No.

MALAYSIA-JAPAN TECHNICAL COOPERATION

PROPOSED PLAN AND DESIGN FOR PILOT FARM NO. 3

TRAINING CENTRE, KOTA BHARU,
KELANTAN, MALAYSIA

JAPAN INTERNATIONAL COOPERATION AGENCY

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PROPOSED PLANNING AND DESIGN FOR
PILOT FARM NO. 3

March, 1983

Shin IMAI
(Japanese Water Management Expert)

Ir. Hidayah Hj. Abdullah
(Senior Engineer)

Mansor Adabi Ahmad
(Technical Assistant)

National Water Management Training Center,
Drainage and Irrigation Department,
Kota Bharu, Kelantan,
MALAYSIA

PREFACE

The National Water Management Training Centre has four Pilot Farms. The main functions of the Pilot Farms are to apply and improve the techniques of rice cultivation and water management which are obtained through proper water management practised at Demonstration Farm in the Centre, and these improved techniques should be extended to the farmers for adoption. The Pilot Farms also provide on-the-job training for technical staff of D.I.D. or other agencies whose work involved with rice cultivation and water management.

The design criteria of the facilities to be constructed on each Pilot Farm, such as density of canals and ditches, construction materials of canals, are different from each other, especially, land consolidation technique is to be adopted in the Pilot Farm No. 3. These differences allow us to implement various types of water management practices, such as from lot-to-lot irrigation to a more advanced technique with independent facilities.

When I consider the background of agriculture in the country, I found that: 1) Income and living standard of the farmers are not high and these must be improved following planning in the 4th. Malaysian Plan. 2) Compared to other advanced regions in the country such as in MADA, Tanjong Karang and Krian, the farm lands located in the East side of Peninsular Malaysia is very small and complicated, therefore, in order to improved the farming practices in the area, land consolidation project is inevitable. 3) Actually, KADA has already started such a project on a small scale, 4) And more such project would be executed in this area in the near future.

Accordingly, the land consolidation project in the PIlot Farm will enhance good progress to the agricultural development in the country.

Mr. S. IMAI, one of the Japanese experts from the Centre, has tackled the design of this land consolidation project together with Ir. HIDAYAH and Mr. Mansor. Although this project is located in the double cropping area of KADA, in most of the area, no crop has been planted for a long time because the elevation of the area is either too high or too low to convey proper irrigation water. It is therefore, considerably difficult to disign the irrigation facilities. As a result, the soil-moving method is to be applied on 7.7 ha. out of a total area of 11.2 ha. In addition the shape of the area is long and narrow, which causes the construction cost to be higher. Construction cost would have been cheaper if it is located in a wider and flatter area in general.

I hope that the farmers will cooperatively carry out their farming works and achieve higher land and labour productivities such that more consolidation works will extended in the near future to improve such complicated farm lands. Lastly, I hope that this will contribute to the development and stability of agriculture in Malaysia.

1st. of March, 1983

Ta. Yano

T. YANO

(Japanese Team Leader)

CONTENTS

2/-

3/-

4/-

APPENDIX - 1 LIST OF TABLES

2/-

APPENDIX - 2

LIST OF FIGURES

2/-

3/-

BACK GROUND.

To render necessary training on water management techniques to various technical staff of DID and other relevant government agencies, four pilot farms are to be implemented by the Centre. These pilot farms provide improved on-farm facilities such as irrigation canals, drainage ditches, farm roads and their related structures but water distribution within the block remains as from lot to lot depending on the topography. However, since the proposed on-farm development is not of the intensive type, land levelling and farm lots rearrangement(replotting) is not included in these pilot farms.

For further agricultural development in the country in order to achieve higher output for the farmers and to be in line with the KADA II Project, the centre proposes to implement an additional pilot farm whereby land levelling and rearrangement of farm-lots would be the primary objective. Furthermore, the farmers in the proposed pilot farm area have requested for additional on-farm facilities, thus indicating that they are in favour of the project.

With the independent on-farm facilities to individual farmlots, the proposed pilot farm would set an example for the practice of water management techniques at the on-farm level to improve cultivation and possibly raise production through yield increase. Furthermore the proposed pilot farm would also provide an ideal farm for training and demonstration purposes.

Description of the area

The area is located on the south western part of Kota Bharu about 11 km from the town in the mukim of Kadok. The area of padi land is approximately 11.2 hectares and is situated in Kampong Seberang Lating. The area is bounded by the Sungai Mulong in the north, whilst the Limbat Main Canal is on the south eastern side. The general topography of the area is sloping from south to north towards Sungai Mulong.

Existing conditions

The area is situated in one of the irrigation units of the Kemubu Irrigation Project which has been gazetted for double cropping. The whole irrigation unit of about 112 hactares is supposed to be irrigated by the quartenary offtake P23 along the Limbat Main Canal. Unfortunately most of the area is not able to get a water because of the problem of irrigation canal. After some improvement work was done by the farmers concerned in order to get the irrigation water, about 3.6 hactares are now double-cropped and the yield is about 2.0 T/ha. This is lower than the average yield obtained in the Kemubu area of about 4.1 T/ha whilst in Pasir Mas, the yield is about 3.7 T/ha.

As for the area at the end of the irrigation unit which has been deprived of irrigation water since the Kemubu Project was implemented, the land is mostly left idle.

Sungai Mulong which is flowing on the north side of the area serves as the drainage for the irrigation unit.

As for the transportation purpose, there is a narrow path leading to the padi area from the kampong laterite road.

In other words, although this area is situated in the Kemubu irrigation unit, there is no on-farm facilities for the farmers to do double cropping.

I PLANNING

1) Definition of Land Consolidation Project

1)-1 What is a Land Consolidation?

Land Consolidation Project is defined as block reformation project which change the block size-shape and the quality of the land and/or the combined project with land reclamation project or the necessity project for improvement of land for agricultural use or the farm land conservation, and those following extremely connected project in an agricultural land, such as irrigation and drainage project, underdrainage project and farm road readjustment project are able to enforce simultaneously in spite of the supposed area been located outside of the land readjustment area.

Consequently, this project is identified as follows:

1. Ground levelling works in order to change the block size-shape and it's quality centering around the block reformation project.

2. Involving all of the necessary improvement works for the agricultural land such as irrigation and drainage works, underdrainage works, farm road readjustment works and soil layer improvement.

3. And moreover, like a disposal of replotting which has been operated by itself before owing to make a farm land grouping for the scattered agricultural land able to enforce simultaneously under this Land Consolidation Project Plan.

Therefore, the work of "Land Consolidation Project" means the integrated readjustment works for the fundamental agricultural land.

1)-2 *Object and Signficance of Land Consolidation*

The object of the Land Consolidation Project is said as follows:-

1. To improve the size-shape and the quality of land as fundamental production area for agricultural machinery.

2. To readjust of field facilities such as irrigation and drainage, farm road, underdrain and so on.

3. To group the farm land.

These matters would be taken into account and afterwards the agricultural land should be changed to high productivity land through the effective usage of agricultural machinery and rationalized water management.

Excess labor force which is produced as the result of the various works as above-mentioned is able to use effectively in the multi-purpose farm land and selective increasing of agricultural production is taken as a countermeasure against tha various changes of domestic food demand.

In fact, the following effects will be expected.

1. Decrease the labor force in farming

   It goes without saying that the main effect of the Land Consolidation Project is to increase the productivity of labor, that is to say the effect of decreasing the labor force, and after completing this project, the labor hours in farming will be shorten a lot due to the promotion of high performance of agricultural machinery.

2. Stabilized food supply by the consolidated multi-purpose farm land.

   The multi-purpose farm land which can use both ways as paddy field and for upland field is able to be to provide by readjustment of each field conditions, so that the fundamental farming field to produce various crops in order to satisfy the food demand will be established, moreover the stability/intergrated food supply is to be available as the result of the multi-purpose and high utilization of paddy field.

3. Enlargement of farm size by the cooperation of farming activities.

   The effective utilization of agricultural machinery is expected as the result of the land readjustment works therefore through the enlargement of the farm size it will be easier to plan the co-operative work and contract farming.

4. Multiple use of excess labor force.

   The excess labor force which is produced as the result of the intensive farming is to be useful for another multiple agricultural activities such as horticulture, animal industry and it will be contribute to the selective increasing of agricultural production, in addition it is expected to make the chance to work in another industry and finally the income of the farmers will be expected to increase.

5. Improvement of the rural enviroment

   Owing to the reproductive disposal of replotting method, it can be provide the new land for the agricultural modernized facilities, rural parks for the public use, and not only it release farmers from their severely works but also their standard of living and social life are increased with this project.

In another words, it is extremely expected to improve the rural environment and economical standard.

1)-3 Role of Land Consolidated in The Agricultural Policy

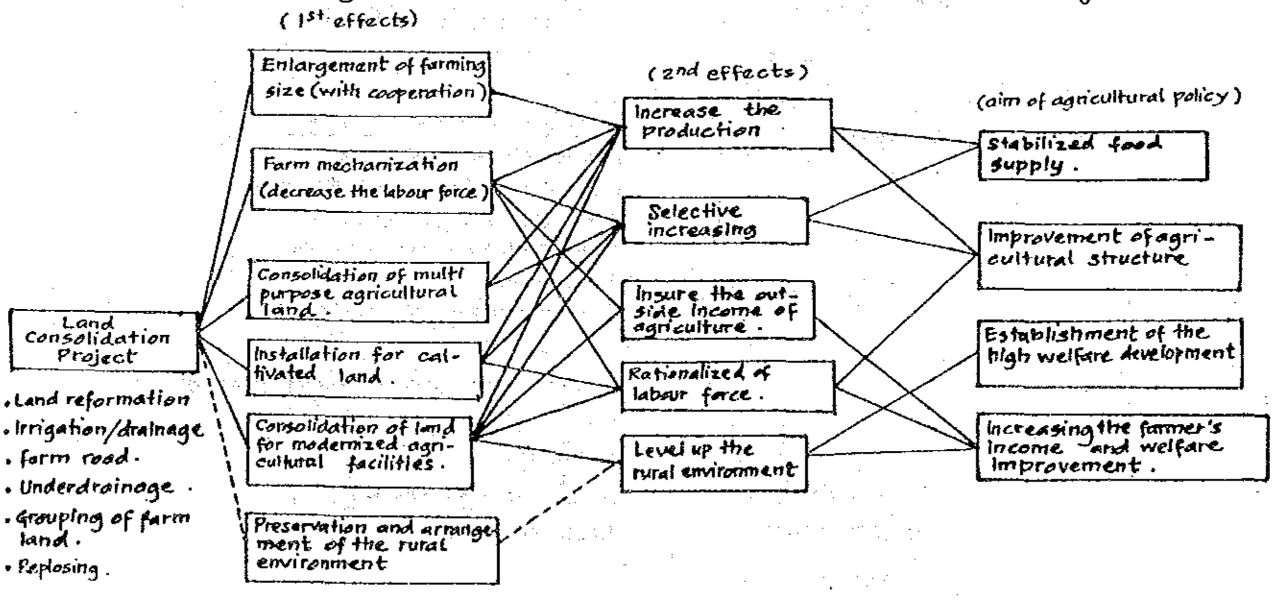
Aim and it's role is to be as follows.

The aim of the agricultural policy in the future is supposed to establish the stability for food supply, increasing the productivity of agriculture, selective increasing of agricultural production and improve the agricultural infrastructures.

In order to establish these aims it is necessary to readjust the paddy field to be able to set up the high efficient agriculture and the high welfare rural development with multi purpose farm-land.

In this point of view, the role of the Land Consolidation owing to enforce these aims are shown as shown in Fig-1

Fig-1 Role of the Land Consolidation Project



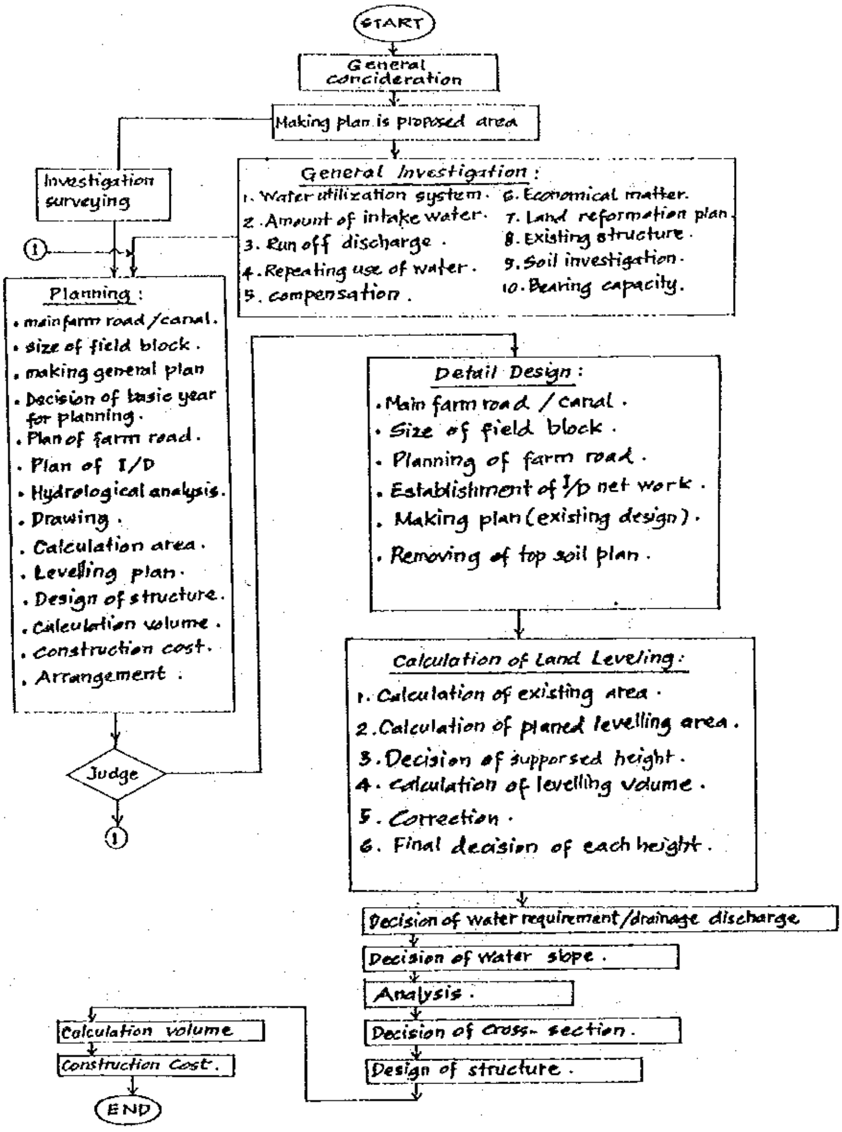
2) How to carry out the Land Consolidation

The Land Consolidation scheme is including not only irrigation/drainage, farm road and underground drainage construction but also plane construction so that it is quite necessary to connect between the point and line works and plane works at the same time.

Moreover, the construction period reached almost one year as usual therefore the construction schedule should be planned not to obstruct another existing field facilities thus the well considered temporary work is to be necessary.

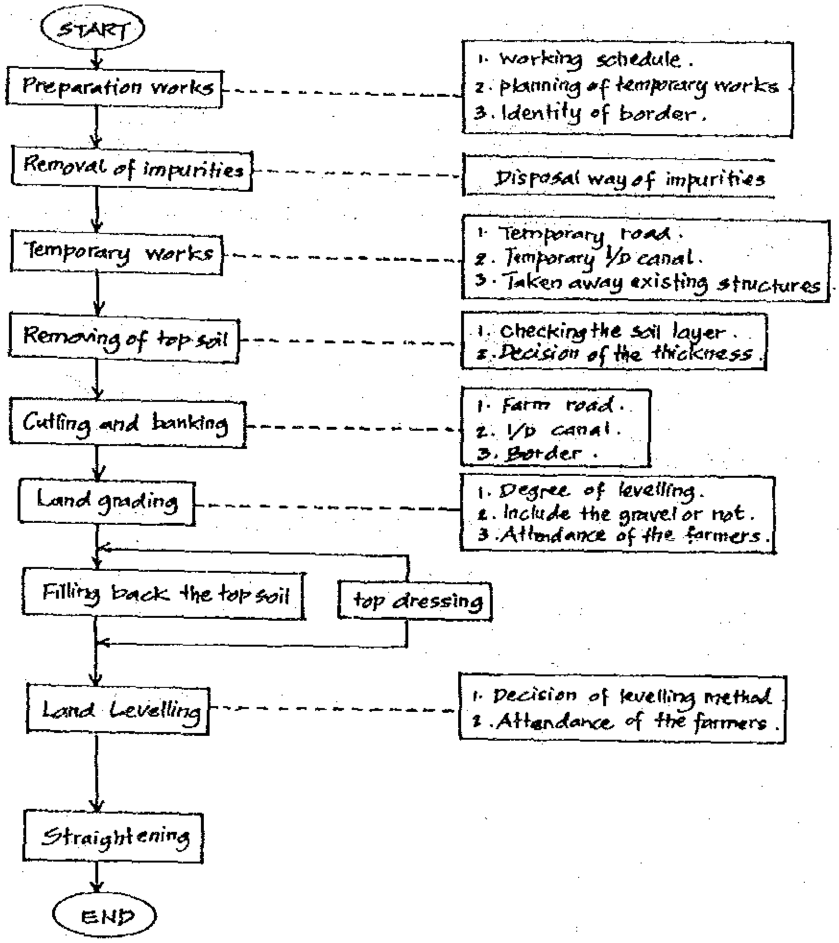
The expected flow chart is shown as in Fig-2

Fig-2 Flow chart for planning



And also the standard construction schedule is shown as in Fig-3

Fig-3 Standard construction schedule.



3) Study on the existing condition

3)-1 Ownership and area

Total area in this project is decided as follows

1. total area by C.P $A_1 = 111,991\ m^2$.

2. total area by planimeter $A_2 = 111,835\ m^2$.

As an example in Japan long time ago it is said that the registered area and on the ground area is different because the people is apt to report his area rather smaller than the actual area in order to avoid the land tax.

These difference of the area is called "NAWA NOBI". In this area the "NAWA NOBI" rate is to be 99.9 percent so that it can be said the land in this area is governed properly, thus the C.P area is adopted into the benefitted area.

The total number of the ownership is 42 within 44 field lot.

In fact, the 44 field lot is sub-devided into 98 sub-field-lot, therefore the area average sub-devided lot size is to be 1,143 m2.

The ownership with his/her name is arranged as in Tab-1.

tab-1 Ownership and area/elevation.

Ownership NO.	Reference NO.	Name	Lot NO.	C.P Area (ha)	Total C.P Area (ha)	E.L (m)
1	1	CHE SEMAN B. PUTEH	400	0.1822		7.085
	2	"	413	0.2267	0.4089	5.612
2	3	SAFAIAN B. IBRAHIM	312	0.3724		7.258
	4	"	379	0.1558	0.5282	7.114
3	5	FATIMAH BTE. DAUD	380	0.1417		7.046
	6	"	416	0.5141	0.6558	6.306
4	7	MERIAM BTE. SAADU	321	0.0810		5.892
	8	"	1592	0.2247	0.3057	7.106
5	8-i	LIMAH BTE. DAUD	1592	0.2247	0.2247	7.059
6	9	MAHMUD B. CHE HARUN	1590	0.1963	0.1963	6.945
7	9-i	MINAH B. STOPA	1590	0.1963	0.1963	7.319
8	10	MOHAMAD B. ABDULLAH	448	0.2745		5.932
	11	"	447	0.0587	0.3332	6.614
9	12	MUDA B. SENIK	418	0.2064		5.864
	13	"	403	0.1660	0.3724	6.578
10	14	WAN KALTHOM BTE. W.YUSOF	417	0.1063	0.1063	6.108
11	14-i	MEK NOR BTE YUSOFF	417	0.1063		5.893
	15	"	414	0.1194	0.2257	5.201

Ownership NO.	Reference NO.	Name	Lot NO.	C.P Area (ha)	Total C.P Area (ha)	E.L (m)
12	16	SEPIAH BTE MAT AMIN	320	0.1316		5.984
	17	"	401	0.2854	0.4170	6.780
13	18	MOHD. NOR B. MASHOR	1569	0.0374		6.806
	19	"	1570	0.2226	0.2600	6.663
14	20	ABDULLAH B. SALLEH	378	0.2834		7.138
	21	"	399	0.0951	0.3785	7.104
15	22	BIDAH BTE ALI	398	0.3765	0.3765	6.859
16	23	MEK NAH BTE ISMAIL	412	0.2793	0.2793	6.078
17	24	SALLEH B. IDRIS	1533	0.3242	0.3242	7.305
Sub-total				5.5890	5.5890	

Ownership NO.	Reference NO.	Name	Lot NO.	C.P Area (ha)	Total C.P Area (ha)	E.L (m)
18	25	MAT ALI B. JEMAL	317	0.1336	0.1336	6.418
19	26	AWANG B. SAADU	1591	0.2429	0.2429	7.057
20	27	LIMAH BTE YAAKUB	410	0.1619	0.1619	6.358
21	28	AWANG NOR B. SALLEH	314	0.2780	0.2780	6.954
22	28-i	MUNAH BTE AWAN NOR	314	0.5020	0.5020	6.442
23	29	RAHIMAH BTE SALLEH	1534	0.6315	0.6315	7.077
24	30	AIDA BTE ABDULLAH	450	0.1291	0.1291	5.770
25	31	MEK JAH BTE DIAH	449	0.0648	0.0648	5.724
26	32	FATIMAH@ MEK JAH BTE DOLLAH	409	0.1619	0.1619	6.451
27	33	MEK EMBONG BTE YAAKUB	316	0.1194	0.1194	6.682
28	34	SAPIAH BTE YUSUF	315	0.1397	0.1397	7.019
29	35	SITI HABESAH BTE AWANG HIM	405	0.3259	0.3259	6.765
30	36	MEK NOH BTE SENIK	381	0.1771	0.1771	7.143
31	36-i	MAT DIAH B. AWANG BESAR	381	0.1771	0.1771	6.957
32	37	FATIMAH BTE MAT DIAH	382	0.2550	0.2550	7.170
33	38	WAN HASSAN B. W.MD.AMIN	415	0.0753	0.0753	6.169
34	38-i	KHATIJAH BTE JUSOH	415	0.1506	0.1506	6.364
35	38-ii	FATIMAH BTE JUSOH	415	0.1506	0.1506	6.437

Ownership NO.	Reference NO.	Name	Lot NO.	C.P Area (ha)	Total C.P Area (ha)	E.L (m)
36	39	YAAKUB B. ABDULLAH	1568	0.8723	0.8723	6.666
37	40	AWANG B. MAJID	397	0.0870	0.0870	6.856
38	41	MAJID B. OMAR	323	0.1750	0.1750	6.729
39	42	MERIAM BTE AWANG KECHIK	395	0.0769	0.0769	8.383
40	43	MOHD. B. ALI	419	0.1215	0.1215	5.723
41	44	MAT DIAH B. AWANG BESAR	328	0.2005	0.2005	7.091
42	44-1	MINAH BTE DERAMAN	328	0.2005	0.2005	7.067
Sub-total				5.6101	5.6101	
Total				11.1991	11.1991	

3)-2 Statistical analysis

Based on the layout plan of the Pilot Farm, the geographical characteristics of the area should be studied and understand, where by altitude and area analysis are very important aspects to be considered for planning and design of the water management facilities as well as the agro-solio and other fudamental studies.

Accordingly, this paper gives an account of the altitude and area analysis using a statistical method.

1) Existing altitude analysis

Each field lot is classified in accordance with the elevation, so that the parameter such as frequency (f), mean (X), sum of squares of deviation (Sx), variance (V) and standard deviation ( ) can be determined as follows.

Tab-2 Statistical calculation of altitude.

Range of E.L. (m)	Centre of Range (x) , (m)	Frequency (f)	$(U) = \frac{X' - 6.75}{0.25}$	(f).(U)	$(f).(U)^2$
5.00 - 5.25	5.125	1	- 6.5	- 6.5	42.25
5.25 - 5.50	5.375	0	- 5.5	0.0	0.00
5.50 - 5.75	5.625	3	- 4.5	-13.5	60.75
5.75 - 6.00	5.875	6	- 3.5	-21.0	73.50
6.00 - 6.25	6.125	3	- 2.5	- 7.5	18.75
6.25 - 6.50	6.375	7	- 1.5	-10.5	15.75
6.50 - 6.75	6.625	6	- 0.5	- 3.0	1.50
6.75 - 7.00	6.875	8	0.5	4.0	2.00
7.00 - 7.25	7.125	14	1.5	21.0	31.50
7.25 - 7.50	7.375	3	2.5	7.5	18.75
7.50 - 7.75	7.625	0	3.5	0.0	0.00
7.75 - 8.00	7.875	0	4.5	0.0	0.00
8.00 - 8.25	8.125	0	5.5	0.0	0.00
8.25 - 8.50	8.375	1	6.5	6.5	42.25
total		52		-23	307

1. means $(\bar{X})$

$$\bar{X} = 0.25 \cdot \bar{U} + 6.75$$

$$\text{where} \quad U = \frac{\Sigma(f) \cdot (U)}{\Sigma(f)} = \frac{-23}{52} = -0.442$$

$$\therefore \bar{X} = 0.25 \times (-0.442) + 6.75$$
$$= 6.639 \text{ m.}$$

2. Sum of squares of deviation (Sx)

$$Sx = 0.25^2 \cdot Su$$

$$\text{where, } Su = \Sigma(f) \cdot (U)^2 - \frac{[\Sigma(f) \cdot (U)]^2}{\Sigma(f)}$$

$$= 307 - \frac{(-23)^2}{52}$$

$$= 296.827.$$

$$\therefore Sx = 0.25^2 \times 296.827$$

$$= 18.552.$$

3. variance (V)

$$V = \frac{Sx}{\Sigma(f)} = \frac{18.552}{52} = 0.357.$$

4. Standard deviation (σ)

$$\sigma = \sqrt{V}$$

$$= \sqrt{0.357} = 0.597$$

Therefore,

a) lower limit $= \bar{X} - 3\sigma$

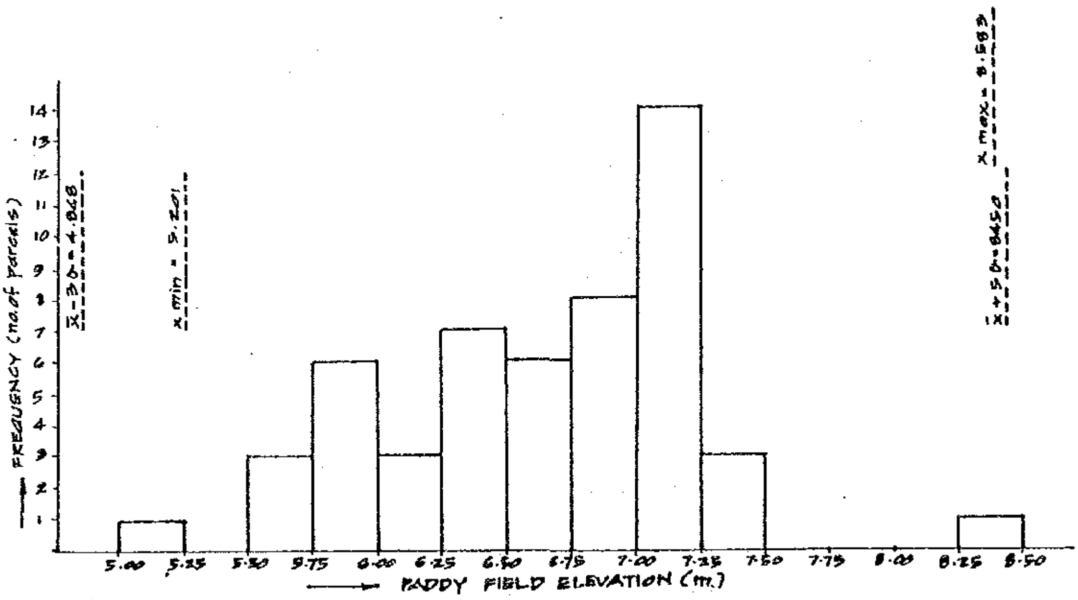
$$= 6.639 - 3 \times 0.597 = 4.848 \text{ m}$$

b) upper limit $= \bar{X} + 3\sigma$

$$= 6.639 + 3 \times 0.597 = 8.430 \text{ m}$$

Fig-4 shows it's histogram

Fig-4: HISTOGRAM OF ALTITUDE



2) Existing area analysis

Each field lot area is classified as well as the altitude analysis.

Tab-3 Statistical calculation of area (C.P area)

Range of Area (ha)	Centre of Range (x) (ha)	Frequency (f)	$(U) = \frac{x - 0.45}{0.1}$	(f).(U)	$(f).(U)^2$
0.0 - 0.1	0.05	4	-4	-16	64
0.1 - 0.2	0.15	15	-3	-45	135
0.2 - 0.3	0.25	9	-2	-18	36
0.3 - 0.4	0.35	7	-1	- 7	7
0.4 - 0.5	0.45	2	0	0	0
0.5 - 0.6	0.55	2	1	2	2
0.6 - 0.7	0.65	2	2	4	8
0.7 - 0.8	0.75	0	3	0	0
0.8 - 0.9	0.85	1	4	4	16
Total		42		-76	268

1. means $(\bar{X})$

$$\bar{U} = \frac{-76}{42} = -1.809$$

$$\bar{X} = 0.1 \times (-1.809) + 0.45$$

$$=0.269$$

2. Sum of square of deviation (Sx)

$$Su = 268 - \frac{(-76)^2}{42} = 130.476$$

$$Sx = 0.1^2 \times 130.476 = 1.305$$

3. variance (V)

$$V = \frac{1.305}{42} = 0.031$$

4. Standard deviation (σ)

$$\sigma = \sqrt{0.031} = 0.176$$

Therefore,

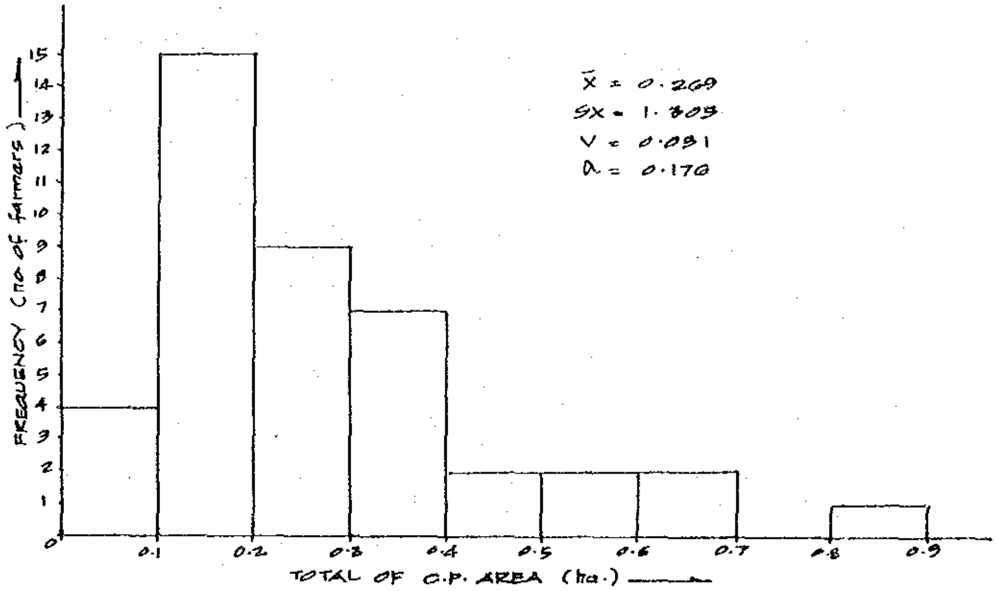
a) lower limit $= \bar{X} - 3\sigma$

$= -0.259$

b) upper limit $= \bar{X} + 3\sigma$

$= 0.797$

Fig-5 Histogram of area (C.P area)



3) Analysis

Take into consideration about the Full Supply Water Level (F.S.L) at P23L, that is determined as 7.76 m, then almost all the area could be irrigated without any new facilities, however, only about 3.5 ha is irrigated now,

It can be found that something wrong and the detail analysis is going to carry out in the design on external canal.

On the other hand, the average area in this project is determined as 0.269 ha.

This value is very small compared to the average area in KADA which is about 0.7 ha. Maybe the farmers owned other lots outside the project area. Anyhow the average area is small if compared to the standard lot size, that is 0.3 ha.

In addition due to the irregular shape of the project area as the whole, it is unavoidable that the density of the facilities is increased thus construction cost is going to high as the result.

However, as explain on the comparative design of the representative area in KADA if the benefitted area is chosen from such area, the density of the on farm facilities is to be very small and construction cost also become cheaper.

Thus, the purpose of this land consolidation is not extend into such kind of narrow area but for water management purpose i.e how it can be improved due this land consolidation project.

3)-3 Existing land possession

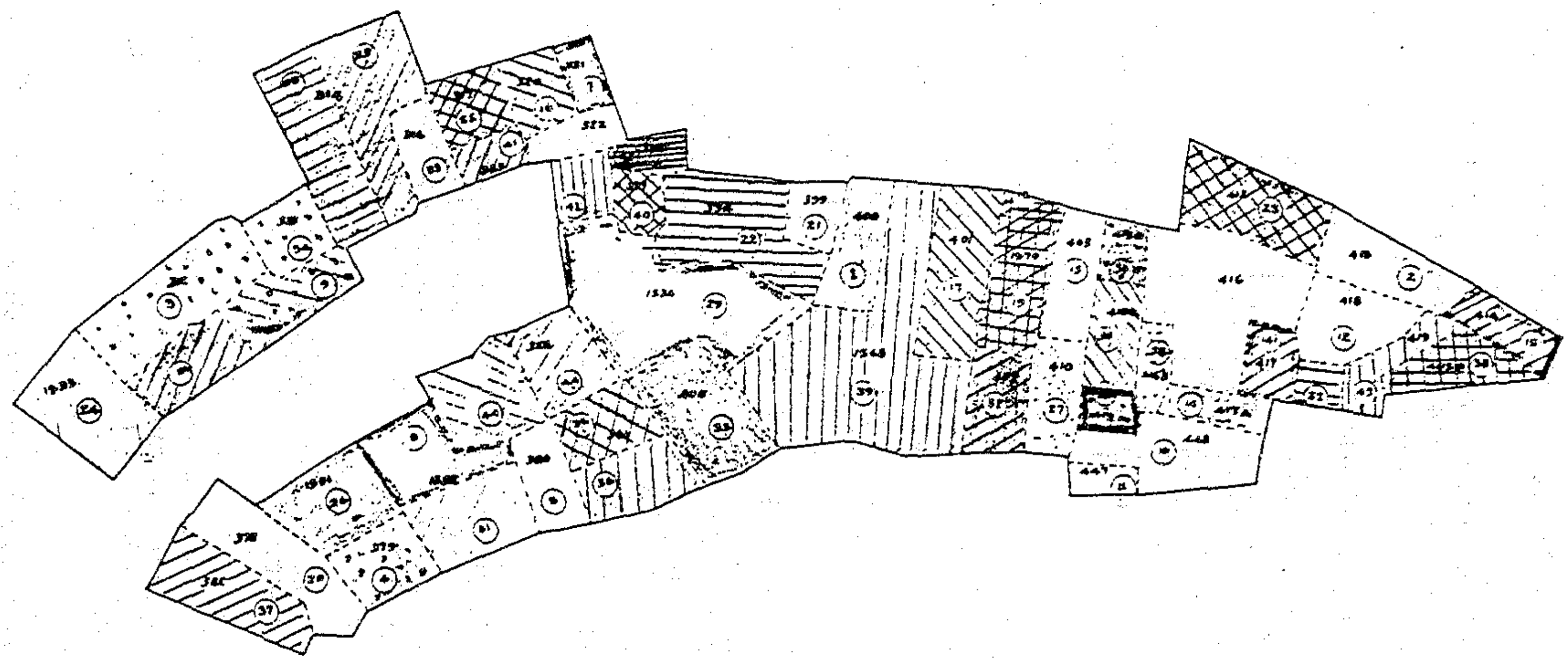
One of the main purpose of the Land Consolidation is the grouping of the land.

