IV.4-2 Observation and Analysis on Discharge of Pat Feeder

1) Objectives of Observation and Analysis

Pat Feeder Canal is large scale unlined earth canal with a proposed discharge of 11,000 cusec (311 cu.m/sec).

Selection of hydraulic calculation formula and decision of hydraulic roughness are most important matter for planning of the widening of the canal.

By using the current meter, observation of discharge of the canal were made during the field survey for the Feasibility Study on the Widening of Pat Feeder Canal analysis on the said observation is carried out during the home office work for the purpose.

2) Method of Observation

The observations were made on velocity by current meter, cross-section survey and hydraulic gradient by the level.

Observation of velocity by current meter of TOHO CM-1B was made on boat fixed with the wire-rope in an interval of 10 feet. Taking into consideration the velocity of about 2 feet per second and a depth of around 7 feet, mean velocity at 0.6 of the depth in each vertical (0.6 H).

Hydraulic gradient is determined by surveying the elevations of water level at the downstream and the upstream 1,500 feet far from the observation site of the velocity.

3) Selection of Observation Site

For the calibration of the canal by observing the hydraulic data explained in the former Para, the portion of the canal with straight line, uniform section and no influence of the back water at RD 433, RD 447 and RD 453 were choosen.

4) Results of Observation

a) Hydraulic Gradient

The hydraulic gradient is calculated by using the following equation;

$$S = \Delta H/L$$

where; S = Hydraulic Gradient

ΔH = Different in Elevation of the Water Level between the two points(feet).

L = Distance between the two points(feet).

The observation are as follows:

Observation Point	Different Elevation ΔH (feet)	Distance L (feet)	Hydraulic Gradient S
RD 433	0.154	3,062.6	0.0000503
RD 447	0.151	3,011.4	0.0000501
RD 453	0.154	3,308.1	0.0000466

b) Depth and Velocity

Observation of depth and velocity are shown in the Tables.

5) Analysis

Generally speaking, Manning formula and Lacey's formula are applied for hydraulic calculation on the open channel.

$$\text{Manning Formula : } V = \frac{1.486}{n} \cdot R^{2/3} \cdot S^{1/2}$$

$$\text{Lacey's Formula : } V = \frac{1.3458}{Na} \cdot R^{3/4} \cdot S^{1/2}$$

$$Na = 0.0225 \cdot f^{1/4}$$

$$f = 1.75\ m^{1/2}$$

Applying the two formulas, hydraulic roughness "n" of the Manning and silt factor "f" of the Lacey are comparatively studied.

The results of the analysis show that the hydraulic roughness "n" ranges at 0.0201 to 0.0209 and the silt factor "f" is considered at 0.72 to 0.86 as shown the details in the Tables.

These numbers are just observation and analysis by a few points to review and check the said hydraulic data on the open channel. The observation is not enough for the detailed academic discussions, however, it can be used as the countercheck of the hydraulic study.

Table IV.2-3 Value of Water Depth & Velocity on Measurement (1)

Survey point; RD 433
Survey time; Mar.13,1982 10:20-10:40
Current mater; Tohondentan CM-1B.No.8701
Surveyor; S.Konishi T.Inoue

No.	Interval (meters)	Water Depth (meters)	Velocity of 0.6H (m/sec)	Area of unit Section (sq.meters)	Discharge of unit Sec. (cum/sec)	Wetted Perimeter (meters)
0		0.00				
1	1.524(5feet)	0.80	0.28	2.25	0.63	1.72
2	"	1.35				1.62
3	"	1.65	0.48	4.84	2.32	1.55
4	"	1.70				1.52
5	"	1.80	0.49	5.37	2.63	1.53
6	"	1.75				1.52
7	"	1.80	0.50	5.45	2.73	1.52
8	"	1.80				1.52
9	"	1.85	0.53	5.56	2.95	1.52
10	"	1.80				1.52
11	"	1.85	0.47	5.60	2.63	1.52
12	"	1.85				1.52
13	"	1.85	0.45	5.64	2.54	1.52
14	"	1.85				1.52
15	"	1.85	0.52	5.60	2.91	1.52
16	"	1.80				1.52
17	"	1.70	0.49	5.07	2.48	1.53
18	"	1.45				1.54
19	"	0.95	0.28	2.55	0.71	1.60
20	"	0.00				1.80
Total	30.48			47.93	22.53	31.13

Table IV.2-4 Value of Water Depth & Velocity of Measurement (2)

Survey point; RD 447
Survey time; Mar.13,1982 11:35-11:50
Current meter; Tohodentan CM-1B No.8701
Surveyor; S.konishi T.Inoue

No.	Interval (meters)	Water Depth (meters)	Velocity of 0.6H (m/sec)	Area of Unit Section (sq.meters)	Discharge of Unit Sec. (cu.m/sec)	Wetted Perimeter (meters)
0		0.00				
1	1.524(5feet)	0.85	0.28	2.36	0.66	1.75
2	"	1.40				1.62
3	"	1.65	0.48	4.99	2.40	1.54
4	"	1.85				1.54
5	"	1.90	0.50	5.68	2.84	1.52
6	"	1.80				1.53
7	"	1.85	0.48	5.60	2.69	1.52
8	"	1.85				1.52
9	"	1.85	0.47	5.68	2.67	1.52
10	"	1.90				1.52
11	"	1.85	0.48	5.72	2.75	1.52
12	"	1.90				1.52
13	"	1.85	0.48	5.64	2.71	1.52
14	"	1.80				1.52
15	"	1.75	0.47	5.45	2.56	1.52
16	"	1.85				1.53
17	"	1.70	0.42	5.03	2.11	1.52
18	"	1.35				1.56
19	"	1.00	0.32	2.55	0.82	1.56
20	"	0.00				1.82
Total	30.48			48.70	22.21	31.17

Table IV.2-5 Value of Water Depth & Velocity of Measurement (3)

Survey point; RD 453
Survey time; Mar.13,1982 12:20-12:40
Current meter; Tohodentan CM-1B No.8701
Surveyor S.Konishi T.Inoue

No.	Interval (meters)	Water Depth (meters)	Velocity of 0.6H (m/sec)	Area of Unit Section (sq.meters)	Discharge of Unit Sec. (cu.m/sec)	Wetted Perimeter (meters)
0		0.00				-
1	1.524(5feet)	0.65	0.20	1.83	0.37	1.65
2	"	1.10				1.59
3	"	1.50	0.45	4.34	1.95	1.58
4	"	1.60				1.53
5	"	1.75	0.44	5.22	2.30	1.53
6	"	1.75				1.52
7	"	1.75	0.42	5.37	2.26	1.52
8	"	1.80				1.52
9	"	1.80	0.43	5.41	2.33	1.52
10	"	1.70				1.53
11	"	1.70	0.42	5.22	2.19	1.52
12	"	1.75				1.52
13	"	1.75	0.51	5.30	2.70	1.52
14	"	1.70				1.52
15	"	1.70	0.51	5.11	2.61	1.52
16	"	1.60				1.53
17	"	1.60	0.49	4.88	2.39	1.52
18	"	1.60				1.52
19	"	1.55	0.38	4.57	1.74	1.52
20	"	1.30				1.54
21	"	1.10	0.38	2.67	1.01	1.54
22	"	0.00				1.88
Total	33.53			49.92	21.85	34.14

Table IV.2-6 Analysis Table of Roughness Coefficient "n" & Silt Factor "f" on Pat Feeder Canal

Description			RD 433	RD 447	RD 453
Discharge	Q	(cusecs)	796.11	784.81	772.08
		(cusecs)	(22.53)	(22.21)	(21.85)
Area of section	A	(sq.feet)	515.93	524.22	537.35
		(sq.meter)	(47.93)	(48.70)	(49.92)
Wetted perimeter	P	(feet)	102.13	102.26	112.01
		(meter)	(31.13)	(31.17)	(34.14)
Hydraulic mean depth	R	(feet)	5.052	5.126	4.797
		(meter)	(1.504)	(1.652)	(1.462)
Hydraulic gradient	S		0.0000503	0.0000501	0.0000466
Velocity	V	f.p.s.	1.543	1.497	1.437
		m.p.s.	(0.4701)	(0.4561)	(0.4377)
Roughness coefficient	n		0.0201	0.00209	0.0201
Silt factor	f		0.74	0.86	0.72

IV.4-3 Hydraulic Design of Canal Structure

Basin hydraulic design of canal structure are made based on the following hydraulic equations :

Hydraulic calculation of discharge is applied equations specified in " Design of Small Dams " published by the USBR.

1) Discharge over Ogee Crest

$$Q = C\ L\ He^{3/2}$$

where, Q : Discharge (cusec)
C : Variable coefficient of discharge
L : Effective length of crest
He : Total head on the crest, including velocity of approach head, ha

2) Pier and Abutment Effect

$$L = L' - 2(N.Kp + Ka)He$$

where, L : Effective length of crest
L' : Net length of crest
N : Number of Pier
Kp : Pier construction coefficient
= 0.01 for round - nosed Piers
Ka : Abutment construction coefficient
= 0.20 for square abutment with headwall at 90° to direction flow
He : Total head on crest

3) Discharge Coefficient

a) Discharge coefficients for vertical-faced ogee crest can be read on the Fig.IV.4-1.

b) Ratio of discharge coefficients due to tailwater effect can be read on the Fig.IV.4-2.

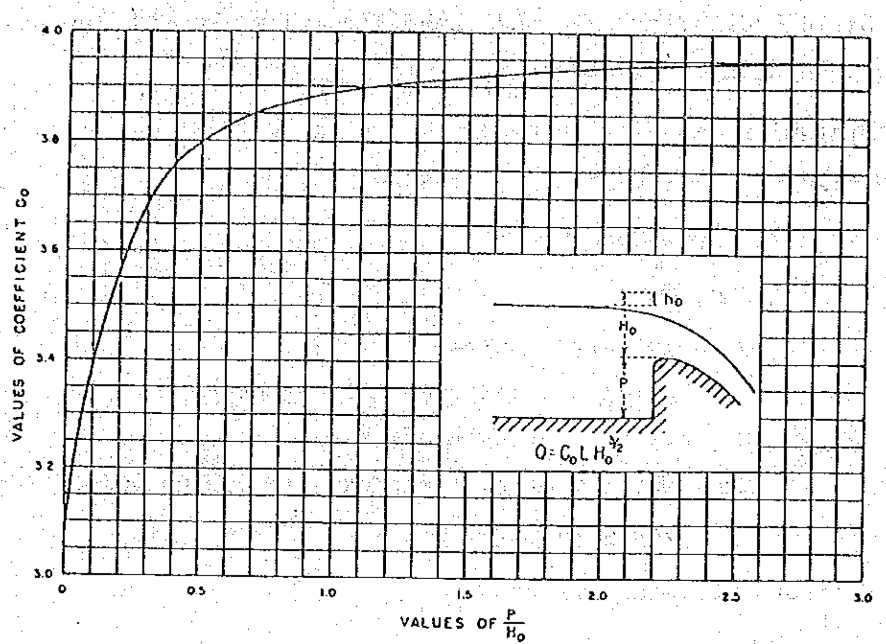


Fig.IV.4-1 Discharge Coefficients for Vertical-Faced Ogee Crest

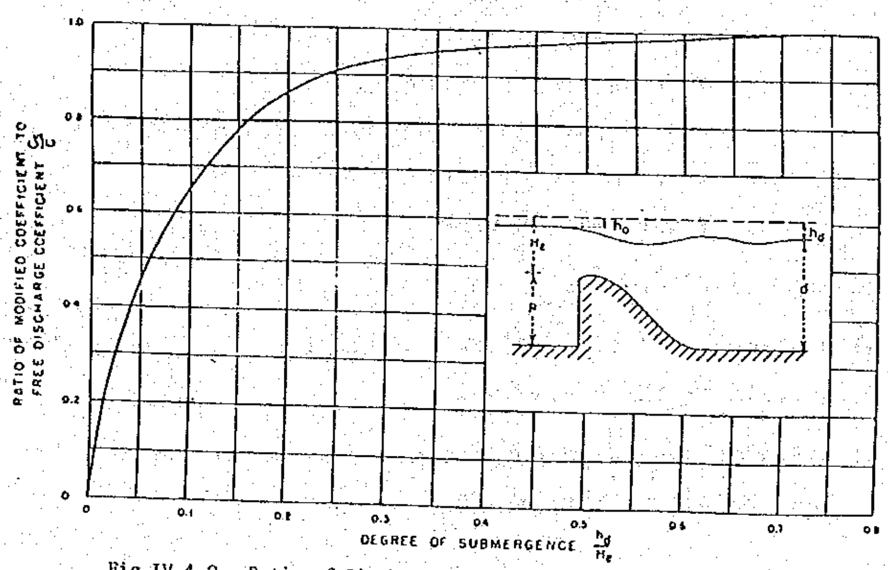


Fig.IV.4-2 Ratio of Discharge Coefficients due to Tailwater Effect

Table IV.4-7 Hydraulic Caluculation of Distributary - Type "A"

Type of Canal	Discharge Q(cusecs)	Bed Width W(feet)	Water Depth D(feet)	Area of Section (sq.feet)	Wetted Perimeter (feet)	Hydraulic Gradient S	Mean Velocity V(feet/sec)	Silt Critical Velocity Vo(feet/sec)	V/Vo
A - 1	5 ~ 16	5.0	2.0	14.00	10.66	1/4,500	1.16	1.05	1.10
- 2	~ 25	8.0	-do-	20.00	13.66	-do-	1.25	-do-	1.27
- 3	~ 52	17.0	-do-	38.00	22.66	-do-	1.39	-do-	1.32
- 4	~ 80	20.0	2.4	53.76	26.79	1/5,000	1.50	1.18	1.27
- 5	~ 105	26.0	-do-	68.16	32.79	-do-	1.54	-do-	1.31
- 6	~ 130	26.0	2.8	80.64	33.92	1/5,500	1.63	1.30	1.25
- 7	~ 152	30.0	-do-	91.84	37.92	-do-	1.66	-do-	1.27
- 8	~ 180	29.0	3.2	103.04	38.05	1/6,000	1.72	1.41	1.22
- 9	~ 200	33.0	-do-	115.84	42.05	-do-	1.74	-do-	1.23
- 10	~ 232	35.0	3.4	130.56	44.62	1/6,300	1.78	1.47	1.21
- 11	~ 260	39.0	-do-	144.16	48.62	-do-	1.80	-do-	1.22
- 12	~ 272	38.0	3.6	149.76	48.18	1/6,600	1.82	-do-	1.18
- 13	~ 310	43.0	-do-	167.76	53.18	-do-	1.84	-do-	1.20
- 14	~ 336	40.0	4.0	176.00	51.31	1/7,000	1.91	1.63	1.17
- 15	~ 353	42.0	-do-	184.00	53.31	-do-	1.92	-do-	1.18
- 16	~ 375	42.0	4.2	194.04	53.28	1/7,300	1.93	1.68	1.15
- 17	~ 402	45.0	-do-	206.64	56.88	-do-	1.95	-do-	1.16
- 18	~ 423	42.0	4.5	209.25	54.73	-do-	2.02	1.76	1.15

(Cont'd)

