JICA

COMPILATION OF SERIES OF
TECHNICAL REPORTS
(Engineering Report)

VOL. 1

26 FEBRUARY 1982

Prepared by: Detailed Design Team JICA

LETTER OF TRANSMITTAL

Mr. Boonthai Otaganonta
Chief Engineer for Civil Engineering
Royal Irrigation Department

February 26, 1982

In response to the request made by the Royal Government of Thailand for technical cooperation in conducting a detailed design on the pipeline system between Dok Krai and Mab Ta Pud, the Government of Japan dispached the Study Team for detailed design hereafter referred as the study team to Thailand through the Japan International Cooperation Agency (JICA).

The study team arrived in Thailand on November 18th, 1981, and started the necessary studies and surveys on the various fields in close cooperation with the Royal Irrigation Department (RID), National Economic and Social Development Board (NESDB) and other authorities concerned of the Thai Government.

This report titled "Engineering Report" is prepared in accordance with the reporting schedule designated in the Inception Report include the result of topographic survey and geological survey along the planned pipe setting line to be connected from Dok Krai Dam to Mab Ta Pud, and review as the pipeline system in the Feasibility Study Report, and compilation of Series of Technical Reports.

The 30 coppies of engineering report are submitted to the Goverment of Thailand.

Y. Katayama
YUICHI KATAYAMA
Leader of the Study Team

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1. INTRODUCTION

During the meeting of presenting Progress Report 1, held in the middle of December 1981, JICA team, discussing on major issues, explained about the stage of decision.
Since then, the team has decided matters of both major and minor importance.

Each member of the team, with the assigned role and responsibility as a specialist, was requested to write reports. On the team's meeting not infrequently held since, each report was checked about the soundness of principles, the sufficiency of reasoning, the right approach of tackling the problem and the consistency with other reports.

The decisions made so far have not been reached by a particular person(s)'s reasoning but by the judement and concensus of the team members concerned, working on the above checkpoints.

As a part of Engineering Report, this paper is presented as the product of every participant's effort and comradeship.

2. LIST OF REPORTS

As of the end of February 1982, the following reports are in preparation or completed.

No.	Title	Author, Role		Stage
1.	Water Demand and Supply	KATAYAMA, Leader		completed
2.	Facilities	WAKAMOTO, Co-Leader		"
3.	Filtration Plant	"	"	"
4.	Organization for Operation and Maintenance	"	"	"
7-1.	Comparative Study of Flow Control System	TOMIOKA, Facilities		completed
7-2.	Hydraulics and Operation of Pump Unit Number Control System	"	"	"
8.	Head Tank	"	"	"
9.	Comparative Study of Pipeline, D.K.-M.T.P. - Sattahip	"	"	"
10-1.	Pumping System and Water Hammer	EMONOTO, Pumping		completed
10-2-A	Intake Structure	"	"	"
11.	Control and Emergency Shut-off Valve	NISHIDA, Pipeline		"
12.	Pipe Wall Thickness	"	"	completed
13.	Receiving Well, Receiving Reservoir	MIYAKE, Facilities		"
14.	Survey	ISHIZUKA, Surveyor		"

No.	Title	Author, Role	Stage
15.	Geology	NAKAJIMA, Geologist	completed
16.	Electrical Equipments	NAKAO, Electrical	"
17-1.	Comparison of Intake-Pump	SHIBATA, Mechanical	"
17-2.	Optimum Range of Pump Operation	" "	"
18.	Pipelaying	NISHIDA, Pipeline	"

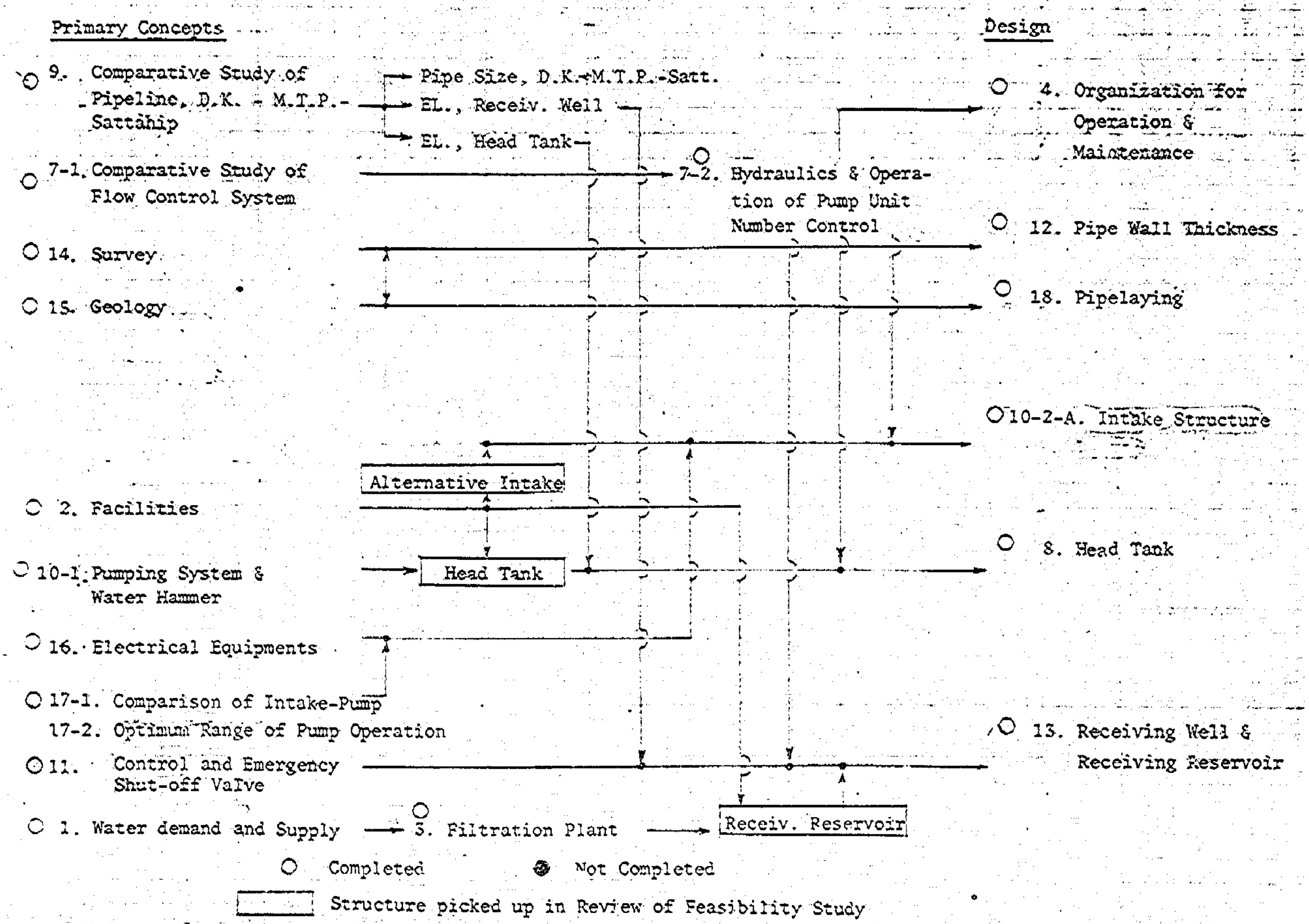
As No. 7 , 10 and 17 are made of two separate reports for each, and No. 5, 6 is removed, the number of reports is 20, all completed.

3. RELATIONSHIP OF REPORTS AND DEVELOPMENT OF CONCEPTS

These reports are related to each other as it can be seen on the next page.

The table will be helpful for the reader of reports as it shows that how the primary concepts have raised and focused on the technical issues of importance and how the issues have been narrowed down to decision making of the design.

Relationship of Reports and Development of Concepts



4. PROJECT FEATURES AND DECISION

The principal features of the pipeline system, including issues decided so far, are as follows:

4.1. General

(1) Design Discharge
$2.62\ m^3/sec$, (57.8 MG /yr)

(2) Filtration Plant (No.3 report)
Decided on no further study.

(3) Flow Control System (No.7-2 report)
Pump Unit Number Control.

(4) Organization for Operation and Maintenance (No.4 report)
Proposal presented.

(5) Mab Ta Pud - Sattahip Line (No.9 report)
Recommendation presented.

4.2. Dok Krai Intake, Pump Station

(1) High & Low Water Level for Intake (No.6, 10-2 reports)
54.10 and 42.00

(2) Type of Intake, Pump Station (No. 10-2 reports)
Concrete one is ongoing in design stage. Steel one is under detailed planning. The two shall be compared sooner.

(3) No. and KW of Pump (No.16, 17 reports)
6 No.s, 550-600 KW

(4) Type, Head of Pump (No.16, 17 reports)
Related to Structure of (2), either vertical shaft volute type or mixed flow type.
80-85 m Head is anticipated now and in the beginning of March, TOMIOKA brings the exact figure.

(5) P.C. Bridge
Locally made box girder of American Standards seems to be useful. Checking load condition is required. Piles for pier are now under study. Pipe bridge with ordinary thickness can be used.

(6) Electrical Equipments (No.16 report)
Discussions about principal matters.

(7) Air Chamber
Computer Analysis already requested to Japan.

4.3. Head Tank (No.8 report)

(1) Level, Size
102.00-103.00 possibly, TOMIOKA's arrival decides it.
Internal dia. 16m., Height 22-23 m possibly.

(2) Number of Tank
Needs discussion

(3) Level Meter (No.16 report)
Float Type most possible

(4) Overflow & Drain Pipeline
0.7 m dia. pipe, 400 long line is required with purchase of land.

(5) Construction Method
Reinforced and prestressed concrete tanks are now under conceptal design to be compared. Locally, prestressed tanks of large size (58 m H x 28 mD) have been built for sile.

4.4. Receiving Well and Reservoir (No.13 report)

(1) Level (No.9 & 14 reports)
61.50 decided. Land purchase plan made, including one more reservoirs lot.

(2) Size
Well : 10 m x 22.5 m x 3.5 m water depth, concrete
Res : 54 m x 189 m x 3 m water depth, 21,000 m^3
earthern pond (No.14, 15 reports)

(3) Outlet

Extra branch for extention shall be prepared. Gravitational flow to industries and pumping to Sattahip shall be expected.

4.5. Pipeline

(1) Wall Thickness (No.12 report)
10.3 mm possible, needs discussion

(2) Other Agencies (No.18 reports)
PTT, IEAT, TOT, PEA, MOT, DOH,
urgent contacts needed.

(3) Corrosion (No.15 report)
Needs study

(4) Alignment, Length
Working in JAPAN, less than 26.7 km

(5) Longitudinal Profile
Working in JAPAN

4.6. Purchase of Land

Started cooperation with L.S.C.D of RID

4.7. Geology (No.15 report)

Pipelaying (soil property), foundation, groundwater, sand and gravel for construction, corrosion

4.8. Office Living Quarter (No.4 reports)

Need studying No.4 and discussion

5. REARRANGEMENT OF REPORTS

The inception report, in 4.1, wanted to review on the interim of feasibility study and the detailed design team has prepared No.1 Water Demand and Supply.
In 3.3 of the inception report, the purification plant is refered and it has been materialized as No. 3 report.

Engineering Report will, therefore, consist of :

1. Compilation of Series of Technical Reports
2. Survey Report (No.14) with maps.
3. Geology Report (No.15) with a data book
4. Meteological and Hydrological Data

2,3,4 are bound separately for other usage.

No 1 WATER DEMAND AND SUPPLY

FEBRUARY 1982

Prepared by YUICHI KATAYAMA
Team Leader
JICA

THE EAST COAST WATER DEVELOPMENT PROJECT

(1) The water resources available to the East Coast Area is very limited because of the annual rainfall of Changwat Chon Buri and Rayong area is 1,300 mm and 1,500 mm. Besides the estimated annual evaporation is about 1,000 mm.

(2) To ease the chronic shortage and to meet the acute water demand of industrial as well as agricultural development schemes, The Thai Government have come up with a comprehensive water resources development scheme in this area.

(3) The comprehensive water resources development plan for Changwat Chon Buri and Rayong has been formulated based on the estimation of water demands and potential water resources, and the construction of Nong Pla Lai Project has been given the first priority to meet the increasing water demands.

(4) To meet the urgent municipal and industrial water demand in Mab Ta Pud - Sattahip Area, ahead of the completion of Nong Pla Lai Dam the pipeline system from Dok Krai to these Area will be constructed and convey 57.8 MCM/year of municipal and industrial water.

Water Demand

(5) The water demand which is expected to occure in Sattahip and Rayong Area has been estimated as follows:

Table 1. Total Water Demand in Sattahip and Rayong Area

Year	1990	2000
Industry	36.8 MCM	45.1 MCM
Municiparity	16.8	34.9
Total	53.6	80.0

(6) The industrial demand which is expected to occure from various proposed industries in Sattahip and Rayong Area has been estimated on the basis of the actural consumption at industrial complexes and ports in operation, shown in Table 4.

(7) Municipal Water Demand has been estimated based on expected population growth of the area. The estimation of future population based on current trend, and subject to industrial development project. (Refer to Table 5, 6)

The estimate shows that the future population in Changwat Chon Buri would reach 900 thousand in 1990, and 1070 thousand in 2000: in Changwat Rayong 500 thousand in 1990 and 620 thousand in 2000.

(8) Estimate Methodology of the municipal water demand is as follows:

$$MWD = ((Pn \times Up \times Wp) + Ip) \times PCC \times 365 \text{ days} \times Rc$$

Where : MWD : Municipal water annual demand
Pn : Population
Up : Urban population ratio
Wp : Water pervation
Ip : Induced population
PCC : Per capita consumption
Rc : Raw water converter (1.1)

Assumption figures such as PCC, Wp, Up of appropriate years are shown below:

Table 2 Assumption

		Up (%)	Wp (%)	PCC t
1980	Chon Buri	30.0	45.3	0.345
	Rayong	9.9	58.6	0.22
1990	Chon Buri	35	60	0.35
	Rayong	30	70	0.30
2000	Chon Buri	45	75	0.35
	Rayong	40	80	0.35

The above assumption was made in line with the Eastern Sea Board Development plan.

(9) The municipal water to be demanded by the growing population of Sattahip and Rayong Area is estimated as shown in Table 7.

Water Resources

(10) To make use of 1300 - 1500 mm/year water resources which flows down as a surface flow, construction of dams is essential owing to the characteristic climate and topography of this area.

(11) The surveys have already been carried out to select the appropriate sites for construction of Dams in the project area of East Coast. Proposed dams include the three listed here which bring about a total of approximately 220 MCM/year.

Table 3 List of Proposed Dams in the Project Area

Proposed Dam	Location	Catchment Area (Km^2)	Storage Capacity (MCM)	Developed Water (MCM/Year)
Nong Pla Lai	Changwat Rayong	426	144	120
Thap Ma	Changwat Rayong	154	50	30
Khlong Yai	Changwat Rayong	223	70	70

(12) In Dok Krai river, the other hand, a tributary of Rayong river, the Dok Krai Dam with catchment area 279 Km^2, storage capacity 54 MCM, developed water 60 MCM/year, had been constructed and is supplying water to Ban Khai Irrigation Project. The demand of this irrigation is relatively small at present, and the Dok Krai Dam and its residual basin have surplus water amount of approximately 20 MCM/year.

Water Resources Development Plans

(13) Priority has been given to the order of construction of the proposed dams based on the nature of arising demand including its time and location.

(14) The highest priority is given to the construction of Nong Pla Lai Dam. The proposed Nong Pla Lai Dam is located relatively closer to Rayong area where the largest demand of water is expected. Nong Pla Lai Dam is scheduled to be completed in 1986 with the water development volume of 120 MCM/year with residual basin flow, the water is to be conveyed to A.M Rayong, a principal development center in the East Coast, to Mab Ta Pud with proposal industrial complex, and to Sattahip.

The water is to be supplied for industrial and municipal use and to supply irrigation water to newly developed tract in the middle reaches fo Rayong river.

Water Balance of Supply and Demand

(15) The water balance study has been made on the basis of the projected water demands in the industrial development centers of Rayong, Sattahip and their neighboring areas, also on the basis of the supply to made available by the development plan in these areas.

(16) Transmission System between Dok Krai Dam and Mab Ta Pud would be meant for the increase of industrial and municipal water demands in 1984. The proposed Nong Pla Lai Dam is scheduled to be completed in 1985, and supply the sufficient volume of water to the project area until 2000.

Water Pipeline System from Dok Krai to Mab Ta Pud and from Mab Ta Pud to Sattahip

(17) To meet the water demand of 8MCM/year expected at the Natural Gas Separation Plant scheduled to start operation in the middle of 1984, a water pipeline system between Dok Krai Dam and Mab Ta Pud will be constructed in advance of Nong Pla Lai Dam. The water pipeline system is to convey about 20 MCM/year of surplus water from Dok Krai Dam for industrial and municipal use until the completion of Nong Pla Lai Dam.

(18) After completion of Nong Pla Lai Dam, the present irrigation water supply function (60 MCM/year) of Dok Krai Dam will be converted to Nong Pla Lai Dam which will then supply only irrigation water. The expected demand of 80 MCM/year for industrial and municipal in Rayong and Sattahip area up to the year 2000 will be fulfilled by Dok Krai Dam.

(19) In Sattahip area, 12 MCM/year of water shortage will occur in 1985. The water from Dok Krai Dam shall be conveyed there by laying out of the pipeline system from Mab Ta Pud to Sattahip.

(20) 57.8 MCM/year of 80 MCM/year will be conveyed to Mab Ta Pud - Sattahip area by pipeline system and remaining 22.2 MCM/year to A. Rayong and its vicinity area by the Rayong river. (refer to Table 8.)

