GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION, WATER DEVELOPMENT AND FLOOD CONTROL
BANGLADESH WATER DEVELOPMENT BOARD

DHAKA INTEGRATED FLOOD PROTECTION PROJECT

MID-TERM CONSULTANCY SERVICES

ADB LOAN - 1124-BAN (SF)

FINAL REPORT

VOLUME - II

ANNEXURE - II : DESIGN OF SLUICES, PROTECTIVE WORKS OF EMBANKMENT, COVERED DRAINS AND FLOOD WALLS

TECHNOCONSULT INTERNATIONAL LIMITED, BANGLADESH
in association with
ASSOCIATED CONSULTING ENGINEERS LIMITED, BANGLADESH
DESH UPODESH LIMITED, BANGLADESH
and
Individual Consultants from LOUIS BERGER INTERNATIONAL, INC., USA

MAY 1993

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ii) HAZARIBAGH KHAL
iii) NAWABGANJ KHAL AND
vi) SHAHIDNAGAR KHAL

ON WESTERN EMBANKMENT

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DHAKA INTEGRATED FLOOD PROTECTION PROJECT

1. HYDROLOGICAL ANALYSIS OF (i) RAYER BAZAR, (ii) HAZARIBAGH KHAL, (iii) NAWABGANJ KHAL AND (iv) SHAHID BAGH DRAINAGE SLUICES.

AREA :

South-west fringe of the city bounded by the embankment from kellar morh to Satmasjid as south-west and western boundary and roads and high ridges from Kellar morth to Satmasjid along Azimpur Road and Satmasjid Road as eastern boundary.

Drainage basin : The area under consideration measures about 5.72 Km^2. Runoff from the area drains into Bachila khal and Buriganga river through 4 major drainage channels. The catchment area of all these 4 sub-basin, as per topography, are as follows:

S-6	: 1.64 Km^2	= 164.00 ha	=	405.25 ac.
S-7	: 2.60 Km^2	= 260.00 ha	=	642.46 ac.
S-8	: 0.52 Km^2	= 52.00 ha	=	128.50 ac.
S-9	: 0.96 Km^2	= 96.00 ha	=	237.20 ac.

DATA :

Rainfall pattern/distribution and runoff model are taken from JICA report "Study on Storm Water Drainage System Improvement of Dhaka City".

In the report, design discharge for drainage pipes and khal improvements are estimated by the rational formula. Considering inflow time of 20 mins.

Short duration intensive rainfall generates high runoff. In the study report of JICA, it was established that duration of heavy rainfalls is six hours.

Accordingly JICA, proposed 2 day- 5 yr. rainfall for pumping stations, and rainfalls of every short duration of 5 yr. frequency for drainage pipes and khals depending on time of inflow in short reaches.

For determing the runoff to the proposed regulators, 1 day rainfall occurring in six hours with a return period of 2 yr. is taken as the basis and the capacity of regulators is again verified with same rainfall of 5 yr. return period.

One day rainfall in the study area, established in the JICA report is:

2 yr. return period = 134.8 mm

5 yr. return period = 191.6 mm

Since duration of heavy rainfall of high intensity in the project area is six hours, so one day rainfall is assumed to occur in 6 (six) hours. The distribution of hourly rainfall is:

1st hr.	= 9%
2nd hr.	= 15%
3rd hr.	= 44%
4th hr.	= 16%
5th hr.	= 9%
6th hr.	= 7%

For maximum runoff, the 6 hourly rainoff is distributed as per established practice and is shown below :

	2 yr. Return Period	5 yr. Return Period
1st hr.	= 12.1 mm	17.3 mm
2nd hr.	= 20.3 mm	28.8 mm
3rd hr.	= 59.4 mm	84.5 mm
4th hr.	= 21.6 mm	30.7 mm
5th hr.	= 12.1 mm	17.3 mm
6th hr.	= 9.5 mm	13.4 mm
	135.0 mm	192.0 mm

Basic assumptions and considerations for calculation of runoff volume from the rainfall intensity are :

1. No loss of precipitation due to evaporation and percolation in rainfall of short duration because of city area.

2. Rational formula, $Q = \frac{1}{3.6} \cdot f.A.R.$, as suggested in JICA report for calculating runoff is used with f=0.6, considering the future development of the area as middle class residential area.

3. The topography of the existing catchment area is not known. Also the whole area will be developed in near future, so storage of runoff is not assumed.

Assumed distribution of rainfall, produces peak runoff in 3rd hr., 4th & 2nd hr. rainfall produces next higher peaks. So runoff from all the basins for 2nd, 3rd and 4th hr. rainfall is calculated to assess the capacity of drainage outlets.

The suggested formula is

$$Q = \frac{1}{3.6} \cdot f.A.R.$$

Where. Q = Runoff (m^3/s)

f = Runoff Co-eff. = 0.6. for middle & low class residential area

A = Drainage Area (Km^2)

R = Rainfall intensity (mm/hr.)

<u>Runoff from the Basins</u> :

Basin : S-6
Area = 1.64 Km^2

2nd yr. ret. period :

Q (2nd hr.) = 0.1667 x 1.64 x 20.3 m^3/sec. = 5.55 m^3/sec.
Q (3rd hr.) = 0.27339 x 59.4 m^3/sec. = 16.24 m^3/sec.
Q (4th hr.) = 0.27339 x 21.6 m^3/sec. = 5.90 m^3/sec.

5 yr. ret. period :

Q (2nd hr.) = 0.27339 x 28.8 m^3/sec. = 7.87 m^3/sec.
Q (3rd hr.) = 0.27339 x 84.5 m^3/sec. = 23.10 m^3/sec.
Q (4th hr.) = 0.27339 x 30.7 m^3/sec. = 8.39 m^3/sec.

Basin : S-7
Area = 2.60 Km^2

2nd yr. ret. period :

Q (2nd hr.) = 0.1667 x 2.6 x 20.3 m^3/sec. = 8.80 m^3/sec.
Q (3rd hr.) = 0.4333 x 59.4 m^3/sec. = 25.70 m^3/sec.
Q (4th hr.) = 0.4333 x 21.6 m^3/sec. = 9.36 m^3/sec.

5 yr. ret. period :

Q (2nd hr.) = 0.4333 x 28.8 m^3/sec. = 12.50 m^3/sec.
Q (3rd hr.) = 0.4333 x 84.5 m^3/sec. = 36.36 m^3/sec.
Q (4th hr.) = 0.4333 x 30.7 m^3/sec. = 13.30 m^3/sec.

Basin : S-8
Area = 0.52 Km^2

2nd yr. ret. period :

Q (2nd hr.) = 0.1667 x 0.52 x 20.3 m^3/sec. = 1.76 m^3/sec.
Q (3rd hr.) = 0.08667 x 59.4 m^3/sec. = 5.15 m^3/sec.
Q (4th hr.) = 0.08667 x 21.6 m^3/sec. = 1.87 m^3/sec.

5 yr. ret. period :

Q (2nd hr.) = 0.08667 x 28.8 m^3/sec. = 2.50 m^3/sec.
Q (3rd hr.) = 0.08667 x 84.5 m^3/sec. = 7.32 m^3/sec.
Q (4th hr.) = 0.08667 x 30.7 m^3/sec. = 2.66 m^3/sec.

Basin : S-9
Area = 0.96 Km^2

2nd yr. ret. period :

Q (2nd hr.) = 0.1667 x 0.96 x 20.3 m^3/sec. = 3.25 m^3/sec.
Q (3rd hr.) = 0.160 x 59.4 m^3/sec. = 9.50 m^3/sec.
Q (4th hr.) = 0.160 x 21.6 m^3/sec. = 3.45 m^3/sec.

5 yr. ret. period :

Q (2nd hr.) = 0.160 x 28.8 m^3/sec. = 4.61 m^3/sec.
Q (3rd hr.) = 0.160 x 84.5 m^3/sec. = 13.52 m^3/sec.
Q (4th hr.) = 0.160 x 30.7 m^3/sec. = 4.91 m^3/sec.

As per assumed distribution of rainfall and runoff calculated from the rainfall, each of the 2nd, 3rd and 4th hr. peak will occur only once 24 hr. period. the other 3 hr. remaining peaks are comparatively very small and in rest 18 hrs.. there is no rainfall. In that consideration only average of the 2nd. 3rd and 4th hr. rainfall of 2 yr. return period is considered for average discharge through the outlets. with the aim to use the main drainage channel as the temporary reservoir.

The section of Approach/ Drainage Channel for different catchment area :

The Approach Channel section is designed on Manning's formula with

$n = 0.03$

s = 0.75 ft./mile = 1.42×10^{-4}.

$$Q = AV$$

$$A = (b - 1.5d)d \qquad \begin{array}{l} A = \text{Area of } X\text{-sec} \\ b = \text{Bed width}\,(m) \\ d = \text{Depth}\,(m) \\ \text{Side slope} = 1.5:1V \\ P = \text{Wetted perimeter}\,(m) \end{array}$$

$$P = b + 2d\sqrt{1 + 1.5^2}$$
$$= b + 3.605d$$
$$R = \frac{A}{P}$$

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$
$$= \frac{1}{0.03} \times R^{\frac{2}{3}} \times (1.42 \times 10^{-4})^{\frac{1}{2}}$$
$$= 0.3972\, R^{\frac{2}{3}}$$

$$Q = AV$$
$$= 0.3972 \times \left(\frac{[(b - 1.5d)d]^{\frac{5}{3}}}{[b + 3.605d]^{\frac{2}{3}}} \right) m^3/\text{sec}$$

By trial and error, section required to accommodate different quantity of flow :

with	b = 3.0 m	d = 2.5 m	Q = 8.41 m^3/sec.
		d = 3.0 m	Q = 12.37 m^3/sec.
		d = 3.5 m	Q = 17.28 m^3/sec.
		d = 4.0 m	Q = 23.20 m^3/sec.

$b = 4.0$ m	$d = 2.5$ m	$Q = 10.03$ m^3/sec.
	$d = 3.0$ m	$Q = 14.55$ m^3/sec.
	$d = 3.5$ m	$Q = 20.06$ m^3/sec.
	$d = 4.0$ m	$Q = 26.65$ m^3/sec.
$b = 6.0$ m	$d = 2.5$ m	$Q = 13.37$ m^3/sec.
	$d = 3.0$ m	$Q = 19.02$ m^3/sec.
	$d = 3.5$ m	$Q = 25.77$ m^3/sec.
	$d = 4.0$ m	$Q = 33.71$ m^3/sec.

Trial section of channel shows that. a channel having bed width of 3.0 m and flow depth ranging from 2.5 m to 4.0 m can. accommodate flow ranging from 8.41 m^3/sec. to 23.20 m^3/sec. and that with bed width of 6.0 m can accommodate flow ranging from 13.37 m^3/sec to 33.71 m^3/sec.

With. sluice invert at (+)1.0 m (PWD) and u/s basin at (+)0.5 m. PWD. (as provided in existing regulators of Dhaka City). the incoming water depth/level at out fall points are in the range of 2.5 m to 4.0 m.

So capacity of various sizes. e.g.. 1-vent. 1.5 m x 1.8 m. 2-vents. 1.5 m x 1.8 m and 1 vent. 1.2 m x 1.5 m. are investigated with incoming water depth in the range of 2.5 m to 4.5 m. and out fall water level in March to July. and October to December.

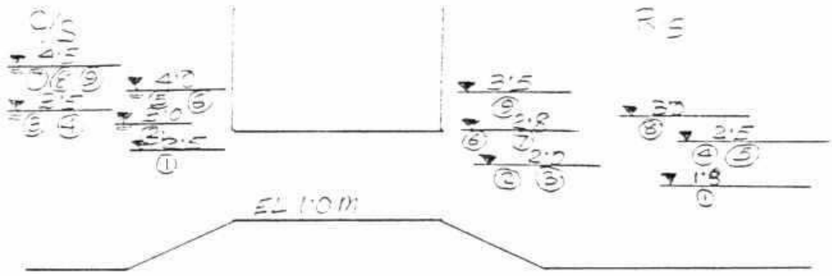
As per W.L. hydrography. water level at average year near the outfall points are :

January to March	= (+)1.8 m to (+)2.0 m (PWD)
April	= (+)2.0 m to 2.5 m (PWD)
May	= (+)2.5 m to 3.0 m (PWD)
June & July	= (+)3.0 m to 4.5 m (PWD)
October	= (+)4.5 m to 4.0 m (PWD)
November	= (+)4.0 m to 2.5 m (PWD)

2. VENT SIZE

In consideration of incoming water depth and outfall water level. the various combinations of water level at U/S and D/S of the structure is investigated to access the dimension of the outfall Regulator.

The combinations of W.L. are :



Flow through Regulators :

Size : 1 Vent - 1.5 m (H) x 1.8 m (V)

Condition : (1)

C/S W.L. = (+) 2.50 m
R/S W.L. = (+) 1.80 m

Q = 4.08 m^3/sec. Free weir flow

Condition : (2)

C/S W.L. = (+) 3.0 m
R/S W.L. = (+) 2.0 m

Q = 6.28 m^3/sec. Free weir flow

Condition : (3)

C/S W.L. = (+) 3.50 m
R/S W.L. = (+) 2.00 m

Q = 8.78 m^3/sec, Weir flow

Condition : (4)

C/S W.L. = (+) 3.50 m
R/S W.L. = (+) 2.00 m

Q = 8.78 m^3/sec, Weir flow

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Condition : (5)

C/S W.L. = (+) 4.00 m
R/S W.L. = (+) 2.50 m

Q = 10.23 m^3/sec. Submerged weir flow

Condition : (6)

C/S W.L. = (+) 4.00 m
R/S W.L. = (+) 2.80 m

Q = 10.23 m^3/sec. Submerged weir flow

Condition : (7)

C/S W.L. = (+) 4.50 m
R/S W.L. = (+) 2.80 m

Q = 11.72 m^3/sec. Submerged weir flow

Condition : (8)

C/S W.L. = (+) 4.50 m
R/S W.L. = (+) 3.00 m

Q = 12.0 m^3/sec. Orifice flow

Condition : (9)

C/S W.L. = (+) 4.50 m
R/S W.L. = (+) 3.50 m

Q = 9.56 m^3/sec Orifice flow

For various combinations of inlet and outfall water depth during pre-monsoon and post-monsoon. discharge capacity of :

1 vent - 1.5 m x 1.8 m is 4.08 m^3/sec to 12.0 m^3/sec and that of 2 vent - 1.5 m x 1.8 m is 8.15 m^3/sec to 24.0 m^3/sec.

Considering average runoff from each basin for 2nd. 3rd and 4th hour of rainfall and the discharge capacity of outlet structure with respect to incoming and outfall water depth. the size of structure as follows :

S-6 :

Catchment Area = 1.64 Km^2

$Q(av.)_2$ = 9.23 m^3/sec.   2 yr. return period

$Q(av.)_5$ = 13.12 m^3/sec.   5 yr. return period

Structure size :

1 vent - 1.5 m x 1.8 m

Q = 4.08 m^3/sec to 12.0 m^3/sec

Total runoff from the basin:

2 yr. return period (1 day rainfall)

= (3.31 + 5.55 + 16.24 + 5.90 + 2.60) x 3600 m^3 = 1.20x 10^5 m^3

5 yr. return period (1 day rainfall)

= (4.73 + 7.87 + 23.10 + 8.39 + 3.66) x 3600 m^3 = 1.72x 10^5 m^3

Selected size of the structure. can drain 1.76 x 10^5 m^3 of runoff, with Q = 4.08 m^3/sec.

S-7 :

Catchment Area = 2.60 Km^2

$Q(av.)_2$ = 14.62 m^3/sec.   2 yr. return period

$Q(av.)_5$ = 20.80 m^3/sec.   5 yr. return period

Structure size :

2 vents - 1.5 m x 1.8 m

Q = 8.15 m^3/sec to 24.0 m^3/sec

Total runoff from the basin:

2 yr. return period (1 day rainfall)

=(5.24+8.80+ 25.70 + 9.36 + 5.24 + 4.12)x3600 m^3 = 2.10x10^5 m^3

5 yr. return period (1 day rainfall)

=(7.49+12.50+36.36+13.30 + 7.49 +5.80)x3600 m^3 =2.98 x 10^5 m^3

Selected size of the structure can drain 3.52 x 10^5 m^3 of runoff, with Q = 8.15 m^3/sec.

S-8 :

Catchment Area = 0.52 Km^2

$Q(av.)_2$ = 2.93 m^3/sec.   2 yr. return period

$Q(av.)_5$ = 4.16 m^3/sec.   5 yr. return period

Required structure size :

1 vent - 1.2 m x 1.5 m

S-9 :

Catchment Area = 0.96 Km^2

$Q(av.)_2$ = 5.40 m^3/sec.   2 yr. return period

$Q(av.)_5$ = 7.68 m^3/sec.   5 yr. return period

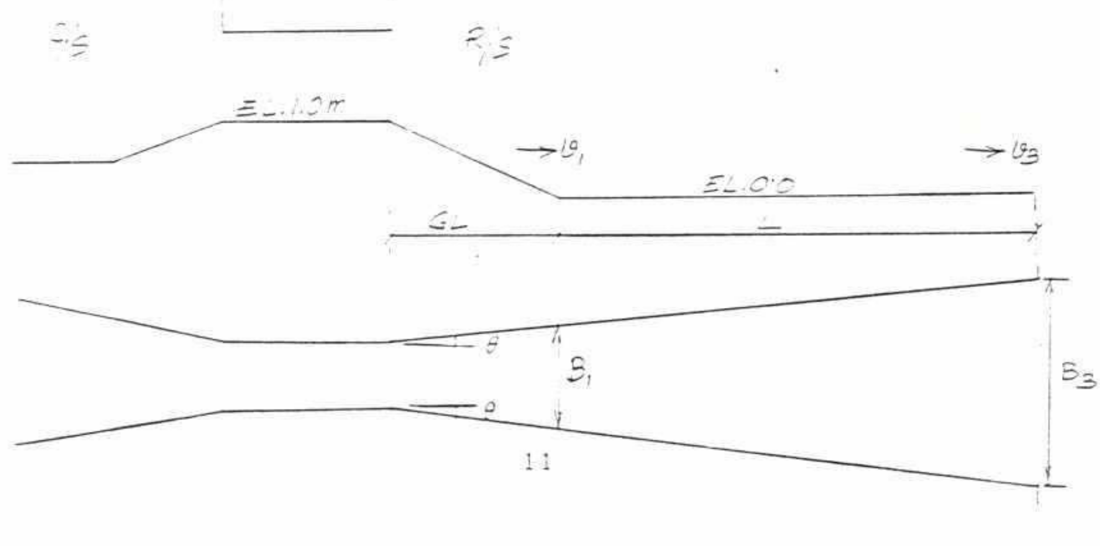
Required structure size :

1 vent - 1.4 m x 1.5 m

In consideration of cleaning of long barrel portion. and also possibilities of solid wastes to be carried by storm runoff of the locality. minimum vent size suggested is 1.5 m x 1.8 m. So. for basins. S-6. S-8 & S-9. One vent 1.5 m x 1.8 m is suggested.

3. BASIN LENGTH

1 vent. 1.5 m x 1.8 m
River side basin



A. C/S WL = (+) 3.50 m
R/S WL = (+) 2.00 m
Invert at = (+) 1.00 m
R/S Floor level = 0.00 m

$Q = 8.78\ m^3/sec$ $\theta = 5^0$
$d_c = 1.52\ m$ $G.L = 3.0\ m$
$v_1 = 7.22\ m/sec$ $B_1 = 2.02\ m$
$d_1 = 0.60\ m$ $d_2 = 2.24\ m$
$F_1 = 2.98\ m$

According to Indian Standard Stilling Basin-I :

$L = 9.0\ m$ and $B_3 = 3.60\ m.$ $V_3 = 0.66\ m/sec$

B. With C/S WL = (+) 4.0 m
R/S WL = (+) 2.5 m

$Q = 10.23\ m^3/sec$
$d_c = 1.68\ m$
$v_1 = 7.43\ m/sec$
$d_1 = 0.68\ m$ $d_2 = 2.45\ m$
$F_1 = 2.88\ m$

According to Indian Standard Stilling Basin-I :

$L = 9.0\ m$ and $B_3 = 3.60\ m.$ $V_3 = 0.56\ m/sec$

Considering the major probabilities of weir flow through the structure the basin details as per detail 'A' is selected.

4. <u>DOWN-STREAM SCOUR DEPTH (1-VENT)</u>

$$Q = 10.23 m^3/sec \qquad B_3 = 3.6m$$
$$q_3 = 2.84 m^3/sec \qquad d_m = 0.03mm$$

$$R = 1.35\left(\frac{q_3^2}{f}\right)^{\frac{2}{3}} \qquad f = 1.76\sqrt{d_m} = 0.03$$
$$= 4.04m$$

D/S scour depth = 1.5 X 4.04 = 6.06 M
D/S scour level = (+) 2.5 - 6.06 = (-) 3.56 m
D/S cutoff depth = 0 - 3.56 = 3.56 m. provided 4.0 m

5. <u>BASIN LENGTH (2 VENTS)</u>

2-VENTS. 1.5 M x 1.8 m
Pier width = 0.5 m

RIVER SIDE BASIN

A. C/S WL = (+) 3.50 m
R/S WL = (+2) 2.00 m
Invert at = (+) 1.00 m
R/S Apron = 0.00 m

$Q = 17.55\ m^3/sec$ $\theta = 8.5^0$
$d_c = 1.52$ m
$d_1 = 0.55$ m $d_2 = 2.17$ m
$v_1 = 7.26$ m/sec TWD = 2.0 m
$F_1 = 3.12$ m

Accordingly to Indian Standard Stilling Basin-I :

L = 9.0 m and $B_3 = 7.10$ m. $V_3 = 0.87$ m/sec

B. C/S WL = (+) 4.0 m
R/S WL = (+) 2.5 m

$Q = 20.47\ m^3/sec$
$d_c = 1.68$ m
$d_1 = 0.62$ m $d_2 = 2.38$ m
$v_1 = 7.51$ m/sec TWD = 2.50 m
$F_1 = 3.04$ m

Accordingly to Indian Standard Stilling Basin-I :

L = length of floor = 9.0 m. $B_3 = 7.10$ m, $V_3 = 0.75$ m/sec

Considering major probabilities of weir flow through the structure, basin details as per 'A' is selected.

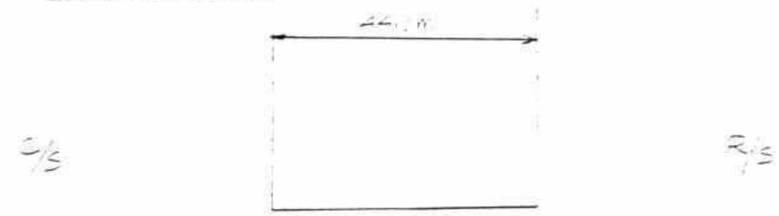
6. DOWN-STREAM SCOUR DEPTH (2-VENTS)

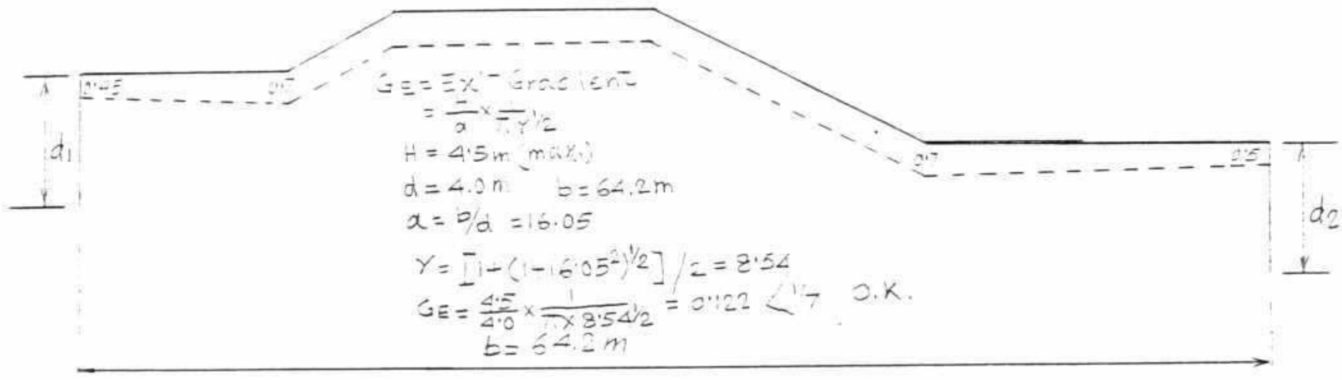
$$Q = 20.47 m^3/sec \qquad B_3 = 7.1 m$$
$$q_3 = 2.88 m^3/sec \qquad TWD = 2.5 m$$

$$R = 1.35\left(\frac{q_3^2}{f}\right)^{\frac{2}{3}} \qquad f = 0.3$$
$$= 4.08 m$$

D/S scour depth = 1.5 X 4.08 = 6.12 m
D/S scour level = (+) 2.5 - 6.12 = (-) 3.62 m PWD
D/S cutoff depth = 0 - (-) 3.62 = 3.62 m, provided 4.0 m

7. EXIT GRADIENT





8. UPLIFT PRESSURE :

Drainage mode : $H = 4.5 - 2.0 = 2.5\,m$

b = 64.2 m
d_1 = 4.0 m
d_2 = 4.0 m

Section	Length from U/S	Uplift Pressure (KN/m^2)
1. (U/S)	0.0	19.13
2.	6.7	17.66
3.	8.2	17.33
4.	52.2	8.10
5.	55.2	7.30
6. (D/S)	64.2	5.40

Flushing mode : no flow

H = 8.50 - 5.0 m = 3.50 m

Section	Length from U/S Section(m)	Uplift Pressure (KN/m^2)
1. (U/S)	0.00	7.60
2.	6.70	9.60
3.	8.20	9.90
4.	52.20	23.00
5.	55.20	23.70
6. (D/S)	64.20	26.80

Imposed load from structure is higher than the uplift. only structural thickness is provided.

9. DESIGN CRITERIA AND CONSIDERATIONS (BARREL)

Embankment Crest Level	-->	9.00 m
Top slab Level	-->	3.25 m
Maximum Water Level	-->	3.50 m
Live Vehicle Loading	-->	H20
Unit Weight of Steel	-->	77.00 KN m^3
Unit Weight of Concrete	-->	23.60 KN m^3
Unit Weight of Soil	-->	18.80 KN m^3
Unit Weight of Water	-->	9.81 KN m^3
Angle of Internal Friction (ø)	-->	25^0

Coeff. of Earth Pressure at rest (Ca) = (1-sinø)

Top Slab Thickness	-->	450 mm
Bottom Slab Thickness	-->	500 mm
Abutment Top Thickness	-->	450 mm
Abutment Bottom Thickness	-->	500 mm
Pier Thickness	-->	500 mm
Inside Height of Barrel	-->	1.80 m
Inside Width of Barrel	-->	1.50 m
Length of Barrel	-->	40.90 m
Barrel Ext. Length (C/S)	-->	1.40 m
Barrel Ext. Length (C/S)	-->	1.70 m

10. STRUCTURAL ANALYSIS OF BARREL (1-VENT)

TOP SLAB

Design Moment	M =	35.73	KNm
Reqd. depth	dr(mom) =	167	mm
Design Shear	V =	116.31	KN
Reqd. depth	dr(shear) =	307	mm
Reqd. Thickness	tr =	367	mm
Provided Thickness	ta =	450	mm

SHRINKAGE REINF.

Ast (reqd. exp. face)	=	760	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	250	mm
Ast (Actual)	=	804	mm^2

TOP REINF.

Ast (mom.)	=	845	mm^2
Ast (shrinkage)	=	380	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	225	mm
Ast (Actual)	=	894	mm^2

BOTTOM REINF.

Ast (mom.)	=	496	mm^2

Ast (shrinkage)	=	760	mm^2
Ast (provided)			
Bar dia	(ø) =	12	mm
Bar spacing c/c	=	200	mm
Ast (Actual)	=	565	mm^2

BOTTOM SLAB
===========

Design Moment	M =	41.68	KNm
Reqd. depth	dr(mom) =	180	mm
Design Shear	V =	135.44	KN
Reqd. depth	dr(shear) =	357	mm
Reqd. Thickness	tr =	417	mm
Provided Thickness	ta =	400	mm

SHRINKAGE REINF.

Ast (reqd. exp. face)	=	760	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	250	mm
Ast (Actual)	=	804	mm^2
Ast (reqd. earth face)	=	370	mm^2
Ast (provided)			
Bar dia	(ø) =	12	mm
Bar spacing c/c	=	200	mm
Ast (Actual)	=	565	mm^2

TOP REINF.

Ast (mom.)	=	546	mm^2
Ast (shrinkage)	=	570	mm^2
Ast (provided)			
Bar dia	(ø) =	12	mm
Bar spacing c/c	=	200	mm
Ast (Actual)	=	565	mm^2

BOTTOM REINF.

Ast (mom.)	=	874	mm^2
Ast (shrinkage)	=	380	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	225	mm
Ast (Actual)	=	894	mm^2

ABUTMENT (SIDE WALL)

Design Moment (Top)	M =	35.73	KNm
Reqd. depth	dr(mom) =	143	mm
Design Shear (Top)	V =	88.41	KN
Reqd. depth	dr(shear) =	123	mm
Reqd. Thickness	tr =	293	mm
Provided Thickness	ta =	450	mm

Design Moment (Bottom)	M =	41.68	KNm
Reqd. depth	dr(mom) =	180	mm
Design Shear (Bottom)	V =	103.06	KN
Reqd. depth	dr(shear) =	272	mm
Reqd. Thickness	tr =	332	mm
Provided Thickness	ta =	500	mm

SHRINKAGE REINF.

Ast (reqd. exp. face)	=	760	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	250	mm
Ast (Actual)	=	804	mm^2

Ast (reqd. earth face)	=	874	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	225	mm
Ast (Actual)	=	894	mm^2

EXPOSED FACE REINF

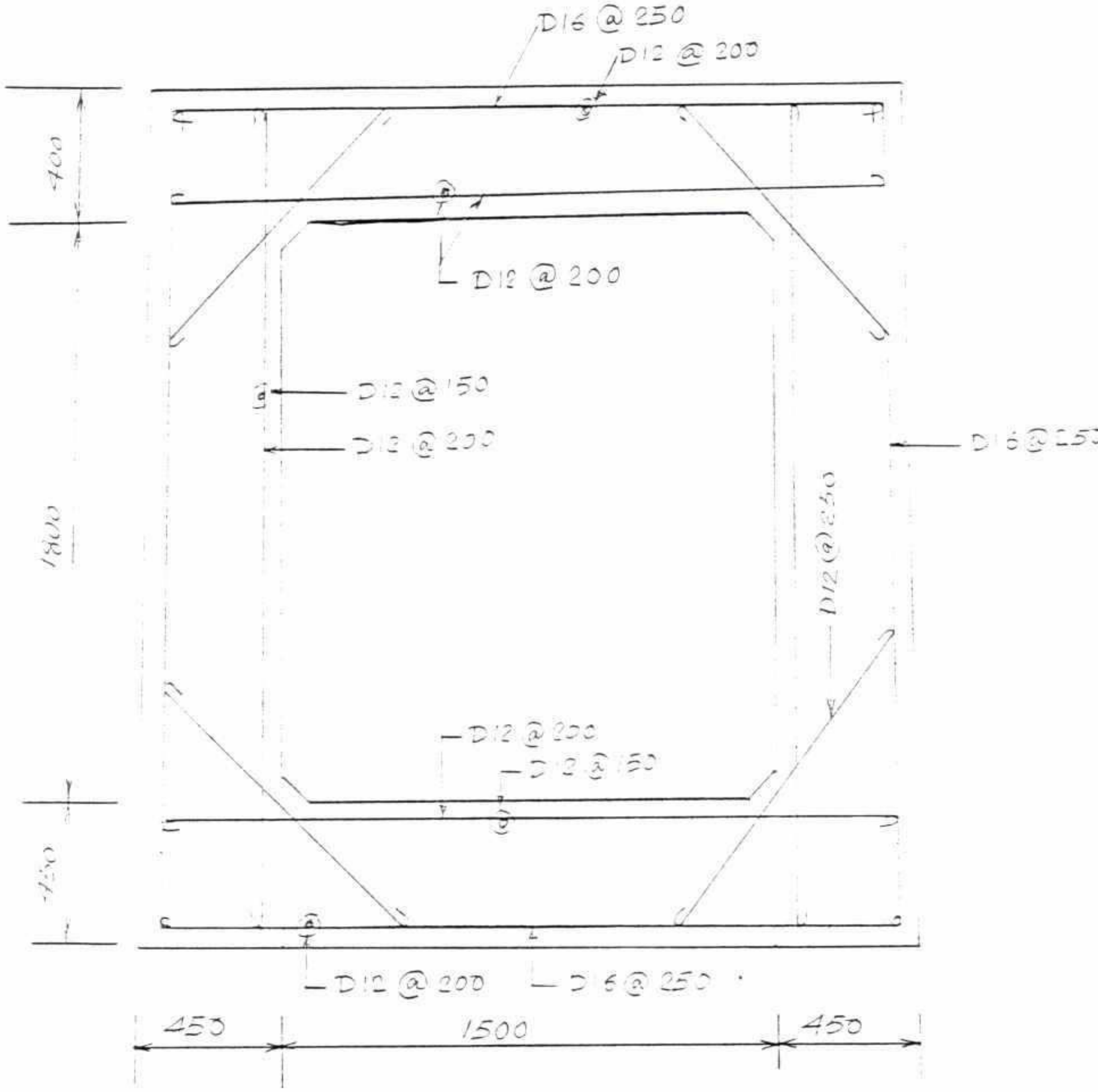
Design Moment	M :	4.23	KNm
Top thickness	ta(top) :	380	mm
Bottom thickness	ta(bot) :	450	mm
Length of member	l :	2.22	mm
Depth at zero shear	d(zs) :	1.10	mm
Actual thickness	ta(zs) :	415	mm

Ast (mom.)	:	109	mm^2
Ast (shrinkage)	:	760	mm^2
Ast (provided)			
Bar dia	(ø) :	16	mm
Bar spacing c/c	:	250	mm
Ast (Actual)	:	804	mm^2

EXPOSED FACE REINF

Moment at top	M(top) :	22.09	KNm
Reinf. top	Ast(top,mom) :	630	mm^2
Moment at bottom	M(bot) :	25.68	KNm
Reinf. botm.	Ast (bot,mom) :	601	mm^2
Shrinkage reinf.	Ast (sk) :	380	mm^2
Ast (provided)			
Bar dia.	(ø) :	12	mm
Bar spacing c/c	:	175	mm
Ast (Actual)	:	646	mm^2

11. REINFORCEMENT DETAILS OF BARREL (1-VENT)
(1.8 m x 1.5 m)



12. STRUCTURAL ANALYSIS OF BARREL (2-VENT)

TOP SLAB

Design Moment	M	=	40.21	KNm
Reqd. depth	dr(mom)	=	177	mm
Design Shear	V	=	120.01	KN
Reqd. depth	dr(shear)	=	317	mm
Reqd. Thickness	tr	=	344	mm
Provided Thickness	ta	=	377	mm

SHRINKAGE REINF.

Ast (reqd. exp. face)		=	760	mm^2
Ast (provided)				
Bar dia	(ø)	=	16	mm
Bar spacing c/c		=	250	mm
Ast (Actual)		=	804	mm^2

Earth face. Ast = 570 mm^2
D12 @ 200 c/c = Ast = 565 mm^2

TOP REINF.

Ast (mom.)		=	951	mm^2
Ast (shrinkage)		=	380	mm^2
Ast (provided)				
Bar dia	(ø)	=	16	mm
Bar spacing c/c		=	200	mm
Ast (Actual)		=	1005	mm^2

BOTTOM REINF.

Ast (mom.)		=	477	mm^2
Ast (shrinkage)		=	570	mm^2
Ast (provided)				
Bar dia	(ø)	=	12	mm
Bar spacing c/c		=	200	mm
Ast (Actual)		=	565	mm^2

BOTTOM SLAB

Design Moment	M	=	46.50	KNm
Reqd. depth	dr(mom)	=	190	mm
Design Shear	V	=	136.25	KN
Reqd. depth	dr(shear)	=	359	mm
Reqd. Thickness	tr	=	419	mm
Provided Thickness	ta	=	500	mm

SHRINKAGE REINF.

Ast (reqd. exp. face)	=	760	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	250	mm
Ast (Actual)	=	804	mm^2
Ast (reqd. earth face)	=	380	mm^2
Ast (provided)			
Bar dia	(ø) =	12	mm
Bar spacing c/c	=	300	mm
Ast (Actual)	=	377	mm^2

TOP REINF.

Ast (mom.)	=	489	mm^2
Ast (shrinkage)	=	570	mm^2
Ast (provided)			
Bar dia	(ø) =	12	mm
Bar spacing c/c	=	200	mm
Ast (Actual)	=	565	mm^2

BOTTOM REINF.

Ast (mom.)	=	975	mm^2
Ast (shrinkage)	=	380	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	200	mm
Ast (Actual)	=	1005	mm^2

ABUTMENT (SIDE WALL)

Design Moment (Top)	M =	35.86	KNm
Reqd. depth	dr(mom) =	143	mm
Design Shear (Top)	V =	89.22	KN
Reqd. depth	dr(shear) =	123	mm
Reqd. Thickness	tr =	295	mm
Provided Thickness	ta =	450	mm
Design Moment (Bottom)	M =	39.96	KNm
Reqd. depth	dr(mom) =	155	mm
Design Shear (Bottom)	V =	102.25	KN
Reqd. depth	dr(shear) =	145	mm
Reqd. Thickness	tr =	270	mm
Provided Thickness	ta =	500	mm

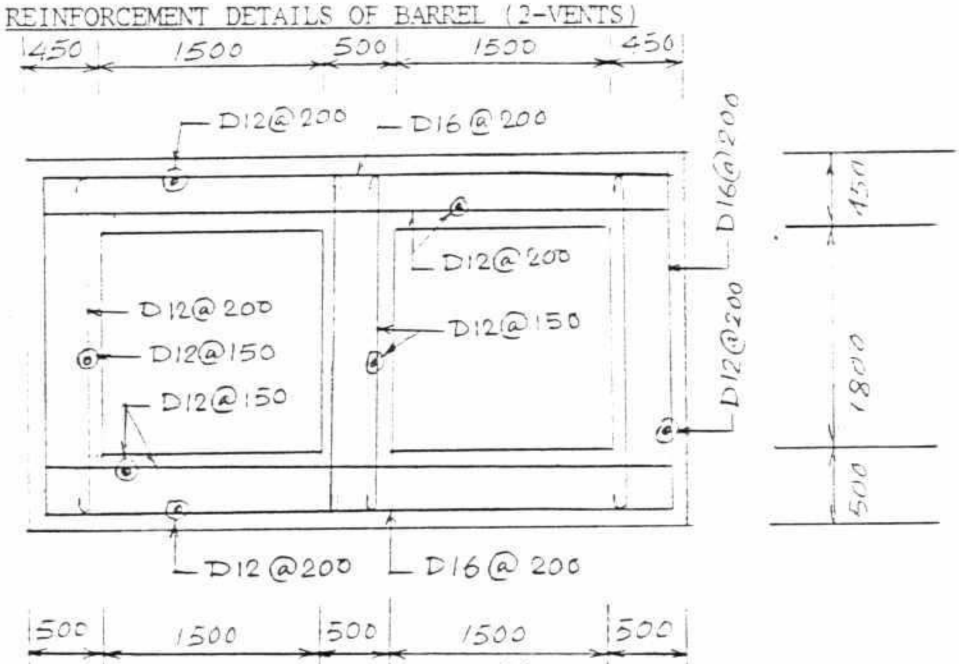
SHRINKAGE REINF.

Ast (reqd. exp. face)		=	760	mm^2
Ast (provided)				
Bar dia	(ø)	=	16	mm
Bar spacing c/c		=	250	mm
Ast (Actual)		=	804	mm^2
Ast (reqd. earth face)		=	380	mm^2
Ast (provided)				
Bar dia	(ø)	=	12	mm
Bar spacing c/c		=	275	mm
Ast (Actual)		=	411	mm^2

EXPOSED FACE REINF.

Design Moment	M	=	16.54	KNm
Top thickness	ta(top)	=	450	mm
Bottom thickness	ta(bot)	=	500.00	KN
Length of member	l	=	2.22	m
Actual thickness	ta(zs)	=	415	mm
Ast (mom)		=	368	mm^2
Ast (shrinkage)		=	570	mm^2
Ast (provided)				
Bar dia	(ø)	=	12	mm
Bar spacing c/c		=	200	mm
Ast (Actual)		=	565	mm^2

13. REINFORCEMENT DETAILS OF BARREL (2-VENTS)



EARTH FACE REINF.

Moment at top	M(top) =	35.81	KNm
Reinf. top. Ast (top. mom)	=	630	mm^2
Moment at bottom	M(bot) =	39.96	KNm
Reinf. bot. Ast (bot. mom)	=	848	mm^2
Shrinkage reinf. Ast (sk)	=	380	mm^2
Ast (provided)	=	368	mm^2
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	225	mm
Ast (Actual)	=	894	mm^2

PIERS

Design Moment	M =	10.37	KNm
Reqd. depth	dr(mom) =	109	mm
Design Shear	V =	22.22	KN
Reqd. depth	dr(shear) =	64	mm
Reqd. Thickness	tr =	150	mm
Provided Thickness	ta =	500	mm

REINFORMCEMENTS.