Type of Canal	Discharge Q(cusecs)	Bed Width W(feet)	Water Depth D(feet)	Area of Section (sq.feet)	Wetted Perimeter (feet)	Hydraulic Gradient S	Mean Velocity V(feet/sec)	Silt Critical Velocity Vo(feet/sec)	V/Vo
A - 19	~ 454	45.0	-do-	222.75	57.73	-do-	2.04	-do-	1.16
20	~ 480	45.0	4.7	233.59	58.29	1/7,500	2.06	1.81	1.14
21	~ 502	47.0	-do-	242.99	60.29	-do-	2.07	-do-	-do-
22	~ 540	41.0	5.3	245.39	55.99	-do-	2.20	1.95	1.13
23	~ 565	43.0	-do-	255.99	57.99	-do-	2.21	-do-	-do-
24	~ 605	-do-	5.5	266.75	58.56	-do-	2.27	2.00	1.14
25	~ 632	45.0	-do-	277.75	60.56	-do-	2.28	-do-	-do-
26	~ 653	-do-	5.6	283.36	60.84	-do-	2.30	2.02	1.14
27	~ 710	49.0	-do-	300.16	63.84	-do-	2.32	-do-	1.15
28	~ 755	52.0	-do-	322.56	67.84	-do-	2.34	-do-	1.16
29	~ 824	55.0	5.7	345.99	71.12	-do-	2.38	2.05	-do-
30	~ 835	-do-	5.8	352.64	71.40	1/7,900	2.37	2.07	-do-
31	~ 900	60.0	-do-	381.64	76.40	-do-	2.37	-do-	-do-
32	~ 940	62.0	-do-	392.24	78.40	1/8,000	2.37	-do-	-do-
33	~ 963	64.0	-do-	404.84	80.40	-do-	2.38	-do-	-do-
34	~ 1,085	72.0	-do-	451.24	85.40	-do-	2.47	-do-	1.20

Note: Inside slope of canal m = 1:1

Table IV.4-8 Hydraulic Calculation of Distributary - Type "B"

Type of Canal	Discharge Q(cusecs)	Bed Width W(feet)	Water Depth D(feet)	Area of Section (sq.feet)	Wetted Perimeter (feet)	Hydraulic Gradient S	Mean Velocity V(feet/sec)	Silt Critical Velocity Vo(feet/sec)	V/Vo
B - 1	5 ~ 20	5.0	2.0	18.00	13.94	1/4,500	1.14	1.10	1.04
2	~ 34	10.0	-do-	28.00	18.94	-do-	1.26	-do-	1.15
3	~ 50	15.0	-do-	38.00	23.94	-do-	1.33	-do-	1.21
4	~ 62	15.0	2.4	47.52	25.73	1/5,000	1.38	1.18	1.17
5	~ 81	18.0	-do-	54.72	27.73	-do-	1.48	-do-	1.25
6	~ 101	20.0	2.6	65.52	31.63	-do-	1.54	1.23	1.25
7	~ 123	26.0	-do-	81.12	37.63	1/5,500	1.52	-do-	1.24
8	~ 151	28.0	2.8	94.08	40.52	-do-	1.61	1.30	1.24
9	~ 165	31.0	-do-	102.48	43.52	-do-	1.62	-do-	1.25
10	450 ~ 482	43.0	4.7	241.58	63.02	1/7,500	1.99	1.81	1.10
11	500 ~ 560	40.0	5.3	268.18	64.33	-do-	2.12	1.95	1.09
12	~ 590	42.0	-do-	278.78	66.33	-do-	2.13	-do-	1.09

Note: Inside slope of canal m = 2:1

IV.4-4 Constant Head Orifice Turnout (Double Gated Turnout)

The Constant Head Orifice Turnout was developed to both regulate and measure the flow of water. At least two gates are required for the structure to operate. The first gate(rectangular gate), the upstream gate, controls the size of the rectangular orifice while the second gate(circular gate), the downstream gate, controls the water depth below the orifice and is operated to maintain the head across the orifice at a constant value.

The discharge flow through the turnout can be computed from the following equation:

$$Q = CA\sqrt{2gh}$$

where: Q = Discharge (cusec)
C = Coefficient of discharge (0.7)
g = Standard gravitational acceleration feet per second squared (32.1741 ff/sec/sec)
h = Differential head (ff)

According to the layout of minor canals, discharge at most of minor canals is about 20 cusec (0.566 cms) or below and is flucuating seasonally in ranging at 90 % of the peak discharge for about 100 days, at 85 % for about 70 days and at 80 % for about 40 days.

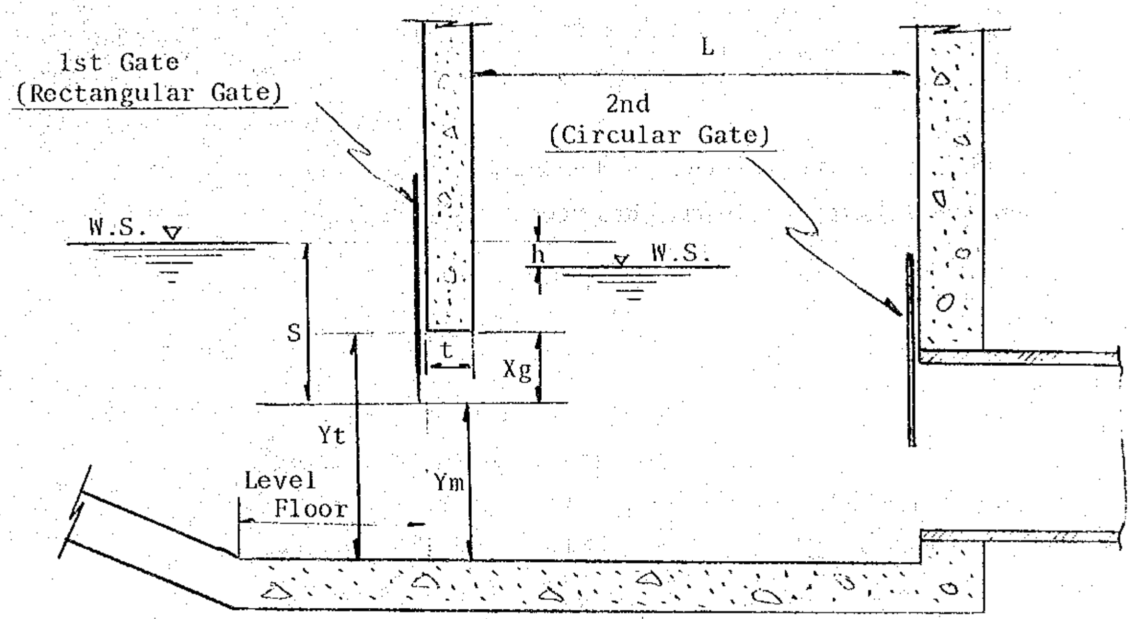
To design of the turnout, the above-mentioned fluctuation of the discharge should be considered. In generally speaking, the constant head orifice turnout with a head differential (h) of 0.2 foot and gate opening of 0.8 is well functioned, but where the head is available a greater differential may be used.

The turnout to divert the flow of water from the distributary to the minor canal should be designed to have a capacity of 20 cusec. Considering the following head differential and discharge, size of the turnout is designed the first gate (rectangular orifice gate) 40 inches wide by 24 inches high and the second gate (circular gate) 32 inches in diameter.

Discharge	Orifice Gate Opening	Head Differential
20 cusec (100%)	Full Opening (0.9)	0.35 ft.
18 " (90%)	" (0.9)	0.29 "
16 " (80%)	" (0.9)	0.23 "
14 " (70%)	" (0.85)	0.20 "

Outlet of the turnout is designed a 32-inch-diameter to be installed to keep a minimum head of 0.4 foot.

Dimensions for Constant Head Orifice Turnout



Ym = Gate opening for maximum Q, Ym/Yt = 0.8 max.

Yt = Full gate leaf lift

S = submergence

"Xg" must be equal to or greater than "t" for max.Q.

"S" is equal to or greater than "Ym" for good accuracy.

For Q up to 10 cfs, L must be at least $2\frac{1}{4}$ Ym or $1\frac{3}{4}$ Yt, whichever is greater (3' - 6" minimum)

For Q above 10 cfs, L = $2\frac{3}{4}$ Ym minimum.

h = 0.2' (normally)

A level floor length equal to the height of the orifice gate opening (Ym) should be provided in front of the 1st Gate (orifice gate).

IV.4-5. Design of Bridges

1. Design Criteria for Canal Structures

From results of the investigation which have been made at field sites in Pakistan, it was clarified that India Cord, BS Cord ard US Cord are generally adopted for the design of canal structures. Therefore, the design of canal structures including this project was made by following processes.

1) Assumption of a temporary design criteria.
2) Examination whether the assumption is suitable or not for the stubility calculation of existing structures.

If this assumption is suitable for the existing structures, the temporary design criteria is adopted as a design criteria for canal structures.

a. Condition of Lands

The safety of structures is confirmed for normal and earthquake conditions, and the condition of loads is as follows:

1) Normal Condition
   The load acted at structures is devided into earth and water pressures, dead and live loads and impact load.
2) Earthquake Condition
   The load is devided into earth pressure, water pressure, dead load and force of inertia for earthquake.

b. Bulk Density

Bulk densities of following materials to be used for structures are determined on the basis the data which is being adopted in Pakistan.

Cement	$\gamma_c = 150^{pcf}$	($\doteqdot$ 2.40 t/m^3)
Water	$\gamma_w = 62.5^{pcf}$	($\doteqdot$ 1.00 ")
Soil	$\gamma_s = 110^{pcf}$	($\doteqdot$ 1.76 ")
Brick	$\gamma_b = 120^{pcf}$	($\doteqdot$ 1.92 ")

c. Seismic Coefficient

A seismic coefficient is determined from the result of trials for existing structures.

1) Design Horizontal Seismic Coefficient $KH = 0.05$

2) Design Vertical Seismic Coefficient $kv = 0$

d. Coefficient of Earth Pressure

Since field soil property consists of sandy silt and/or sand, an internal friction angle for these soils is adopted a 33-degree angle, and earth pressures acting structures are calculated by Rankine Formula.

1) Normal Condition

Coefficient of Active Earth Pressure $KA = 0.29$

Coefficient of Passive Earth Pressure $Kp = 3.39$

2) Earthquake Condition

Compositive Angle of Earthquake $\theta = 29°$

Cofficient of Active Earth Pressure $KAE = 0.30$

Coefficient of Passive Earth Pressure $KPE = 3.38$

e. Live Load

In accordence with the classification of roads applying in Pakistan, roads which the construction of bridges is proposed are classified into Arterial Road (Class AA) and Village Road (Class VR), and the Arterial Road included in this design is National Highway Road where crosses in the project. Following values are adopted as the design load of National Highway Road and Village Road bridges.

1) National Highway Road Bridge (Class AA)

i) Live Load (Weight of tank) 156,800 ℓbs (71.13 ton)

ii) Impact Coefficient $i = 0.1$

iii) While tanks are passing on the bridge, no live loads act on it at the same time excepting that of the tank. Minimum interval between front vehicle and rear one is 300ft(91.4m).

iv) The Position where loads act on the bridge.

Refer to Fig - 1. A

2) Village Road Bridge (Class VR)

i) Live Load (Weight of tank) 28,000 ℓbs (12.70 ton)

ii) Impact Coefficient $i = 0.1$

iii) While tankes are passing on the bridge, no live load act on it at the same time excepting that of the tank. Minimum interval between front vehicle and rear one is 80ft (24.4m).

iv) The position where loads act on the bridge.

Refer to Fig. IV.4-1. B

f. Allowable Stress Intensity

Following values are adopted as allowable stress intensity of materials to be used for structures, and in case of earthquake condition, the stress intensity is increased at 133 percent of the normal one.

1) Concrete

Allowable Compressive Stress Intensity $\sigma c = 750^{psi}$ ($52.73 kg/cm^2$)

Allowable Tensile Stress Intensity $\sigma' c = 75^{psi}$ ($5.27 kg/cm^2$)

2) Steel Bar

Allowable Tensile Stress Intensity $\sigma s = 20{,}000^{psi}$ ($1{,}406 kg/cm^2$)

3) Brick

Allowable Tensile Stress Intensity $\sigma' b = 25^{psi}$ ($1.76 kg/cm^2$)

g. Economical Section for Remforced Concrete

The size of reinforced concrete structures is determined on the basis of a elastic theory and a ratio of elastic coefficients of the concrete and steel bar adopts 15 (= m).

1) General Equetion (Refer to Fig. IV.4-2)

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

$$k = \frac{m.fc}{fs + mfc}$$

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

$$As = \frac{M}{fs.jd} = \frac{M \times 12^2}{2{,}534{,}400 \times d} \text{ (sqin)}$$

$$d = \sqrt{\frac{M}{fs.j\ P.d}} = \sqrt{\frac{M}{17{,}107 \times b}} \text{ (ft)}$$

Where, $fc = 750^{psi} = 108{,}000^{psf}$ ($52,73 kg/cm^2$)

$fs = 20{,}000^{psi} = 2{,}880.000^{psf}$ (1.406 ")

$m = 15$

$P = 0.00675$

d = thickness of structure

M = Moment

2) Rectangular Section (Rfer to Fig.IV-4-3)

$$\sigma c = \frac{2M}{k.j.b.d^2}$$

$$\sigma s = \frac{m\sigma c(1-k)}{K}$$

where: $k = \sqrt{2mp + m^2p^2} - mp$

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

3) T — Beam Section

$$As = \frac{M}{fs.j.d}$$

$$\sigma c = \frac{M.k.d}{b.t(k.d - \frac{t}{2}) \times jd}$$

where: $k = \dfrac{np + \frac{1}{2}(\frac{t}{d})^2}{np + \frac{t}{d}}$ $\qquad b \leqq \frac{1}{4}\ell$ (ℓ; span length)

$b \leqq S$

$jd = d - \dfrac{t}{d}$ $\qquad b \leqq 12 \times t$

Concerning b,d and t, please refere to Fig IV.4-4.

2. Design of National Highway Road Bridge

a. Design Condition

1) Live load acting on the embankment which has filled at the back of walls...... 60^{pst} ($0.29t/m^2$)

2) Live load $156,800^{\ell bs}$ (71.13ton)

3) Percentage which a girder supports the live load.

i) Middle Girder

In case of $S \leqq 10^{\frac{1}{2}t}$ $\qquad$ for $156,800^{\ell bs}$ ..... $\frac{S}{7.5}$

for 28,000 .....$\frac{8}{7}$

where S: Effective Span Length (ft)

ii) Edge Girder

For the edge girder, a fulcrum reaction is adopted.

4) Allowable Bearing Capacity of Foundation

$qa = 3000^{psi}$ ($14.65\ t/m^2$)

In case of earthquake condition

$qak = 3000 \times 1.33 = 3990^{psi}$ ($19.48t/m^2$)

b. Design of Upper Structure

1) General Drawing

Refer to Fig.IV.4-5)

2) Study for Slub

i) Edge

As the stress by a live boad acting on the slub is scarcely established at the edge of slub, the study of structural stability is canceled.

ii) Middle (Refer to Fig.IV-4-6)

Max. bending moment by live loads including impact loads.

$$P = 78,400 \times \frac{1}{1,200} \times 1.10 = 7,187^{ps}$$

$$M = \frac{1}{8} \times P \times \ell = 5,839^{ft\text{-}\ell bs}$$

Max. bending moment by dead loads

$$W = (0.71 + 0.25) \times 150 = 144^{psf}$$

$$M = \frac{1}{12} \times W \times \ell^2 = 507^{ft\text{-}\ell bs}$$

Required effective height of the slub.

$$d = \sqrt{\frac{5839 + 507}{17,107 \times 1.00}} = 0.61 \text{ ft}$$

Quantity of steel bars of the slub

Steel bars of half inch diameter are arranged a piece to every 3 inches. In this case, total arears of the bar are estimated at 0.8 sq.m per ft.