Table 4 WATER DEMAND FOR INDUSTRY IN THE PROJECT AREA

Year	Rayong	Increase demand		Sattahip	Increase demand		Total: Increase demand	Total: Total
1984	Gas Separation Petrochemical	7.8	7.8				7.8	7.8
1985	Sponge Iron	1.0	(8.8)	Soda Ash	10.2	10.2	20.7	28.5
	Chemical Fertilizer	9.5	18.3					
1986	Industrial Estate	2.4	20.7	Sattahip Port	2.1	12.3	4.5	33.0
1990	Industrial Estate	2.4	23.1	Sattahip Port	1.4	13.7	3.8	36.8
1995				Sattahip Port	0.6	14.3	0.6	37.4
1996				Sattahip Port	1.2	15.5	1.2	38.6
2000	Industrial Estate	4.8	27.9		1.7	17.2	6.5	45.1

Table 5 Future Population Based on Current Trend

Changwat Rayong

	1980	1990	2000	1990 - 1980	2000 - 1980
Rayong Municipality*	37,305	56,629	79,773	19,324	42,468
A. Muang*	83,693	90,474	93,065	6,781	9,372
A. Klaeng	100,484	127,383	154,338	26,899	53,854
A. Ban Khai*	71,190	77,522	83,524	6,332	12,334
K. A. Pluak Daeng	25,791	30,804	35,676	5,013	9,885
K. A. Ban Chang	27,594	28,264	29,047	670	1,453
K. A. Wang Chang	12,839	21,414	29,834	8,575	16,995
Sub Total	358,896	432,490	505,257	73,594	146,361

Changwat Chon Buri

	1980	1990	2000	1990 - 1980	2000 - 1980
Chon Buri Municipality	50,106	52,897	55,557	2,791	5,451
Panat Nikhon M.	13,411	14,408	15,392	997	1,981
Tambom Si Racha M	21,632	32,611	43,339	10,979	21,707
A. Muang Chon Buri	119,281	150,115	180,290	30,834	61,009
A. Panat Nikhon	110,203	126,154	142,024	15,951	31,821
A. Pan Thong	38,289	42,069	45,957	3,780	7,668
A. Ban Bung	78,262	83,894	89,555	5,632	11,293
A. Si Racha	84,516	100,426	116,795	15,910	32,279
A. Ban La Mung	43,789	45,824	47,765	2,035	3,976
A. Sattahip*	85,112	98,377	111,528	13,265	26,416
K. A. Ko Si Chang	2,955	3,553	4,157	598	1,202
K. A. Nong Yai	17,386	20,486	23,491	3,100	6,105
K. A. Bo Thong	24,779	36,579	48,372	11,800	23,593
Muang Pattaya	34,706	59,380	84,173	24,674	49,467
Sub Total	724,427	866,773	1,008,395	142,346	283,968
Total	1,083,323	1,299,263	1,513,652	215,940	430,329

* Nong Pla Lai Project

Table 6 Estimation of Future Population
including Induced Population

Changwat Rayong

	1980	1990	2000	1990 - 1980	2000 - 1980
Rayong Municipality*	37,305	56,629	79,773	19,324	42,468
A. Muang*	83,693	160,758	203,349	77,065	119,656
A. Klaeng	100,484	127,383	154,338	26,899	53,854
A. Ban Khai	71,190	77,522	83,524	6,332	12,334
K. A. Pluak Daeng	25,791	30,804	35,676	5,013	9,885
K. A. Ban Chang	27,594	28,264	29,047	670	1,453
K. A. Wang Chang	12,839	21,414	29,834	8,575	16,995
Sub Total	358,896	502,774	615,541	143,878	256,645

Changwat Chon Buri

	1980	1990	2000	1990 - 1980	2000 - 1980
Chon Buri Municipality	50,106	52,897	55,557	2,791	5,451
Panat Nikhon M.	13,411	14,408	15,392	997	1,981
Tambom Si Racha M	21,632	32,611	43,339	10,979	21,707
A. Muang Chon Buri	119,281	150,115	180,290	30,834	61,009
A. Panat Nikhon	110,203	126,154	142,024	15,951	31,821
A. Pan Thong	38,289	42,069	45,957	3,780	7,668
A. Ban Bung	78,262	83,894	89,555	5,632	11,293
A. Si Racha	84,516	124,426	166,483	39,910	81,967
A. Ban La Mung	43,789	45,824	47,765	2,035	3,976
A. Sattahip*	85,112	105,161	122,536	20,049	37,424
K. A. Ko Si Chang	2,955	3,553	4,157	598	1,202
K. A. Nong Yai	17,386	20,486	23,491	3,100	6,105
K. A. Bo Thong	24,779	36,579	48,372	11,800	23,593
Muang Pattaya	34,706	59,380	84,173	24,674	49,467
Sub Total	724,427	897,557	1,069,091	173,130	344,664
Total	1,083,323	1,400,331	1,684,632	317,008	601,309

* Nong Pla Lai Project

Table 7 WATER DEMAND FOR INDUSTRIAL AND MUNICIPAL USE

	1. Rayong				2. Sattahip				3. Total			
Year	Industry	Industry -related municipal	Other municipal	Total	Industry	Industry -related municipal	Other municipal	Total	Industry	Industry -related municipal	Other municipal	Total
1980			1.5	1.5			0.3	0.3	-	-	1.8	1.8
1981			1.8	1.8			0.6	0.6	-	-	2.4	2.4
1982			2.1	2.1			0.9	0.9	-	-	3.0	3.0
1983			2.4	2.4			1.1	1.1	-	-	3.5	3.5
1984	7.8		2.7	10.5			1.4	1.4	7.8	-	4.1	11.9
1985	18.3		3.0	21.3	10.2	0.3	1.7	12.2	28.5	0.3	4.7	33.5
1986	20.7	3.7	3.2	27.6	12.3	0.6	2.0	14.9	33.0	4.3	5.2	42.5
1987	20.7	3.7	3.5	27.9	12.3	0.6	2.3	15.2	33.0	4.3	5.8	43.1
1988	20.7	3.7	3.8	28.2	12.3	0.6	2.5	15.4	33.0	4.3	6.3	43.6
1989	20.7	3.7	4.1	28.5	12.3	0.6	2.8	15.7	33.0	4.3	6.9	44.2
1990	23.1	8.5	4.4	36.0	13.7	0.8	3.1	17.6	36.8	9.3	7.5	53.6
1991	23.1	8.5	4.9	36.5	13.7	0.8	3.3	17.8	36.8	9.3	8.2	54.3
1992	23.1	8.5	5.3	36.9	13.7	0.8	3.5	18.0	36.8	9.3	8.8	54.9
1993	23.1	8.5	5.8	37.4	13.7	0.8	3.8	18.3	36.8	9.3	9.6	55.7
1994	23.1	8.5	6.3	37.9	13.7	0.8	4.0	18.5	36.8	9.3	10.3	56.4
1995	23.1	8.5	6.8	38.4	14.3	0.9	4.2	19.4	37.4	9.4	11.0	57.8
1996	23.1	8.5	8.0	39.6	15.5	1.2	4.4	21.1	38.6	9.7	12.4	60.7
1997	23.1	8.5	9.2	40.8	15.5	1.2	4.6	21.3	38.6	9.7	13.8	62.1
1998	23.1	8.5	10.4	42.0	15.5	1.2	4.9	21.6	38.6	9.7	15.3	63.6
1999	23.1	8.5	11.6	43.2	15.5	1.2	5.1	21.8	38.6	9.7	16.7	65.0
2000	27.9	15.5	12.6	56.0	17.2	1.5	5.3	24.0	45.1	17.0	17.9	80.0

Table 8 Industrial and Municipal Water Demand in Sattahip, Mab Ta Pud Rayong Area.

Year	SATTAHIP AREA				MAB TA PUD AREA				Pipeline
	Industry	Industry - related Municipal	Other Municipal	Sub-total	Industry	Industry - related Municipal	Other Municipal	Sub-total	Total
1984			1.4	1.4	7.8			7.8	9.2
1990	13.7	0.8	3.1	17.6	21.5	5.3	0	26.8	44.4
2000	17.2	1.5	5.3	24.0	21.5	12.3	0	33.8	57.8

Year	A. RAYONG AREA				TOTAL			
	Industry	Industry - related Municipal	Other Municipal	Sub-total	Industry	Industry - related Municipal	Municipal	Sub-total
1984			2.7	2.7	7.8		4.1	11.9
1990	1.6	3.2	4.4	9.2	36.8	9.3	7.5	53.6
2000	6.4	3.2	12.6	22.2	45.1	17.0	17.9	80.0

NO.2 REPORT ON FACILITIES

January 1982

Prepared by OSAMU WAKAMOTO
Co-Leader, Civil Engineering
Detailed Design Team, JICA

CONTENT

1. Introduction

2. Pipeline System as a whole

2.1. Intake Tower with Pump
2.2. Head Tank
2.3. Pipeline
2.4. Receiving Well and Receiving Reservoir

3. Summary

1. Introduction

The detailed design team has to "review on the existing data and the interim of feasibility study" accoridng to the inception report.

The author has reviewed 2. Project Description, 5. Project Formulation of Main Report and VII Pipeline of Supporitng Report, both of them "interim" and like to discuss on the matters concerning facilities.

2. Pipeline system as a whole

From a fluctuating level of Dok Krai Reservoir water must be transmitted to a certain level of receiving tank, supposedly located in Mab Ta Put. Though the most of water is distributed within the area, the rest must be sent to Sattahip therefrom. Exmaing Interim Reports, we think that the following conditions are unchangeable.

(1) Pumping up water, sending it by 1.35 m dia. pipeline and receiving at Mab Ta Put.

(2) Discharging water in Dok Krai Reservoir and lowering the level for construction of the intake is impossible. Then, it must be done under the existing high level condition.

(3) The pipeline's route runs along Highway Route 3191 and 3 for the most part. In the part of downstream, it has to be laid parallel to the natural gas pipeline closely.

(4) As the pipeline has to run over high crests at some points along the route, hydraulic consideration must be given about the condition and necessary steps must be taken for it.

(5) The construction work must be completed before a target time. Implementation of the project will be very tight in the matter of time.

With the above requirements in mind, the Detailed Design Team will discuss about the facilities together with the system.

2.1. Intake Tower with Pump

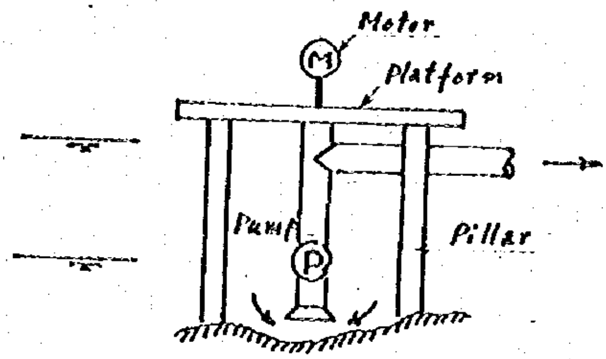
The Feasibility Study, in 4.1.2. of Supporting Report, compares three types of the intake method in combination with different types of pump. But, what is done to conclude is: compare three types of pump from technical and cost points rather qualitatively, match the three to five types of intake tower and compare them again and rank the five in order.

Looking at the conclusion "Priority rating" and comparing with "construction cost" on Table 4-7 of Supporting Report, it is not convincing enough.

The Detailed Design Team will start from considering about the basic viewpoints.

(1) Drawing from Bottom only

If the water drawn from the bottom is always favourable for use of industries and municipalities, the possible intake and pump will be like that is shown below:



Pump

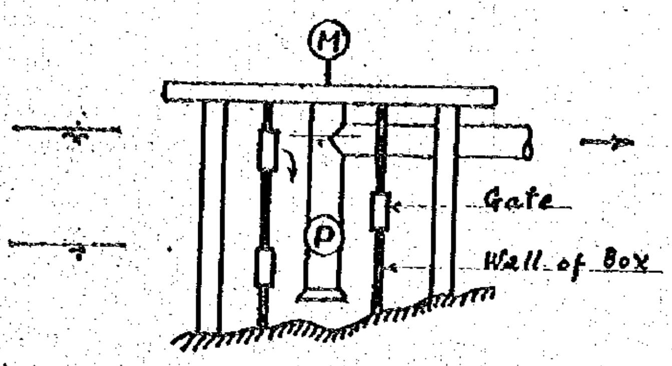
Two types are feasible. One is a pump consisting of submerged pump part and above-the-water motor part, called mixed flow or axial flow type. Another is a pump called submerged motor type in which both the pump and motor are in the water.

Intake structure

The intake structure can take the simplest form, that is; a platform supported by pillars. The platform is for suspending the pump. The points about the type (combination of pump and structure) are:

* The pump's head-capacity curve and the efficiency-capacity % curve are not as good as the volute type's.
* The water can be drawn only at the bottom.
* The pump is not protected from floating matters.
* The structure is simple and easy to construct in the water.

(2) Drawing from Chosen Depth (Type B1 in Supporting Report)

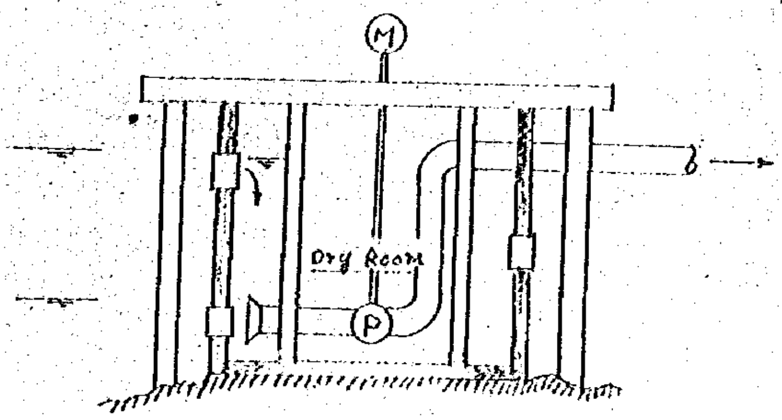


To (1) a box with several gates is added. The box, surrounding the pump at the bottom and the sides, protects the pump and also makes it possible to draw water from a chosen depth.
The inside of box is filled with water and the pump is the same type as (1).
The points about (2) are:

* The pump's curves are the same as (1).
* The water can be drawn from chosen depth so that it would be better suited for industrial and manicipal use.
* The structure is a little more difficult and costly than (1). As the box can be lowered from the platform, construction in the water is as easy as (1).
* The box, being filled with water, need not be very strong.

(3) Drawing from Chosen Depth and Pump in Better Conditions
(Type B3 of Supporting Report)

The volute type pump of better efficiency and flexibility against the water level fluctuation can only be installed in a dry room which is put in the box of (2) with gates.



With the type, the pillars and the wall of box can be made up to a single wall. The dry room in the center of whole structure must be strong enough against the external water pressure and it must be grounded firmly.

The points about the type are:

* The pumps curves are better than (1).
* The water can be drawn from chosen depth.
* The room must be firmly grounded and be strong.
* The cost of structure is higher than (1) and (2) and reliable construction method must be conceived.

As is easily seen, additional or better requirement has forced to change the type, from the simple to the complicated and most possibly increasing the cost. If one requirement is removed or changed, the type can also be changed.

For instance, if the lower efficiency is tolerated and submersible type can be used, the type discussed in (2) can be accepted.

The Detailed Design Team is going to study the type in (2), with possible much more use of steel in place of concrete.

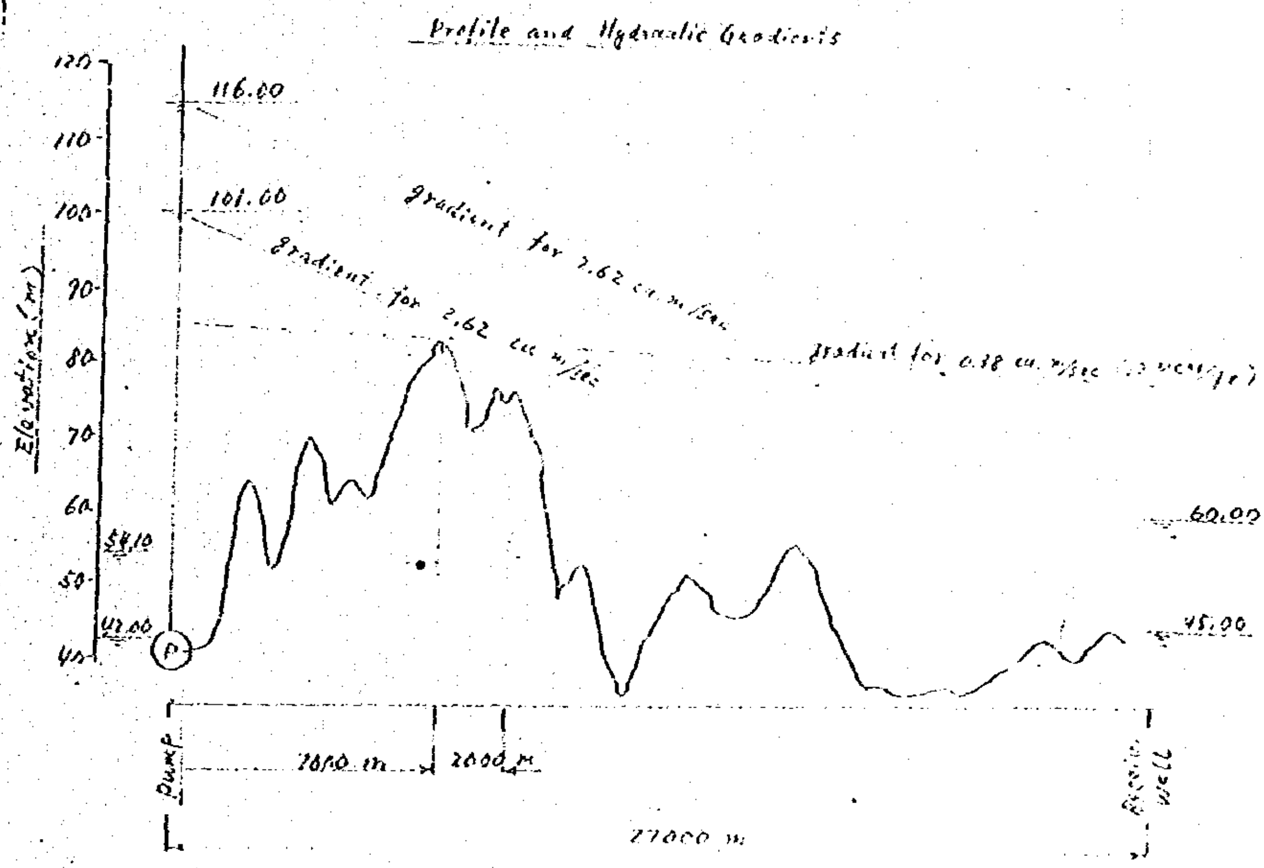
2.2. Head Tank

The reason of necessitating a head tank is not clearly stated. Only in 4.3. of the Supporting Report, the storage capacity is discussed related to the on-off frequency of pumps. In other parts of the Report, preferance of "open type pumping system" to "closed type pumping system" is sometimes described with no explanation of the words, although "Open type" means "with head tank" without doubt.

The Detailed Design Team understands the reason as primarily restricting the adverse effect of water hammer and secondarily giving storage to make the operation easy.

(1) Hydraulic Gradient, Necessary Head

Already the design flow rate 2.62 cu.m/sec and the diameter of pipe 1.35 m are given. The necessary hydraulic gradient is calculated from Hazen Williams Formula with a suitable coefficient of roughness (c) assumed. With c = 120, the hydraulic gradient is approximately 0.00208 or the necessary head for 27,000 m length is 56 m roughly. It means that when the level in the end is 60 m the water must be lifted to 116 (60+56)m level by pumping. As the receiving well's level will be set approximately 45 to 60 m range, the Dok Krai pump's delivery will be 101 to 116 m range. One thing more required with the hydraulic gradient is that every part of the pipeline must below the gradient line. If some parts are above it, the negative pressure there will cause bubbling of the dissolved air and as the result released air will restrict the flow until it clog the pipe.
Shown below is the hydraulic gradient and the topographical profile of the pipeline:



At 7 and 9 km point from Dok Krai, two passes (crests) are on the pipeline route. As is shown on the picture, these two points are a little below the lower gradient line.
The picture means that with the given conditions, negative pressure will not occur on the pipeline.

(2) Possibility of Syphoning

When the pipeline is put under operation is 1984, the initial flow rate will be about 12 MCM/yr (0.38 cu.m/sec disregarding fluctuation and leakage) or 14.5% of the design flow rate 2.62 cu.m/sec. The hydraulic gradient is then 0.0000582 or 2.8% of the gradient 0.00208 for the design flow rate.
Under the condition, the requirements to make it flow is:

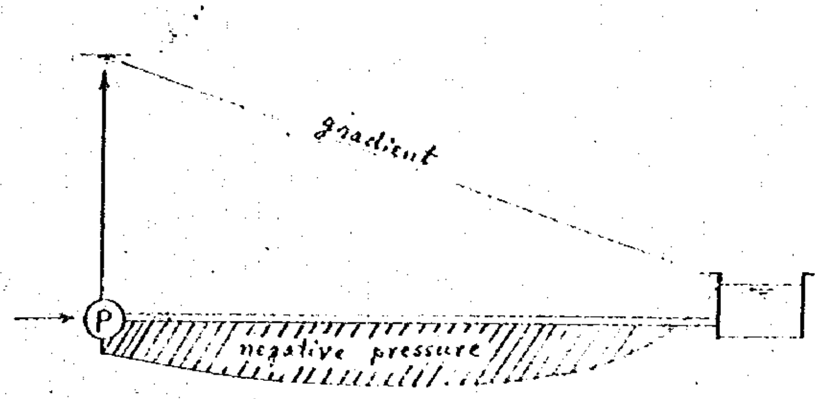
* The valve at the receiving well must be slightly opened or nearly fall-closed.
* The delivery valve of the pump also must be slightly opened.

With the operation of valves with running pump, a gradient line shown as dotted line will be formed and the flow will start.
In a favorable condition like that the Dok Krai level is 52.00 and the receiving well's level 45.00, 7 m head seems to be able to keep the flow even when the pump is stopped. That is, once the flow started, it may continue so with syphoning. Actually it stops instantly as water cannot be lifted up to 10m even theoretically.

(3) In case of Power Failure, Water Hammer

Water hammer occurs with sudden change of flow in the pipeline, caused by pump's start or stop and valve's start or stop and valve's rapid opening or closing.
As for the pipeline under discussion rapid operation of valves is unthinkable, though pump's stoppage due to power failure will certainly occur once in a while and resulting water hammer is inevitable.

