 \times 44^2}$$

$$= 33.8\ kg/cm^2 < 40\ kg/cm^2 \quad OK.$$

$$\sigma_s = \frac{M}{A_s\,j\,d} = \frac{907600}{20.11 \times 0.897 \times 44}$$

$$= 1143\ kg/cm^2 < 1400\ kg/cm^2 \quad OK.$$

2-5-2-7 Distribution of reinforcement.

(1) Tension side

$a = As \cdot \alpha$

$As = 20.11\ cm^2$

$\alpha = \frac{0.6}{\sqrt{l}} \leqq 0.5 \quad (l ;\ Span)$

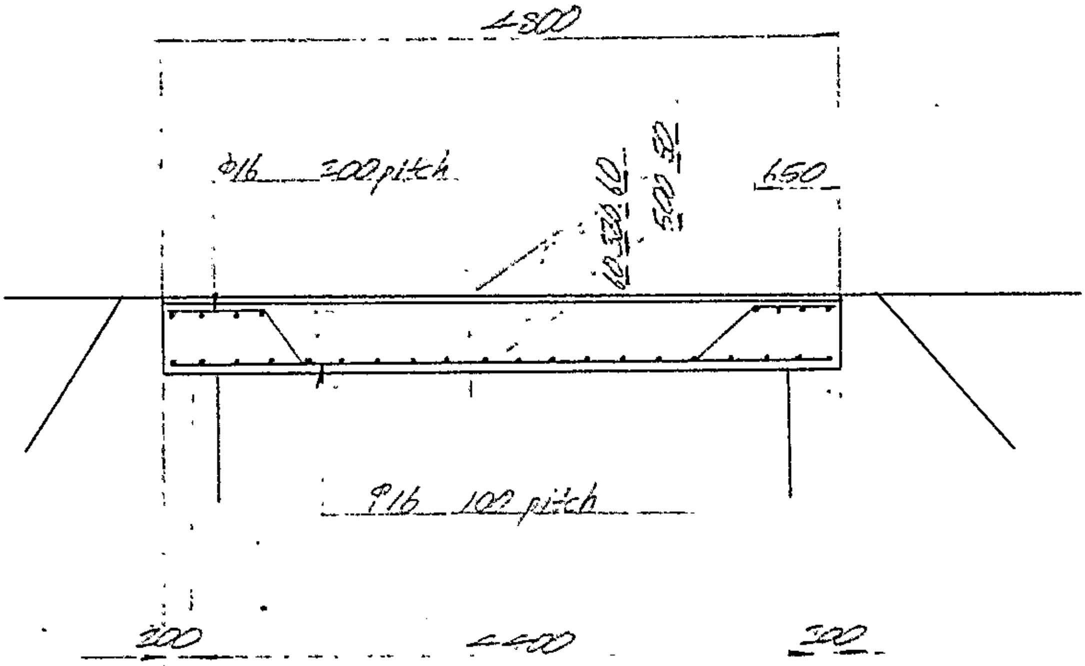
$= \frac{0.6}{\sqrt{4.4}} = 0.286 < 0.5$

$a = 20.11 \times 0.286 = 5.8\ cm^2$

16φ 250 pitch ——— $a = 2.011\ cm^2$

$a' = 2.011 \times \frac{100}{25} = 8.0\ cm^2 > 5.8\ cm^2$

(7) Reinforcement



2-5-3 NO-9 Bridge

2-5-3-1 Design criteria

Type ; Reinforced Concrete Bridge (Single slab)

Bridge Length ; 3m300

Width ; 5m000
(Effective ; 4m500)