Ast (mom)	=	217	mm^2
Ast (shrinkage)	=	760	mm^2
Ast (provided)			
Bar dia	(ø) =	16	mm
Bar spacing c/c	=	250	mm
Ast (Actual)	=	804	mm^2

14. STRUCTURAL ANALYSIS OF R/S RETURN WALL

Top EL = (+) 3.50

Bottom EL = (+) 0.00

Stem Hight = 3.50 m

r_s = 77.0 KN/m^3

r_c = 23.6 KN/m^3

r_s = 189 KN/m^3

r_w = 9.81 KN/m^3

(ø) = 25^0

Ca = 0.41

Co-eff.of friction:- f = 0.5

Length of R/Wall = 5.55 m

Length of Heel = 1.90 m

Length of Toe = 1.00 m

Length of Base = 3.35 m

F.S. against O.T. = 5.02

F.S. against sliding = 1.60

Load = 200.74 KN

Resisting Moment = 411.11 KNm

O.T. moment = 81.82 KNm

P1 = 63.64 KN/m^2

P2 = 56.21 KN/m^2

e = 0.03

Stem:- Ast (reqd.) :- 1297 mm^2, t = 450 mm & 250 m
Ast (provided):- D16 @ 150 c/c (Ast = 1340)
50% curtailment at 1.45 m from bottom

Heel:- t = 500 mm
Ast (reqd.) = 1032 mm^2
Ast (provided):- D12 @ 150 (Ast = 1340)

Toe:- t = 500 m
Ast (reqd) = 407 mm^2
Provided, D12 @ 250 (Ast = 452 mm^2)

15. STRUCTURAL ANALYSIS OF R/S RETURN WALL

Top EL = (+) 3.50
Bottom EL = (+) 0.50

Stem Hight = 3.00 m t = 400 mm & 250 mm

Length of R/Wall = 4.80 m
Length of Heel = 1.70 m
Length of Toe = 0.80 m
Length of Base = 2.90 m

Total load = 154.45 KN
Resit. moment = 269.36 KNm

F.S. against O.T. = 5.13
F.S. against sliding = 1.70

P1 = 58.32 KN/m^2
P2 = 48.20 KN/m^2

Stem:- t (bottom) = 400 mm, t (top) = 250
Ast (reqd) = 937 mm^2
provided:- D12 @ 110 mm c/c (Ast = 1028 mm^2)
Curtailment at 1.46 m from bottom

Heel:- t = 450 mm
Ast (reqd) = 772 mm^2
Provided, D12 @ 125 mm c/c (Ast = 905 mm^2)

Toe :- t = 450 mm
Ast (reqd) = 380 mm^2
Provided, D12 @ 300 mm c/c (Ast = 377 mm^2)

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16. STRUCTURAL ANALYSIS OF WING WALL AND APRON

SECTION	Length from S(1) (m)	WINGWALL Thick(mm) (Top	(Bot)	WINGWALL Height (m)	APRON Thick (mm)	APRON Width (m)	WINGWALL Height (m)	APPRON Width (m)
S	L	Tt	Tb	H	Ta	B	H'	B
1-1	0.00	250	550	4.80	700	3.50	5.15	4.05
2-2	3.00	250	550	4.80	700	4.40	5.15	4.95
3-3	5.00	250	500	4.10	690	5.00	4.44	5.50
4-4	7.00	250	450	3.50	610	5.60	3.81	6.05
5-5	12.00	250	450	3.50	500	7.10	3.75	7.55
6-6	12.00	250	450	3.50	500	7.10	3.75	7.55

H' = Height of Wingwall from apron C/1
B' = Width of apron form C/1 of Wingwalls

Length of Wingwall = $(L2 + ((B - B1)/2)^2)$ 0.5 = 12.13 m

Unit Weight of Steel --> 77.00 KN/m^3
Unit Weight of Concrete --> 23.60 KN/m^3
Unit Weight of Soil --> 18.80 KN/m^3

Angle of Internal Friction (ø) --> 25^0
Coeff. of Active Earth Pressure (Ca) = (1-sinø)/(1+sinø) = 0.41

Ultimate Flexural strength of Steel (fy) --> 2.76E+05 KN/m^2
Ultimate Flexural strength of Concrete (fc') --> 1.72E+04 KN/m^2

Allowable Flexural Strength of Steel (fs) = ------------ 1.24E+05 KN/m^2
Allowable Flexural Strength of Concrete (fc) = 0.45 * fc' 7.74E+03 KN/m^2

Allowable Shear Stress of Concrete(v) = 2.89 * (fc') 0.5 = 3.79E+02 KN/m^2
Allowable Bond Stress (bs(allow)) = 113.40 * (fc') 0.5/d = 1.49E+04/d KN/m^2
(d = Bar diameter, mm) OR --> 1103 KNm^2 (whichever is less)

Modulus of Elasticity of Steel (ES) --> 1.96E+08 KNm^2
Modulus of Elasticity of Concrete (Ec) --> 1.98E+07 KNm^2

Coverage + 1/2 Dia. Reinforcement (c) --> 60 mm

b = Unit Width of Member = 1.00 m
n = Modular Ratio = Es / Ec = 9.90 m
r = fs/fc = 16.02
k = n/(n+r) = 0.382
j = Lever Arm Coefficient = 1 - k/3 = 0.873
R = Resisting Moment Coefficient = fc*j*k/2 = 1.29E+03 KN/m^2

REINFORCEMENT

NUMBER & SEC. No.	SHRINKAGE REINFORCEMENT EXPOSED Ast(req) (mm²)	ø (mm)	spc (mm)	Ast(act) (mm²)	EARTH Ast(req) (mm²)	ø (mm)	SPC (mm)	Ast(act) (mm²)	MAIN REINFORCEMENT TOP/EXPOSED Ast(req) (mm²)	ø (mm)	SPC (mm)	Ast(act) (mm²)	BOTTOM/EARTH Ast(req) (mm²)	ø (mm)	SPC (mm)	Ast(act) (mm²)
WING WALL																
1-1	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	2805
2-2	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	2805
3-3	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	1890
4-4	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	1340
5-5	760	16	250	804	570	12	175	646								

17. DESIGN OF FOUNDATION

FOUNDATION SOIL BEARING PARAMETER

SLUICE	BORE HOLE NUMBER	GROUND ELEVATION	BARREL BASE E.L.	SPT VALUE AT BARREL BASE	ALLOWABLLE BEARING* CAPACITY AT BARREL BASE	
RAYER BAZAR	1	5.6 m	0.5 m	6	0.97 T/□'	2038 lb/□'
(1 VENT)	4	6.3 m		5	0.81 T/□'	1814 lb/□'
HAZARIBAGH	1	7.2 m	0.5 m	13	2.23 T/□'	4995 lb/□'
(2 VENTS)	4	7.5 m		10	1.67 T/□'	3740 lb/□'
NAWABGANJ	1	6.3 m	0.5 m	3	0.48 T/□'	1075 lb/□'
(1 VENT)	4	6.6 m		3	0.48 T/□'	1075 lb/□'
SHAHIDNAGAR	1	6.2 m	0.5 m	4	0.65 T/□'	1456 lb/□'
(1 VENT)	4	6.6 m		7	1.14 T/□'	2553 lb/□'

* ALLOWABLE BEARING VALUES FROM SOIL MECHANICS AND ENGINEERING PRACTICE BY TERZAGHI AND PECK.

FOUNDATION PRESURE

1. EMBANKMENT SOIL

(4.1'x 20.5'+ 78.7'x14'+29.5'x10.6'+ 9.75'x 10.6') x 7.9'x 100 lb/cft.
+ ½ (29.5'x 9.75'+ 19.7'x 6.5'+ 9.75'x 3.28') x 7.9'x 100 lb/cft.
= (289+1102+312.7+103.3) x 7.9'x 100
+ ½ (287.6+128+32) x 7.9' x 100
= (1807+223.8) x 7.9' x 100 = 2030.8 x 7.9 x 120 = 19.25.198 lb

2. STRUCTURE

a. BARREL

(8'x 1.5'+ 8.2'x 1.6'+ 5.9'x 1.5 x 2) x 144 x 145
= (12□'+ 13.1 + 17.7) x 144 x 145 = 42.8 x 144 x 145 = 8.93.664 lb

b. HEAD WALL

8.2' x 10.6' x 1.1' x 2 x 145 = 27.275 lb

c. END SIDE WALL

6.5' x 18'x 1.3'x 2 x 145 = 44.109 lb

3. GATE = 1.000 lb

TOTAL = 28.91.246 lb

THEREFORE. FOUNDATION PRESSURE

$$= \frac{28.91.246}{144 \times 8.2} = 2448 \text{ lb}/\square'$$

SOIL BEARING CAPACITY

SLUICE	FOUNDATION PRESSURE	ALLOWABLE BEARING CAPACITY AT BARREL BASE	REMARKS
RAYER BAZAR (1 VENT)	2448 lb/□'	1814 lb/□'	NEEDS TREATMENT
HAZARIBAGH (2 VENTS)	2448 lb/□'	3740 lb/□'	O.K.
NAWABGANJ (1 VENT)	2448 lb/□'	1086 lb/□'	NEEDS TREATMENT
SHAHIDNAGAR (1 VENT)	2448 lb/□'	1456 lb/□'	NEEDS TREATMENT

FOUNDATION TREATMENT

BARREL BASE WIDTH: 8.2　　　EFFECTIVE DISTRIBUTION ANGLE: 30^0

SLUICE	DEPTH OF SAND FILLING	FOUNDATION WIDTH AFTER TREATMENT	BORE HOLE NO.	SPT VALVE AT 1M BELOW BARREL BASE	ALLOWABLE SOIL BEARING CAPACITY AT THE BASE OF SAND FILLING		FOUNDATION PRESSURE AT THE BARREL BASE	FOUNDATION PRESSURE AT THE BASE OF SAND FILLING
RAYER BAZAR	3.28 ft (1 m)	12 ft.	1	7	1.14 T/□'	2553 lb/□'		1672 lb/□'
			4	5	0.81 T/□'	1814 lb/□'		
NAWABGANJ	3.28 ft (1 m)	12 ft.	1	5	0.81 T/□'	1814 lb/□'	2448 lb/□'	1672 lb/□'
			4	5	-do-	- do-		
SHAHIDNAGAR	3.28 ft (1 m)	12 ft.	1	5	0.81 T/□'	1814 lb/□'		1672 lb/□'
			4	7	1.14 T/□'	2553 lb/□'		

DESIGN REPORT ON

SLUICE ON EMBANKMENT EXTENSION AT KELLAR MORH

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KELLERMORE SLUICE
(1 VENT)

1. DESIGN DISCHARGE

Criteria :-

Catchment Area = 1.64 km² or 164 hectares

Return Period = 2 yr.

Duration of Rainfall = 6 hr.

Rainfall = 135 mm

Peak Run-off = Av. of 2nd, 3rd & 4th hr. rainfall

Formula for Q = $\frac{CiA}{360}$

where

Q = Discharge in m³/sec.

C = 0.6 (Run-off co-efficient)

i = Rainfall Intensity mm/hr.

A = Area in hectare

Rainfall Distribution = 1st hr. 8%

2nd hr. 15%

3rd hr. 44%

4th hr. 16%

5th hr. 8%

6th hr. 7%

Pre-monsoon W.L. = 2.5 m to 3.3 m

Barrel Size = 1.8 m X 1.5 m

Co-efficient of Discharge = 0.75

Discharge:-

Q (2nd hr.) = 5.5 m^3/sec.

Q (3rd hr.) = 16.2 m^3/sec.

Q (4th hr.) = 5.9 m^3/sec.

$\therefore$ Design Discharge = $(5.5+16.2+5.9) \div 3 = 9.2$ m^3/sec.

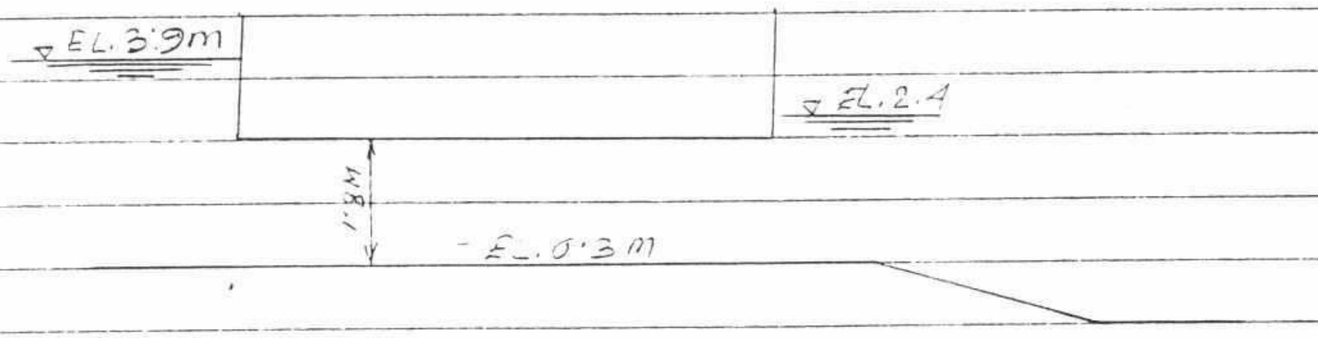
VENT SIZE AND NUMBER

Assumption:

Vent Size = 1.8m × 1.5m

Sill EL. = 0.3 m

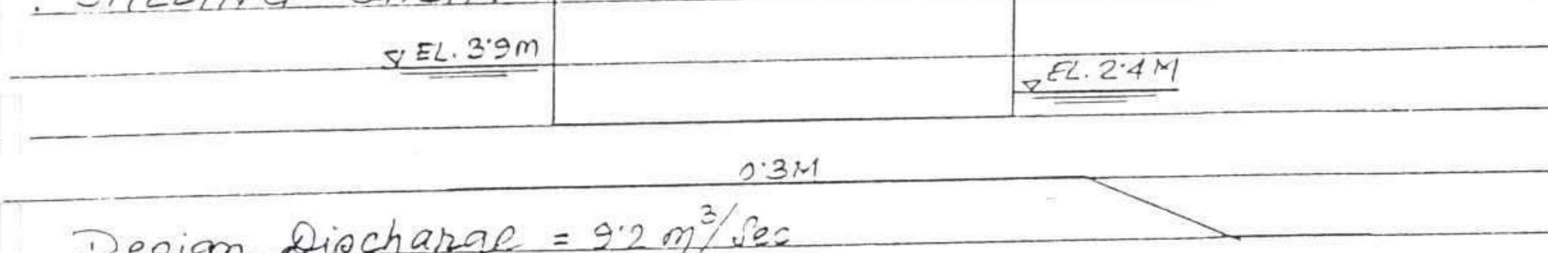
D/S. W. L = 2.5m



Discharge per vent = 10.2 m^3/sec > Design discharge

$\therefore$ Number of vent required is 1

STILLING BASIN



Design Discharge $= 9{\cdot}2\ m^3/sec$

Number of Vent $= 1$ No.

Vent Size $= 1{\cdot}8\,m \times 1{\cdot}5\,m$

Flow per meter width, $q = \frac{9.2}{1{\cdot}5} = 6.1\ m^3/sec/m$

Critical Depth, $D_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{6.1^2}{9.8}\right)^{1/3} = (3{\cdot}8)^{1/3} = 1{\cdot}56\ m$

Neglecting Velocity Head, $H_L = 3{\cdot}9 - 2{\cdot}4 = 1{\cdot}5\ m$

From Blench Curve, $E_{f_2} = 2{\cdot}8\ m$ $\therefore E_{f_1} = H_L + E_f$
$= 1{\cdot}5 + 2{\cdot}8 = 4{\cdot}3\ m$

From Energy of Flow curve

$D_1 = 0{\cdot}63\ m$

$D_2 = 2{\cdot}5\ m$

Length of the Cistern $= 6(2{\cdot}5 - 0{\cdot}63) = 6 \times 1{\cdot}87 = 11.2\ m$
say $11.5\ m$

Apron level $=$ D/S W.L. $- D_2 = 2{\cdot}4 - 2{\cdot}5 = -0{\cdot}1\ m$
Lower the Apron by $0{\cdot}2\ m$

$\therefore$ Apron EL. $= -0{\cdot}3\ m$

SCOUR DEPTH (DOWN STREAM)

$Q = 9.2\ m^3/sec. \qquad B = 3.5\ m$

$q = 9.2/3.5 = 2.63\ m^3/sec \qquad d_m = 0.03\ mm$

$f = 1.76\sqrt{d_m} = 0.3$

Depth of Scour, $R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{2.63^2}{0.3}\right)^{1/3}$

$= 1.35\left(\frac{6.92}{0.3}\right)^{1/3} = 1.35(23.067)^{1/3} = 1.35 \times 2.84$

$= 3.84\ m$

EL. of D/s cut-off bottom $= D/s\ N.L. - 1.5 \times R = 2.4 - 1.5 \times 3.84$

$= 2.4 - 5.8 = -3.4\ m$

DESIGN CRITERIA AND CONSIDERATIONS (BARREL)

Embankment Crest Level = 9.00m
Top Slab level = 3.25m
Max. Water level = 3.00m

Unit Weight of Steel = 77.00 $KN m^3$

Unit Weight of Concrete = 23.6 $KN m^3$

Unit Weight of Soil = 18.8 $KN m^3$

Unit Weight of Water = 9.81 $KN m^3$

Angle of Internal Friction, θ = 25°

Co.eff. of Earth-pressure at rest (Ca) = $(1-\sin\theta)$

Top Slab Thickness = 450mm
Bottom Slab Thickness = 500mm
Abutment Top Thickness = 450 mm
Abutment bottom Thickness = 500 mm
Pier Thickness = 500 mm
Inside Height of the Barrel = 1.8m

Inside width of the Barrel = 1.5m
Length of the Barrel = 40.90m

5. STRUCTURAL ANALYSIS OF BARREL

TOP SLAB

Design moment M = 357.3 KNm
Required Depth dr = 167 mm
Design Shear V = 116.31 KN
Required Depth dr = 307 mm
Required Thickness tr = 367 mm
Thickness Provided ta = 450 mm

Shrinkage Reinf.
Ast (Reqd. Exp. Face) = 760 mm²
Ast (Provided)
Bar Diameter ϕ = 16 mm
Bar Spacing c/c = 250 mm
Ast (Actual) = 804 mm²

Top Reinf.
Ast (mom) = 845 mm²
Ast (Shrinkage) = 380 mm²
Ast (Provided)
Bar Dia ϕ = 16 mm
Bar Spacing c/c = 225 mm
Ast (Actual) = 894 mm²

Bottom Reinf.
Ast (mom) = 496 mm²
Ast (Shrinkage) = 760 mm²

Ast (Provided)

Bar Dia $\phi = 12\ mm$

Bar Spacing c/c $= 200\ mm$

Ast (Actual) $= 565\ mm^2$

BOTTOM SLAB

Design Moment $M = 41.68\ KN\ m$

Required Depth $d_r = 180\ mm$

Design Shear $V = 135.44\ KN$

Required Depth $d_r = 337\ mm$

Required Thickness $t_r = 417\ mm$

Thickness Provided $t_a = 490\ mm$

Shrinkage Reinf.

Ast (Reqd. Exp. Face) $= 760\ mm^2$

Ast (Provided)

Bar Dia. $\phi = 16\ mm$

Bar Spacing c/c $= 250\ mm$

Ast (Actual) $= 804\ mm^2$

Ast (Reqd. Earth Face) $= 370\ mm^2$

Ast (Provided)

Bar Dia. $\phi = 12\ mm$

Bar Spacing c/c $= 200\ mm$

Ast (Actual) $= 565\ mm^2$

Top Reinf.

Ast (min.)	= 546 mm²
Ast (shrinkage)	= 570 mm²
Ast (Provided)	
Bar Dia.	ϕ = 12 mm
Bar Spacing c/c	= 200 mm
Ast (Actual)	= 565 mm²

Bottom Reinf.

Ast (min.)	= 874 mm²
Ast (shrinkage)	= 380 mm²
Ast (Provided)	
Bar Dia.	ϕ = 16 mm
Bar Spacing c/c	= 225 mm
Ast (Actual)	= 894 mm²

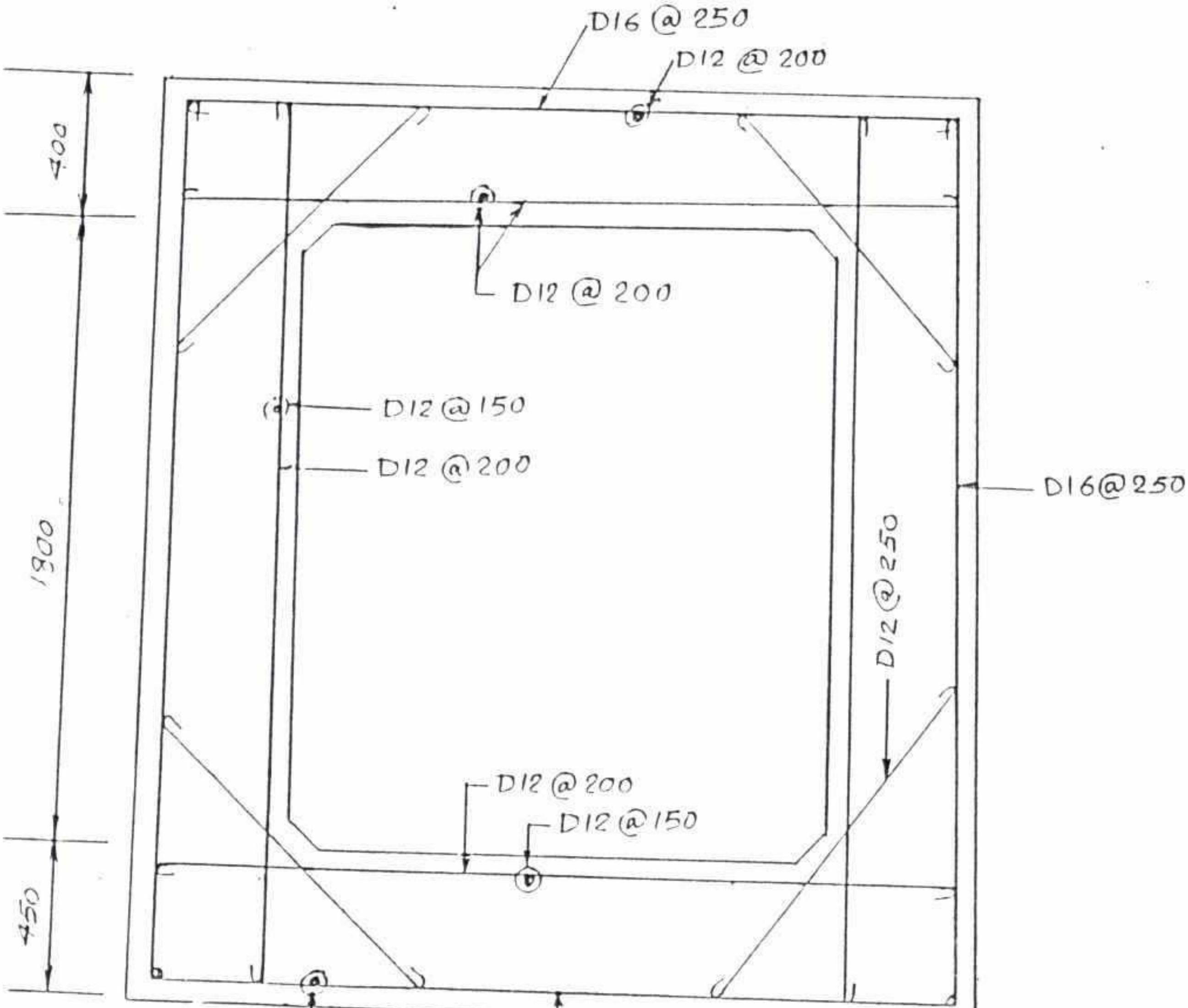
ABUTMENT SIDE WALL

Design Moment (Top)	M = 35.73 kNm
Reqd. Depth	d_r = 143 mm
Design Shear (Top)	V = 60.4 kN
Reqd. Depth	d_r = 133 mm
Reqd. Thickness	T_r = 293 mm
Thickness Provided	T_a = 450 mm

Design Moment (Bottom)	M = 41.62 kNm
Reqd. Depth	d_r = 180 mm
Design Shear	V = 133.06 kN
Reqd. Depth	d_r = 272 mm

Reqd. Thickness $t_r = 332$ mm
Thickness Provided $t_a = 500$ mm

7. REINFORCEMENT DETAILS OF BARREL



8 RIVER SIDE RETURN WALL

Top EL. = 3.5 m — r_b = 77.0 KN/m³

Bottom EL. = 0.0 — r_c = 23.6 KN/m³

Stem Height = 3.5 m — r_s = 18.9 KN/m³

r_M = 9.81 KN/m³

C_a = 0.41

Co-efficient of friction, f = 0.5

Length of wall = 5.55 m

Length of heel = 1.19 m

Length of toe = 1.0 m

Length of base = 3.35 m

F.S. against O.T. = 5.0[illegible]

F.S. against sliding = 1.6

Load = 200.74 KN

Resisting moment = 4[illegible] KNm

O.T. moment = 8.82 KNm

P_1 = 63.64 KN/m²

P_2 = 56.21 KN/m²

e = 0.03

STEM :- As (Reqd) = 1297 mm² t = 450 mm
& 250 mm

Ast (Provided) = D16 @ 150 mm c/c (Ast = 1340)

50% curtailment at 1.45 m from bottom

Heel :- $t = 500mm$

$Ast(Reqd.) = 1032 mm^2$

$Ast(Provided) = D12$ @ 150mm c/c ($Ast = 1340 mm^2$)

Toe :- $t = 500mm$

$Ast(Reqd.) = 407 mm^2$

$Ast(Provided) = D_{12}$ @ 250mm c/c ($Ast = 452 mm^2$)

9. COUNTRY SIDE RETURN WALL

Top E.L. = 3.5 m

Bottom E.L. = 0.5 m

Stem Height = 3.0 m

t = 400mm & 250mm

Length of wall = 4.8 m

Length of Heel = 1.7 m

Length of Toe = 0.8 m

Length of Base = 2.9 m

Total load = 154.45 KN

Resisting Moment = 269.36 KNm

F.S. against O.T. = 5.13

F.S. against sliding = 1.7

$P_1 = 58.32\ KN/m^2$

$P_2 = 48.2\ KN/m^2$

stem :- t (bottom) = 400 mm t (Top) = 250 mm

Ast (Reqd.) = 937 mm^2

Ast (provided) :- D_{12} @ 110 mm c/c (Ast = 1028 mm^2)

Curtailment at 1.46 m from bottom

Heel :- t = 450 mm

Ast (Reqd.) = 772 mm^2

Ast (provided) = D_{12} @ 110 mm c/c (Ast = [illegible] mm^2)

Toe :- t = 450 mm

Ast (Reqd.) = 380 mm^2

Ast (Provided) = D_{12} @ 300 mm c/c (Ast = 3[illegible] mm^2)

6. WING WALL AND APRON

SECTION	LENGTH FROM S(1)	WING WALL Thickness Top	WING WALL Thickness Bot	WING WALL Height	APRON Thickness	APRON Width	WINGWALL Height	APRON Width
	(m)	(mm)	(mm)	(m)	(mm)	(m)	(m)	(m)
S	L	Tt	Tb	H	Ta	B	H	B
1-1	0.00	250	550	4.8	700	3.5	5.15	4.05
2-2	3.00	250	550	4.8	700	4.4	5.15	4.05
3-3	5.00	250	500	4.1	690	5.0	4.44	5.50
4-4	7.00	250	450	3.5	610	5.6	3.81	6.05
5-5	12.00	250	450	3.5	500	7.1	3.75	7.55
6-6	12.00	250	450	3.5	500	7.1	3.75	7.55

H = Height of Wing wall from Apron C/L

B = Width of Apron from C/L of Wingwalls

Length of Wingwall = $(L^2 + ((B - B1)/2)^2)^{0.5}$ = 12.13m

Unit Weight of Steel = 77.00 KN/m^3

Unit Weight of Concrete = 23.6 KN/m^3

Unit Weight of Soil = 18.8 KN/m^3

Angle of internal Friction θ = 25°

Co-efficient of Active Earth Pressure (Ca) = $(1-\sin\theta)/(1+\sin\theta)$ = 0.41

Ultimate Flexural Strength of Steel (f_y) = 2.75E+05 KN/m^2

Ultimate Flexural Strength of Concrete ($f'c$) = 1.72E+04 KN/m^2

Allowable Flexural Strength of Steel (f_s) = 1.24E+05 KN/m^2

Allowable Flexural Strength of Concrete (f_c) = 0.45 x f'c 7.74E+03 KN/m^2

Allowable Shear Stress of Concrete (v) = 2.89 x (f'c) 0.5 = 3.79E+02 KN/m^2

Allowable Bond Stress = 113.4 x (f'c) 0.5/d = 1.49E+04/d KN/m^2

(d = Bar Diameter, mm) or → 1103 KNm^2 which ever is less.

Modulus of Elasticity of Steel (Es) $= 1.96E+08\ KN/m^2$
Modulus of Elasticity of Concrete (Ec) $= 1.98E+07\ KN/m^2$

Coverage + ½ Dia Reinforcement (c) = 60 mm

b = Unit Width of Member = 1.0 m
η = Modular Ratio = E_s/E_c = 9.9 m
$r = f_s/f_c = 16.02$
$K = n/(n+r) = 0.382$
j = Lever Arm Co-efficient $= 1 - K/3 = 0.873$
R = Resisting Moment Co-efficient $= f_c \times j \times K \times \frac{1}{2} = 1.29E+03\ KN/m^2$

REINFORCEMENT

NUMBER & SEC. No.	SHRINKAGE REINFORCEMENT EXPOSED Ast(req) (mm^2)	a (mm)	spc (mm)	Ast(act) (mm^2)	EARTH Ast(req) (mm^2)	a (mm)	SPC (mm)	Ast(act) (mm^2)	MAIN REINFORCEMENT TOP/EXPOSED Ast(req) (mm^2)	a (mm)	SPC (mm)	Ast(act) (mm^2)	BOTTOM/EARTH Ast(req) (mm^2)	a (mm)	SPC (mm)	Ast(act) (mm^2)
WING WALL																
1-1	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	2805
2-2	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	2805
3-3	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	1890
4-4	760	16	250	804	570	12	175	646	570	12	175	646	2666	25	175	1340
5-5	760	16	250	804	570	12	175	646								

II. FOUNDATION

Foundation soil of the flood embankment from Kellarmore to Mitford is very weak. It needs treatment to keep the flood embankment in tolerable shape. Additional Load on the foundation of the sluice is not much because of smaller height of the structure. So no additional foundation treatment is required for the sluice.

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DESIGN REPORT

ON

SLUICE ON SEGUNBAGICHA KHAL ACROSS

CENTRAL SPINE ROAD

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DETAILED DESIGN
OF
SLUICE ON SEGUNBAGICHA KHAL
ACROSS
CENTRAL SPINE ROAD.

CONTENTS

DETALED DESIGN OF SLUICE ON SEGUNBAGICHA-KHAL ACROSS CENTRAL SPINE ROAD

1 DESIGN DISCHAGE:

a) Catchment area: (upstream of central spine road) = 6.95 km^2 (ref. drg. No-)
= 695 ha

b) Return period = 2 yrs

c) Duration of rainfall = 6 hrs

d) 1:2 year return period = 135 mm

e) Hourly distribution of rainfall:

1st hr = 9% = 12.1 mm
2nd " = 15% = 20.3 "
3rd " = 44% = 59.4 "
4th " = 16% = 21.6 "
5th " = 9% = 12.1 "
6th " = 7% = 9.5 "

f) Peak runoff = ave. of 2nd, 3rd & 4th hr. rainfall

g) Rational formula for Q = CiA/360

where, Q = Peak discharge m^3/sec
C = Run off co-efficient = 0.6 (Considering commercial & industrial area by (JICA))
i = rainfall intensity mm/hr
A = Drainage area in hectre

h) Assuming 2nd, 3rd & 4th hour rain fall produce peak run off:

Q (2nd hr.) = 0.6 × 20.3 × 695 × 1/360 = 23.5 m^3/sec
Q (3rd hr) = 0.6 × 59.4 × 695 × 1/360 = 69.0 m^3/sec
Q (4th hr) = 0.6 × 21.6 × 695 × 1/360 = 25.0 m^3/sec

i) Design discharge of the drainage outlet

Q = av. of 2nd, 3rd, & 4th rainfall runoff
= (23.5 + 69.0 + 25.0)/3 = 39.10 m^3/sec

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ref: Main report on the updating study on strom water drainage system improvement project in Dhaka City, Feb. 1990, by (JICA)

2. VENT SIZE & NUMBER:

a) Assumption :

i) Vent size = 1.8 m × 1.5 m

ii) Sill elevation = 0.3 M

iii) Downstream water level during monsoon = 5.25 m PWD (ave. of 10 yr. HWL during end of July at Demra + 0.1m)

iv) Max^m permissible internal (C/S) pond level = 6.75 PWD

v) Head difference = 1.5 m

b) Co-efficient of discharge under submerged condition

$$C_d = \left[1.0 + 0.4 (r)^{.3} + .0045 \times h / r^{1.25} \right]^{-1/2}$$

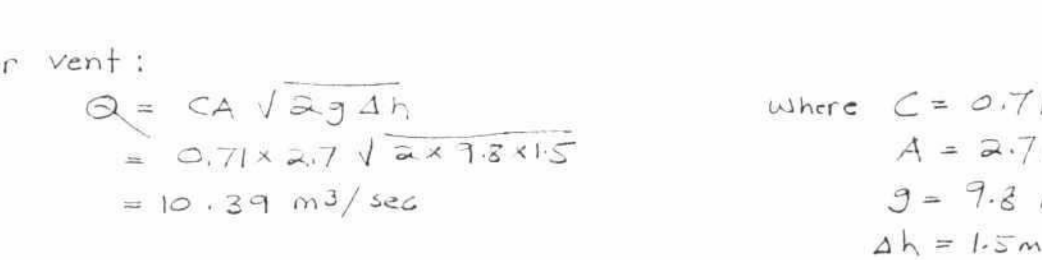
$$= \left[1.0 + 0.4 (.41)^{.3} + .0045 \times 50 / (0.41)^{1.25} \right]^{-1/2} = 0.71$$

where, $A = 1.8 \times 1.5 = 2.7 m^2$

$P = (1.8 + 1.5) \times 2 = 6.6 m$

$r = A/P = 2.7/6.6 = 0.41$

$L = 50 m$



Discharge per vent :

$$Q = CA\sqrt{2g\Delta h}$$

$$= 0.71 \times 2.7 \sqrt{2 \times 9.8 \times 1.5}$$

$$= 10.39 \ m^3/sec$$

where $C = 0.71$

$A = 2.7 m^2$

$g = 9.8 \ m/sec$

$\Delta h = 1.5 m$

for 4-vents total $Q = 10.39 \times 4 = 41.56 \ m^3/sec > 39.1 m^3/sec$ <u>O.K</u>

So <u>4-vents of size</u> (1.5m × 1.8m) to be used

3. <u>SCOUR DEPTH</u>: (Down-Stream)

$Q = 39.1\ m^3/s$ $\quad B = 11.6\ m$ $\quad$ Downstream water = 2.40 m
$q = 3.37\ m^3/s$ $\quad d_m = 0.03\ mm$ $\quad$ (av. of W.C during mid way
$f = 1.76\sqrt{d_m} = 0.3$ $\quad$ at Demra + 0.1m)

Depth of scour, $R = 1.35\left(q^2/f\right)^{1/3} = 1.35\left(3.37^2/0.3\right)^{1/3} = 4.53\ m$

EL of D/S cut-off Bottom = D/S W.L $- 1.5 \times R = 2.4 - 1.5 \times 4.53$
$= 2.4 - 6.8 = -4.4$

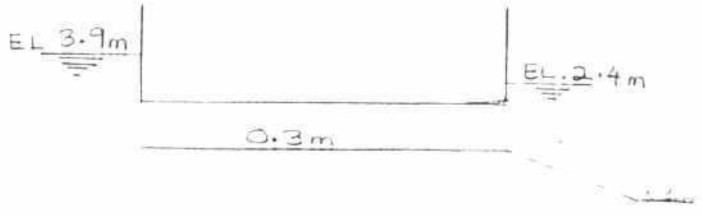
4. <u>EXIT GRADIENT</u>:
Weighted Creep length $= 1.7 + 1 + 1/3 \times 59 + 3.6 + 4.1 = 31\ m$

50 years flood stage = EL. 7.3 m
Inside Innundation Level = 4.5 m $\quad \therefore$ Head Difference = 2.8 m

$\therefore$ Weighted Creep Ratio $= 31/2.8 = 11 > 8.5$

Safe weighted Creep Ratio for very fine sand or silt is 8.5
<u>Structure is safe</u>.

5. <u>STILLING BASIN</u>:



Design Discharge = $39.1\ m^3/sec$
No. of vent = 4 Nos
Vent size = $1.8m \times 1.5m$
Pier Thickness = 380 mm
Flow width $= 1.5 \times 4 + 0.38 \times 3 = 7.14\ m$
Flow per meter width, $q = 39.1/7.14 = 5.47\ m^3/sec$
Critical depth, $D_c = (q^2/g)^{1/3} = (5.47^2/9.8)^{1/3} = 1.45$
Neglecting velocity Head, $H_L = 3.9 - 2.4 = 1.5\ m$

From Blench Curve, $E_{f2} = \overset{3}{2.8}\ m$ $\quad \therefore E_{f1} = H_L + E_{f2} = \overset{3}{4.4}\ m$
From Energy of Flow Curve -
$D_1 = 0.63\ m$
$D_2 = 2.50\ m$.
Length of the Cistern $= \underline{6}\,(2.5 - 0.63) = 6 \times 1.87 = 11.22\ m$ say 11.0 m

Apron Level $= D/S\ W.L - D_2 = 2.4 - 2.5 = -0.1\,m$ (2.7, -0.3)

Lower the Apron by 0.2 m ; Apron EL = -0.3 m (-0.4)

End Sill –

Height $= 0.2 D_2 = 0.2 \times 2.5m = 0.5m$

width & spacing $= 0.15 \times D_2 = 0.15 \times 2.5 = 0.375$, Say 0.4 m

Top width $= 0.02 \times D_2 = 0.02 \times 2.5 = 0.05m$

UP-STREAM CUT-OFF :

Provide 1.4 m deep and 400 mm - Thick concrete cut-off at the upstream floor end.