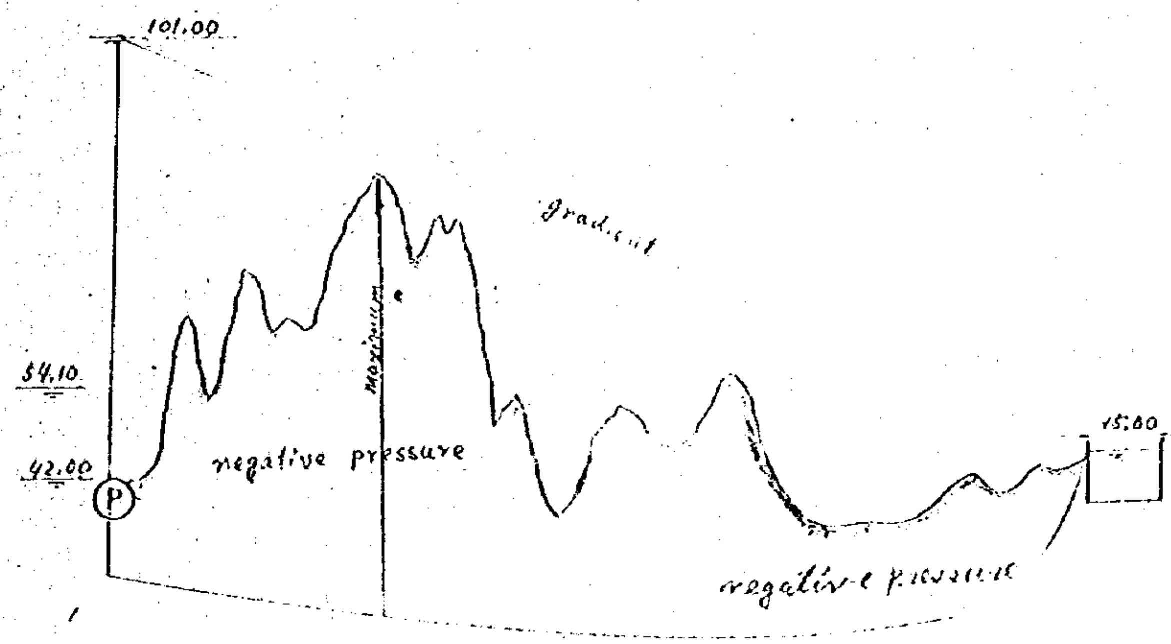
Pipeline on Flat Land



Let us assume a pipeline is laid on a flat land and a flow condition shown by the gradient line is existing.
When the pumps stops, the negative pressure occurs. At the downstream ends it is zero. Somewhere in-between, it takes the maximum value and decreases towards the end.
The hatched line is formed by connecting the minimum value at each points on the pipeline. It does not mean that the negative pressure like the figure occurs simaltaneously all over the pipeline

Pipeline on Hilly Land

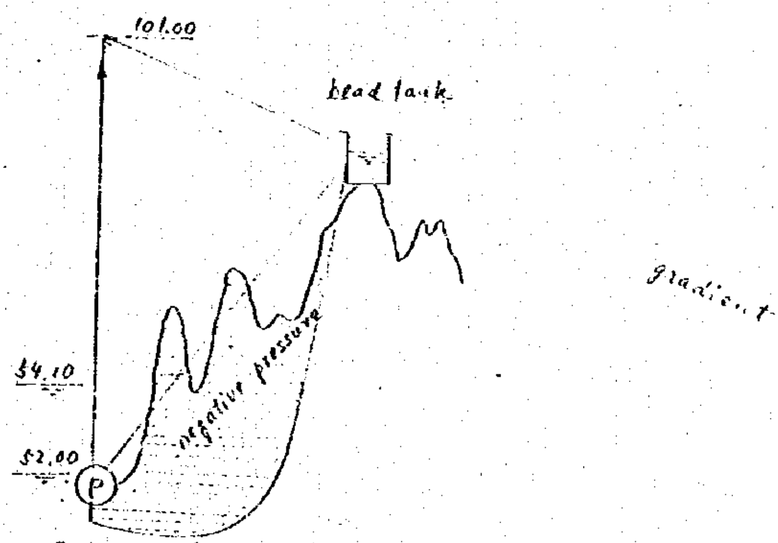
Even on the pipeline on flat land, negative pressure occurd below the line connecting pump and the tank level.



In our case, except some part in the middle and the downstream portion the most part of the pipeline is laid above the line connecting pump and, the level of receiving well. When negative pressure arises in the pipeline it will be like that shown above.

(4) Head Tank at Crest

If a head tank is set up on the high crest, the change will be like that shown below.



Between the head tank and the receiving well no sudden change will occur even when the pump stops. The pressure wave which starts at the pump will reach the head tank and does not go further down. The downstream part can be forgotten as far as the water hammer problem is considered.

Still between the pump and the head tank it exists and must be met with some gadget including air chambers proposed in the Feasibility Study.

The Detailed Design Team accept the necessity and the usefulness of the head tank after reasoning as explained.

(5) Location of Head Tank

The Feasibility Study seems to have decided the location of head tank on a hill of granite (2.2.2. Supporting Report) in the earlier stage.

The hill is about 1 km apart from Route 3191.
Because of the réason explaining the necessity of head tank, for the distance of 1 km two pipelines, one for supplying to and other for drawing from the tank, are necessitated. Dynamite shall be used to clear the hill and to level around it for construction of tank.
As is easily seen, the location of head tank can be changed by the hydraulic condition.
The Detailed Design Team is now studying about it. It may possibly moved closer to Route 3191 where soil conditions allow the construction and it will save the cost and the period.

(6) Material

The Feasibility Study's conclusion that prestressed concrete is the most advantageous cannot be accepted by the Detailed Design Team. It may somehow be related to the preoccupation that the tank is to be located at the top of hill.
Usually the prestressed concrete structure is not suitable where uneven ground subsidence may occur and is not necessarily lower in the cost than other materials.
Most possibly a steel tank will be unsuitable. But a reinforced concrete tank and an earthan pond are worth studying.

(7) Storage for What Use

30 minutes' storage capacity in 4.3. of the Supporting Report seems to have come from protecting a pump which is better be given more than 30 minutes pause between stoppage and restarting. It is not convincing.
As the head tanks' existence is to control the adverse effect of water hammer, so its storage is useful for preventing the pipeline from being emptied in case of pump's stoppage.

The water in the part between the intake and the head tank is checked from flowing by the check valve of pump. The part between the tank and the receiving well tends to flow out unless the valve of receiving well is immediately closed. The storage will cover the flow out between pump's stoppage and valves's closing.

2.3. Pipeline

Selection of Materials

In 3.2.4. of the Supporting Report, the Feasibility Study pointed out four factors, strength, leakage, anti-corrosion and availability while on Table 3-3, 3-4, then comparison is made on the factors. The two Tables shall be examined and discussed here.

Strength

The concrete pipe is said to need "a suitable protective work". Here, the pipeline will be under a light traffic load or no load. Still is it necessary? Usually the type is good against external force like earth pressure.

"not suitable for use under high pressure". Against how big pressure can it stand , must be stated.

The ductile cast iron pipe is not a "substitute" of steel pipe. In the world, it is far more widely used than steel pipe.

Leakage

Mechanical joints' leakage. In Bangkok, a transmission line to Thongburi Treatment Plant, uses 0.9 m dia. ductile cast-iron pipe with practically no problem of leakage for years.
Welded joints are leakage proof.
And so are mechanical joints practically.

Corrosion

The Table says that the ductile cast-iron pipe will corrode, while the steel pipe, though corrosive in quality, can be protected by suitable means.
The description is too misleading.

Pipelaying Works

About the ductile cast-iron pipe. It says that digging wider trench for jointing pipes in more places than for the steel pipes, means "highest construction cost". This will be quite small for the total construction cost, the Detailed Design Team thinks.
About the steel pipe. At poor subsoil area, even the steel pipe needs good foundation against subsidence.

Construction cost and Conclusion

The Table quote the construction cost 619 US$ for the ductile cast-iron pipe and 596 US$ for the steel pipe (per meter?) and concludes the cost is the highest of the three (including the concrete pipe) for the ductile cast-iron pipe.

(3) Detailed Design Teams' Thinking

The Feasibility Study seems to have mixed the general conditions for selecting the pipe material and the specific conditions under which applicability of the general conditions is to be studied.

The Detailed Design Team thinks as follows:

* The concrete pipe (may be of prestressed type) may be acceptable where the construction period is long enough, the importance of supply is lower in such as irrigation etc., the flow is less (smaller size increases reliability), the protection can be better with lighter traffic's load and vibration.
* The ductile cast-iron pipe and the steel pipe will stand equal if the case is that the both are to be imported from overseas.
* The decisive factor for the steel pipe and against the ductile cast iron pipe is that the former can be manufactured here in Thailand. The availability and the lower cost due to far less custom tariff.

2.4. Receiving Well and Reseiving Reservoir

The Detailed Design Team likes to propose a new concept of the Receiving Reservoir here.

(1) The Background

In Japan, the industrial water supply system was introduced in the latter half of 1950s with establishment of the law for it and setting up a department in the Ministry of Industry and Trade. The aim was to supply the industrial belts near major cities water for industrial use, in sufficient quantity and of lower quality and cost. The construction cost was partly covered by the Government grant and implementation of construction, maintenance and operation was put under the major cities' administration. The system should be self-supporting as public enterprise.
In later years together with expansion of the major cities' supply system, new applications were submitted from the middle size cities and the number of existing systems will amount about 100.
Apart from the legislative and institutional matters, some interesting features are to be pointed out here:

* The tariff was about 1/4-1/5 of potable water of the same municipalities.
* The contract between the consumer and the supplier made the former to pay for the agreed amount, notwithstanding the consumption. It was a kind of norm to the consumer.
* The consumer was obliged to build a storage tank to prepare for unexpected failure of supply. The storage capacity was to be agreed upon.
* The supplier guaranteed the quality, related to PH, temperature and turbidity, within a range.

The systems seem to have contributed the progress of industries in Japan since and have been effective in controlling unorderly lifting of ground water and resulting ground subsidence. Actually the government encouraged introduction of the system where the problem of ground subsidence was noticable.

(2) Lesson

For the planned industrial area, the water supply is so vital that storage of water there is apparently important. So far the project has not discussed about it, probably considering about the huge storage at Dok Krai Reservoir and relying on the reliability of pipeline for unceasing supply.
In Japan where the land cost is extremely high and acquisition of land (for the storage) in the existing industrial area is certainly impossible, the precausion of supply stoppage had to be turned over to the consumers who could somehow manage to find the room for it in their compound.
At this stage and in this country, the considerations given by Japanese Government to industries will be impossible, even if conceivable.
One thing which is possible and worth consideration is to prepare storage by the supplier.

(3) Storage : Other Use, Location, Size

The other use of the Receiving Reservoir is that it can work as a sedimentation tank to treat extraordinary turbidity and to produce good quality water for industries. The location shall be adjacent to the receiving well. The size shall be for 3 hrs' storage at the design flow rate 2.62 cu.m/sec.
3 hrs is chosen for the treatment and seems to be sufficient for the industries to make emergency plans to meet the water supply failure of long duration.
The Receiving Reservoir will be discussed in the other report.

(4) Receiving Well

The Feasibility Study specifies 2 minutes volume in 4.4. of the Supporting Report.
As it will be discussed in the other report, 5 minutes of the design flow rate or 786 cu.m capacity is proposed by the Detailed Design Team.

3. Summary

The Detailed Design Team, in the beginning of this Report, found that only a few conditions were unchangeable. Other matters, even when they seemed to have been concluded, were flexible and changeable, where review could be made with more sufficient reasoning.
The Detailed Design Team had tried to reason about the principles of each facility in the beginning and then to follow the logical progress until some conclusion was proved.
The conclusions are summarized as follows.

(1) Intake Tower

The Feasibility Study concluded the plan under three conditions. When one condition is removed, a new alternative can be conceived to save the cost and the construction period. It is worth studying.

(2) Head Tank

The Feasibility Study did not elaborate the necessity. With reasoning, it is proved to be necessary to control the water hammer problem. About the location, structure and size, better choice to save the cost can be made.

(3) Pipeline

About the selection of materials the Feasibility Study Report would better be revised.

(4) Receiving Well

A larger well is disirable to facilitate operation.
The Detailed Design Team recommends a Receiving Reservoir to be built closely next to the Receiving Well.
These matters are to be discussed in the other report.

NO.3 REPORT ON FILTRATION PLANT

December 1981

Prepared by OSAMU WAKAMOTO
Co-Leader, Civil Engineering
Detailed Design Team, JICA

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1. Layout of Filtration Plant 1:1000
2. Overall Layout 1:2000
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1. Foreward

The detailed design teams' scope of work includes some matters about the prospective filtration plant. It says:
The location of the plant will be decided by taking account of the urban and industrial demands in Mab Ta Put area and the topographical conditions there. The location must be fully studied, reviewing the results of the Feasibility Study.

Together with it, the water treatment process and the size of plant shall be approximately discussed and commented.

With the above background, the detailed design team submit a report.

2. Approach

2.1. Necessity of Treatment

Under the Eastern Seaboard Program, an industrial complex is to be materialized in 1980 to 2000, in the area of Sattahip-Mab Ta Put - Rayong along the shoreline and Route 3.
The population will increase remarkably as a large inflow of developement - induced transmigration occur, adding to the natural increase of the original inhabitants.
Some portion of the increased population will live in newly developed urban area provided by industries and the public and private sectors. Other portion will live in and around the existing cities, towns and villages.
Not only the newly developed urban area but also the existing area where no potable water supply is available, certainly necessitate a reliable potable water supply system.
The industries and seaports consumes the water transmitted by the planned pipeline. Water-consuming-type industries like Soda ash, chemical fertilizer, gas separation, sponge iron and steel plate are already either ongoing into construction or under planning.
Non-water-consuming-type industries, some downstream ones of the mentioned and others for supporting of the same will crop out naturally or by inducement.
These industries, heavy and light, use water for widely spread purposes in quality and quantity.
With the situation of water usage in industries, an collective treatment is unrealistic.
Concluding the paragraph, treatment for potable supply is necessary and that for industrial supply is unnecessary.

2.2. Consideration for industries

Though no treatment for industrial supply is concluded, still some consideration will be paid.

When the water level in a reservoir is low and the inflow from upstream is high, the turbidity might grow high, caused by erosion of land and by suspension of fine clay particles.
The measures which will be considered in the detailed design to meet the situation will be :

(1) Intake Tower

Within about 10 m difference between the highest and the lowest water level, 3 to 4 intake gates at different height shall be prepared, so that less turbid water is taken by controlling them. It also aims to prevent sedimentation in the pipeline.

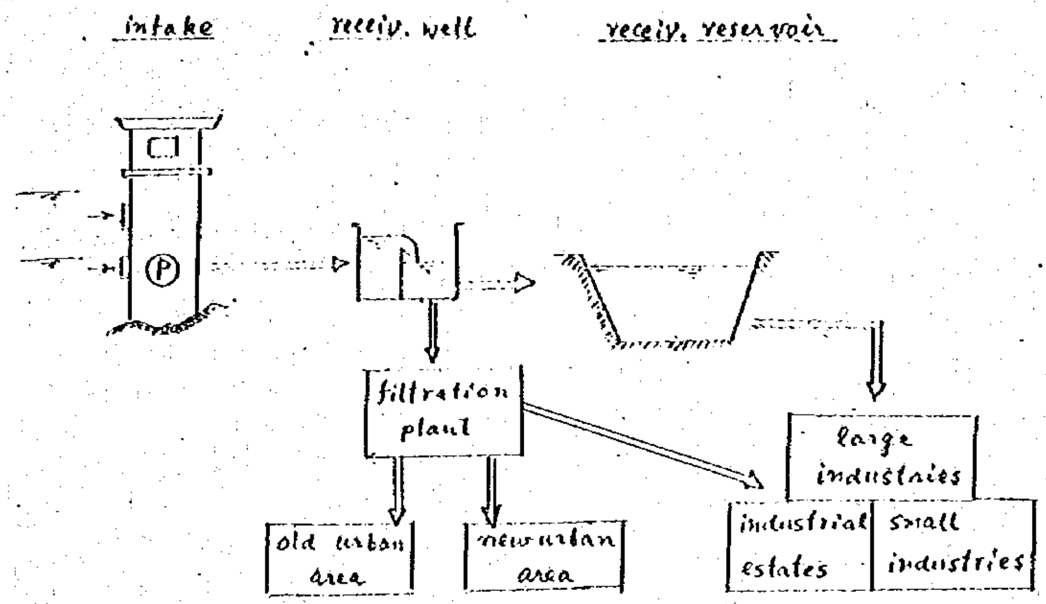
(2) Receiving Reservoir (Raw Water Reservoir)

Adjacent to the planned receiving well, a receiving reservoir shall be constructed, possibly with several hours capacity. The primary purpose is to prepare for the supply shortage from upstream due to accidents like malfunction of pipeline and power failure. The receiving reservoir can be so designed as to make settling of suspended particles. In case of extraordinary high turbidity use of coagulants will accelerate settling and improve the water quality remarkably.

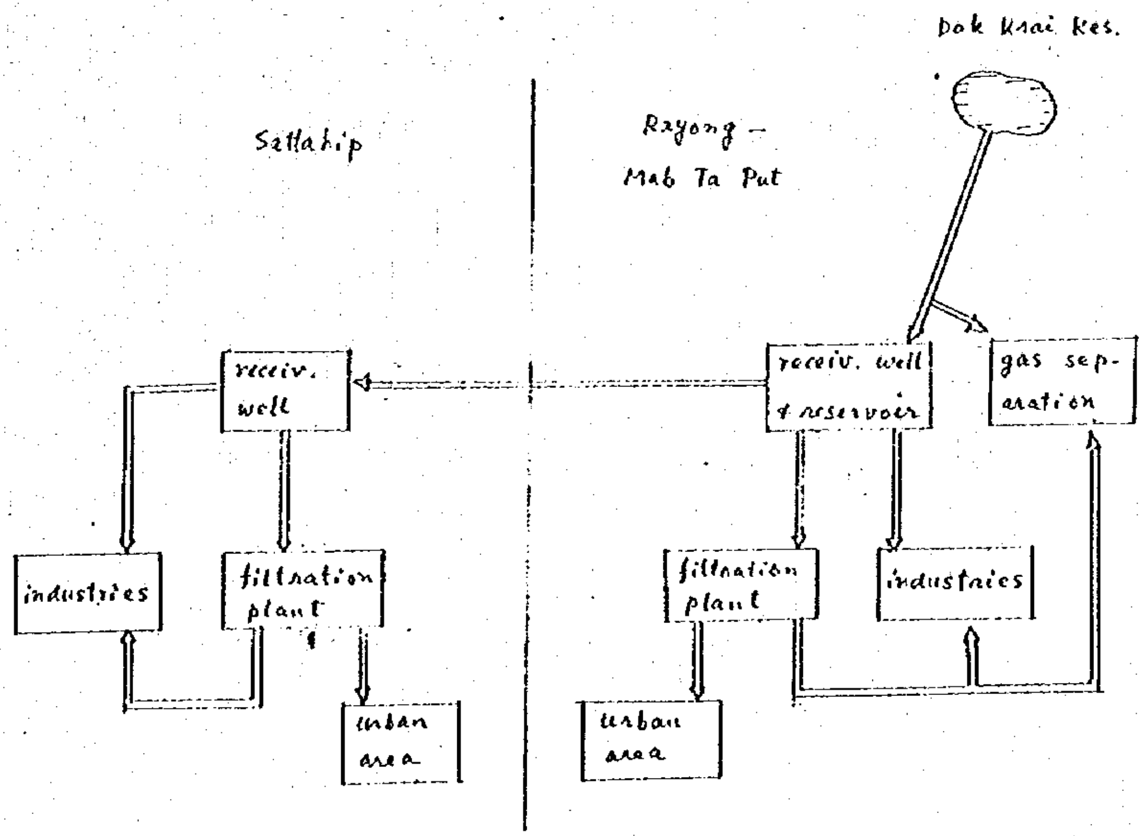
2.3. Water Usage in Sattahip-Mab Ta Put - Rayong

Taking into account of the discussion, the usage system will be shown in the next page:

(1) Qualitative



(2) Area



2.5. Size

The Feasibility Study Report, in 4.1.2, estimated the municipal water demand in 1990 and 2000 as 19.4 and 38.2 (32.8 on page 4-3 seems to be wrong, probably mistypewriting or miscalculation)MCM/yr. The Detailed Design team feels it an underestimate as the population growth rate in the study is rather low. (It will be discussed in other paper with factors like urban population ratio, pervasion ratio and per capita consumption)
The Detailed Design team likes to propose 25 MCM/yr. filtration plant (about 1/10 of Sam Sen Filtration Plant, Bangkok), considering the size to be expanded 2 times in 2000.
The above discussion is based on an assumption that a collective treatment plant is preferable than several plants along the pipeline. The comparison, even if it be done with many assumed factors, seems to be fruitless.