In another words, it can be said to enlarge the farm size consequently reorganizing the existing scattered land here and there.

The scattered lots owned by a single farmer is to be grouped together into one common lot with the independent infrastructure.

The existing land possession is shown as in Fig-6.

Fig-6 Existing land possession



Total Area	11.2 ha.	
Total NOS. of Farmers	42 NOS.	
Total NOS. of field lot	44 NOS.	
Sub-devided lot	98 NOS.	
Average elevation	E.L 6.639 m.	
Average area	A = 0.269 m.	

LEGEND.

○ reference NO of ownership and same colour is to be same ownership.

3)-4 Existing land utilization

The each field lot under existing condition is categorized as in tab-4 and Fig-7.

Tab-4 Existing land utilization

LAND CATEGORY	AREA (ha)	RATE (%)	REMARKS
1.PADDY FIELD	3.5901	32.1	
2.VEGETABLE	0.3242	2.9	LOT NOS. 1533 C.D.
3.GRASS LAND	6.2265	55.6	11.1991 - 4.9726
4.FRUIT	0.8521	7.6	LOT NOS. 314 C, 1534 E, 449 B
5.RUBBER	0.0848	0.7	LOT NOS. 382 C, 138 C, 1569 A.
6.SWAMP	0.1214	1.1	LOT NO. 419 (DRAIN)
7.EXCEPTION	(0.0951)	(0.8)	LOT NO. 322 0.0951 ha.
TOTAL	(11.2942) 11.1991	(100) 100	

More than half of the area is categorized as the grass land which has not been cultivated for long time and only 32.1 percent of the area is under existing paddy field.

The rest is used for the fruit, vegetable, rubber and others respectively.

Accordingly, the productivity of land is to be very low and there is no means to increase the farmer's income though they have a land to produce money.

Existing paddy field area is shown as in tab-5.

Tab-5 Existing paddy field area.

REFERENCE NO.	LOT.NO	AREA (ha)
1	1568 A	0.0731
2	" B	0.3048
3	" C	0.0539
4	" D	0.0624
5	" E	0.0711
6	" F	0.0827
7	" G	0.0575
8	401 A	0.0727
9	409 A	0.0685
10	" B	0.0949
11	410 A	0.0994
12	" B	0.0611
13	1570 A	0.0880
14	403 A	0.0951
15	415 A	0.0569
16	" B	0.0678
17	" C	0.1364
18	" D	0.0811
19	448 A	0.0510
20	" B	0.1451
21	" C	0.0470
22	447	0.0587
23	416 A	0.1440
24	" B	0.0712
25	" C	0.0884
26	" D	0.1892
27	417 A	0.1100
SUB TOTAL		2.532

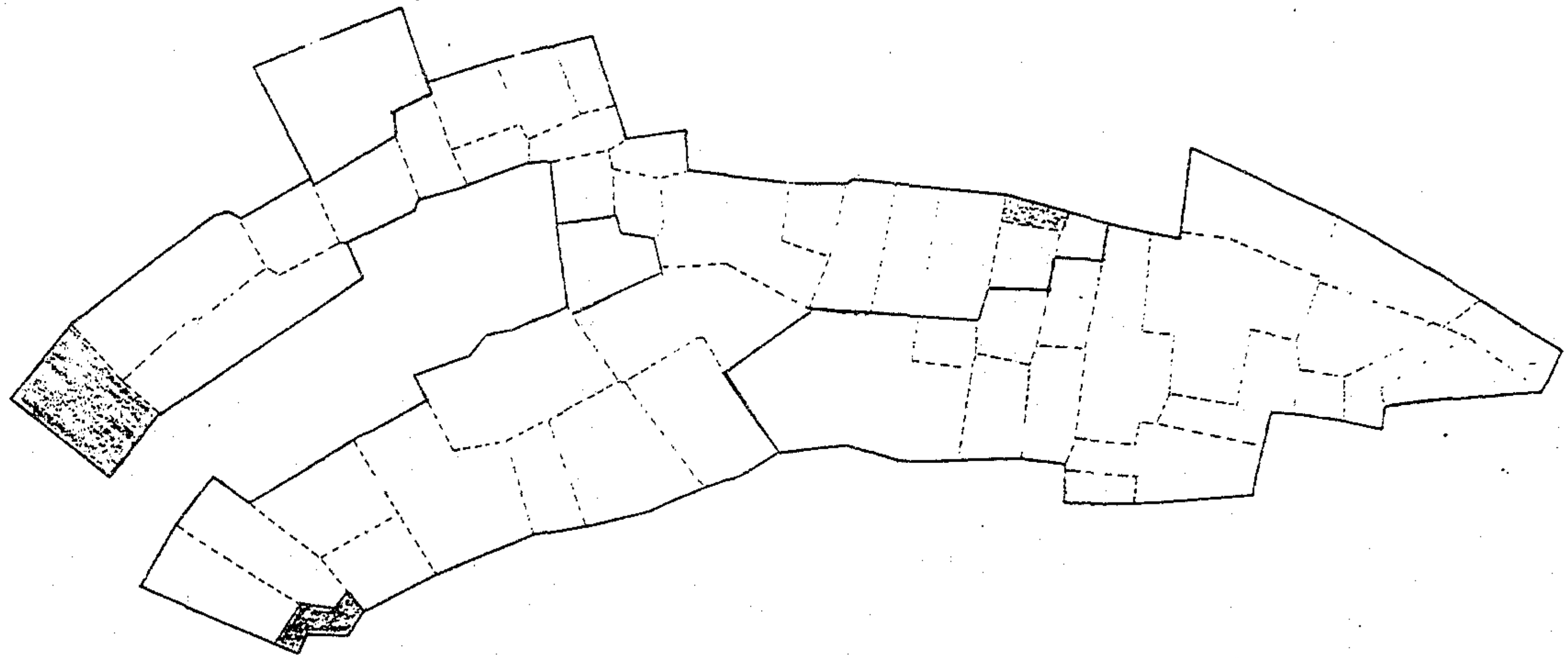
REFERENCE NO.	LOT.NO	AREA (ha)	REMARKS.
28	417 B	0.1104	
29	412 A	0.0952	
30	" B	0.0348	
31	" C	0.0552	
32	" D	0.0683	
33	" E	0.0440	
34	413 A	0.0400	
35	" B	0.1866	
36	418 A	0.0749	
37	" B	0.1002	
38	414	0.1194	
39	450 B	0.1076	
40	" C	0.0215	
SUB TOTAL		1.0581	
TOTAL		3.5901	

TOTAL AREA 11.1991 ha

EXISTING PADDY AREA 3.5901 ha

RATE 32.1 %

Fig-7   Existing land Utilization.



LEGEND

	paddy field	3.6 ha.
	fruit tree	0.9 ha.
	vegetable	0.3 ha.
	swamp	0.1 ha.
	rubber tree	0.1
	grass land	6.2 ha.
	Total	11.2 ha.

3)-5 Existing facilities

1) Out side of the benefitted area

1. External canal

$L = 378.66$ m

2. OFF - TAKE P24L

2) In side of the benefitted area

1. Irrigation canal

Actually, existing irrigation canal which is provided after the external canal run through just out side of the benefitted area, however it should be considered as the facilities which belongs to the benefitted area otherwise the farmers cannot get water.

$L = 280$ m

$$A = 280 \times (\overset{B}{0.50} \times \overset{H}{0.50}) = 70 \text{ m}^2$$

2. Drainage canal

$L = 105$ m

$A = 1.214 \text{ m}^2$ (whole area of Lot NO.419)

3. Farm road

$L = 140$ m; $B = 4.5$ m

overlapping length with drain $L_1 = 35$ m

$$\therefore A = (140 - 35) \times 4.5 = 472.5 \text{ m}^2.$$

3) Density of the facilities

Tab -6 Density of the existly facilities

Items	L (m)	*DENSITY 1) (m/ha)	2) (m/ha)	REMARKS
1. Irrigation canal	280	25.0	78.0	1)11.1991 ha
2. Drainage canal	105	9.4	29.2	2) 3.5901 ha
3. Farm road	140	12.5	39.0	

where, ※1) : Density against the whole benefitted area (11.1991 ha)

※2) : Density against the existing paddy area (3.5901 ha)

4) Batas

$L = 5,477.5$ m

average width $B = 0.25$ m

$A = 1,369\ m^2$

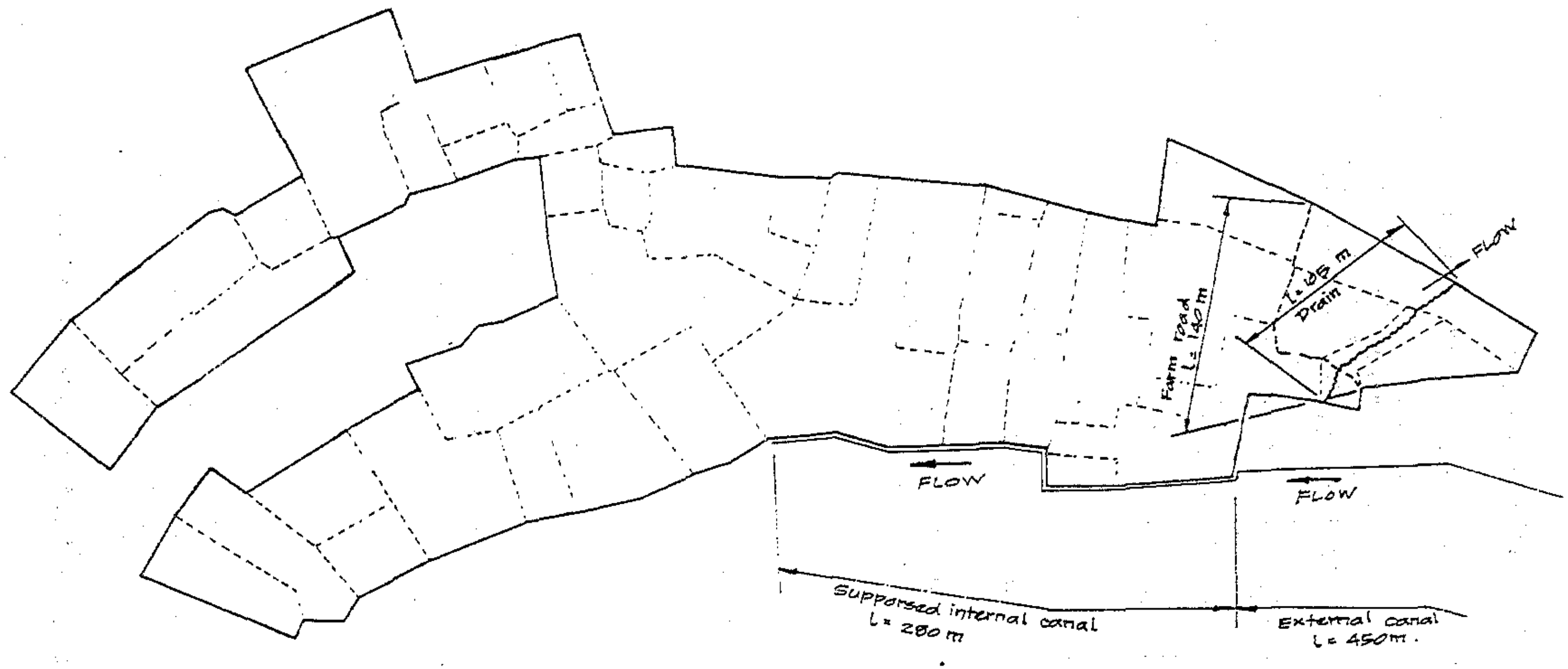
5) TOTAL AREA OF THE FACILITIES

Tab -7 Total area of the existing facilities

Items	A (m^2)	REMARKS
1. Irrigation Canal	70	
2. Drainage Canal	1,214	
3. Farm Road	473	
4. Batas	1,369	Bare = 0.25 m
TOTAL	3,126	

The existing facilities are shown as in Fig -8.

Fig -8   Existing Facilities



1. Existing irrigation canal

    1) Supporsed internal canal   l = 280 m

    2) External canal   l = 450 m

    Total l = 730 m

2. Existing drain   l = 105 m

3. Existing farm road   l = 140 m

3)-6 Soil and Water Study

1) Actual volume of the soil, Eh and PH.

1. Objectives

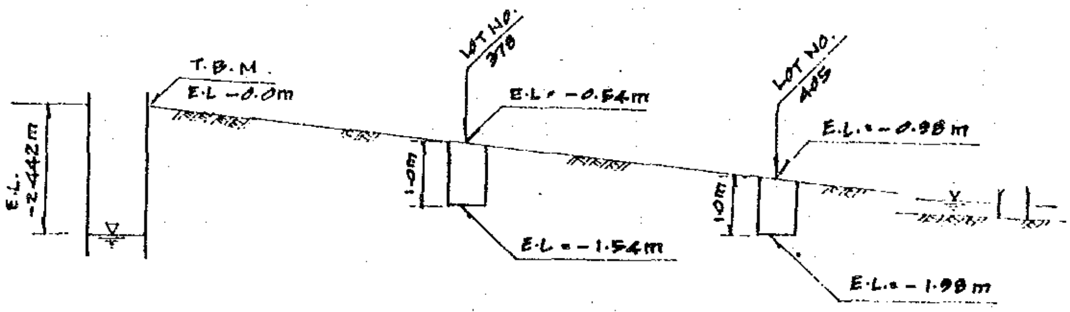
To know the three phases of the soil and its physico-chemical condition such as Eh and PH under existing grass land, the testing hole was excavated and afterwards soil sampling was performed at each 10 cm. depth one after another.

2. Soil condition

a) Ground water

First of all, it is better to study the site condition between Lot No. 378 and 405 which were the surveyed points in P/F No.3. The location is shown as in Fig -9 and topographical condition is shown as follows :-

Fig -10 Site Condition of the soil sampling.



The Lot No. 378 is about 54 cm lower than the highest ground in the area and Lot No. 405 is 98 cm. lower as well. During excavation of testing hole in Lot No. 405, there was no trace of ground water though the lot was adjacent to the saturated paddy lot.

Taking this into consideration, it can be said that the soil is a heavy clay and it constrain an impermeable layer.

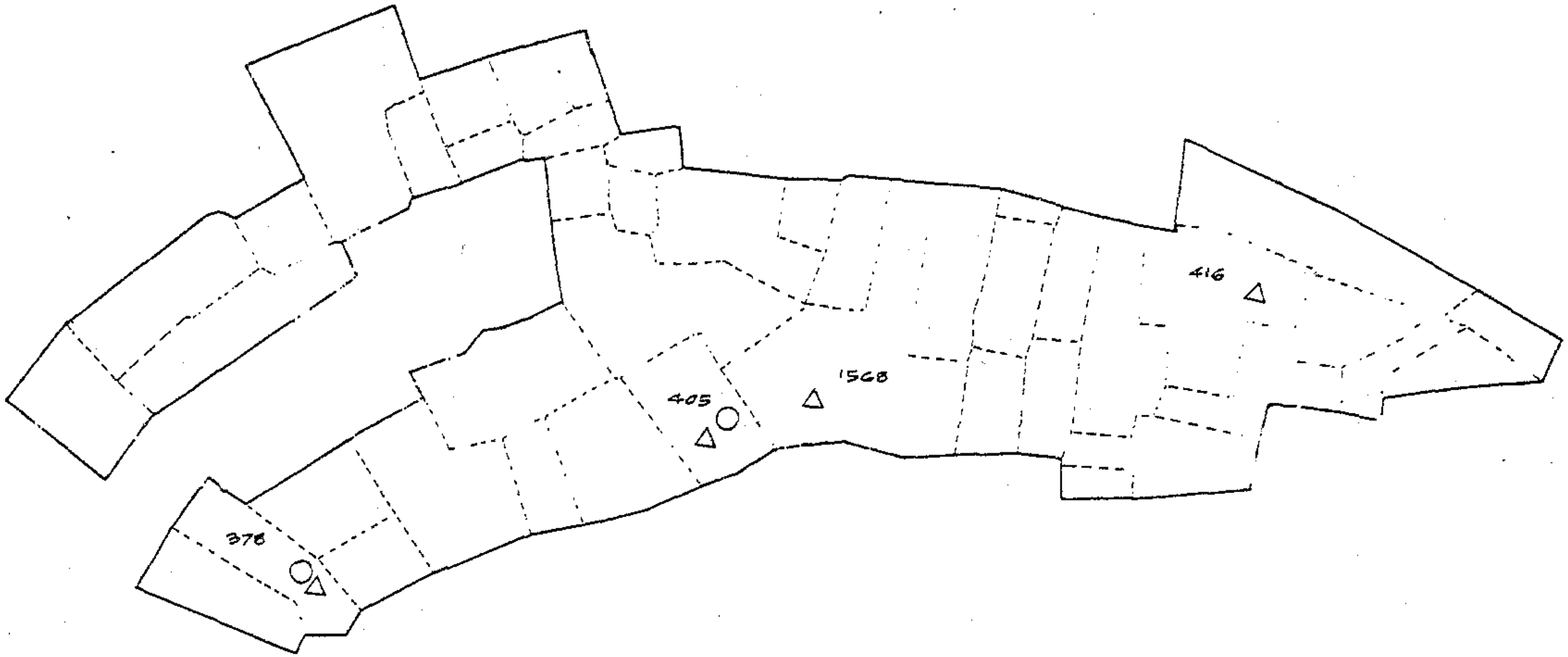
b) Top soil

Top soil thickness was determined in each Lot, that is, 15 cm. in the Lot No. 378 and 20 cm. in the Lot No. 405

c) Soil colour

Top soil colour was identified as not very dark gray compared to existing paddy ground and sub-soil was also brown with a little oxidized red colour.

Fig -9 Location of soil sampling and Cone penetration test.



LEGEND

○ Soil sampling

△ Cone penetration

3. Oxidation reduction potential (Eh) and hydrogen-ion-concentration (PH)

The samples for Eh and PH test were collected at the site at each 10 cm. depth. The site measurement results are shown as follows :-

Tab -8 Eh and PH value at Lot No. 378

Depth(cm)	Eh(mv)	PH	Depth(cm)	Eh(mv)	PH
0 - 10	250	4.9	50 - 60	350	5.1
10 - 20	290	4.9	60 - 70	390	5.1
20 - 30	320	5.1	70 - 80	390	5.1
30 - 40	330	5.0	80 - 90	380	5.1
40 - 50	330	5.0	90 - 100	390	5.1

Tab -9 Eh and PH value at Lot No. 405

Depth(cm)	Eh(mv)	PH	Depth(cm)	Eh(mv)	PH
0 - 10	300	4.8	50 - 60	340	5.0
10 - 20	300	5.2	60 - 70	340	5.0
20 - 30	320	5.0	70 - 80	340	5.0
30 - 40	320	5.0	80 - 90	350	5.0
40 - 50	340	4.9	90 - 100	340	5.0

Distilled water was not used at that time i.e the ground water which was taken from the well near by was used. The results are shown as in Fig -10. The water condition of the well is follows :-

Ground water...... Eh = 320 mv
PH = 5.5
water temperature 29 ^{o}C.

4. Condition of suspension

In case of the Lot No. 378, first sample of 0 - 10 cm, thickness was very much suspended and next 10 - 20 cm. sample was identified a little bit suspended.
Another 20 - 100 cm.depth,the suspension shown the same degree of clearance.

In case of the Lot No. 405, the second sample of 10 - 20 cm. thickness was the most suspended rather than 0 - 10 cm. sample and following samples were almost the same as well as the Lot No. 378.
It is said that the sample which was taken from the grassland part has shown the most suspended one and the top soil thickness is also to meet this result.

5. Three phases of the soil

These phases of the soil were measured by using the actual volume meter and its results are shown as in Tab -10.

Tab -10 Three phases of the soil

Items	Lot No. 378	Lot No. 405
Vapor phase (%)	7.2	9.3
Solid phase (%)	51.7	49.7
Liquid phase (%)	42.9	41.0
Moisture ratio	35.7	36.3
Real-specific gravity	2.32	2.27
Actual-specific gravity	1.72	1.69

* These figure are shown the average between 0 - 100 cm.

These phases of this soil and another peculiarities are shown as above table. The difference between the top soil were measured as below :-

Tab -11 Three phases of the top soil

Items	Lot No. 378 0 - 15	Lot No. 378 15 - 100	Lot No. 405 0 - 20	Lot No. 405 20 - 100
Vapor phase (%)	2.2	7.8	9.0	9.3
Solid phase (%)	54.1	51.4	50.9	49.5
Liquid phase (%)	43.7	42.9	40.1	41.2

Moisture ratio	35.1	35.9	9.0	9.3
Real-specific gravity	2.30	2.32	2.25	2.27
Actual-specific gravity	1.72	1.72	1.70	1.69

6. Integrated analysis

For the first time it is assumed that the Lot No. 405 is supposed to be more saturated than Lot No. 378 because the area is located near the irrigated paddy field, even the moisture ratio measured was a little bit high but almost of the same value.

For the vapor phase, it is realized that the Lot No. 378 was covered by grasses so that it prevent the cracks to happen unlike Lot No. 405 that was not covered with so much grass. This is the reason why the vapor phase in the Lot No. 378 was identified lesser than Lot No.405.

According to the Fig -11, the changes of solid and liquid phases were quite similar. It can be said that there was no ground water in the testing hole and it constrain the impermeable layer so that only the capillary water is available, there not the gravity water. This is the reason why the solid and liquid phases are paralel.

It can be analyzed that there is a tendency for the bulk density to increase within the thickness of 40 - 50 cm. in the soil.

In the Lot No. 378 around 60 cm. below the soil surface the vapor phase is decreasing so much till 0 % and also in the Lot No. 405 about 50 cm. below the surface it goes down very much. And after this the soild phase is increasing again.

In the case of the Eh and PH, the Eh shows more than 300 mv. and PH is also around 5.0 throughout the whole soil. However, in the top soil that was measured at 15 cm. in the Lot No. 378 and 20 cm. in the Lot No. 405 the Eh shown is the smallest. If compared between the Lot No. 378 and 405 the Eh of the former one found to be smaller than the latter because of the grass condition.

Generally speaking, the soil in this grass land has been oxidized for such a long time since the area is located in the high area where the water haven't been supplied so far, however the PH is not decreasing very much.

The results of the Eh and PH are shown in Tab. 8. though the distilled water was not used for the testing at that time as mentioned before.

The water from the well already indicated high oxidation potential about 320 mv. so it can be said that the top soil though covered by the grass or not was affected by the water. It is supposed to show smaller figure, however the results shows 300 mv.

If the assumed Eh given to the top 0 - 5 cm. soil is to be 250 mv. and 5 - 10 cm. is to be 280 mv. in the Lot No. 405, the correlation coefficient would be calculated as 0.83. This figure is assumed to be correct.

Normaly it is realized that there are some correlation between the Eh and PH such that if the Eh goes high the PH goes down in opposition.

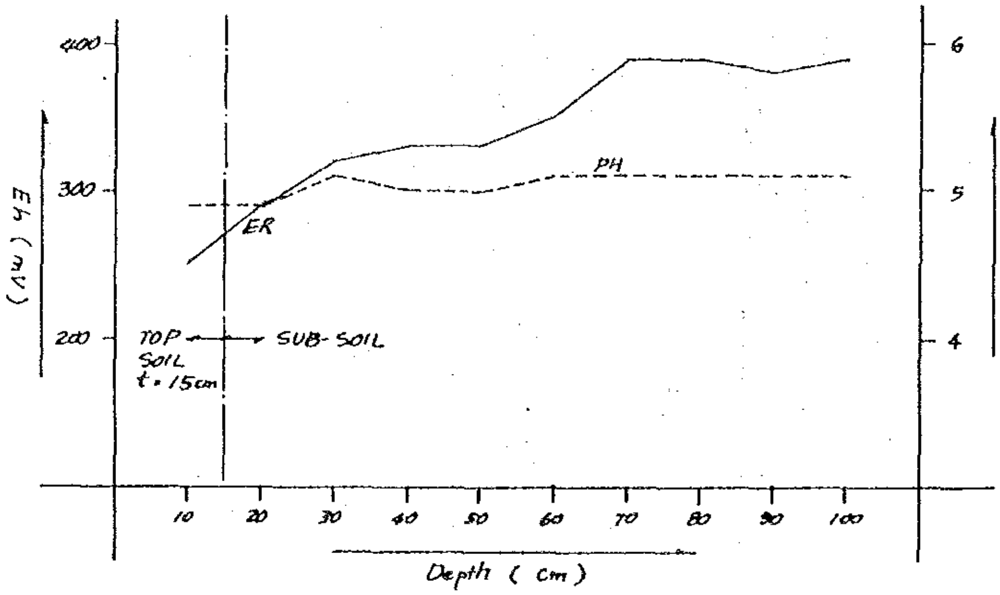
Eventhough the distilled water was not used for the test, however the correlation between Eh and PH can be determined as follows :-

Tab -12 Correlation between Eh and PH.

Items	Lot No. 378	Lot No. 405
Correlation coefficient	0.86	0.83
Variance	3.47	1.87
Average (Eh)	342	322
Average (PH)	5.04	4.96
Formula of linear regression	Y = 0.00153 X + 4.52	Y = 0.00179 X + 4.38

Fig -10　Oxidation reduction potential (Eh)
and Hydrogen-ion-concentration (PH)

Case -1　　Lot No. 378



Case -2　　Lot No. 405

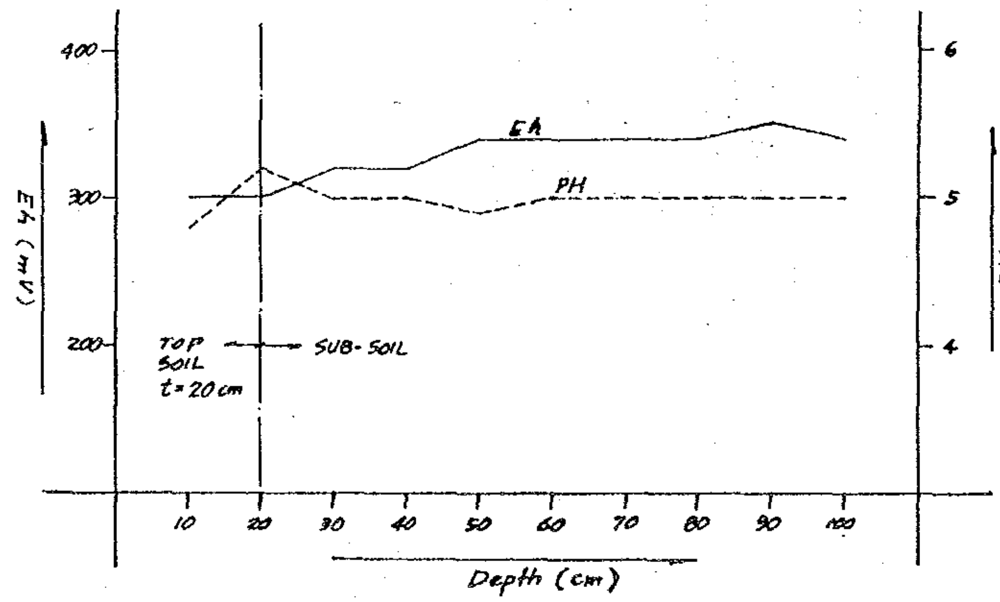
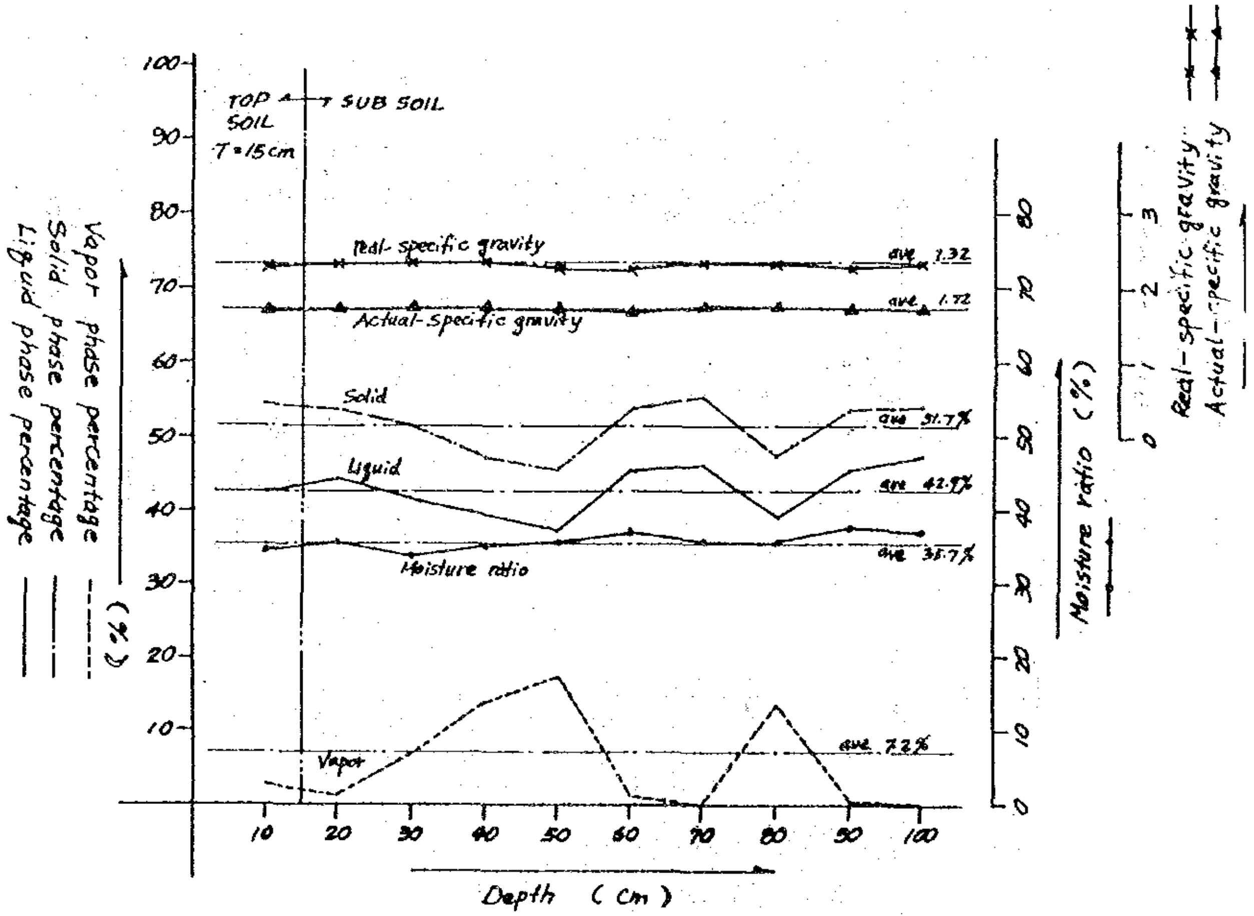
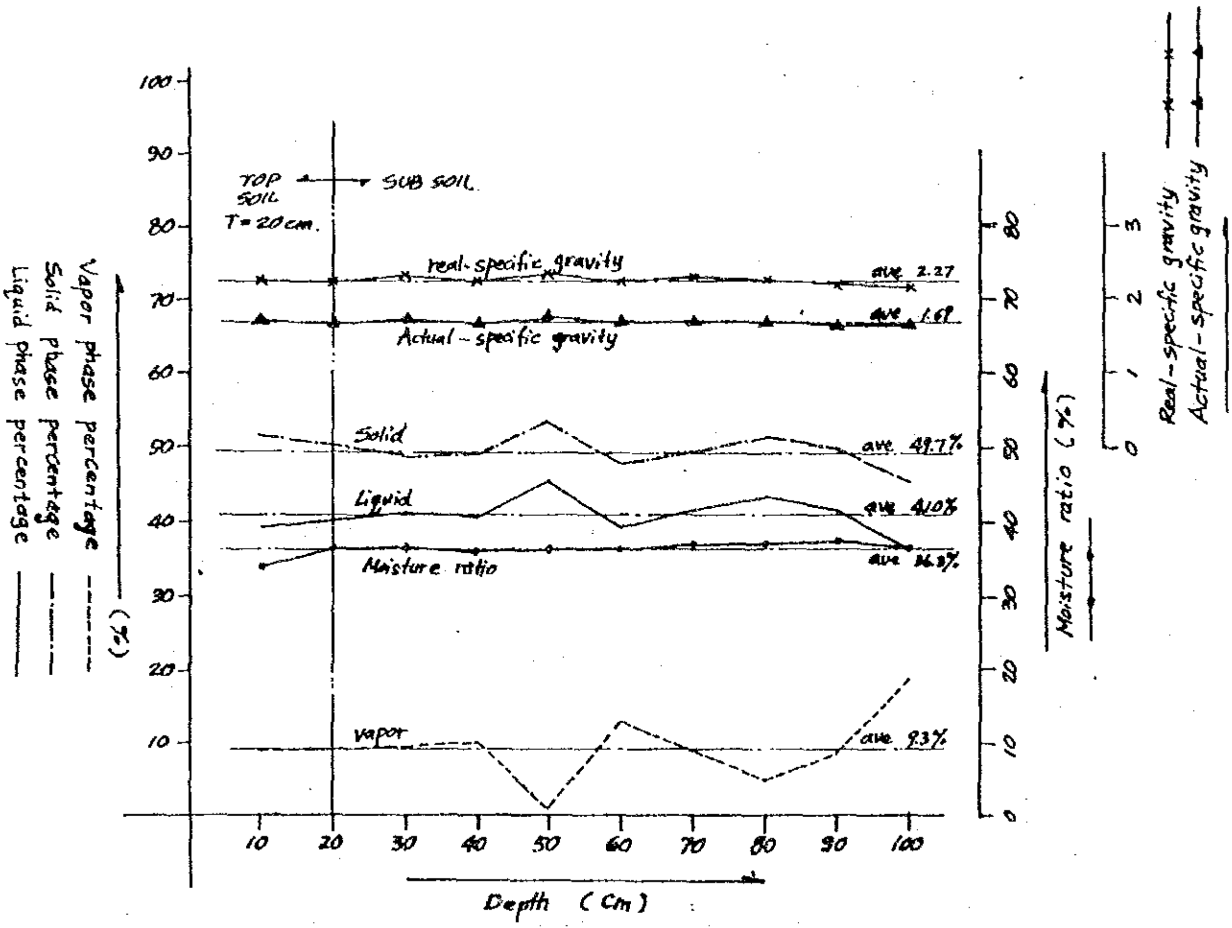


Fig -11 Actual volume measurement results.

Case -1 Lot No. 378



Case -2 Lot No. 405



2) Cone penetration test

1. Objectives

To know the bearing capacity of paddy and grass land under existing conditions.
The grass land has not been planted for a long time such that the soil is found to be highly oxidized.
This survey result can be used to show the change in the grass land after land consolidation.

2. Place and apparatus for measurement

a) Place

1. Paddy land ..... Lot No. 416, 1568
2. Grass land ..... Lot No. 378, 405

Each lot condition was under flood.

b) Apparatus

1. SR-II ...... cone penetrometer

(30^{o}, 6 cm^2)

3. Measurement and results

The survey was carried out by using the SR-II cone penetrometer at the points that are shown as in Fig -9. In the paddy field it was identified that for the Lot No. 1568 the irrigation water was kept to 10 cm deep and the Lot No. 416 was also kept up to 8 cm.

a) Result

In the grass land, the first cone index already shown to be over 6 kg/cm^2 and its soil hardness is going harder and harder. Especially in the Lot No. 378 which is the highest area in the grass land only 5 cm. deep could penetrate.

On the other hand, the cone penetration test could be carried out from 25 cm. to 30 cm., however, the clear hard pan could not be determined at that depth. It can be said that the hard pan could not be determined in this soil.

It means this soil is going harder and harder and soil has good enough soil hardness for the machinery.

The results of the three phases of the soil shows it might required some countermeasurement in-order to increase the soil permeability such as the subsoil breaking using a pan-breaker at the construction time. Because at the construction time the top soil removing and land levelling will be carried out and it could be considered without any difficulty that the bulldozer would compact it harder.

The results of the cone penetration test is shown as follows :-

Tab -13 Results of the cone penetration test.