Span ; 2m900

Live Load ; Truck Load T=14 Ton

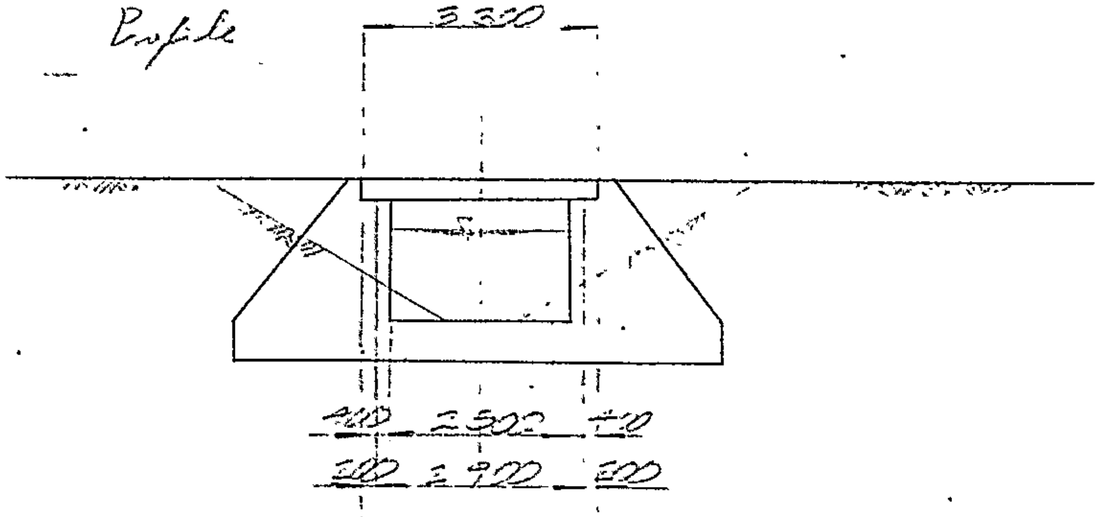
Allowable stress for compression of concrete ; σ_{ca} = 40 kg/cm²

Allowable stress for tension of reinforcement ; σ_{sa} = 1400 "

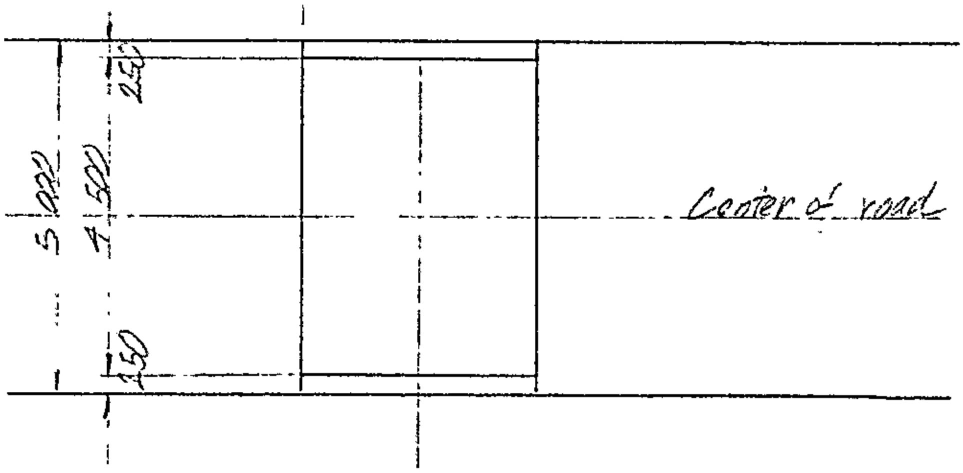
unit weight. | Concrete γ_c = 2.3 t/m³
| Reinforced concrete γ_r = 2.4 t/m³

2-5-3-2 Given dimension.

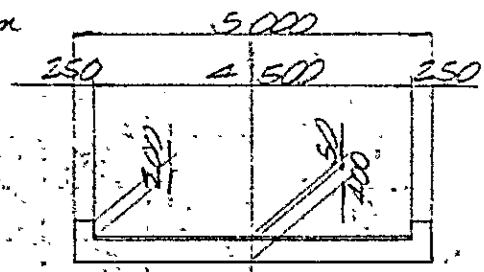
Profile



Plan

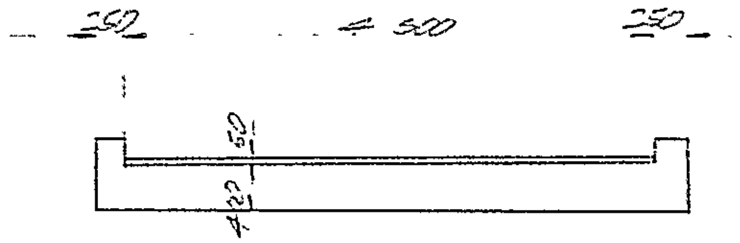


Cross section



2-5-3-3 Load

(1) Dead load



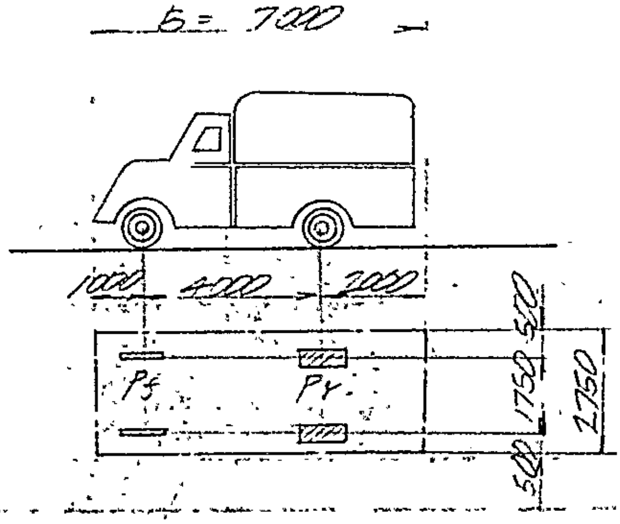
(per one meter of width)