6. UP-STREAM WING WALL

Assumption :

$\phi = 15°$

$\gamma_{sat} = 1844\ kg/m^3$

$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.59$

Earth pressure, P

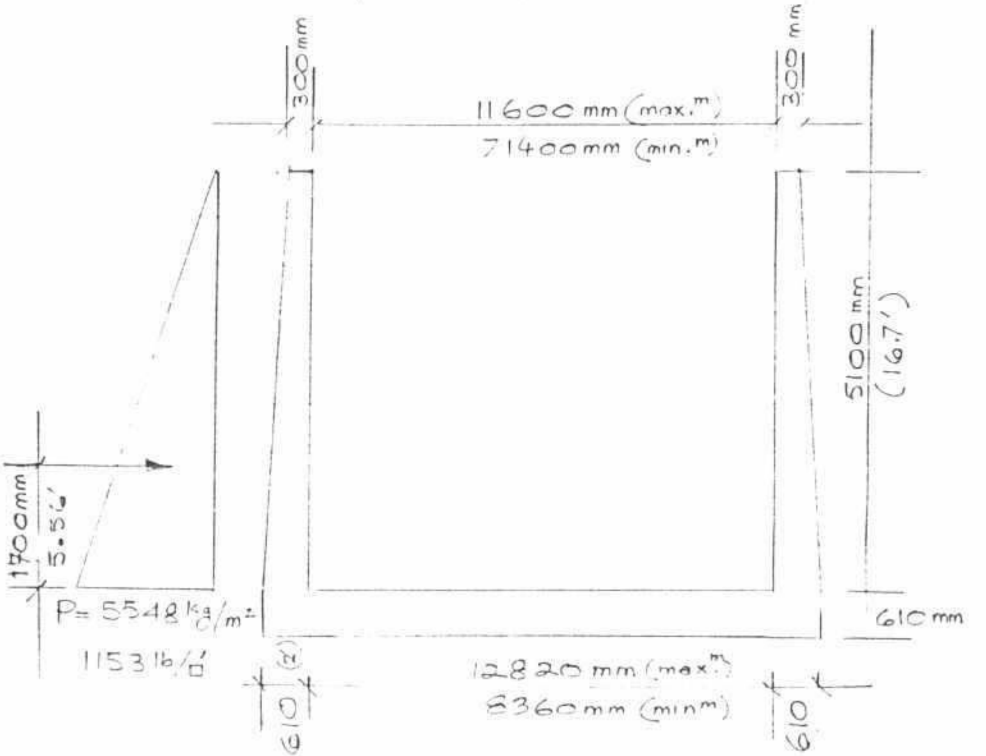
$= 0.59 \times 1844 \times 5.1$

$= 5548\ kg/m^2$

$= 1133\ lb/\square'$

$P_A = \frac{1}{2} \times 5.1 \times 5548$

$= 14147\,kg\ (94601b)$



Stem

Assumption :

$f'_c = 2500\,psi$

$f_c = 0.45 \times f'_c = 1125\,psi$

$f_s = 18{,}000\ psi$

$K = 0.385$ | $R = 187$

$j = 0.87$

Moment at the base of the stem = 9460 × 5.56 = 52597 ft·lb

$d = \sqrt{52597/189} = 16.2''$ provide $\underline{24''}$ Thickness

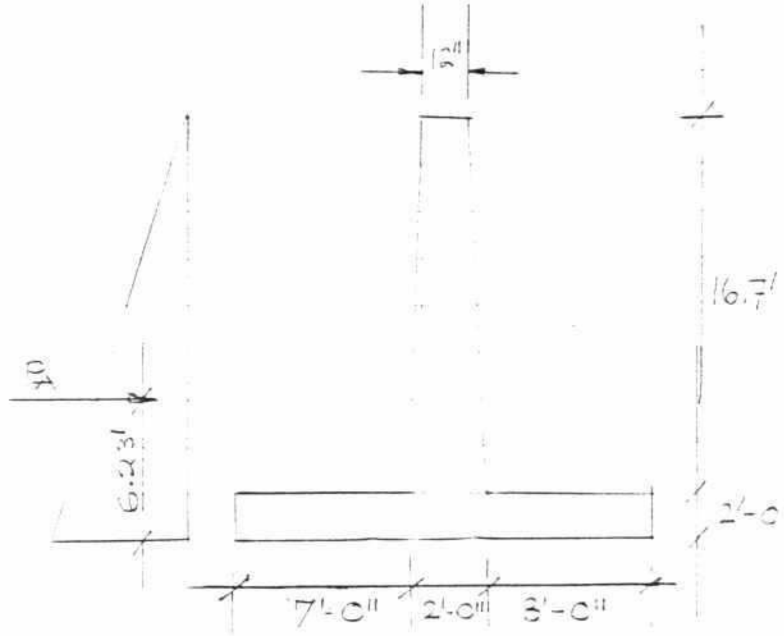
$fs = \sqrt{E\ 2597 \times 12 / 18{,}000 \times 0.33 \times 21} = 1.9$ □"

Provide 3/4"φ (20mmφ) @ 2.75" c/c (70 mm c/c)

Temp. and shrinkage reinf. at The bottom = 24×12×0.0025 = 0.72 □"
each face 0.36 □"; provide 1/2"φ (12mmφ) @ 6" c/c (150mm c/c)
Temp. and shrinkage reinf. at The Top = 12×12×0.0025 = 0.36 □"
each face 0.18 □"; provide 1/2"φ (12mmφ) @ 12" c/c (300mm c/c)

Thickness of the bottom will be same as that of The stem bottom. Main reinforement will also be same. Total Temp. and shrinkage reinforement is 24×12×0.0025 or 0.72 □" in both faces; provide 1/2"φ (12mm φ) @ 6" c/c (150 mm c/c)

7. UP-STREAM FLANK WALL

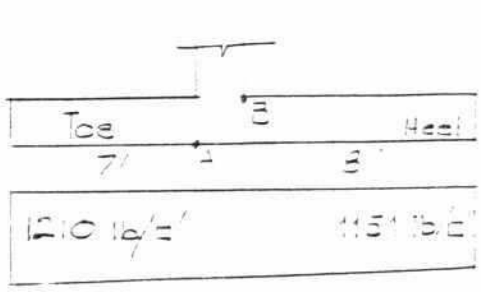


Assumption
$\phi = 15°$
$\gamma_{sat} = 115$ lb/cft

Overturning Force = 11865 lb
Overturning Moment = 73917 ft-lb
Stabilising Force = 25182 lb
Stabilising Moment = 230,163 ft-lb

Max^m. pressure = 1510 lb/□'
Min^m. pressure = 1451 lb/□'

Design of Toe (24" thick)
Moment at A = 29645 ft-lb
A_s = 1.04 □"
provide 3/4"φ @ 5.1" c/c
Temp. & Shirankage reinforcement
= 0.0025×24×12 = 0.72 □"
each face 0.36□'; provide 1/2"φ @ 6" c/c



NET PRESSURE

Design of Heel (24" thick)
Moment at B = 23696 ft-lb
As = 0.83 □" ; Provide 3/4"φ @ 6" c/c

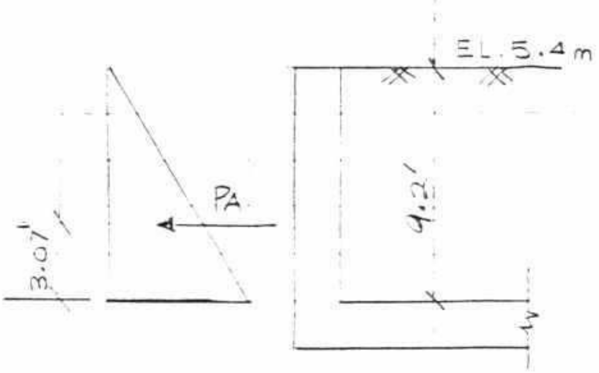
Design of stem (24" Thick)

Moment at B = 52660 ft-lb

As = 1.90 □" ; provide 3/4"φ @ 2.75" c/c

Temp. & Shrinkage renf. = 1/2"φ @ 6" c/c

8. UP-STREAM END WALL (12" Thick) Above Barrel Top



φ = 15°

γ_{sat} = 115 lb/cft

P_A = 2870 lb

Moment (max^m) = 8801 ft-lb

Provide 3/4"φ @ 7" c/c

Temp. & Shrinkage renf. = 12x12x 0.0025

= 0.36 □"

Each face 0.18□" ; Provide 1/2"φ @ 12" c/c

9. DOWN- STREAM WING WALL:

The height of the down stream flank wall is variable, max^m 20.6 ft and min^m 12.5 ft. for convenience of The structural analysis The wall is divided into 3 equal segments which are as follows:

- First Segment 20.5 ft av. 19.5 ft
  Second " 16.5 ft av. 16.5 ft
  Third " 15.0 ft av. 13.6 ft

First Segment (20.5 ft)

φ = 15°

γ_{sat} = 115 lb/□'

Overturning Force P_A = 14399 lb

Overturning Moment = 97913 ft.lb

Max^m thickness 26"

Min^m thickness 12"

As = 3.2□" ; Provide 1"φ @ 3" c/c

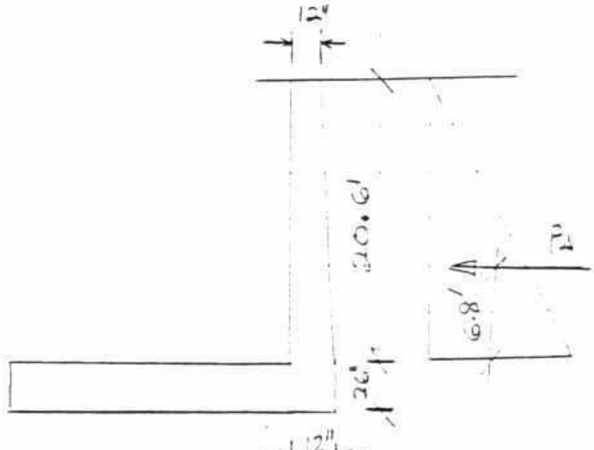
Temp. & shinkage reinf. at the bottom

= 26x12x 0.0025 = 0.78□"

each face 0.39□" provide 1/2"φ @ 6" c/c

At The middle 1/2"φ @ 6" c/c

At The Top 1/2" φ @ 12" c/c



Bottom slab

thickness = 26"
Main Reinforcement 1" φ @ 3" c/c
Temp & Shrinkage Reinf. 1/2" φ @ 6" c/c

Second Segment (16.5 ft)

Overturning Force = 9240 lb
Overturing Moment = 50820 ft-lb

Stem

Thickness 21"
A_s = 2.1 □" ; provide 1" φ @ 4" c/c
Temp. & Shrinkage reinf. = 1/2" φ @ 6" c/c

Bottom slab

Thickness 21"
Main Reinforcement 1" φ @ 4" c/c
Temp. & shrinkage reinf. 1/2" φ @ 6" c/c

Third Segment (15 ft)

Overturning Force = 7635 lb
Overturning Moment = 38175 ft-lb

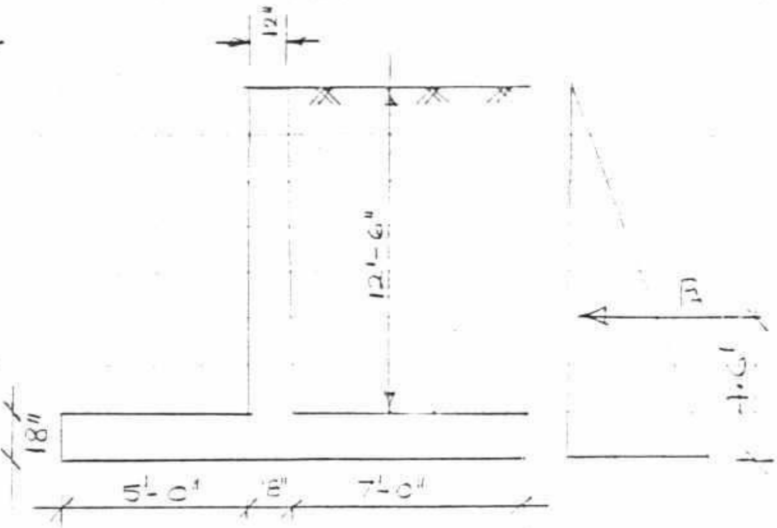
Stem

Thickness 18"
A_s = 1.86 □" provide 1" φ @ 4" c/c
Temp. and shrinkage reinf. 1/2" φ @ 12" c/c

Bottom slab

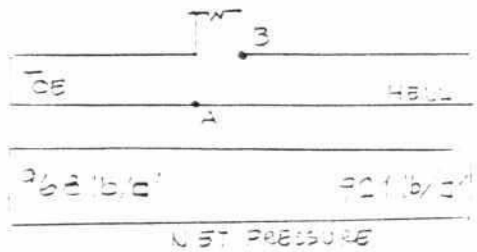
Thickness 18"
Main reinf. 1" φ @ 4" c/c
Temp. & shrinkage reinf. 1/2" φ @ 12" c/c

10. DOWN-STREAM FLANK WALL



$\phi = 15°$
$\gamma_{sat} = 115$ lb/□'
Overturning Force = 6650 lb
Overturning Moment = 30590 ft-lb
Stabilising Force = 15802 lb
Stabilising Moment = 136600 ft-lb
Maxm pressure 1193 lb/□'
Minm pressure 1146 lb/□'

Design of Toe (18" Thick)



Moment at A = 12,100 ft-lb
A_s = 0.59 □"; Provide 3/4" φ @ 8" c/c
Temp. & Shrinkage reinf. = 18 × 12 × 0.0025 = 0.54 □"
Each face 0.27 □"; provide 1/2" φ @ 9" c/c

Design of Heel (18" thick)

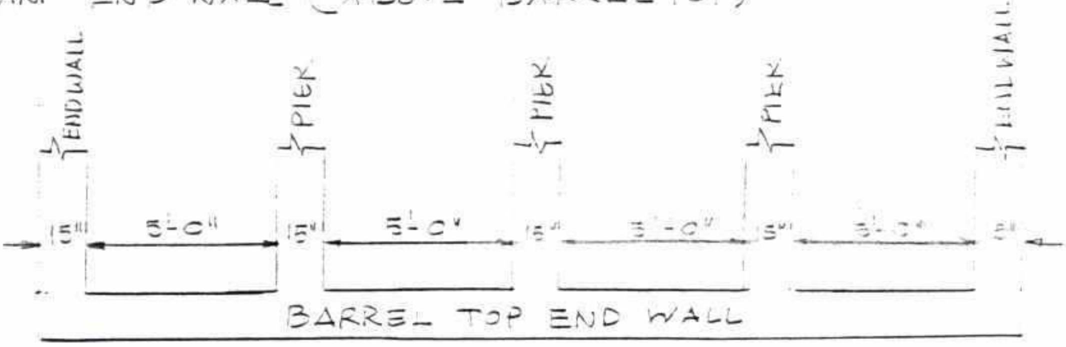
Moment at B = 12679 ft-lb
A_s = 0.62 □"; provide 3/4" φ @ 8" c/c

Design of Stem (18" thick)

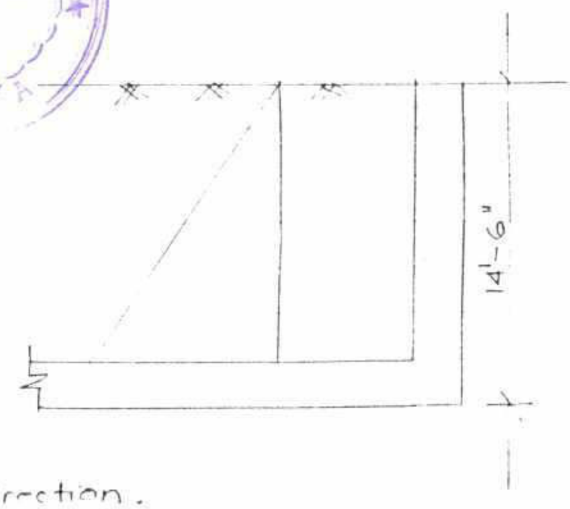
Total Earth Pressure = 5300 lb
Moment at B = 24,300 ft-lb
A_s = 1.13 □'; provide 3/4" φ @ 4" c/c
Temp. & shrinkage reinforement = 1/2" φ @ 9" c/c

11. DOWN-STREAM END WALL (ABOVE BARREL TOP)

$\phi = 15°$
$\gamma_{sat} = 115$ lb/□'

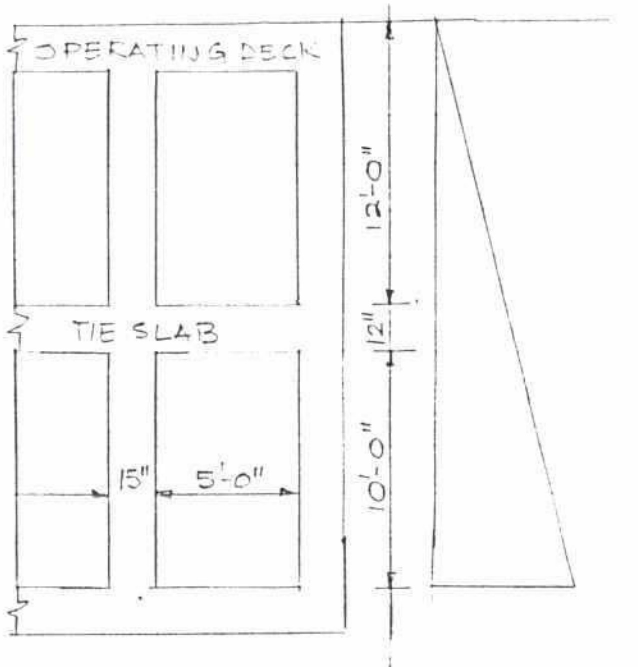


Earth Pressure



Max^m Earth pressure = 984 lb/□'
Max^m -ve moment co-efficient = 0.107
Max^m +ve moment ,, = 0.077
Max^m Moment = 2632 ft-lb
As = 0.2 □"
Temp. and shrinkage renf. = 12x12x0.0025
= 0.36 □"; each face 0.18 □"
Provide 1/2"φ @ 12" c/c in each face both direction.

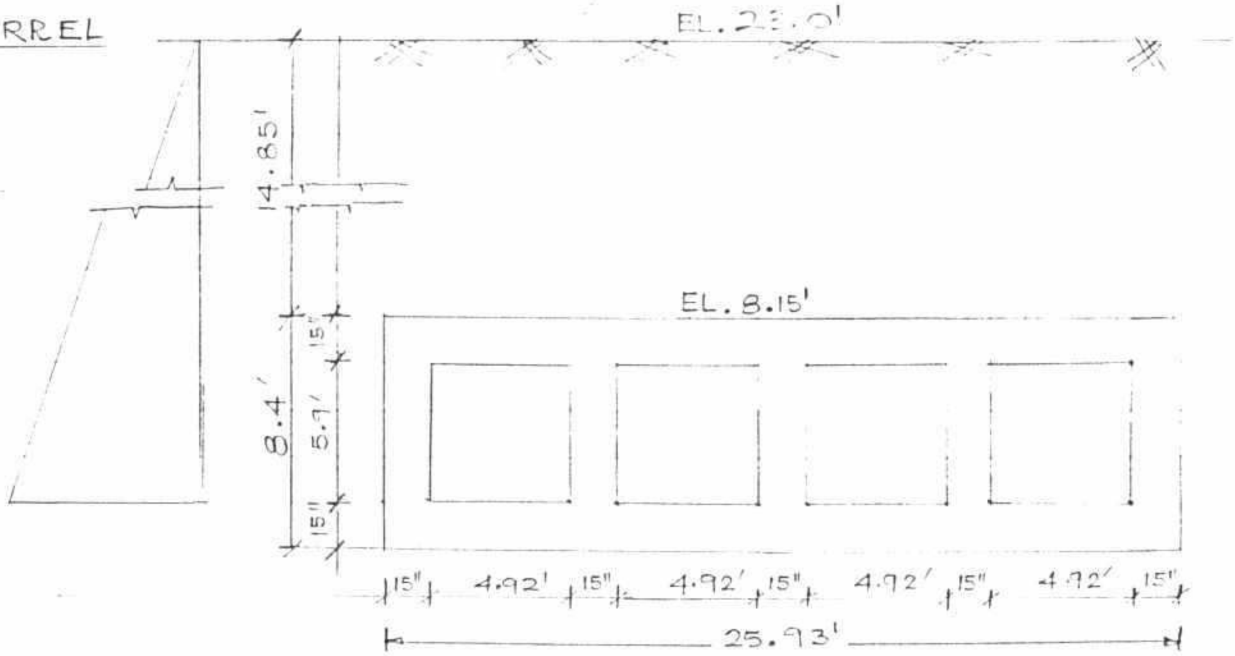
12. WING WALL (ATTACHED TO BARREL)



$\phi = 15°$
γ_{sat} = 115 lb/□'
Max^m Earth Pressure = 1628 lb/□'
Earth pressure at a depth of 13' = 882 lb/□'
Calculate Bending Moment for the lower span from average earth pressure. Average earth pressure = (1628+882) x 1/2 = 1255 lb/□'

Max^m -ve moment co-efficient = 0.125
Max^m +ve " " = 0.100
Max^m -ve moment = 18982 ft-lb
Max^m +ve ,, = 15185 ft-lb
-As = 1.2 □" ; Provide 3/4"φ @ 4" c/c
+As = 1.0 □" ; Provide 3/4"φ @ 5" c/c
Provide 3/4"φ @ 8" c/c in the Top span
- Temp. and shrinkage reinforcement = 15x12x0.0025 = 0.45 □"
each face 0.225 □" ; Provide 1/2"φ @ 10" c/c
- Tie slab (12" Thick)
Provision of Temp. and shrinkage bar will keep the slab in position
Temp. bar = 12x12x0.0025 = 0.36 □" - each face 0.18 □"; provide 1/2"φ @ 1[illegible]
- Bottom slab (26" thick)
Temp. bar = 26x12x0.0025 = 0.78 □"
Each face 0.39 □" ; provide 3/4"φ @ 12" c/c OR 1/2"φ @ 6" c/c

13. BARREL



Load on barrel = 1708 lb/□'

self wt. of the culvert = 15,255 lb/ft or 588 lb/□'

Load on the Top slab = 1896 lb/□'

Load on the foundation = 1708 + 588 = 2296 lb/□'

Net reaction on the bottom slab = 2071 lb/□'

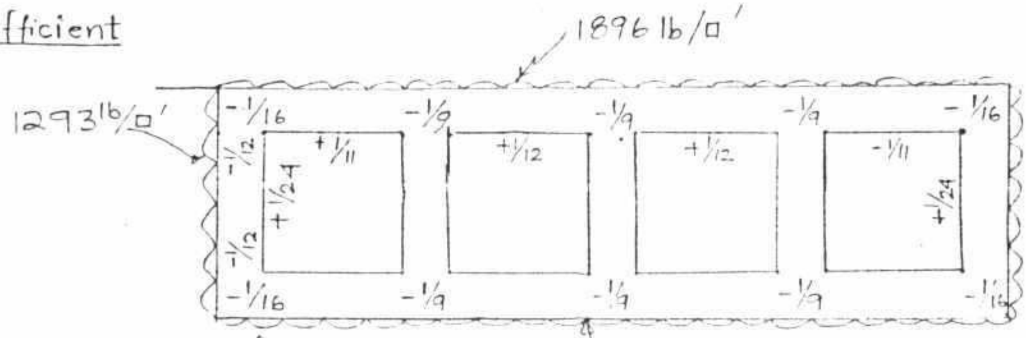
Lateral earth pressure on the side wall —

At a depth of 22.0 ft = 1493 lb/□'

At a depth of 16.1 ft = 1092 lb/□'

Average lateral earth pressure on the side wall = 1293 lb/□'

Moment Co-efficient



Moment :

Top slab :

Max. -ve moment = 5266 ft-lb

Max. +ve " = 4309 "

Bottom slab :

Max -ve moment = 5753 ft-lb

Max +ve " = 4707 ft-lb

Side wall

-ve moment = 3879 ft-lb

+ve moment = 1940 ft-lb

4. OPERATING DECK (12" thick)

A. Distributed load

Wt. of the slab = 150 lb/□' = 250 lb/□'

Live Load = 100 lb/□'

250 lb/□'

B. Concentrated load

Self wt. of the Gate = 400 lb (assumed)

Pull on the hoist = 2570 lb friction factor = 0.7

Head differenc = 2'

Total = 2970 lb,

Say 3000 lb 1500 lb on either side say 2000 lb

Moment

Distributed load

Max. +ve moment = 481 ft-lb

Max. -ve " = 669 ft-lb

Concentrated load

Max. +ve moment = 1690 ft-lb

Max. -ve moment = 1610 ft-lb

Total Moment

+ve moment = 1690 + 481 = 2171 ft-lb

-ve moment = 1610 + 669 = 2279 ft-lb

As = 0.27 □" 1/2" φ 8" c/c

Temp. and shrinkage bar = 12x12x0.0025 = 0.36 □"

each face 0.18 □"; provide 1/2" φ @ 12" c/c

Slab Thickness and Reinforcement.

Top slab (15")

+ve As = 0.27 □" 1/2" φ @ 8" c/c

-ve As = 0.33 □"

Bottom slab (15")

+ve As = 0.30 □" 1/2" φ @ 8" c/c

-ve As = 0.36 □"

Side wall (15")

+ve As = 0.12 □" Min^m reqd. is temp. bar 1/2" φ @ 8" c/c

-ve As = 0.25 □"

Inner wall (15")

Temp. bar = 15x12x0.0025 = 0.45 each face 0.225 □"

1/2" φ @ 10" c/c

15. FOUNDATION :

Foundation soil bearing parameter

Bore Hole	Ground Elevation	Barrel Base	SPT Value at Barrel Base	Allowable Bearing Capacity at Barrel Base
32/A	24'-0"	0.0	5	0.75 T/□' or 1680 lb/□'
33/A	23'-0"	0.0	6	0.90 T/□' or 2016 lb/□'

Allowable bearing values from "Soil Mechanics & Engineering Practice" - by Terzaghi & Peck

Soil Bearing Capacity

Bore Hole	Foundation Pressure	Allowable Bearing Capacity at Barrel Base	Remarks
32/A	2296 lb/□'	1680 lb/□'	Needs Treatment
33/A	- do -	2016 lb/□'	- do -

Foundation Treatment

Barrel Base Width = 7.92 m   Effective distribution angle : 30°

Depth of sand Filling	Foundation width after Treatment	Bore Hole No.	SPT Value at 1m below Barrel base	Allowable Soil Bearing Capacity at the Base of Sand Filling	Foundation Pressure at Barrel Base	Foundation Pressure at Base of sand Filling
4.1' (1.25m)	30.6' (9.33m)	32/A	6	0.9 T/□' or 2016 lb/□'	2296 lb/□'	1950 lb/□'
		33/A	7	1.05 T/□' or 2352 lb/□'		

RE-DESIGN OF FOUNDATION ON THE BASIS OF LABORATORY SOIL TEST RESULTS

FOUNDATION

Soil Bearing Parameter

Bore Hole	SPT at Sand filling Base	Cohesion from Soil sample	Remarks
32/A	3	22.5 KN/m² 472 lb/□'	
33/A	5	13.0 KN/m² 273 lb/□'	Doubtful

Ultimate Bearing Capacity: Unconfined compression test has not been performed. Cohesion, as determined from the soil sample of the bore hole 32/A, has been used for determining the bearing capacity of the foundation soil. The formula (Foundation Design by Teng Page 121) is :-

$$q_{ult} = cN_c$$

N_c for square footing when D/B is 1. ($D = 27'$ & $B = 26'$) $= 7.8$

Therefore, N_c (for rectangle) $= (0.84 + 0.16\ B/L)\ N_c$ (Square)

$$= (0.84 + 0.16 \times 26/33) \times 7.8 = 7.6$$

$q_{ult} = 472 \times 7.6 = 3587$ lb/□'

Factor of Safty:

The factor of safety of the soil bearing pressure varies from 3 to 2. In this case 2 is the factor of safety because,

1. The load for which the foundation is designed may not likely to develop.
2. Additional settlement beyond 1" is not harmful for the structure because it is divided into several segments by contraction joints.
3. Sub-soil dewatering will increase the soil bearing capacity.

ALLOWABLE SOIL BEARING CAPACITY:

Allowable Soil Bearing Capacity

= Ultimate Bearing Capacity ÷ 2

= 3587 ÷ 2

= 1793 lb/□'

Foundation Treatment

Depth of Sand Filling	Foundation width after Treatment	Foundation Pressure		Allowable Soil Bearing Capacity at the Sand Filling Base
		Barrel Base	Sand Filling Base	
6.5 ft.	33.5 ft.	2296 lb/□'	1782 lb/□'	1793 lb/□'

Mahale

YB

DESIGN REPORT

ON

PIPE SLUICE AT D/S OF PROGOTI SARUNI BRIDGE

CONTENTS

PIPE SLUICE
AT
D/S OF PROGOTI SARONI BRIDGE

DESIGN DISCHARGE

AREA = 28 hectares

Rainfall duration = 6 hr.

Max. 6 hourly Rainfall = 135 mm
(2 year return Period)

Rainfall Distribution

1st hr = 9% = 12.1 mm
2nd hr. = 15% = 20.3 mm
3rd hr. = 44% = 59.4 mm
4th hr = 16% = 21.6 mm
5th hr = 9% = 12.1 mm
6th hr. = 7% = 9.5 mm

$Q(2nd\ hr) = 0.6 \times 20.3 \times 28 \times \frac{1}{360} = 0.94\ m^3/sec$

$Q(3rd\ hr) = 0.6 \times 59.4 \times 28 \times \frac{1}{360} = 2.77\ m^3/sec$

$Q(4th\ hr) = 0.6 \times 21.6 \times 28 \times \frac{1}{360} = 1.0\ m^3/sec$

Design Discharge = Average of the 2nd, 3rd and 4th hourly rainfall run-off

$= (0.94 + 2.77 + 1.0) \div 3 = 1.55\ m^3/sec.$

or 54.7 Cusec.

PIPE SIZE

Assumption:

Pipe Diameter = 1200mm

Sill EL. = 3.6 m

Head Difference, Δh = 0.15m

Running full

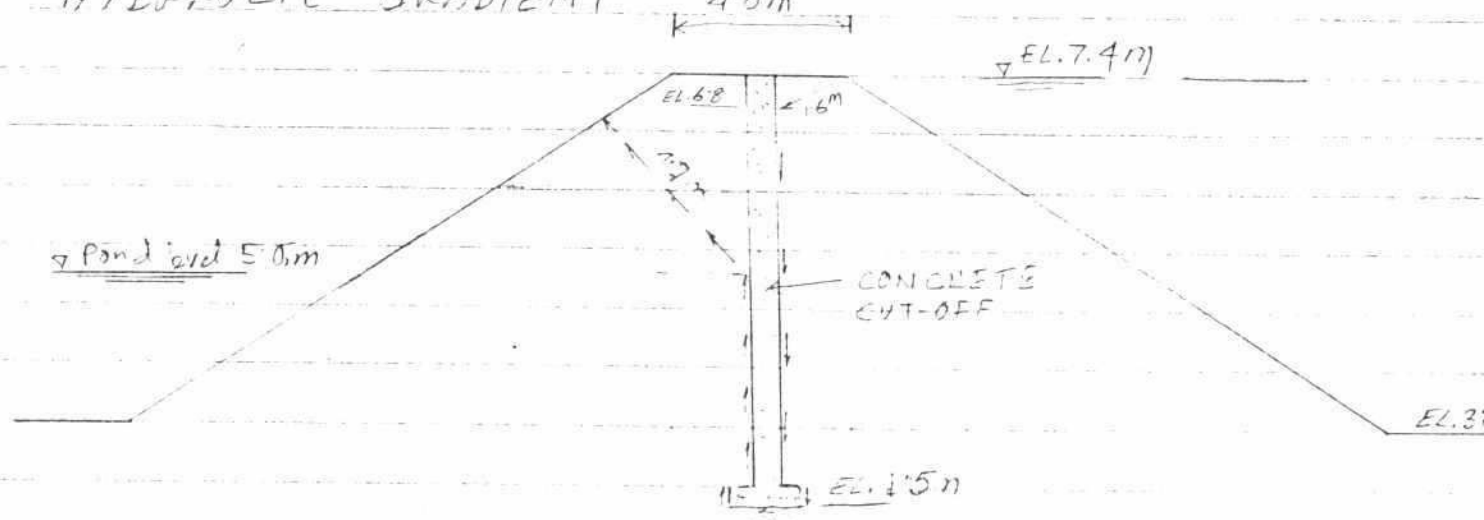
Discharge co-efficient = 0.9

Area = 12.5 □' Hydraulic Radius, R = 1

$v = c\sqrt{2gh} = 0.9\sqrt{64.4 \times .5} = 5.1$ ft/sec.

$Q = 5.1 \times 12.5 = 63.75$ cusec. > 54.7 cusec.

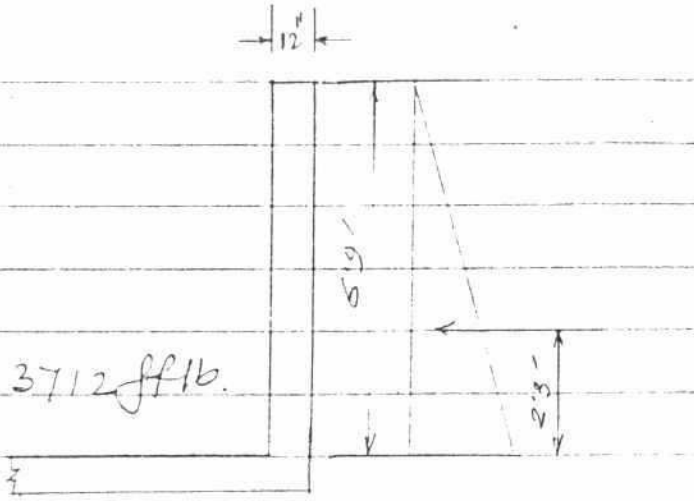
HYDRAULIC GRADIENT



Weighted length of Creep = 5.3 + 3.5 + 4.7 + 1.6
= 14.9

Weighted Creep ratio = 14.9/2.4 = 6.2 > 6 Safe

UP-STREAM WING WALL



$\theta = 15^\circ$

$\gamma_{sat} = 115$ lb/cft.

$P = 0.59 \times 115 \times 6.9 = 468$ lb/□'

$P_A = \frac{1}{2} \times 468 \times 6.9 = 1614$ lb.

Overturning Moment $= 1614 \times 2.3 = 3712$ ft lb.

$d = \sqrt{\frac{3712}{189}} = 4.43$

Provide 12" thick wall

$A_s = \frac{3712 \times 12}{18000 \times .88 \times 9} = 0.31$ □"

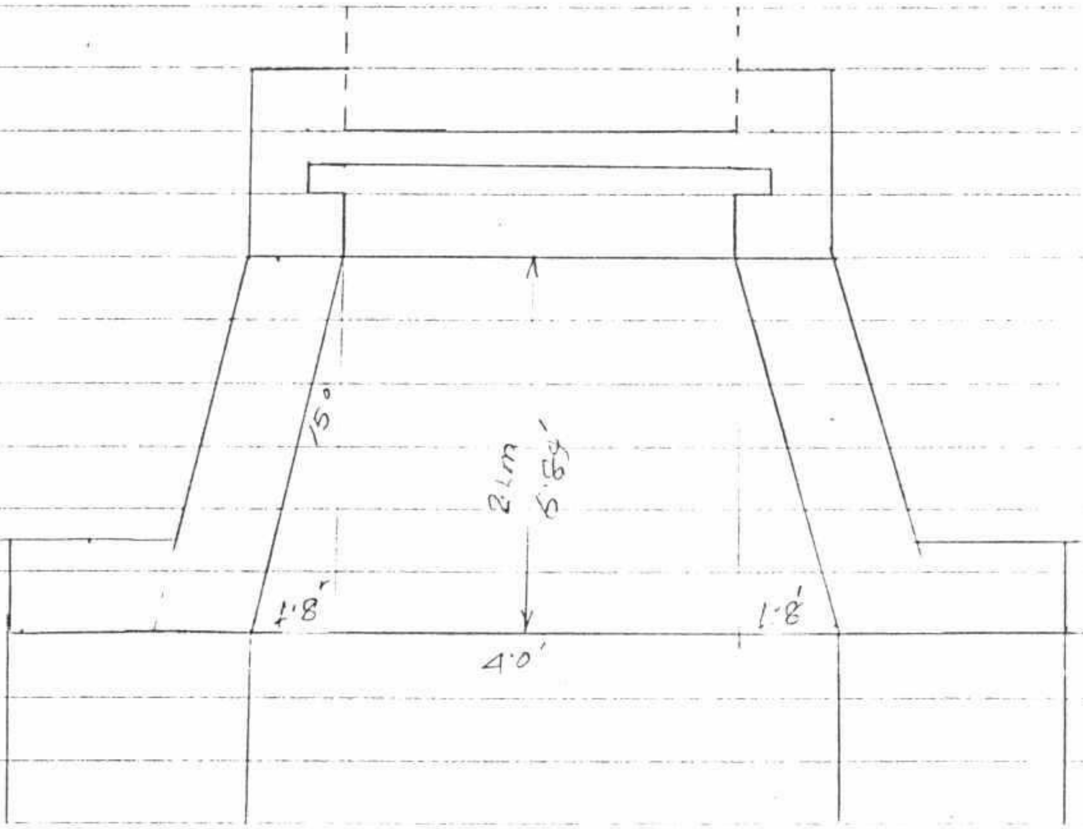
Provide ½" ∅ @ 8" c/c

Temp. reinforcement $= 12 \times 12 \times 0.0025 = 0.36$ □"

each face $= 0.18$ □"

provide ½" ∅ @ 12" c/c

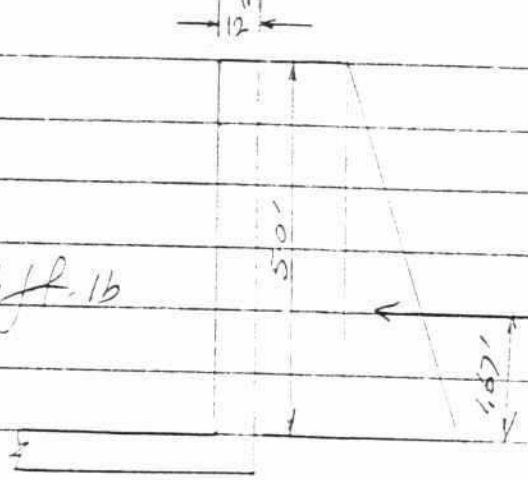
FLARING OF THE BASIN



width at the end of the basin	= 7.6 ft
Depth of the flow	= 1.6m (5.25 ft)
Design Discharge	= 54.7 cusec (1.55 m³/sec

Therefore, velocity at the end of the basin $= \frac{54.7}{7.6 \times 5.25} = 1.37$ ft/sec

DOWN STREAM WING WALL



$P = 0.59 \times 115 \times 5 = 339$ lb/ft'

$P_A = \frac{1}{2} \times 339 \times 5 = 848$ lb

overturning moment $= 848 \times 1.67 = 1416$ ft.lb

$$A_s = \frac{1416 \times 12}{18000 \times .88 \times 9} = 0.12$$

Provide $\frac{1}{2}$" ϕ @ 12" c/c

Temp reinforcement $\frac{1}{2}$" ϕ @ 12" c/c

DESIGN REPORT

ON

SLUICE AT DOWN STREAM OF RLY. BRIDGE NO. 39

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CONTENTS

PIPE SLUICE AT D/S OF RLY. BRIDGE NO. 39.

1. Design Discharge

Catchment area = 4.5 km² = 450 ha (as determined from field survey & topo map)

Return period = 2 years
Duration of rainfall = 6 hours
Rainfall of 6hr. duration on 2yr. return period = 135 mm (ref: Main report on the updating study on storm water drainage system improvement project in Dhaka City, Feb 1990 by JICA)

Hourly distribution of rainfall

1st hr. = 9% = 12.1 mm
2nd hr. = 15% = 20.3 mm
3rd hr. = 44% = 57.4 mm
4th hr. = 16% = 21.6 mm
5th hr. = 9% = 12.1 "
6th hr. = 7% = 9.5 "

Peak run off = av. of 2nd, 3rd & 4th hr. rainfall

Rational formula for $Q = \frac{CiA}{360}$

where Q = Peak discharge m³/sec.
C = Run off co-efficient = 0.6
i = Rainfall intensity mm/hr.
A = Drainage area in hectre

Q (2nd hr.) $= 0.6 \times 20.3 \times 450 \times 1/360 = 15.23$ m³/sec
Q (3rd hr.) $= 0.6 \times 57.4 \times 450 \times 1/360 = 44.55$ m³/sec
Q (4th hr) $= 0.6 \times 21.6 \times 450 \times 1/360 = 16.20$ m³/sec.

Design discharge of the drainage outlet

$= (15.23 + 44.55 + 16.20)/3 = \underline{25.33 \text{ m}^3/\text{sec.}}$

Pipe size & Number

H.F.L. 50 yr. return period = 7.50 M. PWD (Interpolation of levels betn. Tongi & Demra)

Downstream water level during monsoon
= 5.5 + 0.5 = 6.0 M. PWD (av. of 10 yr's H.F.L during August at Demra + 0.5 M)

Downstream water level in pre monsoon
= 2.57 + 0.50 = 3.07 m PWD (10 yrs avg. in end May + 0.5 M)

Sill elevation = 2.0 M PWD.

Assuming 1500 mm dia pipe, $r = \frac{1.5}{4} = 0.375$

Co-efficient of discharge

$$C_d = \left[1 + 0.4(r)^{0.3} + \frac{.0045 \times L}{r^{1.25}}\right]^{-1/2}$$

$$= \left[1.298 + \frac{.0045 \times 10.67}{0.375^{1.25}}\right]^{-1/2}$$

$$= (1.298 + 0.164)^{-1/2} = 0.827 \approx 0.83$$

Max. permissible internal pond level
= 6.50 M PWD (Safe level against inundation of rly. embkt. c/s land)

Min. head difference
$h = 6.50 - 6.0 = 0.5$ M

Velocity $v = C\sqrt{2gh} = 0.83\sqrt{2 \times 9.8 \times 0.5} = 2.60$ m/sec.

Q per pipe $= Av = 1.77 \times 2.60 = 4.60$ m³/sec.

No. of pipes $= \frac{25.33}{4.60} = 5.51$ say 6 nos.

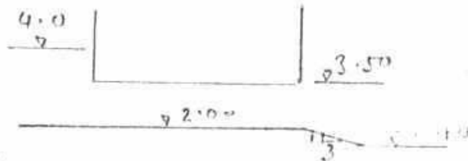
(3)

Stilling Basin :

From energy loss consideration in hydraulic jump

$Q = 25.33$ cumec

Flow width $= 1.5 \times 6 + 0.4 \times 5 = 11$m



$q = \frac{25.33}{11} = 2.3$ cumec / m width.

critical depth $y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{2.3^2}{g}\right)^{1/3} = 0.816$ m.

Neglecting velocity head, loss of energy (H_L) (Ref: Garg pg. 450)

$$H_L = 4.0 - 3.50 = 0.50$$

From Blench curve, $Ef_2 = 1.40$

$\therefore Ef_1 = H_L + Ef_2 = 0.5 + 1.40 = 1.90$

From energy of flow curve

$$y_1 = 0.4 \quad \& \quad y_2 = 1.85$$

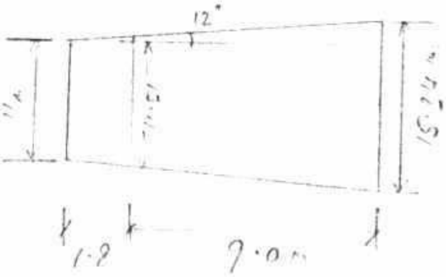
length of cistern $6(y_2 - y_1) = 6(1.8 - 0.4) = 8.7$m
say 9m.

Desired apron level $=$ D/S W.L $- y_2$

$= 3.50 - 1.85 = 1.65$ m PWD.

Apron to be lowered by min $(2.0 - 1.65) = 0.35$ M

Again, $B_1 = 11.51$m.



$q_1 = \frac{Q}{B_1} = \frac{25.33}{11.86}$

$= 2.15$ cumec / m

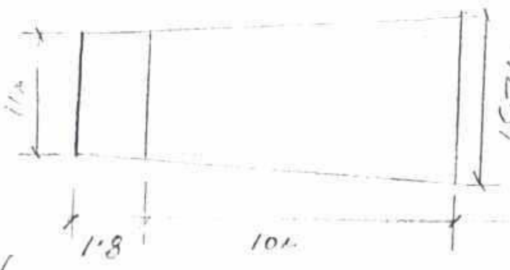
$V_1 = \frac{q_1}{y_1} = \frac{2.15}{0.40} = 5.38$ m/sec

$F_1 = \frac{5.38}{\sqrt{g \times 0.40}} = 2.72$

From curve, $\frac{L}{y_2} = 5.3$, $\therefore L = 5.3 \times 1.85 = 9.81$m
say 10m.

discharge at the end of stilling basin,

$$q_3 = \frac{25.33}{15.76} = 1.61\ m^3/sec\ per\ width$$



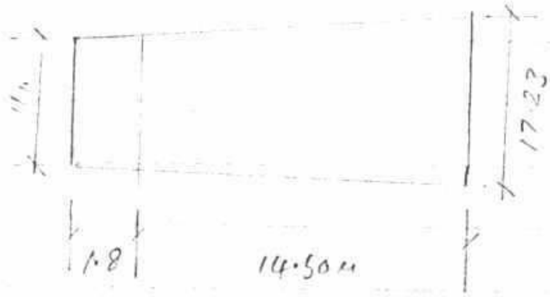
$$V_3 = \frac{1.61}{1.85} = 0.87\ m/sec.$$

Allowing max. permissible velocity of 0.8 m/sec

∴ basin width reqd. at the end of stilling basin $= \frac{25.33}{1.85 \times 0.8} = 17.11\ m$

& Basin length, $L = (17.11 - 11) \times 0.5 \times \frac{1}{\tan 12^\circ}$

$= 14.41\ m$



CHUTE BLOCK

Ht. of the block = D_1 = 0.40 m provided.

end gap betn. wing walls block = $\frac{D_1}{2} = \frac{0.4}{2}$ = 0.20 m

Thickness of the block = $0.75D_1$ = 0.7 × .4 = .30

gap betn. block. = $0.75D_1$ = 0.30

BASIN BLOCK

Ht. of block = D_1 = 0.40

Thickness of block = $.75D_1$ = 0.30

gap between block. = $.75D_1$ = 0.30

Crest width = $.2D_1$ = .2 × .4 = .08 = 80 mm

END SILL

Height of Sill = $.2D_2$ = .2 × 1.85 = 0.370 = 400

Thickness = $.15D_2$ = .15 × 1.85 = .278 = 300

Gap = $.15D_2$ = ≃ 300

Crest = .2 × 400 = 80 mm.

Distance betn. block & basin block = 0.8 D_2

= 0.8 × 1.55 = 1.48 m.

Scour depth :

$Q = 25.33\ m^3/sec.$

$B = 18.50\ m \qquad q = \frac{25.33}{18.50} = 1.37\ m^3/sec$

$d_m = 0.02\ m$

$f = 1.76\sqrt{d_m} = 0.249$

Depth of scour, $R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35 \times \left(\frac{1.37^2}{.249}\right)^{1/3} = 2.63\ m$

El. of D/S cut off bottom $= D/S.\ w.l - 1.5 \times R$

$= 3.5 - 1.5 \times 2.63$

$= -0.445\ m.\ PWD.$

Hydraulic gradient :

50 yr. flood stage el. = 7.50 m PWD.

c/s inundation level = 5.50 " "

∴ Head diff. = 2.0 m.

Weighted creep length (after Lane's)

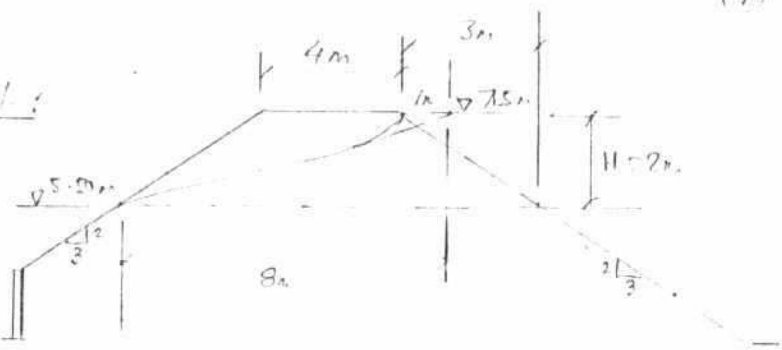
$= 1 \times 2 + (4 + 11.46 + 16.5) \times \frac{1}{3} + 1.95 \times 2$

$= 15.38$

Weighted creep ratio $= \frac{15.38}{2} = 7.69 > 3.0$ for clayey soil ok.