$d = 0.61'$ $b = 1.0'$ $mp = 0.137$ $k = 0.404$ $j = 0.865$

$$\sigma c = \frac{2M}{kjbd^2} = 678^{psi} < 750^{psi} \quad (47.7\text{kg/cm}^2 < 52.7\text{kg/cm}^2)$$

$$\sigma s = \frac{m.\sigma c.(1-k)}{K} = 15,003^{psi} < 20,000^{psi} (1,055\text{kg/cm}^2 < 1,406\text{kg/cm}^2)$$

3) Study for Girder

i) Interior Girder

. Live loads including impact loads

$$S = 6.5' \qquad \frac{S}{7.5} = 0.87$$

$$P = 156,800 \times \frac{1}{2} \times 0.87 \times 1.10 = 75,029^{\ell bs}$$

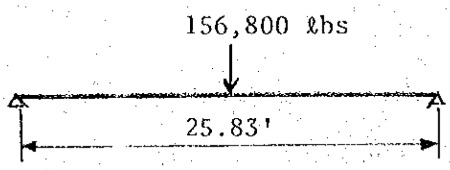
. Dead load

$$3.13 \times 1.25 \times 150 = 587^{pf}$$

$$587 + 144 \times 6.5 = 1,523^{pf}$$

. Span length

$$27.00 - 2 \times 7'' \times \frac{1}{12} = 25.83'$$



Max. bending moment

$$\frac{1}{4} \times 75,029 \times 25.83 = 484,500^{ft\text{-}\ell bs}$$

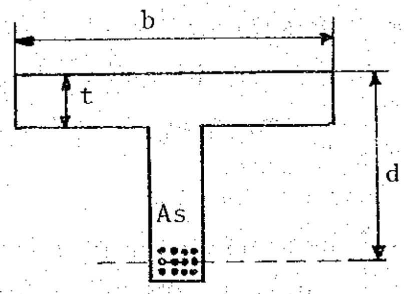
$$\frac{1}{8} \times 1.523 \times 25.83^2 = 127,016 \quad ''$$

Total 611,516$^{ft\text{-}\ell bs}$

Steel bars of one inch diameter are arranged 12 pieces in the beam.

$As = 0.79 \times 12 = 9.48^{sq.i}$ $t = 0.96'$ $d = 3.75'$

$b = 6.5'$



$$jd = d - \frac{t}{2} = 39.24'' \qquad mp = \frac{As \times m}{b.d} = 0.0405 \qquad k = 0.247$$

$$\sigma s = \frac{M}{As.jd} = 19,727^{psi} < 20,000^{psi} (1,387kg/cm^2 < 1,406kg/cm^2)$$

$$\sigma c = \frac{M.k.d}{b.t.(k.d - \frac{t}{2})jd} = 432^{psi} < 750^{psi} (30.4kf/cm^2 < 52.7kg/cm^2)$$

iii) Exterior Girder (Refer to Fig.IV.4-7)

live load Pe $= 78,400 \times \dfrac{4.37}{650} \times 1.10 = 57,980^{\ell bs}$

dead load Pd $= 1646^{pf}$

length of span $27.00' - 2 \times 7'' \times \dfrac{1}{12} = 25.83'$

Max. bending moment M

$$M1 = \frac{1}{4} \times 57,980 \times 25.83 = 374,406^{ft\text{-}\ell bs}$$

$$M2 = \frac{1}{8} \times 1,646 \times 25.83^2 = 137,274 \quad ''$$

Total 511,680 "

Steel bars of one inch diameter are arranged 12 pieces in the beam.

$As = 9.48^{q.i}$ $t = 0.96'$ $d = 3.75'$ $b = 3.0'$

$jd = d - \frac{t}{2} = 39.24''$ $mp = 0.0872$ $k = 0.351$

$\sigma s = 16,506^{psi} < 20,000^{psi}$ $(1,161kg/cm^2 < 1,406kg/cm^2)$

$\sigma c = 594^{psi} < 750^{psi}$ $(41.8kg/cm^2 < 52.7kg/cm^2)$

c, Design of Substructure

1) Pier

i) General Drawing

Refer to Fig

ii) Weight of Structure

$W = 9,060^{\ell bs}$

iii) Reaction of Upper Structure

In case of earthquake condition

$N = 7,137^{\ell bs}$

$H = 7,137 \times 0.05 = 357^{\ell bs}$

Section force for A - A Section

$M = 6,605^{ft-\ell bs}$

Section force for Bottom

$M = 7,319^{5t-\ell bs}$

iv) Uplift

W.L.	A - A Section		Bottom	
	U(ℓbs)	M(ft-ℓbs)	U(ℓbs)	M(ft-ℓbs)
F.S.L.	2,906	0	4,219	0
D.B.L.	1,031	0	2,344	0

v) Total Section Force

W.L.	A - A Section			Bottom		
	N. (ℓbs)	H. (ℓbs)	M. (ft-ℓbs)	N. (ℓbs)	H. (ℓbs)	M. (ft-ℓbs)
F.S.L.	10,891	591	9,413	11,978	591	10,595
D.B.L.	12,766	591	9,413	13,853	591	10,595

vi) Study for A - A Section

In case of earthquake

WL = F.S.L.

$$\sigma c = \frac{N}{A} + \frac{M}{Z} = \frac{12,766}{1 \times 3 \times 12^2} + \frac{6 \times 9413}{1 \times 3^2 \times 12^2} = 73.1^{psi} < 200^{psi}$$

($5.1kg/cm^2 < 14.06kg/cm^2$)

WL = D.B.L.

$$\sigma c = \frac{10,891}{1 \times 3 \times 12^2} + \frac{6 \times 9413}{1 \times 3^2 \times 12^2} = 68.8^{psi} < 200^{psi}$$

($4.8kg/cm^2 < 14.06kg/cm^2$)

vii) Study for Bottom

In case of earthquake

W.L = F.S.L.

$$\sigma c = \frac{13.953}{1 \times 5 \times 12^2} + \frac{6 \times 10.595}{1 \times 5^2 \times 12^2} = 34.3^{psi} < 200^{psi}$$

($2.4kg/cm^2 < 14.6kg/cm^2$)

W.L = D.B.L.

$$\sigma c = \frac{13.953}{1 \times 5^2 \times 12^2} + \frac{6 \times 10.595}{1 \times 5^2 \times 12^2} = 36.9^{psi} < 200^{psi}$$

($1.6kg/cm^2 < 14.6kg/cm^2$)

2) Abutment

i) General Drawing

Refer to Fig - 9

ii) Weight and Moment of Structure

Refer to Fig - 10

Total weight $\overline{W}$ = 38,577 ℓb

Moment A - A Section Mx = $162^{ft-\ell bs}$

My = 328,851 "

Bottom Mx = 12,845 "

My = 406,674 "

iii) Reaction

Condition	A - A Section			Bottom		
	N (ℓbs)	H (ℓbs)	M (ft-ℓbs)	N (ℓbs)	H (ℓbs)	M (ft-ℓbs)
Normal Condition	8,458	0	47,196	8,458	0	47,196
Earthquake Condition	3,599	180	23,412	3,599	180	23,772

iv) Uplift

W.L	A - A Section		Bottom	
	U(ℓbs)	M(ft-ℓbs)	U(ℓbs)	M(ft-ℓbs)
F.S.L.	194	0	244	68
D.B.L.	69	0	109	0

v) Live Load

Normal Condition

Section A - A	N = 600 ℓbs	M = 750 ft-ℓbs
Bottom	N = 660 "	M = 1,155 "

vi) Earth Pressure

Condition	A - A Section		Bottom	
	N(ℓbs)	M(ft-ℓbs)	N(ℓbs)	M(ft-ℓbs)
Normal F.S.L.	3,909	49,873	2,752	56,607
-do- D.B.L.	5,808	60,590	5,020	74,538
Earthquake F.S.L.	3,740	47,301	2,587	53,704
-do- D.B.L.	5,705	58,385	4,933	69,125

vii) Total Section Force

Condition	A - A Section			Bottom		
	N(ℓbs)	H(ℓbs)	M(ft-ℓbs)	N(ℓbs)	H(ℓbs)	M(ft-ℓbs)
Normal F.S.L.	40,882	3,909	96,157	47,451	2,752	89,871
-do- D.B.L.	41,007	5,808	106,874	47,586	5,020	107,734
Earthquake F.S.L.	35,423	5,112	86.994	41.932	4.053	85.033
-do- D.B.L.	35,548	7,077	98,078	42,067	6,399	100,386

viii) Study for A-A Section

Normal Condition

WL = FSL

$$\sigma = \frac{N}{A} + \frac{M}{Z} = \frac{40,882}{1 \times 12.5 \times 12^2} + \frac{6 \times 96,157}{1 \times 12.5^2 \times 12} = 48.4^{psi} < 150^{psi}$$

$(3.4kg/cm < 10.55kg/cm^2)$

WL = D.B.L.

$$\sigma = \frac{41,007}{1 \times 12.5 \times 12^2} + \frac{6 \times 106,874}{1 \times 12.5^2 \times 12^2} = 51.3^{psi} < 150^{psi}$$

$(3.6kg/cm^2 < 10.55kg/cm^2)$

Earthquak Condition

WL = FSL

$$\sigma = \frac{N}{A} + \frac{M}{Z} = \frac{35,423}{1 \times 12.5 \times 12^2} + \frac{6 \times 86,994}{1 \times 12.5^2 \times 12} = 42.9^{psi} < 200^{spi}$$

$(3.0kg/cm^2 \leq 14.06kg/cm^2)$

ix) Bottom

Normal Comdition

WL = FSL

$$\sigma = \frac{47,586}{1 \times 14.5 \times 12^2} + \frac{6 \quad 89,871}{1 \times 14.5^2 \times 12^2} = 40.5^{psi} < 150^{psi}$$

WL = DBL

$$\sigma = \frac{47,586}{1 \times 14.5 \times 12^2} + \frac{6 \times 107,734}{1 \times 14.5^2 \times 12^2} = 44.1^{psi} < 150^{psi}$$

Earthquake Condition

WL = FSL

$$\sigma = \frac{41,932}{1 \times 14.5 \times 12^2} + \frac{6 \times 85,033}{1 \times 14.5^2 \times 12} = 36.9^{psi} < 200^{psi}$$

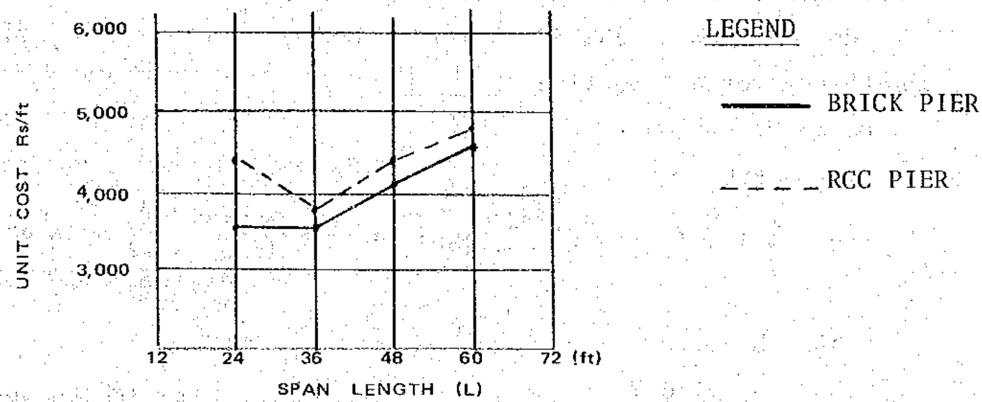
WL = DBL

$$\sigma = \frac{42,067}{1 \times 14.5 \times 12^2} + \frac{6 \times 100,386}{1 \times 14.5^2 \times 12^2} = 40.0^{psi} < 200^{psi}$$

3. Design of Village Road Bridge

a. Study for Span Length and Materials to be used for Pier

In order to determine the most economical bridge, 4 cases of span length (24, 36, 48 and 60 feet) and 2 kinds of pier constructed by bricks and concrete have been studied, and the result is as follows:



From above study, the span length and pier material were determined at 36 ft and brick respectively.

b. Design of Upper Structure

1) General Drawing

Refer to Fig. IV.4-11

2) Study of Slub

i) Edge

Weight $W = 664^{\ell bs/ft}$

Moment $M = 1,045^{ft\text{-}\ell bs}$

$$d = \sqrt{\frac{M}{17,107 \times b}} = 0.25^{ft} = 3.0^{in} < 10^{in}$$

$$As = \sqrt{\frac{M \times 12^2}{2,534,400 \times d}} = 0.09^{sq.in/ft}$$

ii) Middle

Max. bending moment for live load(including impact load)

$$P = 14,000 \times \frac{1}{9} \times 1.25 = 1,944^{\ell bs/ft}$$

$$M = \frac{1}{8} \times P.\ell = 1,944^{\ ft-\ell bs}$$

Max. bending moment for dead load

$$W = (10'' + 3'') \times 150 \times \frac{1}{12} = 163^{\ell bs/sq.ft}$$

$$M = \frac{1}{12} \cdot W \cdot \ell^2 = 869^{ft-\ell bs}$$

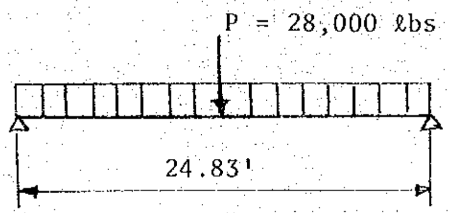
$$d = \sqrt{\frac{1944 + 869}{17,107 \times 1.0}} = 0.41^{ft} \div 5^{in} < 8^{in}$$

$$As = \frac{(1944 + 869) \times 12}{2,534,400 \times 12^2 \times 8/12} = 0.24^{sq \cdot ft/ft}$$

Steel bars of half inch diameter are arranged a piece to every 6 inches. In this case, the total areas of steel bar are estimated at 0.4 sq·ft per ft.

iii) Girder

$$\text{Span } \ell = 36.0' - 7'' \times 2 \times \frac{1}{12} = 34.83^{ft}$$



Live load including impact load

$$L = 28,000 \times \frac{1}{2} \times 1.143 \times 1.5 = 24,003^{\ bs}$$

Dead load

$$4'2'' \times 18'' \times 150 = 938^{\ell bs/ft}$$

$$938 + 163 \times 8.0 = 2,242^{\ ''}$$

Max. bending morment

$$M = \frac{1}{4} \times 24,003 \times 23.83 + \frac{1}{8} \times 2,242 \times 34,83 = 548,985^{ft-\ell bs}$$

Study of section

$M = 548,985^{ft-\ell bs}$ $\quad t = 10''$ $\quad d = 4'10''$

$b = 2 \times 30'' + 18'' = 78'' = 6.5^{ft}$

$$jd = d - \frac{t}{2} = 4'10'' - \frac{10''}{2} = 4'5'' = 53''$$

$$As = \frac{M}{fs \cdot jd} = \frac{548,985 \times 12}{20,000 \times 53} = 6.21^{sq.in}$$

Steel bars of one inch diameter are arranged 5 pieces double.