Items	grass land 378	405	paddy ground 1568	416
Initial No.	7.0/35	7.3/25	6.9/15	7.0/10
Suffix No.	3	3	2	1
Soil type	C	C	C	C
Judgement	7.0/35C3	7.3/25C3	6.9/15C2	7.0/10C1

4) Determination of the land sharing rate.

The most interesting matter for the farmers is a land sharing rate.

Generally, the people doesn't want to give his land to the project without any compensation or the land acquisition in spite being the government project or others.

One of the features of the land consolidation is that the area for the on farm facilities can be shared among the farmers who have their land in the benefitted area.

The sharing rate is recommended as approximately 10 percent. If the sharing rate is exceeding 10 percent then the area which belongs to the farmers is going to be small because they lost a lot of land.

Especially the average holding area of the farmers in this area is very small that is 0.269 ha., so on the occasion of planning this project the sharing rate should be kept smaller as far as possible.

In addition, it must make sure that the land which can be produced by the land sharing among the farmers does not belongs to certain people but to all of the farmers who organize the farmers cooperative and these facilities should be operated/ maintained by the cooperatives.

4)-1 Area for the proposed on farm facilities.

According to the design, the area is determined as follows.

1) Out side of the benefitted area

1. External canal

L = 378.66

Demolishing the canal by base concrete and rip-rap with the proposed slope 1 : 54,000 and 1 : 31,000

2. OFF-TAKE P24L
without any demolish.

2) In side of the benefitted area.

1. Irrigation canal

Tab -14 Length of the proposed irrigation canal.

Items	L (m)	CONSTRUCTION TYPE	REMARKS
Q.C.1	259.175	PIPE LINE 375 (B)	INCLUDING FIELD OFF-TAKE BOX.
Q.C.1	207.270	OPEN CHANNEL, I = $\frac{1}{2}$,700	
Q.C.1	357.50	OPEN CHANNEL, I = $\frac{1}{4}$,400	
Q.C.2	69.79	PIPE LINE, 375 (B)	
Q.C.3	286.80	OPEN CHANNEL, I = $\frac{1}{4}$,400	
Q.C.4	51.50	OPEN CHANNEL, I = $\frac{1}{4}$,400	
SUB TOTAL	1.232.035		
FARM ROAD CROSSING 1	2.09		
" 2	7.50		
FARM ROAD CROSSING	18.60	6 NOS. x 3.1	FIELD BLOCK NO.4
SUB TOTAL	28.19		
TOTAL	1.260.225		

2. DRAINAGE CANAL.

Tab -15 Length of the proposed drainage canal.

Items	L (m)	REMARKS
F.D.1	231.00	$I = \frac{1}{1000}$, t = 0.30
F.D.2	367.00	INCLUDING DRAINAGE
F.D.3	132.500	CONTROL
F.D.4	392.00	
F.D.5	119.00	
F.D.6	227.00	
TOTAL	1,468.60	

3. FARM ROAD.

Tab -16 Length of the proposed farm road.

Items	L (m)	REMARKS
M.F.R	908.00	B = 3,000
F.R.1	84.00	LATERITE PAVEMENT
F.R.2	73.50	t = 0.10 5% SLOPE
F.R.3	302.00	
F.R.4	66.00	
TOTAL	1,433.50	

4. OTHER STRUCTURES.

Tab -17 Number of structures on proposed design.

Items	NOS	REMARKS
M.B.1	1	1.600 x 1.057
M.B.2	1	1.000 x 1.000
PERSHALL FLUME	1	B = 152
CONTROL BOX	1	M.S SLIDE GATE 1.000 x 1.000 x (2)
DIVISION BOX NO.1	1	M.S SLIDE GATE
DIVISION BOX NO.2	1	M.S SLIDE GATE

Items	NOS	REMARKS
CORNER BOX	1	500 x 500
DRAINAGE CONTROL	6	HARD WOOD GATE
BOX CULVERT	1	1.000 x 900
FIELD OFFTAKE	17	WATER TAP 65 BALL VALVE
"	34	BOX TYPE , B = 200
FIELD OUTLET	43	INTO DRAIN 300 P.V.C
"	10	DIRECT, 300 P.V.C
ACCESS TO FIELD LOT	50	SLOPE 31.2 %, B = 3,000

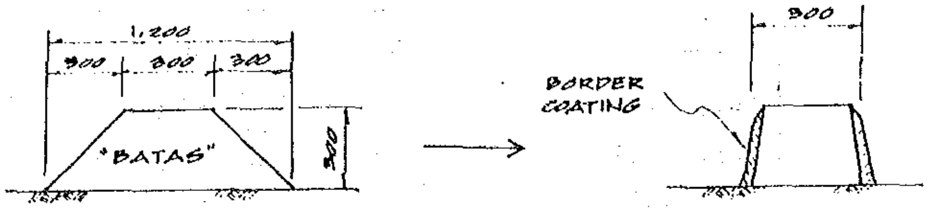
.3) DENSITY

Tab -18 Density of the proposed facilities.

Items	L (m)	DENSITY (m/ha)	REMARKS
1. Irrigation Canal	1,260.^{225}	112.5	A = 11.1991 ha
2. Drainage Canal	1,468.^{50}	131.1	
3. Farm Road	1,433.^{50}	128.0	

4) BATAS.

The "batas" will be constructed by the dimension which are given below, however few years after construction, it means after the batas is compacted naturally, the farmers will reduce the size in order to extend their field.
So, it can be said that the final section of the "batas" will be 0.30 m width.
It goes without saying that, the "batas" is provided to prevent the seepage from the border and save the water for good water management, therefore after the soil is taken by the farmers they have to maintain it by border costing and so on.



L = 2,741.5 m

B = 0.30

A = 822.45 m^2

5) TOTAL AREA OF THE FACILITIES.

Tab -19 Total area of the proposed facilities.

Items	A (m^2)	REMARKS
1. Irrigation Canal	6.560.98	
2. Farm Road		
3. Drainage Canal	6.444.81	
4. Multi Purpose Center	400.0	20 x 20 m
5. Batas	822.45	B = 0.30 m
Total	14,228.24	

4)-2 Determination of the land sharing rate.

The balance between the existing area and proposed area is to be 11,102 m^2 , thus the land sharing rate is to be as follows.

Tab -20 Area for land sharing.

Items	EXISTING 1) A (m^2)	PROPOSED 2) A (m^2)	2)-1) (m^2)
1. Irrigation Canal	70	} 6,561	} 6,018
2. Farm Road	473		
3. Drainage Canal	1,214	6,445	5,231
4. Batas	1,369	822	- 547
5. Multi Purpose Centre	-	400	400
TOTAL	3,126	14,228	11,102

Conversion rate to new facilities is to be :-

$$(11,102 / 111,991) \times 100 = 9.9$$
$$\fallingdotseq 10 \%$$

5) Replotting plan.

5)-1 Significance of the replotting

In principle, the replotting is determined in Japan as follows.

1) Customary land and equivalant customary land.

Certain land before construction is defined as the customary land and new land after construction, that is, replotting area is regarded to the equivalant customary land.

Due to the Land Consolidation Project, it is very difficult to keep the old on-farm facilities such as road and canal because of the changes of land shape and it's quality.

Thus, only The Land Improvement Law and Land Adjustment Law that are concerned to the Land Consolidation Project are allowed to take this replotting method.

2) Objectives of the replotting.

The replotting plan should be established to contribute grouping of farm land and it's improvement of farming structures.

To improved the existing farm management that is approaching to petty farming where the land is scattered, it is quite necessary that the grouping of farm land should be established.

3) Disposal of replotting

The replotting plan is defined as the official connection between customary and replotting land. The disposal of replotting is also defined as the reversion of right of the land is decided by the replotting plan.

In another words, the farmers who are situated in the Land Consolidation area can get the new land without any converting institution in right of land.

5)-2 Method of replotting

1) Normal replotting

The replotting sight of new land utilization is to be the same area as well as the customary sight.

Agricultural land should be replotting in the agricultural land and land for another purpose should be replotted outside the agricultural land.

The replotting area should be situated in the almost the same condition of customary sight.

Judging from the various conditions of customary sight, such as purpose of the land, area, soil conditions, water distribution, topographical conditions, temprature and other natural conditions the replotting area is to be almost the same.

The replotting area that is decided by delivery rate of replotting is to be within a 20 % difference.

It is required to plan the difference between customary and replotting area should be limitted within 20 %.

2) Special replotting

Replotting is exceptional from the normal one.

In this case, it is necessary to get the agreement from the person who have the ownership right, usufruct real right such as superficies right, emphasis right, pawned right, right of lease and another concerned rights.

3) Non replotting

It is not necessary to replot from the customary land due to the request or agreement from the person who have a certain rights but not the ownership right.

In this case, the settlement of accounts is paid and all the rights are lapsed simultaneously.

4) Re-productive replotting.

Under this replotting, it is possible to produce a new land that is mainly used for the public facilities nevertheless there is no customary land.

1. Re-productive replotting by the land sharing.

   New agricultural facilities such as a new road, canal and rice-centre etc. which are mainly used for the public purpose can be distributed equally by land sharing of all replotting area.

5) Exchanging of function

Previous facilities such as road, canal and pond that are owned by the government would be able to exchange their function after the facilities have been constructed.

Though the principle significance and it's method is determined as above mentioned, however the activities should be carried out in accordance with the law.

At the moment there are no institutional way to control these activities in Malaysia therefore the replotting and land sharing is planned to be carried out as follows.

5)-3 How to carry out the replotting

There are a few cases of the land reformation projects in Malaysia, one is in Kemubu area and the other is in Asam Jawa.

In fact, these projects do not involve the replotting, however they are united by the farmers' cooperative and all the lots are to be grouped into one large common lot belonging to the cooperative.

Thus, on cooperative basis, the common lot is to be levelled to meet with the supply level in each field block. Though the present boundary batas separating each individual lot would be demolish, the boundary stones would not be disturbed but to be buried deeper so as not to disturb the replotting and other farm activities.

1) Proposed lot size in each field lot.

The grouping of the land is to be one of the main purposes as mention before and existing condition of the scattered land possession is shown as in Fig -12 and it's average distance is indicated as in tab -21.

Tab -21 Average distance between the scattered land.

No.	Ownership No.	Total 1) Area (ha)	Dis. 2) (m)	Name of ownership.
1	1 - 2	0.4089	283	CHE SEMAN B. PUTEH.
2	3 - 4	0.5282	176	SAFAIAN B. IBRAHIM.
3	5 - 6	0.6558	370	FATIMAH BTE DAUD.
4	7 - 8	0.3057	217	MERIAH BTE SAADU.
5	12 - 13	0.3724	148	MUDA B. SENIK.
6	14i- 15	0.2257	130	MEK NOR BTE YUSOFF.
7	16 - 17	0.4170	245	SEPIAH BTE MAT AMIN.
8	20 - 21	0.3785	336	ABDULLAH B. SALLEH.
9	41 - 41	0.175	52	MAJID B. OMAR.
		3.4672		

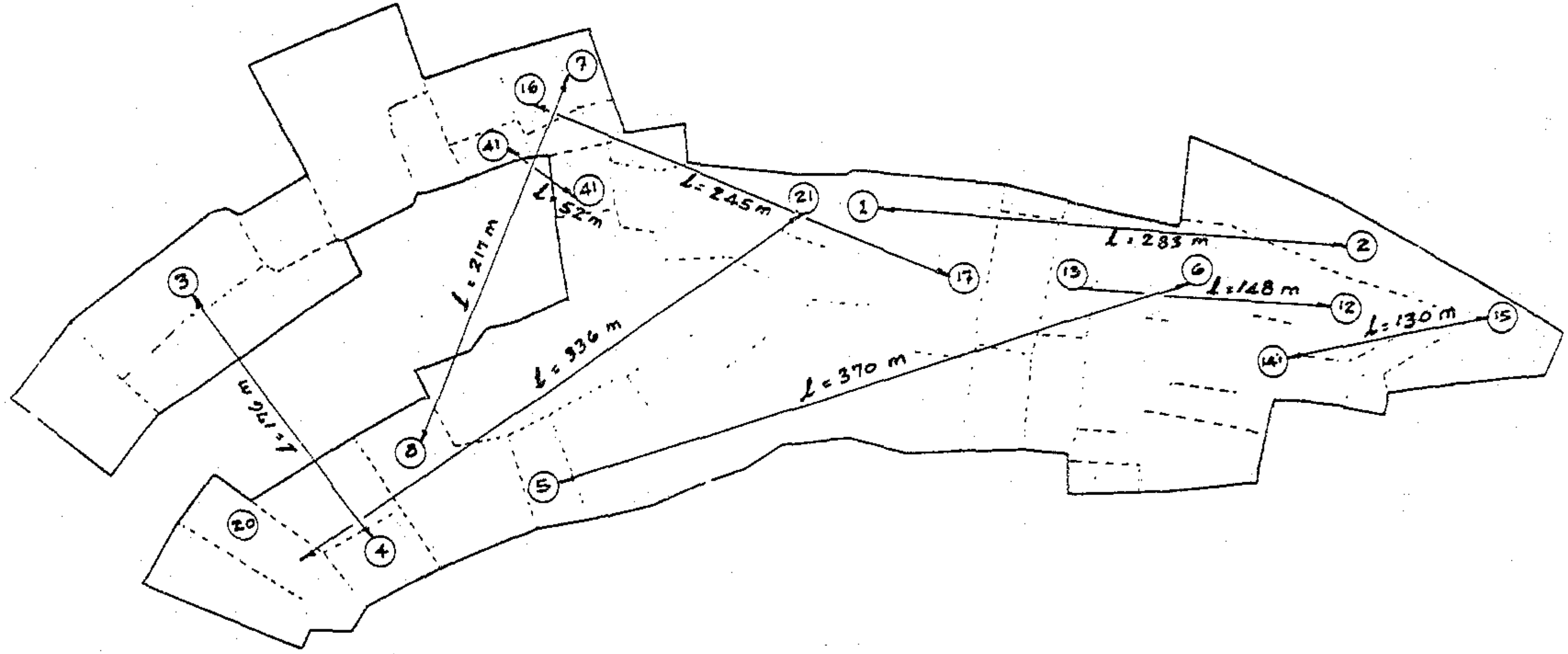
$$\therefore \text{average distance} = \frac{\Sigma\ 1)\ \times\ 2)}{3.4672}$$

$$= \frac{840.562}{3.4672}$$

$$= 242.^{43}\ m.$$

On occasion of the replotting, the insite replotting is carried out as far as possible and it is planned that the smaller lot is to be combined by bigger lot owned by same farmer. The short side of field lot is planned to be not less than 20 m which is the minimum necessary length for the mechanization.

Fig -12 Existing scattered land possession.



LEGEND.

○ Reference No. of ownership.

Average distance 1 = 242 m

Consequently, the lot size of the each field lot is determined as shown in tab -22.

Tab -22 Determination of each field lot size.

Field Block	Lot No.	Area (m^2)	Dimension ($m \times m$)	Weighted elevation (m)
1.	397	783	31.32 x 25.0	5.319
	417	957	45.50 x 21.03	5.269
	415 A	1,356	61.64 x 22.0	5.646
	328 B	1,805	75.2 x 24.0	5.705
	328 A	1,805	67.0 x 26.95	5.810
sub-total		6,706		
2.	418	1,537	53.0 x 29.0	5.664
	412	2,514	70.02 x 35.9	6.226
	1569 1570	2,340	73.12 x 32.0	6.058
	447 448	3,000	irregular	6.387
	419	1,094	57.25 x 19.11	6.382
	409	1,458	59.5 x 24.5	6.483
sub-total		11,943		
3.	403 418	1,800	77.5 x 23.23	6.017
	416 380ii	4,423	90.5 x 48.88	6.214
	416 380	1,480	51.0 x 29.0	6.446
	317	1,203	55.05 x 21.85	6.568
	381 B	1,594	59.03 x 27.0	6.614
	381 A	1,594	61.25 x 26.02	6.879
sub-total		12.094		

Field Block	Lot No.	Area (m^2)	Dimension (m x m)	Weighted elevation (m)
4.	410	1,458	64.8 x 22.5	6.578
	414 417	2,032	65.1 x 31.21	6.642
	1568i	2,125	63.0 x 33.7	6.621
	1568ii	2,640	73.5 x 36.0	6.726
	1568iii	3,080	88.0 x 35.0	6.826
	405	2,934	101.17 x 29.0	6.997
sub-total		14,916		
5.	450	1,162	61.70 x 18.83	6.529
	415 B	1,356	61.5 x 22.05	6.898
	400ii	1,728	62.05 x 27.85	7.174
	400i	1,953	62.0 x 31.5	7.151
	320ii	1,850	64.9 x 28.5	6.877
	320i	1,903	68.0 x 28.0	6.848
	398	3,389	80.7 x 42.0	7.318
	323	1,575	75.0 x 21.0	7.141
sub-total		14,916		
6.	1534i	2,880	96.0 x 30.0	7.072
	1534ii	2,804	95.05 x 29.5	7.031
	1592 A	2,752	80.9 x 34.02	7.062
	1592 B	2,023	69.75 x 29.00	7.100
	1591	2,187	69.0 x 31.69	7.079
	399 378	3,407	irregular	6.786
	382	2,295	81.95 x 28.0	7.167
sub-total		18,348		

Field Block	Lot No.	Area (m^2)	Dimension (m x m)	Weighted elevation (m)
7.	395	693	28.88 x 24.0	5.892
	449	584	32.44 x 18.0	6.129
	415 C	678	35.65 x 19.02	6.246
	316	1,075	48.00 x 22.40	6.448
	315	1,258	50.30 x 25.01	6.721
	314	3,510	irregular	6.379
	314	3,510	irregular	6.627
sub-total		11,308		
8.	312 379ii	2,650	61.98 x 42.75	7.002
	312 379i	2,104	61.0 x 34.5	7.127
	1590 B	1,767	58.9 x 30.0	7.280
	1590 A	1,767	58.3 x 30.31	7.266
	1533	2,918	60.80 x 47.99	7.355
sub-total		11,206		
Total		100,790		

DESIGN OF EXTERNAL CANAL

1) Determination of Full Supply Level (F.S.)

1-1) Difference between the proposed crest height and the one after construction.

First of all, the proposed F.S.L between R8L and R9L is identified as follows:-

1) Distance

(40.972 - 36.661 ft)x0.3048 = 1.314 m.

2) Proposed F.S.L

R8L = 25.39 ft (7.92 m)

R9L = 25.13 ft (7.66 m)

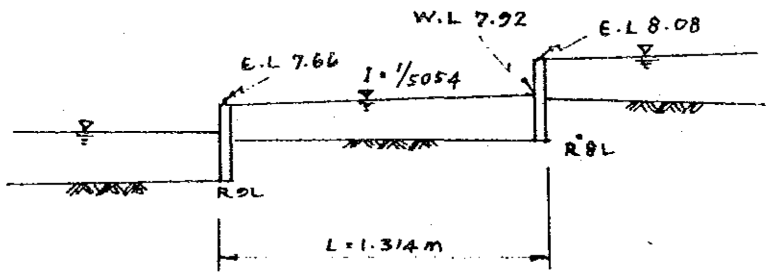
Difference = 7.92 - 7.66 = 0.26 m.

3) Water Slope

I = 0.26/1.314 = 0.0002

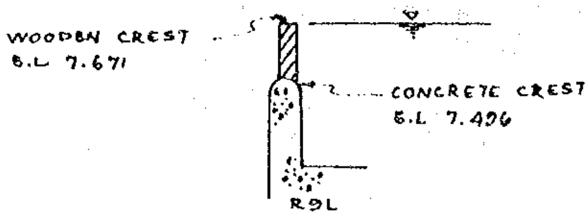
= 1/5,054

Fig. 13 Proposed F.S.L at R8L and R9L



The actual F.S.L which is obtained by survey at R9L is shown as below:-

Fig. 14 Actual F.S.L at R9L



Top of the west ... E.L. 7.496

Top of the wooden crest ... E.L. 7.671

The hardwood timber with 0.175 m width was implemented as temporary crest in order to increase the F.S.L.

This means, in spite of the F.S.L at R9L is designed to the W.L 7.66, however, the actual crest height has been constructed as E.L. 7.496 that is 0.164 m. below the proposed crest height. A piece of wooden timber was placed on top of the concrete crest in order to reach the designed F.S.L. crest as the emergency measure in order to adjust the proposed one.

1-2) Determination of the F.S.L. at P23L.

(1)

The F.S.L at P23L was found to be W.L. 7.76, however, this must be reconsidered by the reason as mentioned above.

1. Distance between P23L and R9L

   (40,972 - 39,298 ft.)x0.3048 = 510 m

2. Supposed F.S.L. at R9L

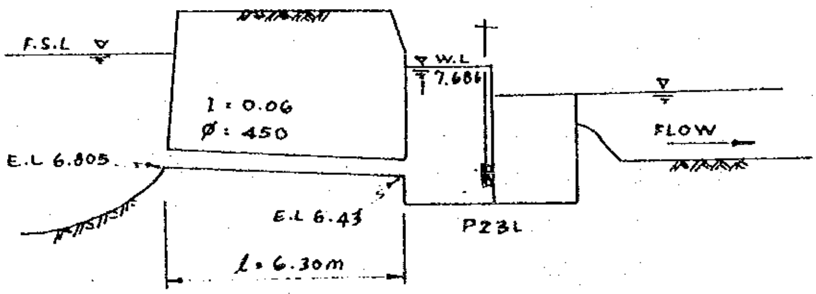
   510x0.0002+7.671 = 7.773 m --- (2)

   Assuming the F.S.L at P23L when the Qmax is obtained as 42 l/sec with W.L. 7.686 at the first box, the supposed F.S.L is to be as follows:-

(Conditions)

1.	the water level at the first box of P23L	... W.L 7.686
2.	Actual quantity of intake water	... Q=42 l/sec
3.	Coefficient of roughness of the pipe	... n=0.014
4.	The water level reached up to the top level of wooden crest at R9L	...

Fig.15 Relation between F.S.L and P23L



3. Velocity and quantity of water in the pipe

<u>Manning's formula</u>

$$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

$$= \frac{1}{n}.R^{1/6}.R^{\frac{1}{2}}.I^{\frac{1}{2}}$$

$$= \frac{1}{n}.R^{1/6}.\sqrt{R.I}$$

<u>Chegy's formula</u>

$$V = C.\sqrt{R.I}$$

where, C = coefficient of velocity $\sqrt{\frac{8.g}{f}}$

f = coefficient of friction losses.

then, the coefficient of velocity can be given as follows:-

$$\frac{1}{n}.R^{1/6} = C = \sqrt{\frac{8.g}{f}}$$

$$\therefore \quad f = \frac{8.g.n^2}{R^{1/3}}$$

where, $n = 0.014$

$$R = \frac{D}{4} = \frac{0.45}{4} = 0.1125$$

$$\therefore \quad F = \frac{8x9.8x0.014^2}{0.1125^{1/3}} = 0.0318$$

Taking into consideration of inlet, outlet and friction losses, the velocity in the pipe is given as follows:-

$$V = \frac{2.g.h}{\sqrt{1.5+f.1/D}}$$

where, g : acceleration of gravity 9.8 $m.sec^{-2}$
h : difference of water head (m)
f : coefficient of friction (0.0318)
l : length of pipe (6.3 m)
D : diameter of pipe (0.45 m)

$$\therefore \quad V = \sqrt{\frac{2x9.8xh}{1.5+0.0318x\frac{6.3}{0.45}}} = \sqrt{10.08\ h}$$

$$Q = A.V$$

$$= \frac{\pi.D^2}{4}.V$$

$$= \frac{\pi x0.45^2}{4} x\sqrt{10.08.h}$$

$$= 0.16\ x\ \sqrt{10.08.h}$$

the measured quantity is 42 l/sec, so that the difference of water head is to be as follows:-

$$0.042 = 0.16x\sqrt{10.08.h}$$

$$\therefore \ h = 0.0068\ m$$

Consequently, the F.S.L at P23L is suppposed to be:-

$$7.686 + 0.0068 = 7.693 \text{ ---- } (3)$$

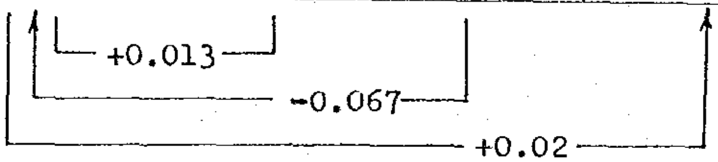
the operating and maintaining water level at P23L by KADA can be obtained by the darker mark on the wall of the first box.

According to this controlled mark, the water level can be read by (4) W.L 7.78.

The quantity of inlet water, however, it can be said that the F.S.L at the P23L should be higher than W.L 7.78.

All of the investigated F.S.L is arranged as follows:-

Proposed F.S.L by KADA (1)	Presumed F.S.L by R9L (2)	Hydraulic Cal. at P23L (3)	Controlled water mark (4)	Wooden crest at R9L
7.76	7.773	7.693	7.78	7.671



The different values of the F.S.L obtained depend on the overflowing condition at the regulator R9L. The overflow depth is kept constant in order to maintain stable intake.

Consequently, the proposed F.S.L by KADA which is shown by the middle figure as W.L 7.76, is determined as the F.S.L at P23L and also the ordinary supply level is to be W.L 7.71 which is considered by 5 cm risk.

F.S.L	7.76 m
O.S.L	7.71 m

2) Determination of the maximum discharge and terminal water level of the external canal.

The existing condition of the external canal is shown as in Fig. -

According to this result, the total water head losses is to be 0.691 m with the total length of 456 m. Since the loss over the short length is quite large, it is proposed that the external canal should be demolished.

In Kemubu scheme, the water duty is decided as 60 acres per one cusec (1.18 l/sec/ha). According to this water duty, the maximum water discharge of the P/F No. 3 is to be as follows:-

P/F No. 3 total area $A = 11.2$ ha

$\therefore$ Maximum discharge $11.2 \times 1.18 = 13.2$ l/sec

Though the maximum discharge is given as 13.2 l/sec, it is too little to operate the area within a certain period and also 42 l/sec discharge was already obtained.

Therefore, it is considered that the maximum water discharge will be taken as much as possible from P23L to meet the Land Consolidation aspects that can be used as the training purposes of FLPAK.

Actually, the water discharge of 42 l/sec was obtained when the assumed F.S.L reached to W.L 7.693.

Now the F.S.L is determined to be W.L 7.76 so that the proposed maximum water discharge to be obtained should be more than 40 l/sec.

The system flow chart of the calculation is shown in Fig. -16

Fig. - 15A EXISTING CONDITION OF EXTERNAL CANAL

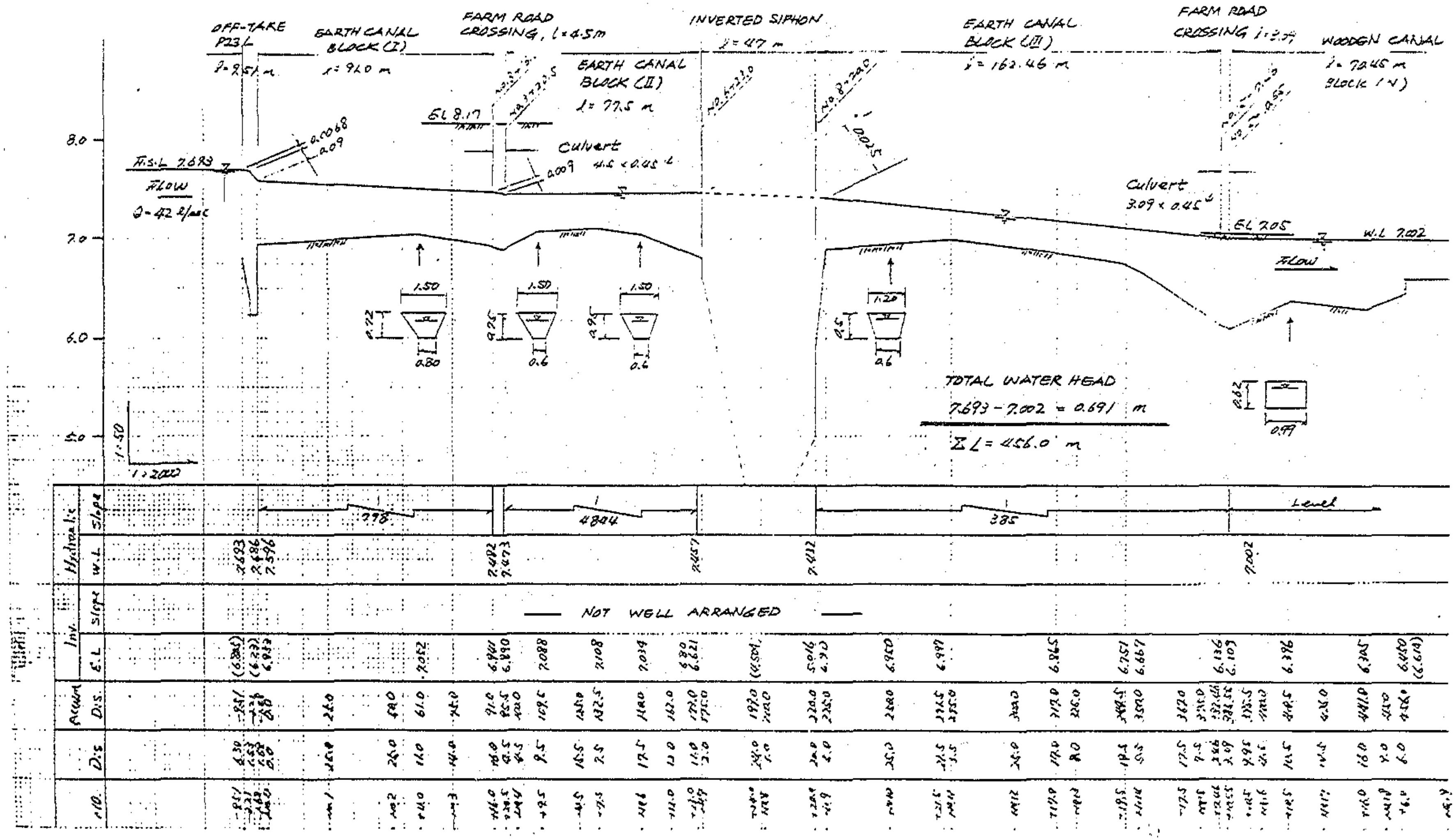
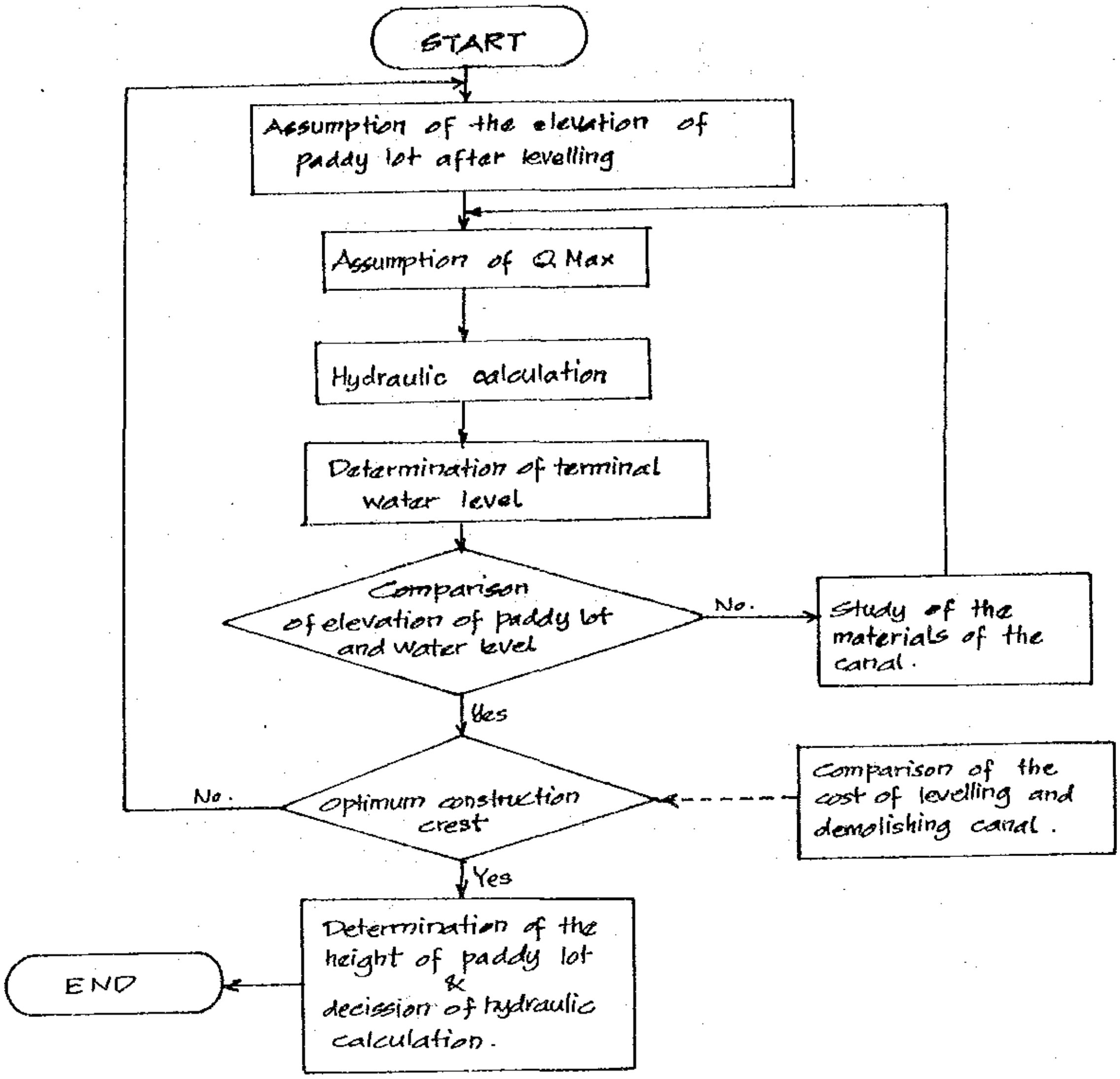


Fig. -16 System flow chart to determine the terminal water level.



Likewise, in determining the terminal water level it is required to consider about not only to demolish the canal but also the relation of the cost of land levelling.

2-1) Assumption of the elevation in each field block

The area of the P/F No. 3 is categorized into two types, one is the existing paddy area and the other is grass land which has not been planted for a number of years.

Tab. 23 Category of the P/F No. 3 area

Land category	Area (ha)	Rate (%)
paddy land	3.59	32.1
grass land	6.23	55.6
others	1.38	12.3
Total	11.20	100

And, after providing the on-farm facilities the average elevation on each field block is given as below:-

Tab.24 Average elevation on each field block

Block	1	2	3	4	5	6	7	8	Total
Area (m^2)	6,706	11,943	12,094	14,269	14,917	18,348	11,308	11,207	100,792
Elevation(m)	5.61	6.21	6.39	6.76	7.05	7.03	6.45	7.20	6.68

The highest block, block No. 5 is the control block where the elevation is E.L. 7.05, so the terminal water level should be planned to provide enough water head to supply to block No. 5 and/or is it possible to increase the terminal water level up to that height.

2-2) Hydraulic calculation

1) Case-1 ... Using the existing external canal without any demolish.

Qmax = 421/sec	(F.S.L 7.693)
terminal water level	: W.L 7.002
irrigable area	: 4.5 ha (Block No. 1, 2, 3 and 4)
trouble area	: 5.6 ha
land levelling volume	: 12,000 m^3

If the irrigation canal is given by the slope of 1/3,000, the supplied water level at the field block No. 5 is to be W.L. 6.8.

In this case irrigable area can cover only the block No. 1, 2, 3, and 4 with the total area of 4.5 ha.

The land levelling is required at field block No. 5, 6, 7 and 8 approximately 12,000 m^3

Consequently the construction cost is to be:-

Item	Cost ($)	Remarks
Demolishing of the canal	-	
*Land levelling	75,600	12,000x6.30
Total	75,600 =	75,000

*it means to take out the surplus soil to meet the proposed elevation to supply the water excluding the grading/levelling cost in each field lot after this.