(7)

<u>Seepage through embankment:</u>
(without core wall)



$S = \sqrt{2^2 + 8^2} - 8$

$= 0.246$ m.

$k = 5 \times 10^{-7}$ m/sec
for clayey soils (Ref. Bowles')

$q = k \times S = 5 \times 10^{-7} \times 0.246 = 1.23 \times 10^{-7}$ cumec/m length of embkt.

Travel distance of flow $= \sqrt{2^2 + 7^2} = 7.28$ m

Time reqd. to reach the D/S face

$= 7.28 \div (5 \times 10^{-7})$

$= 1.456 \times 10^7$ sec $= 169$ days.

Peak flood duration max. 21 days. < 169 days, ok.

STRUCTURAL DESIGN

Up stream wing wall;

Assumption $\phi = 15°$

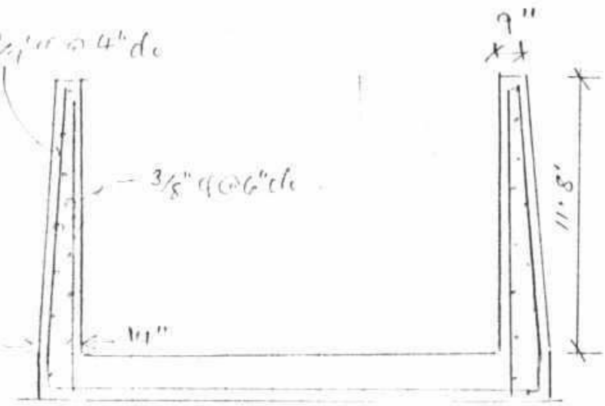
$\gamma_{sat} = 115 \text{ lb/ft}^3$

$K_a = \dfrac{1 - \sin\phi}{1 + \sin\phi}$

$= \dfrac{1 - 0.259}{1 + 0.259} = 0.59$

earth pressure $P = K_a \times \gamma_{sat} \times h$

$= 0.59 \times .115 \times 11.8' = 0.8 k/ft'$



$P_A = \frac{1}{2} \times 11.8 \times 0.8 = 4.7^k$

Overturning moment $= 4.7 \times \dfrac{11.8}{3} = 18.49^{k'}$

$d = \left(\dfrac{18.49}{0.189}\right)^{.5} = 9.89'' \approx 10''$

Provide 14" wall

$A_s = \dfrac{18.49}{1.31 \times 11} = 1.28 in^2$, 3/4"φ @ 4" c/c

Temp. & Shrinkage reinf $= .0025 \times 12 \times 14$

$= 0.42 in^2$

3/8"φ @ 6" c/c ea. face.

Assumption

$f'_c = 2500$ psi

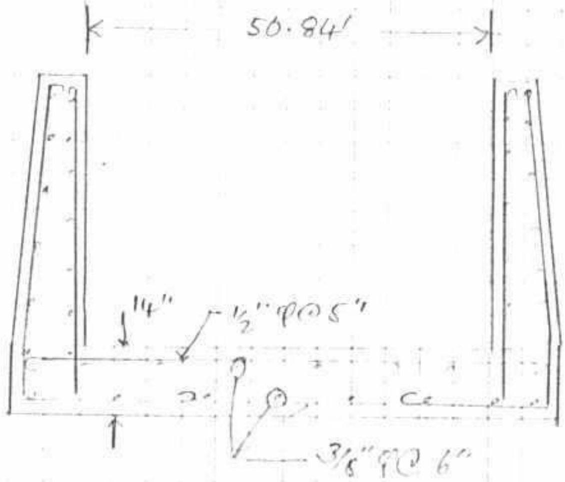
$f_c = 0.45 f'_c$

$R = 189$

$k = 0.389$

$j = 0.87$.

Upstream Apron Slab



Load from soil

$= \frac{1}{2} \times 11.8 \times \frac{5}{12} \times .115$

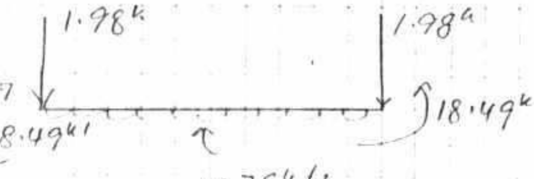
$= 0.28^{k/'}$

Load from wing wall stem

$= \frac{9+14}{2 \times 12} \times 11.8 \times .15$

$= 1.696^{k/'}$

Total load $= 0.28 + 1.696 = 1.976^{k/'}$



$q_{net} = \frac{1.98 \times 2}{50.84} = 0.078^{k/'}$

Mid span moment

$= 18.49 - 1.98 \times \frac{50.84}{2} + 0.078 \times \frac{50.84^2}{8}$

$= 18.49 - 50.33 + 25.20 = -6.64^{k'}$ Top tension.

Considering uplift, slab thickness:

$t = 1.33 \frac{h}{(G-1)}$

$= 1.33 \frac{0.12}{(2.4-1)}$

$= 0.114 < .350$ m ok

$h = \frac{H_2}{C}$, $(H_2 = 7.5 - 5.5 = 2.0\text{M})$

$= \frac{2}{17.83 - \frac{3.63}{3}}$

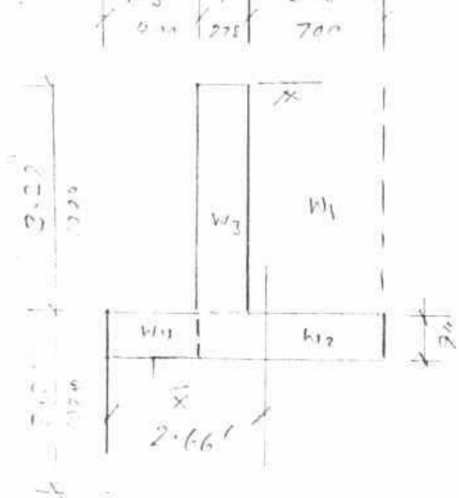
$= 0.12$

$(G = 2.4$ Sp. gr. of concrete$)$

$A_s = \frac{6.64}{1.31 \times 11} = 0.46$ in², 1/2" φ @ 5" c/c

Tempr. & distn. $= .0025 \times 14 \times 12 = 0.42$ in², 3/8" φ @ 6" c/c.

Upstream Flank wall:



$P = K_a \times \gamma_{sat} \times h$

$= 0.59 \times .115 \times 3.28 = 0.223^k$

$P_A = 0.223 \times 3.28 \times \frac{1}{2} = 0.37^k$

Overturning Moment

$= 0.37 \times \frac{3.28}{3} + .75 = 0.50$

Stabilising load & Moment:

	Force	Moment Arm	Moment k'
$W_1 = 2.3 \times 3.28 \times 1 \times .115 =$	0.87^k	3.2'	2.78
$W_2 = \frac{9}{12} \times 3.05 \times 1 \times .15 =$	0.343	2.82	0.97
$W_3 = \frac{9}{12} \times 3.28 \times 1 \times .15 =$	0.37^l	1.68'	0.622
$W_4 = \frac{9}{12} \times 1.3 \times 1 \times .15 =$	0.146^k	0.65'	$0.095^{k'}$
	$\Sigma V_s = 1.729$	ΣM_r	4.47^k

$F.S. = \frac{4.47}{0.5} = 8.93 > 1.5$ OK.

$\Sigma M = M_r - M_o = 4.47 - 0.4 = 4.07$

$\bar{x} = \frac{\Sigma M}{\Sigma V_s} = \frac{4.07}{1.58} = 2.58$ from toe.

$e = \frac{4.35}{2} - 2.58 = -0.405$

Pressure $P = \frac{\text{Stabilising force}}{\text{Width of footing}} \left(1 \pm \frac{6 \times e}{\text{Width of footing}}\right)$

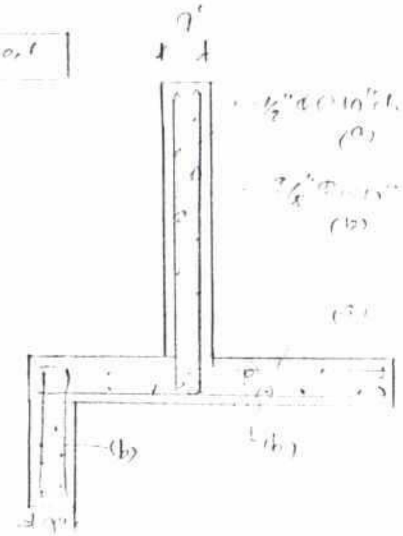
$= \frac{1.73}{4.35} \left(1 \pm \frac{6 \times 0.405}{4.35}\right)$

$= 0.397 \times (1 \pm 0.56)$

$P_{max} = 0.62^{k}$

$P_{min} = 0.175^{k}$

Design of toe



Moment at A $= 0.62 \times 1.3 \times \frac{1.3}{2}$

$= 0.524^{k'}$

$d = \left(\frac{0.524}{.189}\right)^{.5} = 1.66''$

$A_s = \frac{0.524}{1.21 \times 6} = 0.06 \text{ in}^2$ 3/8"φ@9" c/c

Design of keel

Moment at B $= 0.81 \times \frac{2.3}{2} + (\frac{?}{12} \times$ [illegible]$) \times \frac{2.3}{2}$

$= 1.00 + 0.0$[illegible] $= 1.29^{k'}$

$d = \left(\frac{1.30}{.189}\right)^{.5} = 2.62''$

$A_s = \frac{1.30}{1.21 \times 6} = 0.165 \text{ in}^2$ 3/8"φ@3" c/c

Temp. & shr. $A_s = .0025 \times 9 \times 12 = 0.27 \text{ in}^2$

3/8"φ @ 10" c/c each face

Thickness of stem wall has been kept same as top of wingwall.

Cut off has been adopted at toe up to the length with same thickness

Design of stem

Moment at C $= 0.37 \times \frac{2.25}{2} = 0.416^{k'}$

$d = \left(\frac{0.416}{.189}\right)^{.5} = 1.45''$

$A_s = \frac{0.416}{[illegible]} = 0.0$[illegible] in^2 3/8"φ@ 10" c/c

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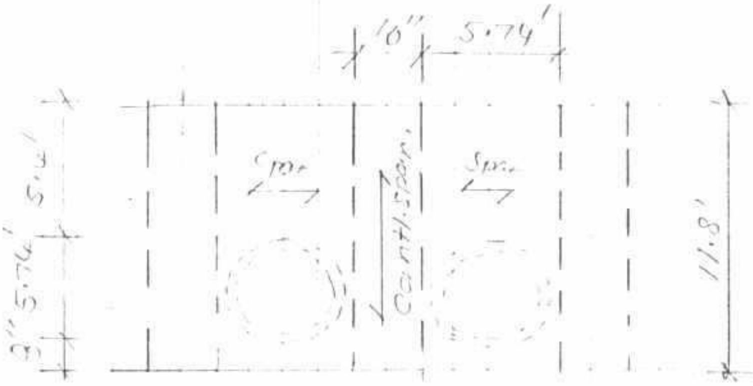
Up stream end wall :



P_a at pipe top

$= 0.59 \times .115 \times 5.4$

$= 0.367\ k/ft^2$

considering 1.33 times of an. load acting on wall as distributed load

$= 1.33 \times \frac{1}{2} \times .367 = .244\ k/ft^2$

$M = 0.244 \times 5.74^2 \times \frac{1}{9} = 0.89^{k'}$

$d = \left(\frac{.89}{.189}\right)^{.5} = 2.17''$

$A_s = \frac{.89}{1.31 \times 6.5} = 0.10\ in^2$

3/8" φ @ 9" ea. face hor.

Temp. & dist. bars $= .0025 \times 9 \times 12$

$= 0.27\ in^2$ 3/8" φ @ 10" c/c ea. face vert.

Load on cantl. wall from wall above pipe,

$= \frac{1}{2} \times .367 \times 5.33 = 0.978^k$

load on cantl. wall from earth load

$= (0.59 \times 11.8 \times .115) \times \frac{1}{2} \times 11.8$

$= 4.72^k$

M at base $= 4.72 \times \frac{10}{12} \times \frac{11.8}{3}$

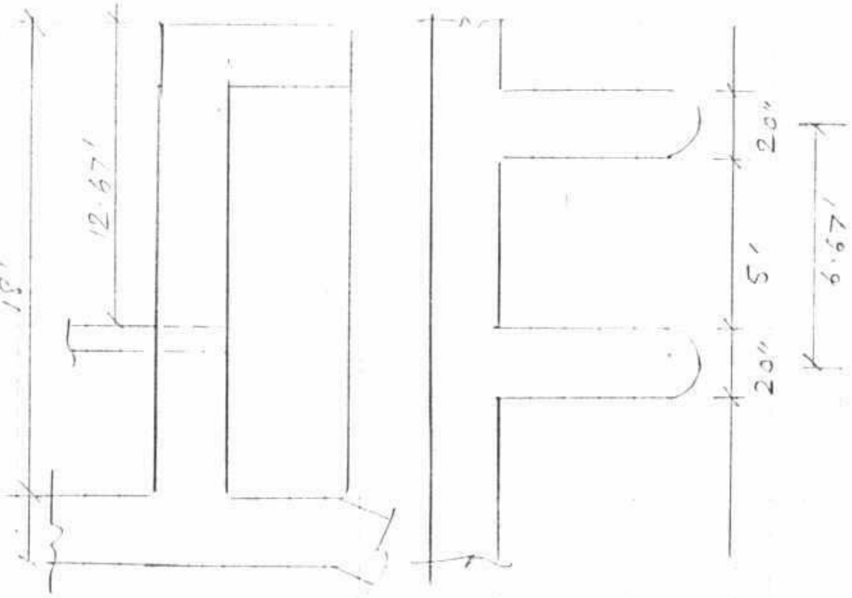
$+ 0.978 \times \left(11.8 - \frac{5.4}{3}\right) \times 5.74$

$= 15.47 + 56.14 = 71.61^{k'}$

$d = \left(\frac{71.61 \times 12}{.189 \times 10}\right)^{.5} = 21.32''$

$A_s = \frac{71.61}{1.31 \times 21.5} = 2.54\ in^2$ 6 - 3/4" φ or 5 - 7/8" φ

Down Stream End Wall



Earth load at base of wall above pipe top

$= 0.57 \times .115 \times 12.67$

$= 0.86^{k/'}$

Considering 1.33 times of ave. load acting on walls dist. load,

$1.33 \times \frac{1}{2} \times .86 = .57^{k/'}$

$-ve\ M = .57 \times 5^2 \times \frac{1}{9}$

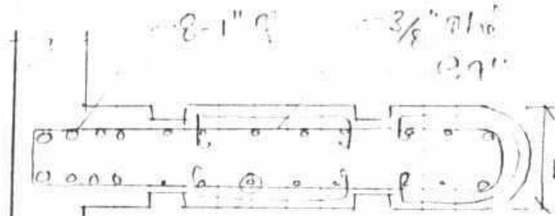
$= 1.59^{k'}$

$d = \left(\frac{1.59}{.185}\right)^{.5} = 2.9"$ provide 10" thick wall

$A_s = \frac{1.59}{1.31 \times 7} = 0.173 in^2$ ½" φ @ 10" c/c each face hor.

Dist. $A_s = .0025 \times 12 \times 10 = 0.3 in^2$ ½" φ @ 10" c/c each face vert.

Down Stream piers



Lateral earth load at above pipe $= 0.86^{k/'}$

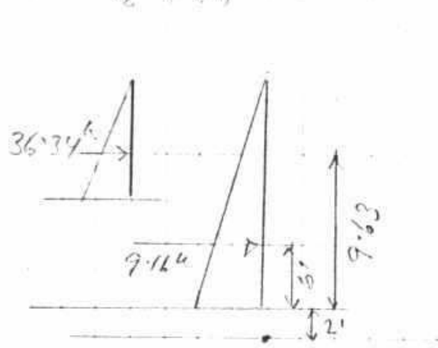
$P = 6.67 \times .86 = 5.74^{k}$

$P_a = \frac{1}{2} \times 5.74 \times 12.67 = 36.34^{k}$

Load on cantl. wall from earth load

$= (.59 \times 18 \times .115) \times \frac{1}{2} \times 18 \times \frac{10}{12}$

$= 9.16^{k}$



M at base $= 36.34 \times 9.63 + 9.16 \times 6$
$= 349.95 + 54.96 = 404.91^{k'}$

$d = \left(\frac{405 \times 12}{.185 \times 10}\right)^{.5} = 50.71"$ provide 4.7 ft deep pier

$A_s = \frac{405}{1.31 \times 57} = 5.42 in^2$ 8-1" φ and 3/8" φ ties @ 9" c/c.

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Overturning Moment at C

$= 36.34 \times 11.63 + 9.16 \times 8$

$= 422.63 + 73.28 = 495.91^{k'}$

$\approx 496^{k'}$

Balancing M:

Soil $= 6.67' \times 12.67' \times 8.36' \times .118 \times 10.58'$

$= 882.0^{k'}$

Pier $\dfrac{20'' \times 72''}{144} \times .15 \times 18' \times 3.2'$

$= 97.4^{k'}$

Endwall $\dfrac{10''}{12} \times 5' \times .15 \times 18' \times 5.93'$

$= 67.39^{k'}$

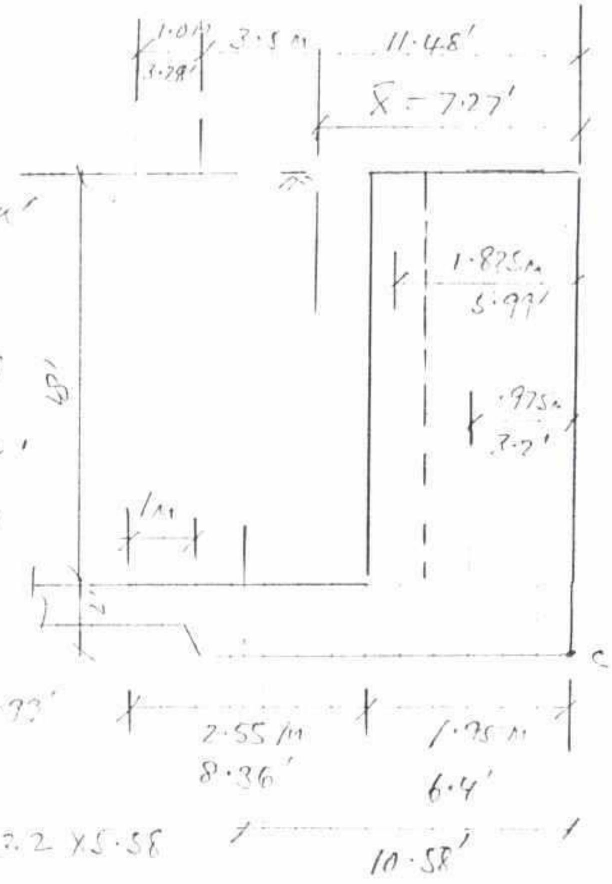
Operating deck $1' \times 5' \times .15 \times 3.2 \times 5.58$

$= 13.39^{k'}$

Foundation slab $2' \times 11.5 \times 6.67 \times .15 \times 5.75$

$= 132.07^{k'}$

Total balancing moment $= 1187.27^{k'}$ ∴ min F.S. $= \dfrac{1187.27}{496} = 2.39$ > 1.5 ok.



ΣV_s

Soil $= \dfrac{882}{10.58} = 83.36$

Pier $= \dfrac{97.4}{3.2} = 28.55$

Endwall $= \dfrac{67.39}{5.75} = 11.25$

Op. deck $= \dfrac{13.39}{5.58} = 2.40$

Fdn. $= \dfrac{132.07}{5.75} = 22.97$

148.86^{k}

$\bar{X} = \dfrac{\Sigma M}{\Sigma V_s} = \dfrac{1187.27}{148.86} = 7.98'$ from c.

$e = \dfrac{4.5m \times 3.28}{2} - 7.98 = -0.6'$ within middle third of ℄ of footing, ok

Pressure on soil

$P = \dfrac{\Sigma V_s}{\text{footing area}} \left(1 \pm \dfrac{6 \times e}{\text{footing area}}\right)$

$= \dfrac{148.86}{14.76 \times 6.67} \left(1 \pm \dfrac{6 \times 1.24}{11.48 \times 6.67}\right)$

$= 1.51 \times (1 \pm .097)$

$= 1.68$ ksf max.

$= 1.36$ ksf min.

Pier foundation

$M_B = 36.34 \times 9.63 + 9.16 \times 6 = 350 + 54.96 = 404.96$

$d = \left(\dfrac{405 \times 12}{.187 \times 6.67 \times 12}\right)^{.5} = 17.97''$

Provide slab thickness = 24"

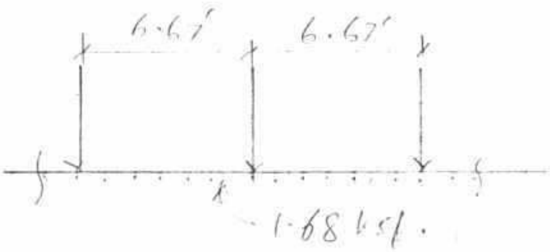
$A_s = \dfrac{405}{1.31 \times 21} = 14.61 \text{ in}^2$

i.e. $\dfrac{14.61}{6.67} = 2.19 \text{ in}^2$ per foot width of slab

3/4" φ @ 7.5in c/c.

$-ve\ M = 1.68 \times 6.67^2 \times \frac{1}{9}$

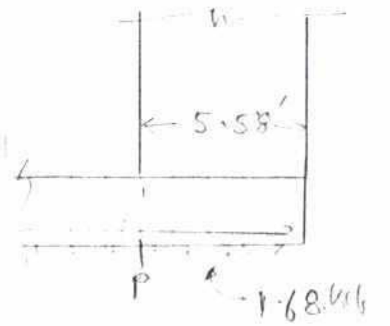
$= 8.30^{k'}$



$d = 21''$

$\therefore A_s = \frac{8.30}{1.31 \times 21} = 0.30\ in^2$

$\frac{1}{2}''\phi$ @ 8" c/c ea face.



M at P

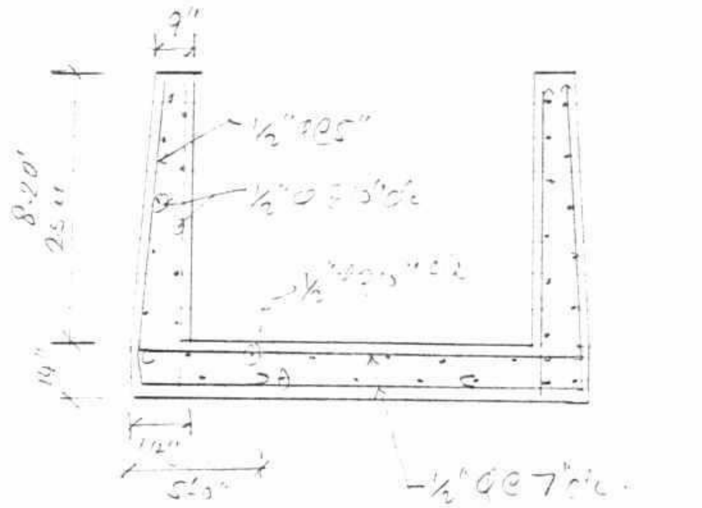
$= 1.68 \times 5.58^2 \times \frac{1}{2}$

$= 26.15^{k'}$

$A_s = \frac{26.15}{1.31 \times 21} = 0.95\ in^2$

$\frac{5}{8}''\phi$ @ 4" c/c

Down Stream wing wall :



earth pressure

$P = 0.59 \times .115 \times 8.20$

$= 0.56^k$

$P_A = \frac{1}{2} \times .56 \times 8.20$

$= 2.28^k$

Moment $= 2.28 \times \frac{8.20}{3}$

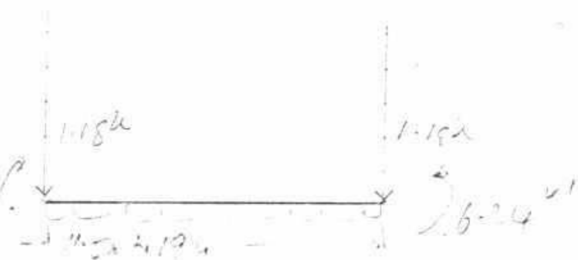
$= 6.24^{k'}$

$d = \left(\frac{6.24}{.189}\right)^{.5} = 5.74"$ Provide 14" thick wall

$A_s = \frac{6.24}{1.31 \times 11} = 0.43\,in^2$ 1/2" φ @ 5" c/c

Temp'l. dist. $A_s = .0025 \times 12 \times 14 = 0.42\,in^2$ 1/2" φ @ 10" c/c

Stilling basin slab :



Load from wing wall $\frac{9+14}{2 \times 12} \times 8.2 \times .115$

$= 1.18^k$

$q_{net} = \frac{1.18 \times 2}{37.72} = 0.063$ k/ft²

Moment at mid span (for 62.22 span)

$= 6.24 - 1.18 \times \frac{62.22}{2} + .063 \times \frac{62.22^2}{8}$

$= 6.24 -$ [illegible] $+ 30.5 = 0.05^{k'}$

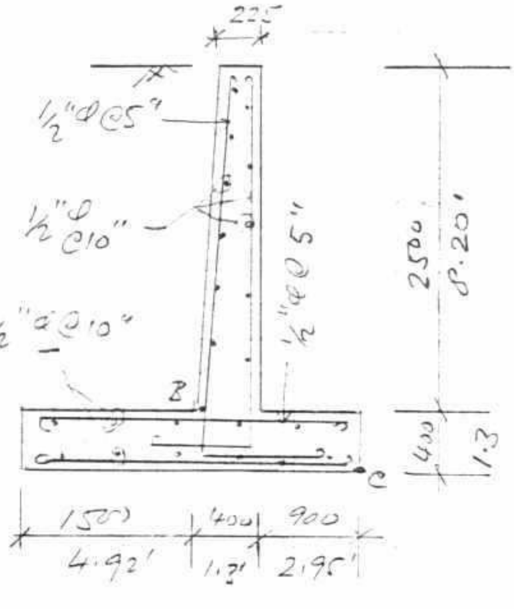
M at mid span (for 37.72' span)

$= 6.24 - 1.18 \times \frac{37.72}{2} + .063 \times \frac{37.72^2}{8}$

$= 6.24 - 22.25 + 11.2 = -4.8^{k'}$ top tension.

Uplift negligible. $A_s = \frac{4.8}{1.31 \times 11} = 0.33\,in^2$ 1/2" φ @ 7" c/c.

Down stream Flank wall :



earth pressure

$P = 0.59 \times .115 \times 8.20 = 0.56$

$P_x = \frac{1}{2} \times .56 \times 8.20 = 2.28^k$

Moment at B

$= 2.28 \times \frac{8.20}{3} = 6.24^{k'}$

$d = \left(\frac{6.24}{.189}\right)^{.5} = 5.74''$

provide 14" stem base

$A_s = \frac{6.24}{1.31 \times 11} = 0.43 in^2$ 1/2" Φ @ 5" c/c

Tempr. & distr. $A_s = .0025 \times 12 \times 14 = 0.42 in^2$; 1/2" Φ @ 10" c/c.

Moment at C (Overturning M)

$= 2.28 \times \left(\frac{8.2}{3} + 1.3\right) = 9.20^{k'}$

Component	Balancing load	Moment arm	Moment $^{k'}$
Soil	$4.92 \times 8.2 \times .115 = 4.64^k$	6.71'	31.13
Stem	$\frac{9+14}{12\times 2} \times 8.2 \times .15 = 1.18^k$	3.43	4.05
Base slab	$1.3 \times 9.17 \times .15 = 1.79^k$	4.59'	8.22
	$\Sigma V = 14.73^k$,	$\Sigma M =$	$43.4^{k'}$

$F.S = \frac{43.4}{9.2} = 4.71 > 1.5$ ok.

Add cut-off wall with toe to el. −0.445 ok.

Foundation :

Both the bore logs show organic clay with trace fine sand and silt with SPT value ranging from 1 to 3 upto a depth of 20' below existing G.L.

Layers of fine sand, with little silt having SPT values ranging from 5 to 23 upto depth of 20' to 60' below E.G.L.

Bore Hole No.	Depth below E.G.L.	Type of Soil	SPT value	Allowable Bearing Capacity	Fdn. level of Sluice	Allow. Bearing Capacity at fdn. level
BH 1	0'-20'	Organic clay, trace fine sand	1 to 3	536 lb/ft²	RL. 1.68 m PWD	536 lb/ft²
	20'-40'	Fine SAND, trace Silt	3 to 18			
BH 2	0'-20'	Organic clay, trace fine sand	1 to 2	610 lb/ft²	RL. 0.875 m PWD	610 lb/ft²
	20'-40'	Fine SAND little Silt	3 to 16			

~~Timber piles upto a depth of 30' below E.G.L. is recommended. 16" piles.~~

~~$$W = \frac{5 \times 600 lb \times 12}{\frac{1}{3} \times 6}$$~~

~~$= 18000 lb = 18 t$~~

~~Wt. of hammer = 600 lb~~
~~Fall " = 5'~~
~~Set = $\frac{1}{3}$"~~
~~F.S = 6~~

~~(energy loss during driving considered negligible)~~

Pre cast r.c.c. pile :

Length of pile, l = 11 m = 36', cut off level = ± 1.0 m below ex. G.L.
Size = 12" × 12", $r_p = \frac{12}{2} = 6" = 0.5'$
N = 16. (av.) at 40' depth below G.L.
Soil type at 30 to 35' depth below G.L. = Medium dense SAND, trace silt & mica.

$$\left.\begin{array}{l}\varphi = 30^\circ \\ N = 13\end{array}\right\} \text{from curve, } N_\gamma = 14 \;\&\; N_q = 16$$

Pile capacity

by, i) End bearing (Rf. Chellis, Pg. 45)

$$R_e = 4r_p^2(1.3cN_c + \gamma D_f N_q + 0.8\gamma r_p N_\gamma)$$
$$= 4 \times .5^2 (0 + .085 \times 36 \times 16 + 0.8 \times 0.5 \times 14)$$
$\frac{(115+55)}{2}$ = 85 pcf
$$= 1 \times (48.96 + 5.6)$$
$$= 54.56^k$$

by ii) skin friction

$$R_f = 8 r_p D_f' s$$
(s = 1200 psf, ultimate, ref, table 2.3 of Chellis Pg. 43.)
$$= 8 \times 0.5 \times 16 \times 1.2$$
$$= 76.8^k$$

$$R_u = R_e + R_f = 54.56 + 76.8 = 131.36^k$$
$$F.S. = 3.0$$
$$R_a = \frac{131.36}{2.5} = 52.54^k \text{ say } 53^k \text{ per pile.}$$

Structural design of piles:

Vertical load = 60^k (max.)

$$R = \cdot 80 A_g(\cdot 225 f'_c + f_s P_g) \qquad 12'' \times 12'' \text{ pile}$$

$$= 0\cdot 18 f'_c A_g + 0\cdot 8 A_s f_s$$

$$= \cdot 18 \times 2\cdot 5 \times 144 + 0\cdot 8 \times 8 \times \cdot 31 \times 18$$

$$= 64\cdot 8^k + 35\cdot 71^k$$

$$= 100\cdot 51^k$$

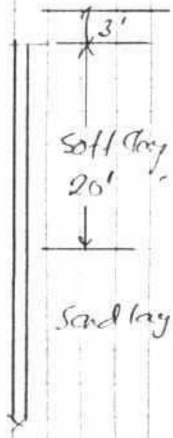
Unsupported length of soft clay layer = 20'

Effective length = $0\cdot 75 \times 20 = 15'$

$$\frac{L}{d} = \frac{15 \times 12}{12} = 15$$

load capacity factor = 0.80

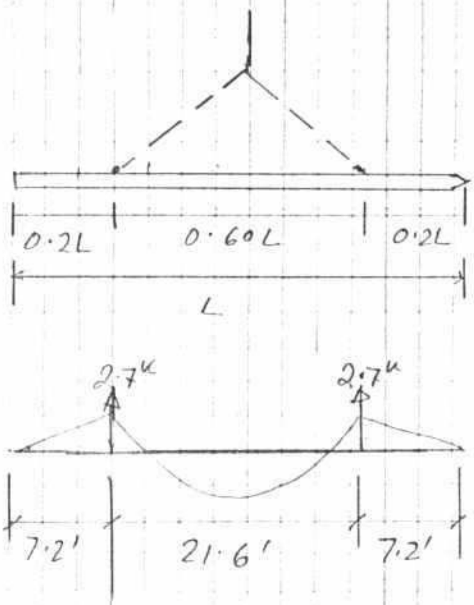
$$\therefore R = 0\cdot 8 \times 100\cdot 51^k = 80\cdot 41^k > 60^k \text{ ok.}$$



Handling stresses:

two point pick-up.

Load per rft = $0\cdot 15^k/'$



$$-ve\ M = \cdot 15 \times 7\cdot 2 \times \frac{7\cdot 2}{2} = 3\cdot 89^{k'}$$

$$+ve\ M = 2\cdot 7 \times \frac{21\cdot 6}{2} - 18 \times \cdot 15 \times \frac{18}{2}$$

$$= 29\cdot 16 - 24\cdot 3$$

$$= 4\cdot 86^{k'}$$

$$d = \left(\frac{4\cdot 86 \times 12}{\cdot 185 \times 12}\right)^{\cdot 5} = 5\cdot 1''$$

$$A_s = \frac{4\cdot 86}{1\cdot 31 \times 10} = 0\cdot 37 \text{ in}^2 \qquad 3\text{-}5/8''\varphi = 0\cdot 93 \text{ in}^2 \text{ ok.}$$

Foundation of pipe:

Load from embkt. soil $= (7.5 - 2.0) \times 3.28 \times .115 = 2.075^{ksf}$

Self wt. of slab $= \frac{20}{12} \times .15 = 0.25^{ksf}$

2.325^{ksf}

area under each pile $= \frac{53}{2.33} = 22.75\ ft^2$

$= 4.76' \times 4.76'$ spacing.

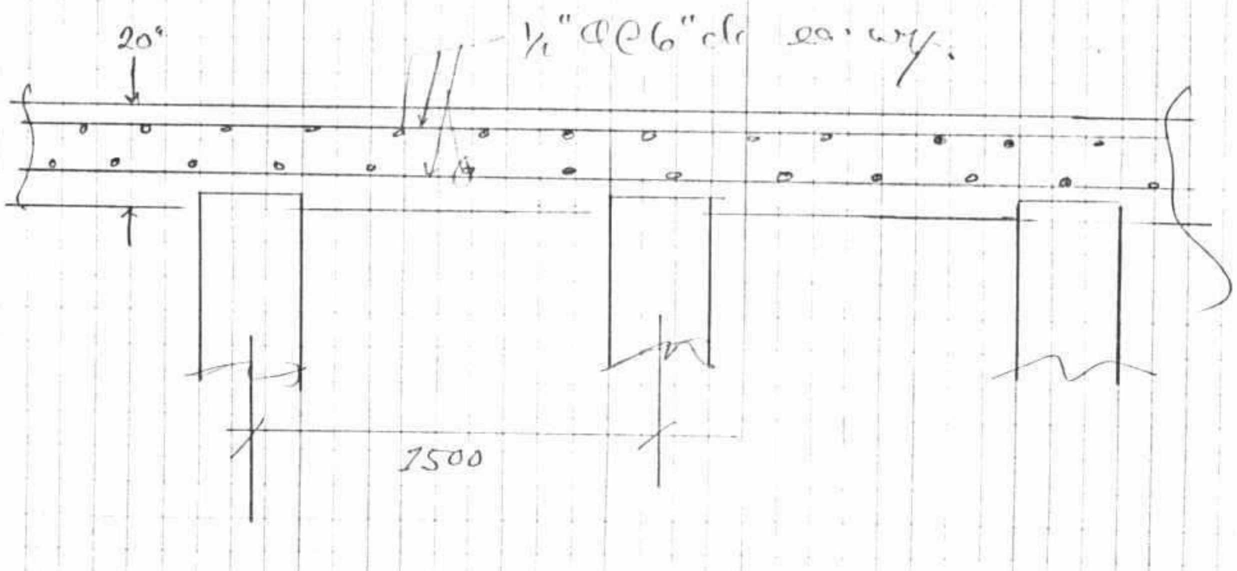
Base slab of pipe:

$B.M = 2.33 \times 4.76^2 \times \frac{1}{9} = 6.12^{k'}$

$d = \left(\frac{6.12}{.189}\right)^{.5} = 5.69''$ / provide thickness $= 20''$

$A_s = \frac{6.12}{1.31 \times 14} = 0.33\ in^2$ $\frac{1}{2}''$ ф @ 7" c/c ea. way, top & bot.

Punching shear $v = \frac{53}{26 \times 4 \times 14 \times .875} = .042\ ksi < .060\ ksi$ ok.



Foundation of D/r piers:

Max. pressure on soil = 2.1 ksf

Pile spacing = $\frac{53}{2.1}$ = 25.23 ft^2 = 5.02' × 5.02'

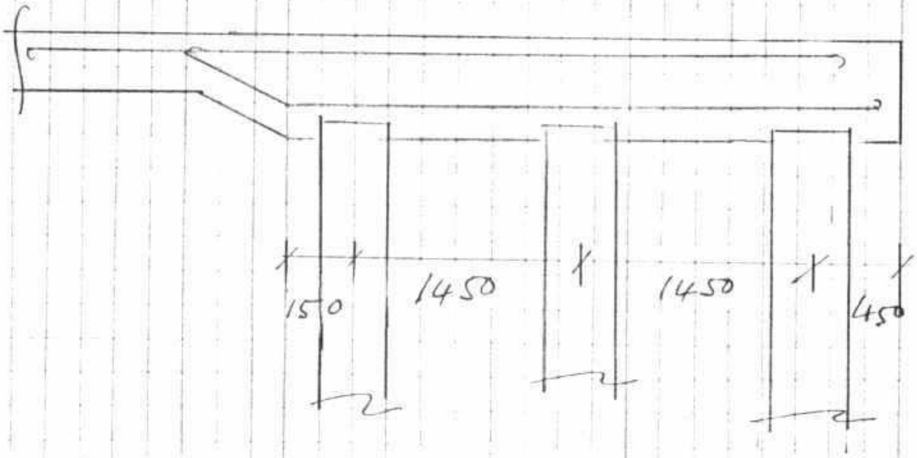
Provide spacing 4.6' × 4.6' upto 3.5 m of fdn slab
& 5.12' × 5.12 spacing beyond.

Base slab:

B.M = 2.1 × 5.12^2 × $\frac{1}{9}$ = 6.12$^{k'}$

d = 18   t = 24"

A_s = $\frac{6.12}{1.31 \times 18}$ = 0.26 in^2 ½" ϕ @ 9" ea way. top & bot.



Founda. of stilling basin slab :

water load = $(7.5 - 1.40) \times 3.28 \times 0.0625$ = 1.25 ksf

Self wt. of slab = $\frac{16}{12} \times .15$ = 0.20 ksf

1.45 ksf

Load from wing wall = 0.273

1.723 ksf

Pile spacing = $\frac{53}{1.723} = 30.76$ sq ft.

$= 5.55' \times 5.55'$, 1.69 m $\times 1.69$ m

$B.M = 1.723 \times 5.55^2 \times \frac{1}{9} \approx 6.04^{k'}$

$A_s = \frac{6.04}{1.31 \times 11} = 0.42$ in² ½" Ø @ 6" c/c. each way top & bot.

Punching shear - $v = \frac{53}{23 \times 4 \times 11 \times .875} = .059$ ksi > 0.60 ksi

increase thickness to 20"

$v = \frac{53}{27 \times 4 \times 15 \times .875} = .037$ ksi < 0.60 ksi

$A_s = \frac{6.04}{1.31 \times 15} = 0.31$ in² ½" Ø @ 7" c/c. each way top & bot.

DESIGN REPORT

ON

PIPE SLUICE AT DOWNSTREAM OF RLY. BRIDGE NO. 40A

000

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①

PIPE SLUICE AT DOWNSTREAM OF RLY. BRIDGE NO. 40A

Design Discharge

Catchment area = 0.06 km² (as determined from field survey & topo map)

Design discharge = 0.34 m³/sec. (computed from av. of max
= 12 cusec. 2nd, 3rd & 4th hr rainfall of 6hr. duration on 2yr. return period. Rational formula used. Ref. JICA report 1990)

Pipe Size and Number:

Sill elevation = 4.20 m PWD.

assumed, pipe dia = 1200 mm.
Head diff = 0.15 m
Running full
Discharge co-efficient = 0.9
Hydraulic radius r = 1
Area = 12.5 □'

$v = c\sqrt{2gh} = 0.9\sqrt{64.4 \times .5} = 5.1$ ft/sec.

$Q = 5.1 \times 12.5 = 63.75$ cusec > 12 cusec.

Use of 1 no. 1200 mm dia pipe ok.

Other design procedure similar to those of sluice at D/S of Prayoti Sarani Bridge.

P.T.O.

Foundation:

Light brown stiff CLAY, trace fine sand.

SPT, $N = 5$ at 5' depth below G.L.

$q_u = 1.61$ ksf (from similar lab. test results of soil report)

Load from structure on soil:-

D/S end wall $= (7.8 - 5.2) \times 1.8 \times 3.28^2 \times .15 = 7.55^k$

Pier $= 1.3 \times (7.8 - 4.2) \times 3.28^2 \times .15 = 7.55$

Operating Slab $= \frac{10}{12} \times 1 \times 1.2 \times 3.28^2 \times .15 = 1.61$

Live load $= 0.1^k$ lb/sft $\times 1 \times 1.2 \times 3.28^2 = 1.29$

Gate & hoist $= 0.4^k + 2.57^k = 2.97$

$= 20.97^k$

Fdn. area $= 1.3 \times 1.8 \times 3.28^2 = 25.17$ ft^2

Pressure on soil $= \frac{20.97}{25.17} = 0.833$ ksf < 1.61 ksf. ok.

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DESIGN REPORT
ON
PIPE SLUICE AT DOWNSTREAM OF RLY. BRIDGE NO. 40

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PIPE SLUICE AT D/S OF RAILWAY BRIDGE NO - 40.

<u>Design Discharge</u>:

Catchment area (Upstream of Rly. lines)	= 83 ha.
Return period.	= 2 yrs.
Duration of Rainfall	= 6 hrs
Rainfall (6hr, 2yr return period)	= 135 mm (JICA report 1990)*

Hourly distribution of rainfall:

1st hr	= 9%	= 12.1 mm
2nd hr	= 15%	= 20.3 mm
3rd hr	= 44%	= 59.4 mm
4th hr	= 16%	= 21.6 mm
5th hr	= 9%	= 12.1 mm
6th hr	= 7%	= 9.5 mm

Peak run-off = av. of 2nd, 3rd & 4th, hr rainfall

Rational formula for $Q = CiA/360$

where

Q = Peak discharge m^3/sec

C = Run-off co-efficient = 0.6*

i = Rainfall intensity mm/hr

A = Discharge area in ha.