In is case $\quad As = 0.79 \times 10 = 7.9^{sq.in} \quad 6.21^{sq.in}$

$$\sigma c = \frac{M.k.d}{bt(kd - \frac{t}{2})jd} = \frac{548,985 \times 0.207 \times 58 \times 12}{78 \times 10 \times (0.207 \times 58 - \frac{10}{2}) \times 53} = 273^{psi} < 750^{psi}$$

$k = 0.207$

b. Substructure

i) Pier (Refer to Fig - 12)

$N_1 = \overline{W} = 1,123 \times 150 = 168,450^{\ell bs}$

$H_1 = 0.05\ \overline{W} = 0.05 \times 168,450 = 8,423^{\ell bs}$

W.L.	ΣW(ℓbs)	ΣH(ℓbs)	ΣMo(ft-ℓbs)	ΣMt(ft-ℓbs)
F.S.L.	372,685	13,304	1,863,425	311,566
D.B.L.	412,560	13,295	2,072,275	311,566

Sliding

$$F = \frac{u.\Sigma W}{\Sigma H} = 11.21 \qquad (12.41)$$

Overturning

$$x = \frac{\Sigma Mo - \Sigma Mt}{\Sigma W} = 4.16 \quad (4.27)$$

$$e = \frac{\beta}{2} - x = 0.84^{ft} \qquad (0.73) < \frac{\beta}{4} = 2.5^{ft}$$

Bearing Capacity

$$q = (1 \pm \frac{6e}{\beta})\ \frac{\Sigma W}{A} = 2,735^{psf} \qquad (2,894) < 3,000^{psf}$$

Note: parenthese mean the value for D.B.L.

ii) Abutment

Refer to Fig - 13

W.L.	ΣW(ℓbs)	ΣH(ℓbs)	ΣMo(ft-ℓbs)	ΣMt(ft-ℓbs)
F.S.L.	461,405	135,572	4,106,556	1,243,936
D.B.L.	615,921	157,942	5,690,345	1,351,859

Sliding

$$F = \frac{u.\Sigma W}{\Sigma H} = 1.36 \qquad (1.6) > 1.2$$

Overturning

$$x = \frac{\Sigma Mo - \Sigma Mt}{\Sigma W} = 6.20 \qquad (7.04)$$

$$e = \frac{B}{2} - x = 2.80^{ft} \qquad (1.96) \quad < \frac{B}{4} = 4.5^{ft}$$

Bearing Capacity

$$q = (1 \pm \frac{6e}{B})\frac{\Sigma W}{A} = 3,418^{psf} \qquad (3,902)$$

iii) Wall

Refer to Fig - 14

$\Sigma W = 21,860^{\ell bs/ft}$

$\Sigma H = 1,656 + 6,774 = 8,430^{\ell bs/ft}$

$\Sigma Mo = 178,481^{ft\text{-}\ell bs/ft}$

$\Sigma Mt = 17,476 + 55,487 = 72,963^{ft\text{-}\ell bs/ft}$

$u = 0.4$

Sliding

$F = 1.04$

Overturning

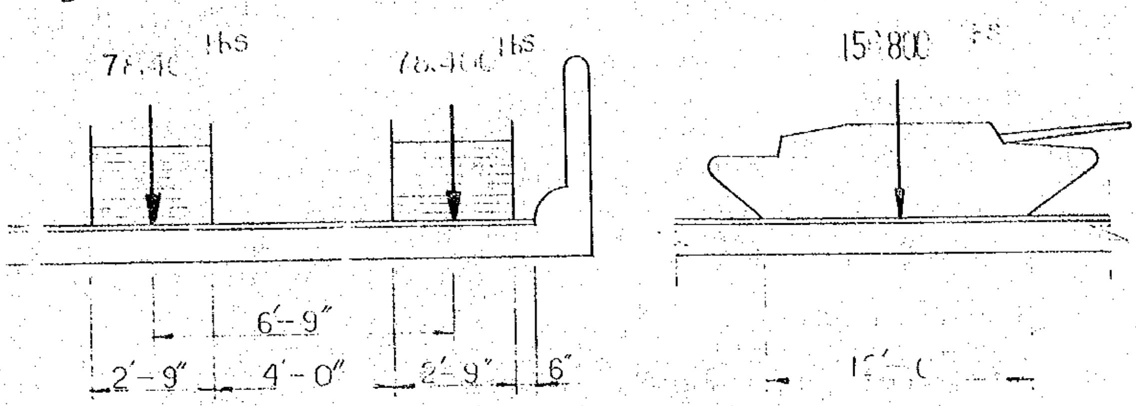
$$x = 4.83^{ft} \qquad e = \frac{B}{2} - e = 2.67^{ft} < \frac{B}{4} = 3.75^{ft}$$

Bearing Capacity

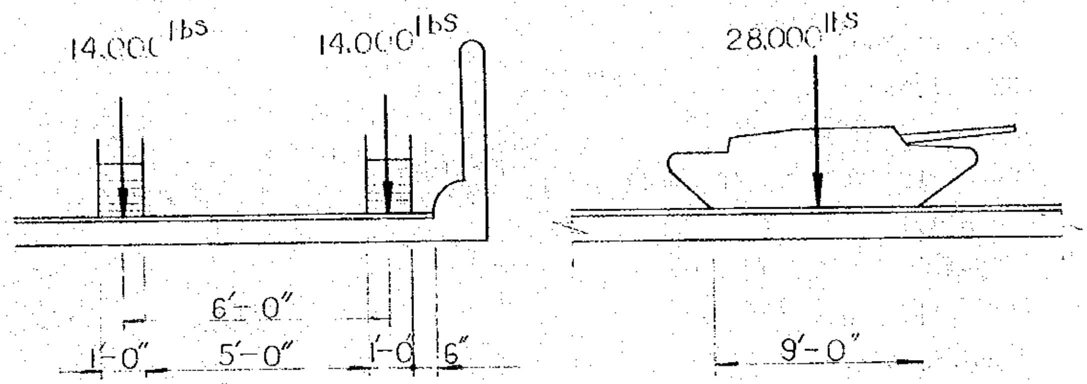
$$q = \frac{2}{3 \times (\frac{1}{2} - \frac{e}{B})} \times \frac{\Sigma W}{A} = 3,017^{psf}$$

$$b = 3 \times (\frac{B}{2} - e) = 14.49^{ft}$$

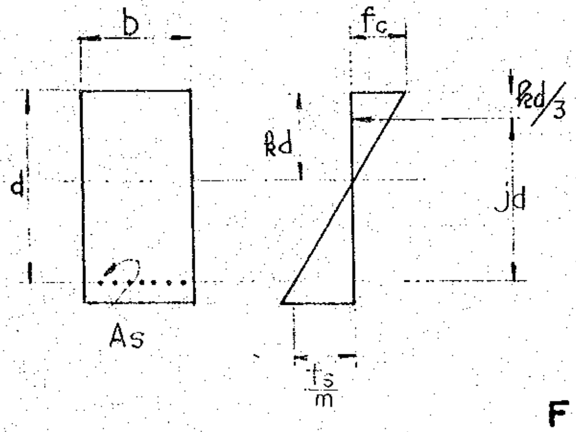
Fig IV.4-3 CONDITON OF LOAD



A. NATIONAL HIGHWAY BRIDGE



B. VILLAGE ROAD BRIDGE



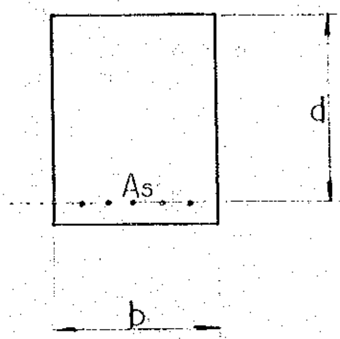


Fig IV.4-5 RECTANGULAR SECTION

Fig IV.4-4 STRESS DISTRIBUTION

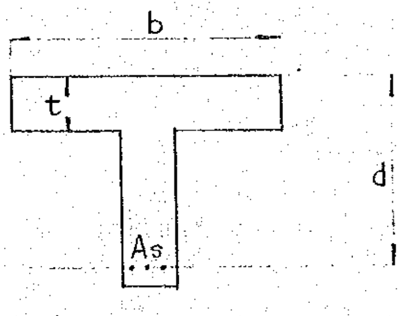
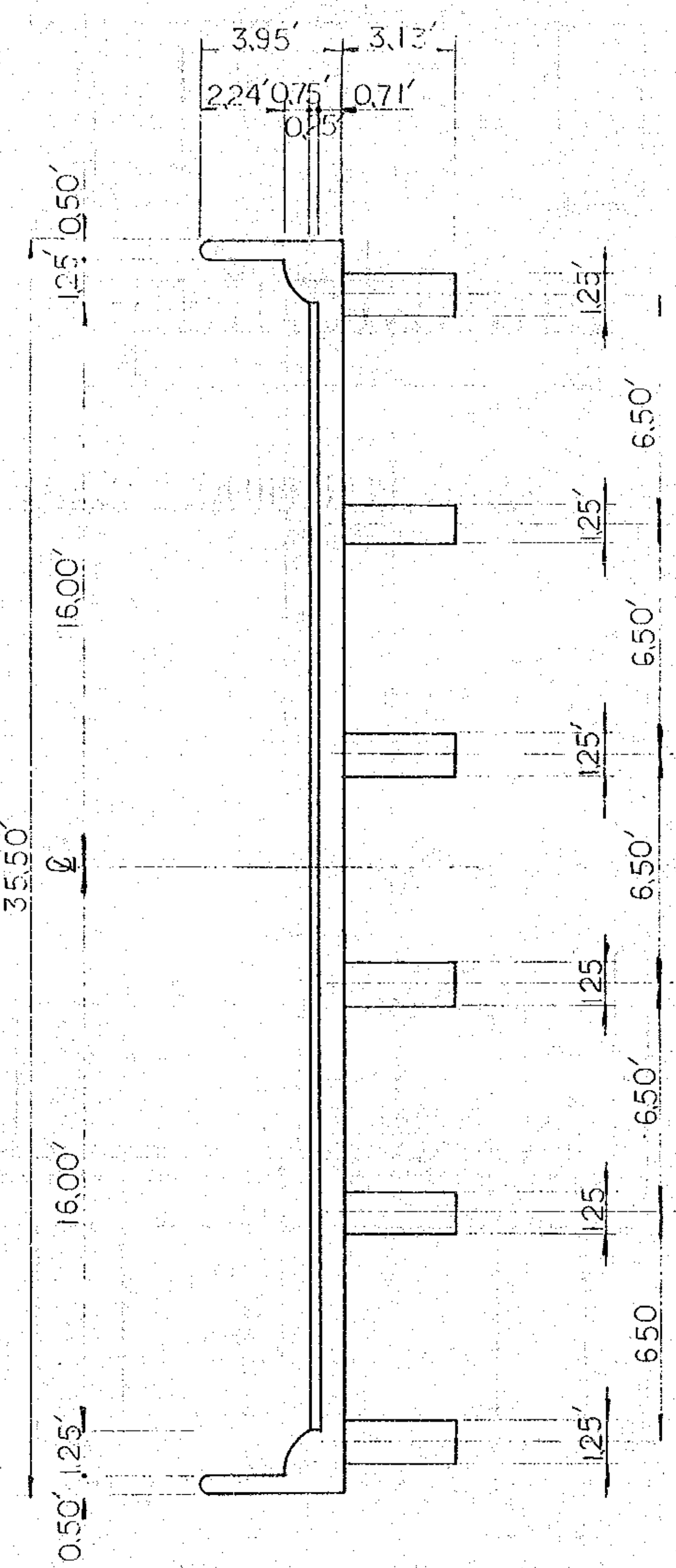