Pavement (concrete)	$2.3 \times 0.05 =$	0.115
Slab (R-concrete)	$2.4 \times 0.40 =$	0.960
	$\Sigma W =$	1.075 t/m

(2) Live load

Truck Load $T = 14$ ton

Impact coefficient $i = \frac{20}{50 + 2.9} = 1.378$



$P_r = 5.6$ ton

$P_f = 1.4$ ton

2-5-3-4 Bending moment

(1) Bending moment by dead load (Md)

DEAD LOAD $w = 1.075$ t/m

$$Md = \frac{1}{8}wl^2 = \frac{1}{8} \times 1.075 \times 2.9^2 = 1.130 \text{ t·m}$$

(2) Bending moment by live load (Ml)

The bending moment (Ml) at the center of bridge span caused by the live load (including inpact load) 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 drive-way and walk-way are not separated.

$$Ml = (1.8l + 0.5) \times 0.7$$

(l: span)

$l = 2.9\ m$

$\therefore M_L = (1.8 \times 2.9 + 0.5) \times 0.7 = 4.004\ t \cdot m$

(3) Total bending moment (ΣM).

$\Sigma M = M_d + M_L = 1.130 + 4.004 = 5.134\ t \cdot m$

2-5-3-5 Reaction force

(1) Dead load.

Curb Load $\quad P_C = 2.4 \times 0.25 \times 0.2 \times 2 \times \frac{1}{5.0}$
$= 0.048\ t/m$

$w' = 1.075 + 0.048 = 1.123\ t/m$

$R_d = \frac{1}{2} w' l' = \frac{1}{2} \times 1.123 \times 3.3 = 1.853\ ton$

(l' = Bridge length)

Per one meter of width

$W_d = \frac{R_d}{B} = \frac{1.853}{5.0} = 0.371\ t/m$

(2) Live load.

$P_r = 5.6^{ton}$

$R_l = 5.6 \times 2 \times (1 + 0.378)$

$\doteq 15.437^{ton}$

Per one meter of width $W_l = \frac{15.437}{5.0} = 3.09\ t/m$

(3) Total reaction force

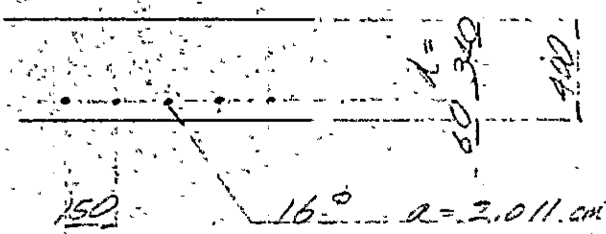
$\Sigma R = R_d + R_l = 17.287^{ton}$

Per one meter of width

$\frac{\Sigma R}{B} = \frac{17.287}{5.0} = 3.46\ t/m$

2-5-3-6 Slab section and amount of main reinforcement

$M = 5.134\ t\cdot m = 513400\ kg\cdot cm$



$\therefore\ As = 2.011 \times \frac{100.0}{15.0} = 13.407\ cm^2$

$p = \frac{As}{bd} = \frac{13.407}{100 \times 34} = 0.0039$

$k = \sqrt{2np - (np)^2} - np = \sqrt{2 \times 15 \times 0.0039 - (15 \times 0.0039)^2} - 15 \times 0.0039$
$= 0.289$

$j = 1 - \frac{k}{3} = 1 - \frac{0.289}{3} = 0.904$

$\sigma_c = \frac{2M}{kj\,bd^2} = \frac{2 \times 513400}{0.289 \times 0.904 \times 100 \times 34^2}$
$= 34\ kg/cm^2 < 70\ kg/cm^2$ ok.

$\sigma_s = \frac{M}{As \cdot j \cdot d} = \frac{513400}{13.407 \times 0.904 \times 34}$

$= 1246\ kg/cm^2 < 1400\ kg/cm^2$ OK

2-8-3-7 Distribution of reinforcement.

(1) Tension side.