Assuming 2nd, 3rd & 4th hour rainfall produce peak runoff:

Q (2nd hr) = $0.6 \times 20.3 \times 83 \times 1/360 = 2.81\ m^3/sec$

Q (3rd hr) = $0.6 \times 59.4 \times 83 \times 1/360 = 8.22\ m^3/sec$

Q (4th hr) = $0.6 \times 21.6 \times 83 \times 1/360 = 2.99\ m^3/sec$

Design discharge of the drainage outlet.

Q = av. of 2nd, 3rd & 4th rainfall runoff

$= (2.81 + 8.22 + 2.99)/3 = \underline{4.66}\ m^3/sec$

<u>Pipe Size & Number</u>

H.F.L 50 yr. return period = 7.75 m PWD

Downstream water level during monsoon:

= 5.5 + 0.75 = 6.25 m PWD (av. 10 yrs HWL during August at Demra + 0.75 m)

Sill elevation = 4.40 M

Max^m permissible internal pond (C/S level) = 6.40 m PWD

Min^m head difference = 6.40 − 6.25 = 0.15 m

Assuming 1200 dia pipe $r = 1.2/4 = 0.30$

Co-efficien of discharge, $Cd = [1 + 0.4\,(r)^{\cdot 3} + \cdot 0045 \times L / r^{1.25})^{-1/2}$

$= 1 + 0.4\,(.3)^{.3} + 0.0045 \times 11 / 0.3^{1.25})^{-1/2} = 0.816$

* Main report on the updateng study on storm water drainage system improvement project in Dhaka City, Feb 1990, by JICA

$v = Cd.\sqrt{2gh} = 0.816 \times \sqrt{2 \times 9.8 \times 0.15} = 1.40\ m/sec$

$Q = Av = 1.13 \times 1.40 = 1.582\ m^3/sec$

No. of pipe required $= 4.66/1.582 = 2.94 \approx 3$ Nos.

3 Nos. 1200 mm dia RCC pipe OK.

Scour Depth

$Q = 4.66\ m^3/sec$

$B = 8.45\ m$

$q = 4.66/8.45 = 0.55\ m^3/sec$

$dm = 0.03\ mm$

$f = 1.76\sqrt{dm} = 1.76\sqrt{.03} = 0.30$

depth of scour $= 1.35\ (q^2/f)^{1/3} = 1.35\ (0.55^2/0.3)^{1/3} = 1.35$

EL of D/S cutoff level $=$ D/S W.L $- 1.5 \times R = 4.4 - 1.5 \times 1.35 = 2.37$ PWD

Hydraulic Gradient

Exit gradient by Lanes Weighted Creep theory

Weighted Creep Length $L = 1.72 + 1.52 + (7.0 + 1.52 + 1.0 + 1.830 \times 2 + 2.45 \times 2 + 3) \times 1/3 + 0.9 + 1.2$
$= 12.38$

50 yrs Flood stage $= 7.75$ PWD

In sedimentation level $= 5.50$ PWD

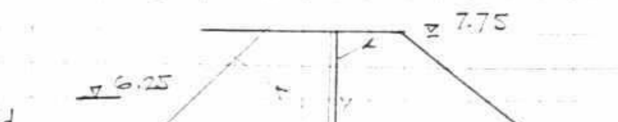
Head difference $= 7.75 - 6.25 = 1.50\ m$

Weighted creep ratio $= 12.38/1.50 = 8.25 > 3.5$ for clayey soil OK.

Providing RCC core wall upto EL 1.5 M

Weighted creep length $= 12.38 + (4.40 - 0.30 - 1.50) \times 2 = 17.58\ m$

Weighted creep ratio $= 17.58/1.50 = 11.72$ ok.



Again Considering seepage through embankment constructed with clayey soil

Weighted creep Length $= (7.75 - 1.5) \times 2 = 12.50\ m$

Weighted Creep ratio $12.50/1.50 = 8.33 > 3.5$ ok

SLUICE AT D/S OF RLY. BRIDGE NO. 40

Stilling basin

A) $q_1 = 1.06\ m^3/sec$

$y_c = (q^2/g)^{1/3} = (1.06^2/9.8)^{1/3} = 0.489 \simeq 0.49\ m$

Applying Bernoullis eqn. betn Critical section & section 1-1 in horizontal Channel

$y_c + V_c^2/2g + Z_c = y_1 + V_1^2/2g$

$y_c + 0.5 y_c + 0 = y_1 + V_1^2/2g$

$1.5 y_c = y_1 + q^2/y_1^2 \times 2g$

$1.5 \times 0.49 = y_1 + (1.06)^2/y_1^2 \times 2 \times 9.8$

by trail $y_1 = 0.44\ m$

$V_1 = 1.06/0.44 = 2.41\ m/sec$

$F_1 = V_1/\sqrt{g y_1} = 2.41/\sqrt{9.81 \times 0.44} = 1.16$ undular jump / weak jump

$y_2/y_1 = 1/2[(1+8F^2)^{1/2} - 1] = 1/2[(1+8\times 1.16^2)^{.5} - 1] = 1.215$

$\therefore y_2 = y_1 \times 1.215 = 0.44 \times 1.215 = 0.535\ m$

Fig 15-4 "CHOW" for weak jump, $L/y_2 = 4.0$

$L = 4.0 \times 0.535 = 2.14$

$F.S. = 1.50$ $\therefore L = 2.14 \times 1.50 = 3.21\ m$

$q_1 = 4.66/5.53 = 0.84\ m^3/sec$

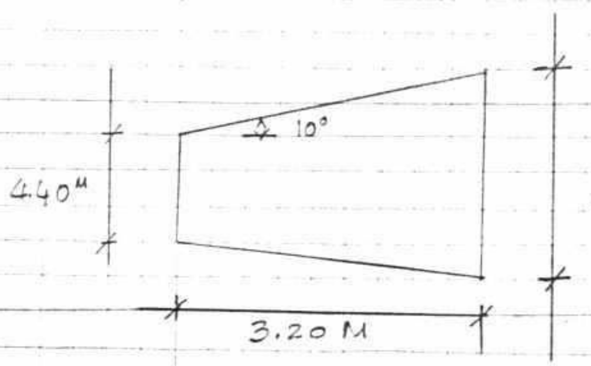
$V_2 = 0.84/0.535 = 1.57\ m/sec$

To reduce the velocity upto 0.6 m/sec

(maxm permissible velocity for organic clay trace fine sand)

$W = 4.66/(0.535 \times 0.60) = 14.50\ m$

$L = (14.50 - 4.40) \times 0.50 \times 1/\tan 10^\circ = 31.53\ m$



Stilling Basin

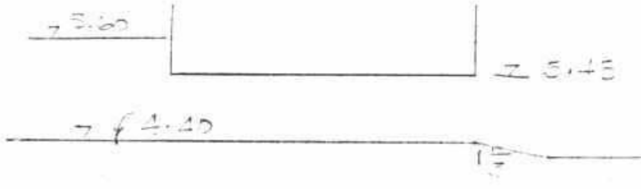
$Q = 4.66$ m³/s

Flow width = 1.2 × 3 + 2 × 0.40 = 4.40 m

Flow per m width $q = 4.66/4.40 = 1.06$ cumec

Critical depth $y_c = (q^2/g)^{1/3} = (1.06^2/9.8)^{1/3} = 0.49$ m

From energy loss consideration by hydraulic jump, neglecting velocity, Head loss of energy (H_L) (Ref: BARG Pg. 487)



$H_L = 5.60 - 5.45 = 0.15$

From Blench Curve

$E_{f_2} = 0.70$ m

$\therefore E_{f_1} = H_L + E_{f_2} = 0.15 + 0.70 = 0.85$ m

From energy of flow curve

$y_1 = 0.20$
$y_2 = 0.87$

Length of cistern = $6(y_2 - y_1) = 6 \times (0.87 - 0.20) = 3.60$ m

Lowering the apron by 0.30 m

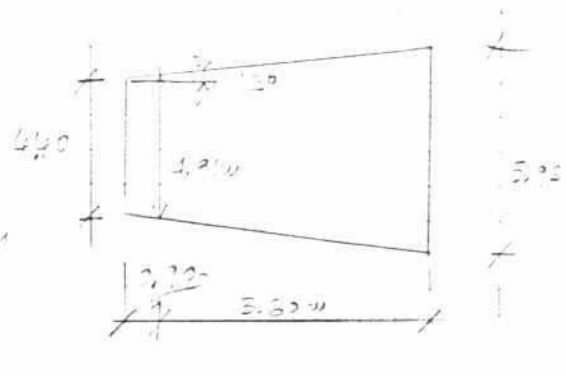
$B_1 = 4.90$ m

$q_1 = Q/B = 4.66/4.90 = 0.95$ cumec/m

$v_1 = q_1/y_1 = 0.95/0.20 = 4.75$ m/sec

$F = 4.75/\sqrt{g \times 0.20} = 3.37$

From curve $L/y_2 = 5.6$ $\therefore L = 5.6 \times 0.80 = \underline{4.48\text{ m}}$



discharge at the end of stilling basin

$q_2 = 4.66/5.93 = 0.79$ cumec/m

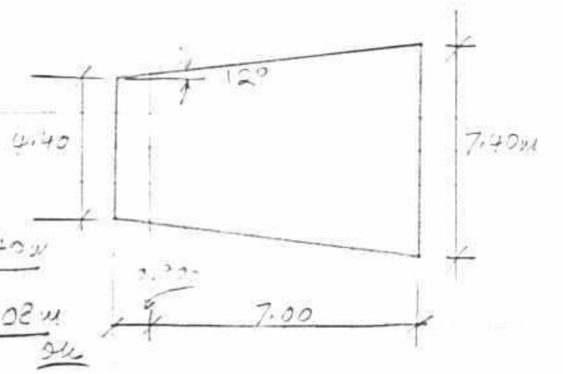
$(y_2 = 5.45 - 4.40 = 1.05\text{ m})$

Velocity $v_3 = 0.79/1.05 = 0.75$ m/sec

Maxm permissible velocity = 0.6 m/sec

So, basin width reqd at the end of stilling basin = $\frac{4.66}{1.05 \times 0.6} = \underline{7.40\text{ m}}$

& basin length = $(7.40 - 4.40) \times 0.50 \times 1/\tan 12^\circ = \underline{7.02\text{ m}}$



Upstream Wing Wall:

Assumption,

$\phi = 15^\circ$

$\gamma_{sat} = 115$ lb/cft

$K_a = (1 - \sin\phi)/(1 + \sin\phi) = 0.59$

Earth pressure, $P = 0.59 \times 0.115 \times 5.27' = 0.392^k$

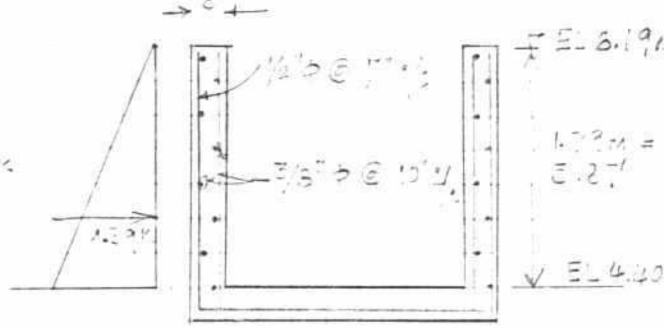
$P_A = \frac{1}{2} \times 0.392 \times 5.27 = 1.17^k$

Moment $= 1.17 \times 5.27/3 = 2.29^{k'}$

$d = (2.29/0.120)^{0.5} = 3.42''$, provide 8" thick wall

$A_s = 2.29/1.31 \times 5 = 0.35$ in² — ½" φ @ 7" c/c

Temp. distribution steel $A_s = 0.0025 \times 12 \times 8 = 0.24$ in²
3/8" φ @ 10" c/c each face



Assumption :-

$f'_c = 2500$ psi

$f_c = 0.45 f'_c$

$f_s = 18000$ psi

$k = 0.387$

$j = 0.875$

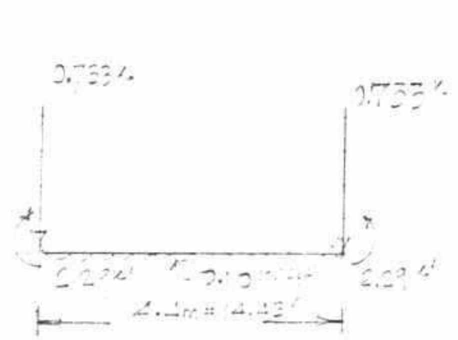
$R = 189$

Upstream Apron Slab:

Load from wingwall $= \frac{8}{12} \times 5.27 \times 0.15 = 0.733^k$

$q_{net} = 0.733 \times 2/14.43 = 0.10$ k/ft²

M at mid span $= 2.29 - 0.733 \times 14.43/2 + 0.10 \times 14.43^2/8$
$= 2.29 - 5.29 + 2.60$
$= 0.397^{k'}$ top tension



$d = (0.397/0.120)^{0.5} = 1.48''$

Thickness for uplift,

$t = 1.33 \times h/(G-1)$ (G = 2.4 Sp. gr. of Concrete)

$= 1.33 \times 0.104/(2.4 - 1)$

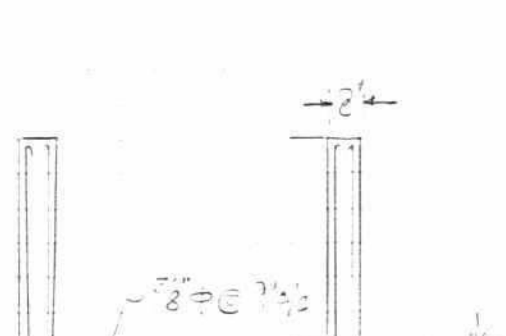
$= 0.125$ m > 0.200 m Thickness provided.

$A_s = 0.397/1.31 \times 5 = 0.06$ in²

3/8" φ @ 8" c/c

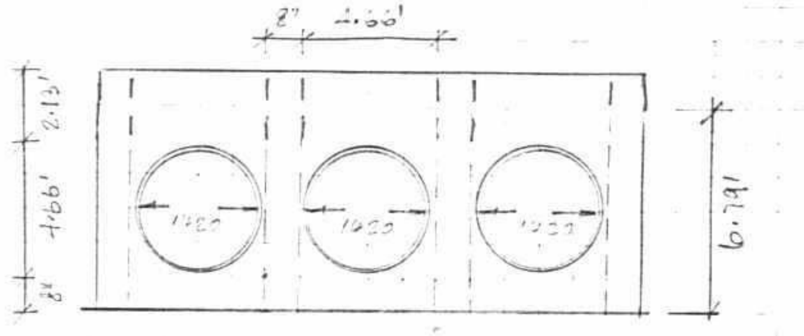
Temp & Distr. steel, $A_s = 0.0025 \times 8 \times 12 = 0.24$ in²

3/8" φ @ 8" c/c



Continue reinforcement of wing wall into apron slab as well.

U/s stream end wall



Pa at pipe top

= 0.39 × 0.115 × 2.13'

= 0.145 k/ft²

Considering 1.33 times of av. load acting on wall as distributed load

= 1.33 × 1/2 × 0.145 = 0.096 k/ft²

M = 0.096 × 4.66² × 1/8 = 0.26 k'

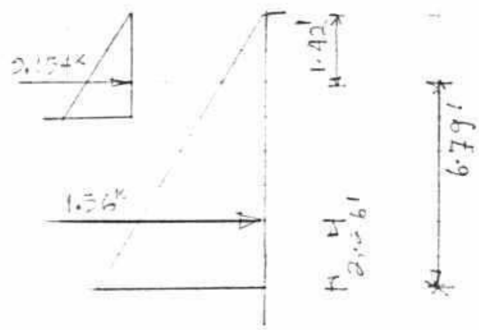
d = (0.28/0.129)^0.5 = 1.50'

As = 0.28/(1.3 × 5) = 0.035 in²

3/8" φ @ 9" c/c

Temp. & distribution = 0.0025 × 8 × 12
= 0.24 in²

3/8" φ @ 10" c/c each face



Load on cantilever wall from earth load

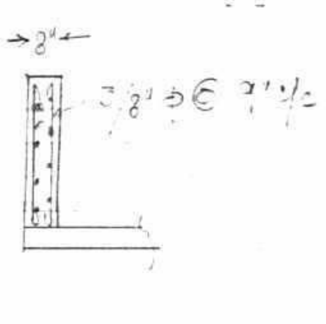
= (0.39 × 6.79 × 0.115) × 1/2 × 6.79

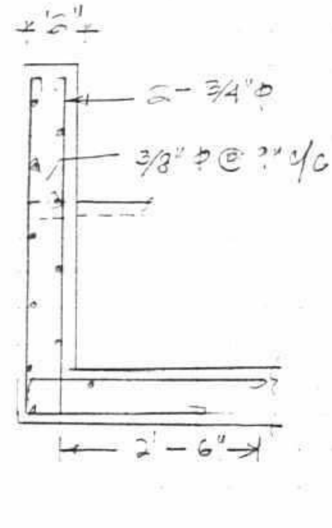
= 1.56k

M at base = 1.56 × 8/12 × 6.79/3 + 0.134 × 4.66 (6.79 − 1.42) = 6.20

d = {(6.20 × 12)/(0.128 × 8)}^0.5 = 7" provide 10" thick wall

As = 6.20/(1.3 × 7) = 0.676 in² - 2 - 3/4" φ ver.





<u>Down Stream end Wall:</u>

Earth load at base of wall above pipe Top

$= 0.39 \times 0.115 \times 6.56' = 0.445^{k/'}$

Considering 1.33 times of a/c load acting on wall as distributed load.

$= 1.33 \times (1/2 \times 0.45) = 0.30^{k/'}$

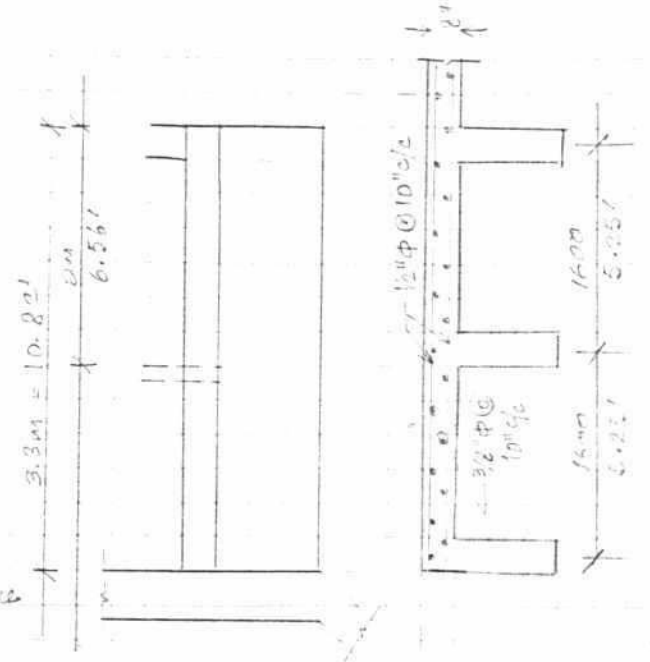
$-ve\ M = 0.39 \times 5.25^2/9 = 0.92^{k'}$

$d = (0.92/.189)^{.5} = 2.20''$ <u>Provide 8'' wall</u>

$A_s = 0.92/1.31 \times 5 = 0.14\ in^2$, $1/2''\phi$ @ $10''$ c/c each face

Distribution $A_s = 0.0025 \times 8 \times 12 = 0.24\ in^2$

use $3/8''\phi$ @ $10''$ c/c



<u>Down Stream Piers:</u>

Earth load at A above pipe $= 0.445^{k/'}$

$P = 5.25 \times 0.445 = 2.34^k$

$F_a = 1/2 \times 2.34 \times 6.56 = 7.68^k$

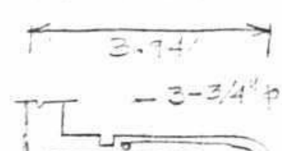
Load on cantilever wall from earth load

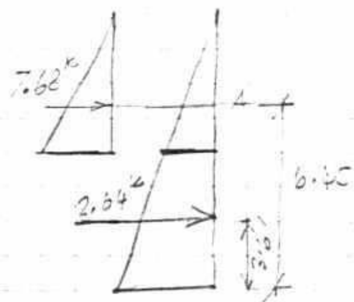
$= (0.39 \times 10.82 \times 0.115) \times 1/2 \times 10.82 \times 8/12 = 2.64^k$

M at base $= 7.68 \times 6.25 + 2.64 \times 3.6 = 60.20^k$

$d = [60.20 \times 12/(.189 \times 8)]^{1/2} = 21.25''$

$A_s = 60.20/(1.31 \times 43) = 1.07\ in^2$ $3-3/4''\phi$ ver.





Over turning Moment at C = $60.20^{k'}$

Balancing M

Soil $= 3.28 \times 10.82 \times 5.25 \times 0.115 \times 5.47 = 120.85^{k'}$

$\therefore$ Min F.S $= 120.85/60.20 = 2.01$ ok.

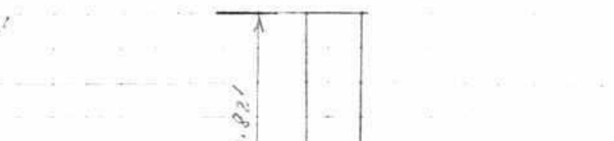
Foundation Slab, $M = 60.20$

$d = [(60.20 \times 12)/(.189 \times 5.25 \times 12)]^{1/2} = 7.79''$

<u>provide 14''</u>.

$A_s = 60.20/$[illegible] $= 4.$[illegible] in^2

ie $4.18/5.25 = 0.8\ in^2$ per footwidth of slab <u>$5/8''\phi$ @ $4\frac{1}{2}''$ c/c</u>



Stilling basin floor (down stream)

Load from wing wall = 8/12 × 0.15 × ... = 0.656^k

q_{net} = 0.656 × 2 / 14.48 = 0.09 k/ft²

M at mid span (Span 14.48')

= 2.32 − 0.656 × 14.48/2 + 0.09 × 14.48²/8

= 2.32 − 4.75 + 2.34 = −0.09 k' Top tension

M at mid span (Span 27.78')

= 2.32 − 0.656 × 27.78/2 + 0.09 × 27.78²/8

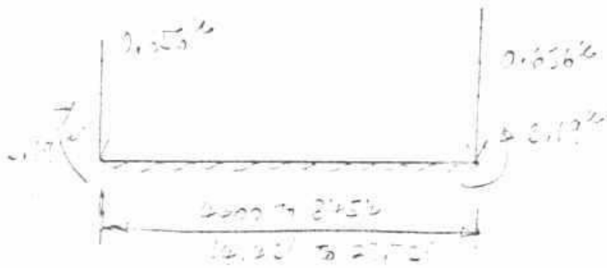
= 2.32 − 9.12 + 8.65 = 1.87 k' bot tension

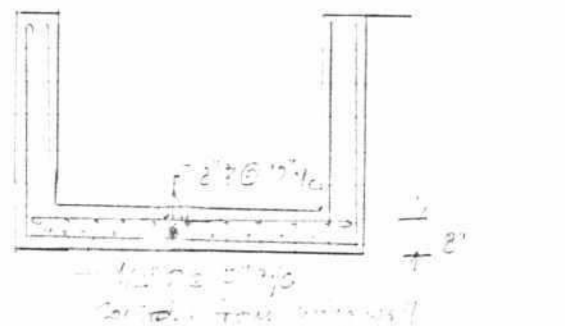
d = (1.87/0.129)^0.5 = 3.80"

Uplift negligible, Provide Thickness of floor 8"

As = 1.87/(1.44×6) = 0.22 in² use 1/2"φ @ 8" c/c

As = 0.0025 × 12 × 8 = 0.24 in² Temp 3/8"φ @ 10" c/c





Down Stream wing wall

Earth pressure

P = 0.30 × 0.10 × 4.58' = 0.143 k

P_h = 1/2 × 0.143 × 4.58 = 0.33k

Moment = 0.33 × 4.58/3 = 0.50 k'

d = (0.50/0.129)^0.5 = 1.97"

Provide thickness = 8"

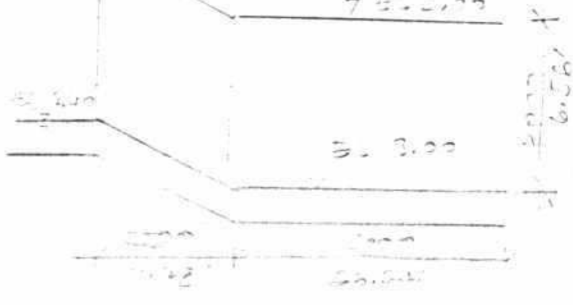
As = (0.50/(1.44×6)) = 0.12 in²

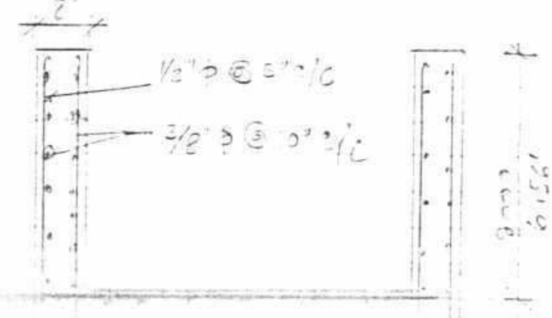
1/2"φ @ 8" c/c

Temp. & shrink. As = 0.0025×12×8

= 0.24 in²

3/8"φ @ 10" c/c





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DESIGN REPORT

ON

PIPE SLUICE AT D/S OF RLY. BRIDGE NO. 41

CONTENTS

PIPE SLUICE AT D/S OF RLY. BRIDGE NO. 41.

Design Discharge:

catchment area = 0.22 sq mile = 58 ha. (as determined from field survey & topo map)

Design Discharge = 3.37 cum/sec. (as computed from av. of max. 2nd, 3rd & 4th hr. rainfall of 6hr. duration on 2yr. return period - JICA report 1990)

Pipe Size & Number:

H.F.L. 50yr. return period = 7.90 M PWD (interpolation of level btn. Tongi & Demra)

Sill elevation = 5.0 M PWD.

Downstream Water level during monsoon = 5.15 + 0.9 = 6.05 M PWD. (av. 10yr. H.W.L. during July at Demra + 0.9 m)

Max. permissible internal pond level (C/S) = 6.20 M PWD. (Safe level against inundation of C/S land)

Assuming 900 mm dia R.C.C. pipe (pipe dia = 0.9 m, A = 0.636, $r = \frac{0.9}{4} = 0.225$, L = 8.22 M)

Co-efficient of discharge

$$C_d = \left[1 + 0.4r^{0.2} + \frac{.0045L}{r^{1.25}}\right]^{-1/2}$$

$$= \left[1 + 0.4 \times .225^{0.2} + \frac{.0045 \times 8.22}{0.225^{1.25}}\right]^{-1/2}$$

$$= (1.256 + 0.234)^{-1/2}$$

$$= 0.82$$

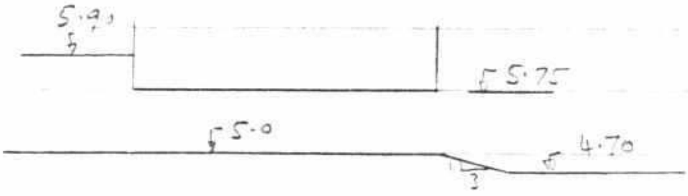
Head difference = 6.20 - 6.05 = 0.15 M

$V = C_d\sqrt{2gh} = 0.82 \times \sqrt{2 \times 9.8 \times .15} = 1.41$ M/sec.

$Q = AV = 0.636 \times 1.41 = 0.898$ m³/sec.

No. of pipes reqd. $= \frac{3.37}{0.898} = 3.75$ nos. i.e. 4 nos.

Stilling Basin:



$Q = 3.37$ cumec

Floor width $= 4.8$ m

Flow per metre width

$$q = \frac{3.37}{4.80} = 0.70 \text{ cumec}$$

Critical depth $y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{0.70^2}{9.8}\right)^{1/3} = 0.42$

From energy loss consideration in hydraulic jump, Neglecting velocity head, loss of energy (H_L)

$$H_L = 5.90 - 5.75 = 0.15 \text{ m}$$

From Blench curve, $Ef_2 = 0.6$

$$\therefore Ef_1 = H_L + Ef_2 = 0.15 + 0.6 = 0.75$$

From energy of flow curve,

$$y_1 = 0.15$$

$$y_2 = 0.70$$

Length of cistern $= 6(y_2 - y_1) = 6(0.7 - 0.15) = 3.3$ m.

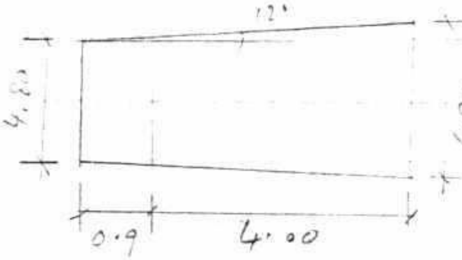
Lowering the apron by 0.2 m.

$B_1 = 5.18$ m.

$$q_1 = \frac{3.37}{5.18} = 0.65 \text{ cumec}$$

$$v_1 = \frac{0.65}{0.15} = 4.33 \text{ m/sec.}$$

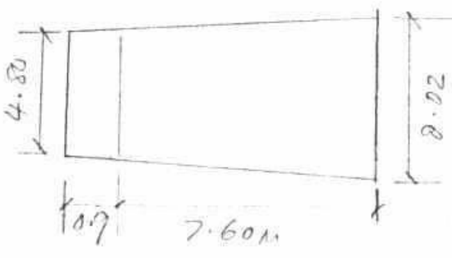
$$F_1 = \frac{4.33}{\sqrt{g \times 0.15}} = 3.58$$



From curve $L/y_2 = 5.70 \therefore L = 5.70 \times 0.7 = 4.00$ m.

$$q_2 = \frac{3.37}{6.87} = 0.49 \text{ cumec.} \quad v_2 = \frac{0.49}{0.70} = 0.70 \text{ m/sec.}$$

max. permissible velocity = 0.6 m/sec.
So, basin width reqd. at the end of stilling basin



$$W = \frac{3.37}{0.7 \times 0.6} = 8.02 m$$

& Basin length

$$L = (8.02 - 4.8) \times 0.5 \times \frac{1}{\tan 12°}$$

$$= 7.60 m.$$

DDt

(4)

CHUTE BLOCK   Basin type IV USBR

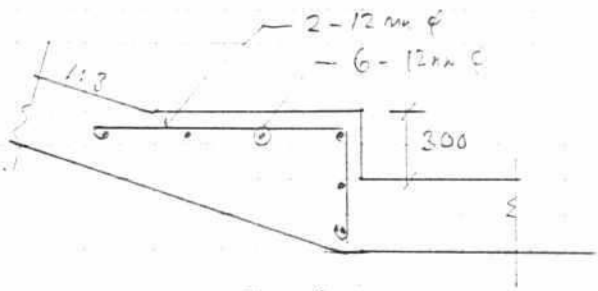
Ht. of block = $2d_1 = 2 \times .15 = 0.30$ m

width of block = $d_1 = 0.15$ m

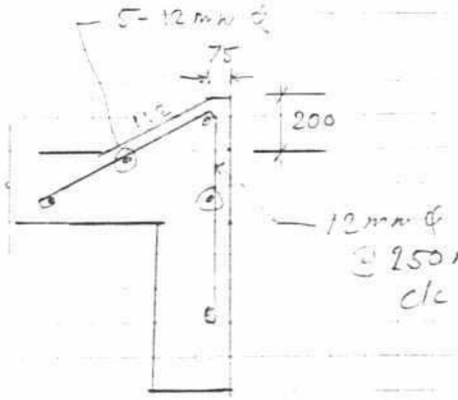
Spacing betn. block = $2.5\,W = 2.5 \times .15 = 0.375$ m.

END SILL

Ht. of sill = $1.25\,d_1 = 1.25 \times .15 = .188$ m



CHUTE BLOCK



END SILL

Scour depth :

$Q_1 = 3.37\ m^3/sec.$

$B = 4.8\ m$

$$q = \frac{3.37}{4.80} = 0.70\ m^3/sec.$$

$d_m = .03\ mm.$

$f = 1.76\sqrt{d_m} = 1.76\sqrt{.03} = 0.30$

$$\text{Depth of scour} = 1.35\left(\frac{q^2}{f}\right)^{1/3}$$

$$= 1.35\left(\frac{.70^2}{0.30}\right)^{1/3}$$

$$= 1.58\ m$$

Rl. of D/S cut off bottom = D/S W.L − 1.5×1.58

$(4.70 + 0.70) - 2.37 = 3.03$ PWD.

Hydraulic gradient :

50yr. flood stage = 7.90 m PWD.

C/S foundation level = 6.20 m PWD.

∴ Head diff. = 1.70 m

Weighted creep length (after Lane)

$= 1 + 0.7 + \frac{1}{3}(8.32 + 5.90) + 1.67 + 1.37$

$= 9.48$

$$\text{Weighted creep ratio} = \frac{9.48}{1.70} = 5.58 > 3.0\ \text{ok.}$$

(6)

Upstream Wing wall :

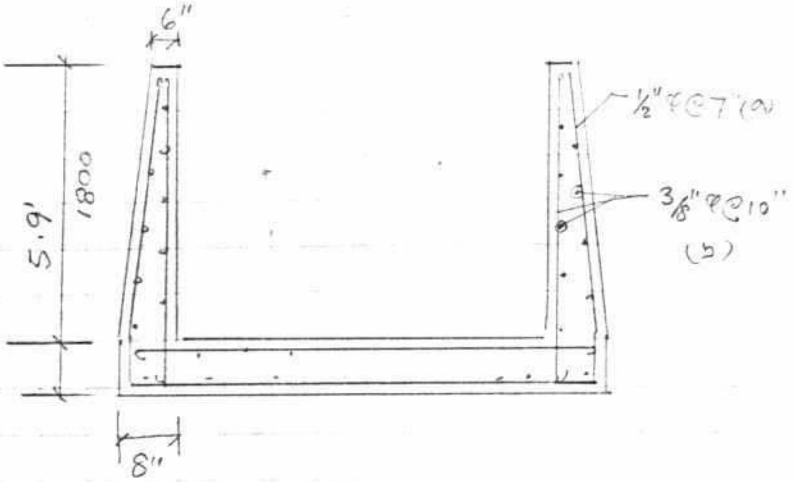
Assumption,

$\phi = 15^\circ$

$\gamma_{sat} = 115\ lb/cft$

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

$= 0.59$



earth pressure

$P = 0.59 \times .115 \times 5.9 = 0.40^{k}/\square'$

$P_A = \frac{1}{2} \times 5.9 \times 0.4 = 1.18^{k}$

Moment $= 1.18 \times \frac{5.9}{3} = 2.32^{k'}$

$d = \left(\frac{2.32}{.189}\right)^{.5} = 3.5''$ provide 8" thick wall

$A_s = \frac{2.32}{1.31 \times 5} = 0.35\ in^2$

½" φ @ 7" c/c

Tempr. & Dist. $A_s = .0025 \times 12 \times 8 = 0.24\ in^2$

3/8" φ @ 10" ea. face.

Assumptions,

$f'_c = 2500$ psi

$f_c = 0.45 f'_c$

$f_s = 18000$ psi

$k = 0.289$

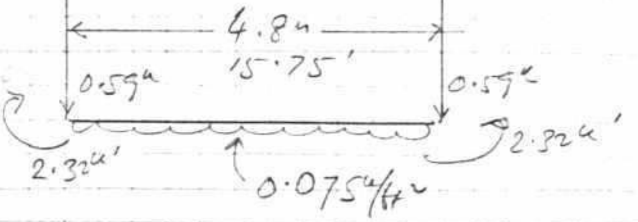
$j = 0.875$

$R = 189.$

Upstream Apron Slab:

Load from wing wall

$$= \frac{8}{12} \times 5.9 \times .15 = 0.59^{k}/'$$



$$q_{net} = \frac{0.59 \times 2}{15.75} = .075^{k}/ft^2$$

$$M \text{ at mid span} = 2.32 - 0.59 \times \frac{15.75}{2} + .075 \times \frac{15.75^2}{8}$$

$$= 2.32 - 4.65 + 2.33$$

$$= -0.0044^{k'} \text{ (top tension)}$$

$$d = \left(\frac{.0044}{0.189}\right)^{.5} = 0.153'', \quad t = 8''$$

$$A_s = \frac{.0044}{1.31 \times 5} = .00067 \text{ in}^2$$

Temp. $A_s = .0025 \times 8 \times 12 = 0.24 \text{ in}^2$

3/8" ⌀ @ 10" c/c ea. way
ea. face

Check slab thickness for uplift:

$$t = 1.33 \left(\frac{h}{G-1}\right)$$

$$= 1.33 \frac{0.228}{(2.4-1)}$$

$$= 0.217 \text{ m}$$

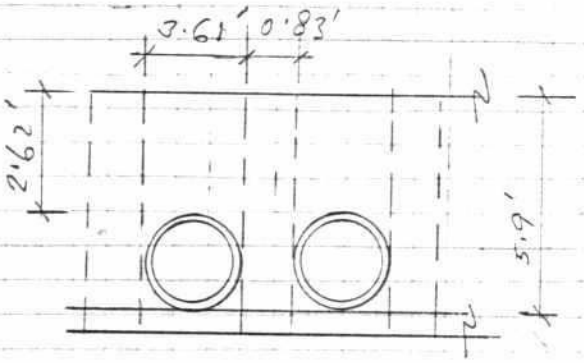
$$= 217 \text{ mm} < 300 \text{ mm}.$$

$$h_c = \frac{H_c}{L} \quad (H_c = 7.9 - 6.0 = 1.9 \text{ m})$$

$$= \frac{1.9}{8.32}$$

$$= 0.228$$

Upstream end wall



P_a at pipe top

$= 0.59 \times .115 \times 2.62$

$= 0.178\ ^{k}/ft^2$

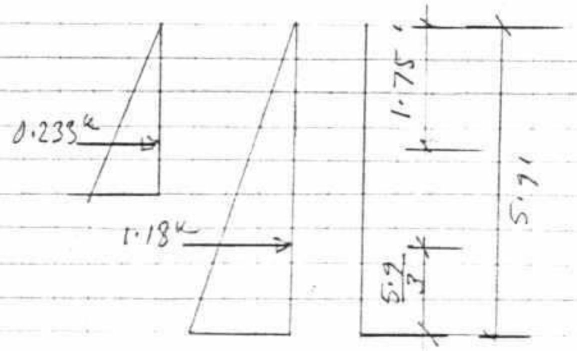
Considering 1.33 times of av. load acting on wall as distributed load

$= 1.33 \times \frac{1}{2} \times .178 = 0.118\ ^{k}/ft^2$

$M = 0.118 \times \frac{3.61^2}{9} = 0.17^{k'}$

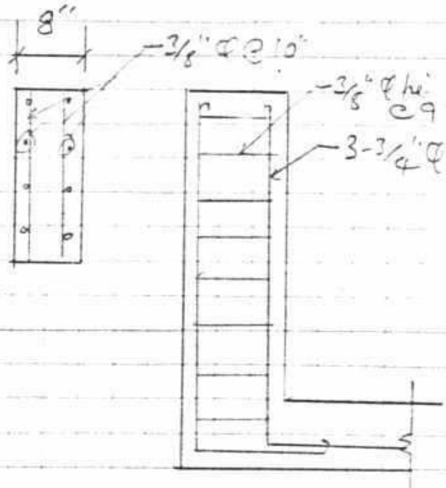
$d = \left(\frac{0.17}{.189}\right)^{.5} = 0.95''$

$A_s = \frac{0.17}{1.31 \times 4} = .0324\ in^2$



Load on cantl. col. from wall above pipe,

$= \frac{1}{2} \times .178 \times 2.62 = 0.233^{k}$



Load on cantl. col. from earth load

$= (0.59 \times 5.9 \times .115) \times \frac{1}{2} \times 5.9$

$= 1.18^{k}$

M at base $= 1.18 \times .83 \times \frac{5.9}{3} + .233 \times 3.61(5.9 - 1.75)$

$= 1.92 + 3.49 = 5.42^{k'}$

$d = \left(\frac{5.42 \times 12}{.189 \times 10}\right)^{.5} = 5.86''$ provide 18" thick col.

$A_s = \frac{5.42}{1.31 \times 1.5} = 0.276\ in^2$ – provide 3-3/4"φ vert.

3/8"φ tie @ 9" c/c.

Down Stream end wall :

Earth load on base:

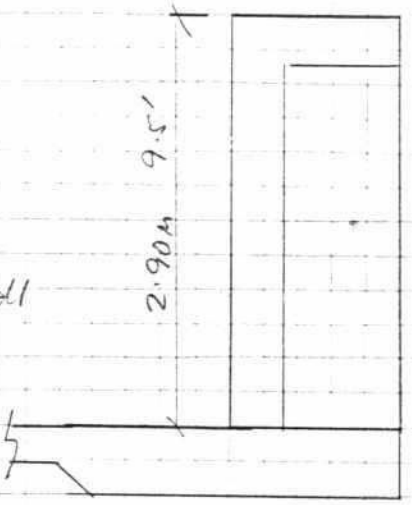
$= 0.59 \times 0.115 \times 9.5$

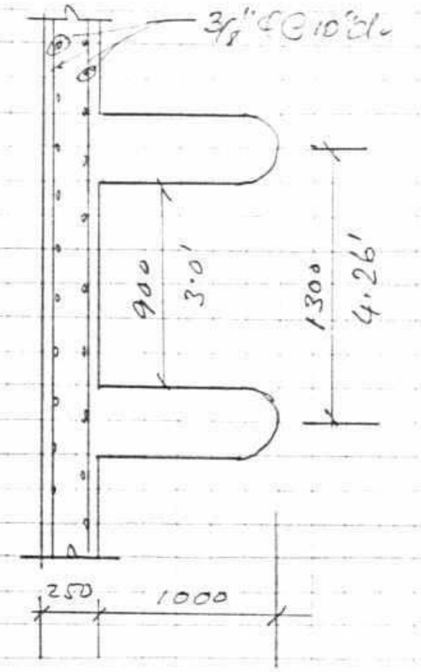
$= 0.645^{k/'}$

Considering 1.33 times of av. load acting on wall as distributed load

$1.33 \times (\frac{1}{2} \times .645)$

$= 0.43^{k/'}$





$-ve\ M = 0.43 \times \frac{4.26^2}{9} = 0.87^{k'}$

$d = \left(\frac{0.87}{.189}\right)^{.5} = 2.14''$ provide 10" wall

$A_s = \frac{0.87}{1.31 \times 7} = .095\ in^2$ 3/8"Φ@10" ea. way ea. face.