2) Case-2 ... the most effective cross-section is given to the external canal with the maximum discharge of 40 l/sec.

(Conditions)

1.	F.S.L	7.76 m
2.	O.S.L	7.71 "
3.	Slope	1 = 1000
4.	Qmax	40 l/sec

1. Head losses between main canal and first box of P23L.

$$Q = 0.16x\sqrt{10.08.h}$$

$$0.040 = 0.16x\sqrt{10.08xh}$$

$$\therefore h = 0.0062 \text{ m}$$

water level at first box of P23L=7.71-0.0062
=7.704 m

2. Water level at second box of P23L.

The water from the first box to second box is given through the submerged orifice 0.345 x 0.345 size.

$$Q = C.A.\sqrt{2.g.H}$$

$$\therefore H = \frac{Q^2}{C^2.A^2.2.g}$$

$$= \frac{0.040^2}{0.6^2 x 0.345^4 x 2 x 9.8} = 0.016 \text{ m}$$

$\therefore$ The water level at second box of P23L

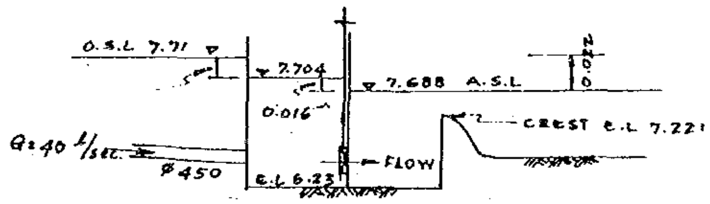
$= 7.704-0.016$

$= 7.688$ m

In fact, this water level is to be an actual supply water level (A.S.L).

The condition of the total head losses between the main canal and second box of P23L is shown as in Fig.-17

Fig.- 17 Total head loss at P23L



3. Most effective cross-section at Block (I)

(Conditions)

1.	A.S.L	7.688 m
2.	Canal type	earth canal
3.	Slope	1:1000
4.	Qmax	40 l/sec
5.	Coefficient of roughness	n=0.03
6.	side gradient	1:0.5

The most effective cross section at Block (I) is calculated by changing the invert width and if the wetted perimeter is given the smallest figure then this is to be the most effective cross-section.

The calculation results are shown as in tab.- 24

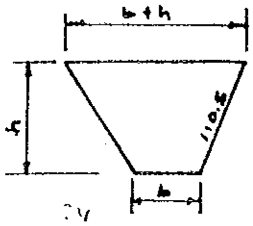


Table-24 Most effective cross section at Block (I) :

b(m)	A(m^2)	Q(l/sec)	P(m)	h(m)	h/b	V(m.s)	Remarks
0.30	0.14	40	0.989	0.308	1.026	0.287	
0.35	0.14	40	<u>0.985</u>	0.284	0.811	0.287	⊘
0.40	0.14	40	0.988	0.263	0.437	0.287	

4. Head losses at the first culvert (ϕ450 x 4,500)

The head losses of the first culvert with ϕ450 diameter and 4.5 m length is calculated as follows:-

$$V = \sqrt{\frac{2.g.h}{1.5+f.\frac{l}{D}}}$$

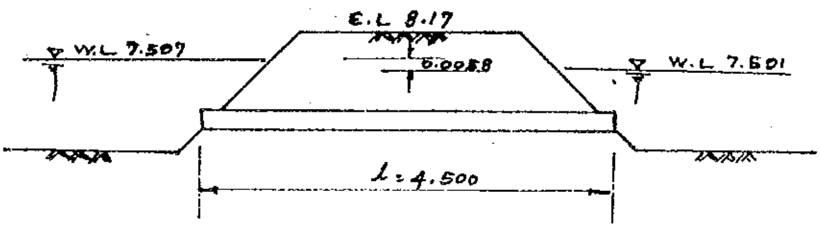
$$= \sqrt{10.78.h}$$

$$Q = A.V$$

$$\therefore\ 0.040 = 0.16\times\sqrt{10.78xh}$$

$$\therefore\ h = 0.0058\ m$$

Fig.-18 Head losses at the first culvert.

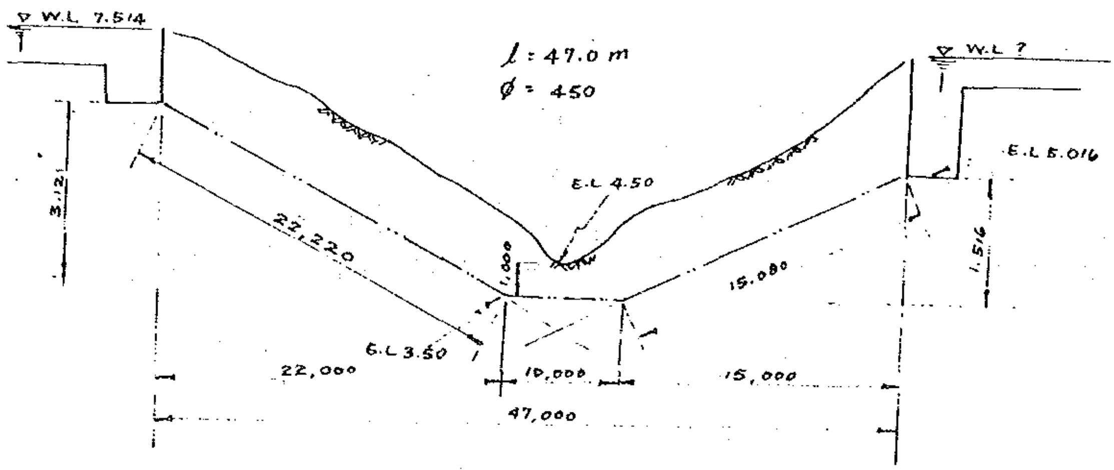


5. Most effective cross-section at Block (II)

This cross-section is the same as the block (I).

6. Head loss at inverted syphon is undetermined since the plan was not available for reference, so there is no choice but to assume the setting condition from the existing site condition.

Fig.-19 Assumed setting condition of the inverted siphon



The head losses which must be calculated are the entrance, outlet and bend losses. The velocity of the water in the pipe is given as follows:-

$$V = \sqrt{\frac{2.g.h}{1.5+f.\frac{l}{D}+fb}}$$

where, fb: coefficient of the head loss of the bend.

$$\tan^{-1}\frac{3.121}{22} = \theta_1 = 8.074^{\circ}$$

$$\tan^{-1}\frac{1.516}{15} = \theta_2 = 5.771^{\circ}$$

According to the "Weisback", if the bended angle (θ) is smaller than 15^{o} at that time the coefficient of the head loss of the bend (fb) is determined by 0.0222.

$$\therefore V = \sqrt{\frac{2x9.8xh}{1.5x0.0318x\frac{47.3}{0.45}+0.0222x2}}$$

$$= \sqrt{4.01xh}$$

$$\theta = A.V$$

$$0.040 = 0.16 \; x\sqrt{4.01xh}$$

$$\therefore h = 0.016 \; m$$

Actually, the head loss at the inverted siphon is found to be 0.025 m., however the former head loss, that is 0.016 m, is the value under good condition that means there is no obstacles in the pipe.

In fact, the velocity in the pipe is not fast enough to clean up the pipe such that sedimentation has occured over a long period thus resulting in the cross-sectional area of flow to be narrower.

Now, the cross-sectional area of flow under existing condition which can be obtained by 0.025 m. head loss is calculated as follows:-

$$V = \sqrt{\frac{2.g.h}{1.5+f.\frac{l}{D}+fb}}$$

$$\text{where,} \quad f = \frac{8.g.n^2}{R^{1/3}} = \frac{0.0244}{D^{1/3}}$$

$$V = \sqrt{\frac{2x9.8x0.025}{1.5+\frac{0.0244x47.3}{D^{4/3}}+0.0222x2}}$$

$$= \sqrt{\frac{0.49}{1.5444+\frac{1.15412}{D^{4/3}}}}$$

$$Q = A.V$$

$$0.040 = \frac{\pi . D^2}{4} . V$$

$$\therefore V = \frac{0.0509}{D^2}$$

$$\therefore \frac{0.0509}{D^2} = \sqrt{\frac{0.49}{1.5444 + \frac{1.15412}{D^{4/3}}}}$$

$$D^{5.3} - 0.00816 . D^{1.3} - 0.0061 = 0$$

$$\therefore D = 0.408 \text{ m.}$$

$$A = \frac{\pi . D^2}{4}$$

$$= \frac{\pi x 0.408^2}{4} = 0.131 \text{ m}^2$$

Consequently, the cross-sectional area of flow under existing condition is determined as 82 percent of the pipe, in other words 18 percent of the pipe is already sedimetted.

The hydraulic condition of the inverted siphon can be arranged as follows:-

Length of the siphon	l = 47.3 m
Size of the pipe	ϕ = 0.45 m
Flow area	A = 0.131 m^2(82%)
Velocity	V = 0.302 m/sec
Hydraulic radius	R = 0.102 m.
Equivalent diameter	D = 0.408 m

Up-stream water level	W.L. 7.514
Down-stream water level	W.L. 7.489

7. Most effective cross-section at Block (III)

This cross-section is the same as well as the block (I).

8. Head losses at the second culvert (Ø450x3,090)

$$V = \sqrt{\frac{2.g.h.}{1.5+f.\frac{l}{D}}}$$

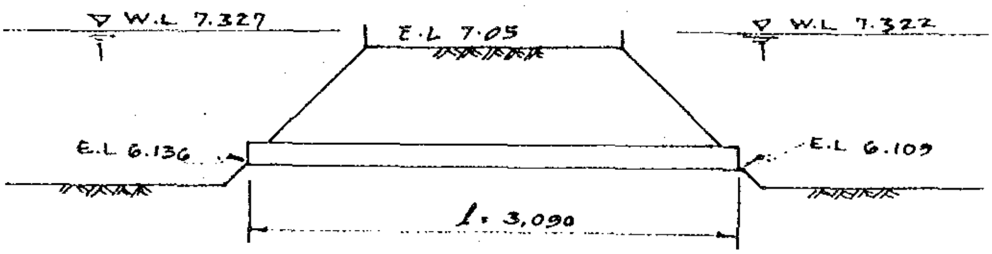
$$= \sqrt{11.41xh}$$

$$Q = A.V$$

$$0.040 = 0.16 \times \sqrt{11.41xh}$$

$$\therefore\ h = 0.0055\ m$$

Fig.- 20 Head losses at the second culvert



9. *Most effective cross-section at Block (N)*

This cross-section is the same as well as the block (I).

Finally, the terminal water level is given by W.L. 7.252 and the level at each existing structure point is shown as in *Fig.*-20

It is clear in case-2 that the cross-sectional area of flow of the most effective cross-section is only to be 0.14 m^2, however the cross-section of the existing external canal is bigger than that, i.e. the invent width of the earth canal part is 0.6 m whilst the timber canal part is 1.0 m wide.

Take into consideration of these facts, it seems very difficult to construct the most effective cross-sectional canal to meet the existing external canal because of the difficulty of the earth work, thus concrete canal or the rip-rap have to be provided as the lining material. Therefore, it is better to consider only minor repair works on the existing external canal by planning to suit the design according to existing condition as far as possible.

Incidentally, if the irrigation canal is lead by 1/3000 slope the irrigable area is to be 4.1 ha and the required land levelling volume is to be 2,100 m^3 as the rest and also the filling back volume to reforme the external canal to the most effective cross-section is around 210 m^3 and this has to be done by hand.

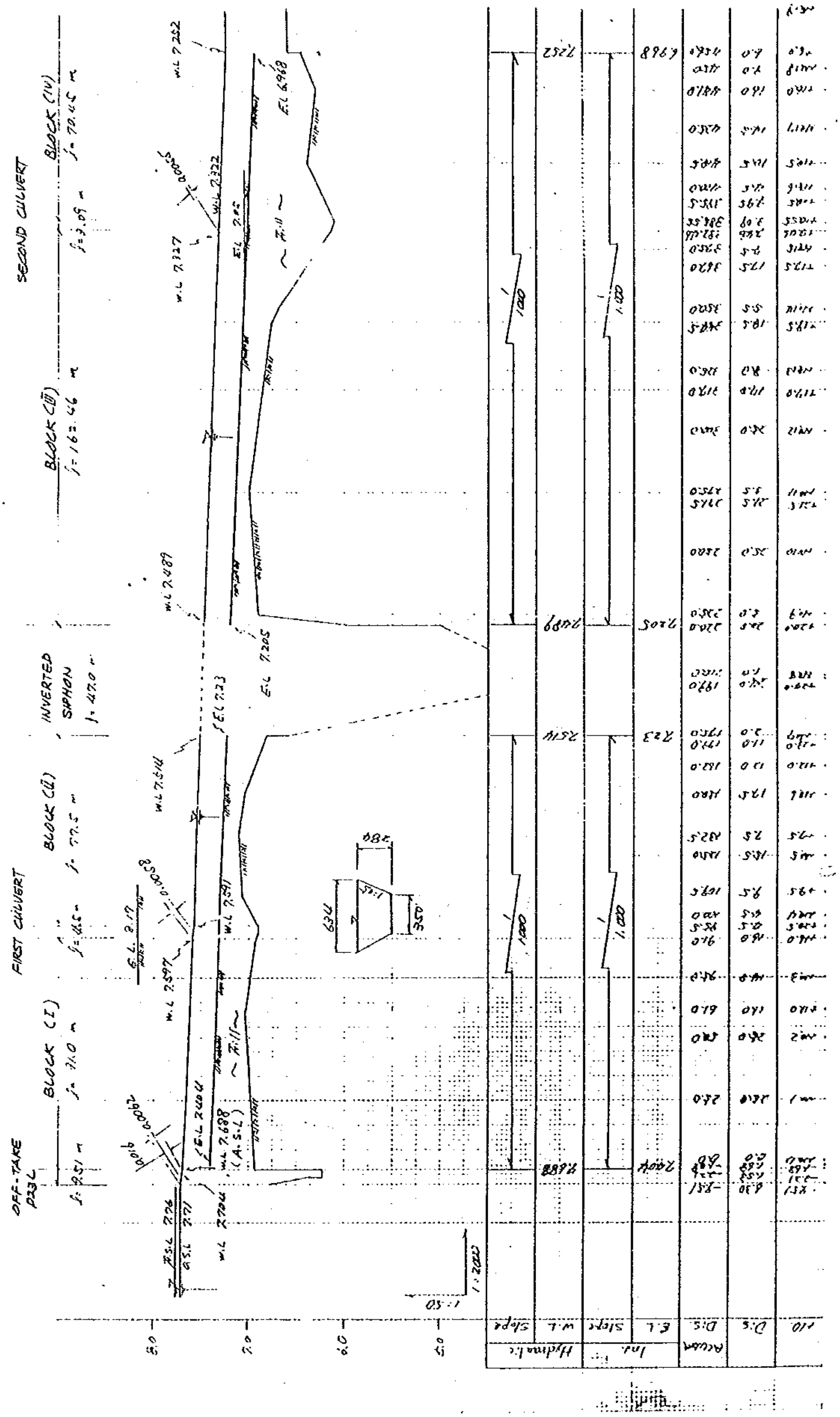
Item	Cost($)	Remarks
Bank	2,478	210x11.80
*Land levelling	13,230	2,100x6.30
Rip-rap	40,872	2.6x400x39.30
Total	56,580 ≒	56,000

*it means to take out the surplus soil to meet the proposed elevation to supply the water excluding the grading/levelling cost in each field lot after this.

3) Case-3 ... Demolishing the external canal with the maximum discharge of 40 l/sec.

In this case, the external canal is planned not to be demolished but to use the existing canal cross-section as far as possible with some minor readjustment.

Fig- 17A TERMINAL WATER LEVEL IN CASE -2



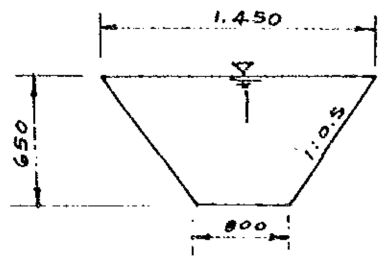
1. Determination of the A.S.L

The A.S.L is given the same as Case 2, that is W.L. 7.688.

2. Head loss at block (I)

The existing cross-section is given as in Fig.-21

Fig.21 Existing canal section at block (I)



$$A = (1.45+0.80) x 0.65 x 0.5 = 0.731\ m^2$$

$$P = 2.253$$

$$R^{2/3} = 0.472$$

the velocity of the canal is to be as follows:-

$$Q = A.V$$

$$0.040 = 0.731 \times V$$

$$\therefore V = 0.055\ m/sec$$

the hydraulic gradient is to be as follows:-

$$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

$$0.055 = \frac{1}{0.03} \times 0.472 x I^{\frac{1}{2}}$$

$$\therefore I = 1/81.831 \doteqdot 1/82,000$$

the velocity of the canal is not fast enough to prevent the sedimentation, however in order to increase the supply water level the hydraulic gradient has to be kept at a gentler slope as far as possible, however maintenance is needed greater than before.

$\therefore$ A.S.L 7.688 m

down stream W.L 2.687 m

3. Head losses at first culvert

the head losses is given the same way as case-2.

$$Q = A.V$$

$$V = \sqrt{10.78.h}$$

$$Q = 0.16 \times \sqrt{10.78.h} = 0.040$$

$$\therefore h = 0.0058 \text{ m}$$

$\therefore$ Up stream W.L 7.687

down stream W.L 7.681

4. Head losses at block (II)

The existing cross-section is given as in Fig.-22

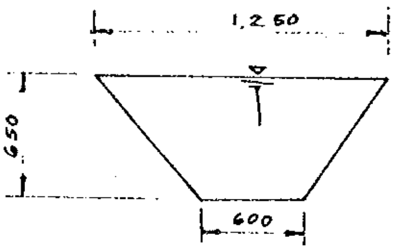


Fig 22

$$A = (1.25+0.60)x0.65x0.5 = 0.601$$

$$P = 2.053$$

$$R^{2/3} = 0.441$$

the velocity and hydraulic gradient are to be as follows:-

$$Q = A.V.$$

$$0.040 = 0.601 \times V$$

$$\therefore V = 0.067 \text{ m/sec}$$

$$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

$$= \frac{1}{0.03} \times 0.441 \times I^{\frac{1}{2}} = 0.067$$

$$\therefore I = \frac{1}{48,000}$$

$\therefore$ up Stream W.L 7.681

down stream W.L 7.679

5. Head losses at the inverted siphon

The diameter of the pipe is given the figure after considering about the sedimentation in the pipe, that is 0.408 m.

$$V = \sqrt{\frac{2.g.h}{1.5+f.\frac{l}{D}+fb}}$$

$$= \sqrt{\frac{2 \times 9.8 \times h}{1.5+0.0329 \times \frac{47.3}{0.408}+0.0222 \times 2}} = \sqrt{3.658.h}$$

$$Q = A.V$$

$$0.040 = \frac{\pi \times 0.408^2}{4} \times \sqrt{3.658 \times h}$$

$$\therefore h = 0.025 \text{ m}$$

up stream W.L. 7.679

down stream W.L. 7.654

6. Head losses at block (III)

The existing cross-section is the same as block (II).

$\therefore$ up stream W.L 7.654

down stream W.L 7.651

7. Head losses at the second culvert

$$Q = A.V$$

$$0.040 = 0.16x \sqrt{11.41xh}$$

$$\therefore h = 0.0055$$

$\therefore$ up stream W.L 7.651

down stream W.L 7.646

the difference between the hydraulic level and existing farm road is to be 0.601 m at this point and also the proposed bed level and existing one is to be 0.865 m.

It is clear that only the certain portion of this second culvert, the existing bed level is going down in order to cross the farm road. Through, the F.S.L is given high enough to supply the water into the benefitted area but because of this matter the head losses is lost a lot unexpectedly.

This is the reason why only a small area can be supplied the water.

8. Head losses at block (N)

In this block, the proposed hydraulic level is going to be higher than existing water level i.e. approximately 0.60 m.

Therefore for this block it is required to replace the existing canal with concrete lined canal.

Particularly, it is better to consider about this block to be the transition to the field quaternary canal.

The assumed cross-section of the quaternary canal is to be 0.5 m, invert width with water depth between 0.25 to 0.30 m.

Considering about the velocity and it's hydraulic gradient of the quaternary canal, the optimum size and conditions are calculated as follows:-

(conditions)

1.	invert width	$B = 0.50$ m
2.	water depth	$H = 0.30$ m
3.	maximum discharge	$Q = 0.040\ m^3/sec$

$$Q = A.V$$

$$0.040 = 0.5 \times 0.30 \times V$$

$$\therefore V = 0.267\ m/sec$$

$$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

$$= \frac{1}{0.015} \times 0.265 \times I^{\frac{1}{2}} = 0.267$$

$$\therefore I = \frac{1}{4.378} = \frac{1}{4,400}$$

If the elevation of the highest field lot is assumed by E.L 7.30, the hydraulic gradient is to be:-

up stream	W.L	7.646 m
highest field lot	W.L	7.40 m (including 10 cm height of standing water layer)

length of the canal $l = 800$ m

$$\therefore \frac{7.646-7.40}{800} \doteqdot \frac{1}{3,200}$$

and the height of the water depth is to be:-

$$\frac{Q}{A} = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

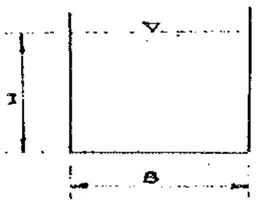
$$\frac{0.040}{0.5.H} = \frac{1}{0.015}.\left(\frac{0.5.H}{2.H.+0.5}\right)^{2/3}.\left(\frac{1}{3200}\right)^{\frac{1}{2}}$$

$$\therefore H^{5/2} - 0.0708.H - 0.0178 = 0$$

$$\therefore H = 0.2665\ m$$

Consequently, the optimum canal size is studied by changing the velocity from 0.25 m to 0.30 m together with the hydraulic gradient between the range of 1:2,000 to 1:5,000

By the calculation, the HP34C is used and it's formula is shown as below:-



$$A = B.H$$

$$P = 2H+B \qquad \text{-----} \quad (1)$$

$$R = \frac{B.H}{2H+B}$$

$$Q = A.V$$

$$= B.H.V \qquad \text{-----} \quad (2)$$

$$\therefore H = \frac{Q}{B.V}$$

substitute (2) for (1)

$$\therefore P = \frac{2.Q}{B.V}+B = \frac{2Q+B^2.V}{B.V}$$

$$\therefore R = \frac{A}{P} = \frac{Q.B}{2.Q+B^2.V} \qquad \text{-----} \quad (3)$$

$$V = \frac{1}{n} x R^{2/3}.I^{\frac{1}{2}} = \frac{R^{2/3}.I^{\frac{1}{2}}}{n}$$

$$\therefore R^{2/3} = \frac{n.V}{I^{\frac{1}{2}}}$$

$$2/3 \log R = \log \frac{n.V}{I^{\frac{1}{2}}}$$

$$\log R = \frac{3}{2}.\log \frac{n.V}{I^{\frac{1}{2}}}$$

$$\therefore R = 10^{3/2(\log \frac{n.V}{I^{\frac{1}{2}}})} \qquad \text{-----} \quad (4)$$

$$R(2Q+B^2.V) = Q.B$$

$$B^2 - \frac{Q}{V.R.}.B + \frac{2.Q}{V} = 0$$

$$\therefore\ B = \frac{\frac{Q}{V.R} \pm \sqrt{\left(\frac{Q}{V.R}\right)^2 - 4\times\frac{2Q}{V}}}{2}$$

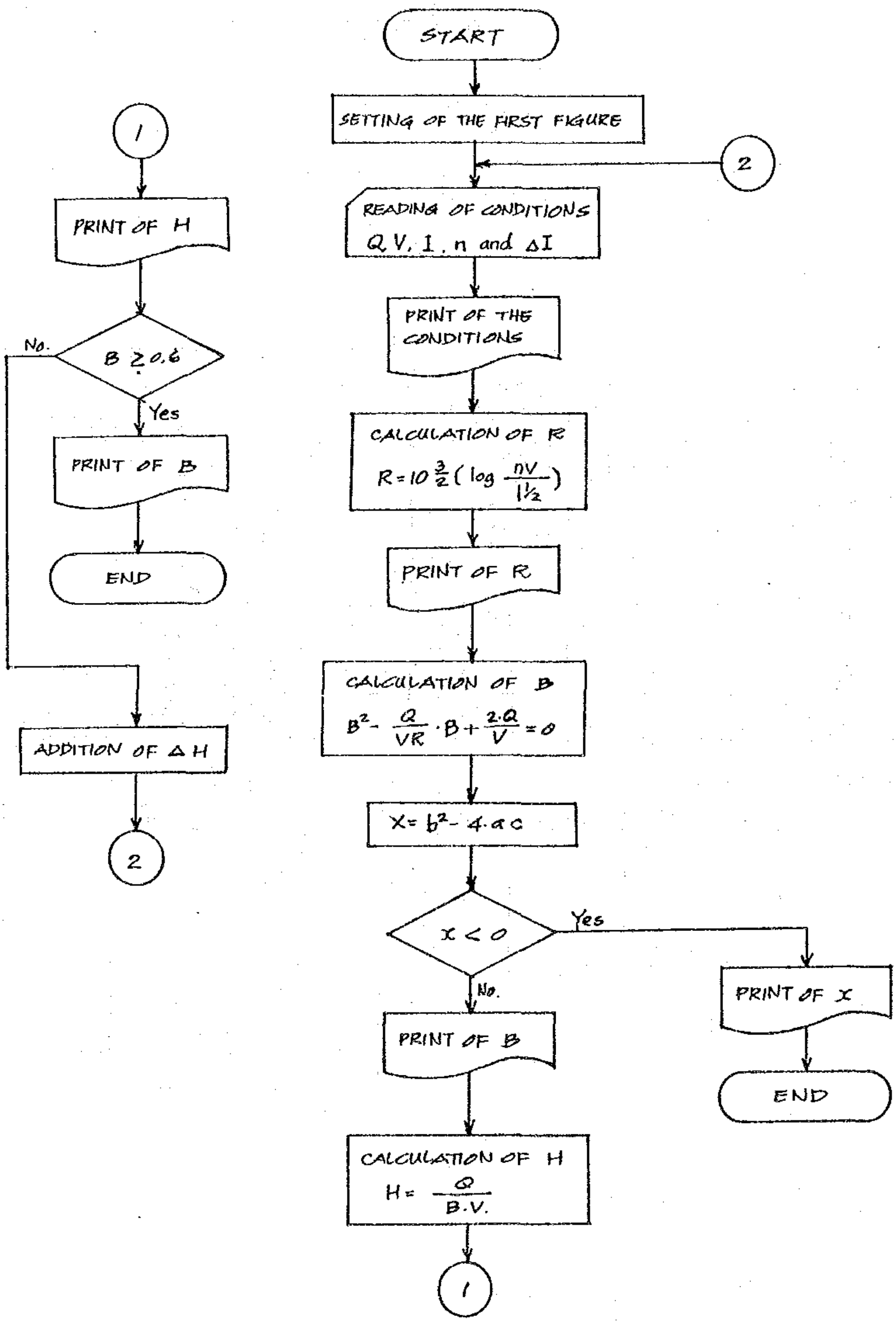
$$= \frac{\frac{Q}{V\times10^{3/2(\log\frac{n.V}{I^{\frac{1}{2}}})}} \pm \sqrt{\left(\frac{Q}{V\times10^{3/2(\log\frac{n.V}{I^{\frac{1}{2}}})}}\right)^2 - \frac{8.Q}{V}}}{2} \quad \text{-----} \quad (5)$$

substitute (5) for (2), and then the water depth in the canal can get.

the systematic flow chart is shown as in Fig.- 24A

the results of the calculation is given as in Tab.- 25.

Fig - 24A: SYSTEMATIC FLOW CHART TO IDENTIFY THE OPTIMUM CANAL SECTION.

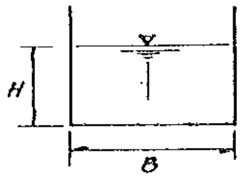


PROGRAM NAME CALCULATION OF OPTIMUM CANAL SIZE

PROGRAM NO

CODING DATE , DEC, '82

1) Schematic drawing



2) Purpose

To identify the canal size depends on the changes of Q·V·I and n

3) Exercise

4) Program list

Coded by :

00	Key. on		16	PSE		32	÷	
01	h. LBL A		17	RCL 5		33	f. LOG	
02	RCL 2		18	PSE		34	3	
03	RCL 4		19	PSE		35	×	
04	−		20	PSE		36	2	
05	h. LBL B		21	RCL 3		37	÷	
06	RCL 4		22	PSE		38	STO 6	
07	+		23	PSE		39	10	
08	STO 5		24	PSE		40	↑	
09	RCL 0		25	RCL 3		41	RCL 6	
10	PSE		26	RCL 1		42	h. y^x	
11	PSE		27	×		43	STO 7	
12	PSE		28	↑		44	PSE	
13	RCL 1		29	RCL 5		45	PSE	
14	PSE		30	h. 1/x		46	PSE	
15	PSE		31	f $\sqrt{x}$		47	RCL 0	

5) Operation step

Step	Contents	Input	Operation	Output	Step	Contents	Input	Operation	Outp
01					11				
02					12				
03					13				
04					14				
05					15				
06					16				
07					17				
08					18				
09					19				
10					20				

48	↑	78	PSE	108	
49	RCL 1	79	RCL 0	109	
50	RCL 7	80	↑	110	
51	×	81	RCL 8	112	
52	÷	82	RCL 1	113	
53	↑	83	×	114	
54	RCL 0	84	÷	115	
55	↑	85	STO 9	116	
56	RCL 1	86	PSE	117	
57	RCL 7	87	PSE	118	
58	×	88	PSE	119	
59	÷	89	RCL 8	120	
60	2	90	↑	121	
61	h. y^x	91	0.6	122	
62	↑	92	f $x \leq y$	123	
63	RCL 0	93	GTO 0	124	
64	8	94	0.6	125	
65	×	95	−	126	
66	RCL 1	96	RCL 5	127	
67	÷	97	GTO B	128	
68	−	98	h.LBL 0	129	
69	g. $x<0$	99	h.RTN	130	
70	GTO 0	100		131	
71	f $\sqrt{x}$	101		132	
72	+	102		133	
73	2	103		134	
74	÷	104		135	
75	STO 8	105		136	
76	PSE	106		137	
77	PSE	107		138	

6) Register List

R0	Q	R1	V	R2	J	R3	n
R4	ΔI	R5	I	R6	$\frac{3}{2}\log(nV/I^{\frac{1}{2}})$	R7	R
R8	B	R9	H	R.0		R.1	
R.2		R.3		R.4		R.5	
R(16)		R(17)		R(18)		R(19)	
R(20)		R(21)		R(22)		R(23)	
R(24)		R(25)		R(26)		R(27)	
R(28)		R(29)					

Tab.- 25 Calculation results of the optimum canal size
For Q =40 l/s.

$Q(m^3/sec)$	V(m/sec)	I	B(m)	H(m)	$A(m^2)$	Remarks
0.040	0.25	$\frac{1}{5,000}$	0.433	0.369	0.160	
0.040	0.26	$\frac{1}{4,690}$	0.504	0.305	0.154	
0.040	0.27	$\frac{1}{4,250}$	0.503	0.294	0.148	
0.040	0.28	$\frac{1}{3,860}$	0.498	0.287	0.143	
0.040	0.29	$\frac{1}{3,520}$	0.497	0.277	0.138	
0.040	0.30	$\frac{1}{3,220}$	0.500	0.266	0.133	

As the results, the most optimum canal size is supposed to be the last one, where the invert width is to be 0.5 m and the water depth is to be 0.266 m and this is the smallest canal section.

therefore, the water level at the end of the external canal is to be as follows:-

$$7.646 - \frac{70.45}{3.220} \fallingdotseq 7.624 \text{ m.}$$

Consequently, the construction cost is amounted as follows:-

Items	Cost($)	Remarks
Demolishing the canal	30,275	(2.253x91.0+2.053x (77.5+162.46)+ 1.032x70.45)x39$^{\underline{30}}$
Bank	1,020	0.8x0.6x180x11.$^{\underline{80}}$
Total	31,295 ≒ 31,000	

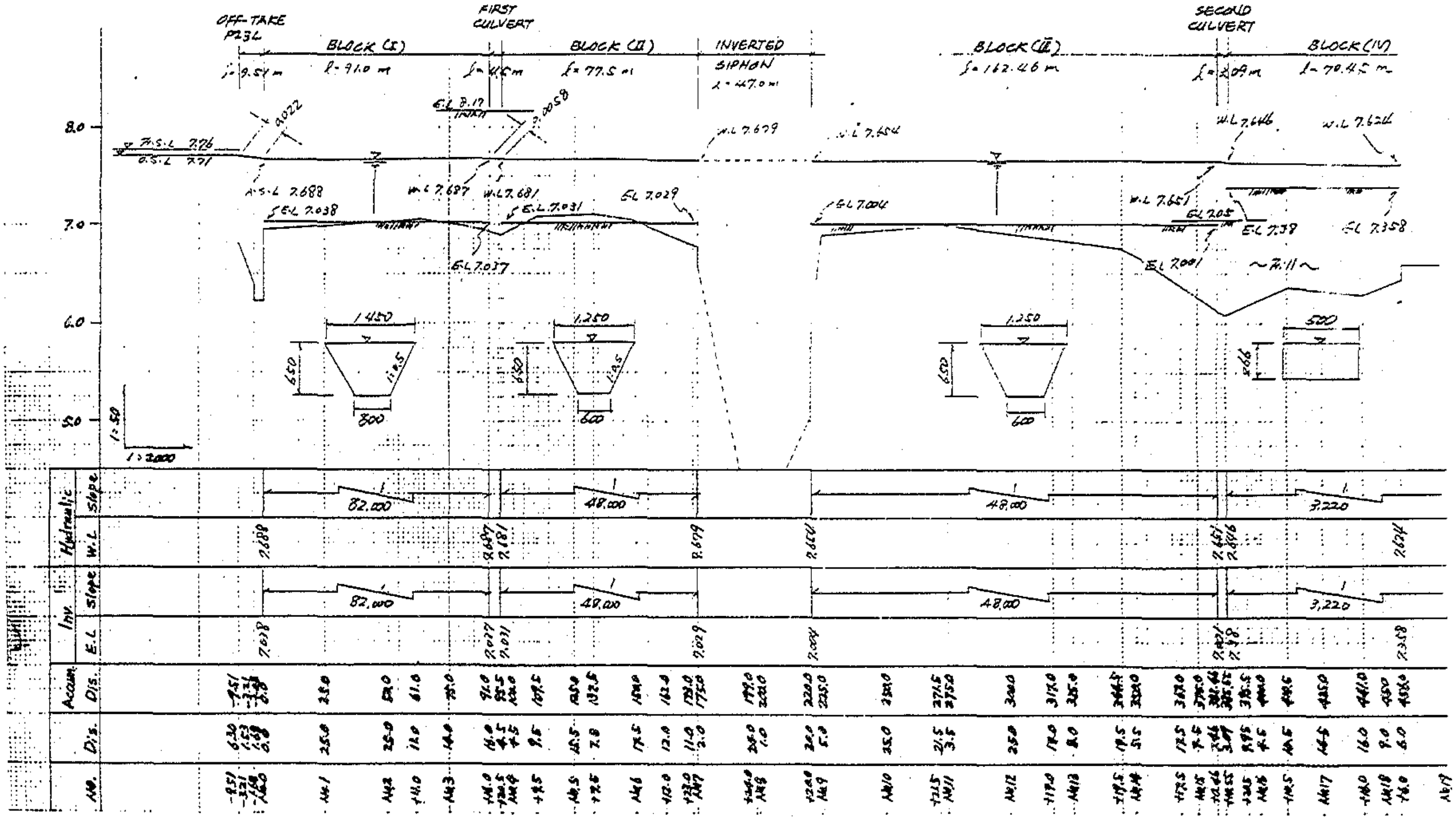
The water level at each point is shown as in Fig.-25A

4) Case-4 ... Demolishing the external canal with the maximum discharge of 50 l/sec.