2.6. Location

From the viewpoints of operation and maintenance, the filtration plant is to be run together with the planned receiving well and receiving reservoir, under an order system within an organization. It eventually leads that the plant is to be located adjacent to the receiving well and the receiving reservoir, if the land condition makes it possible.
The location of the receiving structures are now under study. An approximate plan of the filtration plant will be necessary for selecting and deciding the location of the receiving well and the receiving reservoir.

2.4. Treatment Process

(1) Raw Water Quality

The analysis of Dok Krai water is shown on Table 9-19, -20, -21 of the Feasibility Study Report.
Table 9-19 is about the toxic substances. Cadmium, Cyanide and chromium are not detected. Phosphorus (in form of organic phosphate) and Arsenic is sufficiently low.
Table 9-20 is about the minerals. Total Dissolved Solids and Total solids are 68 ppm, 93 ppm respectively. PH is 7.1.
Table 9-21 is mostly about the substances specified for drinking purpose. Some items such as Color, Cu, Zn, Pb, Ba quoted in World Health Organization's standards are not tested.

(2) Industrial Use

The Feasibility Study Report, in 9.3.1, states "The data acquired in the survey shows that except the TS valve for raw material water, the use in industry would pose no problem in matter of PH, TS, hardness and Fe + Mn."
As the data is of the raw water, the statement means it is good for use of industries without treatment.
The detailed design team will agree and follow the line, with the previously discussed consideration.

(3) Potable Use

In Bangkok area, the existing water treatment plants have already adopted the chemical coagulation-sedimentation-rapid filtration-chlorin-ation system and they have been well operated.
The planned one will follow the process to stabilize quality and quantity, to economize man power, to meet the overload in future.
The raw water quality is fit for the process.
The system requires electricity, periodical supply of chemicals like soda ash (or slaked lime), aluminum sulphate and liquid chlorine (or ozone generator with electricity). The system will discharge liquid sludge, the residue of treatment.

3. Filtration Plant

3.1. Components

An ordinary filtration plant is composed of the following parts. Each parts' description is also given.

No.	Name	Function	Requirement
1	receiving and division well	receive and divide raw water for 2-3 identical treatment lines	10-15 min. retention, rectangular concrete tank of division wall within
2	coagulation basin	mix raw water with coagulants for re-action of absorption and floc formation	about 30 min. retention, rapid and gentle mix-ing machine, rectang-ular concrete tank
2-1	chemicals feed-ing system	feed coagulants with controlled rate, chemicals storage	chemical pumps and tanks with reasonable size
3	sedimentation basin	settling of coagulant formed floc	2-3 hrs. retension, rectangular concrete tank, sludge valves
4	filtration basin	filter unsettled floc	filtration velocity 120-180 m/d, periodi-cal washing, battery of rectangular concrete tanks, flow control system
4-1	backwash system	periodical cleans-ing of clogged sand bed of filter	direct washing by pump, or elevated tank with pump
5	chlorination or ozonization system	disinfection (steri-lization) of filtered water	few min. contact time, small concrete pit, chlorinator with steel container, ozone generator with auxiliary equipment
6	clear water reservoir	storage of potable water for in-plant uses, sometime for distribution stor-age	1 hr. retention time, rectangular concrete tank with cover, several hrs. retention time for distribution storage

No.	Name	Function	Requirement
7	instrumentation system	control and integration of components	electric, electronic preumatic system panel and cables
8	power supply system	supply power to components and instrumentation system	
9	administrative building		
10	building for 2-1 and 5		

3.2. Dimensions of Major Components in 25 MCM/yr Plant

25 MCM/yr. is 68,500 cu.m/day or 2.854 cu.m/hr.

(1) Receiving and Division Well

Capacity : 10 min. of 2.854 cu.m/hr. = 475 cu.m

Dimensions:12.0 mW (width) x 13.2 mL (length) x 3.0 mD (depth)

The treatment system is devided into two identical, symmetrical lines, hereafter.

(2) Coagulation Basin

Capacity : 30 min. retention or 1.427 cu.m.

Dimensions:12.0 mW x 17.0 mL x 3.5 mD x 2 parts

(3) Sedimentation Basin

Capacity : 3 hrs. retention or 8.562 cu.m:

Dimensions:72 mW x 17 mL x 3.5 mD x 2 parts

(4) Filtration Basin

Filtration rate : 140 cu.m/sqm/day or 2.33gpm/sq.ft

No. of filters : 6 x 81 sq.m filtration area

Dimensions : each filter, 11 mW x 9 mL x 3.2 mH

Gallery : 7 mW x 35 mL

Back wash system: elevated tank capacity 400 cu.m.

pump capacity 3.3 cu.m/min x 25 m head

(5) Chlorination System

Consumption : 14 kg/hr at 5 ppm rate
8.4 " 3 "
Feeder : 20 kg/hr
Container : 30 at least (50 kg)

(6) Clear Water Reservoir

Capacity : 1 hr. of 2.854 cu.m
Dimensions : 20 mW x 20 mL x 3.6 mD (effective) x 2 parts

(7) Instrumentation System

Center Panel Meters : turbidity 2, PH 2, flow 2, level 2.
Site Panel Meters : chlorine feed rate, container weight scale, chemical tank level, residual chlorine

(8) Power Supply System

(9) Administrative Building : Office, control room, laboratory, rest rooom

(10) Chemical House : Soda ash and aluminum sulphate store room dissolving and storing tanks two each for soda ash and al. suephate
Chlorinatin House : Containers store room, feeder room

3.3. Layout of Filtration Pland

Shown on the attached sheet.
The lot is 160 m x 180 m and includes the area for the future extension facilities which are shown with dot lines.
It is for a 50 MCM/yr. filtration plant.
If a distribution reservoir is needed within the area, the space between the Chlorine House and the Administrative Building can be used for 2 hrs. storago at 25 MCM/yr.

3.4. Layout of Receiving Well, Receiving Reservoir and Filtration Plant

Shown on the attached sheet.
The receiving well received water and divides it into three compartments, for the future extension to Sattahip, for the filtration pland and for the reservoir.
By the Feasibility Study Report, the flow rates for Sattahip, the possible filtration plant and for the industries in Mab Ta Put - Rayong area are 17.6, 12.9 and 23.1 MCM/yr. respectively and making the total 53.6 MCM/yr.
How to regulate each flow rate and to divide for the three usages will be studied sometime else. It is possible by using a variable - depth weir.
The receiving reservoirs capacity is assumed tentatively as 3 hrs. retention times of the design flow rate, that is, 28,300 cu.m.
It will better be constructed not by concrete but by earth dam with plastic film (membrane) lining.
Assuming the water depth as 3 m, the area of reservoir becomes 9,430 sq.m. For emergency treatment in case of extraordinary turbidy, the flow will zigzag in the reservoir by the walls as shown on the picture.
The receiving well, with 20 min. retention capacity, is 3,144 cu.m and the dimensions are 30 mW x 30 mL x 3.5 mD.
The lot of the receiving reservoir is 155 m square.
The whole area of the receiving well, receiving reservoir, filtration plant is 260 m x 410 m. However, it can be decreased by rearrangement of the facilities.

4. Head Losses After Receiving Well

4.1. In Receiving Well

A receiving well consists of three parts, namely the receiving part, the before-weir part and the after-weir part.
The water in the receiving part is turbulant and it is calmed down by flowing through a perforated wall (with many holes arranged latticewise) to the before-weir part.
The before-weir part is sometimes called the calm flow part.
The after-weir part, often called the division part, is divided into several tanks by the walls.
In this project, the three divided tanks are for Sattahip,the filtration plant and the receiving reservoir.
The weir prevents the upstream flow condition from Dok Krai to the receiving well from interference coming from the downstream flow condition and keeps the receiving part's water level constant.
The dividing walls make it possible to prevent interference to each other, causedby the three downstream flow conditions.
The weir can be used to measure the flow rate also.
The weir loss, consisting of overflow depth and clearance is reasonably assumed about 0.6 m and the loss through the perforated wall is less than 0.1 m.

4.2. From Receiving Well to Next Tank

From the three division tanks the water flows to the next tanks, the receiving tanks of Sattahip and the filtration plant and the receiving reservoir.
Because the design flow rate of Sattahip pipeline is not decided, the loss of it is not discussed here.
For the other two, the loss is rather small due to the short distance.
With reasonable design it can be less than 0.2 m.

4.3. In Receiving Reservoir

The flow in the receiving reservoir is of an open channel flow of 25 mW x 3 mD. The loss is negligible.

4.4. In Filtration Plant

Here the turbulance in the receiving well is slight and the loss in the well and interference, if it occurs, is not serious. The loss in the well and to the coagulation basin is 0.5 m.
In the coagulation basin, the loss is negligible when mechanical mixers are used. When mixing is done, however, by turbulance causing walls, the loss will amount about 0.2 m.
In the sedimentation basin, about 0.2 m at the outlet.
In the filtration basin, a heavy loss due to the san beds' clogging is unavoidable. It is 2.5 m at least.
From the filtration basin to the clear water tank 0.5 m loss must be considered.

4.5. Summery

The above description is summerized and shown on the attach paper.

5. Conclusion

The detailed design teams' conclusion are the following ones:

(1) Location

If a filtration plant of collective treatment for neighbouring urban area is to be constructed, the location shall be adjacent to the receiving well and receiving reservoir.
It will be convenient for operation and maintenance and will save the head for distribution.

(2) Water Treatment Process

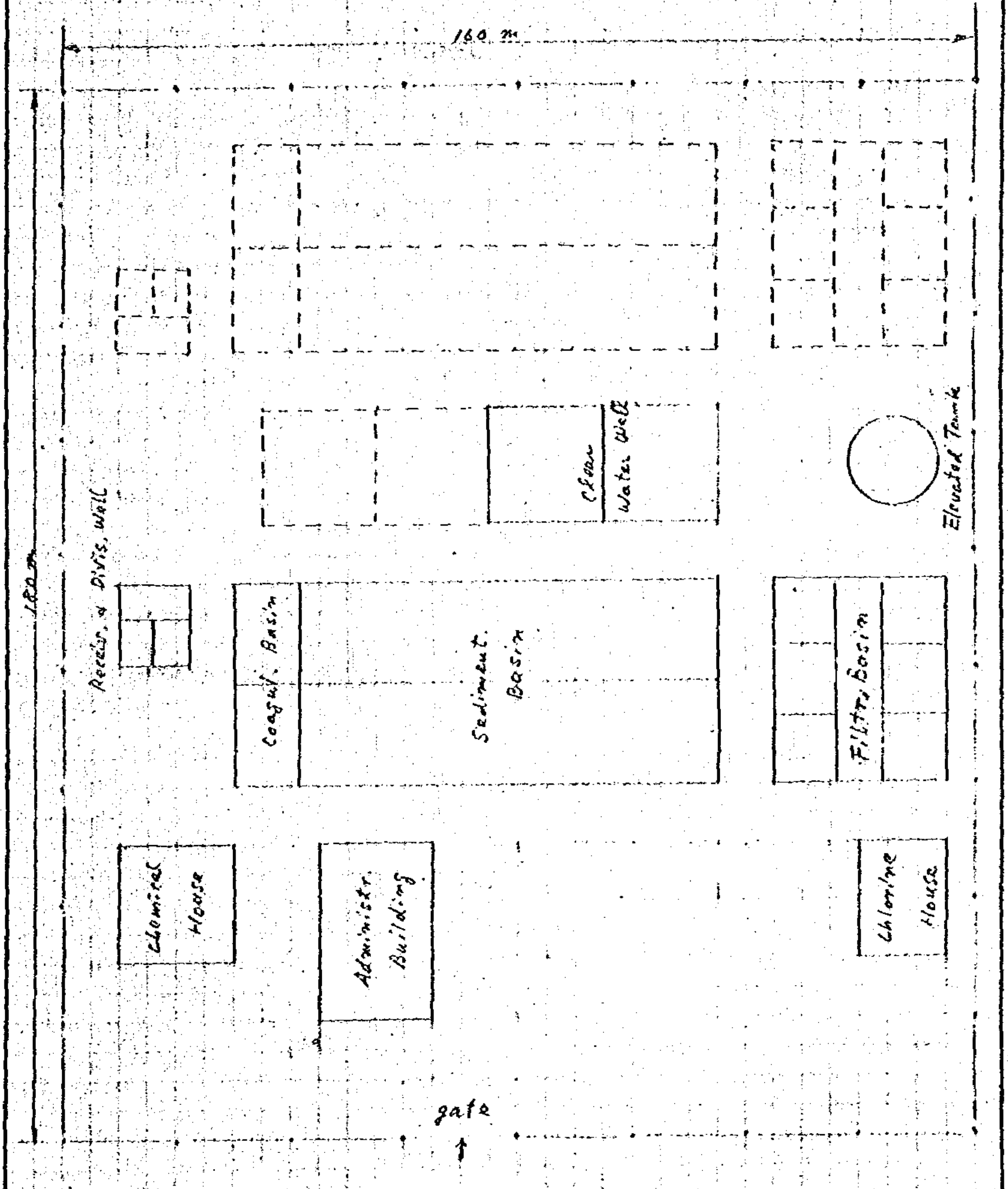
Treatment for industries is not practical and unnecessary. However, consideration for extraordinary turbidity is taken easily and economically.
Treatment for drinking water is indispensable. The process is widely used coagulation - sedimentation - filtration - chlorination and it is suitable for Dok Krai water.

(3) Size of Filtration Plant

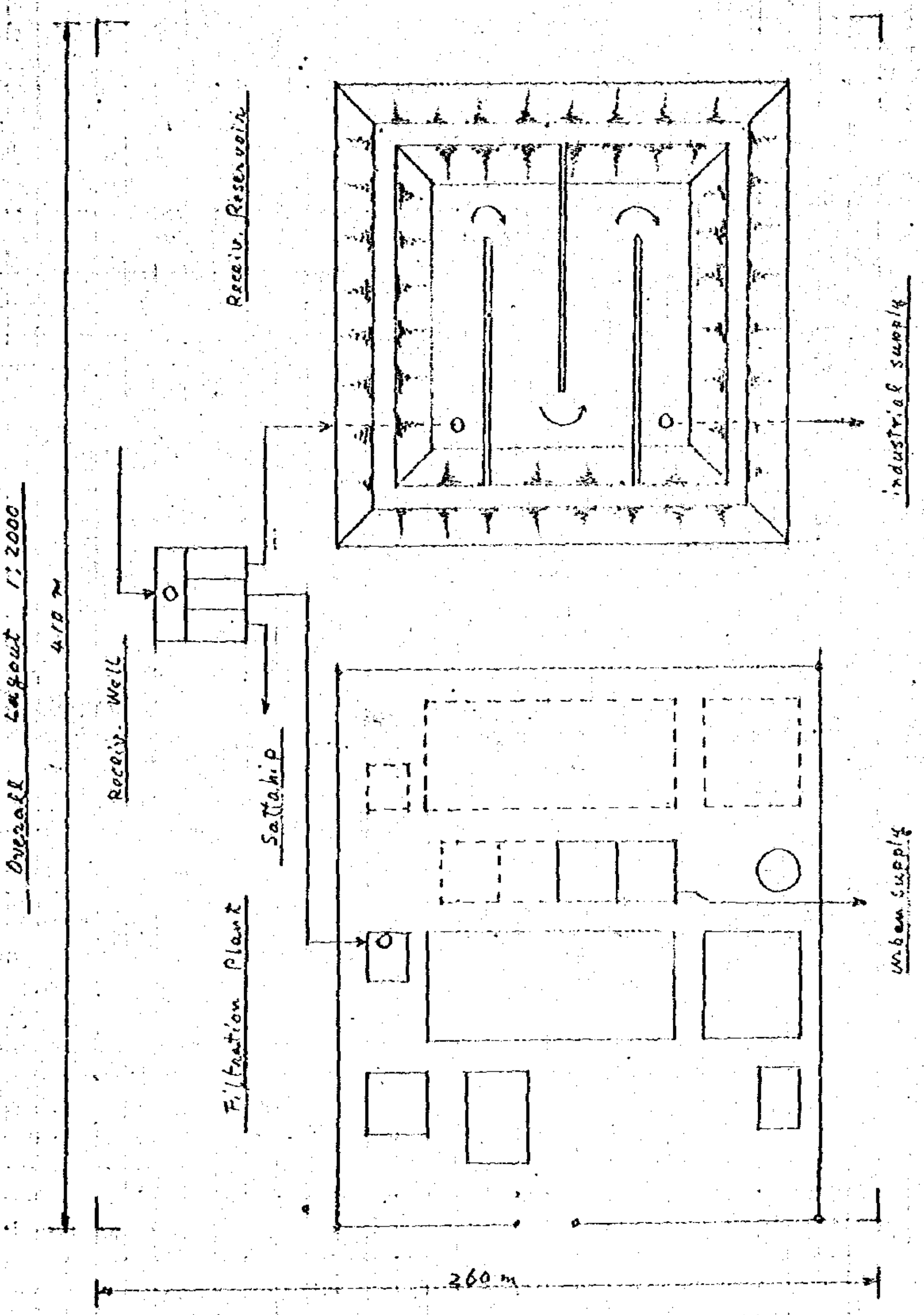
For 1990 25 MCM/yr and for 2000 additional 25 MCM/yr will be reasonable.
Layouts of Filtration Plant and Receiving Well-Receiving Reservoir-Filtration Plant are attached.
Also attached is Head Loss in the mentioned system.

Attached sheet 1 No.

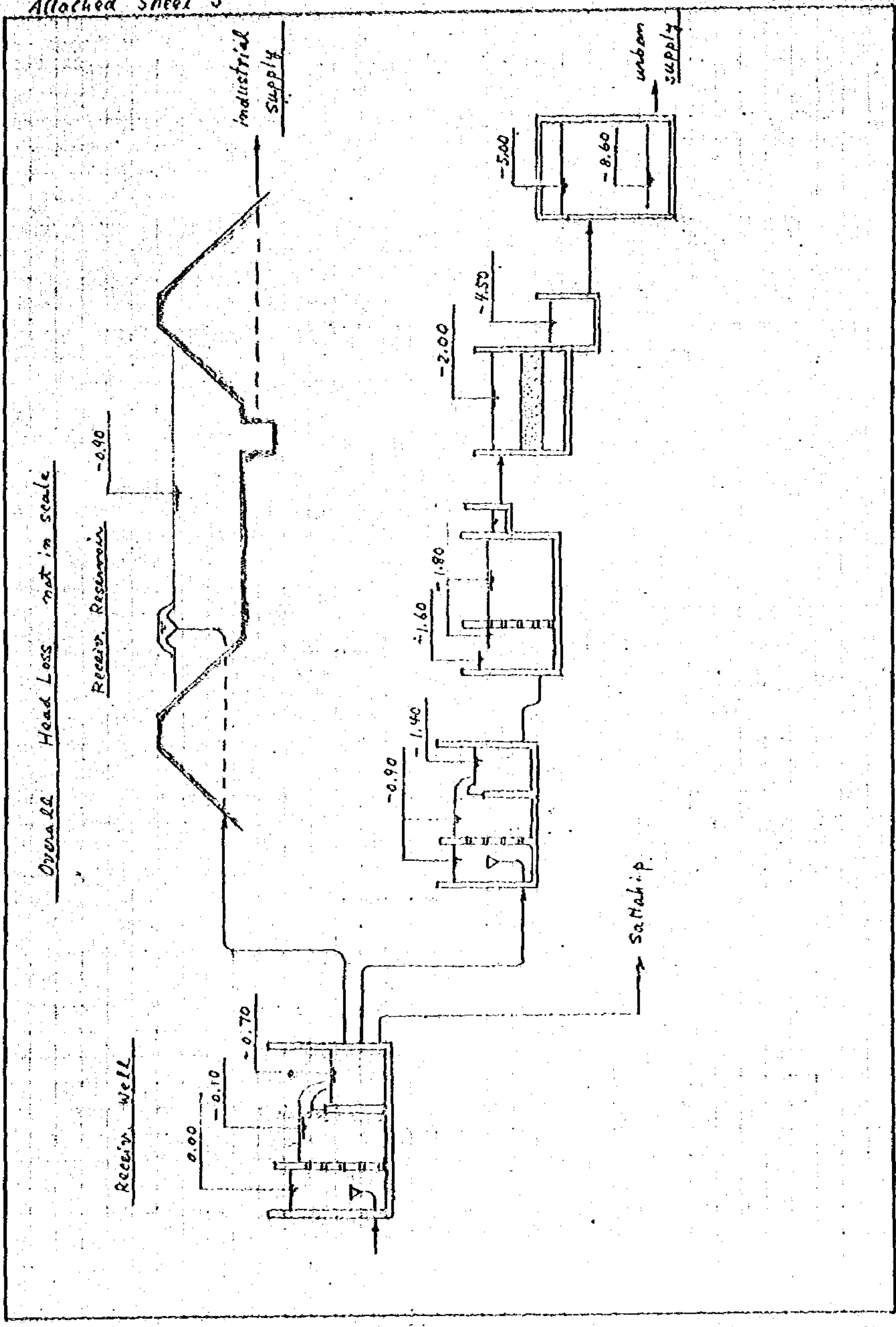
Layout of Filtration Plant 1:1000



Attached Sheet 2 No.