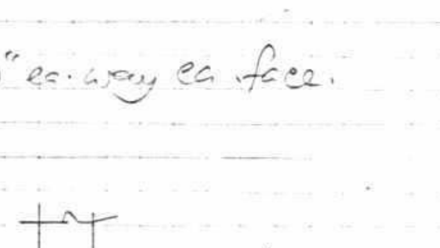
Down Stream Piers :

earth load at base $= 0.645^{k/'}$

For 4.26' width, $P = 0.645 \times 4.26$

$= 2.75^{k}$

$P_a = \frac{1}{2} \times 2.75 \times 9.5 = 13.05^{k}$



M at base $= 13.05 \times \frac{9.5}{3} = 41.33^{k'}$

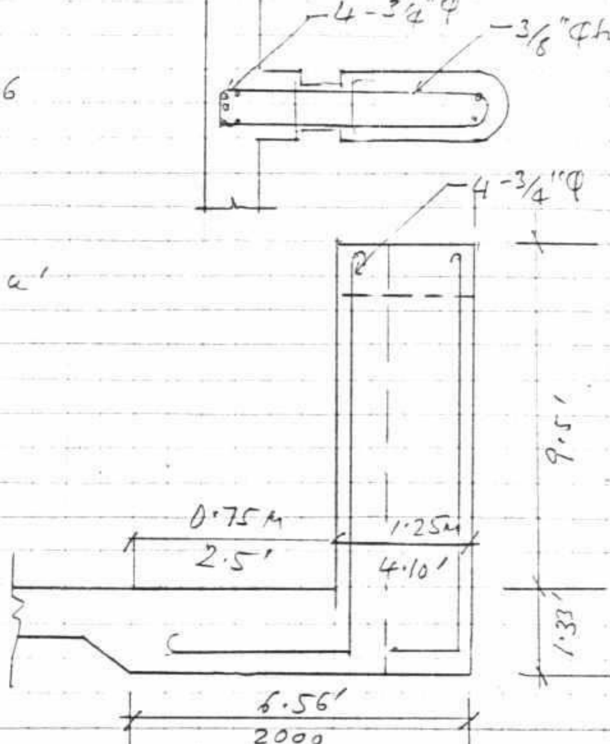
$d = \left(\frac{41.33 \times 12}{.189 \times 10}\right)^{.5} = 16.16''$

Pier width provided = 1.25m

= 49"

$A_s = \frac{41.33}{1.31 \times 45} = 0.70\ in^2$

Use 4-3/4"Φ vert. bars & 3/8"Φ tie @ 9" c/c.

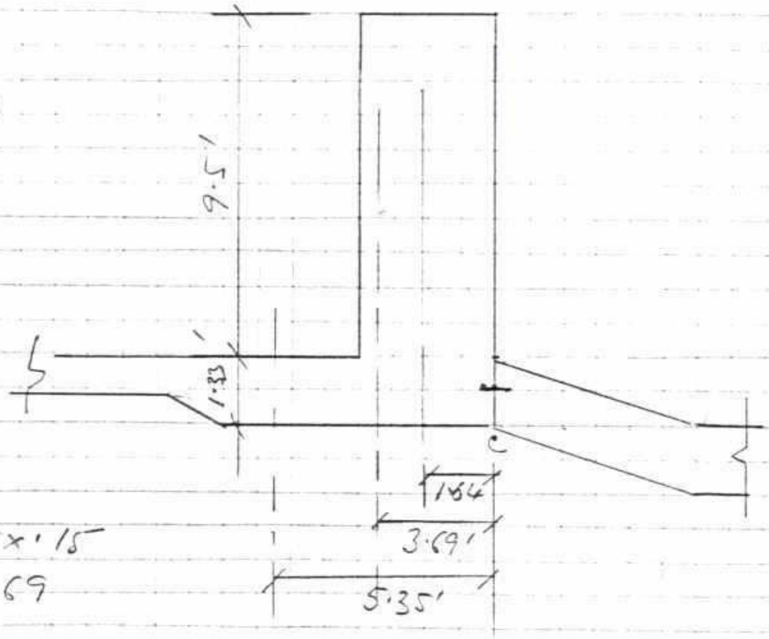


Stability of pier:

Overturning M at C

$= 13.05 \times \left(\frac{9.5}{3} + 1.33\right)$

$= 58.68^{k'}$



Balancing M at C:

Soil $= 4.26 \times 2.5 \times 9.5 \times .118 \times 5.35$

$= 63.87^{k'}$

endwall $= 4.26 \times .83 \times 9.5 \times .15 \times 3.69$

$= 18.59^{k'}$

Pier $= 3.28 \times .83 \times 9.5 \times .15 \times 1.64' = 6.26^{k'}$

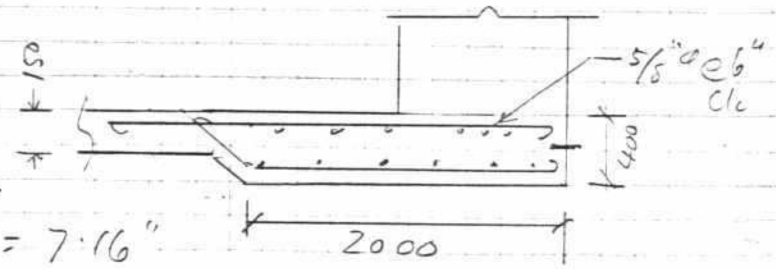
Base slab $= 1.3 \times 6.50 \times 4.26 \times .15 \times \frac{6.50}{2} = 17.87^{k'}$

Deck slab $= 3 \times 3.28 \times 1.0' \times .15 \times 1.64' = 2.42^{k'}$

Total balancing M $= 109.11^{k'}$

$F.S = \frac{109.11}{58.68} = 1.86 > 1.50$ ok.

Base slab of pier:



$M = 41.33^{k'}$,

$b = 4.26'$

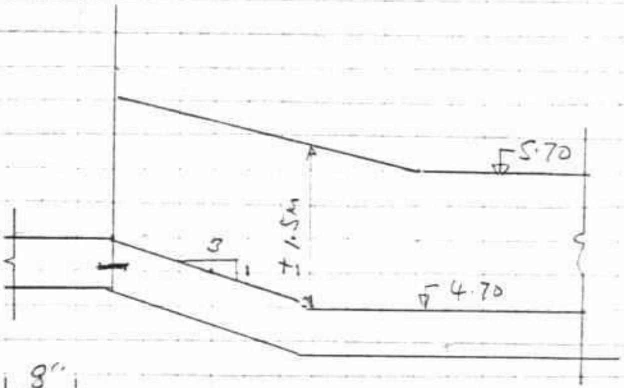
$d = \left(\frac{41.33 \times 12}{.189 \times 4.26 \times 12}\right)^{.5} = 7.16"$

Slab thickness provided = 16"

$A_s = \frac{41.33}{1.31 \times 12} = 2.63 in^2$ i.e. $\frac{2.63}{4.62} = 0.62 in^2$ per ft width.

5/8" φ @ 6" c/c.

Down stream Wing wall :

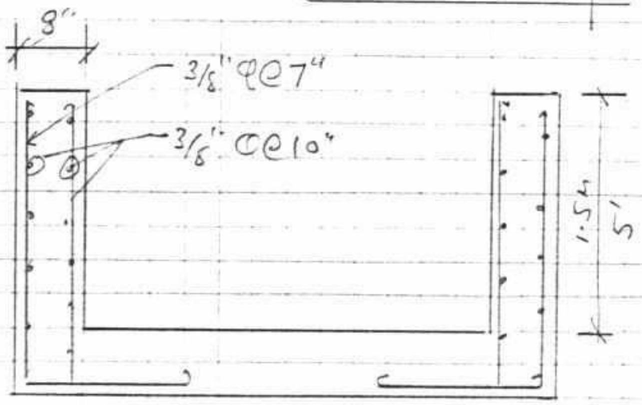


earth pressure

$P = 0.59 \times .115 \times 5'$

$= 0.34^k$

$P_a = \frac{1}{2} \times .34 \times 5 = 0.85^k$



Moment $= 0.85 \times \frac{5}{3} = 1.42^{k'}$

$d = \left(\frac{1.42}{.185}\right)^{.5} = 2.73''$

provide wall thickness = 8"

$A_s = \frac{1.42}{1.31 \times 5} = 0.217 \text{ in}^2$

3/8" φ @ 10" c/c

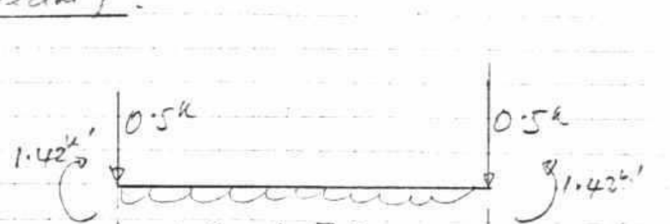
Tempr. & dist. $A_s = .0025 \times 12 \times 8$

$= 0.24 \text{ in}^2$, 3/8" φ @ 10" c/c @ face.

Stilling basin floor slab (downstream) :

load from wing wall

$= \frac{8}{12} \times 5 \times .15 = 0.5^k$



$q_{net} = \frac{0.5 \times 2}{15.74} = .0635 \text{ ksf}.$

M at mid span (span = 15.74')

$= 1.42 - 0.5 \times \frac{15.74}{2} + 0.0635 \times \frac{15.74^2}{8}$

$= 1.42 - 3.935 + 1.97 = -0.546^{k'}$ (top tension)

Again,

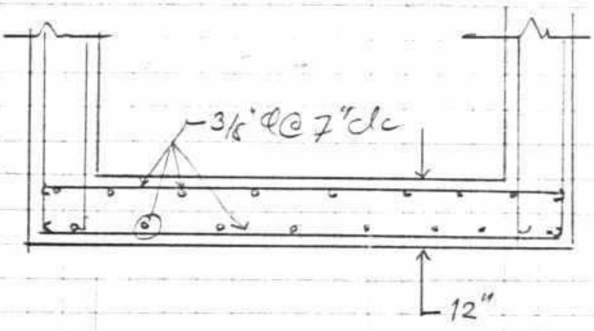
$q_{net} = \frac{0.5 \times 2}{23.94} = 0.042 \text{ ksf}.$

M at mid span (span 23.94') $= 1.42 - 0.5 \times \frac{23.94}{2} + .042 \times \frac{23.94^2}{8}$

$= 1.42 - 5.99 + 3.01 = -1.56^{k'}$ (top tension)

$d = \left(\frac{1.56}{.189}\right)^{.5} = 2.87''$

Uplift negligible,
provide floor thickness = 12"



$A_s = \frac{1.56}{1.31 \times 8} = 0.149 \text{ in}^2$

3/8"φ @ 8"c/c

Tempr & distr.

$A_s = .0025 \times 12 \times 12 = 0.36 \text{ in}^2$

3/8"φ@ 7"c/c

Foundation :

Existing G.L = +5.70 PWD.
Foundation level of sluice (at downstream pier) = +4.425 PWD.

Bore Hole No.	Depth below EGL.	Type of soil	SPT Value	Allow. bearing capacity	Fdn. level of sluice	Allowable bearing capacity at fdn. level	Remarks.
BH 1	5'	CLAY, trace fine sand	4	0.5 Tsf = 1.12 ksf	RL 4.625 M. PWD	1.0 ksf	Unconfined compression test not done. Bearing capacity has been obtained from table-1 of Appendix-B of Soil test Report prepd. by RRI
	5'-32'	Do	4 to 19	1.5 Tsf		—	
	32'-52'	CLAY, little fine sand	19 to 23	2.5 Tsf		—	
BH. 2	5'	CLAY, trace fine sand	11	1.375 Tsf = 3.0 ksf	R.L 4.425 M. PWD.	2.50 ksf	
	5'-32'	Do	11 to 20	2.0 Tsf		-	
	32'-52'	Do	20 to 25	2.80 Tsf		-	

Foundation of downstream pier :

Load from superstructure :

Soil = $\frac{63.87}{5.35}$ = 11.94^k

endwall = $\frac{18.59}{3.69}$ = 5.04^k

Pier = $\frac{6.36}{1.64}$ = 3.88^k

Base slab = $\frac{17.87}{3.28}$ = 5.45^k

Deck slab = $\frac{2.42}{1.64}$ = 1.48^k

a) Sub-Total = 27.79^k

a) = 27.79^k

Load of water
= 2.9 × 3.28 × 4.26 × 1.0
3.28 × .0625 = 8.31

Liveload 0.10 ksf
= 4.26 × 4.1 × .10 = 1.75^k

Self wt. of gate = 0.40^k

Pull on hoist = 2.57^k

Total $\Sigma V = 40.82^k$

$$\bar{X} = \frac{\Sigma M}{\Sigma V} = \frac{109.11}{40.82} = 2.67'$$

$$e = \frac{2 \times 3.28}{2} - 2.67 = 0.61' \text{ ok.}$$

Pressure on soil

$$P = \frac{\Sigma V}{A}\left(1 \pm \frac{6 \times e}{A}\right)$$

$$= \frac{40.82}{2 \times 3.28 \times 4.26}\left(1 \pm \frac{6 \times 0.61}{6.56 \times 4.26}\right)$$

$$= 1.46\,(1 \pm 0.131)$$

$$= 1.65 \text{ ksf max} < 2.50 \text{ ksf. } \underline{\text{ok.}}$$

$$= 1.269 \text{ ksf min.}$$

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DESIGN REPORT

ON

I) OPEN CHANNEL

II) BOX CHANNEL CULVERT

III) R.C.C. PIPE DRAINS

IV) BRICK SEWER

OPEN CHANNEL

IBRAHIMPUR KHAL

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DESIGN OF IBRAHIMPUR KHAL (OPEN CHANNEL)
(OLD AIRPORT RUNWAY TO IBRAHIMPUR ROAD CROSSING)

Area A =1.66 Km^2 (As per survey map)

Runoff factor C =0.5

Rainfall intensity (I) :

a) Upland flow = 6 x 666 = 3996 ft

Considering 1 m/sec upland flow. T=3996 /1 = 3996 sec = 66.6 min

b) Time for start of flow = 10 min

c) Khal length = 4 x 666 =2664 ft = 812 m

Considering 1 m/s velocity T = 812/1 = 812 sec =13.53 min

Therefore Tc (Time of concentration) =66.6 +5 +13.53 = 85.13 min

I from the graph = 65

Runoff (Q) =CIA

$$= \frac{1.66 \times 10^6 \times 0.5 \times 65}{3600 \times 1000}$$

$$= 14.96 = 15 \ m^3/s$$

Considering a khal velocity of 0.8 m/sec as the area is very flat

Therefore X- area of khal = 18.75 m^2

$= 1/2(2b + 2x)y = (b+x)y = (b+1.5y)y = 18.75$

or $b \times 3.0 + 1.5 \times 3.0 \times 3.0 = 18.75$

or $3.0\ b + 13.5 = 18.75$

or $3.0\ b = 18.75 - 13.5$

$b = 5.25/3.0 = 1.75$ m

BOX CHANNEL CULVERT

BEGUNBARI KHAL

CONTENTS

D06

DESIGN OF BEGUNBARI (BOX CULVERT)

Area A = 1.92 Sq.km

Runoff factor C =0.6

Rainfall intensity (I)

a) Upland flow = 1200 m

Assume 0.38 m/sec upland flow. T=1200/0.38 = 3157 sec = 52.61 min

b) Time for start of flow = 10 min

c) Khal length = 1000 m

Considering 1 m/s velocity T = 1000/1 = 1000 sec = 16.66 min

Therefore Tc (Time of concentration) = 52.61 + 10 + 16.66 = 79.27

I from the graph = 70

Runoff (Q) =CIA

$$= \frac{0.6 \times 57 \times 1.92 \times 10^3}{3600}$$

$$= 18.24 \ m^3$$

Area A = $18.24/1 = 18.24 \ m^2$

Size of culvert.

Assumed, H = 3.5 m (Without free board),

$$\frac{18.24}{3.5}$$

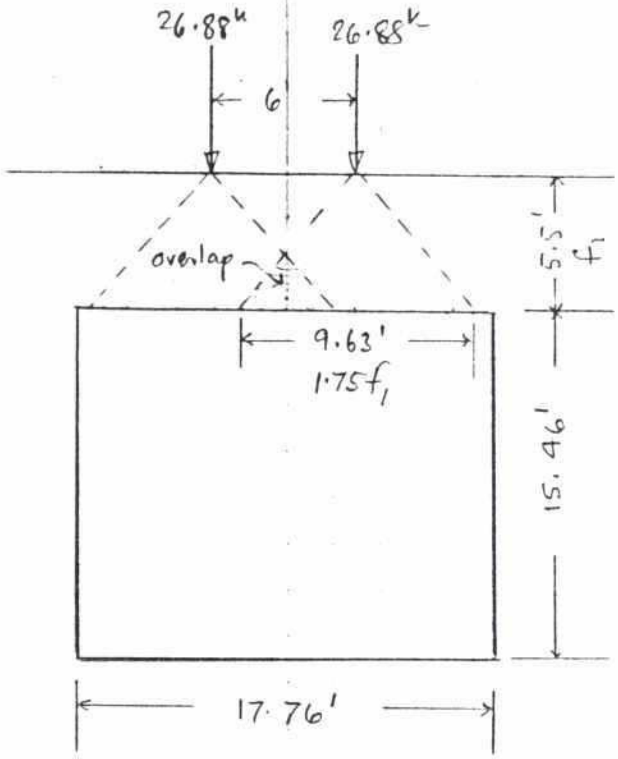
$$= 5.2$$

W = 5 m

Considering free board 300 mm

BEGUNBARI KHAL

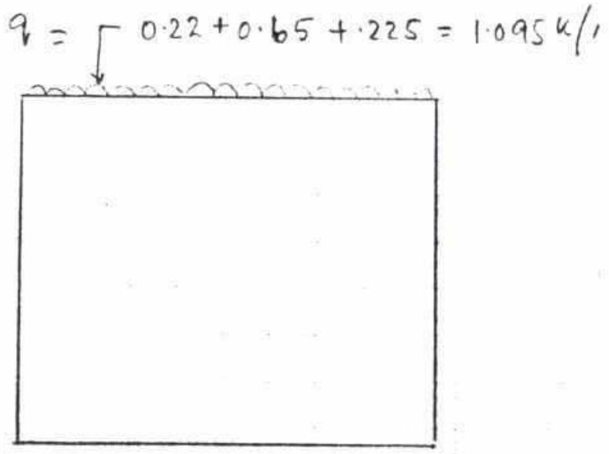
DESIGN OF CHANNEL CULVERT



1. vertical load due to traffic:

H20 loading
rare wheel load = 8T
= 17.92k
Impact 50% = 8.96k
Total = 26.88k

As the depth of fill is more than 2ft, the conc. load will be considered as uniformly distributed over a square, the side of which are equal to $1\frac{3}{4}$ times the depth of fill. (Ref. 1)

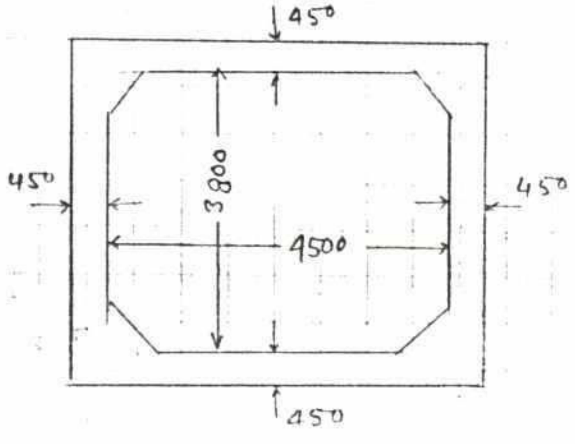


The total load will be considered uniformly distributed over the above mentioned area which is $9.63' + 6' = 15.63'$ square.

For design purpose the load may be considered to be uniformly distributed over the span length of the box.

Uniformly distributed load due to traffic.

$$= \frac{26.88 \times 2}{15.63^2} = 0.22^{k}/ft^2$$



2. load from earth fill & road:

$4.5 \times .115 + 1 \times .130 = 0.648$.

3. Load from top R.C. slab:

$\frac{18}{12} \times .15 = 0.225$ ksf.

4. load from two side walls:

$12.46 \times \frac{18}{12} \times .15 \times 2 = 5.61^{k/l}$

load on soil from wall $= \frac{5.61}{17.76} = 0.316$ ksf

5. Wt. of bot. slab:

$\frac{18''}{12} \times .15 = 0.225$ ksf

* By inspection, the effect of traffic load without overburden found less than the load of overburden.

6. Wt. of water (full depth flow):

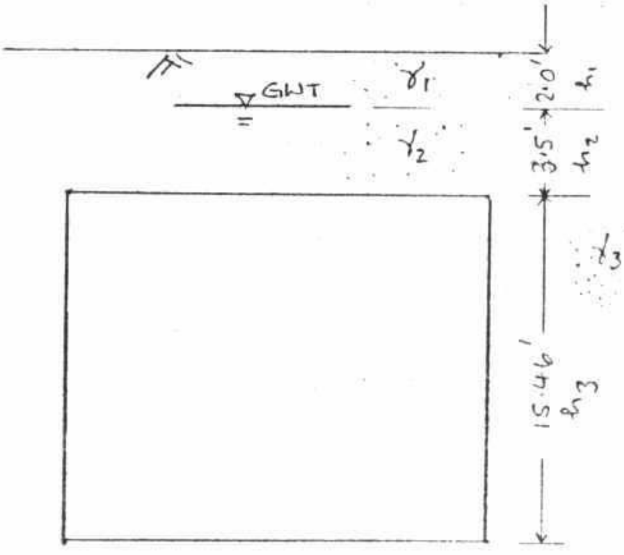
$12.46 \times .0624 = 0.78$ ksf

Total load on soil = 1) + 2) + 3) + 4) + 5) + 6)

$= .22 + .648 + 0.225 + 0.316 + .225 + 0.78$

$= 2.414$ ksf.

Computation of earth pressure:



fill material: Silty clay

assume $\phi = 20°$

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

$$= \frac{1 - \sin 20°}{1 + \sin 20°} = \frac{1 - .34}{1 + .34}$$

$$= 0.492$$

earth pressure on walls :

$$q_{ep_1} = K_o \gamma_3 h_3 + \gamma_w h_3$$

$$= 0.493 \times (115 - 62.4) \times 15.46 + 62.4 \times 15.46$$

$$= 400.9 + 964.70 = 1365.60 \ \#/\text{ft} = 1.365 \text{ ksf}.$$

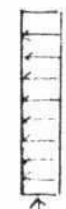
earth pressure on walls due to surcharge :

$$q_{ep_2} = K_o \gamma_1 h_1 + K_o \gamma_2 h_2 + \gamma_w h_2$$

$$= 0.493 \times 115 \times 2 + 0.493 \times (115 - 62.4) \times 3.5 + 62.4 \times 3.5$$

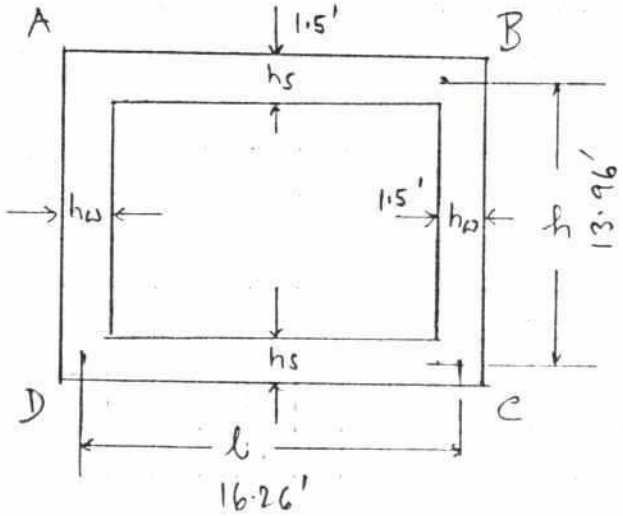
$$= 113.39 + 90.76 + 218.40 = 422.55 \text{ psf}$$

$$= 0.423 \text{ ksf}$$



Computations of Bending Moments:

$$k = \frac{h}{l}\left(\frac{h_s}{h_w}\right)^3 \quad (\text{Ref.2})$$



$K_1 = k + 1 = 0.86 + 1 = 1.86$

$K_2 = k + 2 = 0.86 + 2 = 2.86$

$K_3 = k + 3 = 0.86 + 3 = 3.86$

$K_4 = 4k + 9 = 4 \times .86 + 9 = 12.44$

$K_5 = 2k + 3 = 2 \times .86 + 3 = 4.72$

$K_6 = k + 6 = .86 + 6 = 6.86$

$K_7 = 2k + 7 = 2 \times .86 + 7 = 8.72$

$K_8 = 3k + 8 = 3 \times .86 + 8 = 10.58$

④

$$k = \frac{h}{l}\left(\frac{h_s}{h_w}\right)^3 = \frac{13.96}{16.26} \times \left(\frac{1.5}{1.5}\right)^3 = 0.858 \approx 0.86$$

case II Uniform load on roof:

$$\left.\begin{matrix} M_A \\ M_C \end{matrix}\right\} = -\frac{ql^2}{12k_1} = -\frac{1.095 \times 16.26^2}{12 \times 1.86} = -12.97^{k'}$$

case III wt. of walls:

$$M_A = \frac{q_1 l^2 k}{12 k_1 k_3} = \frac{0.315 \times 16.26^2 \times 0.86}{12 \times 1.86 \times 3.86} = 0.831^{k'}$$

$$q_1 = \frac{2G}{l + h_w} = \frac{5.61}{17.76} = 0.316 \text{ ksf}$$

$$M_C = -\frac{k_5}{k} M_A = -\frac{4.72}{0.86} \times .831 = -4.56^{k'}$$

Case IV Earth pressure on walls:

$$M_A = -\frac{q_{ep_1} h^2 k k_7}{60 k_1 k_3} = -\frac{1.365 \times 13.96^2 \times .86 \times 8.72}{60 \times 1.86 \times 3.86}$$

$$= -4.631^{k'}$$

$$M_C = \frac{k_8}{k_7} M_A = \frac{10.58}{8.72} \times (-4.63) = -5.618^{k'}$$

Case V Earth (Surcharge) pressure on walls:

$$\left.\begin{matrix} M_A \\ M_C \end{matrix}\right\} = -\frac{q_{ep_2} h^2 k}{12 k_1} = -\frac{0.423 \times 13.96^2 \times 0.86}{12 \times 1.86} = -3.28^{k'}$$

Case VI Hydrostatic (internal pressure)

$$q_{ip} = \gamma h = .0624 \times 12.46 = 0.778^{k/'}$$

$$M_A = +\frac{q_{ip} h^2 k k_7}{60 k_1 k_3} = \frac{.778 \times 13.96^2 \times .86 \times 8.72}{60 \times 1.86 \times 3.86}$$

$$= 2.64^{k'}$$

(5)

Case $\underline{VI}$ Contd.

$$M_C = \frac{k_8}{k_7} M_A = \frac{10.58}{8.72} \times 2.64 = 3.20^{k'}$$

Case $\underline{VII}$ Excess hydrostatic (internal pressure):

$$M_A = + \frac{q_{ip}\left(h^2 k k_3 + \ell^2 k_5\right)}{12 k_1 k_3}$$

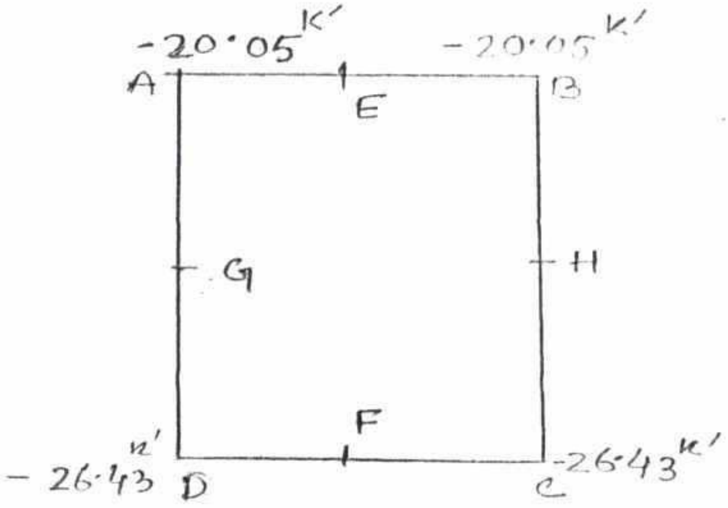
$$= \frac{0.778\left(13.96^2 \times 0.86 \times 3.86 + 16.26^2 \times 4.72\right)}{12 \times 1.86 \times 3.86}$$

$$= \frac{.778(646.9 + 1247.9)}{86.16} = 17.11^{k'}$$

$$M_C = + \frac{q_{ip}\, k\left(h^2 k_3 - \ell^2\right)}{12 k_1 k_3}$$

$$= + \frac{0.778 \times 0.86\left(13.96^2 \times 3.86 - 16.26^2\right)}{12 \times 1.86 \times 3.86}$$

$$= + 3.79^{k'}$$



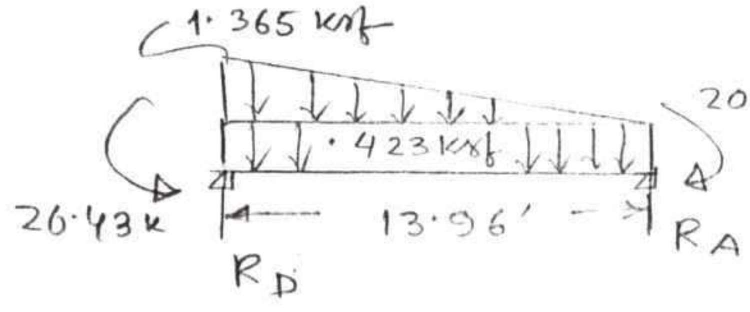
	M_A	M_C
II	-12.97	-12.97
III	0.831	-4.56
IV	-4.68	-5.62
V	-3.88	-3.28
	-20.05 k'	-26.43 k'

$$M_E = \frac{\omega l^2}{8} - \frac{M_A + M_B}{2} = \frac{1.095 \times 16.26^2}{8} - 20.05$$

$$M_E = 16.14^{k'}$$

$$M_F = \frac{1.409 \times 16.26^2}{8} - 26.43$$

$$M_F = 20.14^{k'}$$



$\Sigma M_D = 0$

$$13.96 R_A = \frac{.423 \times 13.96^2}{2} + \frac{1}{2} \times 1.365 \times 13.96^2 \times \frac{1}{3} + 20.05 - 26.43$$

$$R_A = 5.67$$

Let zero shear be distance from R_A

$$\therefore \quad 5.67 - 0.423x - \frac{1.365x^2}{2 \times 13.96} = 0$$

$$.0488x^2 + .423x - 5.67 = 0$$

$$x = \frac{-.423 \pm \sqrt{(.423)^2 + 4 \times 5.67 \times .0488}}{2 \times .0488}$$

$$x = 7.28'$$

$$+M_{7.28} = 5.67 \times 7.28 - \frac{.423 \times 7.28^2}{2} - 20.05$$

$$- \frac{1}{2} \times \frac{7.28^3 \times 1.365}{3 \times 13.96} = 3.73^{k'}$$

DESIGN Data

$f'_c = 2500$ psi $\qquad f_c = 1125$ psi

$f'_y = 60000$ psi $\qquad f_s = 2400$ psi

$n = \frac{E_s}{E_c} = 10 \qquad R = 189$ psi

$$\therefore f_s j = \frac{24}{12} \times .875 = 1.75$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{26.43 \times 12}{.189 \times 12}} = 11.82''$$

Let $t = 16 \qquad \therefore d = 16 - 3'' = 13''$

Steel Area Calculation.

provided

$- A_{sA} = \frac{20.05}{1.75 \times 13} = 0.88 \square''$, $3/4'' \phi$ $6''$ c/c $-$ 20mm ϕ @150 mm c/c

$+ A_{sE} = \frac{16.14}{1.75 \times 13} = 0.71 \square''$ $3/4'' \phi$ $7''$ c/c, 20 mm ϕ @125 mm c/c

$- A_{sc} = \frac{26.43}{1.75 \times 13} = 1.16 \square''$, $3/4'' \phi$ $5''$ c/c, 20 mm ϕ @125 mm c/c

$+ A_{sF} = \frac{20.14}{1.75 \times 13} = 0.88 \square''$, $3/4'' \phi$ $6''$ c/c 20 mm ϕ 160 mm c/c

$+ A_{sG} = \frac{3.73}{1.75 \times 13} = .16 \square''$

using minimum steel = .0025 bd = $0.39 \square''$

$5/8'' \phi$ $10''$ c/c, 16 mm ϕ 250 mm c/c

Foundation

Load on soil = 2.414 ksf (soft soil as shown in bore Log H_1 H_2 H_3)

using timber pile of 18^K capacity

$$\frac{18}{2.414} = 7.456 \text{ ft}^2 = 2.73 \text{ ft} \simeq .83 \text{ m} \times .83 \text{ m}$$

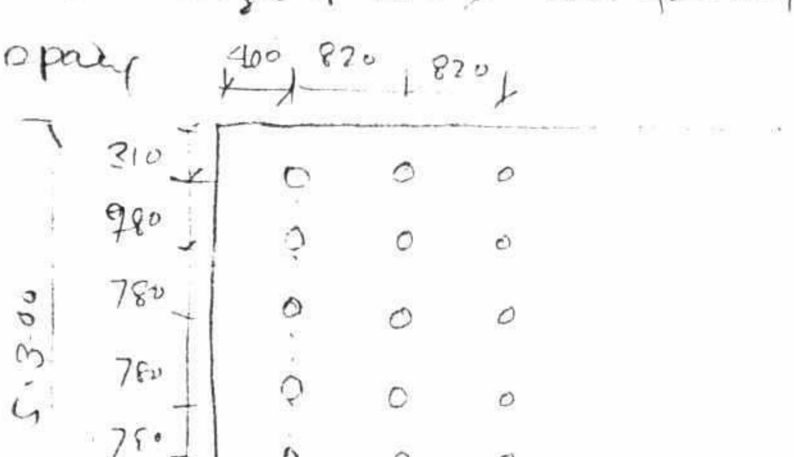
Design of foundation:

From pg. 1. load on soil = 2.04 ksf.

Field bore log of H_1, H_2 & H_3 shows soft layer upto depth of 30 ft. below G.L. The box rests at ± 19 to 20' below G.L. Timber pile is suggested. (Compacted sand filling of soft layer below the box upto hard stratum is not suggested for large thickness of filling and excavation difficulty at deeper depth below G.W.T.)

Using 16k pile, area under each pile = $\frac{16}{2.414}$ = 6.62 sft
2.57' × 2.57'
spacing of pile

For the size of box use following spacing



$$V_J = \frac{6 \times 500'' \times 12''}{\frac{1}{3}'' \times 6}$$

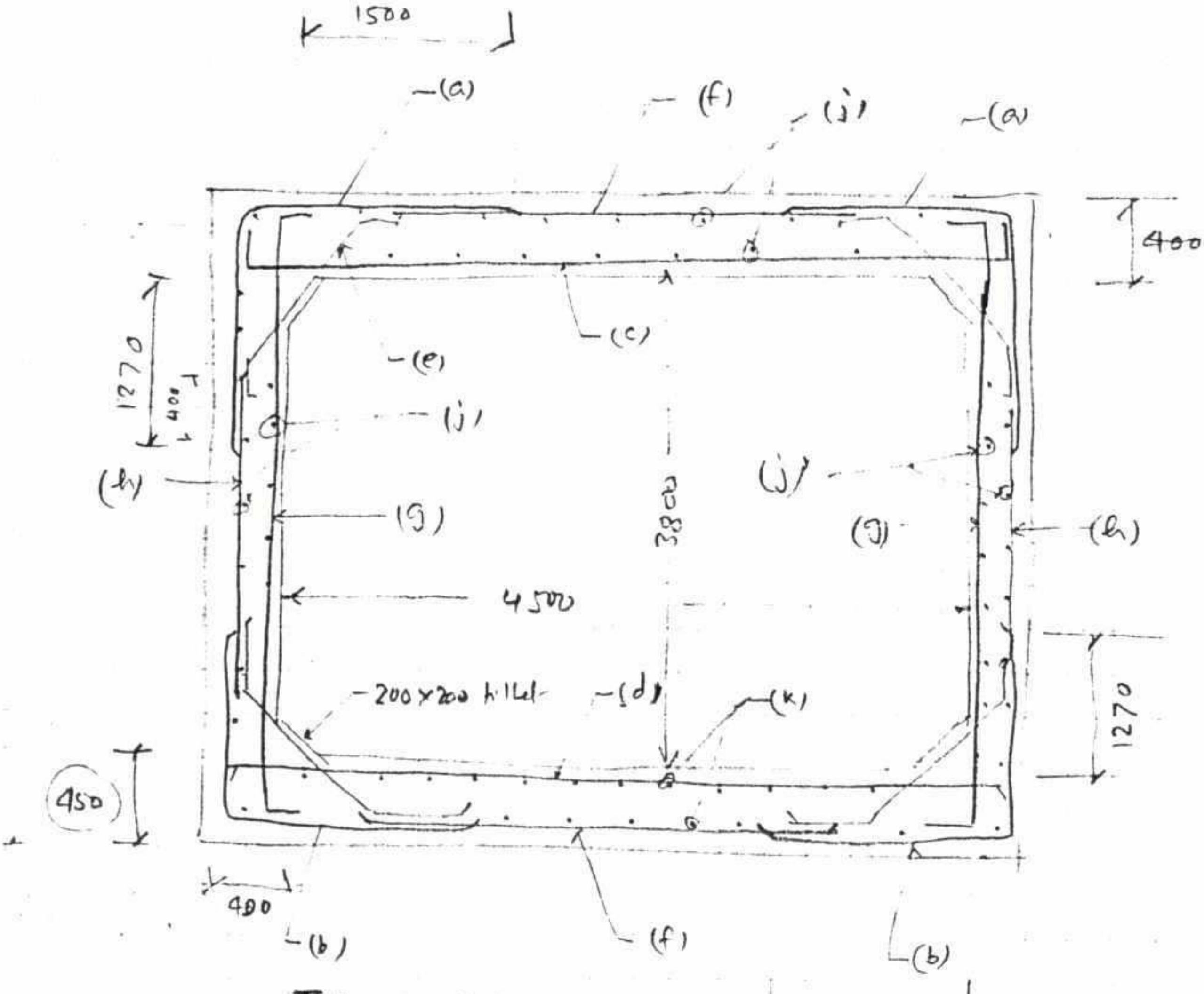
Sct F.S.

= 18,000 #

= 18k.

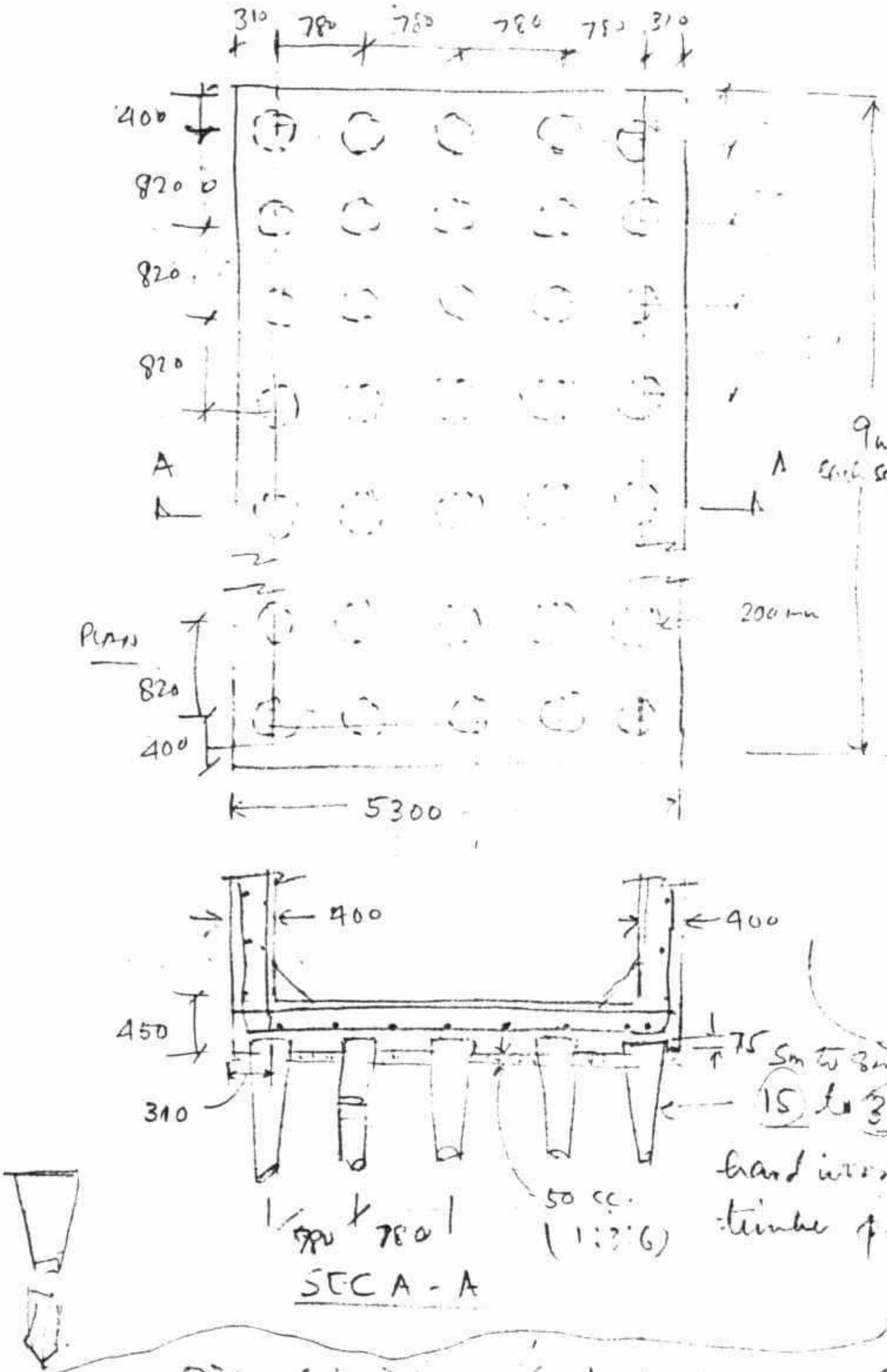
280

(10)



TYPICAL SECTION OF COVERED DRAIN.

(a)	Y20 - 150	—	top bar of top slab.
(b)	Y20 - 125	—	bot bar of bot. slab
(c)	Y20 - 125	—	bot bar of top slab
(d)	Y20 - 100	—	top bar of bot slab
(e)	Y12 - 200	—	fillet bar on each flat
(f)	Y12 - 150	—	top bar of top slab
(g)	Y16 - 200	—	Inner vertical bar of wall
(h)	Y12 - 150	—	outer vertical " " " .
(j)	Y12 - 225	—	Dist. bar of walls.
(k)	Y12 - 180	—	" " " top & bot slabs.



Notes:

Piling:

1. All piles to be untreated barkfree hard wood timber piles having a minimum dia. of 125mm at their end under the bark.

2. Piles to be driven by a 600lb drop hammer from a free fall of 6ft and met with a set of 1/4"

3. Payment for piles will be made on driven length only.

4. Pile driving record to be kept as follows for each pile:

i) Length of pile taken
ii) Length of pile driven
iii) Their end perimeter of the pile driven.
iv) No. of strokes reqd. for last foot penetration of pile.