$a = As \cdot \alpha$

$As = 13.407\ cm^2$

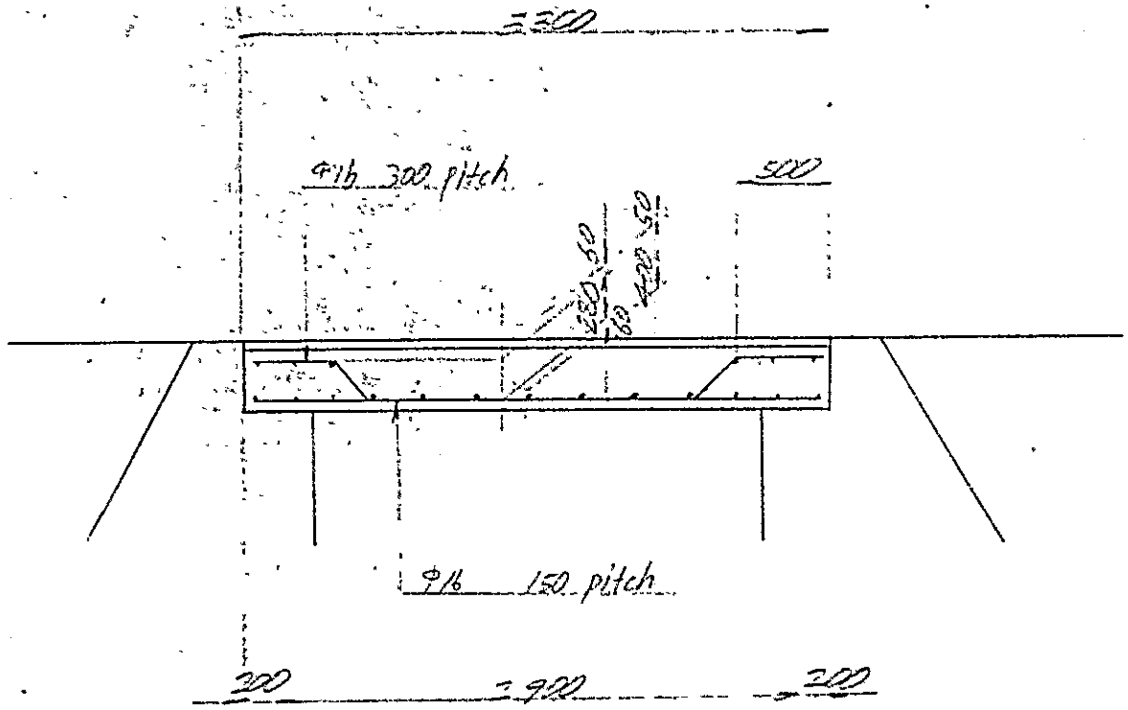
$\alpha = \frac{0.6}{\sqrt{2.9}} = 0.352$

$a = 13.407 \times 0.352 = 4.7\ cm^2$

16^ϕ 300 pitch $\quad a = 2.011\ cm^2$

$a' = 2.011 \times \frac{100}{30} = 6.7\ cm^2 > 4.7\ cm^2$

(2) Reinforcement



2-5-4 Wooden Bridge

2-5-2-1 Design criteria

Type ; Wooden simple beam bridge

Bridge length ; 7 m 300

Width ; 5 m 000 (Effective 4 m 500)