Fig IV.4-6 T-BEAM SECTION



UPPER STRUCTURE

Fig IV.4-7

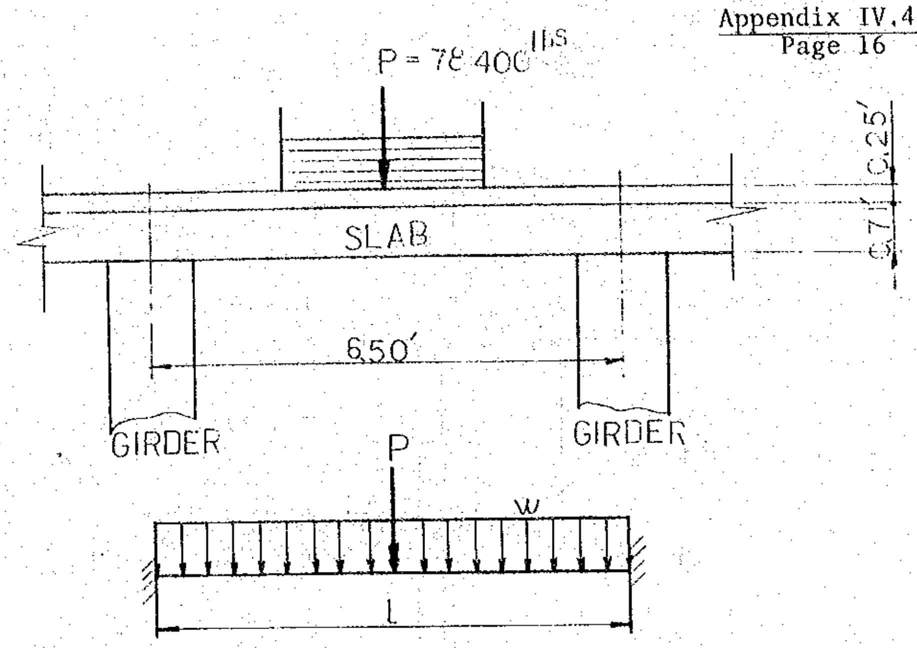


Fig IV.4-8 LOAD FOR MIDDLE SLAB

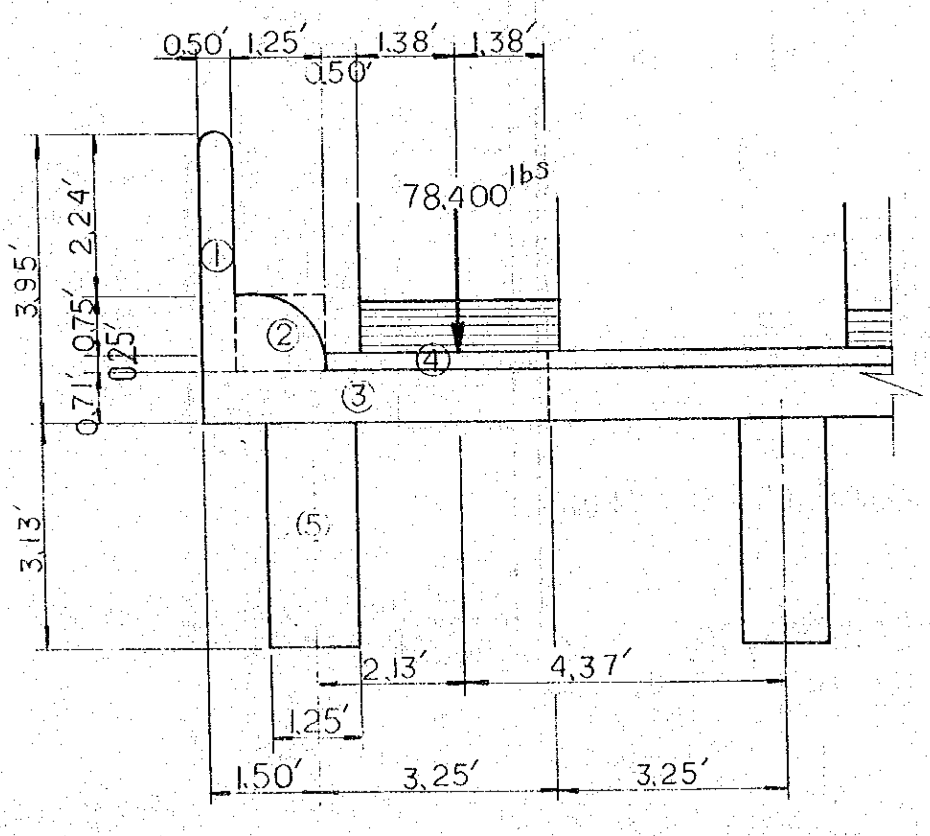


Fig IV.4-9 LOAD FOR EXTERIOR GIRDER

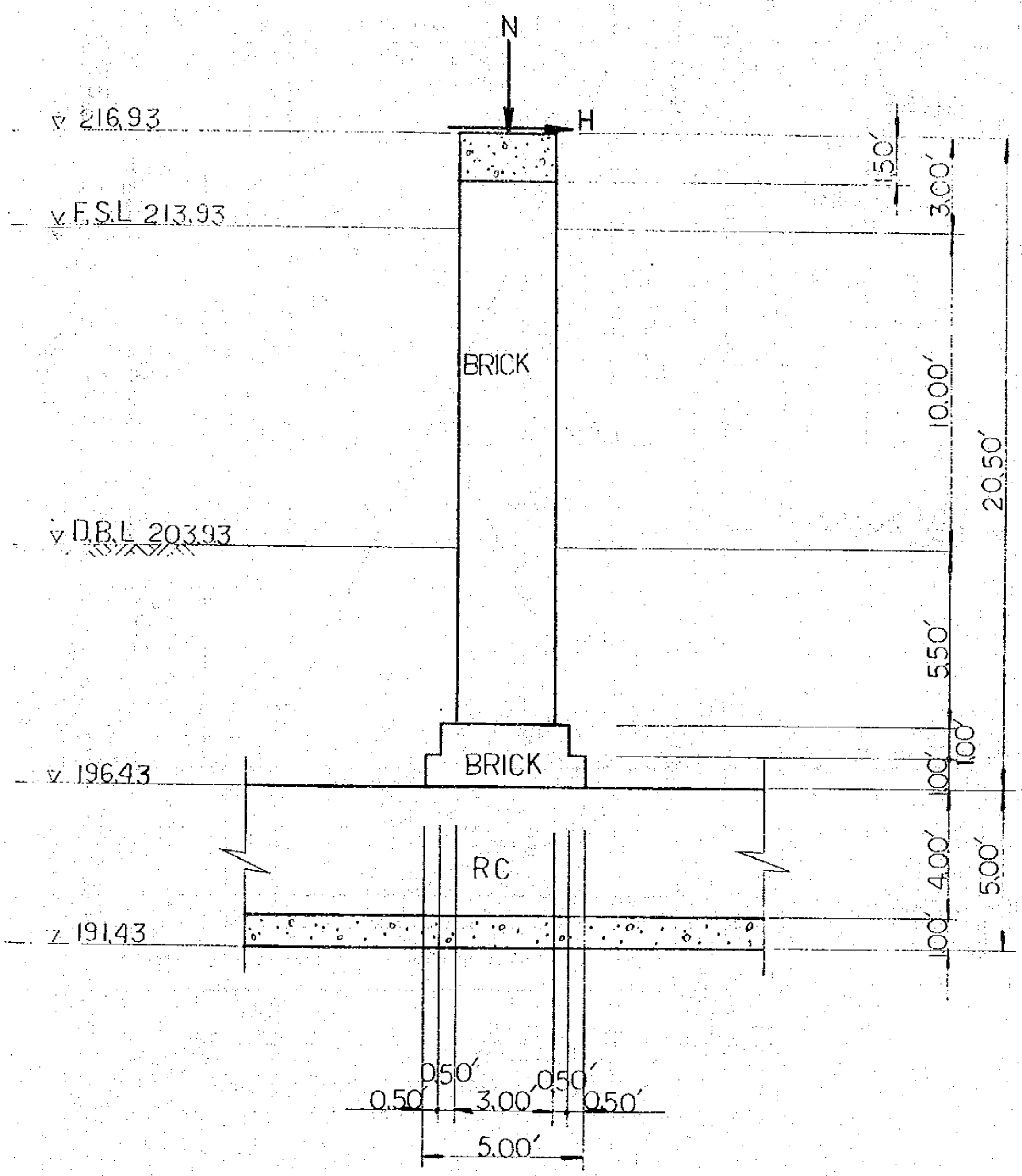


Fig IV.4-10 PIER

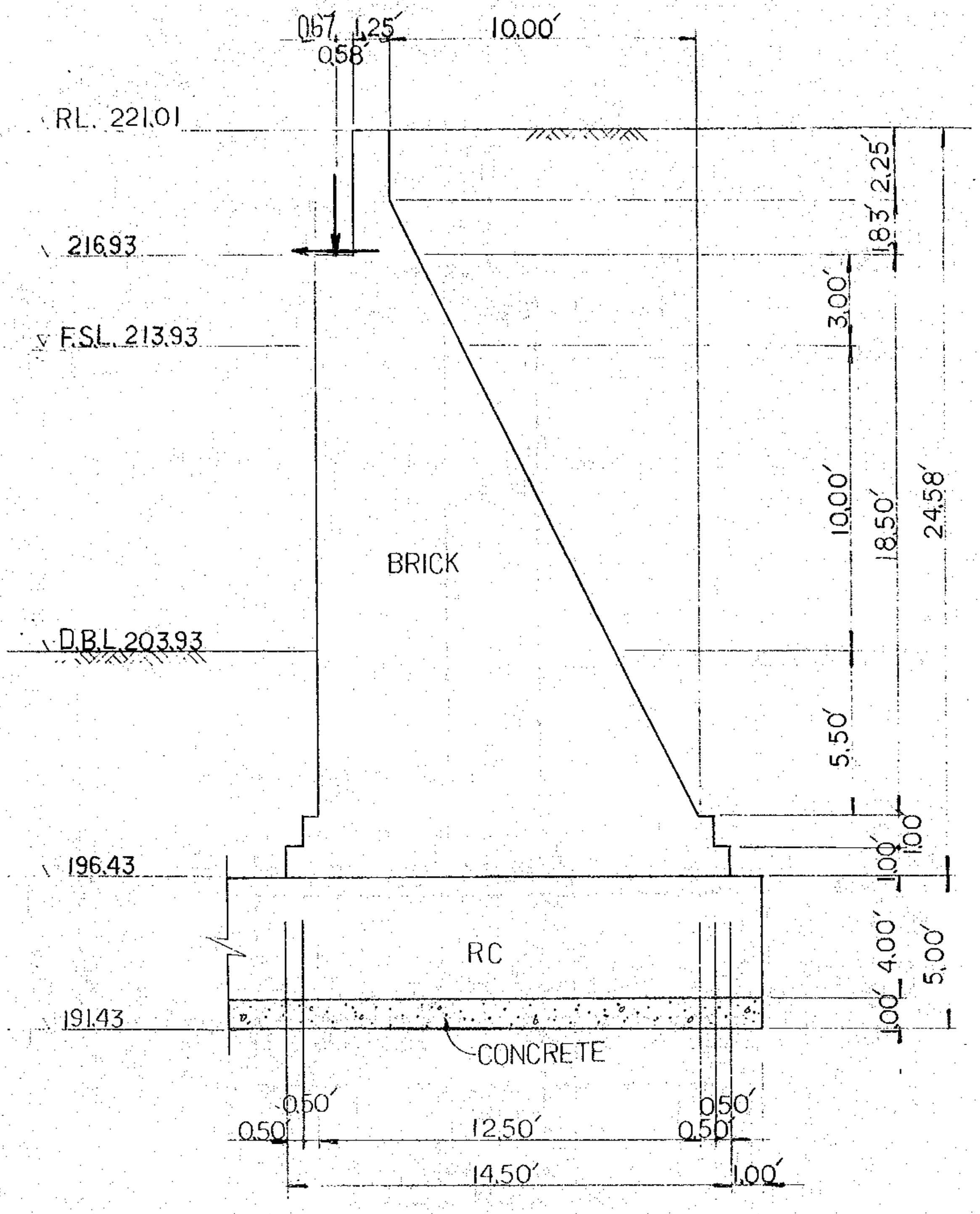


Fig IV.4-11 ABUTMENT

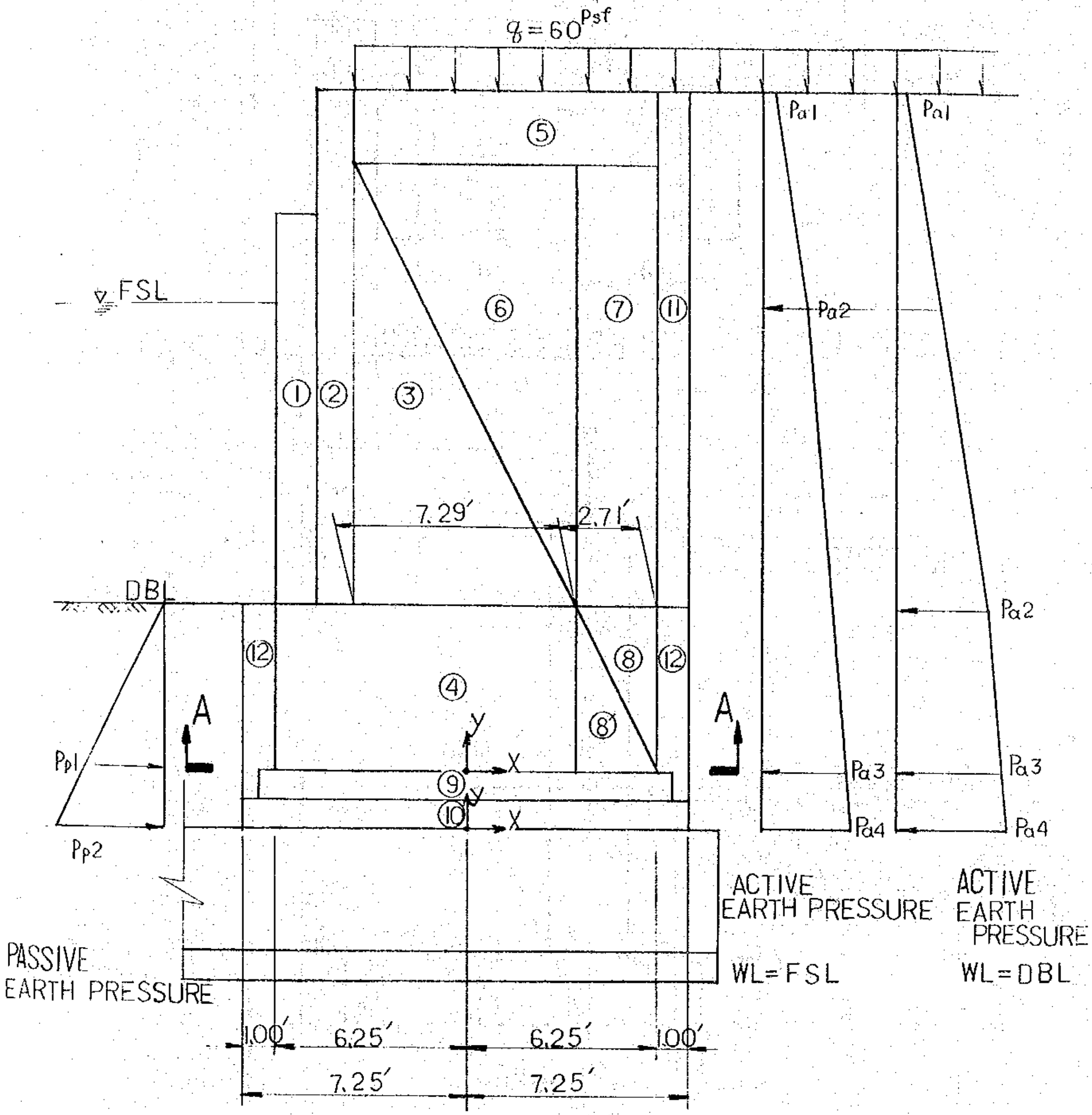


Fig IV.4-12 WEIGHIT AND LOAD

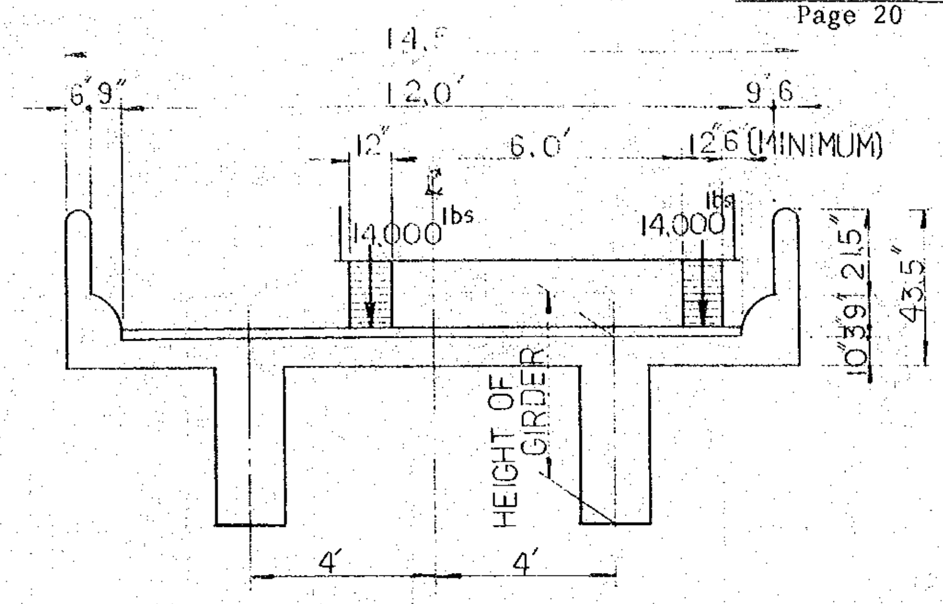


Fig IV.4-13 UPPER STRUCTURE

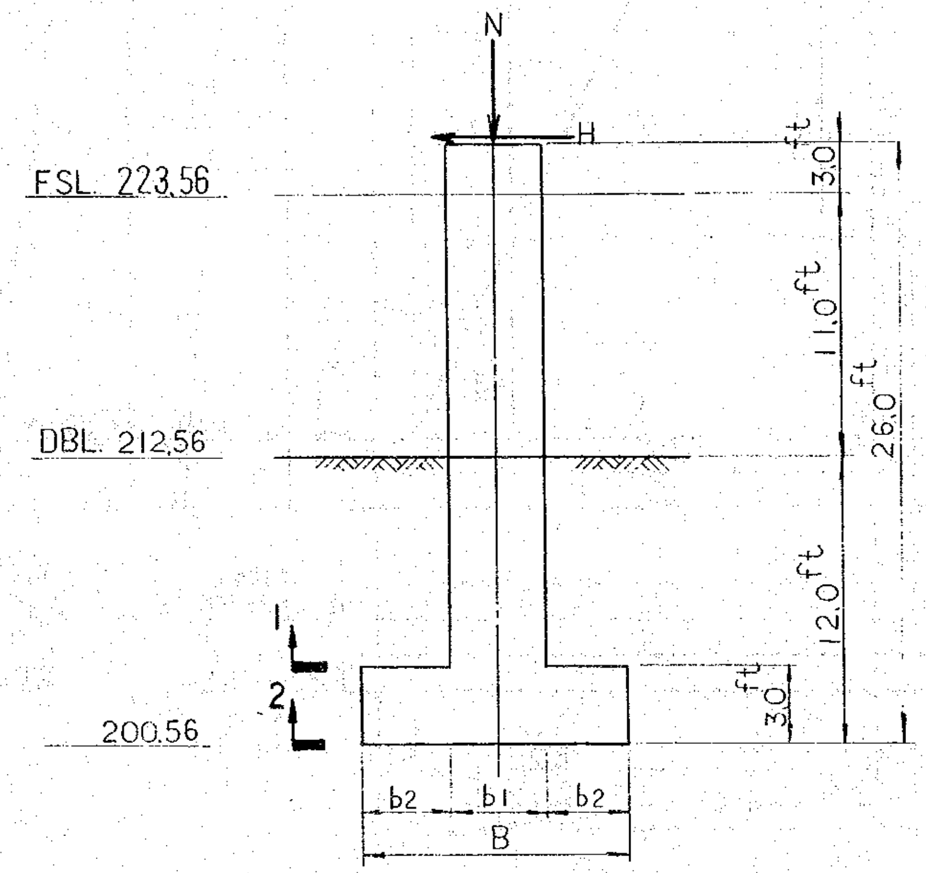


Fig IV.4-14 PIER

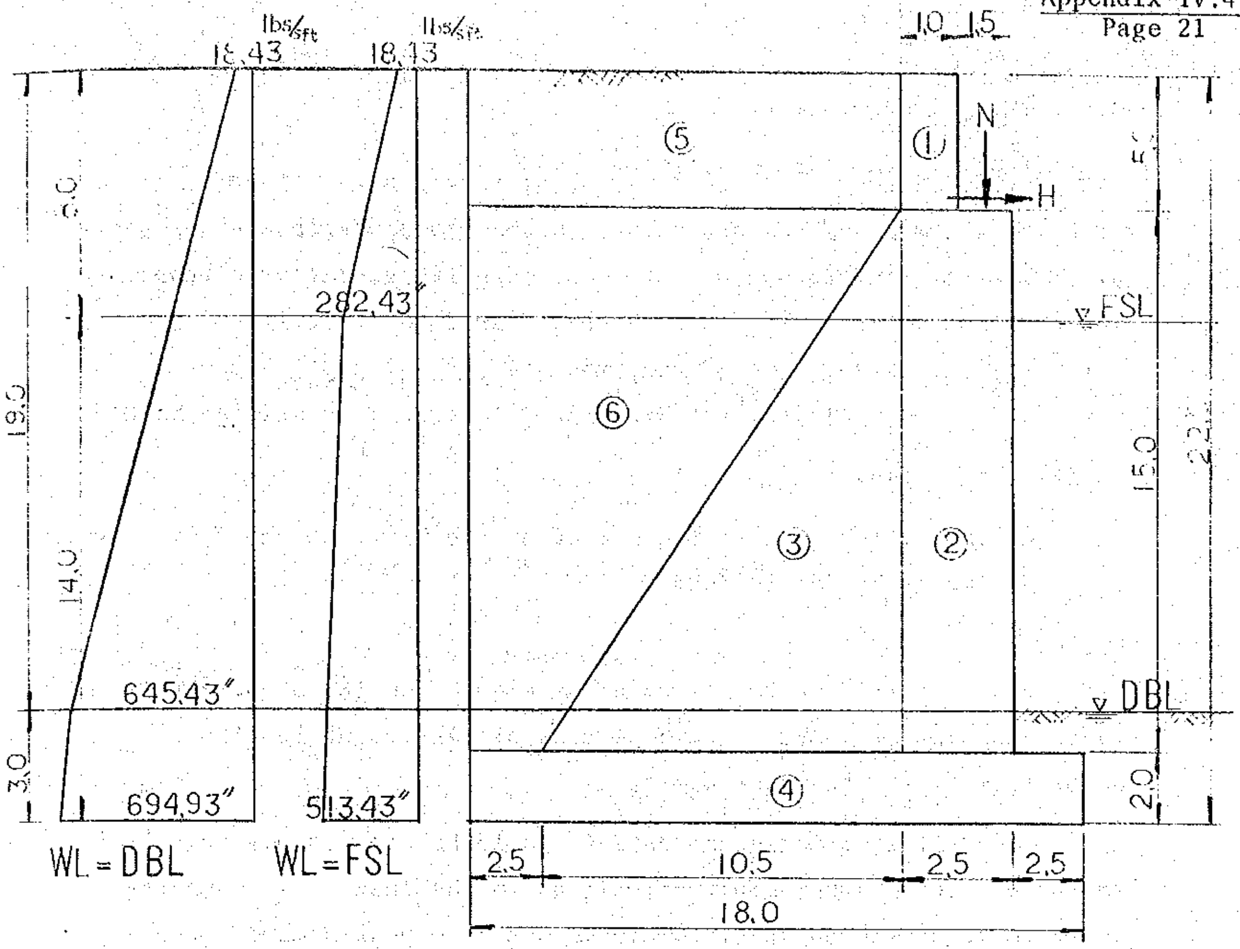


Fig IV.4-15 ABUTMENT

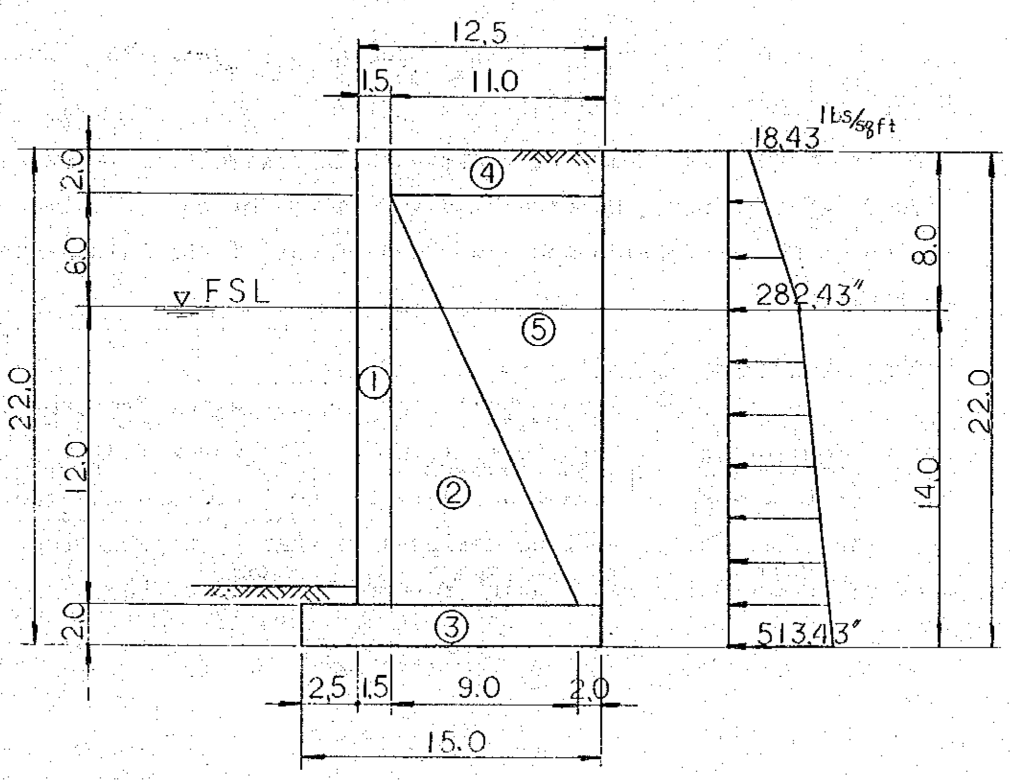


Fig IV.4-16 WALL

IV.4-6. On-farm Development

1) Typical Plan of On-farm Facilities

There exist some water courses in the Project Area, though these are not sufficient in number for the modern irrigated agriculture. The existing water courses have been designed to be utilized to their fullest extent. Based upon the plan, the typical layout on-farm facilities including the Minor is illustrated in drawing No.016.

2) On-farm Facilities

The facilities ranging the outlet on the Minor to the farm plots are described as follows:

Outlet

This facilities divers irrigation water from the minor canal to the main water course, and also controls and regulates the water flowing to the water course.

The ideal location of an outlet should be closest possible to the upstream of the minor as well as to the chak. Its capacity is to be determined by taking into account the peak demand water allowance of 9.23 cusecs per 1,000 acres (for the cases I and III).

The type of the outlet is the module as shown in the drawing No.015, with the mean capacity of 3.23 cusecs.

The type of the outlet is the module as shown in the drawing No.015, with the mean capacity of 3.23 cusecs.

The outlet is a part of the minor canal structure to be designed, constructed, operated, and maintained by the operation agency of the Irrigation and Power Department of the Baluchistan Government.

Main Water Course (M.W.C)

This is the irrigation canal that conveys water from the outlet to the internal water course (I.W.C). The water from the M.W.C will be supplied simultuneously to the I.W.C and therefore, the size of the M.W.C will be designed so that it tapers stage wise as it moves from the outlet forward the head of the last I.W.C.

The designed discharge of the M.W.C will be determined by the peak demand water allowance of 9.23 cusecs per 1,000 acres, however the discharge of the M.W.C should not be less than that of the last I.W.C.

The hydraulic computation follows Lacey's formula as follows:

$$Q = A \times V$$

$$V = \frac{1.3458}{Na} \times R^{3/4} \times S^{1/2}$$

where,
- Q : Discharge (cusec)
- A : Flow area (sq.feet)
- V : Mean velocity (feet per sec)
- Na : 0.0225 x f (f = 0.8)
- f : Lacey's silt factor = 1.75 $m^{1/2}$
- m : Mean diameter of silt in millimeter = 10 percent of the mean diameter of silt particles in the river water. (m = 0.2)
- R : Hydraulic mean depth in feet
- S : Slope of the Canal

The freeboard of the M.W.C is a harf feet (about 15 m).

Nakka

This facility diverts irrigation water from the M.W.C to the Internal Water Course. Its structural type is shown in the drawing No.017 and the designed discharge of the Nakka will be the same as that of the Internal Water Course, or namely, 0.90 cusecs (0.025 cu.m/s).

Internal Water Course (I.W.C)

This is the irrigation canal that conveys water from M.W.C to L.W.C each I.W.C will have a different size of service area and hence a different discharge. However, the mean CCA covered by the I.W.C is about 80 acres.

The I.W.C supplies water simultaneously to the link water course (L.W.C) and the 40 % of the CCA will remain follow, therefore, the size of the same from the Nakka to the head of the third

L.W.C, then tapers toward the last L.W.C.