The aspects of this case is also the same as the case-3, however taking into consideration about the existing discharge which is obtained at the P23L by using a current meter is 42 l/sec, the maximum discharge should be studied in order to get greater value than that of 42 l/s.

So, the assumed maximum discharge is given as 50 l/sec, so the terminal water level of the external canal can be identified.

Fig. - 25A : TERMINAL WATER LEVEL IN CASE - 3



1. Determination of the A.S.L

F.S.L	W.L	7.76
O.S.L	W.L	7.71

The head losses between the main canal and off-take P23L is calculated as follows:-

$$V = \sqrt{\frac{2.g.h}{1.5+f\frac{1}{D}}}$$

$$f = \frac{8.g.n^2}{8^{1/3}} = \frac{8x9.8x0.014^2}{(\frac{0.45}{4})^{1/3}} = 0.0318$$

$$V = \sqrt{\frac{2x9.8xh}{1.5+0.0318x\frac{6.30}{0.45}}} = \sqrt{10.08.h}$$

$$Q = A.V$$

$$0.050 = \frac{\pi.0.45^2}{4}.\sqrt{10.08.h}$$

$$0.050 = 0.16.\sqrt{10.08.h}$$

$$\therefore h = 0.0097 \text{ m}$$

therefore, the water level at the first box is to be W.L 7.700.

The head loss of the submerged orifice is calculated as well.

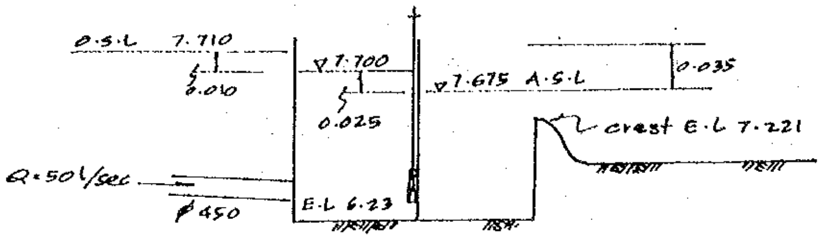
$$Q = C.A.\sqrt{2.g.H}$$

$$\therefore H = \frac{Q^2}{C^3.A^2.2.g}$$

$$= \frac{0.050^2}{0.6^2x0.345^4x2x9.8} = 0.025 \text{ m}$$

then the water level at the second box is to be W.L 7.675 and this elevation also to be the A.S.L.

Fig.- 23. Total head loss at P23L



2. Head loss at block (I)

The average cross-section is the same as the case-3.

$$A = 0.731 \ m^2$$

$$P = 2.253$$

$$R^{4/3} = 0.472$$

$$Q = A.V$$

$$0.050 = 0.731.V$$

$$\therefore V = 0.068 \ m/sec$$

$$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

$$= \frac{1}{0.03}\times 0.472\times I^{\frac{1}{2}} = 0.068$$

The velocity is not fast enough to prevent the sedimantation, thus hindering the performance of the canal

$$\therefore I = \frac{1}{53,533} = \frac{1}{54,000}$$

∴ Up stream W.L 7.675

down stream W.L 7.673

3. Head losses at the first culvert

$Q = A.V$

$V = \sqrt{10.78\ h}$

$Q = 0.16x\sqrt{10.78xh} = 0.050$

$\therefore h = 0.0091$ m

$\therefore$ up stream W.L 7.673

down stream W.L 7.664

4. Head losses at block (II)

The average cross-section is the same as the case-3.

$A = 0.601$

$P = 2.053$

$R^{4/3} = 0.441$

$Q = A.V$

$0.050 = 0.601xV$

$\therefore V = 0.083$ m/sec

$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$

$= \frac{1}{0.03}x0.441xI^{\frac{1}{2}} = 0.083$

$\therefore I = \frac{1}{31,376} = \frac{1}{31,000}$

$\therefore$ up stream W.L 7.664

down stream W.L 7.662

5. Head losses at the inverted siphon

$V = \sqrt{3.658.h}$

$Q = A.V$

$0.050 = 0.131 x V$

$\therefore V = 0.382$ m/sec

$\therefore h = 0.040$ m

∴ up stream	W.L	7.662
down stream	W.L	7.622

6. Head losses at block (III)

The average cross-section is the same as well as block (V)

$\therefore I = 1/31,000$

$V = 0.083$ m/sec

∴ up stream	W.L	7.622
down stream	W.L	7.617

7. Head losses at second culvert

$Q = A.V$

$0.050 = 0.16 x \sqrt{11.41 x h}$

$\therefore h = 0.009$ m

∴ up stream	W.L	7.617
down stream	W.L	7.608

8. Head losses at block (N)

If the optimum canal section which is 0.5 m invert width with 0.3 m water height is given at that time the hydraulic gradient is to be:-

$$Q = A.V$$

$$0.050 = 0.5 \times 0.3 \times V$$

$$\therefore V = 0.333 \text{ m/sec}$$

$$V = \frac{1}{n}.R^{2/3}.I^{\frac{1}{2}}$$

$$0.333 = \frac{1}{0.03} \times 0.265^{2/3} \times I^{\frac{1}{2}}$$

$$\therefore I = \frac{1}{2,815} \doteqdot \frac{1}{2,800}$$

On the other hand, when the highest lot elevation is assumed as E.L 7.30 the hydraulic elevation is to be:-

up stream W.L 7.608

down stream W.L 7.40 (including 10 cm standing water layer).

length of the canal l=800 m

$$\therefore \frac{7.608-7.40}{800} = \frac{1}{3,846} \doteqdot \frac{1}{4,000}$$

In this case, the cross-section is identified as follows:-

$$Q = A.V. = B.H.V$$

$$\therefore V = \frac{0.050}{0.5.H} = \frac{1}{0.015} \times \left(\frac{0.5 \times H}{2 \times H + 0.5}\right)^{2/3} \times \left(\frac{1}{4,000}\right)^{\frac{1}{2}}$$

$$\therefore H^{5/2} - 0.1168.H - 0.0292 = 0$$

$$\therefore H = 0.344$$

$$V = 0.29 \text{ m/sec}$$

Consequently, the optimum cross-section is determined by changing the velocity from 0.25 m/sec to 0.35 m/sec with the hydraulic gradient between the range of 1:2500 to 1:5,000.

The calculation results are shown as in tab.- 26.

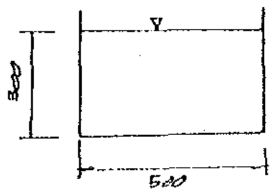
$Q(m^3/sec$	V(m/sec)	I	B(m)	H(m)	$A(m^2)$	Remarks
0.050	0.25	$\frac{1}{5,000}$	0.363	0.551	0.200	exception
0.050	0.26	$\frac{1}{5,000}$	0.427	0.451	0.193	
0.050	0.27	$\frac{1}{4,830}$	0.501	0.369	0.185	
0.050	0.28	$\frac{1}{4,400}$	0.500	0.357	0.179	
0.050	0.29	$\frac{1}{4,030}$	0.504	0.342	0.172	
0.050	0.30	$\frac{1}{3,700}$	0.509	0.327	0.166	
0.050	0.31	$\frac{1}{3,390}$	0.501	0.322	0.161	
0.050	0.32	$\frac{1}{3,120}$	0.498	0.314	0.156	
0.050	0.33	$\frac{1}{2,880}$	0.497	0.305	0.152	
0.050	0.34	$\frac{1}{2,670}$	0.507	0.290	<u>0.147</u>	smallest
0.050	0.35	Nil				

Table 26. Calculation results of optimum canal size Q = 50 l/s

the optimum canal section can be obtained when velocity is 0.34 m/sec and its hydraulic gradient is 1:2670, however for the construction it's better to round up the hydraulic gradient figure.

therefore, the hydraulic gradient for the optimum canal section is determined by 1:2,700.

(checking)



$$A = 0.5 \times 0.3 = 0.15 \ m^2$$

$$P = 0.3 \times 2 + 0.5 = 1.1 \ m$$

$$R^{2/3} = (\frac{0.15}{1.1})^{2/3} = 0.265$$

$$V = \frac{1}{0.015} \times 0.265 \times (\frac{1}{2,700})^{\frac{1}{2}}$$

$$= 0.340 \ m/sec$$

$$\therefore \ Q = 0.15 \times 0.34 = 0.050 \quad \ldots \quad O.K$$

the terminal water level at the external canal is to be as follows:-

$$7.608 - \frac{70.45}{2,700} = 7.582 \ m$$

Consequently, the construction cost is amounted as follows:-

Items	Cost($)	Remarks
Demolishing the canal	30,464	2.253x91.0+2.053x (77.5+162.46)+ 1.1x70.45)x39.30
Bank	1,020	
Total	31,484 ÷	31,000

the water level at each point is shown as in Fig.- 25B

the comparison of the construction cost among the case studies are arranged as in tab.-27.

Tab. 27. Arrangement table of the construction cost.

Case	Cost($)	Remarks
Case-1	75,000	without any demolishing
Case-2	56,000	most effective cross-section
Case-3	31,000	demolishing the canal, Q = 40 l/sec
Case-4	31,000	demolishing the canal, Q = 50 l/sec

According to this result, the most effective case is to be No. 4 which can take 50 l/sec maximum discharge and can demolish the external canal.

When the maximum discharge is given by 50 l/sec the presaturation period is estimated as fllows:-

$$Qp = q \times A \times D \quad (m^3)$$

where Qp : Total presaturation water

q : water duty per ha. (1.18 l/sec/ha)

A : total benefitted area (11.2 ha)

D : period for presaturation (30 days)

$$\therefore\ Qp = 1.18 \times 0.001 \times 11.2 \times 10{,}000 \times 30 \times 86400$$

$$= 34{,}256\ m^3$$

$$T = 34.256/(86400 \times 0.050)$$

$$= 7.9$$

$$\doteqdot 8 \text{ days}$$

It is supposed to be a limited period to finish the presaturation work within a eight days so that the maximum discharge, of 50 l/sec, is not necessary to be increased.

3) Conveyance loses from the external canal

The maximum discharge is given by 50 l/sec at the moment, however it is sure that all of the discharge is not be able to supply to the field. In another words, the conveyance losses from the external canal should be considered prospectively.

E.A Moritz's formula

$$S = 0.0619.C.\sqrt{\frac{Q}{V}}$$

where, S : volume of seepage loss (m^3/sec/km)

Q : discharge (m^3/sec)

V : velocity (m^3/sec)

C : coefficiency of seepage loss (m/day/km)

(Conditions)

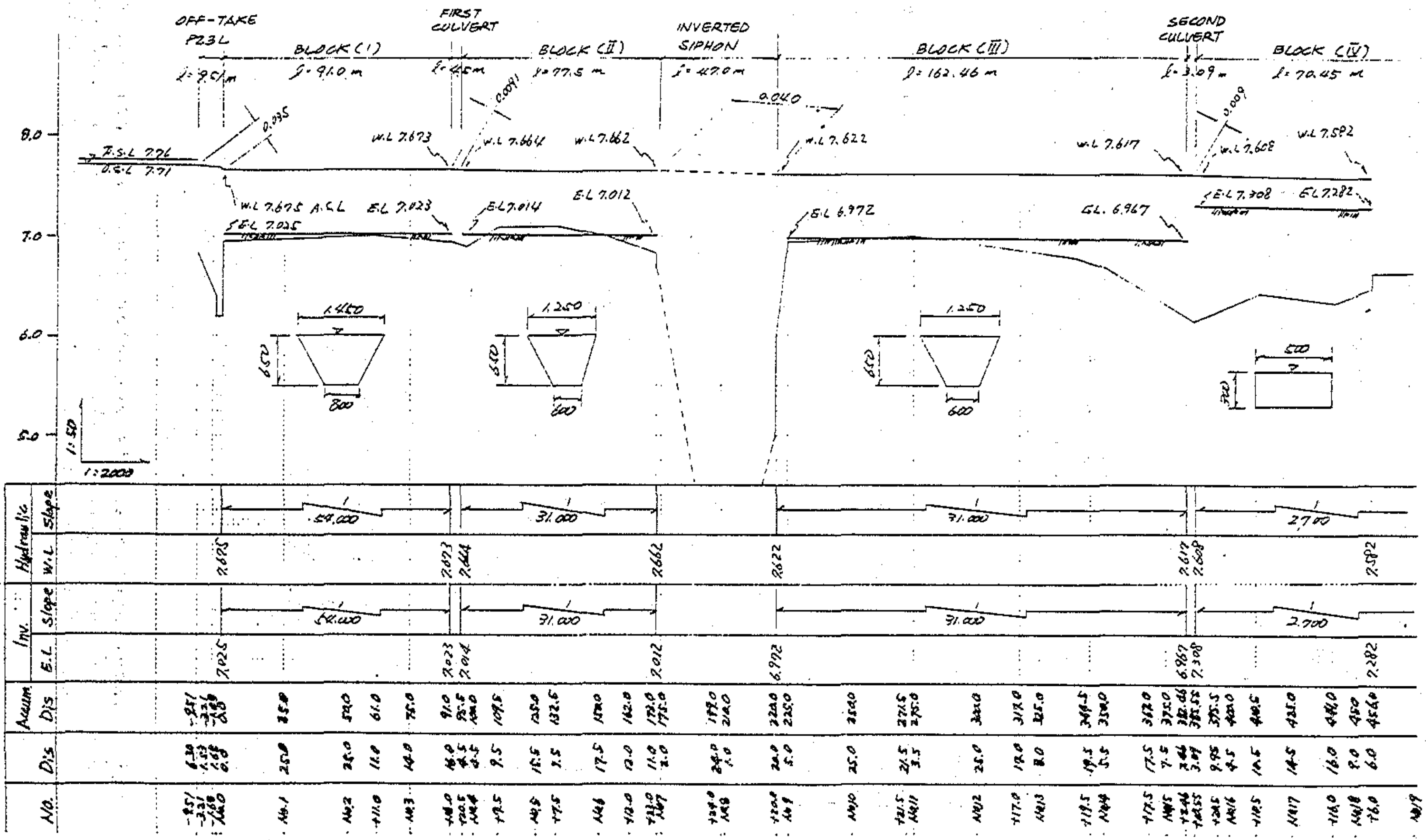
1. Q = 0.050 m^3/sec
2. V = 0.083 m/sec
3. C = 0.08 (clay)
4. L = 456 m

$$\therefore\ S = 0.0619 \times 0.08 \times \sqrt{\frac{0.050}{0.083}} \times 0.456$$

$$= 0.0018\ m^3/sec$$

this conveyance losses is amounted about 3.6 percent of the maximum discharge so that it must be considered this losses when the presaturation period are going to calculate.

Fig - 25B TERMINAL WATER LEVEL IN CASE-4



4) Head loss of the submerged orifice at P23L

The head loss of the submerged orifice at P23L was found to be 0.09 m., however at that time the orifice was not opened fully but only about 0.15 m.

this head loss at 0.16 m height can be proved as below:-

$$Q = C.A.\sqrt{2.g.h}$$

where,

$Q = 0.042\ m^3/sec$ ... the discharge when the investigation was done

$$\therefore 0.042 = 0.6 \times 0.345 \times 0.15 \times \sqrt{2 \times 9.8 \times h}$$

$$\therefore h = 0.09\ m$$

If the submerged orifice is opened fully, then the head losses is going to reduce the height up to 0.016 m. as mentioned in case-2.

In another words, the difference of the head losses between the former and the latter is to be 0.074 m. height, such that it can be said that the water level can be increased up to 0.074 m by proper gate operation.

3. DESIGN ON THE FIELD IRRIGATION CANAL

1) Fixture of the terminal water level of the external canal to the field irrigation canal.

1)-1 Layout plan

There are two types of layout plans being considered as shown in Fig.- 24 and Fig.-.25.

Take into consideration about the existing gradient of the area, that is sloping from south to north east with 1:500 slope, the irrigation canal should be provided to the highest portion as shown in Fig.-21 and Fig. 22.

The widest width of the area is to be 140 m so that the long side of the field lot is supposed to be very long for the water management, and from the certain portion the benefitted area is separated into two area.

Therefore, the main farm road which is provided together with irrigation and drainage canal is planned to run through the center of the area like in case-1.

The density of the on farm facilities is to be as follows:-

Items	Density (m/ha)
irrigation canal	142.9
drainage canal	132.6
farm road	184.8

On the other hand, this project is going to include the land grading and levelling so that the elevation in each field lot is to be the same height.

This means, it is not necessary to consider about the land gradient so much as the conventional lot-to-lot irrigation system, however it goes without saying that if the land gradient is found to be more than 1:100, it must be careful to do the land grading and/or levelling unless otherwise the earth work cost is increased so much and becomes not economical.

The main farm road run through the center of the area together with the irrigation canal as the case-1.

The density of the on farm facilities is to be as follows:-

Items	Density (m/ha)
irrigation canal	112.5
drainage canal	131.1
farm road	128.0

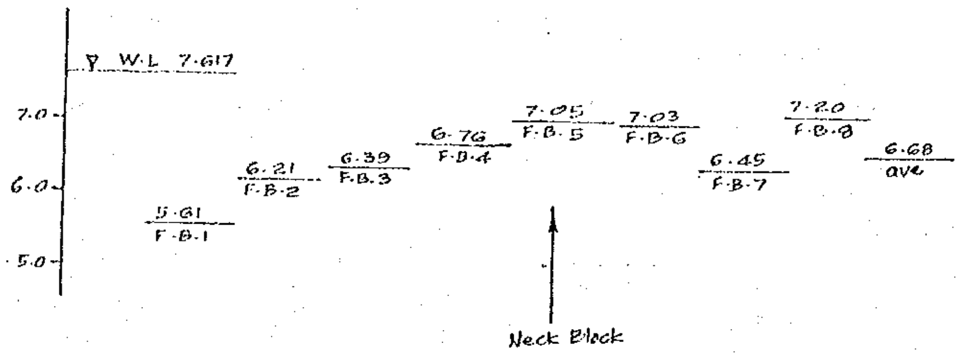
Consequently, case-2 can save a lot of construction cost than case-1.

Therefore, the terminal water level of the external canal is determined to be W.L 7.617 which is the water level at the up stream of the second culvert.

Take into consideration of the average elevation in each field block, the block No. 1, 2, 3 and 4 is located at a very low area and block No. 5 is in high area.

This means, it might be said that block No.5 is to be a neck block, so that the irrigation canal should be provided by the pipe line to those field block No. 1, 2 and 3.

Fig.- 24 Longitudinal section of the benefited area.



The distance from the end of the external canal to the field block No. 5 is about 280 m. and the field irrigation canal size is supposed to be 300 x 500 and the hydraulic slope is to be 1:2700.

Fig. — 24: LAYOUT PLAN (CASE-1)

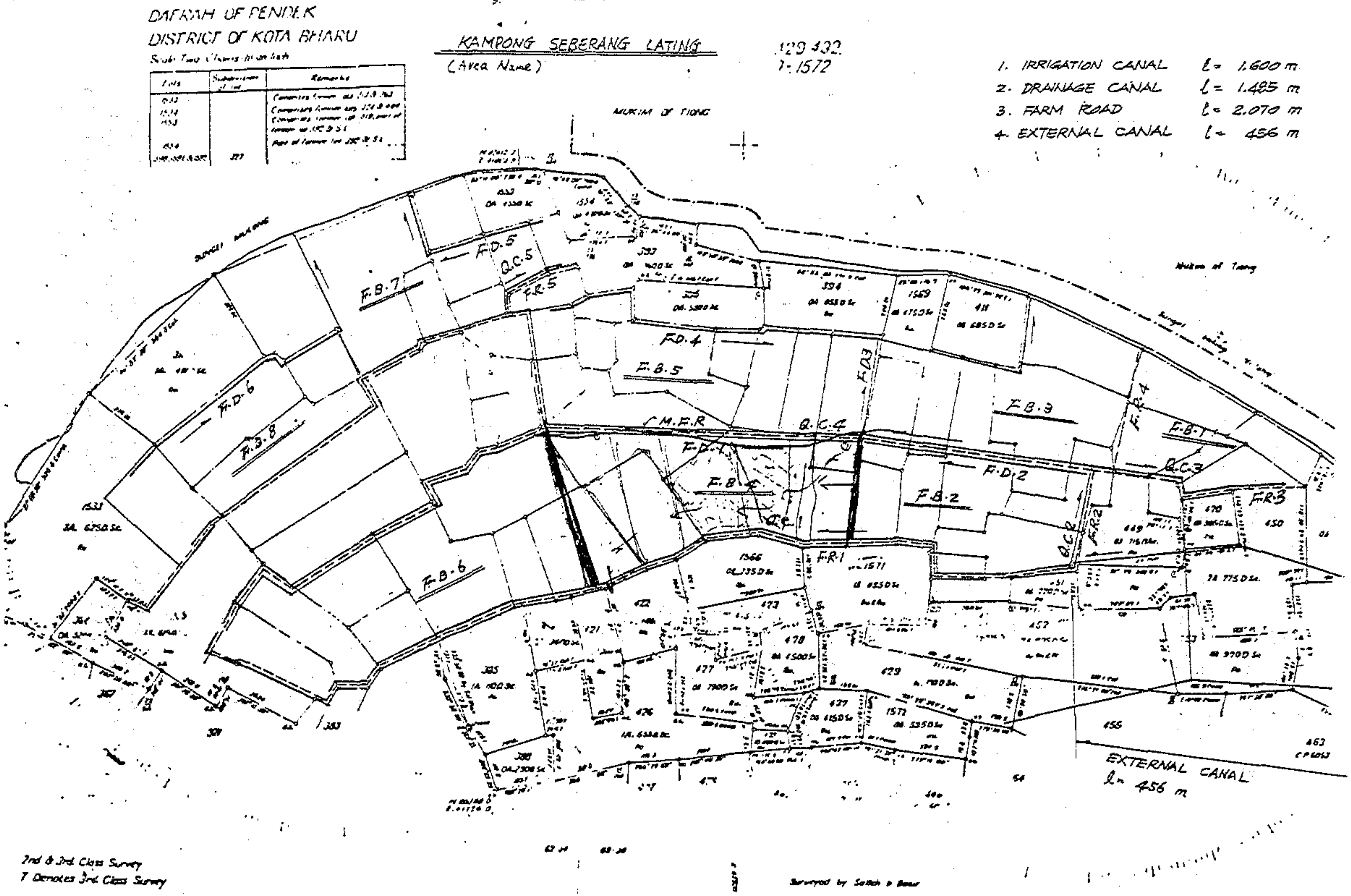


Fig-25 : LAYOUT PLAN (CASE-2)

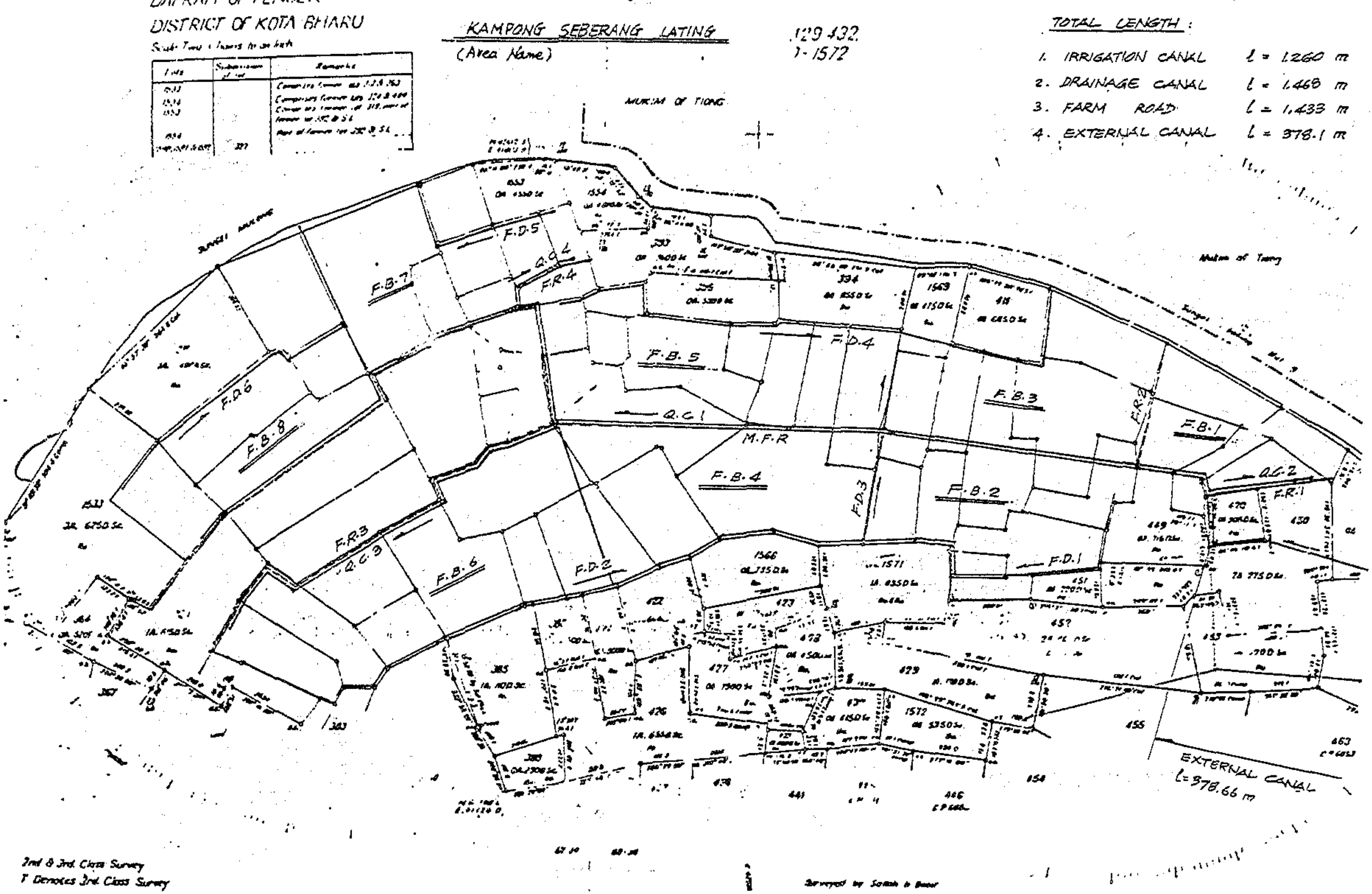
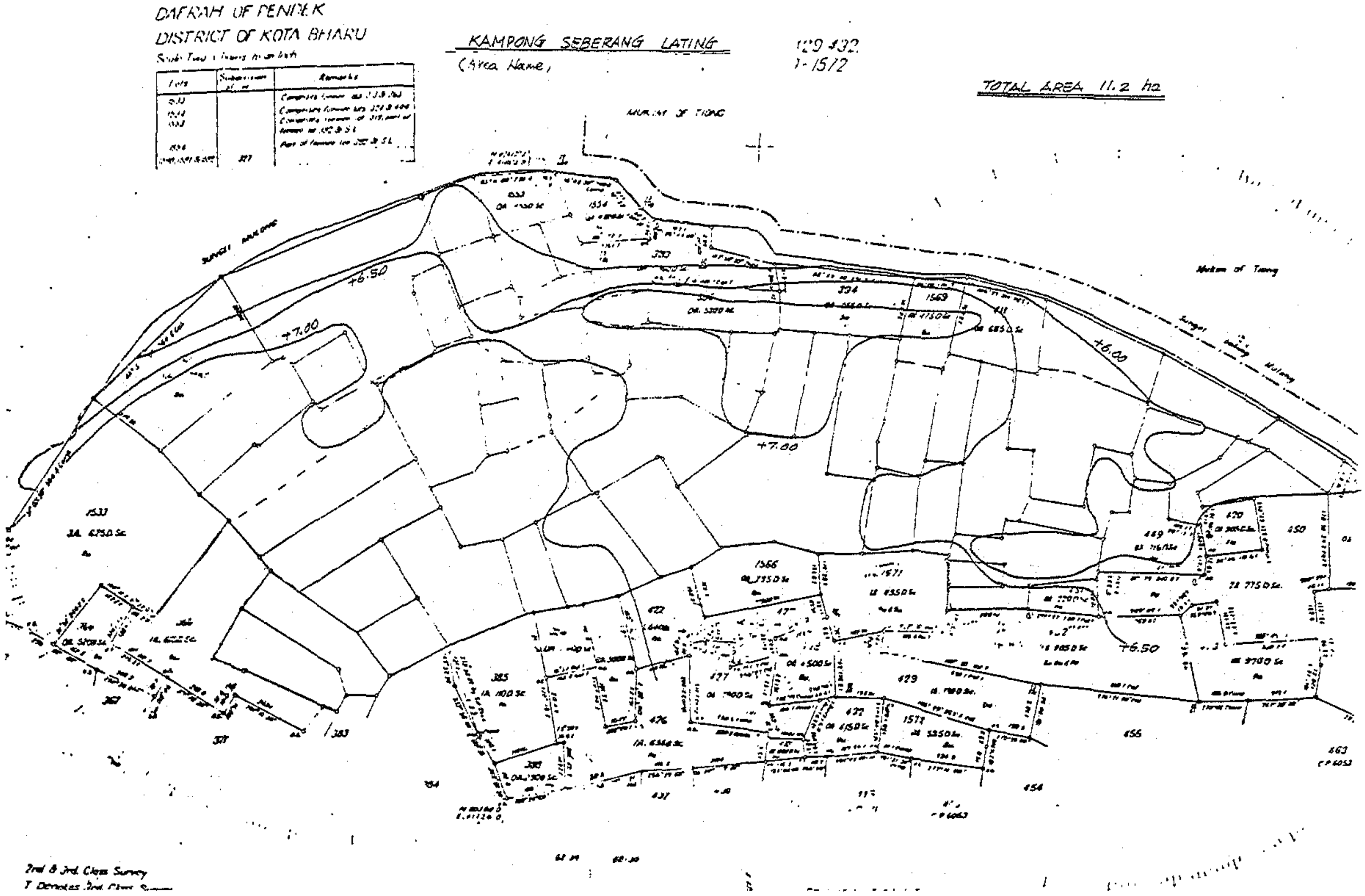


Fig - 26 CONTOUR MAP



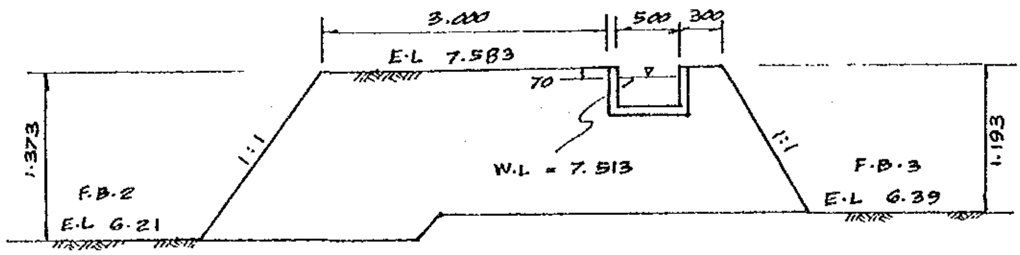
If the irrigation canal is lead by the open channel therefore, the water level in the field block No. 5 is supposed to be as follows.

$$W.L\ 7.617 - \frac{280}{2.700} = 7.513$$

It means if the irrigation canal is to be of the open channel, wider area would be required for the facilities and this not economical.

The proposed cross section for the farm road and irrigation canal is shown as in Fig.- 25.

Fig.- 25 Proposed cross-section of the M.F.R and Q.C. 1



Showing as in Fig.-25 the difference between the top level of the farm road and field lot is to be over 1.0 m. such that it is very difficult for the agricultural machinery to go into each field lot.

Normally the difference in height is given as 0.5 m for the main farm road and 0.3 m for the branch farm road.

Consequently, the pipe line system should be taken to supply the irrigation water to those low area.

2) Measurement device

Generally speaking, this matter is the very important device to control the supplying water however, though the measurement ruler is provided at the existing off-take P23L, this measurement device is not functioning well due to the submerged off-take.

In this project, the parshall flume is recommended for measurement purpose and to be provided at the end of the external canal.

2)-1 Determination of the size of parshall flume

In order to measure the maximum discharge, that is 50 l/sec, the up stream water depth of the parshall flume (Ha) is to be as follows:-

1) in case of 6" size (15.24 cm)

$$q = 0.264 \cdot Ha^{1.58}$$

$$50 = 0.264 \times Ha^{1.58}$$

$$Ha = 27.6 \text{ cm}$$

2) in case of 9" size (22.86 cm)

$$q = 0.466 \cdot Ha^{1.53}$$

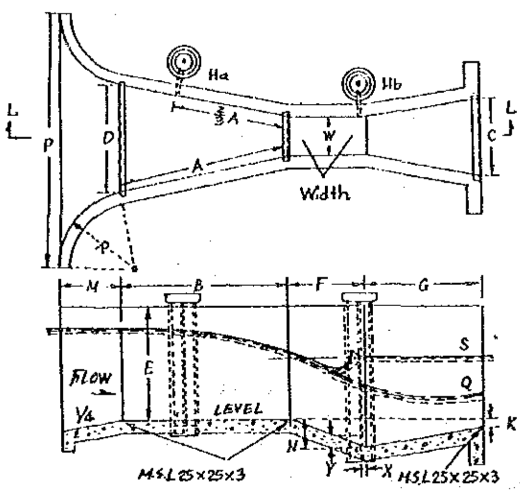
$$50 = 0.466 \times Ha^{1.53}$$

$$Ha = 21.2 \text{ cm}$$

The 6" size of the parshall flume which can give deeper water depth than 9" one is adopted for this project.

The dimension of each part of the 6" parshall flume is shown as in Fig.- 26

Fig.- 26 Dimension of the 6" Parshall Flume



W	A	$\frac{2}{3}A$	B	C	D	E	F	G
7.62 (3 in)	46.7	31.1	45.7	17.8	25.9	61.0	15.2	30.5
15.24 (6 in)	62.1	41.4	61.0	39.4	39.7	61.0	30.5	61.0
22.86 (9 in)	88.0	58.7	86.4	38.1	57.5	76.2	30.5	45.7
30.48 (1 ft)	137.2	91.4	134.3	61.0	84.5	91.4	61.0	91.4
45.72 (1 ft 6 in)	144.8	96.5	141.9	76.2	102.6	91.4	61.0	91.4
60.96 (2 ft)	152.4	101.6	149.5	91.4	120.7	91.4	61.0	91.4

(W~Y : cm)

K	N	R	M	P	X	Y	Q (m^3/sec)	
							Min.	Max.
2.5	5.7	40.6	30.5	76.8	2.5	3.9	0.00085	0.0538
7.6	11.4	40.6	30.5	90.2	5.1	7.7	0.00142	0.110
7.6	11.4	40.6	30.5	108.0	5.1	7.6	0.00255	0.252
7.6	22.9	50.8	38.1	149.2	5.1	7.6	0.00311	0.456
7.6	22.9	50.8	38.1	167.6	5.1	7.6	0.00425	0.697
7.6	22.9	50.8	38.1	185.4	5.1	7.6	0.0119	0.937

2)-2 Determination of the water depth (Ha) at the measurement point "a".

What is the control water depth at the "a" point when the maximum discharge is given by 50 l/sec?

The water depth (Ha) at the measurement point "a" and the control point C are determined as follows.

$$q = 0.264\ Ha^{1.58}$$

where,

q = maximum water discharge (l/sec)

∴ Ha = water depth at the "a" point (cm)

and $Hc = \sqrt[3]{\dfrac{Q^2}{B^2 \cdot g}}$

where

Hc = critical depth (m)

Q = maximum water discharge (m^3/sec)

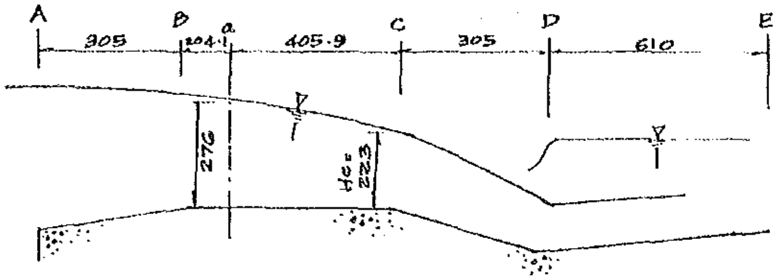
B = width of the canal (m)

∴ Ha = 0.276 m

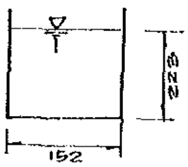
Hc = 0.223 m

However, this formula is adoptable for wide range of the discharge ranging from 0.00142 m^3/sec. It is also necessary to check the water depth Ha in order to reach the maximum discharge of 50 l/s.