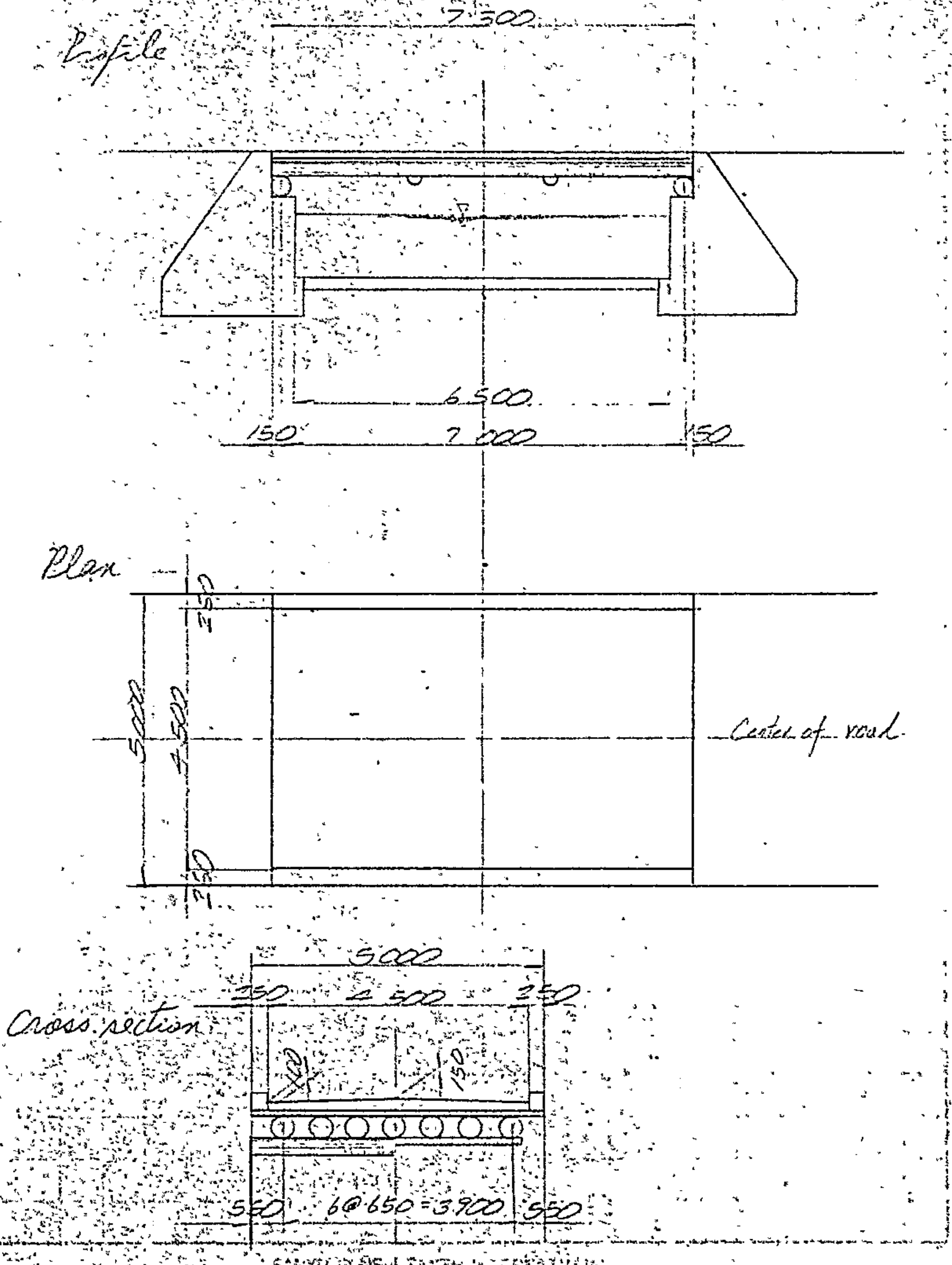
Span ; 7 m 000

Live load ; Truck load $T = 14$ ton

Allowable stress for tension ; Wood $\sigma_{wa} = 120.$ kg/cm^2

unit weight ; Wood $\gamma_w = 0.8$ t/m^3

2-5-A-2 General view



2-5-4-3 Load and bending moment

(1) Dead load

(Per one meter of width)

Curb	$0.8 \times 0.25 \times 0.25 \times 2 \,/\, 5.0 =$	0.020
Wet soil	$1.8 \times 0.125 \times 4.5 \,/\, 5.0 =$	0.203
Board	$0.8 \times 0.09 \times 5.0 \,/\, 5.0 =$	0.072
Girder	$0.8 \times \pi \times 0.2^2 \times 7 \,/\, 5.0 =$	0.141
		0.436

(2) Live load

Truck load $T = 14^{ton}$

$\left(\begin{array}{l} P_f = 1.4 \text{ ton} \\ P_r = 5.6 \text{ ''} \end{array} \right.$

Impact coefficient $i = \dfrac{20}{50+7} = 0.351$

(3) Bending moment

$$Md = \frac{1}{8} \times 0.436 \times 7^2 = 2.671^{\,t \cdot m}$$

$$Ml = (1.8 \times 7 + 0.5) \times 0.7 = 9.170$$

$$\Sigma M = 11.841^{\,t \cdot m}$$

2-5-4-4 Section of main girder

$M = 11.841 \ t \cdot m = 1184100 \ kg \cdot cm$

seven woods of φ450 are used for Bridge's width of 5.00M,

$$I = \frac{\pi \cdot 45^4}{64} \cdot \frac{7}{5} = 281.662 \ cm^4$$

$$\sigma_w = \frac{M}{I} \cdot y = \frac{1184100}{281662} \times \frac{45}{2} = 94.5 \ kg/cm^2 < 120 \ kg/cm^2$$

OK