The formula for the hydraulic computation is that Lacey's as described in the section of main water course. The freeboard of the I.W.C is 0.2 feet (6 cm) and the typical cross section of the I.W.C is shown in the attached drawing.

Division Box

This diverts irrigation water from the I.W.C to the L.W.C, and consists of the check facilities on the I.W.C and intake facilities on the head of the L.W.C. The structures of the check and the intake facilities are the same as shown in the drawing No.017.

Link Water Course (L.W.C)

This is the irrigation canal which conveys water from the I.W.C to the farm lots. Each link water course has a different size of service area and hence a different discharge. It is therefore designed independently of each other, but is uniform in cross-section.

Lacey's formula will be employed as well for the hydraulic computation as discharged in the section of main water course, and freeboard of the I.W.C is 0.2 feet (6 cm). Typical cross section of the L.W.C is shown in the attached drawing.

Farm Drain

Farm drains are to be provided at the lowest portion of the service area. They are the terminal drainage canals made of earth and installed along the next I.W.C. Typical cross section of a farm drain is shown in the attached drawing.

3) On-farm Facilities in Sample Area

Proposed land uses in the sample area are classified as follows :

Land Uses in the Sample Area

Item	Acreage ac	ha
Farm Lot	15,562.0	(6,297.9)
Canal - Minor	4.6	(1.9)
M.W.C	94.7	(38.3)
I.W.C	89.2	(36.1)
L.W.C	85.7	(34.7)
Farm Design	36.0	(14.6)
Branch Drain	110.2	(44.6)
Residential Area and Others	1,507.9	(610.2)
	ac 17,490.3	ha (7,078.3)

The land parcelling and the layout of sample area were made on the map of four inches to one mile in scale. The results obtained from the study of Sample Area are presented in the Table IV.3- in the Appendix IV.3- , and these are applied to the whole Project Area as well the CCA's covered by the irrigation aanal and the canal intensities per acre are summarised as follows :

Summary od Land Parcelling and Canal Length

Description	Quantities	
Mean CCA covered by Minor	2,223 ac	(899.6 ha)
Mean CCA covered by M.W.C (mean chak)	346 "	(140.0 ")
Mean Rotation Area	82.3 "	(33.3 ")
Canal Intensity		
Minor	6.7 feet/ac	(5.0 m/ha)
M.W.C	17.7 "	(13.3 ")
I.W.C	27.7 "	(20.9 ")
L.W.C	30.0 "	(22.6 ")
Farm Drain	20.1 "	(15.1 ")
Branch Drain	20.6 "	(15.5 ")

The layout of sample area is shown in the drawing No.018.

4) Preliminary Design of Sample Area

The designed discharge of the M.W.C depends upon the extent of its service area. The water from the M.W.C will be supplied simultaneously to the I.W.C, so tha the size of the M.W.C becomes smaller towards the down stream. However, the designed discharge of the M.W.C should not be loss than the diverting discharge to the last I.W.C.

The CCA covered by one M.W.C varies in acres from 593 to 194 and its average is about 350 acres the peak demand water allowance including water conveyance losses of the M.W.C is 9.23 cusecs per 1,000 acres. Therefore, the designed discharge of the M.W.C ranges from 5.5 cusecs (156 ℓ/s) to 1.8 cusecs (51 ℓ/s) and its average is 3.23 cusecs (91 ℓ/s). The designed discharge and dimensions of each M.W.C are tabulated as follows :

Designed Discharge and Dimension of Main Water Courses

Description	Unit	Design discharge and Dimensions						
CCA	acres	5.93	500	400	350	300	200	194
Discharge	cusecs	5.47	4.62	3.69	3.23	2.77	1.85	1.79
Bottom Wideth(B)	feet	2.0	2.0	2.0	2.0	2.0	1.5	1.5
Water Depth(D)	"	1.6	1.4	1.3	1.2	1.1	1.0	1.0
Velocity(V)	feet/sec	1.05	0.98	0.94	0.90	0.85	0.77	0.72

Design of Internal Water Course and Link Water Course

Design of I.W.C amd L.W.C shall be made depending upon the irrigation water required for the land soaking and land preparation time of rice. Its requirement is 12.8 inch. Accordingly the designed discharge of the L.W.C is computed as 0.28 cusecs (8 ℓ/sec) amd 0.0177 cusec/acre (1.24 ℓ/sec/ha), whereas tha of the I.W.C is 0.90 cusecs (25 ℓ/sec).