Fig.- 27 Section between the measurement point "a" and control point "c".



Section of the control point is to be:-



$$Q = 0.050\ m^3/sec$$

$$Ac = 0.152 \times 0.223 = 0.034 m^2$$

$$P = 0.223 \times 2 + 0.152 = 0.598\ m$$

$$R = \frac{0.034}{0.598} = 0.057\ m$$

$$Vc = \frac{0.050}{0.034} = 1.478\ m/sec$$

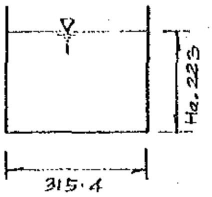
$$n = 0.015$$

$$C = \frac{R^{1/6}}{n} = \frac{0.057^{1/6}}{0.015} = 41.36$$

1) Calculation of the first approximate value

If the water depth (Ha) at the measurement point "a" is given by the same figure as 0.223 m., the section is to be:-

$$Q = 0.050 \ m^3/sec$$



$$A_a = 0.315x0.223 \quad 0.070$$

$$P_a = 0.223x2+0.315 = 0.761$$

$$R_a = \frac{0.070}{0.761} = 0.092$$

$$C_a = \frac{1}{0.015}x0.092^{1/6} = 44.79$$

$$V_a = \frac{0.05}{0.070} = 0.714$$

The formula of the back water curve is given as follows:-

$$\Delta h = \frac{V1^2}{2g} - \frac{V2^2}{2g} + \frac{Vm^2 \Delta l^1}{Cm^2 . Rm}$$

下　上.

where,

V1 : velocity at the control point (m/sec)

V2 : velocity at the measurement point (m/sec)

Vm : average velocity of V1 and V2 (m/sec)

Rm : average hydraulic radius (m)

Cm : average coefficiency of velocity

Δl' : distance between two points (m)

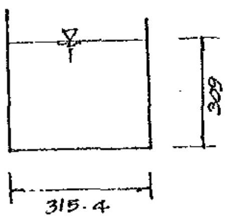
$$
\begin{aligned}
Vm &: (1.478+0.714)\times0.5 = 1.096 \\
Rm &: (0.057+0.092)\times0.5 = 0.075 \\
Pm &: (0.598+0.761)\times0.5 = 0.680 \\
Am &: (0.034+0.070)\times0.5 = 0.052 \\
Cm &: (41.36+44.79)\times0.5 = 43.08
\end{aligned}
$$

$$\therefore \Delta 1 = \frac{1.478^2}{2\times9.8} - \frac{0.714^2}{2\times9.8} + \frac{1.096^2\times0.4059}{43.08^2\times0.680}$$

$$\therefore Ha = 0.223 + 0.086 = 0.309$$

2) Calculation of the second approximate value

Using the new figure which is given to Ha, the most approximate value is calculated one after another.



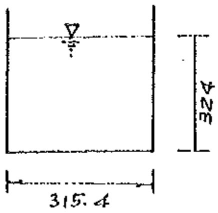
$$
\begin{aligned}
Q &= 0.050 \\
Aa &= 0.315\times0.309 = 0.097 \\
Pa &= 0.309\times2+0.315 = 0.933 \\
Ra &= \frac{0.097}{0.933} = 0.104 \\
Ca &= \frac{1}{0.015}\times0.104^{1/6} = 45.72 \\
Va &= \frac{0.050}{0.097} = 0.515
\end{aligned}
$$

$$
\begin{aligned}
\therefore\ Cm &= (41.36+45.72)\times0.5 = 43.54 \\
Rm &= (0.057+0.104)\times0.5 = 0.081 \\
Vm &= (1.478+0.515)\times0.5 = 0.997
\end{aligned}
$$

$$\therefore \Delta h = \frac{1.478^2}{2\times9.8} - \frac{0.515^2}{2\times9.8} + \frac{0.997^2\times0.4059}{43.54^2\times0.081} = 0.101$$

$$\therefore Ha = 0.223 + 0.101 = 0.324$$

3) Calculation of third approximate value



$$Q = 0.050$$

$$A = 0.324 \times 0.315 = 0.102$$

$$P = 0.324 \times 2 + 0.315 = 0.963$$

$$R = \frac{0.102}{0.963} = 0.106$$

$$C = \frac{1}{0.015} \times 0.106^{1/6} = 45.86$$

$$V = \frac{0.050}{0.102} = 0.49$$

$$\therefore Cm \doteqdot (41.36+45.86) \times 0.5 = 43.61$$

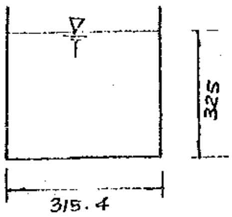
$$Rm = (0.057+0.106) \times 0.5 = 0.082$$

$$Vm = (1.478+0.49) \times 0.5 = 0.984$$

$$\therefore \Delta h = \frac{1.478^2}{2 \times 9.8} - \frac{0.49^2}{2 \times 9.8} + \frac{0.984^2 \times 0.4059}{43.61^2 \times 0.082} = 0.102$$

$$\therefore Ha = 0.223 + 0.102 = 0.325$$

4) Calculation of forth approximate value



$$Q = 0.050$$

$$A = 0.315 \times 0.325 = 0.102$$

$$P = 0.325 \times 2 + 0.315 = 0.965$$

$$R = \frac{0.102}{0.965} = 0.106$$

$$C = \frac{1}{0.015} \times 0.106^{1/6} = 45.86$$

$$V = \frac{0.050}{0.102} = 0.49$$

$\therefore$ $Cm = (41.36+45.86)x0.5 = 43.61$

$Rm = (0.057+0.106)x0.5 = 0.082$

$Vm = (1.478+0.49)x0.5 = 0.984$

$$\therefore \Delta h = \frac{1.478^2}{2x9.8} - \frac{0.49^2}{2x9.8} + \frac{0.984^2 x0.4059}{43.61^2 x0.082} = 0.102$$

$\therefore$ $Ha = 0.223 + 0.102 = 0.325$

Consequently, the back water from the control point to the measure point is determined by 0.102 m and it's operation water depth is to be 0.325 m.

In addition, the critical depth at control point is to be 2/3 height of the energy head at the measurement point "a", so that the energy head is to be as follows.

$$Hc = \frac{2}{3} He$$

$$0.223 = \frac{2}{3} \cdot He$$

$\therefore$ $He = 0.3345$

The velocity head at the "a" point is to be:-

$$\Delta h = \frac{V_2^2}{2 \cdot g} = \frac{0.49^2}{19.6} = 0.012 \text{ m}$$

$\therefore$ $Ha = He - 0.0074$

$= 0.3345 - 0.012$

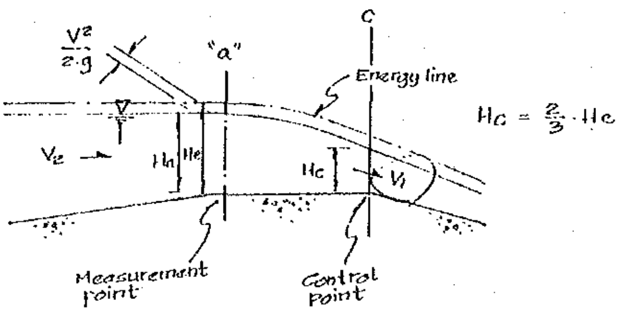
$= 0.323$

From calculation, the value of H_a is 0.323 m and this value is assured to be the correct figure.

Finally, the correct figure which is to meet the theoritical and hydraulical figure is determined by measuring the depth at point 'a'.

Ha	0.33 m
Hc	0.223

Fig.- 28 Relation between the critical depth and energy head.



2)-3 Determination of the back water curve at the parshall flume.

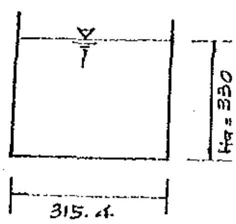
Irrigation water is lead from the off-take P23L through the external canal then controlled by the parshall flume as the measurement device.

To determine the water level to be supplied to the pipe line system the back water curve have to be decided.

The measurement water depth at "a" point is given by 0.33 m as mention in 2.2).2)-1.

Then, the back water curve at the upper point of A and B is determined as follows.

1) Calculation of the back water between "a" and B point.



Q = 0.050

A = 0.330x0.315 = 0.104

P = 0.330x2+0.315 = 0.975

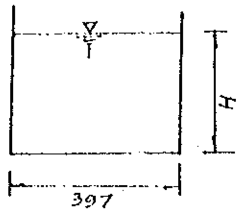
R = 0.107

C = 45.9

V = 0.481

1. Calculation of the first approximate value.

Section at B point is to be:-



If the water depth is assumed as the same height as the "a" point then each hydraulic factors are to be as follows.

Q = 0.050

A : 0.33 x 0.397 = 0.131

P = 0.33x2+0.397 = 1.057

R = 0.124

C = 47.1

V = 0.382

$\therefore$ Cm = (45.9+47.1)x0.5 = 46.5

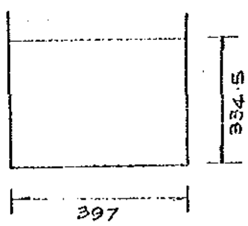
Rm = (0.107+0.124)x0.5 = 0.116

Vm = (0.481 + 0.382x0.5= 0.432

$$\therefore \Delta h = \frac{0.481^2}{19.6} - \frac{0.382^2}{19.6} + \frac{0.432^2 x0.2041}{46.5^2 x0.116} = 0.0045$$

$\therefore H_B$ = 0.33+0.0045 = 0.3345

2. Calculation of the second approximate value



Q = 0.050

A = 0.397x0.3345 = 0.133

P = 0.3345x2+0.397 = 1.066

R = 0.125

C = 47.1

V = 0.376

$\therefore$ Cm = (45.9+47.1)x0.5 = 46.5

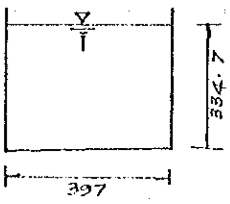
Rm = (1.107+0.125)x0.5 = 0.116

Vm = (0.481+0.376)x0.5 = 0.429

$$\therefore \Delta h = \frac{0.481^2}{19.6} - \frac{0.376^2}{19.6} + \frac{0.429^2 x 0.2041}{46.5^2 x 0.116} = 0.0047$$

$$\therefore H_B' = 0.33+0.0047 = 0.3347$$

3. Calculation of the third approximate value



Q = 0.0050

A = 0.397x0.3347 = 0.133

P = 0.3347x2+0.397 = 1.066

R = 0.125

C = 47.1

V = 0.376

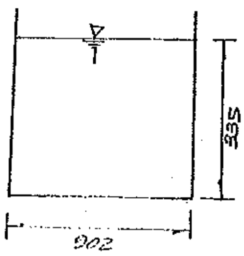
These indicated figure are the same value as the second case, so that the back water at the B point is to be 0.0047 m. and it's water depth is to be 0.335 m.

2) Calculation of the back water between B and A point.

The section at the B point is determined just now and the back water at the A point is calculated as well.

1. Calculation of the first approximate value

Section at A point is to be:-



Q = 0.050

A = 0.902x0.335 = 0.302

P = 0.335x2+0.902 = 1.572

R = 0.192

C = 50.6

V = 0.166

$\therefore$ Cm = (47.1+50.6)x0.5 = 48.9

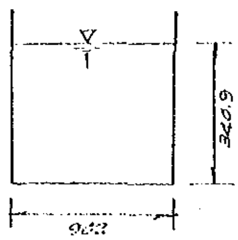
Rm = (0.125+0.192)x0.5 = 0.159

Vm = (0.376+0.166)x0.5 = 0.271

$$\therefore \Delta h = \frac{0.376^2}{19.6} - \frac{0.166^2}{19.6} + \frac{0.271^2 \times 0.305}{48.9^2 \times 0.159} = 0.0059$$

$$\therefore H_A = 0.335 + 0.0059 = 0.3409$$

2. Calculation of the second approximate value.



$Q = 0.050$

$B = 0.902 \times 0.3409 = 0.307$

$P = 0.3409 \times 2 + 0.902 = 1.584$

$R = 0.194$

$C = 50.7$

$V = 0.163$

$Cm = (47.1 + 50.7) \times 0.5 = 48.9$

$Rm = (0.125 + 0.194) \times 0.5 = 0.160$

$Vm = (0.376 + 0.163) \times 0.5 = 0.270$

$$\therefore \Delta h = \frac{0.376^2}{19.6} - \frac{0.163^2}{19.6} + \frac{0.270^2 \times 0.305}{48.9^2 \times 0.160} = 0.0059$$

$$\therefore H_A = 0.335 + 0.0059 = 0.3409$$
$$\doteqdot 0.341$$

3) Calculation of head losses from the external canal to the parshall flume.

The approaching velocity head from the external canal to the parshall flume is to be as follows.

$$\Delta h = \frac{V^2}{2.g} = \frac{0.083^2}{19.6} = 0.00035 \text{ m}$$

Therefore, the energy head at that point is to be:-

W.L $7.617 + 0.00035 = 7.61735$
$= 7.617$ m.

The velocity head at the point of A is to be:-

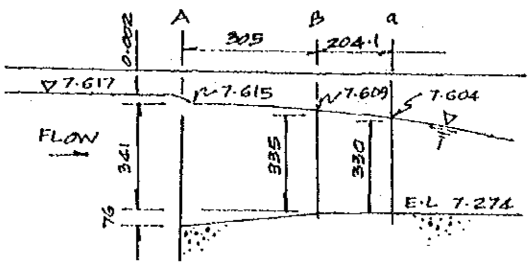
$$\Delta h = \frac{V^2}{2.g} = \frac{0.163^2}{2.g} = 0.0014 \text{ m}$$

$$= 0.002$$

so that, the water head at the point of A is to be:-

W.L $7.617 + 0.002 = 7.615$ m.

Fig.- 29 Water level from the external canal to the parshall flume

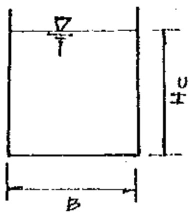


4) Calculation of the back water between "a" and C point.

In this Parshall flume, the control point appear at the C point and it's critical water depth is to be 0.223 m

(checking)

The critical depth is determined as follows.



$$Hc = \sqrt[3]{\frac{Q^2}{B^2 \cdot g}}$$

$$\therefore Hc = \sqrt[3]{\frac{0.050^2}{0.152^2 x9.8}} = 0.223 \text{ m}$$

And also the critical velocity is determined as follows.

$$Vc = \sqrt{Hc \cdot g}$$

$$\therefore Vc = \sqrt{0.223x9.8}$$

$$= 1.478 \text{ m/sec}$$

In this condition, the Froude number is to be:-

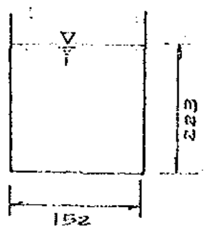
$$Fr = \frac{V}{\sqrt{g.h}}$$

$$Fr = \frac{1.478}{\sqrt{9.8x0.223}} \doteq 0.999 \leqq 1.0$$

Therefore, such condition is proofed as the critical condition and it's critical gradient is determined as follows:

$$Ic = \frac{Vc^2 xn^2}{Rc^{4/3}} = \frac{1.478^2 x0.015^2}{0.057^{4/3}} = 0.002 \doteq \frac{1}{45}$$

Consequently, the hydraulic factors at the "a" point is decided as below.



Q = 0.050

A = 0.152x0.223 = 0.034

P = 0.223x2+0.152 = 0.598

R = 0.057

C = 41.36

V = 41.36

The canal gradient after the control point is shown as in fig.- 30 , and it's gradient is to be 1:2.7.

This gradient is steeper than the critical hydraulic slope shich is determined by 1:45, so jet flow would occur afterwards.

5) Calculation of the back water between C and D point.

The Ernest Drescot Hill's formula is adopted into this calculation.

$$V = \left\{ \frac{2.g}{k} - e^{-k.H} \left(\frac{2.g}{k} - U^2 \right) \right\}^{\frac{1}{2}}$$

where, V : velocity at D point (m/sec)

$$k = \frac{2 \cdot g}{R_m \cdot C^2 \cdot \sin\Theta}$$

U : critical velocity (m/sec)

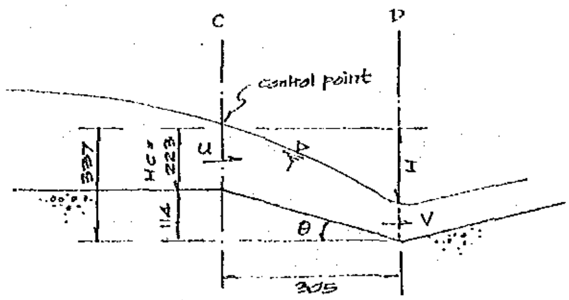
V : velocity of the chute canal (m/sec)

H : difference of the vertical height between up-stream and certain point of the chute canal (m)

Rm: average hydraulic radius between the chute canal (m).

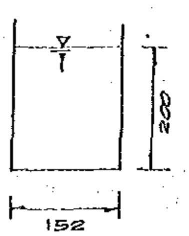
The longitudinal section of this part is shown as in Fig.- 30

Fig.- 30 Longitudinal section of the chute canal



1. Calculation of the first approximate value

   If the water depth at the D point is assumed as 0.20 m height the each hydraulic factors are to be as follows.



$Q = 0.050\ m^3/sec$

$A = 0.152 \times 0.20 = 0.0304\ m^2$

$P = 0.20 \times 2 + 0.152 = 0.552\ m$

$R = 0.055\ m$

$R_m = (0.057 + 0.055) \times 0.5 = 0.056$

$C = 41.24$

From the E.D. Hill's formula

$H = 0.337-0.200 = 0.137$

$U = 1.478$ m/sec

$\tan^{-1} \dfrac{0.114}{0.305} = 20.49°$

$\sin\theta = 0.35$

$$\therefore k = \frac{2.g}{Rm.C^2.\sin\theta}$$

$$= \frac{2x9.8}{0.056x41.24^2x0.35} = 0.588$$

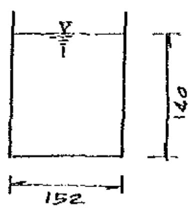
$$\therefore V = \left[\frac{19.6}{0.588} - e^{-0.588x0.137}\left(\frac{19.6}{0.588} - (1.478)^2\right)\right]^{\frac{1}{2}}$$

$$= 2.141 \text{ m/sec}$$

$Q = 0.0304x2.141 = 0.065\ m^3/sec > 0.050\ m^3/sec$ .. No

2. Calculation of the second approximate value.

If the water depth is assumed as 0.14 m, then each hydraulic factors are to be as follows.



$Q = 0.050\ m^3/sec$

$A = 0.152x0.140 = 0.0213\ m^2$

$P = 0.140x2+0.152 = 0.432$ m

$R = 0.0493$ m

$Rm = (0.057+0.0493)x0.5 = 0.0532$ m

$C = 40.88$

$V = 2.347$ m/sec

$H = 0.337 - 0.14 = 0.197$ m

$$\therefore k = \frac{2x9.8}{0.0532x40.88^2x0.35} = 0.630$$

$$\therefore V = \left\{\frac{19.6}{0.63} - e^{-0.63x0.197}\left(\frac{19.6}{0.63} - (1.478)^2\right)\right\}^{\frac{1}{2}}$$

$$= 2.360 \text{ m/sec}$$

$\therefore Q = 2.360x0.0213 = 0.050\ m^3/sec$ ... o.k

In this case, the Froude number is to be:-

$$Fr = \frac{V}{\sqrt{g.h}} = \frac{2.360}{\sqrt{9.8x0.14}} = 2.0 > 1.0$$

Therefore, it is necessary to make sure that the flow is to be a complete jet flow and the water depth at D point is to be 0.14 m.

Take into consideration about the parshall flume in submerged condition, the following formula is given to judge it's condition, that is:-

$$Hb/Ha \leq 0.6$$

where, Ha : water depth at the measure point "a"

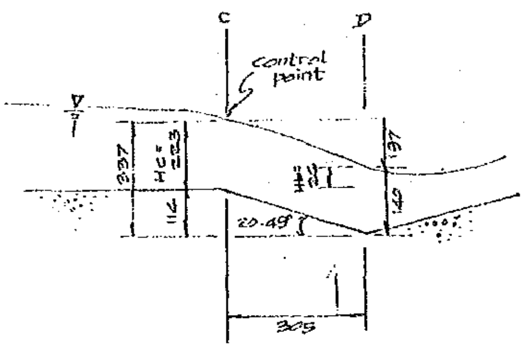
Hb : check point water depth at "b"

$$\therefore Hb = 0.6 \times Ha$$

$$= 0.6 \times 0.330 = 0.198 \text{ m}$$

To satisfy the E.D. Till's formula, the Hb is given as 0.026 m., so that the parshall flume can be provided as the over flowing parshall flume and not the submerged one.

Fig.- 31 Longitudinal section of the chute canal after determining the water depth.



6) Calculation of the back water between D and E point.

After the D point, the jet flow is going to change it's flow to ordinary flow, such that the hydraulic jump will occured.

Regarding to the hydraulic jump, when the Froude number is more than 1.7, it is to be a complete hydraulic jump.

Since, the Froude number is determined as 2.0, so in this case it must be counter-measured against it.

1. Calculation of the hydraulic jump.

The bed slope between the D and E point is given as 1:16 which can be considered the equivalant hydraulic condition that will happen on the normal flat canal with the limited gradient of less than 1:4.

The water depth after the hydraulic jump is determined as follows.

$$h_2 = \frac{h_1}{2} \left(\sqrt{8 \cdot \lambda + 1} - 1 \right)$$

where, h_2 : water depth after hydraulic jump

h_1 : water depth before hydrauic jump

$$\lambda = \frac{V^2}{g \cdot h_1}$$

V : velocity before hydraulic jump

$$\therefore \lambda = \frac{2.36^2}{9.8 \times 0.14} = 4.059$$

$$\therefore h_2 = \frac{0.14}{2} \left(\sqrt{8 \times 4.059 + 1} - 1 \right)$$

$$= 0.335 \ m$$

And, the length of the hydraulic jump is also determined as follows.

$$l = a(h_2 - h_1)$$

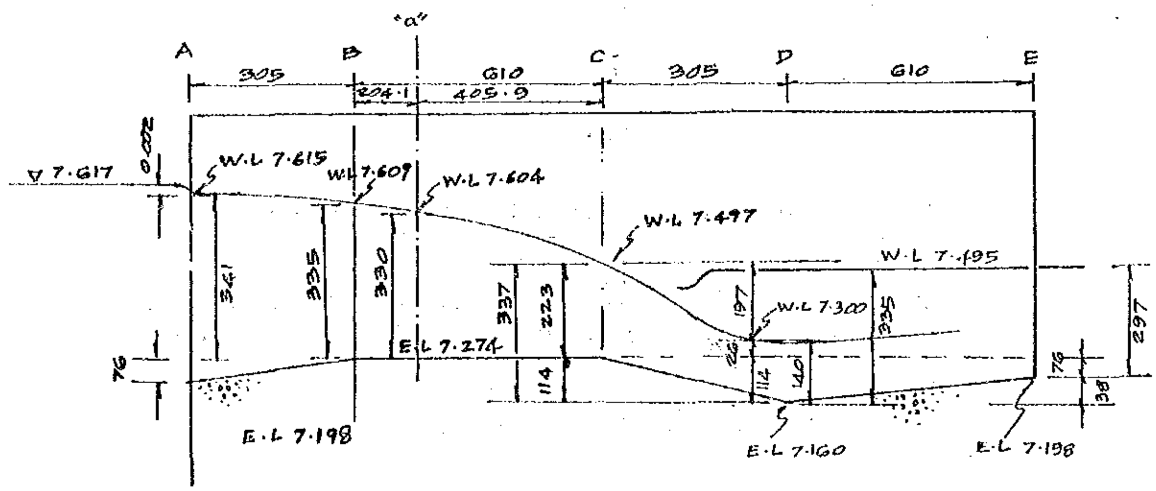
where, a : coefficient of the length of hydraulic jump $4.0 \leq a \leq 6.4$

$$\therefore\ l = a\ (0.335 - 0.140)$$

$$= 0.78 \text{ to } 1.248 \text{ m}$$

The calculation results up to here, from A to E, are arranged together with the water level as below.

Fig.- 32 Dimension of each part of the parshall flume together with the water level.



The water level after the hydraulic jump is given as W.L 7.495, and the head losses of this hydraulic jump is determined as follows.

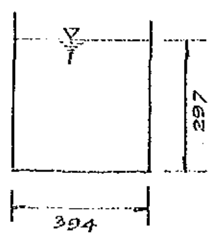
$$\Delta h = \frac{(h_2-h_1)^3}{4 \cdot h_1 \cdot h_2}$$

where, h_1 : water depth of the jet flow

h_2 : water depth of the ordinary flow.

$$\therefore \Delta h = \frac{(0.335-0.14)^3}{4 \times 0.14 \times 0.335} = 0.0395 \text{ m}$$

Now, the cross-section of the E point is given as follows:-



$Q = 0.050 \text{ m}^3/\text{sec}$

$A = 0.297 \times 0.394 = 0.117 \text{ m}^2$

$P = 0.297 \times 2 + 0.394 = 0.988 \text{ m}$

$R = 0.118 \text{ m}$

$C = 46.7$

$V = 0.427 \text{ m/sec}$

The Froude number at this time is to be:-

$$Fr = \frac{V}{\sqrt{g \cdot h}} = \frac{0.427}{\sqrt{9.8 \times 0.297}} = 0.25 < 1 \quad .. \text{ ordinary flow}$$

The velocity head is also to be:-

$$\Delta h = \frac{V^2}{2 \cdot g} = \frac{0.427^2}{2 \times 9.8} = 0.0093$$

Consequently, the energy head after hydraulic jump is determined as follows.

W.L 7.495 + 0.0395 + 0.0093 = 7.544

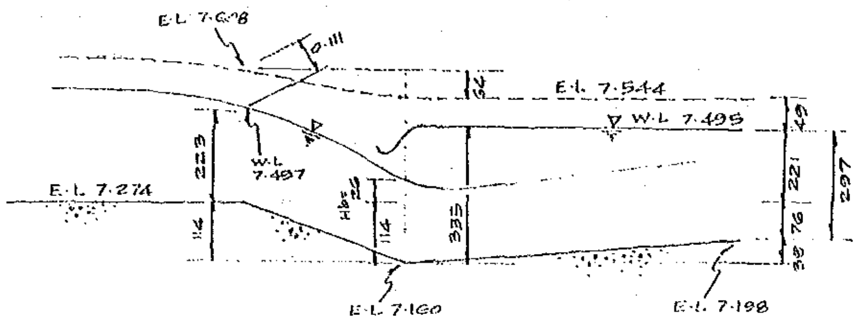
On the other hand, the energy head at the control point C is to be:-

$$\text{velocity losses } \Delta h = \frac{V^2}{2 \cdot g} = \frac{1.478^2}{2 \times 9.8} = 0.111 \text{ m}$$

$\therefore$ W.L 7.497 + 0.111 = 7.608 m

The difference between both energy head is obtained as 0.064 m.

Fig.- 33 Energy head of the control point and end point of the Parshall flume



However, take into consideration about the difference of the water level between up stream and down stream, normally when the difference happen as 2/3 He then the critical depth happen.

$$\frac{2}{3}He = \frac{2}{3} \times 0.223 \doteq 0.150$$

therefore, the water level at the end point of the parshall flume is to be:-

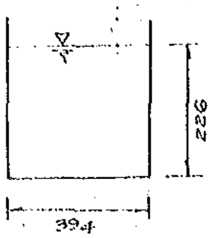
E.L 7.274+0.150 = 7.424 m

difference is to be.

7.495-7.424 = 0.071 m

Finally, in this project the later figure is taken, that is W.L 7.424 and the difference of the head loss can be considered as the safety factors.

So, the cross section at the end point of the parshall flume is determined as follows.



$Q = 0.050\ m^3/sec$

$A = 0.226 \times 0.394 = 0.089\ m^2$

$P = 0.226 \times 2 + 0.394 = 0.62\ m$

$R = 0.144\ m$

$C = 48.27$

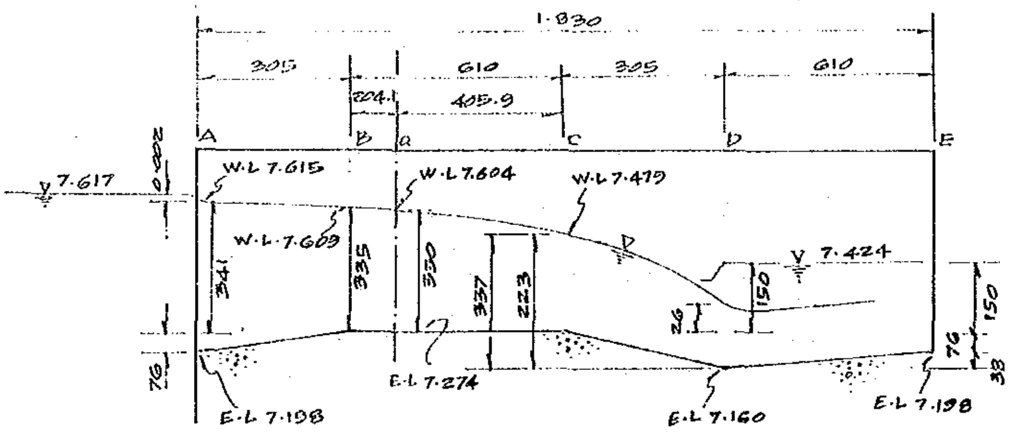
$V = 0.562\ m/sec$

The Froude number is also determined as follows.

$$Fr = \frac{V}{\sqrt{g \cdot h}} = \frac{0.562}{\sqrt{9.8 \times 0.226}} = 0.38 \leq 1 \ \ldots \text{ ordinary flow.}$$

Consequently, the final section of the Parshall flume is shown in in Fig.-34.

Fig.- 34 Final section of the parshall flume.

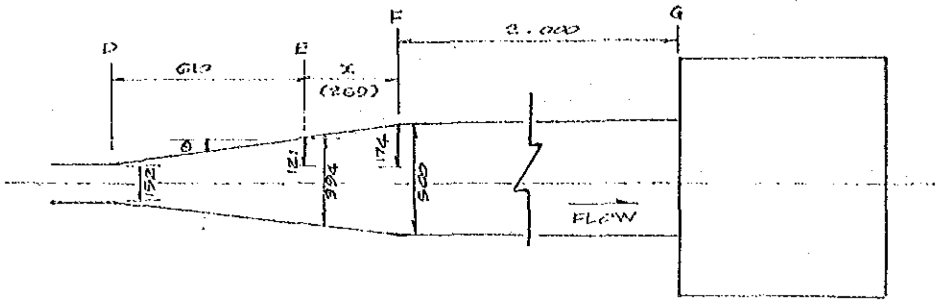


2)-4 Design of the transition from the parshall flume to the pipe line.

The transition will be provided after the parshall flume and try to keep the hydraulic pressure as far as possible.

When the dimension of the transition is given as below, the hydraulic characters are determined as follows.

Fig.- 35 Dimension of the transition.



$$\tan^{-1}\frac{0.121}{0.610} = 11.22^{o}$$

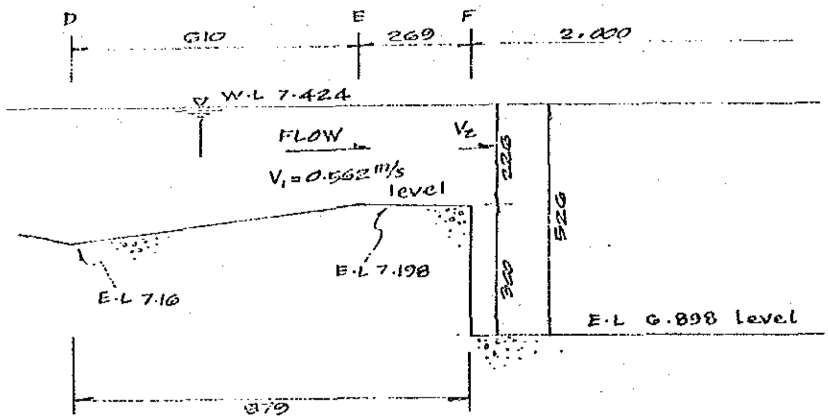
$$\therefore \tan 11.22^{o} = \frac{0.174}{(0.61+x)} = 0.198$$

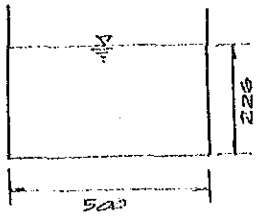
$$\therefore x = 0.269$$

the length of the transition is to be:-

0.610+0.269 = 0.879 > 0.78 (min. length of the hydraulic jump)
O.K

Fig.- 36 Longitudinal section of the transition





$Q = 0.050\ m^3/sec$

$A = 0.226 \times 0.50 = 0.113\ m^2$

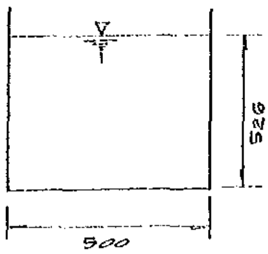
$P = 0.226 \times 2 + 0.50 = 0.952\ m$

$R = 0.119\ m$

$C = 46.76$

$V = 0.442\ m/sec$

The section of the down stream at the F point is given as follows.



$Q = 0.050\ m^3/sec$

$A = 0.526 \times 0.50 = 0.263\ m^2$

$P = 0.526 \times 2 + 0.50 = 1.552\ m$

$R = 0.167$

$C = 49.57$

$V = 0.19\ m/sec$

Then, the approaching velocity can be decreased, that is from 0.442 to 0.19 m/sec.

If bigger cross-section is given to the down stream, it is sure to decrease the approcahing velocity however it is not economical.

Two meters length of the regulating canal is also provided afterwards.

2)-5 Determination of the head losses by the screen

The screen should be provided to the main box No. 2, and it's head losses is calculated as follows.

$$\Delta h = \beta \cdot \sin\alpha \cdot \left(\frac{t}{b}\right)^{4/3} \cdot \frac{v^2}{2 \cdot g}$$

where,

β : coefficient of the Type of the screen bar 2.42

b : interval of the screen 4 cm

t : thickness of bar

α : setting angle 60°

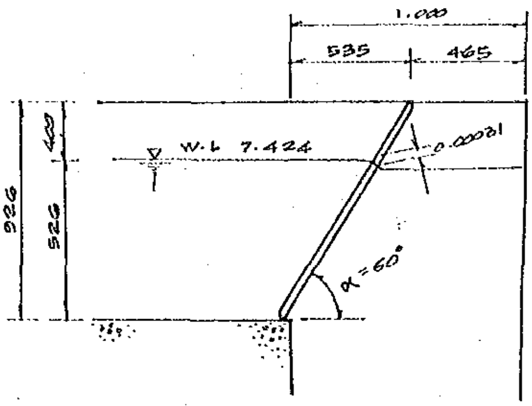
V : approaching velocity.

$$\therefore \Delta h \doteqdot 2.42 \times \sin 60^{o} \times \left(\frac{0.006}{0.04}\right)^{4/3} \times \frac{0.19^2}{2 \times 9.8}$$

$$= 0.00031 \text{ m}$$

Consequently, the head losses by the screen is not affected very much, so that the water level at the main box No. 2 is to be W.L 7.424

Fig.- 37 Head losses of the screen.



3) Design of the Pipe line

3)-1 Province of the turbulent flow and laminar flow.

Normally, the flow of the pipe is categorized to the turbulent flow and it's province is determined by the Reynolds number.