Attached Sheet 3 No.



NO.4 PROPOSAL ON ORGANIZATION FOR PIPELINE OPERATION AND MAINTENANCE

January, 1982

Prepared by OSAMU WAKAMOTO
Co-Leader, Civil Engineering
Detailed Design Team, JICA

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6. Organization Chart with Number of Personnel

1. Introduction

How to manage the pipeline system after completion in 1984 raises problems which are of legal, administrative, institutional, technical nature.
They are now mostly being prepared for by Thai Government.
The Detailed Design team will propose about some issues here from the view points of operation and maintenance.

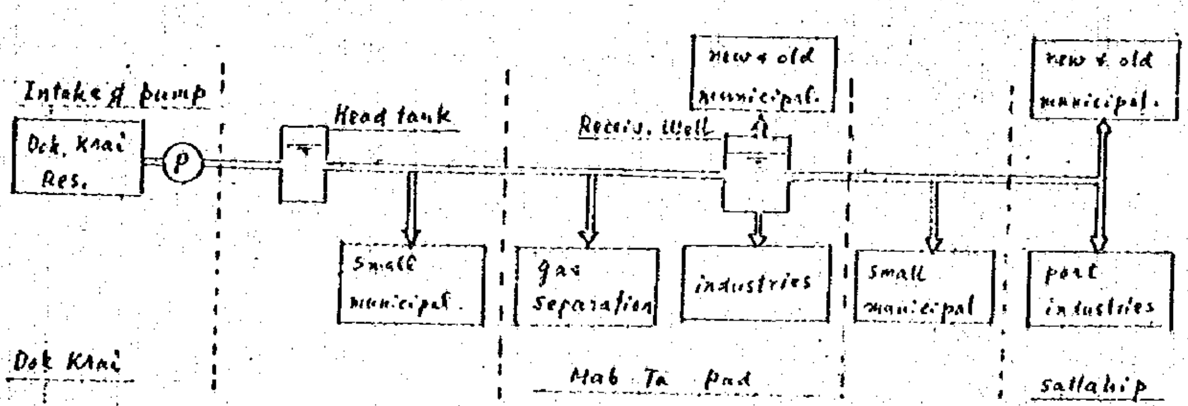
The proposal is only about the matters concerning to the operation and maintenance.
Other matters shall be conseived by Thai side and together with this proposal, a larger organizational concept shall be formed.

2. Displays of System

2.1. Flow Diagram

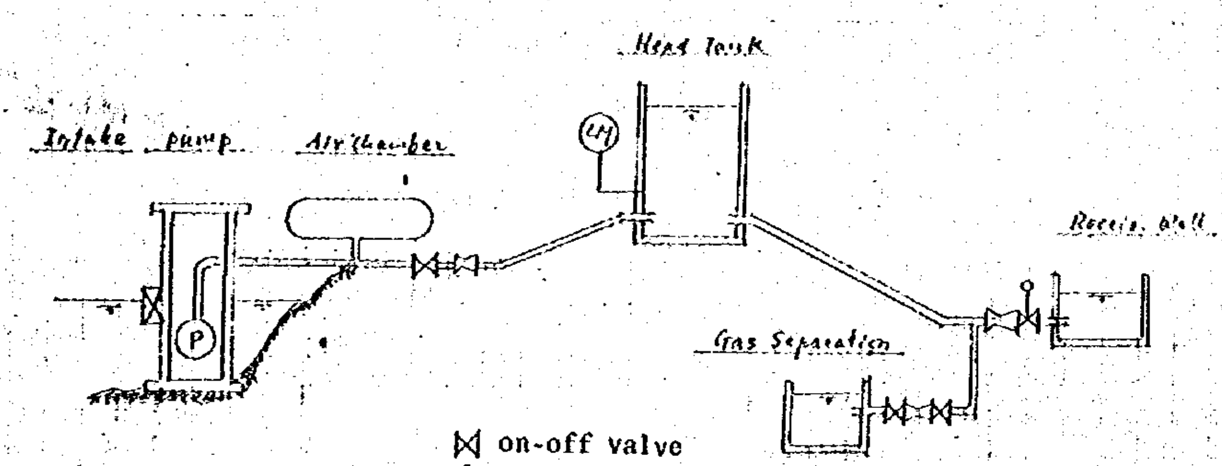
The flow diagram concerning the supply, demand and the locations of demand is as follows:

Flow Diagram



2.2. Facilities in System

The major facilities to be included in the system are shown as follows:



on-off valve
control valve
flow meter
level meter
intake gate

3. Operational Principle

3.1. Background

The Detailed Design Team had a meeting with the engineers of Metropolitan Water Works Authority (MWWA) to collect informations about operational practice in Thailand. The findings are as follows:

* A similiar pipeline is a 0.90 m dia. ductile cast-iron pipeline of 10 km length for transmission from Chaophya river to Thonburi filtration plant.
* The pump MWWA is using are mostly of axial flow or mixed flow type. The largest ones' capacity is 400 m^3/sec.
* Control of the flow rate is by changing the number of pumps under operation.
* Power failure will occur 1 to 2 times per year.
* Orders from the headquater to the stations are sent by public telephone and/or wireless. MWWA has its own radio stations authorized officially.
* Operations are carried out throughout 24 hrs. In each key station, 4 teams are engaged alternatively on 8 hrs' shift. A team is consisted of one engineer, one to two technicians, two skilled laborers.

3.2. Conditions to be considered

3.2.1 Location of Operation Center

To control the flow rate and the level of pipeline system under planning, various methods have been conceived and evaluated comparatively. The conclusion is that the center of operation is to be at the receiving well. The discussion will be reported in other Report. Aside from the said technical reason, other following factors are considered in the conclusion:

* Adjacent to the Receiving Well, a large Receiving Reservoir will be constructed very possibly for storage of water. Furthermore, in case a collective filtration plant is found necessary in the area, it will be better set up near the Well and Reservoir than in other locations. These facilities can be easily operated in coordination because of closeness.

* From the Receiving Well, the water shall be transmitted to Sattahip area. When the line comes under operation, the Receiving Well will be located around the middle of Dok Krai and Sattahip in a strategie point.

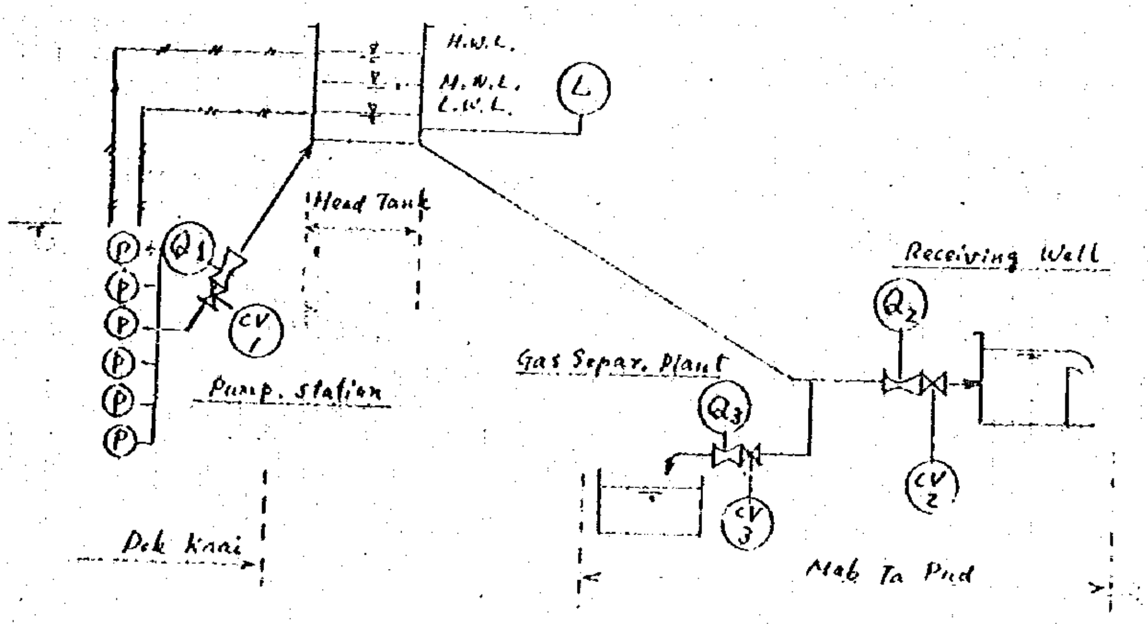
* In principle, a water supply system is an institution to serve the customer. It must be sensitive and responsive to the customers' changing requirements, alert and waiting. The largest and the second largest customer is Mab Ta Pud and Sattahip area.

3.2.2 Conditions of Operation, Normal and Extraordinary

In order to understand the operation, we will see what conditions will occur during normal operation and what will be the extraordinary conditions we must prepare for.

(1) Normal Operation

Normal Operation



The above picture contains following apparatus:

* Flow Meter	(Q1) (Q2) (Q3)	at pumps' delivery, Receiving Well, Gas Separation Plant
* Level Meter	(L)	at head tank
Control Valve	(CV 1) (CV 2) (CV 3)	at pumps' delivery, Receiving Well, Gas Separation Plant
* Pumps		5 No.s at the maximum with one Stand-by
* Electrode		High and Low water level of Head Tank

The functions of each component will be :

* Flow Meter	(Q2)	for the Engineer's judgement to control the flow
	(Q1)	for confirmation of pump's delivery
	(Q3)	for confirmation of Gas Separation Plant's inflow
* Level Meter	(L)	for the Engineer's judgement to control the flow.
* Control Valve	(CV 2)	for controlling the flow and the head tank level
	(CV 1)	usually open, closed only when necessary
	(CV 3)	usually open, closed in case of the power failure by the Engineer's order.
* Pumps		Seasonally and yearly the number is set. One pump is operated in relation with the electrolde in the head tank.
* Electrode		The high and the low actuate on-off of one pump.

Under the normal operation, the Engineer set a flow rate after controlling (CV 2) and the operator at the Receiving Well is ordered to keep watch on (Q2) whether the flow rate is kept. The operator is also to keep contact with the pump station and the Gas Plant to be informed about periodic (Q1) (Q3) reading.

As the flow rate is controlled at the Receiving Well with (CV 2), the pumps' number under operation is controlled by the head tank's level. For instance when the set flow rate is between two and three pumps'capacity, two pumps run

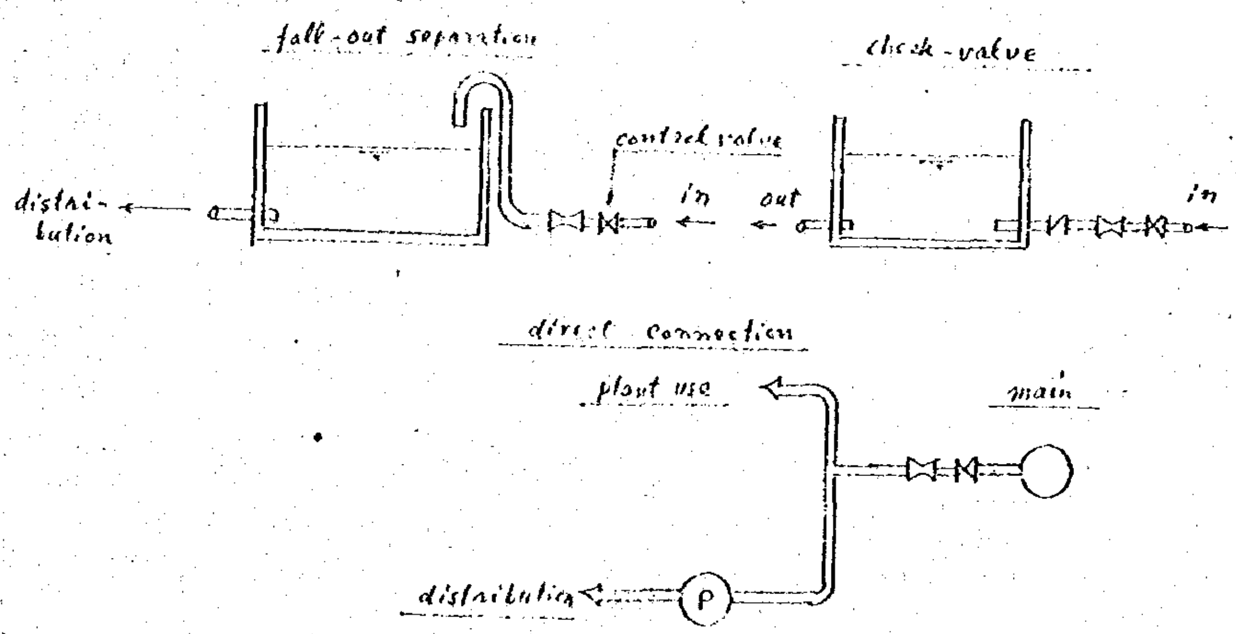
continuously and the number three pump, controlled by the head tank's level, starts and stops periodically according to the set flow rate.
In this way the pump operation is automatic.

When the set value of the flow rate is changed by adjustment of the control valve (CV 2), the period of on-off of the number three pump changes corresponding to the new flow rate.

(2) Requirements on Gas Separation Plant

The Gas Plant is requested of the following conditions;

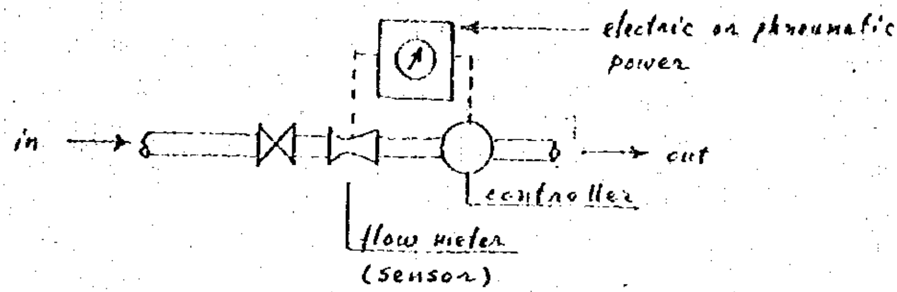
* Prepare its own storage tank in the plant so that the plant can manage with the storage, in case of emergency.
* As the connection is branched before the Receiving Well, in order to avoid any effect on the main's flow condition, the "fall-out" separation is most preferable. Installing a check value may be tolerable. The direct connection to the plant's distribution, especially with a booster pump in it must be prohibited.



* The control valve, even if it belongs to the plant ownership and it is installed within the property, must be operated only under the approval of the Engineer or by the operator of the supplier.

* To keep a steady flow, an automatic control system consisting of the flow meter and the control equipment is preferably to be installed by the Plant.

Automatic Control

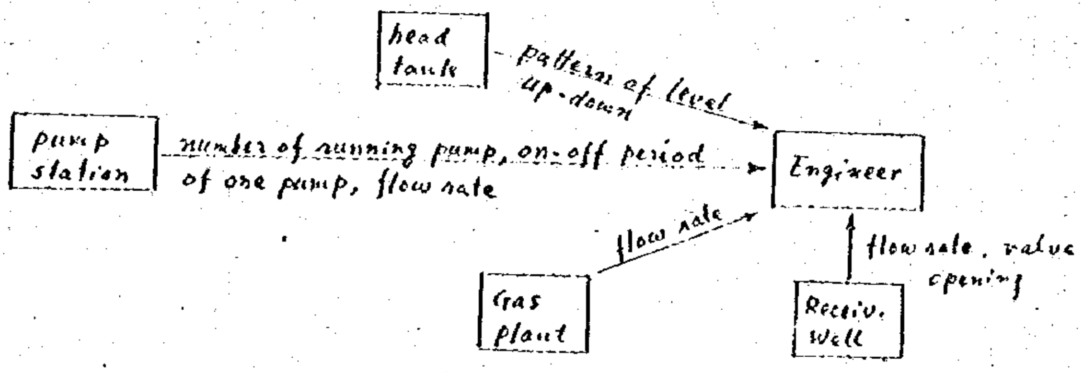


(3) Flow of Information under Normal Operation

The flow of information necessary to operate the system is shown below.

is

everyday operation



When the flow rate is changed

The Engineer orders the Receiving Well to control the valve and wait to see if the desired rate is attained.

(4) Extraordinary Operation, Power Failure

In case of the power failure, all the running pumps will stop. The level of the head tank starts to fall down until the valves at the receiving well and the gas plant are closed immediately.

Precaution by Detailed Design

The detailed design team takes precaution against emptying of the head tank. It assumes that the time required for closing the valves is 20 minutes and the tank is to have 20 minutes minimum storage for it.

Flow of Information and Means taken

The information shall be conveyed and the means be taken as follows:

* The pump station report the power failure to the Engineer and then checks about pump, air chamber, electrical equipments.
* The Engineer alert all stations and put the system in "alert station".
* The Engineer orders the receiving wall and the Gas plant to close the valve and request reporting back to him after the operation.
* The Engineer orders the head tank to report to him the change of level, continuously untill it stops.
* When the situation of pump station, head tank, receiving well is confirmed concerning the complete stop of the pipeline, the Engineer reports to his superior and makes inquiry about the possibility of power's resumption. Then if necessary, he informs the customer about the situation.
* The Engineer orders the operator in reserve to make inspection of the pipeline, especially about the air valves on it.

When the power supply is resumed, the following steps are needed until the pipeline comes under the normal operation.

* The pump station reports about the power supply's resumption to the Engineer.

* The Engineer informs all stations and checks about the water level of head tank.

* The Engineer orders the pump station to start the pump one by one, checking with the head tank's level and with the valves opening of the receiving well and of the gas plant.

* The Engineer orders the air valves' situation to be reported by the operators in reserve. Where air-sucking was apparent in the inspection after the power failure, they shall be waiting to receive the order, preferably with a walkie-talkie and stay till the air is exhausted.

* At each stage of the increase of running pumps number, the steady flow condition as indicated by stable flow rate and the head tank level, must be attained before proceeding to the next stage.

* The operators at the valves of Receiving well and the Gas plant must report the flow rate and the valve opening simaltaneously.

* When the flow condition, same as before the power failure, is attained, the Engineer will order the operators in the field back to office and release all stations from "alert station" to "normal station".

3.3 . Other Equipments

With the above discussion of operational conditions, the equipments will be necessitated as follows:

(1) Communication Means

The discussion includes automatic start and stop of the pumps corresponding the limit of head tanks' level. It necessitates a signal line between the pump station and the head tank. Although only the electrodes are mentioned there, the level sensor for continuous measurement can also be used.