788

GWD-1 / FORM-3

BANGLADESH WATER DEVELOPMENT BOARD
GROUND WATER DIVISION-1
GROUND WATER CIRCLE

FIELD BORE LOG

+7.409 m

PROJECT Dhaka Integrated Flood Protection

SITE Begunbari Khal

LOCATION Begunbari Khal

HOLE NO. — GROUND LEVEL N/S

CLIENT Project Director, Project management off ...

GROUND WATER LEVEL depth 10'-2" AT 0700 HRS OF DATE 24-2-92

DATE STARTED 23-2-92 DATE COMPLETED 23-2-92

DRILLED BY GWD-1; UNIT 2.5

TOTAL DEPTH 72'

NO OF SAMPLE	TYPE OF SAMPLE	SCALE	DESCRIPTION OF MATERIALS	PERMEABILITY	DENSITY	COLOUR	MOISTURE	DILATANCY	BLOWS ON SPOONS 3-6	6-12	12-18	18-24	BLOWS ON CASING	PENETRATION TEST NO. OF BLOWS PER 1 FT. PENET	REMARKS
D_1			SILT AND CLAY trace very fine sand		SOFT	Light Brown	MOIST	PLASTIC	0	1	1	2			19.70'
D_2		10'							0	1	1	2			
D_3					M/S				1	3	4	5			
			18'												
U_1		20'	SILT AND CLAY			GREY		SEMI-PLASTIC	Shelby Tube						4.60 Size - 36"x3" Recovery = 30"
			24'												
D_4			SILT, some VFS, little clay, trace mica						0	1	1	2			
			29'												
U_2		30'	SILT AND clay, trace VFS						Shelby Tube						Size - 36"x3" Recovery = 23"
			34'												
D_5			VERY FINE SAND, little Silt, trace mica		M. DENSE				8	13	15	20			
D_6		40'				Brown	WET		10	15	18	24			
D_7									10	16	20	26			
D_8		50'			DENSE				11	17	24	30			23/2/92
			54'												
D_9			FINE SAND trace Silt, mica			Grey		NON-PLASTIC	11	18	23	30			
D_{10}		60'							12	18	22	30			
D_{11}									12	15	22	27			
D_{12}		70'	72'						13	20	25	30			

(মোঃ ... আলী)
উপ-সহকারী প্রকৌশলী
... গ্রাউন্ড ওয়াটার ডিভিশন-১
পাউবো, ঢাকা

BWD-1 / FORM-3

BANGLADESH WATER DEVELOPMENT BOARD
GROUND WATER DIVISION-1
GROUND WATER CIRCLE

FIELD BORE LOG

PROJECT Dhaka Integrated Flood Protection
LOCATION Begunbari Khal
CLIENT PD. Project Management (F) BWDB Dhaka
DATE STARTED 25-2-92 DATE COMPLETED 25-2-92
DRILLED BY GWD-1 UNIT 2, 5

SITE Begun Bari Khal
HOLE NO [redacted] GROUND LEVEL N/S
GROUND WATER LEVEL 2'-6" AT 0700 HRS OF DATE 26/2/92
TOTAL DEPTH 72'

NO OF SAMPLE	TYPE OF SAMPLE	SCALE	DESCRIPTION OF MATERIALS	PERMEABILITY	DENSITY	COLOUR	MOISTURE	DILATANCY	BLOWS ON SPOONS 6	12	18	24	BLOWS ON CASING	PENETRATION TEST (NO OF BLOWS PER 1 FT. PENET)	REMARKS
D1			VERY FINE SAND, some silt, trace mica		LOOSE	GREY	MOIST	PLASTIC	1	2	2	3			D1 → Sample missed
D2									2	2	3	4			
			13'												
D3			SILT AND CLAY		SOFT				1	2	2	3			
			18'												
U1			SILT, some VFS, little clay						Shelby Tube						L = 36" x 3' Recovery = 36"
			24'												
D4			VERY FINE SAND little silt, trace clay		MEDIUM DENSE	GREYISH BROWN	WET	SEMI-PLASTIC	5	8	10	12			
			29'												
D5		30	VERY FINE TO FINE SAND, little silt, trace mica						6	9	13	17			
D6									7	10	13	16			
D7									6	10	14	17			
D8									7	11	15	19			
D9		50							10	15	19	25			
D10					DENSE				11	16	20	26			
			58'												
D11			VERY FINE SAND, little silt, trace mica		V. DENSE			NON PLASTIC	13	18	30	35			
D12									16	30	32	46			
D13		70	72'						16	22	35	-			

27/2/92

(মোজাম্মেল আলী)
ভূ-তত্ত্ববিদ
ভূগর্ভস্থ পানি ডিভিশন-১
পাউবো, ঢাকা

BWD-17 FORM-3

BANGLADESH WATER DEVELOPMENT BOARD

GROUND WATER DIVISION-I

GROUND WATER CIRCLE

FIELD BORE LOG

PROJECT Dhaka Integrated Flood Protection

SITE Begunbari khal

LOCATION Begunbari Khal

HOLE NO [illegible] GROUND LEVEL N/S

CLIENT F.D. Project Management (I)

GROUND WATER LEVEL 7'-11" AT 1700 HRS OF DATE 27/2/97

DATE STARTED 26-2-97 DATE COMPLETED 26-2-97

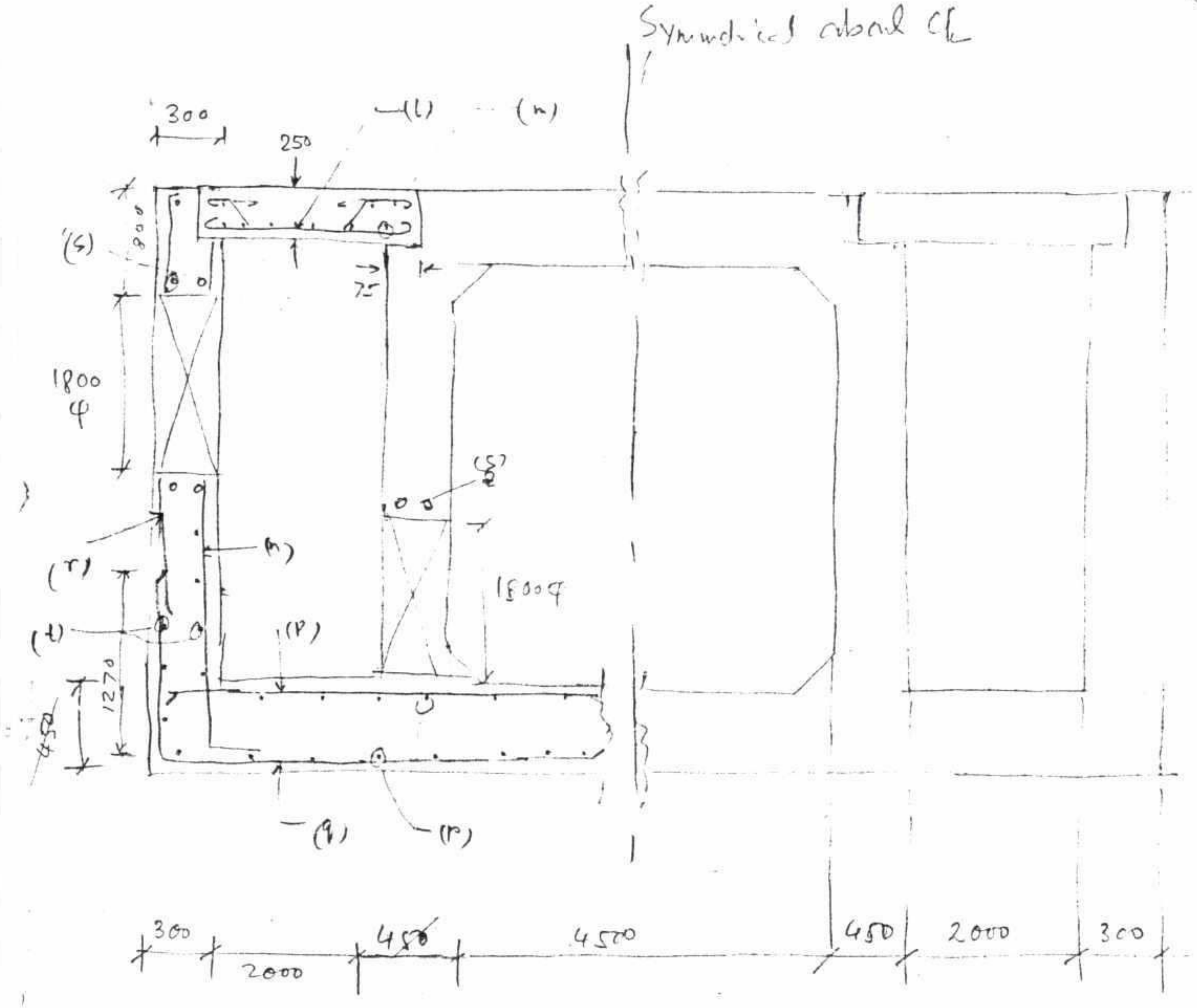
TOTAL DEPTH 72'

DRILLED BY GWD-I UNIT No. 5

NO OF SAMPLE	TYPE OF SAMPLE	SCALE	DESCRIPTION OF MATERIALS	PERMEABILITY	DENSITY	COLOUR	MOISTURE	DILATANCY	BLOWS ON SPOON				BLOWS ON CASING	PENETRATION TEST NO. OF BLOWS PER 1 FT. PENET.	REMARKS
			SILT AND CLAY, little Organic matter												
D_1									0	1	1	2			
			8'												
D_2			SILT AND CLAY, little v.f. Sand trace Organic						0	1	2	3			
D_3			SILT AND CLAY, trace very fine Sand						1	2	2	4			
U_1															Shelby Tube 36" x 3", Recovery = 28"
			23'												
D_4			SILT AND VERY FINE SAND, trace clay						4	5	6	7			
D_5									4	6	7	9			
			34'												
D_6			VERY FINE SAND, little Silt, trace mica		MEDIUM DENSE				5	7	10	13			
D_7									6	8	11	15			
D_8									8	13	17	21			
D_9									11	14	18	23			
D_{10}									12	17	21	26			
D_{11}									12	18	22	28			
D_{12}									17	20	24	30			

27/2/97

(বাদশাহ মিয়া)
প্রকৌশলী
ভূগর্ভস্থ পানি বিভাগ-১
পাপাবো, ঢাকা



(l) Y16 - 125
(m) Y12 - 150
(n) Y16 - 100
(p) Y12 - 200
(q) Y20 - 200
(r) Y16 - 200
(s) Y12 - 2nos. rinf.
(t) Y10 - 150

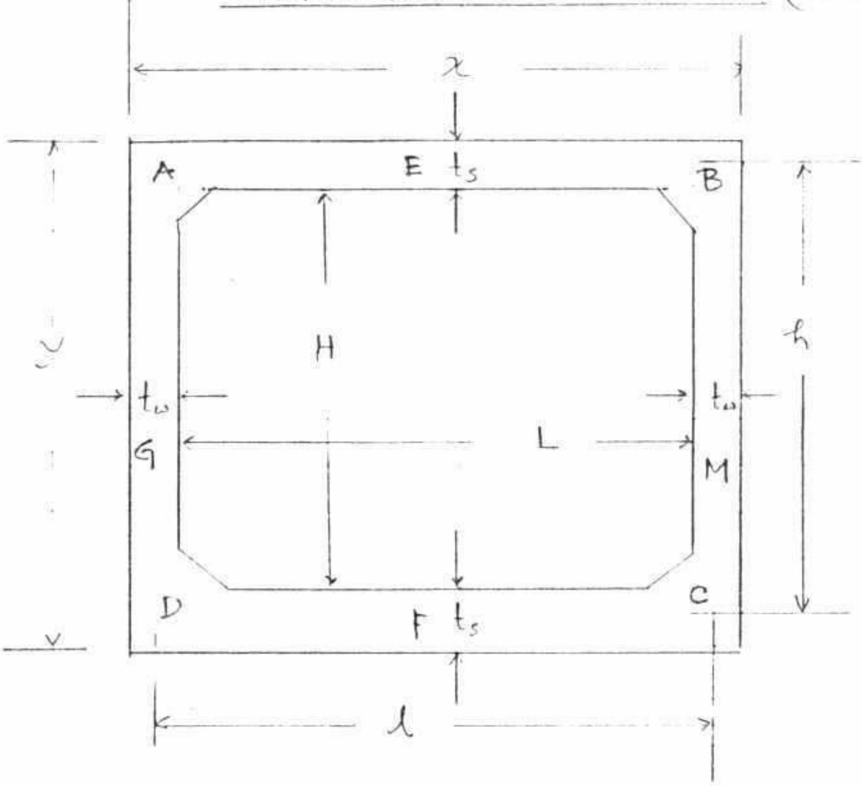
TYPICAL SECTION OF BOX SEWER WITH AT GULLY PIT

BOX CHANNEL CULVERT

SEGUNBAGICHA KHAL (SECTION K 5-3)

CONTENTS | PAGE

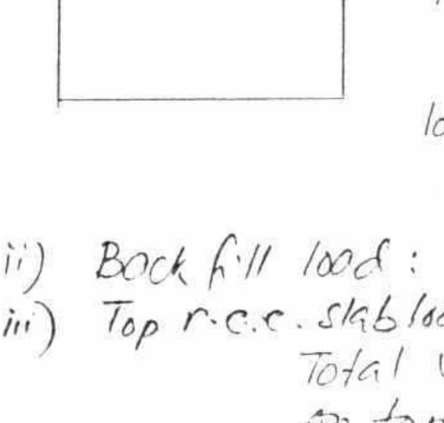
SEGUNBAGICHA KHAL (SECTION NO. K-5-3. JICA REPORT) ①



t_s = 500 = 1.64'
t_w = 450 = 1.476'
H = 4300 = 14.10'
L = 5500 = 18.04'
h = 4800 = 15.74'
l = 5950 = 19.35'
x = 6400 = 20.99'
y = 5300 = 17.38'

Computation of loads:

i) Traffic load: H20 loading



rare wheel load = 8^T = 17.92^k
Impact 30% = 5.38
23.30^k

load distribution area
= 9.63 + 6 = 15.63

load = $\frac{23.30 \times 2}{15.63^2}$ = $0.191^k/ft^2$

ii) Back fill load: 5.5 × .115 = $0.6325\ ^k/ft^2$
iii) Top r.c.c. slab load: $\frac{20}{12}$ × .15 = 0.250 ksf
Total vertical load = 0.191 + 0.6325 + .25
on top slab = 1.074 ksf.

lateral loads:

Triangular distribution of earth fill load on side walls: $\phi = 20^\circ$, $k_o = \frac{1-\sin\phi}{1+\sin\phi}$

$= 0.493$

i) $q_{ep} = K_o \gamma h + \gamma_w h$

$= 0.493 \times (115 - 62.4) \times 17.38 + 62.4 \times 17.38$

$= 450.69 + 1084.51 = 1535.20 = \underline{1.54\ ksf}$

Rectangular Distribution of surcharge load on side walls:

ii) $q_{ep} = k_o \gamma h_1 + k_o \gamma_1 h_2 + \gamma_w h_3$

$= 0.49 \times 115 \times 4 + 0.49 \times (115 - 62.4) \times 1.5$
$+ 62.4 \times 1.5$

$= 225.40 + 38.66 + 93.6 = 357.66 = \underline{0.358\ k/ft}$

Vertical load of side walls:

$q_1 = 14.10 \times 1.476 \times .15 \times 2 = 6.24$

Distributed on soil $= \frac{6.24}{20.99} = \underline{0.297\ k/ft^2}$

$$k = \frac{h}{\ell}\left(\frac{t_s}{t_w}\right)^3 = \frac{15.74}{19.35}\left(\frac{1.64}{1.476}\right)^3 = 1.115$$

case II Uniform load on roof:

$$\left.\begin{matrix} M_A \\ M_C \end{matrix}\right\} = -\frac{w\ell^2}{12\,(k+1)}$$

$$= -\frac{1.074 \times 19.35^2}{12\,(1.115+1)} = -15.84\ k'$$

case III wt. of side walls:

$$M_A = +\frac{q_1 \ell^2}{12}\left\{\frac{k}{(k+1)(k+3)}\right\}$$

$$= +\frac{.297 \times 19.35^2}{12} \times \frac{1.115}{2.115 \times 4.115}$$

$$= +1.187^{k'}$$

$$M_C = -M_A \times \frac{2k+3}{k}$$

$$= -1.187 \times \frac{2 \times 1.115 + 3}{1.115} = \underline{-5.42^{k'}}$$

case IV Earth pressure on walls:

$$M_A = -\frac{q_{ep} h^2}{60} \times \left\{\frac{k \times (2k+7)}{(k+1) \times (k+3)}\right\}$$

$$= -\frac{1.54 \times 15.74^2}{60} \times \frac{1.115\,(2 \times 1.115 + 7)}{(1.115+1)(1.115+3)}$$

$$= \underline{-7.52^{k'}}$$

$$M_C = +M_A \frac{3K+8}{2K+7}$$

$$= 7.52 \times \frac{3\times1.115+8}{2\times1.115+7}$$

$$= 7.52 \times \frac{11.345}{9.23} = \underline{+9.24^{k'}}$$

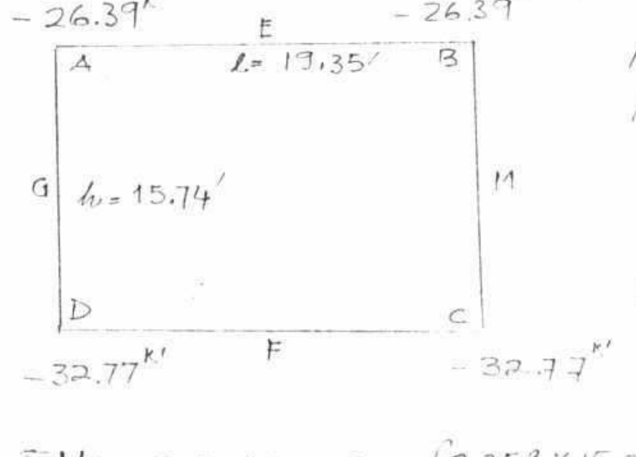
Case $\underline{V}$ Earth (Surcharge) pressure on walls:

$$\left.\begin{matrix} M_A \\ M_C \end{matrix}\right\} = -\frac{q_e p h^2}{12} \times \frac{K}{K+1}$$

$$= -\frac{0.358\times15.74^2}{12} \times \frac{1.115}{2.115} \underline{= -3.90^{k'}}$$

$$M_A = -15.84 + 0.807 - 7.52 - 3.90 = -26.39\ k'$$

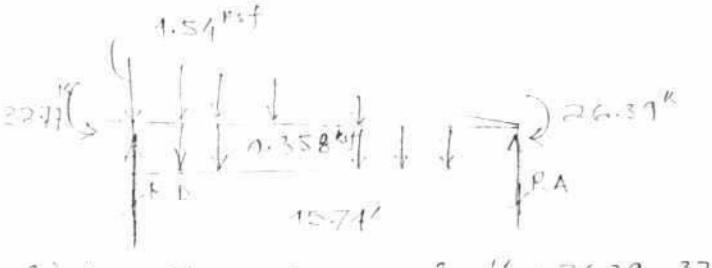
$$M_C = -15.84 - 3.755 - 9.24 - 3.90 = -32.77\ k'$$



$w = 1.074 \quad w_1 = 1.371$

$$M_E = \frac{wL^2}{8} - \frac{M_A + M_B}{2} = \frac{1.074 \times 19.35^2}{8} - 26.39 = \underline{23.88^{k'}}$$

$$M_F = \frac{wL^2}{8} - \frac{M_C + M_D}{2} = \frac{1.371 \times 19.35^2}{8} - 32.77 = \underline{31.40^{k'}}$$



$\sum M_D = 0 ;\quad 15.74 R_A = (0.358 \times 15.74^2)/2 + \tfrac{1}{2} \times 1.54 \times 15.74^2 \times \tfrac{1}{3} + 26.39 - 32.77$

$R_A = 6.45^{k}$

Considering 0-shear at x from "A, then, $6.45 - 0.358x - \dfrac{1.54x^2}{2 \times 15.74} = 0$

or, $0.0489x^2 + 0.358x - 6.45 = 0$

$x = \left(-0.358 + \sqrt{(0.358)^2 + 4 \times 6.45 \times 0.0489}\right) / 2 \times 0.0489$

$x = \underline{7.82'}$

$+M_{7.82} = 6.45 \times 7.82 - (0.358 \times 7.82^2)/2 - 26.39 - \tfrac{1}{2}(7.82^3 \times 1.54)/3 \times 15.74 = \underline{5.32^{k}}$
(G)

Checking d, $d = \sqrt{M/Rb} = \sqrt{32.77 \times 12 / 189 \times 12} = 12.16''$ $\quad \boxed{R = 189}$

Let $t = 20''$ Therefore $d = 17''$ $\quad f_s j = 1.75$

<u>Steel Area Calculation</u>

$-A_{sA} = \dfrac{26.39}{1.75 \times 17} = 0.88 \square''$, $\quad$ 20 ф 150

$+A_{sE} = \dfrac{23.88}{1.75 \times 17} = 0.80 \square''$; $\quad$ 20 ф-125 or 22 ф 170

$-A_{sc} = \dfrac{32.77}{1.75 \times 17} = 1.10 \square''$, $\quad$ 20 ф-125 or 22 ф 170

$+A_{sF} = \dfrac{31.40}{1.75 \times 17} = 1.05 \square''$. $\quad$ 20 ф-100 or 22 ф 130

$+A_{sG} = \dfrac{5.32}{1.75 \times 17} = 0.18 \square''$.

$Min^{m} = 0.0025 \times 12 \times 17 = 0.51 \square''$: $\quad$ 16 ф-325

2228

6

Foundation:

Load on soil : $1.074 + .262 + .879 + .275 = 2.43$ ksf.

Wt. of full flow water
$= .0624 \times 14.10 = 0.879$

Wt. of bottom slab =
$= \frac{22}{12} \times .15 = 0.275$

Using 16k pile,
spacing $= \frac{16}{2.43}$
$= 6.57 ft^2$
$= 2.56' \times 2.56'$
$= .782 \times .782$ m.

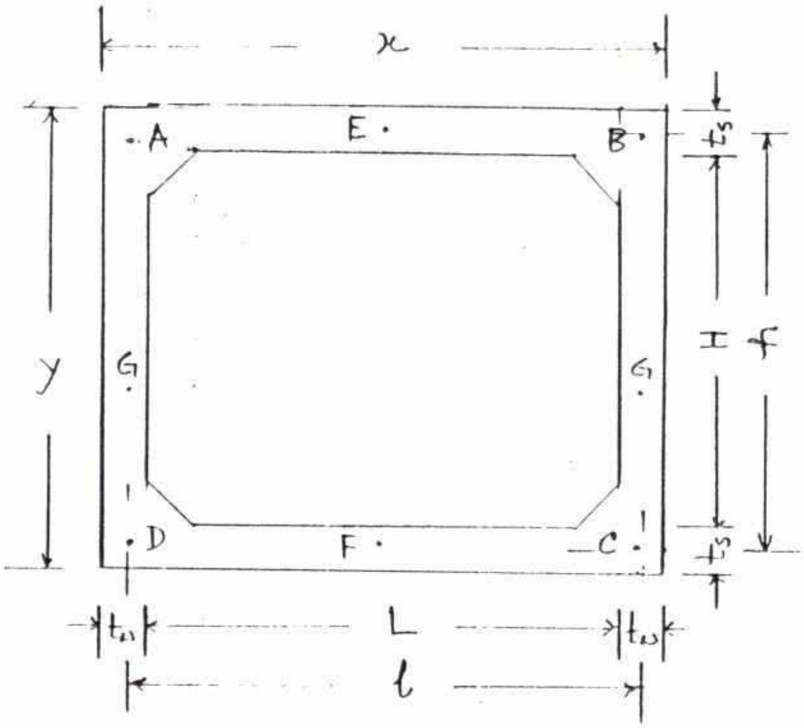
BOX CHANNEL CULVERT

SEGUNBAGICHA KHAL (SECTION K5-4)

CONTENTS

SEGUNBAGICHA KHAL

Section k-5-4 JICA REPORT



$H = 4.3m = 14.10'$
$L = 5.0m = 16.40'$
$t_s = 450\,mm = 1.48'$
$t_w = 400\,mm = 1.31'$
$h = 4.75m = 15.58'$
$l = 5.40m = 17.71'$
$x = 5.80m = 19.02'$
$y = 5.20m = 17.06'$

$$k = \frac{h}{l}\left(\frac{t_s}{t_w}\right)^3$$
$$= \frac{15.58}{17.71} \times \left(\frac{1.48}{1.31}\right)^3$$
$$= 1.269 \approx 1.27$$

$k_1 = k + 1 = 1.27 + 1 = 2.27$

$k_2 = k + 2 = 1.27 + 2 = 3.27$

$k_3 = k + 3 = 1.27 + 3 = 4.27$

$k_4 = 4k + 9 = 4 \times 1.27 + 9 = 14.08$

$k_5 = 2k + 3 = 2 \times 1.27 + 3 = 5.54$

$k_6 = k + 6 = 1.27 + 6 = 7.24$

$k_7 = 2k + 7 = 2 \times 1.27 + 7 = 9.54$

$k_8 = 3k + 8 = 3 \times 1.27 + 8 = 11.81$

1. Traffic load: ·191 ksf

2. Back fill load: ·633 ksf

3. Top r.c.c. slab: ·222 "

1·046

4. Earth fill load on side walls: 1·51 "

5. Surcharge : ·358 "

6. Vertical load on side walls: : ·268 ksf.

Case II uniform load on roof:

$$\left.\begin{matrix}M_A\\M_C\end{matrix}\right\} = -\frac{ql^2}{12}\times\frac{1}{k_1} = -\frac{1.046\times 17.71^2}{12}\times\frac{1}{2.27} = \underline{-12.04^{k'}}$$

Case III wt. of walls:

$$M_A = \frac{q_1 l^2}{12}\times\frac{k}{k_1 k_3} = \frac{.268\times 17.71^2}{12}\times\frac{1.27}{2.27\times 4.27} = \underline{0.918^{k'}}$$

$$M_C = -M_A\times\frac{k_5}{k} = -.918\times\frac{5.54}{1.27} = \underline{-4.00^{k'}}$$

Case IV earth pressure on walls:

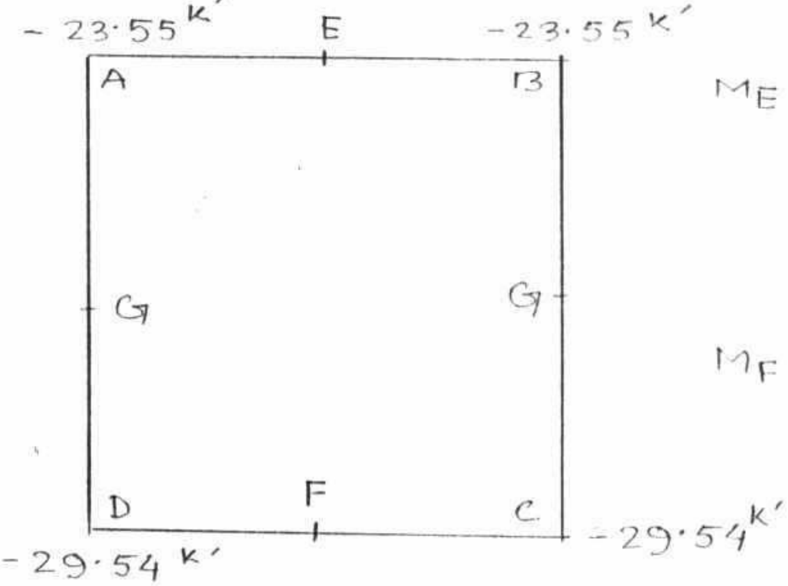
$$M_A = -\frac{q_{ep}h^2}{60}\times\frac{k k_7}{k_1 k_3} = -\frac{1.51\times 15.58^2}{60}\times\frac{1.27\times 9.54}{2.27\times 4.27} = \underline{-7.64^{k'}}$$

$$M_C = M_A\times\frac{k_8}{k_7} = -7.64\times\frac{11.81}{9.54} = \underline{-9.45^{k'}}$$

Case V: Surcharge pressure on walls:

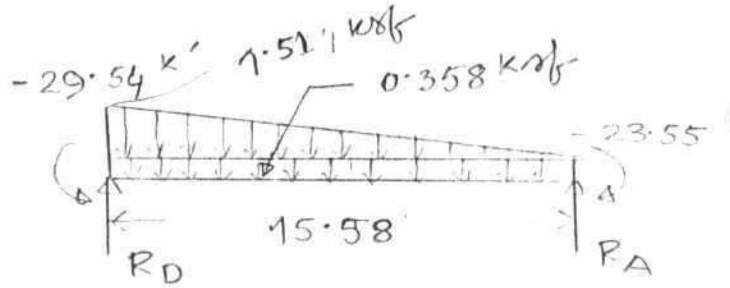
$$\left.\begin{matrix}M_A\\M_C\end{matrix}\right\} = -\frac{q_{eR}h^2}{12}\times\frac{k}{k_1} = \frac{-.358\times 15.58^2}{12}\times\frac{1.27}{2.27} = \underline{-4.05^{k'}}$$

$M_A = -12.04 + .918 - 7.64 - 4.05 = -23.55^{K'}$
$M_c = -12.04 - 4.00 - 9.45 - 4.05 = -29.54^{K'}$



$$M_E = \frac{\omega l^2}{8} - \frac{M_A + M_B}{2}$$
$$= \frac{1.046 \times 17.71^2}{8} - 23.55$$
$$= 17.46^{K'}$$

$$M_F = \frac{1.314 \times 17.71^2}{8} - 29.54$$
$$= 21.98^{K'}$$



$\Sigma M_D = 0$

$$15.58 R_A = \frac{.358 \times 15.58^2}{2} + \frac{1}{2} \times \frac{1.51 \times 15.58^2}{3} + 23.55 - 29.54$$

$$R_A = 6.33^K$$

Let zero shear x distance from R_A

$$6.33 - .358x - \frac{1.51 x^2}{2 \times 15.58} = 0$$

$$.048x^2 + .358x - 6.33 = 0$$

$$x = \frac{-.358 \pm \sqrt{(.358)^2 + 4 \times .048 \times 6.33}}{2 \times .048}$$

$$x = 8.37'$$

$$M_{8.34} = 8.34 \times 6.33 - \frac{.358 \times 8.34^2}{2} - 23.55 - \frac{1 \times 8.34 \times 1.51^3}{2 \times 3 \times 15.58}$$

$$M_{8.34} = 7.42^{k'}$$

$$d = \sqrt{\frac{29.54}{.189}} = 12.5'' \quad \text{Prd} \quad d = 13''$$

$$-A_{sA} = \frac{23.55}{1.75 \times 13} = 1.03 \, \square'' \quad Y20 - 135 \text{ mm c/c}$$

$$-A_{sC} = \frac{29.54}{1.75 \times 13} = 1.3 \, \square'' \quad Y20 - 125 \text{ mm c/c}$$

$$+A_{sE} = \frac{17.46}{1.75 \times 13} = 0.76 \, \square'' \quad Y20 - 135 \text{ mm c/c}$$

$$+A_{sF} = \frac{21.98}{1.75 \times 13} = 0.96 \, \square'' \quad Y20 - 110 \text{ mm c/c}$$

$$-A_{sG} = \frac{7.42}{1.75 \times 13} = 0.33 \, \square'' \quad Y16 - 200 \text{ mm c/c}$$

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Foundation:

Total load on soil

$1.046 + .268 + .88 + .25 = \underline{2.44 ksf}$.

Water load $= 14.10 \times .0624 = 0.88 ksf$

Bot slab load $= \frac{20}{12} \times .15 = 0.25$

16^k pile, $\frac{16}{2.44} = 6.56$ Sft $= 2.56' \times 2.56'$ pile spacing.

BOX CHANNEL CULVERT

SEGUNBAGICHA KHAL (SECTION K-5-5)

CONTENTS

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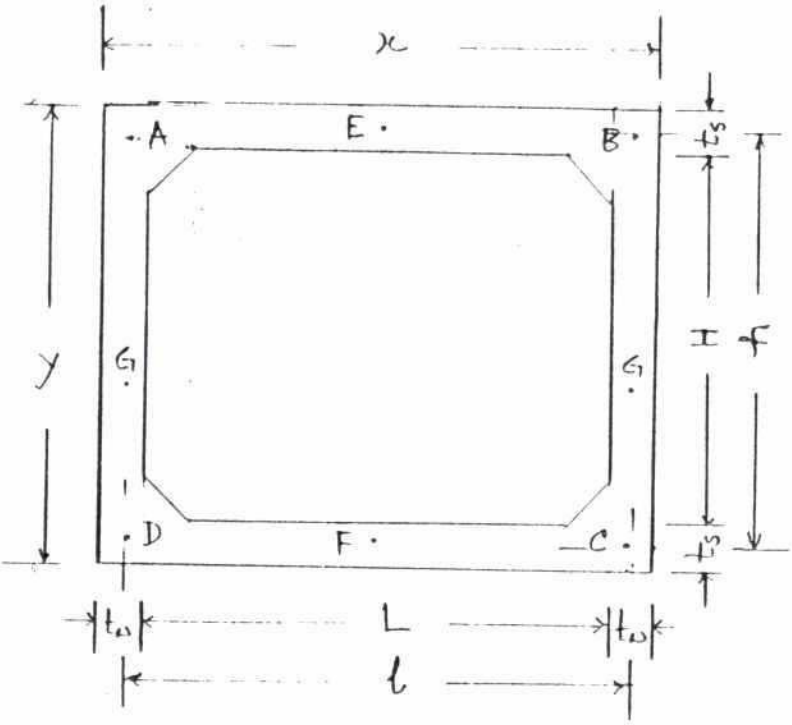
SEGUNBAGICHA KHAL.   SECTION K-5-5 JICAREPORT



$H = 4.30M = 14.10'$

$L = 4.00M = 13.12'$

$t_s = 400 = 1.31'$

$t_w = 400 = 1.31'$

$h = \qquad = 15.41'$

$l = \qquad = 14.43'$

$x = \qquad = 16.72'$

$y = \qquad = 15.74$

$$k = \frac{h}{l}\left(\frac{t_s}{t_w}\right)^3$$

$$= \frac{15.41}{14.43}\left(\frac{1.31}{1.31}\right)^3$$

$$= 1.068$$

$k_1 = k+1 = 1.068 + 1 = 2.068$

$k_2 = k+2 = 1.068 + 2 = 3.068$

$k_3 = k+3 = 1.068 + 3 = 4.068$

$k_4 = 4k + 9 = 4 \times 1.068 + 9 = 13.272$

$k_5 = 2k + 3 = 2 \times 1.068 + 3 = 5.136$

$k_6 = k + 6 = 1.068 + 6 = 7.068$

$k_7 = 2k + 7 = 2 \times 1.065 + 7 = 9.136$

$k_8 = 3k + 8 = 3 \times 1.065 + 8 = 11.204$

Case II: $\left.\begin{matrix} M_A \\ M_C \end{matrix}\right\} - \dfrac{qL^2}{12} \times \dfrac{1}{k_1} = \dfrac{-1.046 \times 10.43^2}{12} \times \dfrac{1}{2.068} = -8.78^{k'}$

Case III: $M_A = \dfrac{q_1 L^2}{12} \times \dfrac{k}{k_1 k_3} = \dfrac{.265 \times 10.43^2}{12} \times \dfrac{1.068}{2.068 \times 4.068} = 0.59^{k'}$

$M_C = -M_A \times \dfrac{k_5}{k} = -.59 \times \dfrac{5.136}{1.068} = -2.84^{k'}$

Case IV $M_A = -\dfrac{q_{ep} h^2}{60} \times \dfrac{k k_7}{k_1 k_3} = \dfrac{-1.51 \times 15.41^2}{60} \times \dfrac{1.068 \times 9.136}{2.068 \times 4.068} = -6.93^{k'}$

$M_C = M_A \times \dfrac{k_8}{k_7} = -6.93 \times \dfrac{11.204}{9.136} = -8.50^{k'}$

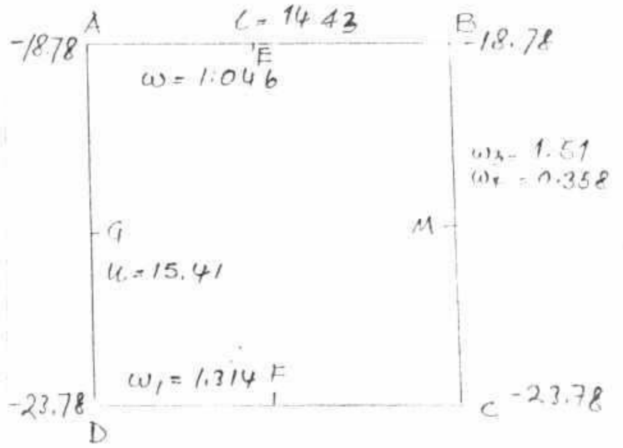
Case V $\left.\begin{matrix} M_A \\ M_C \end{matrix}\right\} = -\dfrac{q_{ep} h^2}{12} \times \dfrac{k}{k_1} = -\dfrac{.358 \times 15.41^2}{12} \times \dfrac{1.068}{2.068} = -3.66^{k'}$

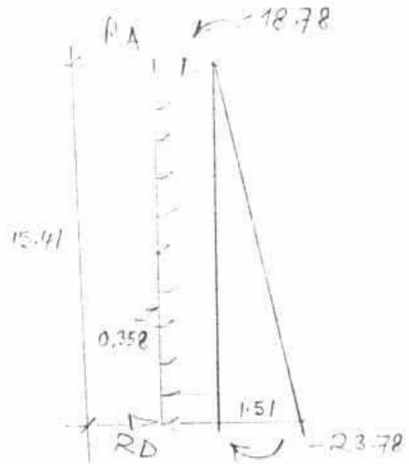
$M_A = -8.78 + 0.59 - 6.93 - 3.66 = -18.78^{k'}$

$M_C = -8.78 - 2.84 - 8.50 - 3.66 = -23.78^{k'}$

Dy8 (3)

SECK- 5-5





$M_E = \omega L^2/8 - (M_A + M_B)/2 = 1.046 \times 14.43^2/8 - 18.78 = 8.45^{k'}$

$M_F = \omega_1 L^2/8 - (M_C + M_D)/2 = 1.314 \times 14.43^2/8 - 23.78 = 10.42^{k'}$

$\Sigma M_D = 0;\ 15.41 RA = (0.358 \times 15.41^2)/2 + 1/2 \times 1.51 \times 15.41^2 \times 1/3 + 18.78 - 23.78 = 0$

$RA = 6.31$

Considering zero shere at x from RA, $\therefore 6.31 - 0.358x - 1.51x^2/2 \times 15.41 = 0$

or $0.049x^2 + 0.358x - 6.31 = 0 \Rightarrow x = \dfrac{-0.358 \pm \sqrt{(0.358)^2 + 4 \times 6.31 \times 0.049}}{2 \times 0.049}$

$x = 8.27$

$+M_{8.27} = 6.31 \times 8.27 - (0.358 \times 8.27^2)/2 - 18.78 - 1/2 \times (8.27^3 \times 1.51)/3 \times 15.41 = 11.92$
(G)

d, checking $d = \sqrt{M/Rb} = \sqrt{23.78/0.187} = 11.21$

Let $t = 16''$ $\qquad d = 13''$

Steel Calculation

$A_s A = 18.78/1.75 \times 13 = 0.826 \square''$; $20\phi - 175$

$A_s E = 8.45/1.75 \times 13 = 0.371 \square''$; $20\phi - 185$

$A_s C = 23.78/1.75 \times 13 = 1.045 \square''$; $20\phi - 135$

$A_s F = 10.42/1.75 \times 13 = 0.458 \square''$; $20\phi - 150$

$A_s G = 11.92/1.75 \times 13 = 0.524 \square''$; $16\phi - 175$

DESIGN OF IBRAHIMPUR KHAL (BOX CULVERT)

Area A =1.66 + 0.5

=2.16 sq km

Runoff factor C =0.5

Rainfall intensity (I)

a) Upland flow = 3996 ft

Considering 1 ft/sec upland flow. T=3996/1 = 3996 sec = 66.6 min

b) Time for start of flow = 10 min

c) Khal length = 1020 m

Considering 1 m/s velocity T = 1020 sec = 17 min

Therefore Tc (Time of concentration) =66.6 + 10 + 17 =93.6 min

I from the graph = 60

Runoff (Q) =CIA

$$= \frac{.50 \times 60 \times 2.16 \times 10^{3}}{3600}$$

= 18 m3/sec

Area A = 18 m2 ,

Size of culvert.

Assumed, H = 3.9 m (With free board), W = 5 m
Considering free board 300 mm

IBRAHIMPUR (PIPE DRAIN)

Area A =68400 m2

Runoff factor C =0.6

Rainfall intensity (I) Five years max. intensity is considered.

a) Upland flow = 600 ft

Considering 1 ft/sec upland flow. T=600/1 = 600 sec = 10 min

b) Time for start of flow = 10 min

c) Khal length = 1190 m

Considering 1 m/s velocity 1190 m/1 = 1190 sec = 19.8 min.

therefore Tc (Time of concentration) =10 + 10 + 20 = 40 min.

I from the graph = 100 mm/hr.

Runoff (Q) =CIA

$$= \frac{.60 \times 100 \times 68400}{1000 \times 3600} = 0.95 \text{ m3 /sec}$$

$A = 0.95 \text{ m}^2$, Considering velocity 1 m/s

Size of sewer.

$$\text{Diameter } D = \sqrt{\frac{0.95 \times 4}{3.14}}$$

$= 1.21$ m

Say 1210 mm dia

DWASA PROVIDED 1220 MM DIA. OK.

1

DILKUSHA & MOTIJHEEL COMMERCIAL AREA

Area A =1 00 800 m2

Runoff factor C =0.6

Rainfall intensity (I) Five years max. intensity is considered.

a) Upland flow = 600 ft

Considering 1 ft/sec upland flow. T=600/1 = 600 sec = 10 min

b) Time for start of flow = 5 min

c) Khal length = 771 m

Considering 1.2 m/s velocity 771 m/1.2 = 642.5 sec

= 10.70 min.

therefore Tc (Time of concentration) =10 + 5 + 10.70 = 25.70

I from the graph = 125 mm/hr.

Runoff (Q) =CIA

$$= \frac{.60 \times 125 \times 1,00,800}{1000 \times 3600} = 2.1 \text{ m3 /sec}$$

$$Q = AV \text{ or } A = \frac{Q}{V} = \frac{2.1}{1.2}$$

Area A = 1.375 m2 .

Size of brick sewer.

$$\text{Diameter } D = \sqrt{\frac{1.375 \times 4}{3.14}}$$

$$= 1.36 \text{ m}$$

1375 mm dia provided so its OK.

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References

Hydraulic design

1. Catchment area has been determined from "Updating study on storm water drainage system improvement project in Dhaka City", by JICA 1990 and/or from topographic survey.