Canal dimensions of the I.W.C and L.W.C are as follows :

Dimensions of I.W.C and L.W.C

Description	I.W.C	L.W.C
Average CCA	80 acres	16 acres
Discharge	0.90 cusecs	0.28 cusecs
Bottom Width	1.0 feet	1.0 feet
Water Depth	0.8 feet	0.5 feet
Velocity	0.63 feet/sec	0.40 feet/sec

Table IV.4-8 Rotation Area and Quantities of On-farm Facilities (1)

Name of Distributary	Name of Minor	Name of Chak	GCA ac	CCA ac	Number of Rotation Area	Mean Rotation Area ac	Quantities: Minor 1,000ft	Outlet	M.W.C 1,000ft	Nakka	I.W.C 1,000ft	Farm Drain 1,000ft	Branch Drain 1,000ft	Main Drain 1,000ft
Jhatpat	A	1	574.6	111	7	73.0			10.2	7	16.2	13.2	12.5	
	A	2	519.1	462	6	77.0			8.7	6	14.5	11.2	11.3	
	A	3	378.0	336	4	84.0			6.9	4	7.9	5.5	7.9	
	A	4	321.9	286	4	71.5			5.5	4	7.9	5.5	7.9	
	A	5	294.0	262	3	87.3			4.5	3	7.1	4.5	5.8	
	A	6	217.6	194	3	64.7			3.0	3	7.1	4.5	5.4	
	Sub-total		2,305.2	2,051	27	76.0	16.5	5	38.8	27	60.7	44.4	50.8	0.0
Jhatpat	B	1	371.1	330	4	82.5			3.6	4	9.1	5.0	8.4	
	B	2	232.9	207	3	69.0			2.2	3	6.9	4.5	5.8	
	B	3	277.1	247	3	82.3			3.2	3	6.9	4.5	5.8	
	B	4	319.6	284	4	71.0			4.0	4	9.1	6.7	7.7	
	B	5	432.0	384	5	76.8			4.6	5	14.9	10.4	5.0	
	B	6	304.0	271	3	90.3			3.7	3	6.5	4.2	4.4	
	B	7	307.2	273	3	91.0			4.2	3	6.3	4.2	4.4	
	B	8	304.8	271	3	90.3			4.1	3	6.3	4.2	4.4	
	Sub-total		2,548.7	2,267	28	81.0	21.5	7	29.6	28	66.0	43.7	45.9	10.6
Muhoad Pur	C	1	329.0	293	4	73.3			4.8	4	9.5	7.1	4.6	
	C	2	334.8	298	4	74.5			4.8	4	9.5	7.1	4.6	
	C	3	326.1	290	4	72.5			4.6	4	9.5	7.1	4.6	
	C	4	329.4	293	4	73.3			4.6	4	9.5	7.1	4.6	
	C	5	329.4	293	4	73.3			4.6	4	9.5	7.1	4.5	
	C	6	325.8	290	4	72.5			4.5	4	9.5	7.1	4.6	
	C	7	344.2	306	4	76.5			4.6	4	9.5	5.9	6.6	
	Sub-total		2,318.7	2,063	28	73.7	12.1	6	32.5	28	66.5	48.5	34.1	18.5
Bari	D	1	364.7	325	4	81.3			6.3	4	7.4	7.4	0.0	
	D	2	513.6	457	5	91.4			6.9	5	13.2	10.6	9.1	
	D	3	438.6	390	4	97.5			6.5	4	9.0	6.7	9.0	
	D	4	420.2	374	4	93.5			6.5	4	9.0	6.7	8.6	
	D	5	420.1	374	4	93.5			6.5	4	9.0	6.7	8.7	
	D	6	390.4	347	4	86.8			6.5	4	7.9	5.9	8.4	
	D	7	390.3	347	4	86.8			6.5	4	7.9	5.9	8.4	
	Sub-total		2,937.9	2,614	29	90.1	11.2	6	45.7	29	63.4	49.9	52.2	0.0

Table IV.4-9 Rotation Area and Quantities of On-farm Facilities (2)

Name of Distributary	Name of Minor	Name of Chak	GCA	CCA	Number of Rotation Area	Mean Rotation Area	Quantities							
							Minor	Outlet	M.W.C	Nakka	I.W.C	Farm Drain	Branch Drain	Main Drain
			ac	ac		ac	1,000ft		1,000ft		1,000ft	1,000ft	1,000ft	1,000ft
Ballan	E	1	667.2	593	7	84.7			12.5	7	21.3	21.3	7.9	
	E	2	480.0	427	5	85.4			6.1	5	14.1	11.1	5.1	
	E	3	278.4	248	3	82.7			4.2	3	5.9	4.0	5.1	
	E	4	288.0	256	3	85.3			4.2	3	5.9	4.0	5.1	
	E	5	297.6	265	3	88.3			4.2	3	5.9	4.0	5.1	
	Sub-total		2,011.2	1,789	21	85.2	17.2	4	31.2	21	53.1	44.4	28.3	0.0
Ballan	F	1	341.8	304	4	76.0			9.5	4	9.4	6.5	9.6	
	F	2	248.2	221	2	110.5			2.2	2	4.5	2.2	4.4	
	F	3	411.4	366	4	91.5			8.4	4	9.0	6.7	7.3	
	F	4	554.4	493	5	98.5			8.4	5	11.2	9.0	0.7	
	F	5	395.6	352	4	88.0			9.9	4	9.2	7.1	5.9	
	F	6	423.2	377	5	75.4			7.4	5	9.9	7.9	10.4	
	Sub-total		2,374.6	2,113	24	88.0	17.2	10	45.8	24	53.2	39.4	38.3	0.0
Ballan	G	1	420.8	375	5	75.0			7.4	5	9.9	7.9	10.3	
	G	2	496.0	441	5	88.2			7.4	5	9.9	7.9	11.9	
	G	3	579.6	516	6	86.0			9.2	6	11.9	9.9	13.5	
	G	4	600.8	535	6	89.2			11.3	6	11.9	9.9	11.7	
	G	5	441.6	393	4	98.3			5.5	4	11.7	7.5	10.2	
	G	6	455.2	405	6	67.5			10.6	6	13.3	0.0	12.7	
	Sub-total		2,994.0	2,665	32	83.3	8.3	5	51.4	32	68.6	43.1	70.3	0.0
	Total		17,490.3	15,562	189	82.3	104.0	43	275.0	189	431.5	313.4	319.9	29.1
Average in feet/acre							6.7		17.7		27.7	20.1	20.6	1.9

IV.5 PILOT PROJECT

IV.5.1 Design of Field Irrigation Facilities

1. Dimention of Design

a. Soil ------------------------------ Loam or Silty Loam

b. Apprication Efficiency and Conveyance Efficiency

Apprication Efficiency ----- 80 percent

Conveyance Efficiency ------ 90 percent

c. Irrigation Interval and Working Hour

Irrigation Interval -------- 5 days

Working Hour --------------- 8 hours/day

d. Water Requirement

From the result of field investigation which have been performed in Pakistan, vegetable and sugarcane were proposed as crops in this pilot farm, and water requirement were determined on the basis the consumptive use of water of the proposed crop.

1) Vegetable

Consumptive use of water per day	3.52"/10 days = 9 mm/day
Irrigation interval	5 days
Net water requirement	9 x 5 = 45 mm
Gross water requirement	45/(0.8 x 0.9) = 51 mm

2) Sugarcane

Consumptive use of water per day	2.85"/10 days = 7.3 mm/day
Irrigation interval	5 days
Net water requirement	7.3 x 5 = 36.5 mm
Gross water requirement	36.5/(0.8 x 0.9) = 51 mm

e. Shape of Pilot Farm

Refer to Fig.IV.5-1.

2. Irrigation System

Taking into account the topographical conditions and farm areas. following irrigation systems are adopted at the farm.

Sprinkler Irrigation (Fixed type)
- do - (Portable type)
Raingun Irrigation
Trincle Irrigation

a. Sprinkler Irrigation System (Fixed type)

1) Size of Lot

long side 275 ft (83.3m)
short side 64 " (19.4m)

2) Spray Intensity I

I = Gross water requirement/(Irrigation hours per day ÷ Movement distance per day)

≑ 63 ÷ (8 ÷ N)

N(time)	I(mm/hr)
1	7.9
2	15.8

3) Model of Sprinkler

Sprinkler of 7 numbers is set at a blanch pipe at an interval of 12m.

Capacity of sprinkler q

$q = I \times \ell_1 \times \ell_2 \div 60$

where, q : Capacity of sprinkler ℓ/mm
I : Spray intensity mm/hr
ℓ_1: Length of short side for a lot m
ℓ_2: Interval of sprinklers m

$q = 7.9 \times 20 \times 12 \div 60 = 31.6$ ℓ/min

Model of sprinkler	30B (3/16" x 3/32")
Max. water pressure	3.2 kg/cm^2
Available spray radius	30 m

Spray capacity	32.4 ℓ/min
Interval of sprinkler	12 m
Interval of branch pipe	20 m
Spray intensity	8.5 mm/hr
Net irrigation hours	7.4 hrs

b. Sprinkler Irrigation System (Portable type)

1) Size of Lot

long side 340 ft (103.0m)
short side 80 " (24.2m)

2) Design Condition

A branch pipe is moved at once during one day.
A firm is devided into 8 rotation blocks.

3) Model of Sprinkler

$q = 60.6$ ℓ/min
$I = q \times 60 \div (\ell_1 \times \ell_2)$
$= 60.6 \times 60 \div (24.2 \times 20.0) = 7.5$ mm/hr
H = Gross water requirement/I = 51/7.5 = 6.8 hr

Model of sprinkler	70 CW (1/4" x 1/8")
Max. water pressure	3.5 kg/cm^2
Available spray radius	125 ft (37.8m)
Spray capacity	60.6 ℓ/min
Interval of sprinkler	60 ft (20m)

c. Raingun Irrigation System

1) Size of Lot

long side 590 ft (179m)
short side 105 " (31.8m)

2) Design Condition

Spray intensity	I = 51 mm / 8 hrs = 6.4 mm/hr
Numbers of sprinkler	5 Nos.
Interval of sprinkler	ℓ_2 = 590/5 = 118 ft (35.8m)

Spray capacity $q = I \times \ell_1 \times \ell_2/60$
$= 6.4 \times 31.8 \times 35.8/60$
$= 121.4$ ℓ/min

3) Model of Sprinkler

Model	102 C
Spray radius	202 ft (61m)
Max. water pressure	3.5 kg/cm^2
Spray capacity	196.8 ℓ/min

Spray internsity I

$I = 60 \times 196.8 \div (31.8 \times 35.8) = 10.4$ mm/ha

Net irrigation hours H

$H = 51/10.4 = 4.9$ hrs

d. Trickle Irrigation System

1) Size of Lot

long side	170 ft (51.5m)
short side	45 " (13.6m)

2) Design Condition

Interval of tube	3 ft (0.92m)
Interval of emitter	3 " (0.92m)

Spray Capacity q

$q = 9.0 \div (0.9 \times 0.8 \times 0.92 \times 0.92) = 10.58$ ℓ/day

3) Model of Trinkle

Model	EMj10
Irrigation hour	2 hrs
Emittee	170'/3' = 57 emittees/tubu
Tube	45'/3' = 15 tubes/valve

Numbers of emittee for valve

57 x 15 = 855 emittees

Spray capacity 10.58/2 x 855/60 = 75.1 ℓ/min

Valve numbers of one rotation

8 hrs/2 hrs x 1 valve = 4 valves

Water pressure 2.5 kg/cm^2

3. Hydraulic Calculation

a. Branch Loss

1) Friction Loss

The irrigation loss of branch pipes can be calculated from Scobey formula.

$$Hf = 2.59 \times ks \times L \times V^{1.9}/1000 \times D^{1.1}$$

where, Hf : Friction loss head (m)
Ks : Coefficient of Scobey
L : Pipe length (m)
V : Velocity in the pipe (m/sec)
D : Pipe diameter (m)

2) Fixed Type of Sprinkler

$Hf < Pf$

where, Pf = Operation water pressure of sprinkler x 0.2

Allowable variaties of water pressure should be controlled within 20 percents of operation water pressures in order to control the variation of spray quantity lower than 10 percents.

$Pf = 3.2\ kg/cm^2 \times 0.2 = 0.64\ kg/cm^2$ ----- 6.4 m
$Hf = \Sigma hf = 4.85$ m (Refer to Table IV.5-1)
$Hv = 3.2 \times 1.10 = 3.52\ kg/cm^2$

Model of valve ----- Electromagnetic valve. EP-150-F (1.5")

3) Portable Type of Sprinkler

$Pf = 3.5\ kg/cm^2 \times 0.2 = 0.7\ kg/cm^2$ ----- 7.0 m
$Hf = \Sigma hf = 5.0$ m (Refer to Table IV.5-1)
$Hv = 3.2 \times 1.10 = 3.85\ kg/cm^2$

Model of valve ----- EP 200F (2")

4) Raingum

$Pf = 3.5\ kg/cm^2 \times 0.2 = 0.7\ kg/cm^2$ ----- 7.0 m
$Hf = \Sigma hf = 5.76$ m (Refer to Table IV.5-1)
$Hv = 3.5 \times 1.10 = 3.85\ kg/cm^2$

Model of valve ----- EAV 400M (4")

b. Branch Line

Hydraulic calculations of branch lines which are layed under the ground parallel with the long of side of each farm are performed for every irrigation type.

1) Fixed Type of Sprinkler

q = 32.4 ℓ/min x 7 seets x 2 blocks/60 sec = 7.56 ℓ/sec
Σhf = 2.67 m ----- 0.27 kg/cm^2 (Refer to Table IV.5-2)
Hp = Water pressure at the beginning point of branch pipe + Σhf

where, Hp : A water pressure required at the beginning point of branch lines.

Hp = Hv + Σhf = 3.52 + 0.27 = 3.79 kg/cm^2

2) Portable Type of Sprinkler

q = 60.6 ℓ/min x 5 seets x 2 blocks/60 sec = 10.10 ℓ/sec
Σhf = 3.2 m ----- 0.32 kg/cm^2 (Refer to Table IV.5-2)
Hp = 3.85 + 0.32 = 4.17 kg/cm^2

3) Raingun

q = 196.8 ℓ/min x 5 seets/60 sec = 16.4 ℓ/sec
Σhf = 2.26 m ----- 0.23 kg/cm^2 (Refer to Table IV.5-2)
Hp = 3.85 +0.23 = 4.08 kg/cm^2

4) Trickle

q = 75.4 ℓ/min x 2 blocks/60 sec = 2.51 ℓ/sec
Σhf = 2.88 m ----- 0.29 kg/cm^2 (Refer to Table IV.5-2)
Hp = 2.5 + 0.29 = 2.79 kg/cm^2

C. Main Pipe Line

The Required water pressure at the delivery of pumps and the diameter of main pipes connected between a pumping station and each branch line are calculated..

(Refer to Table IV.5-3 and Fig.IV.5-2)

4. Pump

a. Conbeyance System of Irrigation Water by Pumps

Three alternative plans have been studied regarding the method which conbeys irrigation water from a farm pond to farms by the pumps.

1) Plan - 1 (Direct Conbeyance)

Since open and close of valves and/or sprinklers to be set at the end point of pipes are directry linking to switch on and off of pumps, and when the pump frequently repeats start and stop, the motor connected to the pump will overheat by the load required at the start.

2) Plan - 2 (Distributing Tank)

There is no hills in the project area where are able to obtain the water pressure bigger than 4 kg/cm^2 (height = 40 m). In case of elevated water tanks, the height will be required more than 40 meters and this plan is uneconomical.

3) Plan - 3 (Surge Tank)

Tanks installed at the delivery side of pumps, and the variation of terminal water pressures is detected in the tank, and the pump operation is controlled by this detection.

From view points of topographical and economical conditions, the surge tank plan was adopted for the irrigation farm.

b. Model of Pump

Pump	high head pump
Delivery capacity	0.35 ~ 200 m^3/min
Total head	9 ~ 300 m
Motor	220 V 50 Hz

c. Numbers of Pump

Generally, pumps to be used for irrigation have low priolity compared with that of for water supply and industrial purposes. No spare pumps therefore are proposed for this project. However, taking into account the pump troubles, pump numbers are determined at more than two (2) numbers.

1) Sprinkler and Raingun Irrigation

Fixed type (for vegitable)	H = 38.4 m	Q = 37.8 ℓ/sec
-do- (for sugarcane)	= 41.9 "	= 49.2 "
Portable type (-do-)	42.5 "	= 40.4 "
Total		127.4 ℓ/sec
Numbers of pump		
Pump diameter	3 Nos	
Power of motor	φ = 150 mm	
	55 kw	

2) Trickle Irrigation

First branch line	H = 31.1 m	Q = 20.1 ℓ/sec
Second branch line	= 31.3 "	= 17.6 "
Numbers of pump	2 Nos	
Pump diameter	φ = 100 m	
Power of motor	15 kw	

5. Farm Pond

a. Capacity

The capacity of farm pond was determined on the basis the follwing conditions.