Besides the signal line, a telephone system is necessary for everyday's operation. However, the problem with the public telephone system is the limitation of number of the circuit. In case an emergency occurs when they are fully engaged, nothing can be done.
Both for everyday's operation and for emergency, the radio communication seems desirable. Approval of a radio system by the relevant authority including allotment of the frequency, preparing for qualified radio operators, purchasing the equipment and setting up the radio stations at the pump station, head tank and the receiving well must be arranged as early as possible. Walkie-talkie will be very handy for inspection patrol of the pipeline to communicate. In case its range is short to reach the Engineer, the message can be relayed through a nearby station within the range.

(2) Mobility

For inspection patrol, a jeap and/or a land cruiser is convenient. 3-4 cars will be enough.

4. Maintenance Principle

4.1. Conditions to be considered

Generally speaking, transmission pipelines do not have much maintenance problems, when compared with distribuiton pipelines and service connections.
Usually along the pipeline, the pipe stockyards for spare pipes, joints, small valves shall be prepared, in order to meet accidents like leakage and burst. Spare pipes will be almost unnecessary as the pipeline is made of steel and other materials can be easily brought to any place as the pipeline runs along the good highway.
Warehouse, Yard, Workshop
A warehouse with yard, located at Dok Krai, shall be used to store the small idems like air valves, check valves, joints and the spare parts like rubber rings, packings, bolts and the large items indoors and outdoors.

Also to be stored are the spare small electrical and mechanical gadgets for upkeeping the pumping station.
A workshop with tools and simple machineries shall be prepared for minor works of repairing.

4.2. Check List and Record

Major items like pumps, motors, compressors for the air chamber, flow and level meters (including sensors) and electric circuits shall be inspected regularly. Also needed will be overhauling of some equipments.
The checking schedule must be fixed and observed and the checking list for various items be prepared.
The record of inspection, overhauling and reparing must be preserved.

4.3. Water Analysis

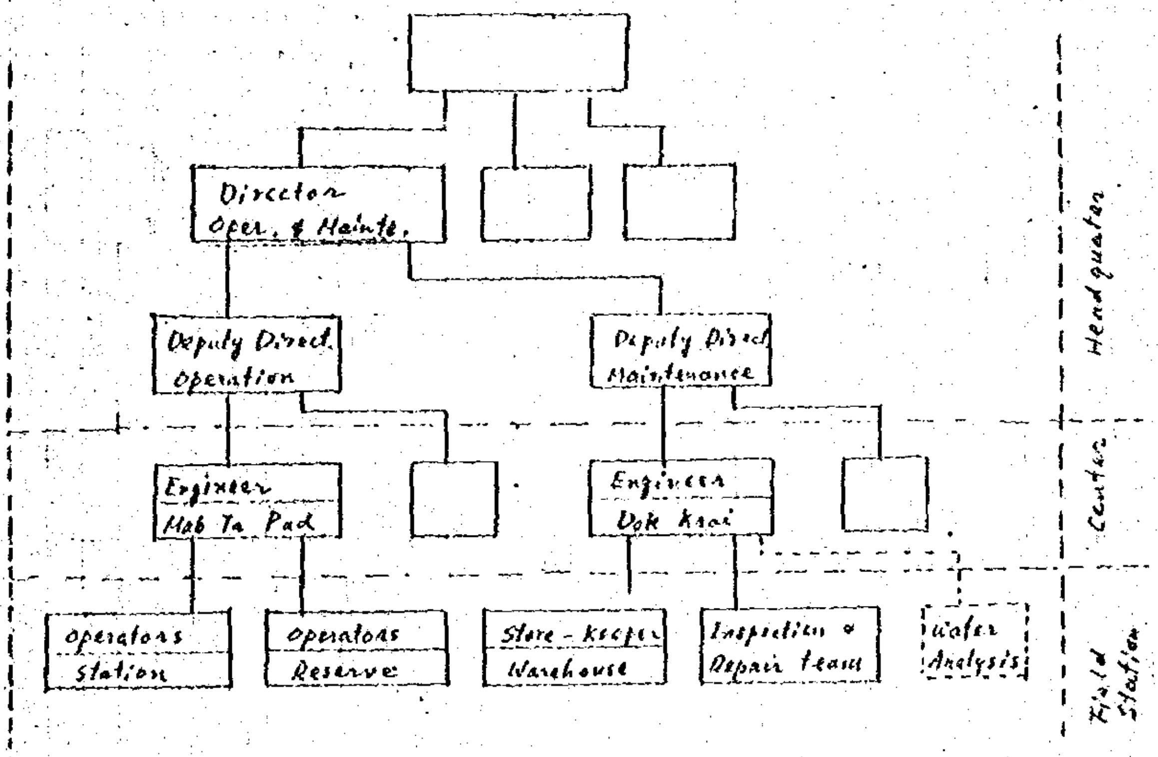
The pipeline system is for delivering water to the customers. The customers are entitled to learn about the quality of delivered goods, the water. Consequently,making the water analysis and reporting about the result is the suppliers obligation, if not responsibility.
The water of Dok Krai Reservoir is satisfactory for general use of industries as far as the available information shows. One thing which may occur but has not been experienced is high turbidity.
The frequency of water analysis is to be considered here. About inorganic matters, remarkable difference will be seen between the dry season and the rainy season. About the inorganic matters, seasonal change will be slight while it is sometimes noticable due to the algae growth in countries of the temperate zone. Probably four times per year including the dry and rainy seasons will be sufficient in making the analysis. A laboratory will be unnecessary for some years until the water is put to use for drinking purpose. Then the frequency must be increased, the items of analysis must be decided and the laboratory shall be set up.

Till then, the water analysis will be better taken care by some public institution which has facilities of water analysis.

5. Organization for Operation and Maintenance

An organization chart including some requirements is to be discussed,then a table of personnel with working condition is to be proposed.

5.1. Organization Chart, Qualification of Key Personnel



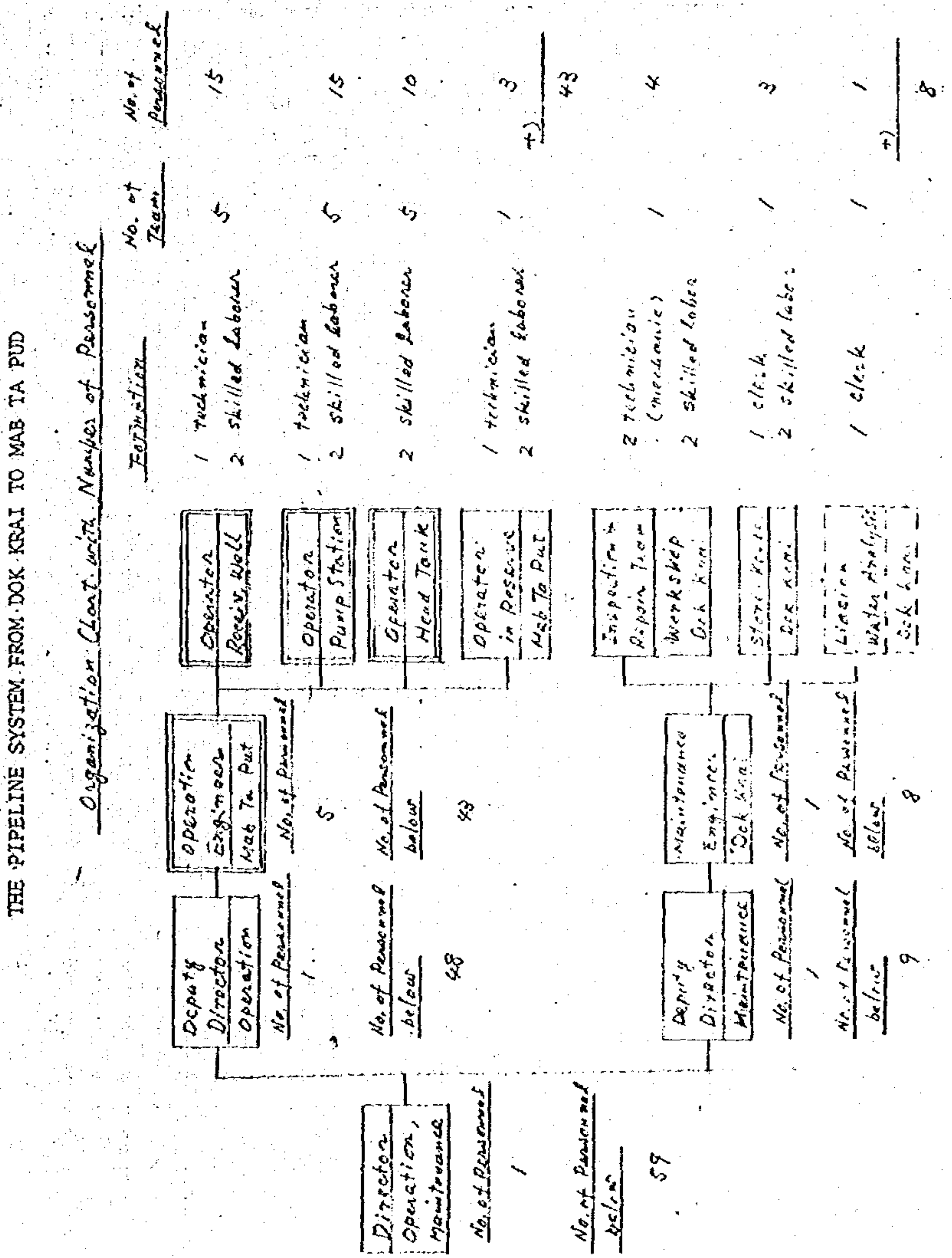
The above chart shows how the operation and maintenance will be organized and how it will fit in the whole organization. The requirements of personnel will be desribed below:

* The director and deputy directors are to have sufficient educational and vocational background and some administrative experience.

* The Engineer in charge of the operation and the maintenance are to have educational background of civil and/or mechanical engineering.

6. Organization Chart with Number of Personnel

The organization chart in 5.1. will be completed with the number of personnel for each part as shown below;



* The Chief Operator heading two to three operators and the chief of inspection and repair team are to be technician, having years of vocational experience.
* The operators are to be skilled laborers.
* The store-keeper shall be selected mostly on the experience.
* The water analysis is included in the work of Engineer in charge of the maintenance.

5.2. Formation of Operators' team and Inspection & Repair team

(1) Operators Team

Depending upon the importance of each station and considering the hardness of operating conditions, both physical and mental, the following formation is thought out;

Station		
* Receiving Well	1	Chief operator
	2	Operators
* Head Tank	2	Operators
* Pump station	1	Chief operator
	2	Operators
Reserve		
* Engineer's office	1	Chief operator
	2	Operators

(2) Inspection and Repair Team

* Engineer's office	1	Chief mechanic
	1	Mechanic
	2	Skilled laborers

5.3. Working Conditions

(1) Daytime only

The headquarters staff, the store keeper, the operators in reserve and the inspection and repair team are to work only in the daytime, following the organization's rule.

(2) Shift

The Engineers and operators are to work at 8 hrs. shift in rotation, so that 24 hrs operation is being kept.
MWWA seems to take 8 hrs shift by 4 teams' rotation.
By calculation it will mean 42 hrs work per week, more working hours than ordinary which will need some extra pay. 5 teams' rotation and 33.6 hrs per week may be better.
It should be concluded considering labour law, government regulation and labor practice.

Note

* The table does not include common laborer for miscellaneous use.

* The Gas Separation Plant's operation is to be under Operator at the Receiving Well. It is not on the table.

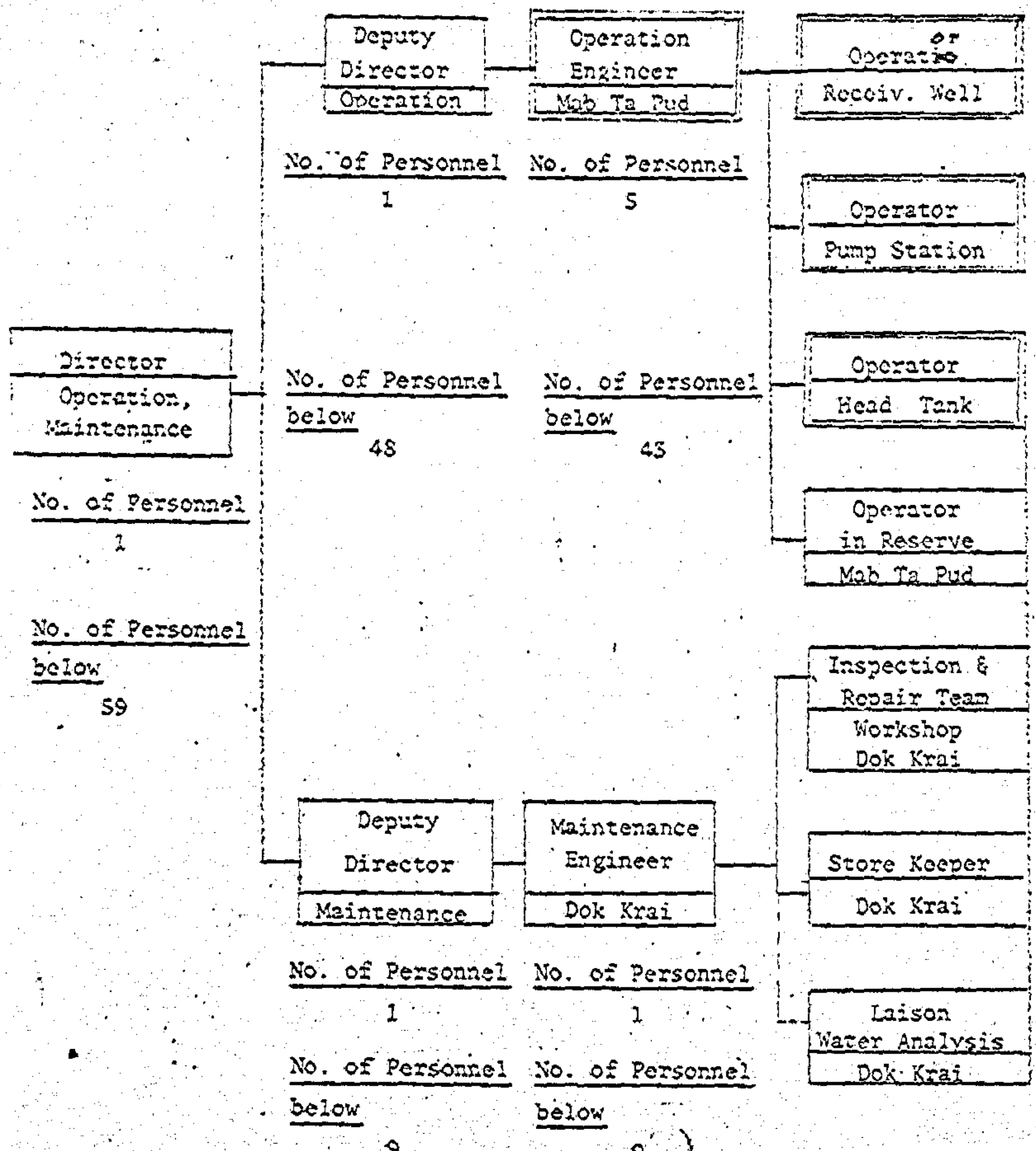
* No. of shift team is assumed as 5.

* The double framed part is for shift work.

THE PIPELINE SYSTEM FROM DOK KRAI TO MAB TA PUD

Organization Chart with Number of Personnel

Fig.-2



Formation	No. of Team	No. of Personnel
1. technician 2. skilled laborer	5	15
1. technician 2. skilled laborer	5	15
2. skilled laborer	5	10
1. technician 2. skilled laborer	1	3
		+) 43
2. technician (mechanic) 2. skilled labor	1	4
1. clerk 2. skilled labor	1	3
1. clerk	1	1
		+) 8

NO.7-1 REPORT ON COMPARATIVE STUDY OF FLOW CONTROL SYSTEM

January 1982

Prepared by YOSHIYUKI TOMIOKA
Water Facilities Engineer
Detailed Design Team
JICA

CONTENT

1. Introduction

2. Control by Pump Unit Number

(1) Setting of Flow Rate by Control Valve
(2) Betteen Pump and Head Tank
(3) Betteen Head Tank and Receiving Well
(4) Flow Pattern

3. Control by Valve Opening

(1) Principle
(2) Setting of Flow Rate by Control Valve
(3) Control by Valve Opening
(4) Head Tank's Role
(5) Flow Pattern

4. Control by Pump Rotation Speed

(1) Principle
(2) Setting of Flow Rate by Control Valve
(3) Control of Rotation Speed
(4) Flow Pattern

5. Discussion before Comparison

(1) Principle
(2) Actual uses of Three Methods
(3) Requirements of Planned Pipeline

6. Comparison

(1) Flow Rate and Response, Simplicity and Reliability
(2) Cost
(3) Head Tank's Capacity

7. Conclusion

1. Introduction

The study aims to compare various methods of controling the pipeline and to decide which one is most suitable for the project.

This study stands on a prerequisite.
In the pipeline system between Dok Krai and Mab Ta Pud, a head tank is found indispensable in order to control the water hammer effect and to protect the pipeline from being emptied.
The head tank's necessity and usefulness are discussed and proved in other reports.

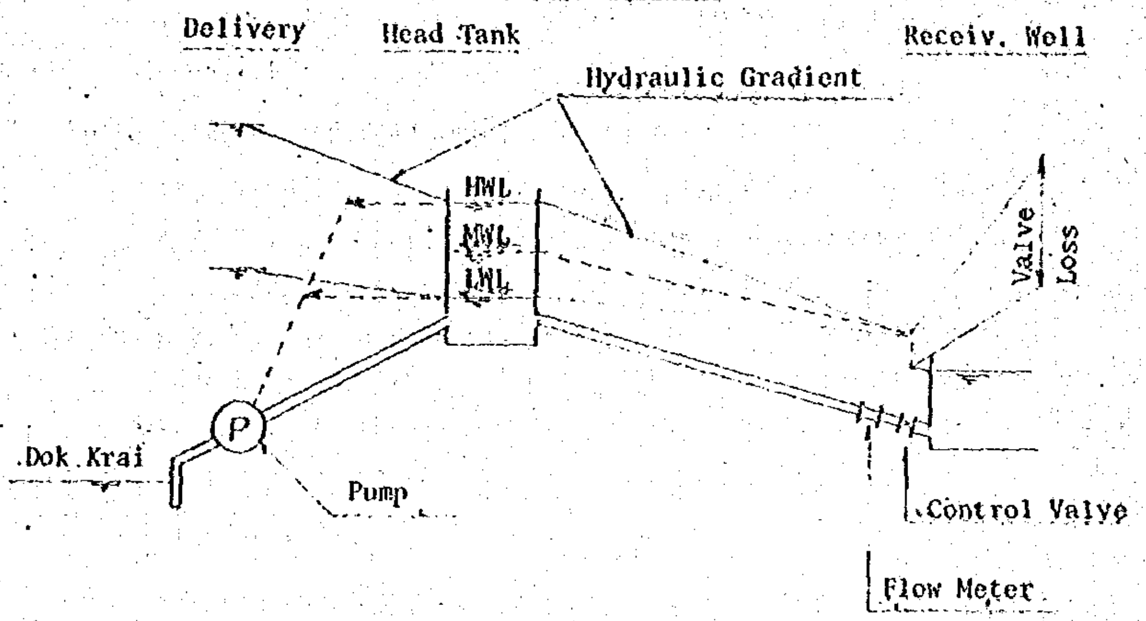
The necessity of flow control is, needless to say, in operating the pipeline system. At the upstream end of pipeline is Dok Krai reservoir of 58 MCM and lifting water does not cause the change of water level practically. At the downstream end of pipeline are Receiving Well and Receiving Reservoir having 21,000 m^3 or 3 hrs' storage for the design rate of flow.
The control of flow in the pipeline which has such sufficient storage capacity at the ends does not need to be very sensitive and delicate. In the actual operation in the foreseeable future, once a flow rate is set up, it will be kept unchanged for several months at least.

Here, three methods will be compared in the study.
They are by changing the unit number of pumps, the valve opening and the rotation speed of pumps.