$Re < 2,100$ ... laminar flow

$Re > 2,100$ ... turbulent flow

where Re : Reynolds number

$$Re = \frac{V.D}{\nu}$$

ν : coefficient of kinematic viscosity 0.01 cm^2/sec

V : velocity in the pipe

D : diameter of the pipe

3)-2 Head losses of the pipe line

The head losses of the pipe can be determined as follows.

$$\Delta h = fi\frac{V^2}{2.g} + f.\frac{L}{D}.\frac{V^2}{2.g} + f_o\frac{V^2}{2.g} + \Sigma fn.\frac{V^2}{2.g}$$

$$\therefore V = \sqrt{\frac{2.g.h}{fi + f_o + f.\frac{L}{D} + \Sigma fn}}$$

where, fi : coefficient of in let loss
f_o : " out let loss
f : " friction loss
fn : " others

1) Coefficient of the friction losses.

The coefficient of the friction losses are identified by the following great peoples as mention below.

1. Experimental formula by Blasius

$$f = 0.3164.Re^{-\frac{1}{4}}$$

2. Manning and Chizy's formula

$$f = \frac{8.g.n^2}{R^{1/3}}$$

In case of the pipe, the hydraulic radius is to be:-

$$R = \frac{D}{4}$$

$$\therefore f = \frac{8.g.n^2}{(\frac{D}{4})^{1/3}} = \frac{124.5.n^2}{D^{1/3}}$$

3. Hazen-Williams's formula

$$f = \frac{133.7}{C^{1.852}.D^{0.167}.V^{0.148}}$$

where, C : coefficient of roughness

and these following formula are established as well.

$$V = 0.35464.C.D^{0.63}.I^{0.54}$$

$$Q = 0.27853.C.D^{2.63}.I^{0.54}$$

$$D = 1.6258.C^{-0.38}.Q^{0.38}.I^{-2.05}$$

$$I = \frac{hf}{L} = 10.666 \cdot C^{-1.85} \cdot D^{-4.87} \cdot Q^{1.85}$$

where,

D : diameter of the pipe (m)

Q : water discharge (m^3/sec)

hf: head losses by the friction loss (m)

L : length of the pipe (m)

The coefficient of roughness is determied as follows:-

Items	C	Remarks
P.V.C. pipe	150	D < 150 → C=140
A.C. pipe	140	
P.C. pipe	130	
Steel pipe	130	
Cast iron pipe	100	

3)-3 Determination of the kinds of the pipe and it's diameter

(conditions)

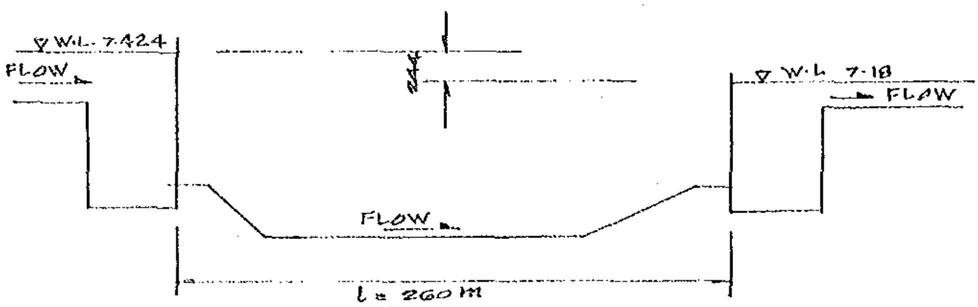
1. length of the pipe line	l = 260 m
2. min. velocity in the pipe	V min.=0.30 m/sec
3. max. discharge	Qmax = 0.050 m^3/sec
4. assumed head loss	△h = 0.244 m

The proposed velocity of the pipe which is classified in between Ø200 to 400 is expected to be about 0.9 to 1.6 m/sec, however it is recommended that the minimum velocity should be 0.3 m/sec in order to prevent sedimentation in the pipe.

Take into consideration about the total head loss of the pipe line, it is assumed approximately 0.244 m only.

The minimum velocity in the pipe should be approximately 0.3 m/s in order to reduce the friction losses.

Fig.-38 General longitudinal section of the pipe line



1) Case-1 ... (P.V.C pipe ∅ 300, C=150)

The biggest diameter of the P.V.C pipe which is available in Malaysia, that is ∅300 is taken in case-1.

$$V = 0.35464.C.C^{0.63}.I^{0.54}$$

$$0.30 = 0.35464 x 150 x 0.3^{0.63} x I^{0.54}$$

$$\therefore I^{0.54} = 0.012$$

$$\therefore Q = 0.27853.C.D^{2.63}.I^{0.54}$$

$$= 0.27853 x 150 x 0.3^{2.63} x 0.012$$

$$= 0.021 \ m^3/sec \quad < 0.050 \quad ... \quad No.$$

$$hf = (10.666.C^{-1.85}.D^{-4.87}.Q^{1.85}).L$$

$$= (10.666 x 150^{-1.85} x 0.30^{-4.87} x 0.021^{1.85}) x 26$$

$$= 0.072 \ m < 0.244 \quad ... \quad o.k$$

In this case, the total head loss excluding the bend loss satisfied it's limit but the maximum discharge is very small.

To determine the optimum size of the P.V.C pipe the following calculation has done i.e. by changing the velocity is the pipe and the results are shown as in tab.- 28.

Tab.-28 Results of the hydraulic calculation of the P.V.C. pipe

Kinds of the pipe	Diameter	V(m/sec)	$I^{0.54}$	Q(m^3/sec)	hf(m)	Remarks
P.V.C pipe	Ø300 C=150	0.30	0.012	0.021	0.072	
		0.60	0.0241	0.042	0.261	
		0.70	0.0281	0.049	0.347	
		0.72	0.0289	0.050	0.360	0.244
		1.00	0.040	0.070	0.670	

As the results, it can be said that this P.V.C pipe is not suitable to be used because the total head loss already exceeded it's limit when the maximum discharge is taken.

To solve this prOblem bigger size pipe must be take in order to decrease the total head loss, however the size of the Ø300 is the biggest size available at the moment.

Therefore, it is required to select the another kind of the pipe such as A.C pipe which is used for domestic purpose.

2) Case-2 ... (A.C. pipe, C = 140)

The calculation will be carried out using the Hazen-Williams' formula as in the case-1 and it's results are shown as in Tab-

When the pipe with the diameter of Ø400 is taken for this case, it is more than sufficient to supply the water but it is not economical.

The pipe with the diameter of Ø350 is supposed to be the most suitable pipe for this purpose, however this size is not available in Malaysia so that the equivalant pipe with the diameter of Ø370 is taken.

Then the results of this pipe is determined as follows.

by the Hazen-William's formula

$$I = \frac{h}{l} = 10.666.C^{-1.85}.D^{4.87}.Q^{1.85}$$

$$= 10.666 \times 140^{-1.85} \times 0.37^{-4.87} \times 0.050^{1.85}$$

$$= 0.00057$$

Tab.- 29 Results of the hydraulic calculation of the A.C. pipe.

Kinds of the pipe	Diameter	V(m/sec)	$I^{0.54}$	$Q(m^3/sec)$	hf(m)	Remarks
	ø350	0.30	0.012	0.029	0.071	
A.C pipe		0.50	0.020	0.048	0.179	
C = 140		0.52	0.020	<u>0.050</u>	<u>0.193</u>	< 0.244
		0.53	0.021	0.051	0.200	
	ø400	0.30	0.011	0.035	0.052	
		0.40	0.014	<u>0.050</u>	<u>0.079</u>	< 0.244
		0.41	0.015	0.053	0.112	
		0.50	0.018	0.063	0.155	

$$V = 0.35464 . C . D^{0.63} . I^{0.54}$$

$$= 0.35464 \times 140 \times 0.37^{0.63} \times 0.00057^{0.54}$$

$$= 0.470 \text{ m/sec}$$

$$\therefore hf = 0.00057 \times 260$$

$$= 0.148 \text{ m}$$

Now, the assumed pressure head is supposed to be 2.0 m so that the pressure in the pipe is to be 0.2 kg/m^2.

Therefore, the class B type of the A.C pipe is recommended to be used.

Finally, each factors of this pipe is determined as in tab.- 30 .

Tab.- 30 Determination of the A.C. pipe

1.	Kinds of the pipe	A.C pipe (B)
2.	Diameter	Ø370
3.	Length	l=260 m
4.	Max. discharge	Q = 0.050 m^3/sec
5.	Velocity	V = 0.470 m/sec
6.	Flow area	A = 0.108 m^2
7.	Hydraulic radius	R = 0.093 m
8.	Coefficient of roughness	C = 140

3)-4 Determination of the total head losses

In case-2, the head loss is determined as 0.148m which is only the friction loss excluding the bend loss.

The bend loss and the total head loss are calculated hereby.

Tab.-31 Head losses of the bend pipe

Kinds of bend pipe	hb(m)	fb	r(m)	R(m)	$\frac{R}{r}$	Remarks
45° Short bend	0.001	0.088	0.185	0.5334	2.9	
45° Medium bend	0.001	0.068	0.185	1.067	5.8	
$22\frac{1}{2}^{\circ}$ Short bend	0.0004	0.034	0.185	1.067	5.8	
$11\frac{1}{4}^{\circ}$ Short bend	0.0004	*0.032	0.185	2.134	11.5	>6.0
90° Medium bend	0.002	0.159	0.185	0.6096	3.3	

*The value of the fb should be taken the double if the $\frac{R}{r}$ is given more than 6.0

1) Head loss of the bend pipe.

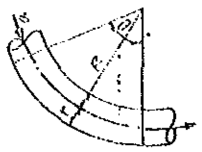
There are several kinds of bend pipe in this pipe line, such as 45^{o} short bend, 45^{o} medium bend, 22½^{o} short bend and 11¼ short bend together with the 2.779^{o}, 4.789^{o} and 6.358^{o} deflected bend.

The head losses of the bend pipe is determined by the Weiback as follows.

$$hb = fb.\frac{V^2}{2g}$$

where hb : head losses of the bend pipe

fb : coefficient of the bend loss



$$fb = \left\{0.131+1.847\ (\frac{r}{R})^{3.5}\right\}\ \frac{\theta}{90}$$

Then the head losses of each bend pipe can be determined as shown in tab.-

2) Head losses of the defelcted bend pipe.

The head losses of this bend pipe is given as follows.

$$hb = fb.\frac{V^2}{2g}$$

where $fb = 0.946.\sin^2\frac{\theta}{2}+2.05.\sin^4\frac{\theta}{2}$

Then the head losses of each deflected pipe can be determined as shown in Tab.- 32.

Tab.- 32 Head losses of the deflected pipe.

angle (θ)	hb (m)	fb
2.779	0.000007	0.0006
4.789	0.00002	0.0017
6.358	0.00003	0.003
Total	0.000057	

Consequently, the total head losses of this deflected pipe is very small such that it can be ignored.

3) Total head losses of the pipe line (Q.C.1)

The total head losses of the pipe line can be determined by the friction, bend and deflected losses.

1. In-let losses.

$$h = fi.\frac{V^2}{2g}$$

where, fi : coefficient of in-let losses (0.5)

$\frac{V^2}{2g}$: velocity head $\frac{0.47^2}{2x9.8}=0.012$

$h = 0.5x0.012 = 0.006$ m

∴ Energy head 7.424-0.006 = 7.418 m

Pressure head 7.418-0.012 = 7.406 m

2. 40.0+5.87 - 40.0+14.69

Kinds of pipes	Quantity
Cast iron fittings	5.035 m
A.C pipe	2.244 m
45^{o} Short bend	2 Nos. l=0.511x2=1.022
11¼ Short bend	2 Nos. l=0.532x2=1.064
Total	9.362 m

a) Head losses of the cast iron pipe

$$I=\frac{h}{l}=\frac{h}{5.035}$$

$$=10.666\times100^{-1.85}\times0.37^{-4.87}\times0.50^{1.85}$$

$$=0.001$$

$$\therefore h=0.006\ m$$

b) Head losses of the A.C pipe

$$I=\frac{h}{l}=\frac{h}{2.244}$$

$$=10.666\times140^{-1.85}\times0.37^{-4.87}\times0.050^{1.85}$$

$$=0.0006$$

$$\therefore h=0.002$$

c) Head losses of the 45^{o} short bend

$$h=0.001\times2=0.002$$

d) Head losses of the 11¼ short bend

$$h=0.0004\times2=0.0008$$

∴ total head losses = 0.011

∴ Energy head 7.418-0.011 = 7.407 m

Pressure head 7.407-0.012 = 7.395 m

Average hydraulic gradient $I=\frac{0.011}{9.362}=0.12\ \%$

3. No.0+14.69 - No. 1+5.69

A.C pipe l = 16.00 m

$$I = \frac{h}{l} = \frac{h}{16.00}$$

$$= 0.0006$$

h = 0.010 m

Energy head	7.407-0.010 =	7.397 m
Pressure head	7.397-0.012 =	7.385 m
Hydraulic Gradient	I =	0.06%

4. No.1+5.69 - No.1+13.69

Kinds of pipes	Quantity
Cast iron pipe	2.08 m
A.c pipe	5.92 m
Total	8.00 m

a) Head losses of the cast iron pipe

$$I = \frac{h}{l} = \frac{h}{2.08}$$

$$= 0.0011$$

$\therefore$ h = 0.003

b) Head losses of the A.C pipe

$$I = \frac{h}{l} = \frac{h}{5.92}$$

$$= 0.0006$$

$\therefore$ h = 0.004

$\therefore$ total head losses = 0.007 m

$\therefore$ Energy head 7.397-0.007 = 7.390

Pressure head 7.390-0.012 = 7.378

Average hydraulic gradient $I = \frac{0.007}{8.0} = 0.09\%$

5. No.1+13.69 - No.2+0.69

A.C pipe l = 12.00 m

$$I = \frac{h}{l} = \frac{h}{12.0}$$

$$= 0.0006$$

$\therefore$ h = 0.008

$\therefore$ Energy head	7.390-0.008	= 7.382
Pressure head	7.382-0.012	= 7.370
Hydraulic gradient		= 0.06%

6. No.2+0.69 - No.2+7.62

Kinds of pipes	Quantity
Cast iron pipe	4.672 m
A.C. pipe	1.50 m
45^{o} Medium bend	1 Nos. l=0.970 m
$22\frac{1}{2}^{o}$ Short bend	1 Nos. l=0.528 m
$11\frac{1}{4}^{o}$ Short bend	1 Nos. l=0.532 m
Total	8.202 m

a) Head losses of the cast iron pipe.

$$I=\frac{h}{l} = \frac{h}{4.672}$$

$$= 0.0011$$

$\therefore$ h = 0.005 m

b) Head losses of the A.C pipe

$$I=\frac{h}{l} = \frac{h}{1.50}$$

$$= 0.0006$$

$\therefore$ h = 0.001 m

c) Head losses of the bend pipe.

Kinds of bend pipe	h(m)
45^o Medium bend	0.001
$22\frac{1}{2}^o$ Short bend	0.0004
$11\frac{1}{4}^o$ Short bend	0.0004
Total	0.0018 m

∴ Total head losses = 0.008 m

∴ Energy head 7.382-0.008 = 7.374

Pressure head 7.374-0.012 = 7.362

Average hydraulic gradient $I = \frac{0.008}{8.202} = 0.10\%$

7. No.2+7.62 - No.7+7.12

A.C pipe l = 124.50 m

$$I \quad \frac{h}{l} = \frac{h}{124.50}$$

$$= 0.0006$$

$$\therefore \; h = 0.075 \text{ m}$$

∴ Energy head 7.374-0.075 = 7.299

Pressure head 7.299-0.012 = 7.287

Hyrdaulic gradient I = 0.06%

8. No. 7+7.12-No.7+11.22

Kinds of pipes	Quantity
cast iron pipe	3.07 m
$11\frac{1}{4}^o$ short bend	2 Nos. l=0.532x2=1.064 m
Total	4.134 m

a) Head losses of the cast iron pipe

$$I = \frac{h}{l} = \frac{h}{3.07}$$

$$= 0.0011$$

$$\therefore h = 0.004 \text{ m}$$

b) Head losses of the bend pipe

$$h = 0.0004\times2 = 0.0008 \text{ m}$$

$\therefore$ Total head losses = 0.0048

$\fallingdotseq$ 0.005 m

$\therefore$	Energy head	7.299-0.005 = 7.294
	Pressurehead	7.294-0.012 = 7.282
	Average hydraulic gradient	$I = \frac{0.005}{4.134} \fallingdotseq 0.12\%$

9. No. 7+11.22 - No. 10+12.35

Kinds of pipes	Quantity
A.C pipe	72.5 m
Cast iron pipe	3.211 m
90^o medium bend	2 Nos. l=0.6314 = 1.263
Total	76.974 m

a) Head lossesof the A.C pipe

$$I = \frac{h}{l} = \frac{h}{72.5}$$

$$= 0.0006$$

$$\therefore h = 0.044 \text{ m}$$

b) Head losses of the case iron pipe

$$I = \frac{h}{l} = \frac{h}{3.211}$$

$$= 0.0011$$

$$\therefore \ h = 0.004 \text{ m}$$

c) Head losses of the bend pipe

$$h = 0.002 \times 2 \quad 0.004 \text{ m}$$

$\therefore$ total head losses = 0.052 m

Energy head	7.294-0.052 = 7.242	
Pressure head	7.242-0.012 = 7.230	
Average hydraulic gradient	$I = \frac{0.052}{76.974} = 007\%$	

10. Out-let losses

$$h \quad f_e \cdot \frac{v^2}{2g}$$

$$1.0 \times \frac{0.47^2}{2 \times 9.8}$$

$$0.012 \text{ m}$$

$\therefore$ Pressure head 7.230 - 0.012 = 7.218

Energy head 7.218

Finally, the total head losses of the pipe line Q.C. 1 can be determined as in tab.- 33.

Tab.- 33 Total head losses of the pipe line, Q.C. 1

Section	h(m)	energy head (m)	pressire jead (m)	I(%)
M.B. 2	-	7.424	7.424	-
in-let	0.006	7.418	7.406	-
No.0+5.87-No.0+14.69	0.011	7.407	7.395	0.12
No.0+14.69-No.1+5.69	0.010	7.397	7.385	0.06
No.1+5.69-No.1+13.69	0.007	7.390	7.378	0.09
No.1+13.69-No.2-0.69	0.008	7.382	7.370	0.06
No.2+0.69-No.2-7.62	0.008	7.374	7.362	0.10
No.2+7.62_No.7+7.12	0.075	7.299	7.287	0.06
No.7+7.12-No.7+11.22	0.005	7.294	7.282	0.12
No.7+11.22-No.10+12.35	0.051	7.242	7.230	0.07
out-let	0.012	7.218	7.218	-

$\therefore$ Total head losses 7.424-7.218 = 0.206 $<$ 0.244 ... o.k

So, the difference of the head between the assumed head losses and actual head losses is to be 0.038 m and it is to be a safety factor.

4) Total head losses of the pipe line (Q.C 2)

The calculation method is the same as the pipe line Q.C. 1 and it's results are shown as in tab.- 34.

3)-5 Structural design of the pipe line.

1) Study of the buried depth

Buried depth is determined from the top of the surface of the ground to the top of the pipe.

1. Study of the buoyancy

The minimum cover of the pipe is calculated as follows.

$$H \geq \frac{\pi . Dc}{4} . \frac{S - \left\{ 1 - \left(\frac{D}{Dc}\right)^2 \right\} \gamma p}{W - 1.0}$$

where,

- H : minimum cover of the pipe (m)
- S : safety factor 1.2
- Dc: external diameter of the pipe 0.413 m
- D : internal diameter of the pipe 0.370 m
- γp: unit weight of the pipe 2.08 t/m^3
- W : bulk dencity of the soil 1.7 t/m^3

$$\therefore H \geq \frac{\pi \times 0.413}{4} \times \frac{1.2 - \left\{ 1 - \left(\frac{0.370}{0.413}\right)^2 \right\} \times 2.08}{1.7 - 1.0}$$

$\therefore H \geq 0.284 \doteqdot 0.30$

Tab.- 34 Total head losses of the pipe line Q.C 2

Section	h(m)	energy head (m)	pressure head (m)	I(%)
No.1+10.28 (Q.C.1)	-	7.393	7.381	-
No.0+1.5-No.0+4.49	0.007	7.386	7.374	0.23
No.0+4.49-No.1+10.50	0.019	7.367	7.355	0.06
No.1+10.50-NO.2+21.29	0.021	7.346	7.334	0.06

Normally, it is said that the buried depth of the pipe is recommended as below.

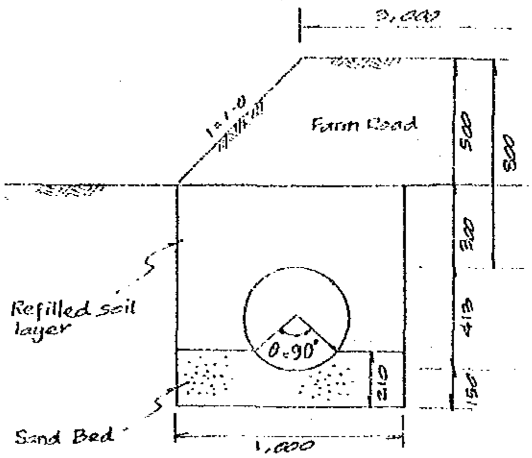
a) under the cultivated land ---- $H \geq 0.6$ m

b) under the farm road ---- $H \geq 1.0$ m

In this project, the pipe line is provided under the road shoulder and take into consideration of the load of the vehicles, the buried depth is given 0.80 m which is the total depth of the road height and at least necessary height of the buoyancy in order to reduce the earth work cost.

The cross -section of the pipe line is shown as in Fig.- 39

Fig.- 39 Cross section of the pipe line.



Road height = 0.50 m

min. cover = 0.30 m

Buried depth= 0.80 m

standard excavation width is given by 1.0 m in case of this pipe size.

2) Starical earth pressure

When the pipe is laid in the excavated ditch there are two types condition, that is ditch conduit and conduit in wide ditch, however in this project all of the condition is supposed to be ditch conduit.

1. Statiscal earth pressure under the ditch conduit.

A.C pipe is categorized as unflexible pipe so that the Marston's formula is adopted.

$$qv = Cd.W.Bd^2/Dc$$

$$Cd = \frac{1-e^{-2.k.u.\frac{H}{Bd}}}{2.k.u'}$$

where

qv : statical earth pressure ($k8/m^2$)

W : bulk density of the soil 1.7 t/m^3

Bd : width of the ditch at the top of the pipe (cm)

Dc : external diameter of the pipe (cm)

Cd : coefficiency of the load.

H : buried depth (cm)

K : coefficient of the Rankine's earth pressure

μ : coefficient of the internal friction of the refilled soil u = tan Ø

μ' : coefficient of the friction between refilled soil and side wall of the ditch

Ø : internal friction angle of the refilled soil. in this case Ø 11°

Ø' : friction angle between refilled soil and side wall of the ditch. Ø. Ø'

$$\therefore K = \frac{\sqrt{\mu^2+1}-\mu}{\sqrt{\mu^2+1}+\mu} = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45-\frac{\phi}{2})$$

$$= \tan^2(45-\frac{11}{2})$$

$$= 0.6795 \doteqdot 0.68$$

$$\therefore K.\mu = 0.68 \times \tan\phi$$

$$= 0.68 \times \tan 11^{\circ}$$

$$= 0.132$$

$$\therefore Cd = \frac{1-e^{-2\times0.68\times\frac{80}{100}}}{2\times0.132} = 0.721$$

$$\therefore qv = 0.721\times\frac{1.7\times1000}{10^6}\times\frac{100^2}{41.3}$$

$$= 0.297\ kg/cm^2$$

3) Dynamic earth pressure

1. Group load

in case of the farm road ... 300 kg/m^2

2. Track load.

The dynamic earth pressure of the track load can be calculated by the Bussinesq, Frohlick, Janssen, Marston and Frohling's formula.

In this design, the Bussinesq's formula is adopted.

The vehicle that can be considered for the calculation is adopted by 14 ton track.

In fact, 14 ton track is very rarely operated on the farm road, however the following calculation is based on this track as the safety way.

$$P_v = P(1+i)\sum \sigma_h / L.D_c = \alpha \cdot P.(1+i)$$

where,

$$\alpha = \sum \sigma_h / L.D_c$$

P_v : dynamic earth pressure by the 14 ton track.

P : weight of the vehicle assigned to one back wheel. 0.4 W

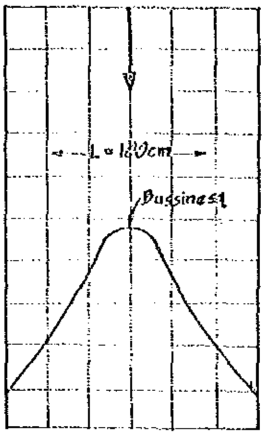
W : Total weight of the vehicle 14 Ton

L : length of distribution of the earth pressure along the pipe line.

D_c : external diameter of the pipe.

i : coefficient of impact 0.5

$\sum \sigma_h$: sum of the vertical earth pressure which is appeared on the distributed area of the load.



@ H<120cm

the length of distribution of the earth pressure along the pipe line is determined by the Bussinesq.

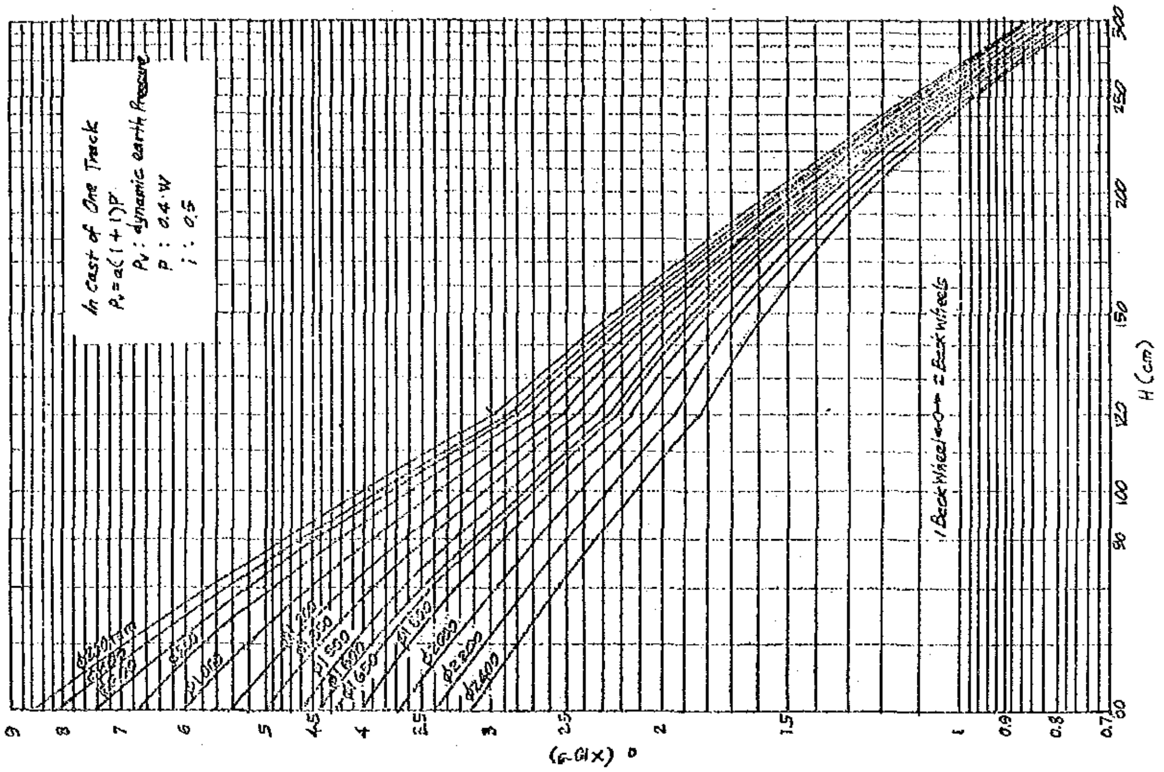
$H = 0.80 \sim 1.20$ m

$\therefore L = 0.50 \times 2 = 100$ cm

The sum of the vertical earth pressure which appeared on the distributed area of the load is determined by the D.L. Holl's integration.

The results of this integration are shown as in Fig.-40.

Fig.-40 Coefficient of the vertical dynamic earth pressure by track load.



$\therefore \alpha = 5.4 \times 10^{-5}$

$\therefore P_v = \alpha . P . (1+i)$

$= 5.4 \times 10^{-5} \times 14 \times 1000 \times (1+0.5)$

$= 0.45\ kg/cm^2$

Regarding to the weight of water in the pipe while the weight of the pipe itself can be ignored because the diameter of the pipe in less than Ø1,000, thus the horizontal earth pressure which would happen on the side of the pipe can be ignored as well.

4) Internal water pressure.

1. Dynamic water pressure.

This pipe line is to be a open conduit and its maximum dynamic water pressure is supposed to be $0.2\ kg/cm^2$.

2. Water hammer

In case of the open conduit, the water hammer can be decided as 20 percent of the dynamic water pressure.

$0.2 \times 0.2 = 0.04\ kg/cm^2$

The A.C pipe which can be used in this project is classified by B and it's recommended internal pressure is to be 6.1 kg/cm^2, so that this class B pipe can be adopted.

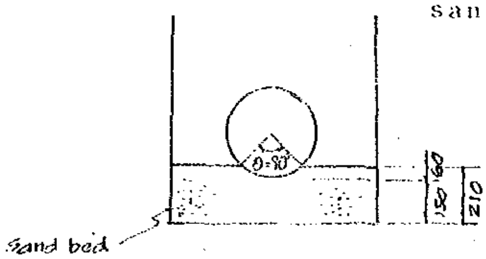
5) Invert reaction

The sand bed is provided in order to make the uniform invert reaction.

The invert reaction varies and depends on the supporting condition of the pipe and if the invert level is not well arranged before putting the pipe, the point load will happen and it can cause damage to the pipe.

Therefore, in this design the sand bed is provided by the minimum thickness of 0.15 m together with the supporting angle of 90°.

Fig. 41 Supporting of the pipe by the sand bed.



min. thickness t=15 cm (in case of Ø250 to Ø450) supporting angle $\theta = 90^0$

1. The maximum bending moment by the external load.

Generally, the maximum bending moment will happen at the invert portion of the pipe and it can be calculated by the following formula.

$$Mmax = C.q.R^2$$

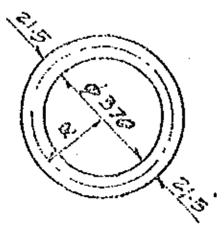
where,

Mmax. : maximum bending moment kg-cm

C : coefficient of the bending moment in case of the 90^o supporting angle C=0.314

q : q = qv+Pv

R : radius from the center of the thickness of the pipe.



$$R = \frac{370+21.5}{2} = 196$$

$$= 19.6 \text{ cm}$$

$$q = qv+Pv$$

$$= 0.297+(0.03+0.45)$$

$$= 0.48 \text{ kg/cm}^2$$

$$\therefore Mmax. = 0.314 \times 0.48 \times 19.6^2$$

$$= 93.7 \text{ kg-cm.}$$

6) Determination of the thickness of the pipe.

The thickness of the pipe is determined by following formula.

$$t = \sqrt{\frac{6(Md.S_3+Ml.S_4)}{\sigma_c\left(1-\frac{(Hs.S_1+Hw.S_2).R}{t\,\alpha i}\right)^{\frac{1}{2}}}}$$

where,

t : thickness of the pipe (cm)

R : radius from the center of the thickness of the pipe (cm)

Md : maximum bending moment by the statical earth pressure (kg-cm)

Md : maximum bending moment by the dynamic earth pressure (kg-cm)

Hs : statical water pressure (kg/cm^2)

Hw : water hammer (kg/cm^2)

σc : external breaking strength

$\phi < 600$ mm $\sigma c = 500\ kg/cm^2$

σi : internal breaking strength 200 kg/cm^2

S_1 : safety factor 2

S_2 : " 1

S_3 : " 2

S_4 : " 2

$$\therefore t = \sqrt{\frac{6 \times 93.7 \times 2}{500 \times (1 - \frac{(0.2 \times 2 + 0.04) \times 19.6}{200 \times t})^{\frac{1}{2}}}}$$

$$\cdot\ t^4 - 0.043 . t^3 - 5.06 = 0$$

$$\therefore t = 1.51 \qquad < 2.15 \text{ cm} \quad \ldots \text{ o.k}$$

The thickness of the recommended A.C pipe class B is to be 21.5 m and the pipe can be adopted because the value indicated is more than 1.51 cm.

7) Thrust blocks.

The bending portion is supposed to be the most dangerous place because the thrust force would happen due to the unbalanced water pressure and the centrifugal force of the velocity.

Therefore, the throust blocks are provided at these places.

The thrust force is determined as follows.

$$RH = S.PH$$

$$PH = 2\left(P.ac + \frac{a.w_o.V^2}{g}\right).\sin.\frac{\theta}{2}$$

where,

- RH : regisiance force (t)
- PH : horizontal thrust force (t)
- P : internal pressure (t/m^2) (statical water pressure + water hammer)
- a : flowing area (m^2)
- ac : external area of the pipe(m)
- w_o : bulk density of the water (1.0 t/m^3)
- V : average velocity in the pipe (m/sec)
- S : safety factor 1.5

in case of θ 90^o

$$\therefore PH = P'$$

$$= 2(6.244-5.494)\times1.2\times\frac{\pi.0.413^2}{4} + \frac{\frac{\pi.0.37^2}{4}\times1.0\times0.47^2}{g})\times\sin\frac{90^o}{2}$$

$$= 0.246 \text{ ton}$$

$$\therefore RH = 0.246\times1.5$$

$$= 0.369 \text{ ton.}$$

Then the volume of the mass concrete is to be:-

$$0.369/2.3 = 0.16 \ m^3$$

In addition, for all the fittings at the bend place the flanged pipe are preferable because it can prevent the thrust force from pulling out the pipe rather than the spigot type pipe.

8) Blow off

The blow off pipes together with the blow off value are provided at the end of pipe line in order to flash out the sedimented matter.

If the size of the pipes are given as Ø250 mm the velocity in the pipe is to be as follows.

Up stream water level = W.L=7.424 m

end point water level W.L=$5.76+\frac{0.25}{2}$

=5.885 m

. difference of the head 7.424-5.885= 1.539m

$$\therefore\ I = \frac{hf}{L} = \frac{1.539}{260} = 0.006$$

$$\therefore\ V = 0.35464.C.D^{0.63}.I^{0.54}$$

$$= 0.35464 \times 140 \times 0.25^{0.63} \times 0.006^{0.54}$$

$$= 1.31\ m/sec$$

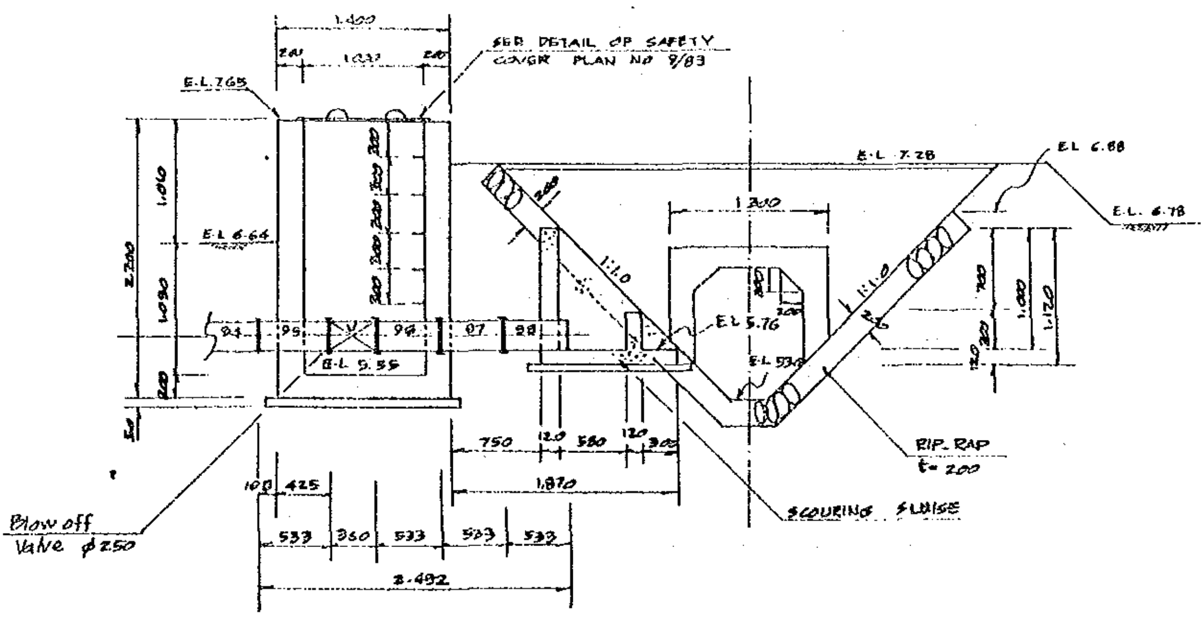
Take into consideration about this velocity in the pipe is faster than the ordinary condition, that is 0.47 m and meet to the recommended velocity of this size of the pipe as mention in tab.- 35

Tab.- 35 Recommended Velocity on the Pipe line.

size of the pipe (mm)	Proposed velocity (m/sec)
75 - 150	0.7 - 1.0
200 - 400	0.9 - 1.6
450 - 450	1.2 - 1.8
900 - 1,500	1.3 - 2.0
1,600 - 3,000	1.4 - 2.5

Therefore, the pipes of the Ø250 mm size can be taken and scouring sluise box should be provided together with rip-rap unless otherwise the side slope of the drainage canal can be spoiled easily.