2. Runoff coefficient has been taken from JICA report.

3. Rainfall intensity has been taken from JICA report Pg. 4-20 Fig. 41.

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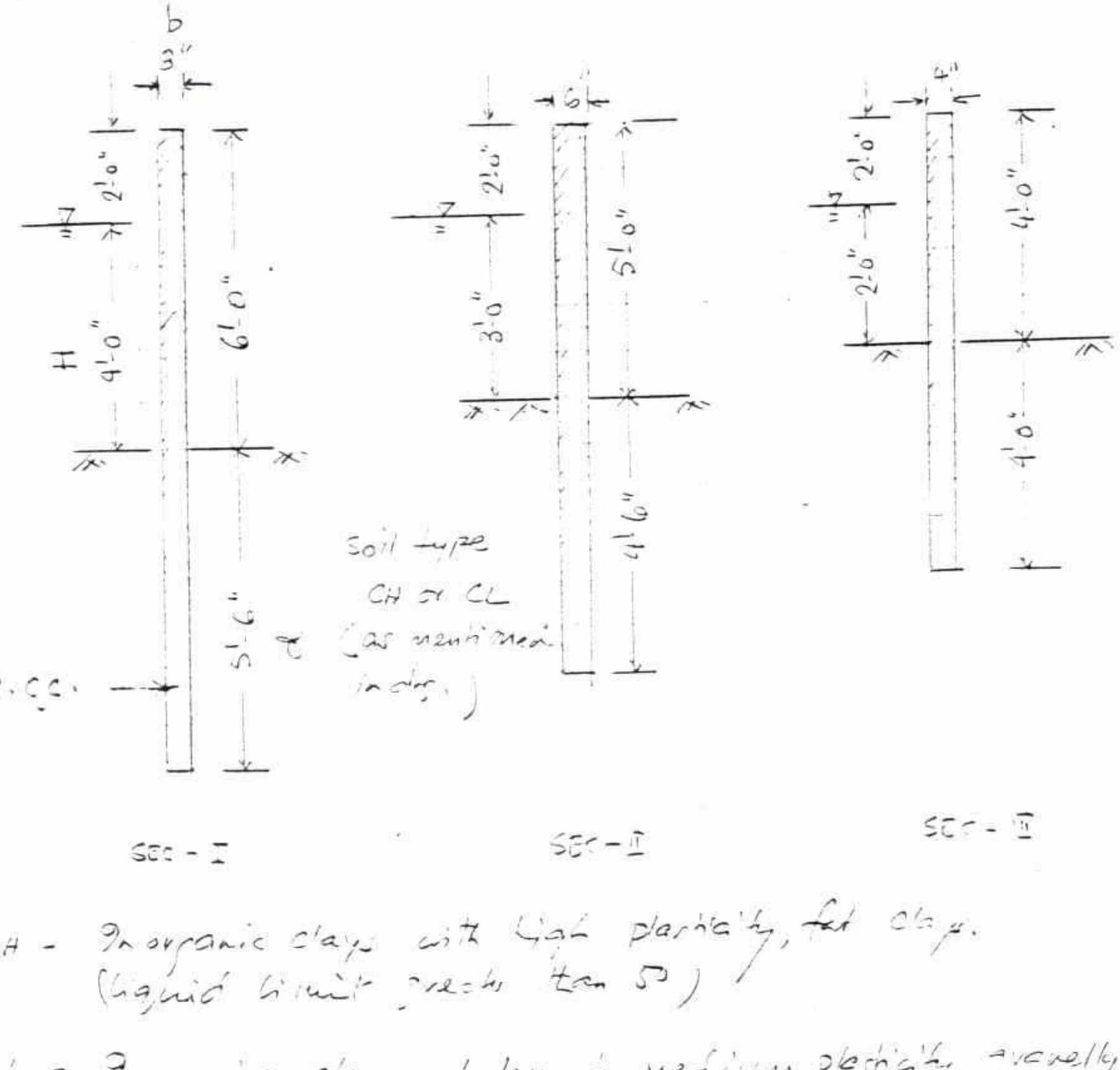
DESIGN REPORT ON
REMEDIAL MEASURES OF FLOOD WALL FROM FRIENDSHIP BRIDGE TO MITFORD HOSPITAL AND FROM DIABARI TO MIRPUR BARO BAZAR

D90

CONTENTS :

1. REVIEW OF FLOOD WALL DESIGN: -
(Ref. Drg. No F/01 or SD/DHA $\frac{40}{89}$ G of BRTC, BUET, DHAKA)



SEC - I SEC - II SEC - III

CH - Inorganic clays with high plasticity, fat clays. (liquid limit greater than 50)

CL - Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. (liquid limit less than 50)

Checking for exit gradient:

SEC- I $H = 4'0"$, $d = 5.5'$, $b = 0.66'$, $\alpha = \frac{b}{d} = \frac{.66}{5.5} = 0.12$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+.12^2}}{2} = 1.003$$

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$$= \frac{4}{5.5} \cdot \frac{1}{\pi\sqrt{1.003}} = 0.231 \quad \text{i.e. } \underline{\underline{1:4.3}} \text{ ok.}$$

0?2

SEC II $H = 3'\text{-}0''$ $d = 4.5'$, $b = 0.5'$ $\alpha = \frac{b}{d} = \frac{0.5}{4.5} = 0.11$

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} \qquad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 1.00$$

$$= \frac{3}{4.5} \cdot \frac{1}{\pi\sqrt{1.0}}$$

$= 0.212$ i.e. $1:4.7$ ok.

SEC III $H = 2'\text{-}0''$ $d = 4.0'$ $b = 0.33'$ $\alpha = \frac{b}{d} = \frac{0.33}{4} = .083$

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} \qquad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 1.0$$

$$= \frac{2}{4.0} \cdot \frac{1}{\pi\sqrt{1.0}}$$

$= 0.159$ ie: $1:6.3$ ok.

Again according to Lane's weighted creep theory, safe hydraulic gradient for clayey soils should not be less than

$\frac{1}{3}$ to $\frac{1}{1.6}$

SEC - I Weighted creep length

$$L_f = 5.5 \times 2 + \frac{.66}{3} = 11.22$$

$$\frac{L_f}{H_L} = \frac{11.22}{4.0} = 2.8 \qquad G_E = \frac{1}{2.8} \quad \text{ok.}$$

SEC-II $L_f = 4.5 \times 2 + \frac{.5}{3} = 9.167$

$$\frac{L_f}{H_L} = \frac{9.167}{3} = 3.06 \qquad G_E = \frac{1}{3.06} \quad \text{ok.}$$

SEC III $L_f = 4 \times 2 + \frac{.33}{3} = 8.11$

$$\frac{L_f}{H_L} = \frac{8.11}{2} = 4.06 \qquad G_E = \frac{1}{4.06} \quad \text{ok.}$$

Checking for structural strength & stability:

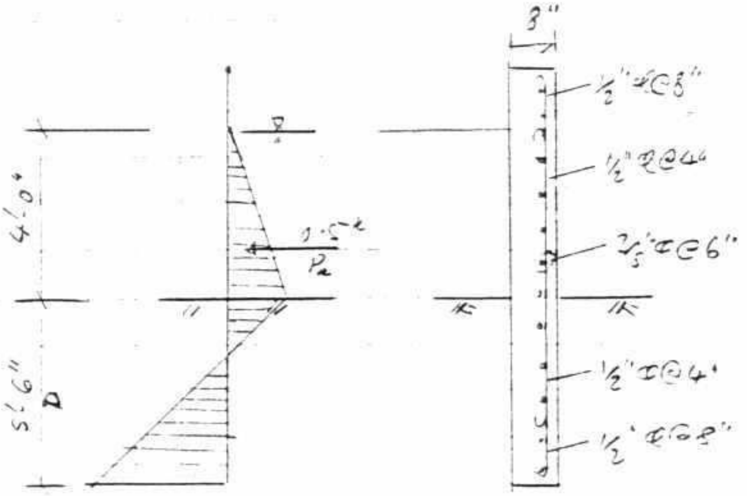
SEC - I

$\gamma_w = 62.5$ pcf

$h = 4'-0''$

$\phi = 30°$

d = effective thickness of wall



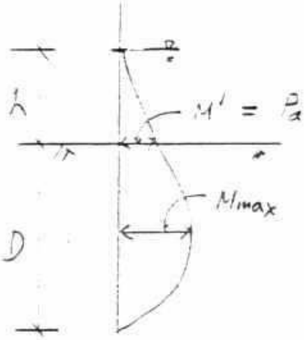
From graph I of fig. 12-10 of Teng.

$D/h = 1.35 \quad \therefore D = 1.35 \times 4 = 5.4' < 5.5'$ ok.

$P_a = 0.0625 \times 4 \times \frac{4}{2} = 0.5^k$

$M' = 0.5 \times \frac{4}{3} = 0.667^{k'}$

From graph II of fig 12-10 of Teng,



$\frac{M_{max}}{M'} = 2.25$

$M_{max} = 2.25 \times 0.667 = 1.50^{k'}$

$d = \left(\frac{1.50}{.189}\right)^{.5} = 2.81'' < 5''$ ok.

$A_s = \frac{1.50}{1.31 \times 5} = 0.23 in^2$ ½"Φ@ 10"c/c. < ½"Φ@8" ok.

SEC – II

$D/_h = 1.35 \quad D = 1.35 \times 3 = 4.05' < 4.5'$

$P_a = .0625 \times 3 \times \frac{3}{2} = 0.281^k$

$M' = 0.281 \times \frac{3}{3} = 0.281^{k'}$

$\frac{M_{max}}{M'} = 2.25$

$M_{max} = 2.25 \times 0.281 = 0.63^{k'}$

$d = \left(\frac{0.63}{.189}\right)^{.5} = 1.82'' < 3''$

$A_s = \frac{0.63}{1.31 \times 3} = 0.16\ in^2$ ½" Φ @ 12" c/c ok.

provided ½" Φ @ 8" c/c.

SEC – III

$D/_h = 1.35. \quad D = 1.35 \times 2 = 2.7' < 4'$

$P_a = 0.0625 \times 2 \times \frac{2}{2} = 0.125^k$

$M' = 0.125 \times \frac{2}{3} = 0.083^{k'}$

$\frac{M_{max}}{M'} = 2.25$

$M_{max} = 2.25 \times 0.083 = 0.187^{k'}$

$d = \left(\frac{0.187}{.189}\right)^{.5} = 0.99'' < 2''$

$A_s = \frac{.187}{1.31 \times 2} = .071\ in^2$ ⅜" Φ @ 15" c/c

Provided ½" Φ @ 9" c/c ok.

2. Evaluation of existing flood wall structures based on actual findings in the field :

Data and information obtained from field investigations through random sampling shows variations and much inconsistency with the design and drawing of the flood walls.

i) Foundation soil:

Out of the locations investigated one third of the locations shows sandy silt and the rest two third areas show clay & silt

ii) Height / Depth of wall above & below G.L.:

Out of the locations investigated flood walls at 66% locations have higher depth than shown in drawings and at 33% locations depth is shorter.

iii) Wall Thickness is found as per drawing.

iv) Clear cover of m.s. bars have been found varying from 2" to 3½".

v) Compressive strength of concrete as obtained from impact hammer test at 12 locations varied from 1560 psi to 3373 psi with an av. strength of 1919 psi.

vi) Main reinforcement found as per drawings

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Considering the above field data, the flood walls have been analysed on the following criteria:

i) Foundation and back filled soil — Sandy Silt.

ii) Depth of wall — 10% less than shown in drawing.

iii) Wall thickness — Same as shown in drawing.

iv) Clear cover of reinf. -- 3"

v) Compressive strength of concrete f'_c = 1900 psi & 1500 psi

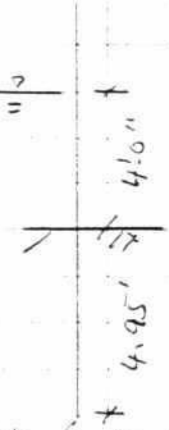
vi) Main reinforcement placement — as per drawing.

vii) f_s = 18000 psi

viii) Max. water level = 2' below top of wall. Hydrostatic pressure only, No impact or wave thrust or higher water level considered.

SEC I

$H = 4'\text{-}0''$; $d = 4.95'$, $b = 0.66'$



$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} \quad \lambda \approx 1$$

$$= \frac{H}{d} \cdot \frac{1}{\pi}$$

$$= \frac{4}{4.95} \cdot \frac{1}{\pi} = 0.257$$

ie: 1:4 Not safe for sandy silt

Safe G_E for sandy silt is $\frac{1}{6}$ to $\frac{1}{7}$ (0.17 to 0.14)

Saf G_E for clayey silt is $\frac{1}{3}$ to $\frac{1}{1.6}$ (.33 to 0.625)

From graph I of fig. 12-10 - "Teng"

$D/h = 1.25 \quad \therefore D = 1.25 \times 4 = 5.4' > 4.95'$ N.G.

$P_a = 0.5^k$

$M' = 0.5 \times \frac{4}{3} = 0.667^{k'}$

From graph II of fig 12-10 - Teng.

$$\frac{M_{max}}{M_1} = 2.25$$

$M_{max} = 2.25 \times .667 = 1.5^{k'}$

$R = \frac{1}{2} f_c k j$

$= \frac{1}{2} \times .45 \times 1.90 \times .34 \times .875$

$= 0.127$

$$d = \left(\frac{1.50}{0.127}\right)^{.5} = 3.43'' < 5''$$

$$A_s = \frac{1.5}{1.31 \times 5} = 0.23\ \text{in}^2\ \text{oh.}$$

$\frac{1}{2}''\ \phi$ @ 8" c/c.

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$f_r f_c' = 1560$ psi , $\quad R = \frac{1}{2} \times .45 \times 1.53 \times .34 \times .875$
$= 0.104$

$d = \left(\frac{1.5}{.104}\right)^{.5} = 3.8'' < 5''$ ok.

$A_s = \frac{1.5}{1.31 \times 5} = 0.23$ in^2 ok.

SEC – II

$D/h = 1.35 \quad D = 1.35 \times 3 = 4.05 \approx 4.05$ ok. $\left(\begin{array}{l} .9 \times 4.5 \\ = 4.05 \end{array}\right)$

$P_a = 0.281^k$

$M' = 0.281 \times \frac{3}{3} = 0.281^{k'}$

$M_{max} = 2.25 \times 0.281 = 0.63^{k'}$

$d = \left(\frac{0.63}{.104}\right)^{.5} = 2.46'' < 3''$ ok.

3. DESIGN CRITERIA FOR RAMPS, HALF HT. & FULL HT. STAIRS ACROSS FLOOD WALL OPENINGS

RAMPS:

Ramps have been provided on flood wall openings for movement of vehicular traffic where sufficient space is available on both R/S and C/S to accomodate the slope length of ramp.

Structure of the ramp have been designed to carry H10 loading with 125mm thick R.C.C. pavement over 125mm brick on edge + herring bond brick soling and 75mm brick flat soling on compacted sand. 250mm high curb have been provided on each side of the ramp. 375mm to 500mm thick brick wall have been provided on both longitudinal side of ramp.

HALF HT. & FULL HT. STAIRS:

Half & full ht. stairs have been provided for pedestrain movement across the wall. Height and width of the wall have been kept according to the space available on both R/S and C/S of the wall.

The opening of the flood walls will be closed by extending the walls upto the top level of the stairs. The stairs to be constructed with brick having G.I. pipe railing on the outer side of the steps only.

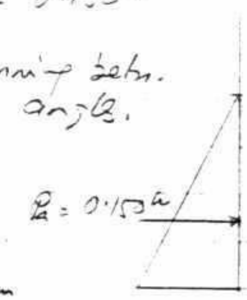
4. Sliding Steel Gates for closure of flood wall openings 10

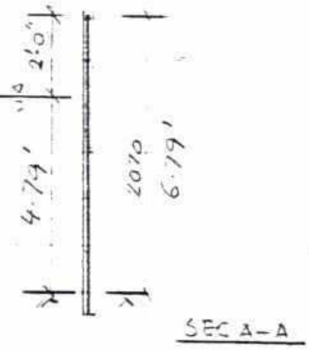
$P_a = 0.0625 \times 4.79 \times \frac{1}{2} = 0.150^k$

B.M. on m.s. plate spanning betn. angle.

$= .150 \times 1.91^2 \times \frac{1}{9}$

$= .061^{k'}$



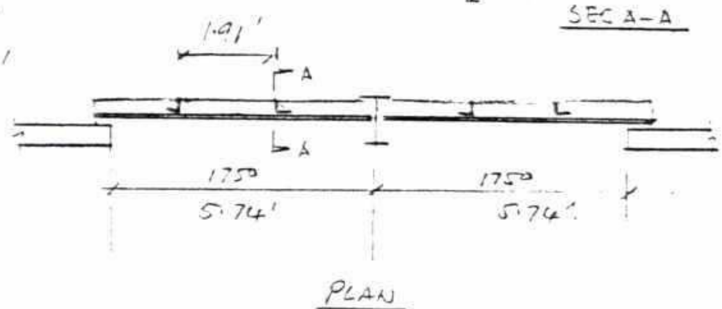


M.s plate thickness = 3mm

$= 0.118''$

$= .0098'$

$I = \frac{.0098^3}{12} \times \frac{4.79}{2} = 1.88^{-7}$

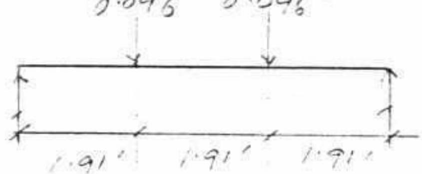


$S = \frac{My}{I} = \frac{.061 \times .0098}{2 \times 1.88^{-7}}$

$= 1746$ ksf $= 12.13$ ksi < 18 ksi ok.

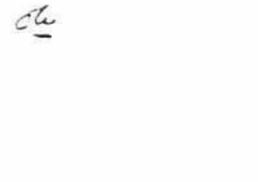
Load on each hor. angle (3 angles)

$= = \frac{1}{3} \times .150 \times 1.91 = .096^k$



B.M $= .096 \times 1.91 = 0.183^{k'}$

2"×2" angle $I_{x-x} = 12.9$ cm^4
(50×50×6 L) $= 0.31$ in^4 $= 1.49^{-5}$ ft^4



$S = \frac{My}{I} = \frac{0.183 \times 0.149}{1.49^{-5}}$ $y = 4.55$ cm $= 0.149'$

$= 1830$ ksf $= 12.71$ ksi < 18 ksi ok

Reaction on flood wall

$= 0.15 \times \frac{5.74}{2} = 0.431^k$

$M = 0.431 \times \frac{4.79}{3} = 0.69^{k'}$

$d = \left(\frac{0.69}{.104}\right)^{.5} = 2.57''$ $A_s = \frac{0.69}{1.31 \times 3} = 0.176$ in^2 Flood wall ½" ø @ 6" ok. Section ok.

5. Design for reconstruction of damaged flood wall   11

Damaged locations: - CH 0+974 to CH 0+990 m (from Friendship Bridge) at Archin Gate.
Existing Ht. of wall above G.L. - 5'

Exist gradient:

$H = 3'\text{-}0''$, $b = 0.66'$, $d = 8'$ (assumed)

$\alpha = \frac{b}{d} = \frac{0.66}{8} = .0825$

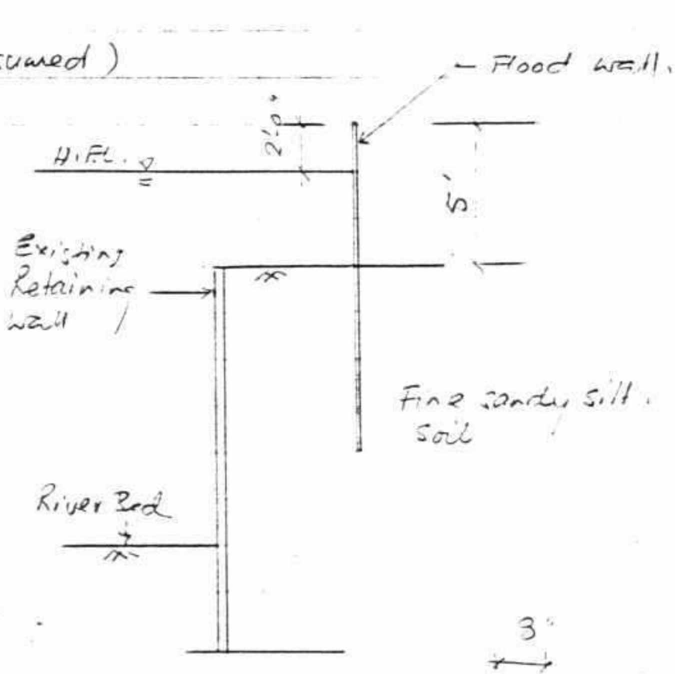
$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 1.0$

$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

$= \frac{3}{8} \cdot \frac{1}{\pi\sqrt{1.0}}$

$= 0.119$ i.e. $1/8.4$ ok.

Safe G_E for sandy silt is $1/7$.



Structural design:

$\gamma_w = 62.5$ pcf, $\phi = 30°$, $h = 3'\text{-}0''$

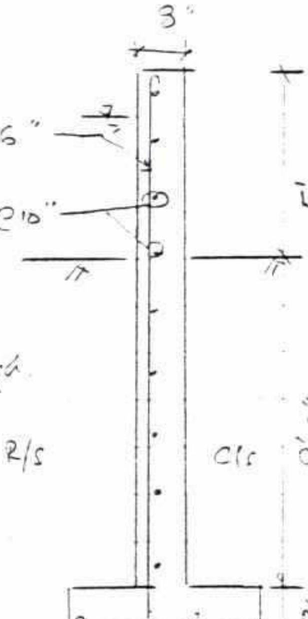
From graph I of fig 12-10 of "Teng"

$D/h = 1.35$, ∴ $D = 1.35 \times 3 = 4.05' < 8'$ ok.

$P_a = .0625 \times 3 \times \frac{3}{2} = 0.281^k$

$M' = 0.281 \times \frac{3}{3} = 0.281^{k'}$

from graph II of fig 12-10 of "Teng"



$\frac{M_{max}}{M'} = 2.25$. $M_{max} = 2.25 \times 0.281 = 0.63^{k'}$

$d = \left(\frac{0.63}{.189}\right)^{.5} = 1.82'' < 5''$ ok.

$A_s = \frac{0.63}{1.31 \times 5} = 0.1 6 in^2$ ½" φ @ 6" c/c.

6. Conclusion :

On field survey and investigation of the flood wall, it is concluded that:

i) Drawings and specification were not properly followed in construction works of the flood walls in most of the locations investigated.

ii) From "exit gradient" consideration, the flood walls on sandy silt is not safe against piping. Flood wall on clayey soil is, however, safe against piping.

iii) Structurally, the wall is marginally safe against normal water load, but might be vulnerable to failure under extreme loading conditions.

7. Recommendations:

i) Further detail investigations should be carried out for soil condition, depth of wall and concrete strength for ascertaining the locations and type of remedial measures to be undertaken.

ii) The openings in the flood walls are to be closed by gate/stairs/ramps as stated in the Schedule of proposed measures for protection against flood.

iii) The openings above the half height stairs are to be closed by sand filled gunny bags when necessary.

iv) Where site condition does not permit closing of openings by gate/stair/ramps, the openings are to be closed by sand-filled gunny bags when necessary.

v) Damaged flood walls are to be reconstructed as per revised design. This design is applicable to locations with similar site and subsoil condition.

Note on 7. v): The total length of flood wall under study is [illegible]. Current investigations show wide range of variations in concrete strength & type of sub soil along the whole length of floodwall. The floodwall is constructed with expansion joints at intervals of 12m. So, to determine the appro-

-priate type of design for each segment of wall, concrete hammer tests and sub-soil borings are to be done at least at 400 points which is beyond the scope of work of this mid term consultancy services.

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DESIGN REPORT ON

SLOPE PROTECTION WORK OF WESTERN EMBANKMENT AGAINST WAVE ACTION

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DIFPP

DESIGN OF SLOPE PROTECTION WORK OF WESTERN EMBANKMENT DUE TO WAVE ACTION :

DESIGN DATA :

Crest level of Embankment = +9.80m (PWD)

H.F.L. = 8.00m ,,

SIDE SLOPE OF EMBKT = 1:3

Toe level on R/S section = +2.00 (PWD)

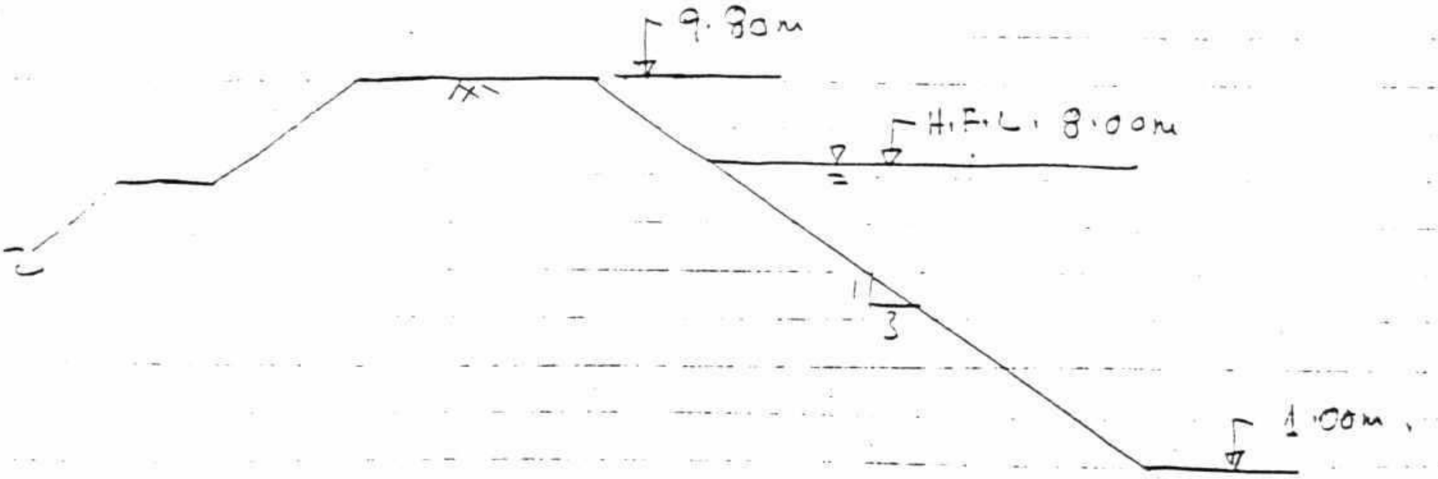
Fetch length, F, a) Towards west = 2.5KM

b) Towards south = 5.0KM.

Average depth of water, d = 8 - 2 = 6.00m.

Type of soil - a) Fine sandy SILT

b) Clayey SILT



Determination of wind speed:

Last 10 years data during monsoon (May to Sept) in Dhaka city.

i) Highest daily av. wind speed,

ii) Monthly max. wind speed;

iii) Monthly av. wind speed and direction.

From the above 3 tables, 10 yr. highest daily av. and inst. wind speed & direction is obtained.

From the above data the following estimation for design purpose may be considered:

1) 24-hr duration max. wind speed = 19 knots/hr

2) 12 hr " " " " = 20 knots/hr

wind direction from south or South East.

U_{24} = Wind Speed = $\frac{19 \times 6080}{60 \times 60}$ = 32.08 ft/sec.

U_{12} = " " = $\frac{20 \times 6080}{60 \times 60}$ = 33.71

≈ 34 ft/sec. (Design speed)

Wave forecasting factors:

$gd/u^2 = \frac{32.2 \times 6 \times 3.28}{34^2} = 0.548$

$gF/u^2 = \frac{32.2 \times 5000 \times 3.28}{34^2} = 456.87 = 4.57 \times 10^2$

[illegible]

Using values of eq. [illegible] ([illegible])

$$\frac{gH}{u^2} = 7\times10^{-2} \quad \ldots \quad \text{(I)} \quad [illegible]$$

$$\frac{Lg}{2\pi u^2} = 0.110 \quad \ldots \quad \text{(II)} \quad [illegible]$$

From eq (I) $\quad H = \dfrac{u^2\times 7\times10^{-2}}{g}$

$$= \frac{32^2\times 7\times10^{-2}}{32.2} = 2.2[illegible] \text{ ft.}$$

From eq (II) $\quad L = \dfrac{0.110\times 2\times\pi\times u^2}{g}$

$$= \frac{0.110\times2\times3.14\times32^2}{32.2}$$

$$= 2[illegible] \text{ ft}$$

Time for generated a wave

$$T = \frac{2\pi L}{g \tanh \frac{[illegible]}{[illegible]}}$$

$$= \frac{2\pi\times 2[illegible].80}{32.2 \tanh \left[\frac{2\times\pi\times [illegible].80}{2[illegible].80}\right]}$$

$$= \frac{[illegible]\times 2[illegible].80}{32.2 \tanh [illegible]}$$

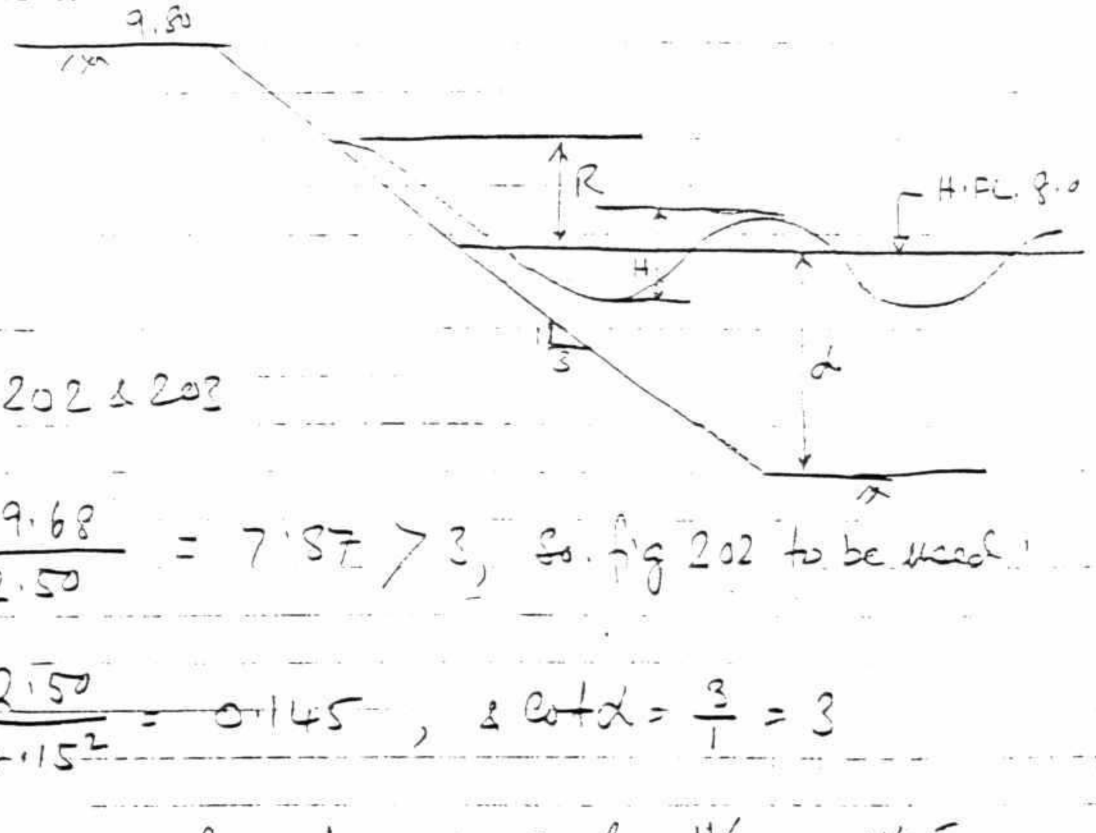
4A5

$$= \frac{155.74}{32.2 \tanh 285.68^\circ} = \frac{155.74}{32.2 \tan(285.68-270)}$$

$$= \frac{155.74}{32.2 \times \tan 15.68^\circ} = \frac{155.74}{32.2 \times .2807}$$

$$= 17.23 \text{ sec}^2 \quad \therefore T = 4.15 \text{ sec.}$$

WAVE RUN-UP :



from figures 202 & 203

$$\frac{d}{H} = \frac{19.68}{2.50} = 7.87 > 3, \text{ so fig 202 to be used.}$$

$$\frac{H}{T^2} = \frac{2.50}{4.15^2} = 0.145, \quad \& \cot\alpha = \frac{3}{1} = 3$$

From fig. 202, for $\cot\alpha = 3$ and $H/T^2 = .145$,

the corresponding value of $R/H = 1.9$

$$\therefore R = 1.9 \times 2.50' = 4.75'$$

$$= 1.45 \text{ m.}$$

5 of 5

Height of embkt should be (for covering 2)
$8.00 + 1.45 = 9.45m < 9.8m$ ok.

DESIGN OF ARMOUR STONES

The wt. of stone reqd. for dissipating wave may be determined by Hudson's formula:

$$W = \frac{W_r H^3}{K_d (S_r - 1)^3 \cot\alpha}$$

W_r = Unit wt. of stone lb/cft

H = Wave ht.

S_r = Sp. gr. of stone

$S_r = \frac{W_r}{W_w}$ (W_w = unit wt of water = 62.4 lb/cf)

α = angle of slope measured from horizontal degree

K_d = Roughness coefficient of surface from table 5 pg. 243 = 2.9

Considering C.C. block (primary ...)

$W_r = 125$ lb/cf (ref Reynold's R.C.C. Handbook Pg 147 table 2)

$\& S_r = \frac{125}{62.4} = 2.003$

$\cot\alpha = 3$

$$\therefore W = \frac{125 \times 2.50^3}{2.9 \times (2.003 - 1)^3 \times 3}$$

$$= \frac{125 \times 15.625}{2.9 \times 1.009 \times 3}$$

$= 222.49$ lbs ≈ 225 lb.

$= \frac{225}{125} = 1.80$ cft

$= 1.34' \times 1.34' \times 1'$ block

DESIGN REPORT ON

SLOPE AND TOE PROTECTION WORK ON WESTERN EMBANKMENT AGAINST EROSION AND SCOUR DUE TO RIVER CURRENT

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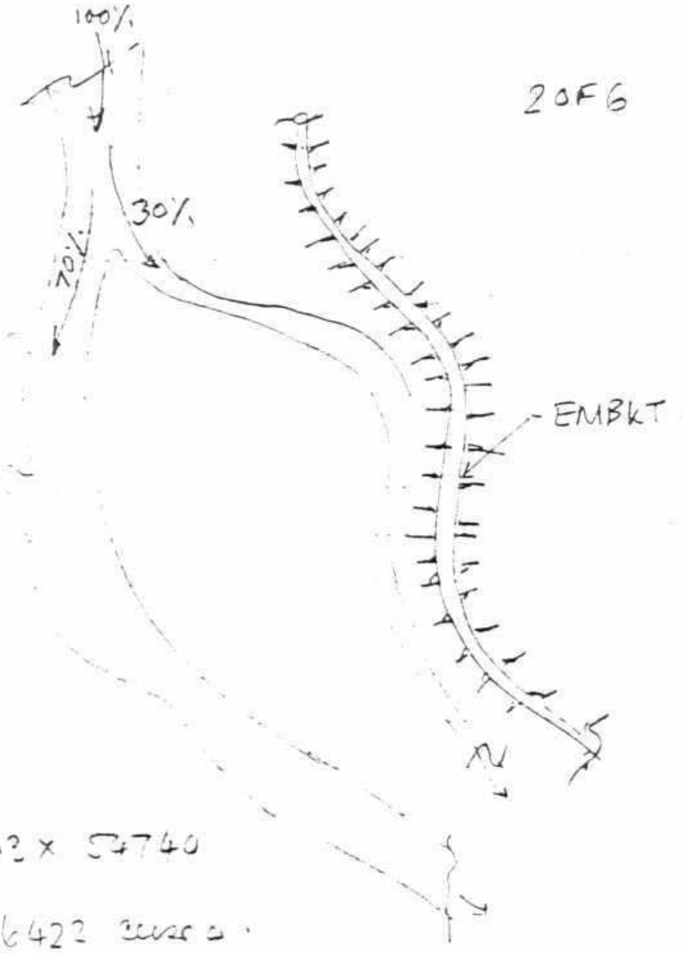
DIFPP

DESIGN OF BANK REVETMENT AGAINST EROSION AND SCOUR DUE TO CURRENT

SLOPE & TOE PROTECTION WITH LIGHT BLOCKS BETWEEN CH 28+450 TO CH 29+300 ON WESTERN EMBANKMENT

Available data:

Maxm. Discharge Q_{max}	=	54740 cusec. (1987)
Highest flood level	=	7.70 m + PWD or 25.26 ft
Lowest Water level	=	2.0 m + PWD or 6.56 ft.
Bank level	=	9.80 m + PWD or 32.14 ft
River Bed level	=	1.00 m + PWD or 3.28 ft.
Av. dia. of river bed material, dm	=	0.10 mm
Maxm. velocity	=	1.52 m/sec. or 5 ft/sec.
Maxm. velocity at bridge section	=	2.44 m/sec = 8 m/sec.
River slope	=	4" per mile.



The main channel carries about 70% flow and the channel adjacent & parallel to the embankment have 30% of the total flow.

So,

Max. Discharge $Q_{max} = 0.3 \times 54740$

$= 16422$ cusec.

A. THICKNESS OF PITCHING ON SLOPE:

Dominant Discharge $= 16422 \times .7 = 11495$ cusec.

According to Inglis formula

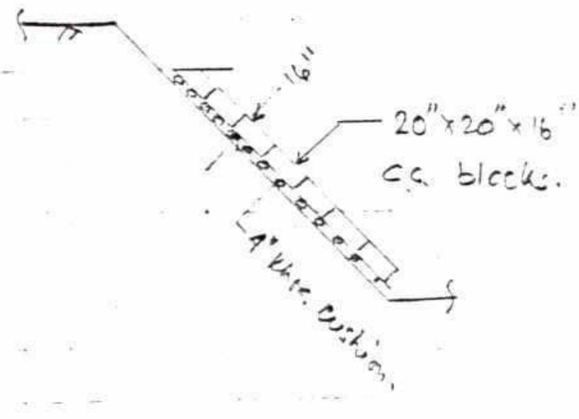
$t = .06\, Q^{1/3}$

$= .06 \times (11495)^{1/3}$

$= 1.35$ ft

$= 16.25''$

$\simeq 16''$



B. DESIGN OF LAUNCHING APRON :

According to Lacey, regim scour depth

$D = 0.47 \left(\frac{Q}{f}\right)^{1/3}$

$= 0.47 \left(\frac{11495}{0.56}\right)^{1/3}$

$= 12.86'$

Lacey's silt factor

$f = 1.76\sqrt{d_m}$

$= 1.76 \times \sqrt{0.1}$

$= 0.56$

i) For moderate to severe bend,

emperical multiplying factor = 1.75

So, Total Scour Depth = $1.75 \times 12.86 = 22.51$

Depth of Scour from existing bed level

$D_s = 22.5 - (25.26 - 3.28) = 0.52'$

Length of Launching apron = $.52 \times 1.5 = \underline{0.78'}$

ii) Max. Scour downstream of a bridge

$D = 1.9 \left(\frac{Q}{f}\right)^{1/3}$

$= 1.9 \left(\frac{11495}{0.56}\right)^{1/3}$

$= 52.02'$, $D_s = 52.02 - 21.98 = 30.04'$

$L = 1.5 \times 30 = \underline{45'}$

iii) Max. scour round bridge pier.

$D = .95 \left(\frac{Q}{f}\right)^{1/3} = .95 \times 27.35 = 25.98'$

$D_s = 25.98 - 21.98 = 4'$ $L = 4 \times 1.5 = \underline{6'}$

For case ii) <u>at down stream of bridge.</u>

L, length of launching apron = 1.5 × 30 = 45'

Av. thickness of apron = 1.25 t

So, total qnty. of hard materials reqd.

$= 1.25 \times 1.33' \times 30 \times \sqrt{1+2^2}$

$= 111.52$ cft.

According to Spring, the thickness at the inner end of apron is of same thickness to that on slope and increasing upto <u>2.76 times</u> at the outer end. So, qnty of hard material reqd.

$= \{1.33 + (1.33 \times 2.76)\} \times \frac{1}{2} \times 45'$

$= 112.52$ cft. ok.

C. SIZE AND GRADATION OF HARD MATERIALS:

Wt. of single unit of stone for a velocity of 5ft/sec. (assumed)

a) As per California Hwy curve - 1.5 lbs.

b) As per Tentative curve for guide to bridge hydraulics. - 3.0 lbs

adopting 3.0 lbs stone, size i.e.

dia of stone comes to =

Sp. Gr. of stone = 2.65

≈ 165 #/cft

∴ 3 lb = ·0182 cft

Unit wt. of C.C. blocks 125 lb/cft (brick bloc)

3 lb = ·024 cft = 0.29 ft cube.

$\underline{\underline{D_{40}}}$. i.e. 3.5" x 3.5" x 3.5" cube

from table-4

Gradation D_{40}

20% larger than 7"

60% larger than 3.5"

80% " " 3"

(very light)

Use next higher size

9" cube 45%

6" " 35%

3" " 20%

Again considering a velocity of 8 ft/sec. at Bridge Section
from curve 5, wt. of single unit of block = 25 lb.
= 0.20 cft
= 0.584 ft cube

from curve (3) wt = 40 lb
= 0.32 cft $\simeq$ 0.684' cube
= 8.21" "

20% larger than 12"
40% larger than 9" cube
80% larger " 6" "

Provide 45% — 12" cube
35% — 9" " } mixed dumping.
25% — 6" cube

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DESIGN REPORT ON

CENTRAL SPINE ROAD RAISING WORKS

DESIGN CRITERIA OF CENTRAL SPINE ROAD RAISING

As recommended in the FSR, the design level of Central Spine Road has been fixed at 50 yr. flood level. 50 yr flood level of Tongi khal at Tongi and Lakhya river at Demra is 8.11 m PWD and 7.09 m PWD respectively and accordingly the design level of the median top/road surface varies from 7.40 m PWD to 7.20 m PWD. The longitudinal profile of the Central Spine Road showing the 50 yr. flood level is attached herewith.

From aesthetic point of view and other practical consideration, the height of median has been kept limited between 25 cm and 50 cm which determines the length and height of road surface raising. The width of the new median has been kept same as that of the existing median, and the bottom of the new median has been kept below the bottom level of macadam with a view to preventing seepage of flood water through the road pavement and for stability of the median. The new median is of RCC which will resist the accidental impact of vehicle without being cracked. The reaches of the road from ch. 6.90 to 7.70, ch. 8.82 to 9.02 and ch. 12.82 to 13.11 are not required to be raised as they are above the 50 year flood. But from practical consideration provision for median in the said reaches has been made.
The openings kept along the median will have to be closed by sand gunny bags during flood.

The criteria followed in designing the road raising works have been fixed in consultation with the concerned Engineers of RAJUK and the Project Director, PMO.