2. Control by Pump Unit Number

Fig.1 Shows the system and Component.

Fig.1 System and Components



(1) Setting of Flow Rate by Control Valve

The flow rate is decided after considering various conditions. The operator at the receiving well adjusts the opening of control valve, watching the flow meter, until the desired rate flows corresponding MWL (Middle Water Level) in the head tank.
Then, the valve will give a constant loss to the system as shown on Fig.1.

(2) Between Pump and Head Tank

The pumps are connected electrically to the HWL (High Water Level) and LWL (Low Water Level) in the head tank.
Suppose the set flow rate is in between the flow of two pumps and three pumps.
Two pumps are so set that they run unceasingly and the third pump is controled by the two levels. As the set flow rate is short of two pumps flow, when the level comes down to LWL, the third pump starts automatically.

and three pumps flow is excessive for the set flow rate, then the water level gradually comes up until it reaches HWL where the third pumps stops automatically again.
In the head tank, the water level repeats its up and down cycle.

(3) Between Head Tank and Receiving Well

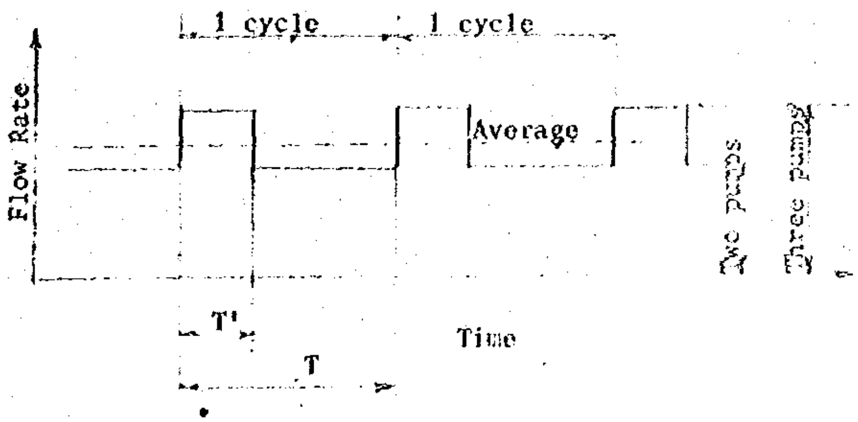
Corresponding the undulation of water level in the tank, the flow rate between the head tank and receiving well also changes periodically.
However, because of the pipeline length, the given head loss by the control valve and the selected height between HWL and LWL, the change of flow rate is small and the flow rate is close to the set one.

(4) Flow Pattern

Between Pump and Head Tank

The third pump runs intermittently following the pattern shown below:

Fig.2 Flow Pattern, Pump to Head Tank



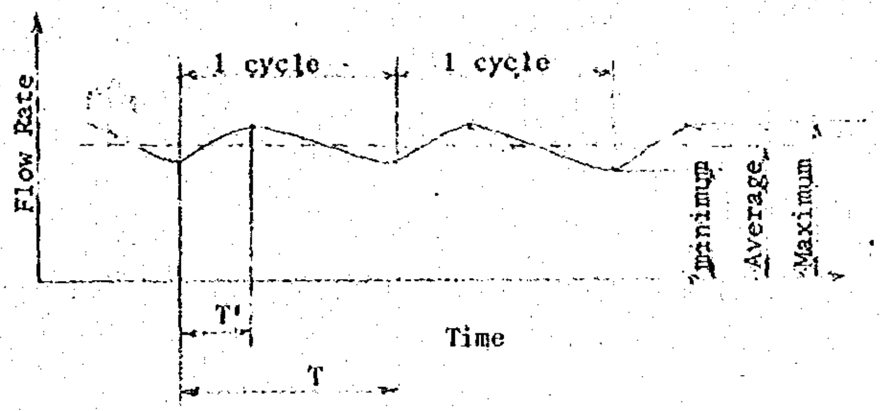
Within one cycel T, the third pump runs for T'. If the flow rate per pump is given as q, 3 pumps run for T' and 2 pumps do for T-T' in one cycle T, making the average flow rate:

$$\frac{3q.T' + 2q(T-T')}{T} = 2q + q.T'/T$$

Between Head Tank and Receiving Well

As is easily understood, the set flow rate quoted in (1) must be the average flow rate, 2q + q.T'/T.
The flow pattern between the head tank and the receiving well will be shown below:

Fig.3 Flow Pattern, Head Tank to Receiving Well

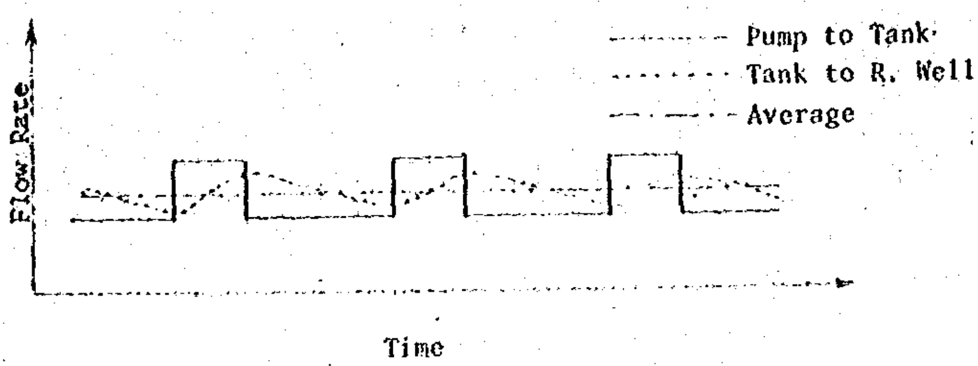


Due to the friction loss of pipeline and the valve loss, the stepped flow pattern of Fig.2 is smoothened to the wavelike flow pattern of Fig.3.

Overlapping two flow patterns

Fig.2 and Fig.3 overlapped, as for the time and flow rate, will be like Fig.4.

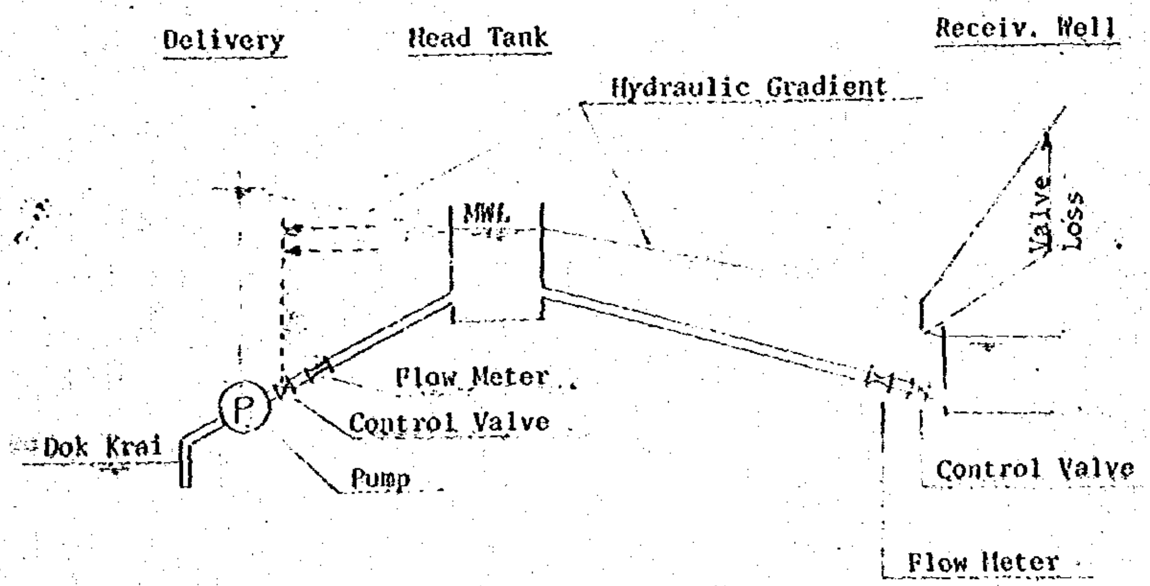
Fig.4 Flow Pattern, Overlapped



3. Control by Valve Opening

Fig.5 Shows the System and Components

Fig.5 System and Component



(1) Principle

The principle is that the pumps supply capacity must be always kept larger than the set flow rate and the excessive flow is cut by the valve at the pump.
Suppose 2.3 q, q being the capacity per unit, is required and 3 pumps must be run. The excessive 0.7 q must be cut by the valve.

(2) Setting of Flow Rate by Control Valve

Setting of the flow rate by the control valve, while keeping the water level around MWL in the head tank, is the same as in the case of control by Pump Unit Number.

(3) Control by Valve Opening

The flow from the pumps is controlled by adjusting the opening of valve. The change in opening causes the change of flow and it is seen on both the reading of near by flow meter and the water level in the head tank.

The judgement on whether open or close the valve at the pump can be done solely from the water level's position. If it is above MWL close and vice versa.
The most possible cause which necessitate the control will be the change in Dok Krai water level.

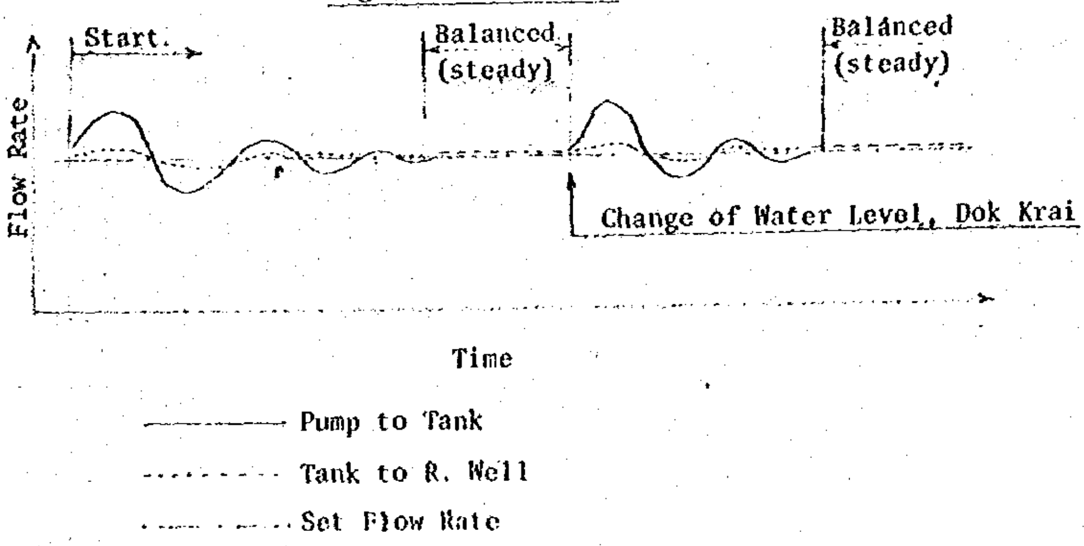
(4) Head Tank's Role

Whithin the head tank, to equalize the inflow and the outflow completely is impossible and even after the water level is stabilized it will change gradually due to the slight difference between the in - and out-flow.
The level's rise or fall must be counteracted by the valve's closing or opening manually.
To control the level change automatically, two level switches above and below of MWL can be set to operate the control valve towards closing and opening direction.
The electrical connection will work the control system so that the surging level slowly converges to MWL as shown on Fig.6.
When external disturbance like Dok Krai's level change occurs and affects the flow, the valve at the pump station shall be operated until the flow rate wi-l be controlled to converge the set rate.

(5) Flow Pattern

The flow pattern is not so specific as in the case of pump unit number. It is random as shown on Fig.6.

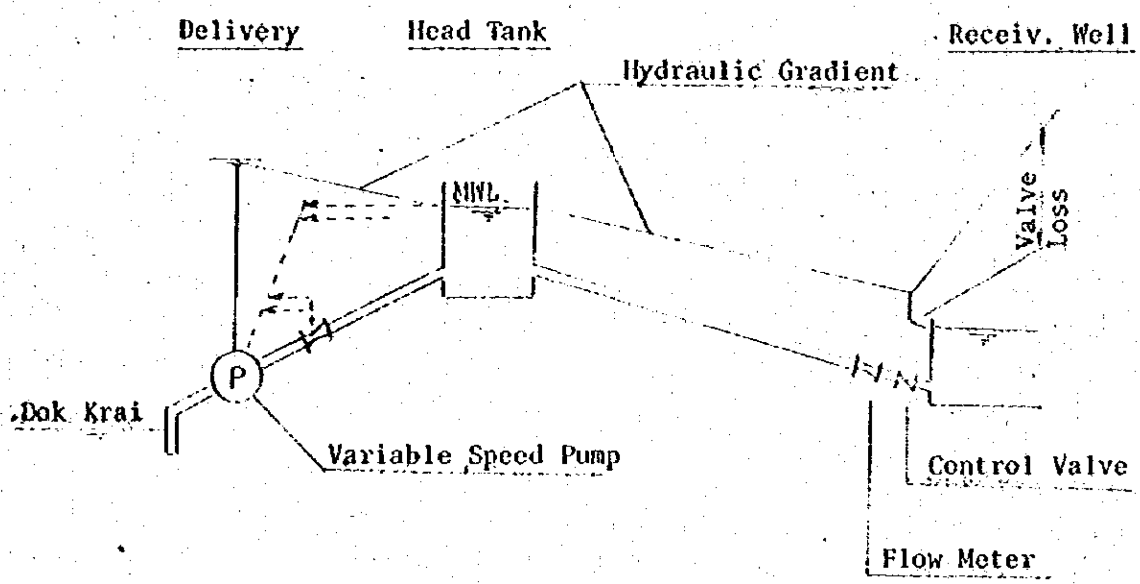
Fig.6 Flow Pattern



4. Control by Pump Rotation Speed

Fig.7 Shows the system and Components

Fig.7 System and Component



(1) Principle

The rotation speed of motor is varied and the flow rate changes accordingly.

(2) Setting of Flow Rate by Control Valve

As was explained in the previous two methods, the same operation is carried out here.

(3) Control of Rotation Speed

As was explained with control by Valve Opening, the water level in the head tank must be relied on in controlling the speed for both manual and automatic operation as in case of control by Valve Opening.

(4) Flow Pattern

The flow pattern is similar to Fig.6.

5. Discussion before Comparison

(1) Principle

The principle is the same with three methods.
Once the flow rate is set between the head tank and the receiving well, the control of flow is done by three different methods. However all of the three are in common in that the control can be relied on the water level in the head tank.
All three methods can be worked both manually and automatically.

(2) Actual Uses of Three Methods

The Pump Unit Numer Control is widely used in transmission lines, chemicals feeding lines and small treatment systems where the flow rate is approximate, a slow response to change of condition is tolerable and a simple control system is favorable. The Valve Opening Control is also used widely in the filter control of treatment plants and in the pipeline of reacting equipments in chemical plants, where the flow rate must be exact and a quick response to change of condition is wanted.
The Pump Speed Control has been used in recent years in the distribution network of the water supply. The change of speed is activated by the pressure of distribution line. For instance, when the water consumption is remarkably reduced and the pressure in the pipeline increases as the result, the pump speed is slowed down to protect the pipe and to conserve the electric power.

(3) Requirement of Planned Pipeline

Before comparing the three methods, the requirements must be reviewed.

Flow Rate and Response

Exact flow rate is not needed, as the receiving reservoir of large capacity is prepared.
Response can be slow with the same reason.

Simple and Reliable Control

As the operation personnel are expected inexperienced at the initial stage, the simpler method is better understood, operated and maintained.

The reliability involves both human and nonhuman factors and if the nonhuman factors like easier understanding, operation and maintenance are considered, the simpler one will be more reliable.

Conservation of Energy

Needless to say, the method which consumes less energy is preferable.

6. Comparison

(1) Flow Rate and Response, Simplicity and Reliability

By Principle, the control by Pump Unit Number is the simplest as it needs not be exact and quick respondent. It can be seen from the application of the method. To make the matters worse for the control by Valve Opening, it may cause the pump's and valve's cavitation resulting in the poorer control functioning.

(2) Cost

As for the initial cost, the Pump Unit Number Control is less expensive than the other two methods because of its simplicity. As for the energy consumption or running cost, Pump Rotation Speed Control is the lowest, as the method has been developed for that purpose. The Pump Unit Number Control will use a little more and the Valve Opening Control will use much more, than the Pump Rotation Speed Control.
The maintenance cost for the Valve Opening and Pump Rotation Speed control methods are pretty higher than for the Pump Unit Number method, as the latter is far simpler mechanically.

(3) Head Tank's Capacity

The Pump Unit Number Control needs a little more depth, about 2 meter, than the other two. The extra depth does not affect much in the cost of about 20 m high head tank.

7. Conclusion

As can be seen in the previous discussion and comparison, the control by Pump Unit Number is ;

* the simplest and the most reliable for the planned pipeline

* the cheapest in the initial and maintenance cost

* the second cheapest in the running cost

* in need of a slightly extra height of the head tank but the cost increase does not matter much.

Conclusively, it shall be the choice.

NO.7-2 HYDRAULICS AND OPERATION OF PUMP UNIT NUMBER CONTROL SYSTEM

JANUARY 1982

Prepared by YOSHIYUKI TOMIOKA
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1. INTRODUCTION

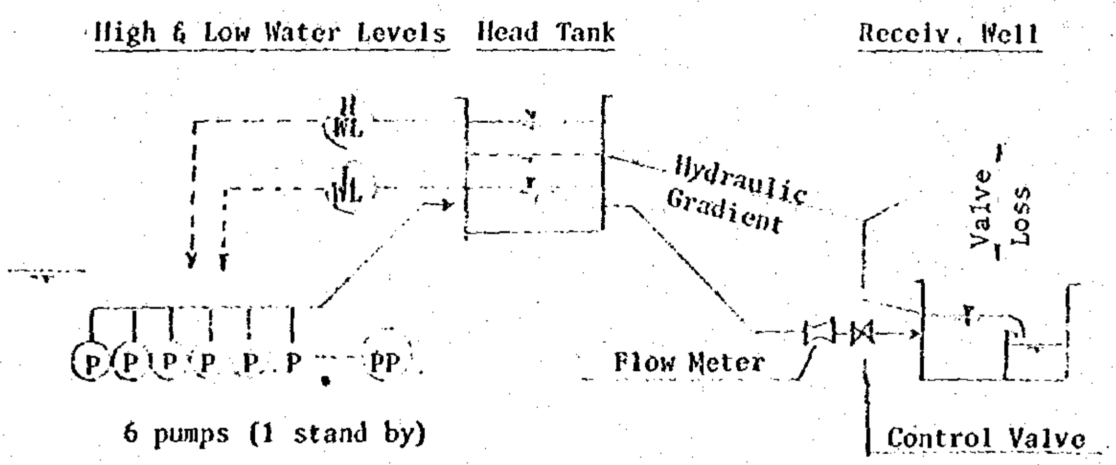
Comparative Study of Flow Control System has proved and concluded that the simplest, lowest in cost, most reliable method in controling the flow is Pump Unit Number Control.
Needless to say, in the report the control method is discussed in principle and practical matters, in order to be compared with two other methods.
In this report, hydraulic study will be made to examine the fluctuation of flow and the periodical surging of the head tank's water level.
Also here, the pumps' schedule of operation will be studied, in relation to the head tank's water level.

2. EXPLANATION OF CONTROL

2.1. Facilities and Equipments

Fig.1 shows the facilities and equipments used in controling the flow.

Fig.-1 Facilities



(1) Pumps and Pumps' Programmer

Depending on the set flow rate, from 1 to 5 pumps will lift water. Actually 6 pumps are prepared to have 1 stand-by in case 5 pumps are run. For maintenance reason, each pump is run in turn and the programmer is used for the purpose. On Fig.1 (P) and (PP) are the pumps and pump programmer respectively.

(2) Level (Pressure) Switches in Head Tank

To start and stop the pump, two level switches must be installed in the head tank, one for the high water level and another for the low water level. They are symbolized as (H WL) and (L WL) on Fig.1.

The gap of the two levels relates to the period or cycle of surging in the head tank. The level switches can be of various types, such as electrode, contacts on pressure gauges, etc.

(3) Control Valves and Flow Meter at Receiving Well

These are for setting the flow rates.