Fig.- 42 Blow Off and scouring sluice box



4) Design of the open channle

The terminal water level of the pipe line is determined i.e 7.18 m as mention before, afterwards the irrigation water is lead by the open channel to each field block.

Each proposed field lot elevation can be arranged as follows.

Tab.- 36 Proposed field lot elevation

Field Block	Lot No.	elevation (m)
No. 4	410	6.78
	414, 417	6.78
	1568 i, ii	6.78
	1568 iii	6.82
	405	6.91
No. 5	450	6.78
	the rest	6.91
No. 6	All	6.80
No. 7	All	7.76
No. 8	All	6.83

The special feature of this area is that the gradient between the irrigation canal and the ground is to be reversed because the hydraulic level which is to be supply to each field lot is limitted.

4)-1 Determination of the canal size

Take into consideration on this mater the most optimum slope is calculated as follows.

(conditions)

1. proposed canal size

width $B = 0.50$ m

water depth $H \geqslant 0.30$ m

2. proposed maximum discharge

$Q = 0.50\ m^3/sec$

3. proposed velocity of the flow

$V \leqq 0.333$ m/sec

The water level in the canal should be kept about 0.20 m above the paddy field elevation in order to keep the complete over flow the field-off-take.

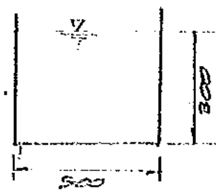
Consequently, the proposed hudraulic gradient of the open channel is determined as follows.

1. Terminal water level at the end point of the pipe line is given as

W.L = 7.18 m

2. Afterwards the hydraulic gradient is to be:-

for F.B. 4 and 5



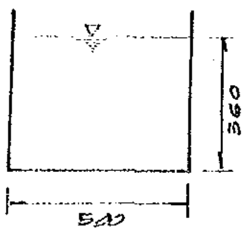
A = $0.15\ m^2$

Q = $0.05\ m^3/sec$

V = 0.34 m/sec

I = $\frac{1}{2,700}$

for F.B. 6, 7 and 8



A = $0.18\ m^2$

Q = $0.050\ m^3/sec$

V = 0.28 m/sec

I = $\frac{1}{4,400}$

The calculation table is shown as in Tab.- 26.

4)-2 Determination of the free board

The freeboard is calculated as follows.

$$Fb = 0.05d+hv+0.05$$

where, Fb : freeboard

d : water depth in the canal

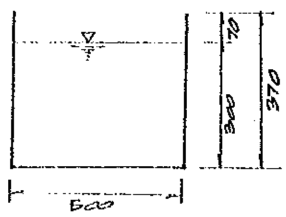
h : velocity head.

Thus, the freeboard is determined as below.

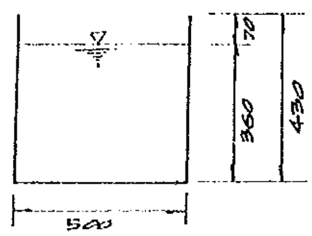
	d	0.05d	V(m/sec)	$\frac{V^2}{2 \cdot g}$	Fb
Case 1	0.30	0.015	0.34	0.006	0.071 ÷ 7 cm
Case 2	0.36	0.018	0.28	0.004	0.004 ÷ 7 cm

Finally, the cross-section on each canal is decided as follows.

Case-1



Case-2



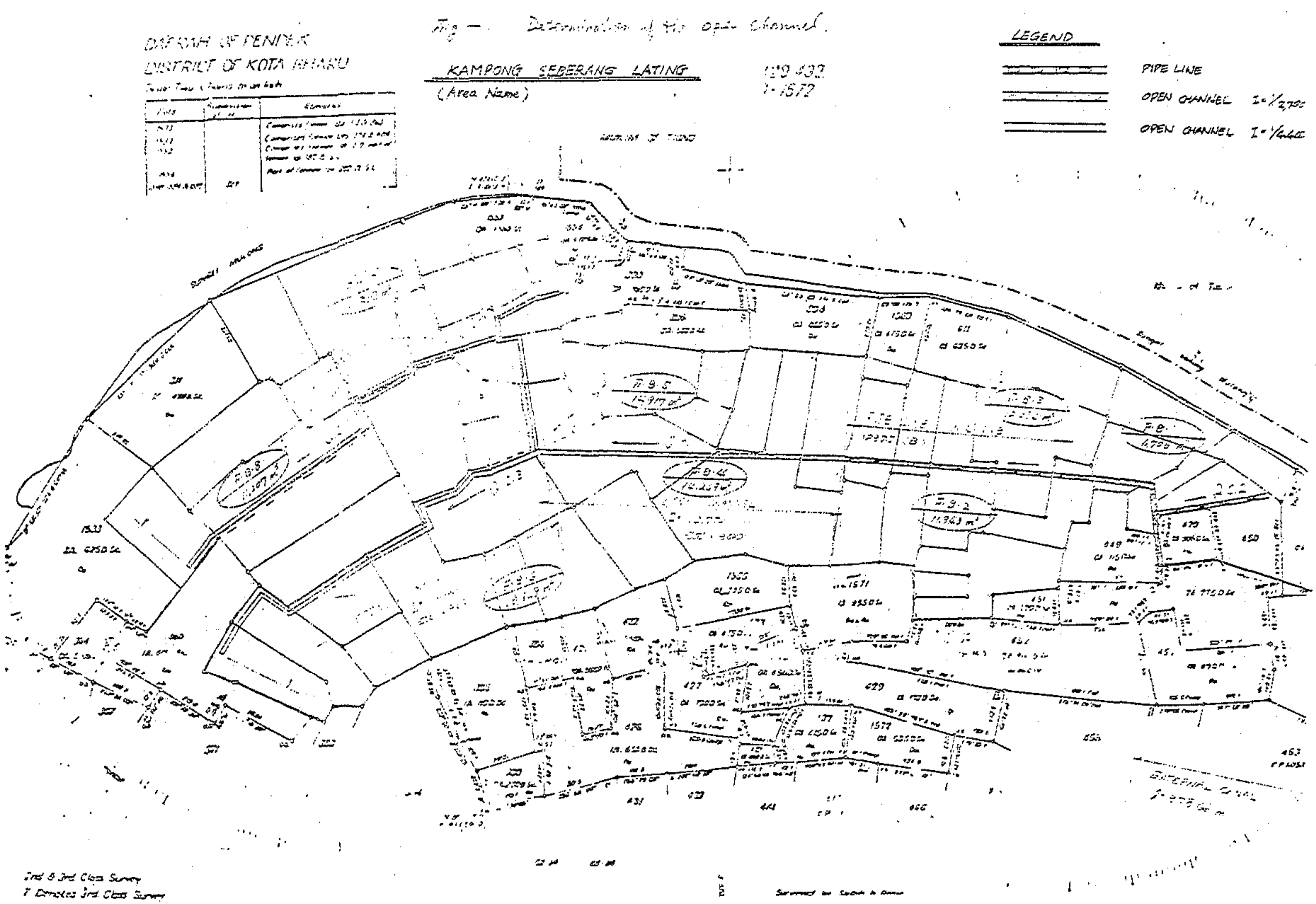


Fig — Determination of the open channel.
DISTRICT OF KOTA BHARU
KAMPONG SEBERANG LATING
(Area Name)
LEGEND
PIPE LINE
OPEN CHANNEL
OPEN CHANNEL
2nd & 3rd Class Survey
7 Denotes 3rd Class Survey

4. DESIGN ON THE FIELD DRAIN

1) Standard daily precipitation

The daily precipitation with the probability 1 in 5 years is adopted for the daily drainage from each field block.

The rainfall data since 1970 to 1977 at Kota Bharu are obtained and it's 5 years probability rainfall is given by 351.0 mm per day.

There are some argument to decide the drain out period, for example within a day, two days and three days, however it is not identified yet how the flooding water will affect to the yield

Incidentally, according to the experimental result at the Demonstration Farm at P.L.P.A.K, in fact it is still under experimental stage, it can be said that the complete flooding will seriously affect to the plant.

Consequently, though it remains how to fix the drainage period, in this design it will adopted that the one day rainfall should be drain out within one day at the moment.

2) Drainage discharge

The drainage discharge is calculated by following equation.

$$Q = \frac{1}{86,400} \times t_1 \times 10^{-3} \times A \times 10^{4} \times f$$

where,

- Q : drainage discharge (m^3/sec)
- t_1 : standard daily precipitation 35 mm/day
- A : catchment area (ha)
- f : run-off ratio (0.7)

The specific discharge is given as follows.

$$Q = \frac{1}{86,400} \times 351 \times 10^{-3} \times 1 \times 10^{4} \times 0.7$$

$$= 0.02844 \ m^3/sec/ha$$

Then the drainage discharge from the each field block is determined as in Tab.-37.

Tab.- 37 Determination of the drainage discharge

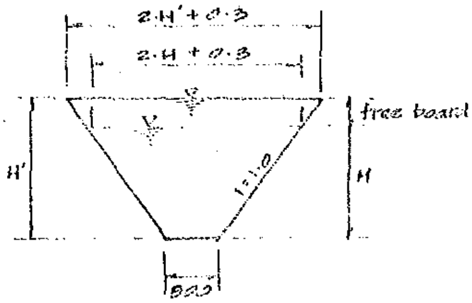
Field Drain	Catchment Area (ha)	Length (m)	Drainage Discharge (m^3/sec)
1	1.34	231.00	0.038 *(0.0456)
2	3.64	367.00	0.104 (0.1248)
3	4.98	132.50	0.142 (0.1704)
4-1	1.66	155.60	0.047 (0.0564)
4-2	7.3	236.40	0.208 (0.2496)
5	0.42	119.00	0.012 (0.0144)
6	1.24	227.00	0.035 (0.042)
Total		1,468.5	

*the figure in the brackets are considered about the free board.

$Q' = Q \times 1.2$

The cross-section of the drainage canal is given as in Fig-44.

Fig.-44 Cross-section of the drainage canal



$$I = \frac{1}{1,000}$$

n 0.03

1:m 1:10

b 300 mm

The water depth in each field drain is calculated and arranged as in Tab.- 38.

3) Determination of the size

The water depth of these drainage canal is very small, such as the deepest one is shown as 0.564 m after combining the drainage canal of F.D 3 and F.D 4.

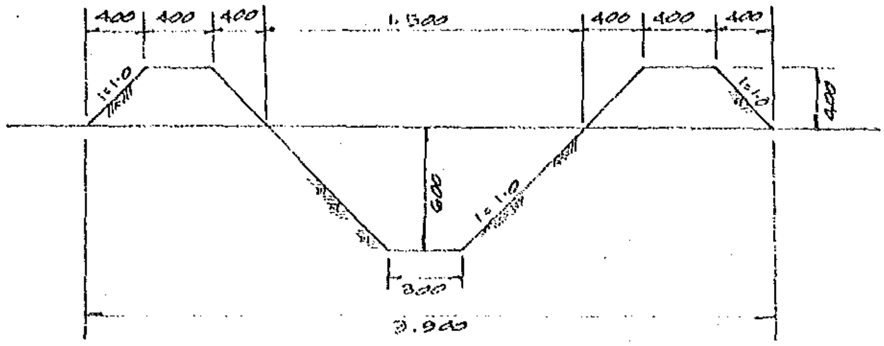
However, to encourage the permeability of the ground water the minimum water depth is recommended to be kept about 0.60 m.

Then the cross-section of the drainage canal at the highest portion is shown as follows.

Tab.- 38 Water depth in each field drain

Field drain	$\frac{Q.n}{I^{\frac{1}{2}}.8/3}$	water depth (m)	$Q(m^3/sec)$	$A(m^2)$	V(m/sec)
1	(1.0726) 0.8938	(0.273) 0.249	(0.0456) 0.038	(0.1564) 0.1367	(0.294) 0.278
2	(2.9355) 2.4462	(0.445) 0.408	(0.1248) 0.104	(0.3315) 0.2889	(0.377) 0.360
3	(4.0081) 3.3400	(0.515) 0.473	(0.1704) 0.142	(0.4197) 0.3656	(0.405) 0.388
4-1	(1.3266) 1.1055	(0.303) 0.277	(0.0564) 0.047	(0.1827) 0.1598	(0.307) 0.294
4-2	(5.8710) 4.8925	(0.613) 0.564	(0.2496) 0.208	(0.5597) 0.4873	(0.447) 0.427
5	(0.3387) 0.2823	(0.149) 0.135	(0.0144) 0.012	(0.0669) 0.0587	(0.209) 0.204
6	(0.9879) 0.8233	(0.262) 0.238	(0.042) 0.035	(0.1472) 0.1280	(0.285) 0.373

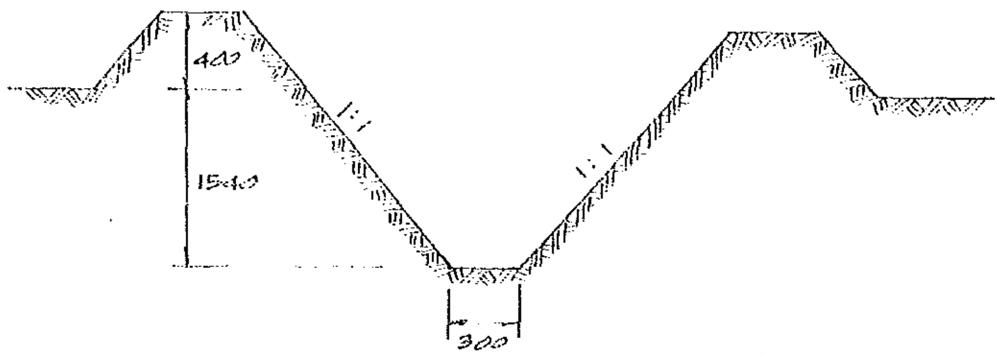
Fig.- 45 Cross section of the drainage canal



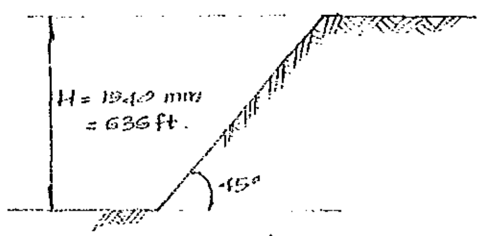
3) Slope stability analysis

We wish to determine the factor of safety, FS, for the slope of the drain.

The worst case occurs at FD-4 i.e. C/S No. 6 + 3.50 as illustrated below.



In this analysis the diagram is simplified as shown below;



In this case we assume that;

i) The cohesive component of shearing resistance of soil, $c = 250$ PSF ($122\ gm/cm^2$)

ii) The angle of shearing resistance of soil, $\phi = 5^o$

and iii) Unit weight of soil, $\gamma = 110$ PCF ($1.75\ gm/cm^3$)

<u>Calculation</u>

Angle of shearing resistance of soil required, ϕreq. can be determined by using the formula.

$$\phi req = \tan^{-1}\left(\frac{\tan \phi}{F\phi}\right)$$

where $F\phi$ is the factor of safety with respect to frictions.

Try $F\phi = 1$

so that,

$$\phi req = \tan^{-1}(\tan \phi)$$
$$= \phi$$
$$= 5^o$$

Therefore from figure 12.10 the value of $\frac{Creq}{\gamma H} = 0.135$

but,

$$\frac{Creq}{\gamma H} = \frac{C}{Fc\gamma H}$$

where

c = cohesive component of shearing resistance

$Creq$ = cohesive component of shearing resistance required

Fc = factor of safety with respect to cohesion

γ = unit weight of the soil

H = height of the embankment

therefore,

$$\frac{C}{Fc\ \gamma H} = 0.135$$

or

$$Fc = \frac{C}{0.135\ \gamma H}$$

Fig. 46 – Stability Chart for slopes in homogeneous material.

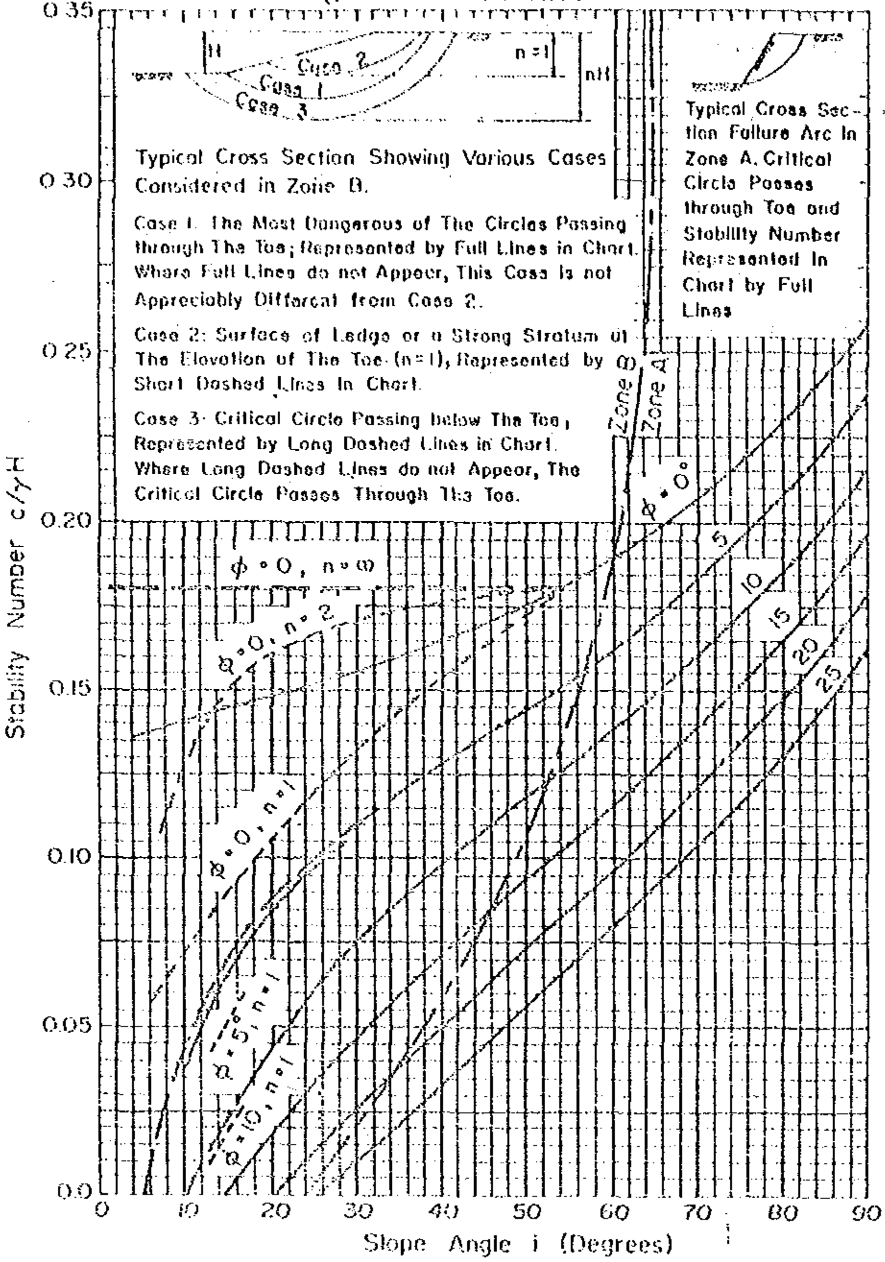


Fig. 12.10 Stability chart for slopes in homogeneous material (After Taylor, 1948)

Substitute the values of C, γ and H into the equation, we have;

$$Fc = \frac{250}{(0.135)(110)(6.36)}$$

$$\therefore Fc = 2.65$$

Then try $F\phi = 1.20$

Likewise,

$$\phi req = \tan^{-1}\left(\frac{0.0875}{1.2}\right)$$

$$= 4.17^{0}$$

so,

$$\frac{Creq}{\gamma H} = 0.150$$

therefore,

$$Fc = \frac{250}{(0.150)(110)(6.36)}$$

$$\therefore Fc = 2.38$$

Then try $F\phi = 1.50$

$$\phi req = \tan^{-1}\left(\frac{0.0875}{1.5}\right)$$

$$= 3.34^{0}$$

So,

$$\frac{Creq}{\gamma H} = 0.153$$

Therefore,

$$Fc = \frac{250}{(0.153)(110)(636)}$$

$$\therefore Fc = 2.33$$

Then try $F\phi = 1.70$

$$\phi req = \tan^{-1}\left(\frac{0.0575}{1.70}\right)$$

$$= 2.95^{0}$$

$$\frac{Creq}{\gamma H} = 0.154$$

$$\therefore Fc = \frac{250}{(0.154)(110)(6.36)}$$

$$\therefore Fc = 2.32$$

Then try $F\phi = 2$

$$\phi req = \tan^{-1}\left(\frac{0.0875}{2.00}\right)$$

$$= 2.5^{0}$$

$$\frac{Creq}{\gamma H} = 0.155$$

$$\therefore Fc = \frac{250}{(0.155)(110)6.36)}$$

$$\therefore Fc = 2.31$$

Lastly try $F\phi = 2.4$

$$\phi req = \tan^{-1}\left(\frac{0.0875}{2.4}\right) = 2.09^{0}$$

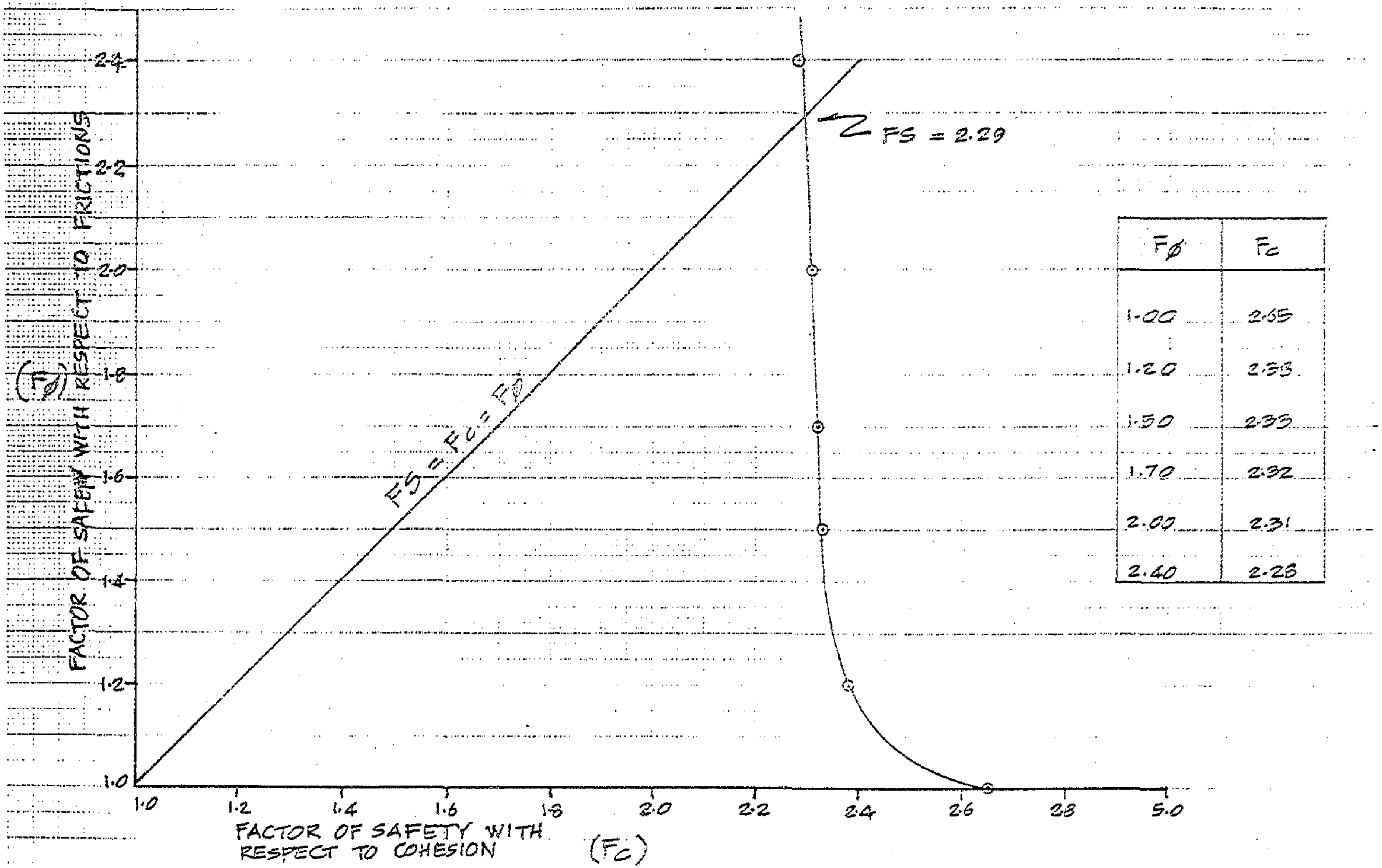
$$\frac{Creq}{\gamma H} = 0.157$$

$$\therefore Fc = \frac{250}{(0.157)(110)(6.36)}$$

$$\therefore Fc = 2.28$$

Conclusion:

Although the assumed value of cohesive component of shearing resistance of soil, C is rather small it has been clearly proved that the factor of safety with respect to the shearing resistance by ϕ-circle method is greater than I. Therefore with the value of FS is 2.29 the slope is stable.



F∅	Fc
1.00	2.65
1.20	2.33
1.50	2.33
1.70	2.32
2.00	2.31
2.40	2.28

Fig. 47. CALCULATION OF FACTOR OF SAFETY WITH RESPECT TO SHEARING RESISTANCE BY ∅-CIRCLE METHOD

5. DESIGN ON THE FARM ROAD

1) Determination of the width

Generally speaking, the width of the farm road is decided by the kinds of vehicles which are concerned to the agricultural activities such as tractors, combines, tracks and so on.

It is said that the width of the main farm road should be provided with enough space to pass each other, that is 0.5 m, outside surplus width, that is 0.3 m, and road shoulder of 0.5 to 0.75 m width.

The effective width of main farm road is recommended as 5.0 to 6.0 m in order to pass each truck and tractor and also the branch farm road is 3.0 to 4.0 m.

The vehicles width which are likely to use the farm road are shown in Tab.- 39.

Tab.- 39. Vehicles for the agriculture

Kinds of Vehicles	width (m)
passanger car	2.0
truck	2.4
tractor (40 ps)	2.0
trailer	1.9
combine (w = 3.0 m)	3.5

In this design, the expected farm road width is supposed to be 5.0 m, however it is said that a lot of land will be occupied by these farm road width.

Take into consideration about the converted rate, it should be as small as possible, and existing road which is connected to the main farm road is very narrow, that is 2.5 m so that there is no point if the 5.0 m width of the farm road was provided as the facilities in this area.

Therefore, the minimum width of the farm road, that is 3.0 m is given to this design with 0.10 m thickness of the laterite pavement.

2) Determination of the height

The height of the farm road from the paddy ground is said more than 0.50 m for the main far road and 0.30 m for branch road.

Careful consideration to be given before deciding the height of it because of the access to the field lot.

In this design the height of the farm road is given in between 0.30 to 0.50 m depends on the proposed ground level.

3) Longitudinal slope

In general, the maximum longitudinal slope of the main farm road is said less than 8% as usual and also 12% as special case.

If the longitudinal slope is given more than 8% at that time it s length should be limitted by 100 m with the control distance which is consisted by the appropriate slope that is less than 2.5% slope with more than 30 m length.

4) Cross-section

The cross section of the farm road should be constructed that the centre portion is higher than the road shoulder side in order to drain out the rainfall water immediately.

The cross sectional slope of the farm road which is made by soil and gravel is supposed to be 3 to 6%.

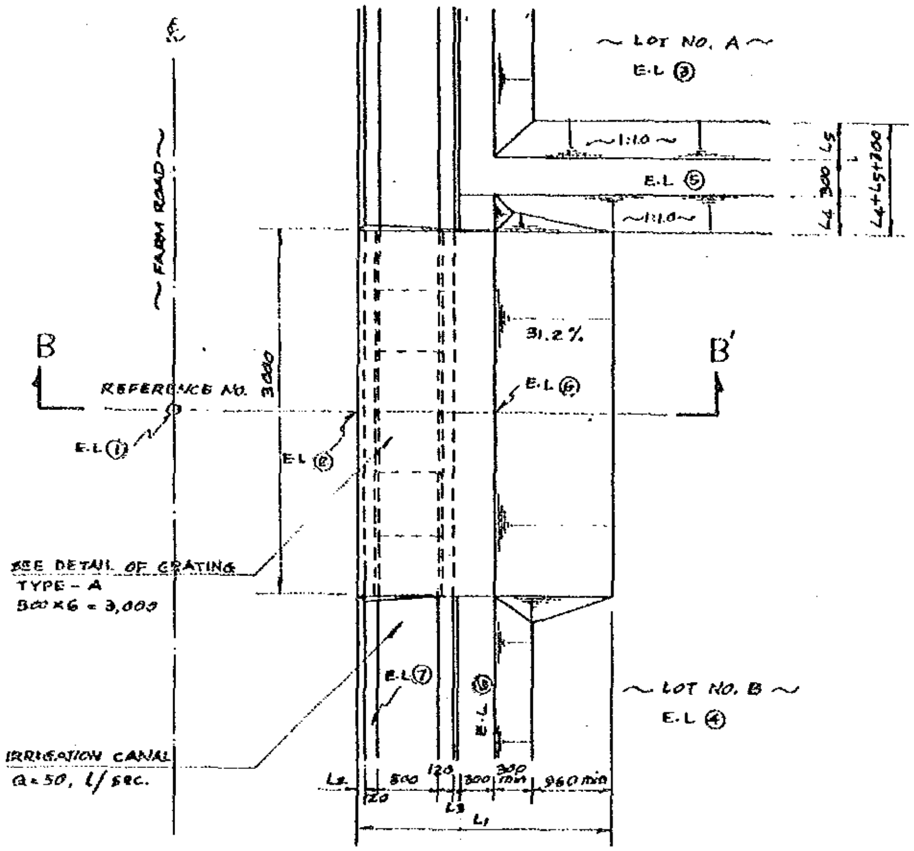
5) Access to the field lot

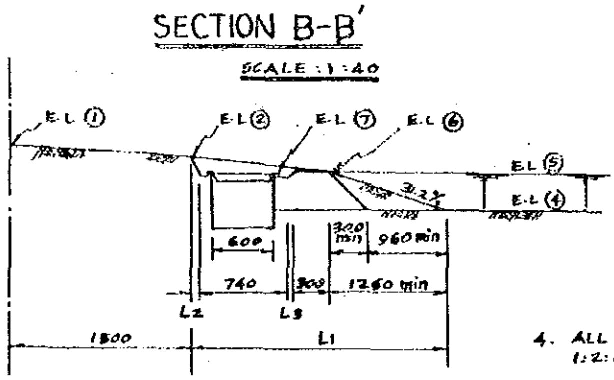
The width of the access is the same width as well as the farm road, that is 3.0 m and it s slope is determined less than 32.5% because of the limitted slope for tractor.

All of the paddy lots which along the farm road will be possessed the access at least on place to each lot, and the location of it is recommended to be the left side of the field lot due to the combine used to work right revolution as usual.

In this design, the access to the field lot is given by 31.2 percent slope and it's tipical cross-section is shown as in Fig.- 48.

Fig.- 48 Typical cross-section of access to the field lot





4. ALL REINFORCED 1:2:4 MIX. WITH ∅ 19

6) Corner cut-off

The corner cut-off can be provided at the intersection of the farm road if necessary.

The side of the corner cut-off is recommended as 1.5 to 2.0 m for one corner.

The shape of the benefitted area is like a man hand and only one farm road is connected to the main farm road, that is F.R-2.

Therefore, the length of the farm road in one section is going to be longer and the width is detemined by 3.0 m, so that when two vehicles which are passing from opposite side of the farm road can pass conveniently.

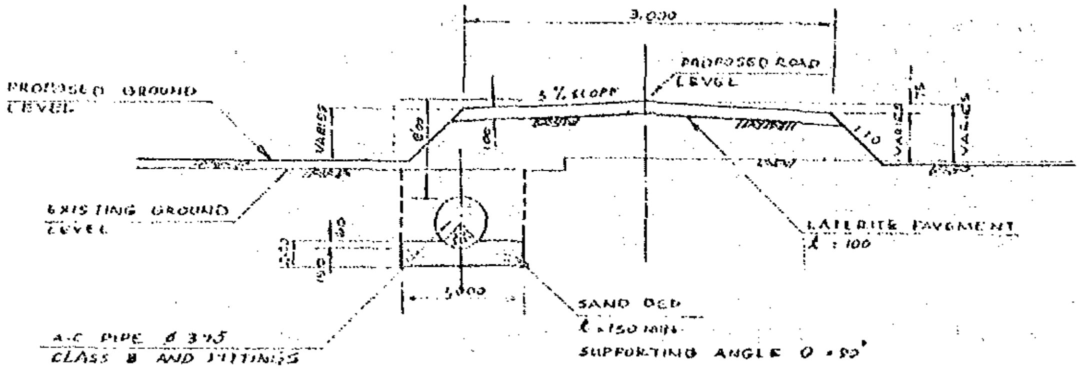
Thus, in this design the size of the corner cut-off is given by 3.0 m for one corner as the purpose of the refuge area.

Finally, the cross section of the farm road is shown as in Fig.- 49.

Fig.- 49 Typical cross-section of the farm road.

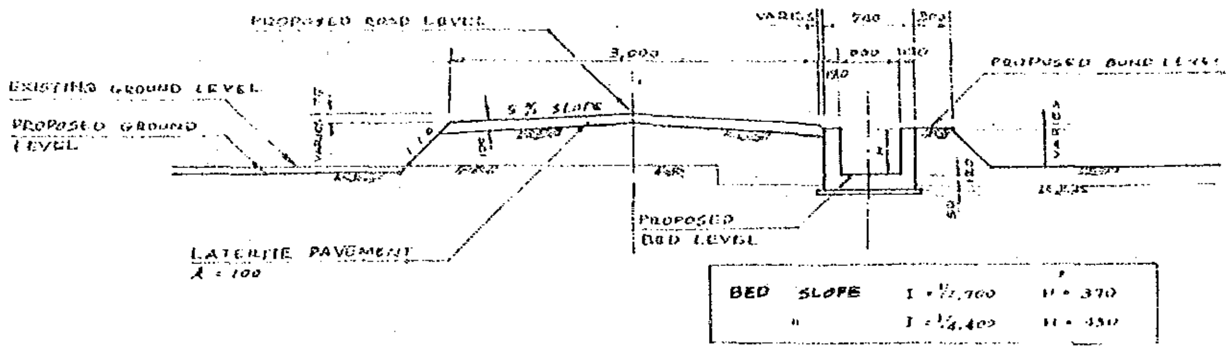
FARM ROAD AND PIPE LINE

SCALE 1:40



FARM ROAD AND OPEN CHANNAL

SCALE 1:40



F.R-2

SCALE 1:40