1) Max. water requirement per day during irrigation season.

2) Water capacity dissipated at one day.

3) Water capacity of one month supplied for offices.

From the result of studies mentioned above, the farm pond capacity is determined at 4,100 cu.m.

6. Pipe

The pipe to be utilized for this irrigation is selected so as to satisfy the follwing conditions, and determined at vinyl pipe.

1) Lightweight pipe

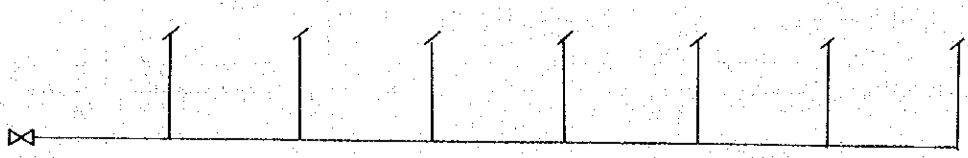
2) Easy construction

3) Joins are flexible

4) Chiep cost

Table IV.5-1 Hydraulic Calculation for Branch Pipe

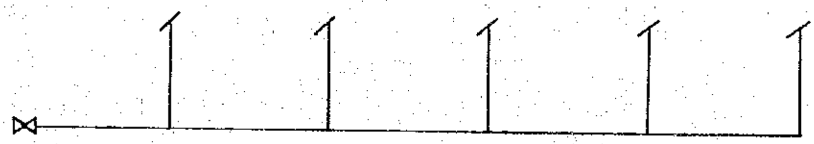
1) Sprinkler (Fixed Type)



L (ft)	19	39	39	39	39	39	39
L (m)	5.8	11.8	11.8	11.8	11.8	11.8	11.8
q(ℓ/s)	3.78	3.24	2.70	2.16	1.62	1.08	0.54
D (mm)	50	50	40	40	40	40	40
V(m/s)	1.92	1.65	2.15	1.72	1.29	0.86	1.43
hf(m)	0.91	0.68	1.44	0.95	0.55	0.25	0.07

$$Hf = \Sigma\ hf = 4.85m$$

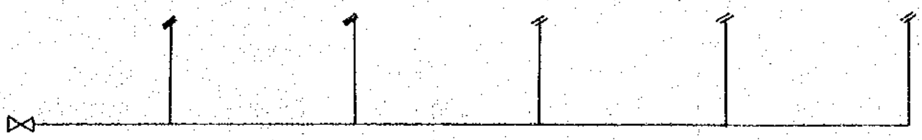
2) Sprinkler (Portable Type)



L (ft)	33	66	66	66	66
L (m)	10	20	20	20	20
q(ℓ/s)	5.05	4.04	3.03	2.02	1.01
D (mm)	50	50	50	40	40
V(m/s)	2.57	2.06	1.54	1.61	0.80
hf (m)	1.34	1.77	1.01	1.41	0.37

$$Hf = \Sigma\ hf = 5.90m$$

3) Raingun

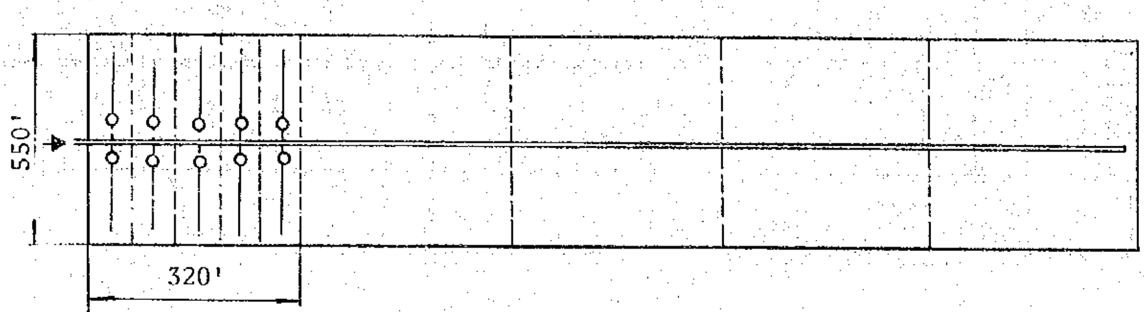


L (ft)	59	118	118	118	118
L (m)	17.9	35.8	35.8	35.8	35.8
Q(ℓ/s)	16.4	13.12	9.84	6.56	3.28
D (mm)	100	100	75	75	65
V(m/s)	2.05	1.67	2.22	1.49	0.99
hf (m)	0.76	0.99	2.33	1.09	0.59

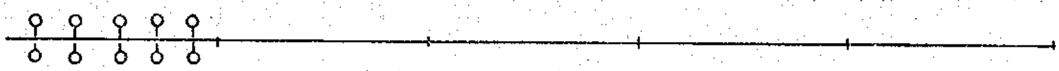
$$Hf = \Sigma hf = 5.76m$$

Table IV.5-2 Hydraulic Calculation for Branch Line

1) Sprinkler (Fixed Type)



one rotation block 275' x 320'

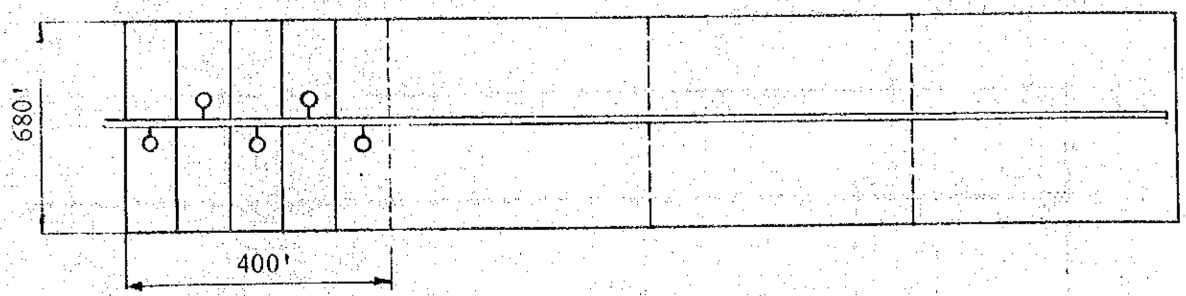


L (ft)	320	320	320	320	320
L (m)	97	97	97	97	97
Q(ℓ/s)	37.8	30.24	22.68	15.12	7.56
D (mm)	200	200	200	150	100
V(m/s)	1.20	0.96	0.72	0.86	0.96
I	0.006	0.004	0.002	0.004	0.009
hf (m)	0.64	0.43	0.21	0.43	0.96

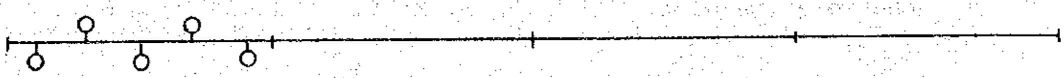
where $I = 10.666 \cdot C^{-1.85} \cdot D^{-4.87} \cdot Q^{1.85}$ $C = 150$

$Hf = \Sigma hf = 2.76m$

2) Sprinkler (Portable Type)



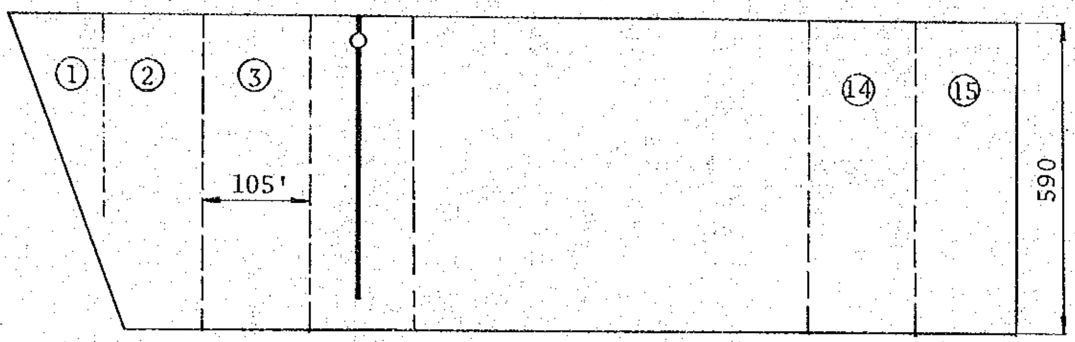
one rotation block 340' x 400'



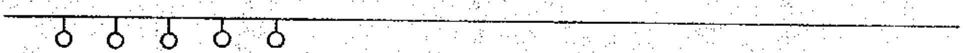
L (ft)	400	400	400	400
L (m)	121.2	121.2	121.2	121.2
Q(ℓ/s)	40.4	30.3	20.2	10.1
D (mm)	200	200	150	125
V(m/s)	1.29	0.96	1.14	0.82
I	0.007	0.004	0.008	0.005
hf (m)	0.93	0.53	1.07	0.67

$Hf = \Sigma hf = 3.20m$

3) Raingun



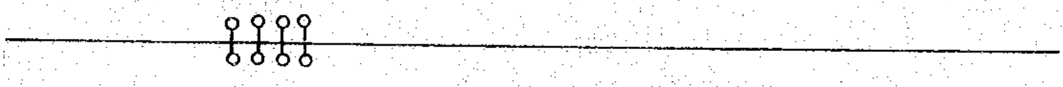
one rotation block 105' x 5 x 590'



L (ft)	525	525	525
L (m)	159	159	159
Q(ℓ/s)	49.2	32.8	16.4
D (mm)	250	200	150
V(m/s)	1.00	1.04	0.93
I	0.003	0.005	0.005
hf (m)	0.52	0.87	0.87

$Hf = \Sigma hf = 2.26m$

4) Trickle



L (ft)	180	180	180	180	180	180	180	180
L (m)	54.5	54.5	54.5	54.5	54.5	54.5	54.5	54.5
Q(ℓ/s)	20.08	17.57	15.06	12.55	10.04	7.53	5.02	2.51
D (mm)	150	150	150	125	125	100	100	75
V(m/s)	1.14	0.99	0.85	1.03	0.82	0.96	0.64	0.57
I	0.007	0.006	0.004	0.008	0.005	0.009	0.004	0.005
hf (m)	0.42	0.36	0.24	0.48	0.30	0.54	0.24	0.30

$Hf = \Sigma hf = 2.88m$

Table IV.5-3 Required Water Pressure and Pipe Diameter

No.	L(feet)	Q(ℓ/s)	D (mm)	V	I	hf (m)
1 - 1	75 (229)	37.8	200	1.20	0.006	0.14
- 2	50 (152)	127.4	300	1.80	0.008	0.12
- 3	540 (1,646)	89.6	300	1.27	0.004	0.66
- 4	340 (1,036)	49.2	250	1.00	0.003	0.31
2 - 1	1,735 (5,288)	37.7	200	1.20	0.006	3.17
- 2	360 (1,097)	17.6	150	1.00	0.006	0.66

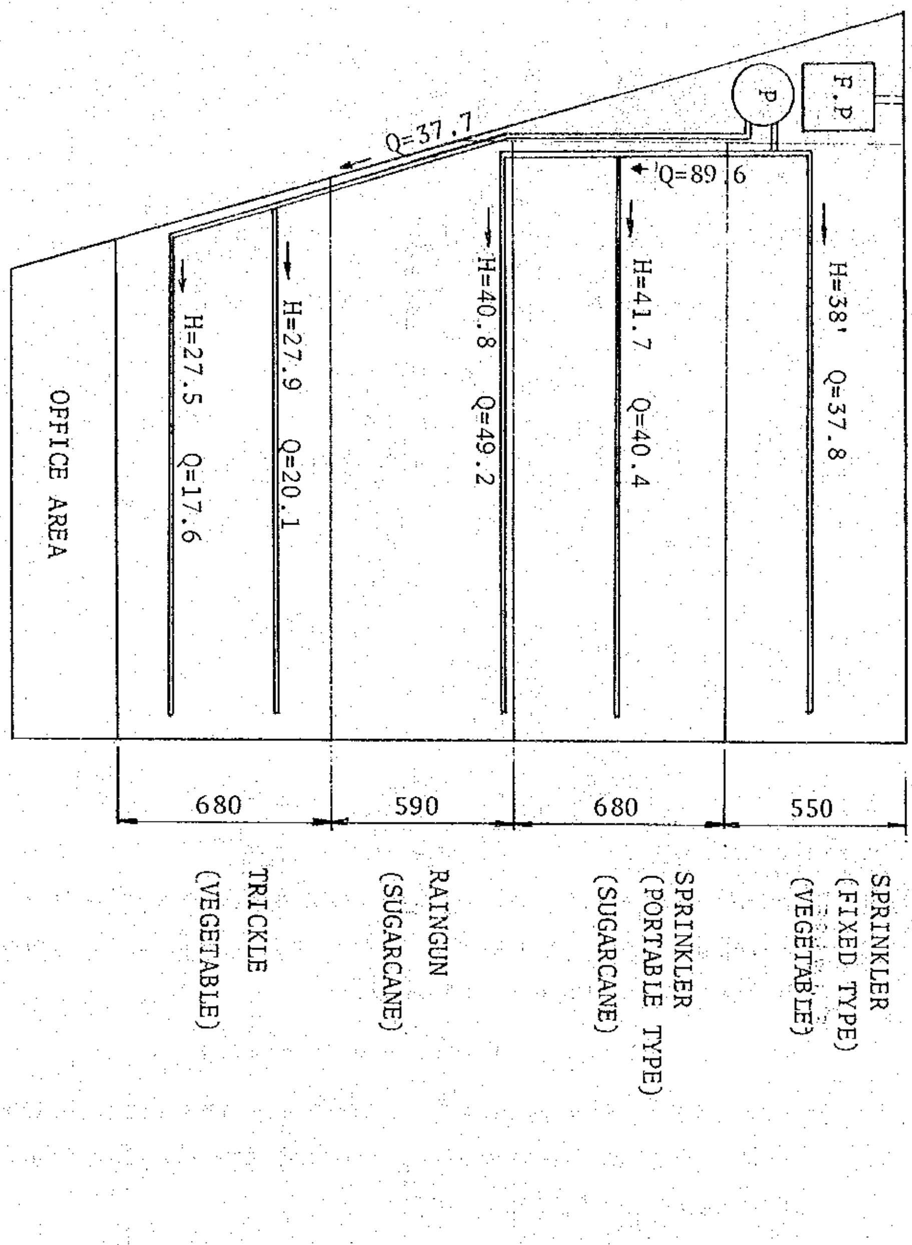
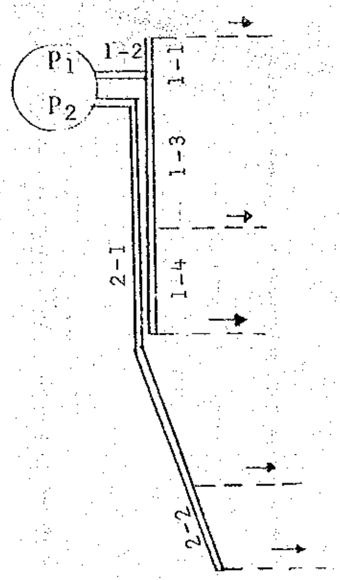


Fig. IV.5-1 Shape of Pilot Farm

Fig. IV.5-2 Main Pipe Line



Note: 1. Pumps of symbol mark P1 are used for the sprinklers and raingun, and the water pressure required at the pumping station is estimated as follows: -

$$P1 = 41.7 + 0.66 + 0.12 = 42.48 \text{ m}$$

2. Pumps of symbol mark P2 are used for the trickle irrigation, and the water pressure required at the pumping station is estimated as follows: -

$$P2 = 27.5 + 3.17 + 0.66 = 31.33 \text{ m}$$