The control valve, electrically driven, can be turned both directions and be stopped at any position. The operator can adjust the valve delicately, checking with the reading of flow meter, until the desired rate is set and steadied. Here, caused by the surge in head tank, the reading of flow meter will sway and it will seemingly make the adjustment rather difficult.

The sway is practically very small by hydraulic reason, as it is explained later.

The valve causes the loss in the pipeline and it is effective in steadying the flow.

2.2. Hydraulics of Pipeline

(1) Hydraulic Gradient

Substituting the roughness coefficient C = 120 and the pipe diameter D = 1.35 m into Hazen-Williams formula, it can be transformed to :

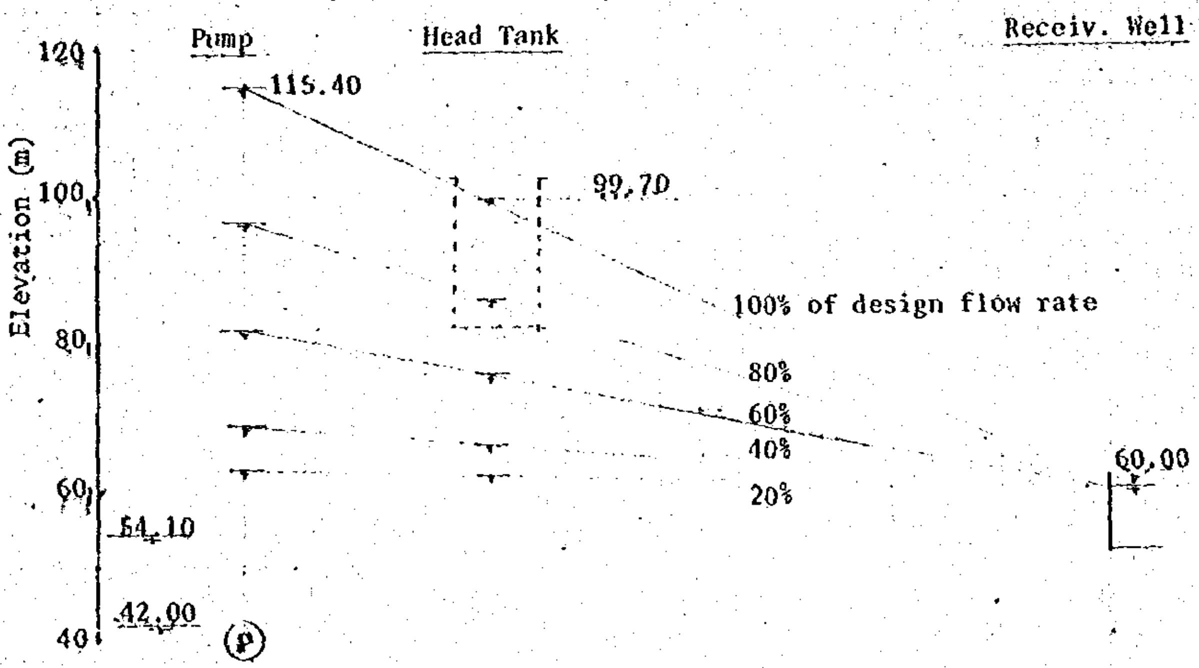
$$I = 3.522 \times 10^{-4} \times Q^{1.85}$$

Q : flow rate, m^3/sec

I : hydraulic gradient

With the fomula, calculation is made on 5 cases of Q, that is, 100, 80, 60, 40, 20% of the design flow rate 2.62 m^3/sec. The result is shown on Fig.2.

Fig-2 Hydraulic Gradient

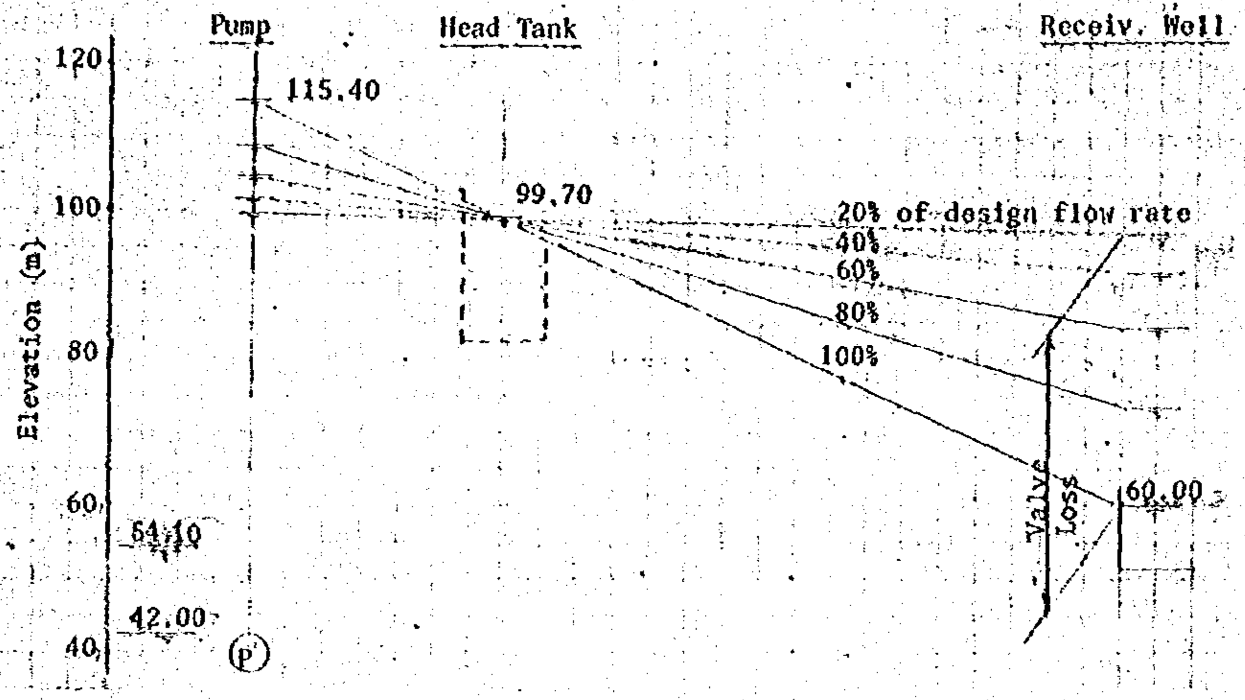


(2) Controling Valve at Receiving Well

As is seen on Fig.2, except the two cases 100% and 80% the 60, 40, 20% lines fall below the head tank, also shown by dotted line.

It leads that the valve must be so controlled that all the lines can stay in the tank, in order to let the pipeline work and to make pump unit control possible.

Fig.3 Controling Valve at Receiving Well



As is seen on Fig.3, nearly 40 m head loss shall be given by the valve, in case of 20% flow of the design rate.
For purpose of control, the head tanks level shall not necessarily be kept at 99.70 accurately.

2.3. Fluctuation of Flow between Head Tank and Receiving Well

As it is quoted before, the flow rate is proportional to 0.54 powers of the hydraulic gradient.
Generally, the equation is :

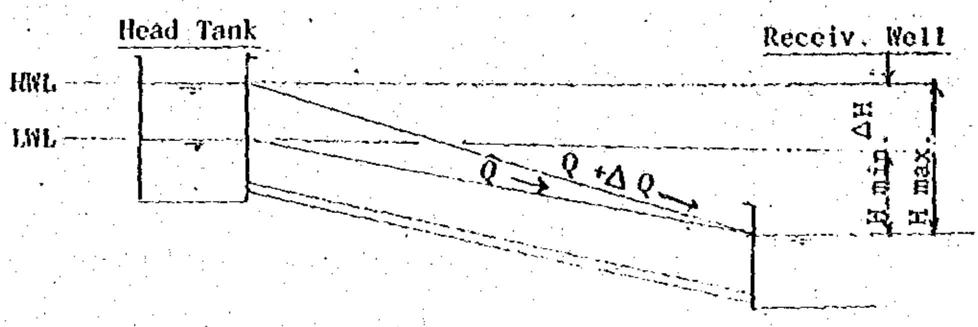
$$Q = k\ I^{0.54}$$

As the hydraulic gradient is equal to the head loss divided by the pipeline length, the above equation becomes :

$$Q = k'\ H^{0.54}$$

When the water level in head tank rises from LWL (Low Water Level) to HWL (High Water Level) and accordingly the head loss changes from H min to H max, as shown on Fig.4, the flow rate also changes from Q to Q + ΔQ.

Fig-4 Fluctuation of Flow



ΔQ, the increase of flow, is caused by the increase of head loss which is designated as $\Delta H = H\ max - H\ min$.
Using the previously quoted equation :

$$Q = k'\ H\ min$$
$$Q + \Delta Q = k'\ H\ max = k'(H\ min + \Delta H)$$

From the two equations, an approximate equation is lead:

$$\frac{\Delta Q}{Q} \doteqdot 0.54\ \frac{\Delta H}{H\ min}$$

In the case of the project pipeline, as shown on Fig.3, H max = 99.70 - 60.00 = 39.70 m and H, the gap between HWL and LWL can be reasonably assumed about 2.50 m.
Then, H min = 39.70 - 2.50 = 37.20
Substiting the values of H min and H :

$$\frac{\Delta Q}{Q} \doteqdot 0.54 \times \frac{2.50}{37.20} = 0.036 = 3.6\%$$

Under the discussed condition, the flow rate increases only by 3.6 per cent and as the change is so small that the flow rate can be considered as steady for practical purpose.

2.4. Fluctuation of Flow between Pump and Head Tank

(1) Setting Flow Rate

At the receiving well, the inflow is practically steady and the desired flow rate can be set without difficulty by controling the valve.

The control of flow by Pump Unit Number is a stepping control flow as it is explained in Report on Comparative Study of Flow Control System.

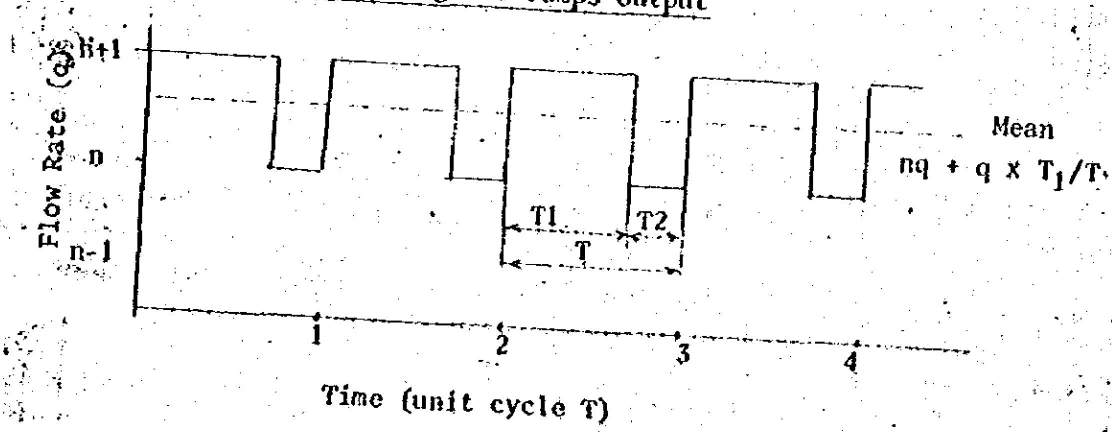
Accordingly, the set flow rate can be put down as $nq+\alpha q$, where q is the flow capacity per pump and α is between o and 1.

The set flow rate is between nq and (n+1)q and it shall be attained by running n number pumps continuously and the (n+1)th pump intermittently.

(2) Pump's Operation

Fig.5 shows how the pumps output changes against time. The abscissa and ordinate are graduated with the cycle (period) time and the capacity per unit pump.

Fig-5 Change of Pumps Output



As shown on the figure, n number pumps run continuouly while the (n+1)th pump repeats T1 time running and T2 time stopping during one cycle time T, the mean output is calculated as :

$$nq + q \times \frac{T1}{T1+T2} \quad \text{or} \quad nq + q \times T1/T$$

While in the foregoing paragraph, the set flow rate is expressed as $nq + \alpha q$.

As the pumps' output is equal to the set flow rate :

$$\alpha q = q \times T1/T \text{ or } \alpha = T1/T,\ o<\alpha<1$$

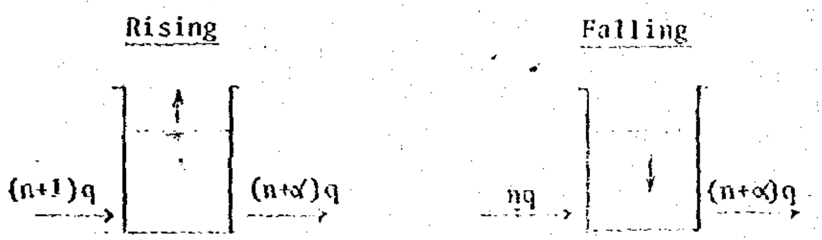
Needless to say, the (n+1)th pump stops when the water level comes up to HWL, as (n+1)q is larger than the set rate nq + αq and the balance is gradually stored in the tank, pushing the level up. And the pump starts when the water level comes down to LWL.

3. HEAD TANK'S ROLE IN CONTROL

3.1. Change of Water Level

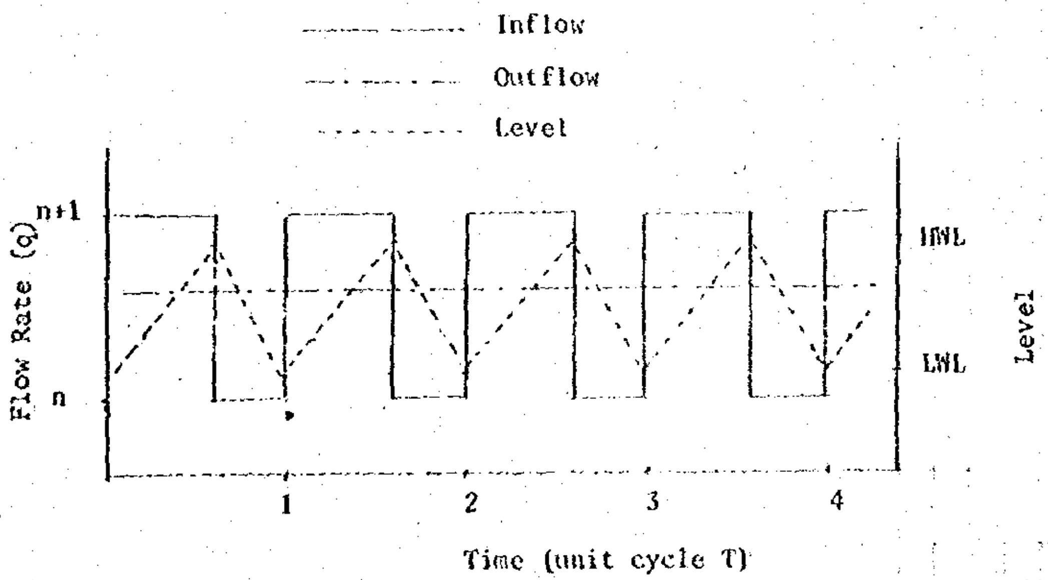
Fig.6 shows the water level's rising and falling.

Fig-6 Water Level Change



When the (n+1) pump is running, the balance of inflow and outflow (1-α)q pushes the level up and when the pump is kept stopped, the balance αq causes the level fall.
In both cases of rise and fall, the speed of level change is constant as the balance of flow is constant.
Accordingly, the change of head tank's level, the inflow and outflow relates as shown on Fig.7

Fig-7 Change of Inflow, Outflow, Level



3.2. Cycle Time

The cyclic time of rise and fall must be studied.
The tank's volume between HWL and LWL is designated as V.
The times required for rising and falling can be expressed as :

$$\text{rising} \qquad \frac{V}{(1-\alpha)q}$$

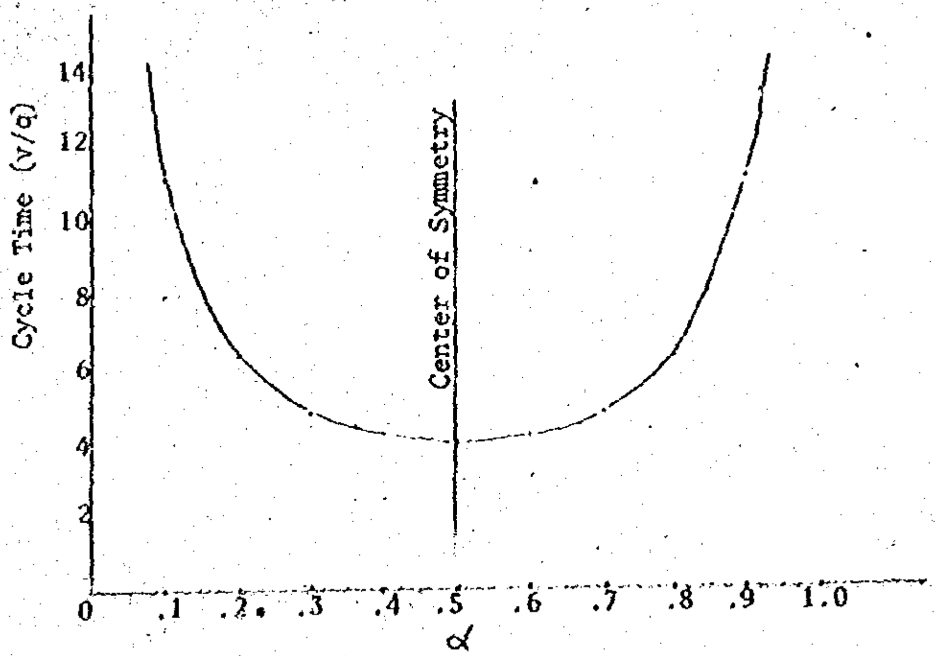
$$\text{falling} \qquad \frac{V}{\alpha q}$$

And one cycle time is :

$$\frac{V}{q}\left(\frac{1}{1-\alpha}+\frac{1}{\alpha}\right)=\frac{V}{q}\cdot\frac{1}{\alpha(\alpha-1)}$$

As V/q becomes a constant valve which is determined by the tank's size, the position of HWL and LWL, unit pump's capacity, the parameter affects the cycle time.
Fig.8 shows the relationship between α and the cycle time.

Fig-8 and Cycle Time



The curve on Fig.8 is symmetrical. The cycle time is the shortest at α= 0.5 and it becomes longer for both way as α increases and decreases. At α = 0.15 and α = 0.85, the cycle time is about two times of that at α=0.5, and at α=0.91, it is about 3 times.

Clearly Fig.8 shows that the cycle time becomes infinite when α and $1-\alpha$ approaches to o and 1. Hydraulically meaning, the flow rate now equals nq or (n+1)q and all the pumps are run continuously and the level is steadied, making no rise and fall any more.

The effect of cycle time on the pumps operation shall be discussed later.

As it is understood already, the cycle time of rise and fall is independent of the flow rate. Of the set flow rate $nq + \alpha q$, only αq affects the cycle time and nq does not.

3.3. Head Tank's Shape

The tanks volume between HWL and LWL is a factor to determine the cycle time. How the cycle time relates to the pump operation shall be studied later.

For control of the flow, the narrower gap between HWL and LWL is better as long as the two levels are sensed distinctively. A flat tank is preferable.

4. CYCLE TIME AND PUMP OPERATION

4.1. Pumps Capacity and Head Tank's Size

The capacity of one pump q is 0.524 m^3/sec.
The head tanks size, now in the stage of conceptual design, is 16 m in diameter and about 21 m in water depth. For purpose of controling the flow, HWL and LWL is set 2.50 m apart around the top of water depth. The volume of surging is $0.785 \times 16^2 \times 2.5 = 502.4\ m^3$

4.2. Cycle Time

The cycle time as against αq can be tabulated as follows:

α	q m^3/sec	Cycle time min.	sec.
0	0		
0.1	0.052	177	33
0.2	0.105	99	52
0.3	0.157	76	6
0.4	0.210	66	34
0.5	0.262	63	52
0.6	0.314	66	34
0.7	0.367	76	6
0.8	0.419	99	52
0.9	0.472	177	33
1.0	0.524		

As it is seen in the table, the surge is remarkably slow. Even the shortest cycle is 64 min and as the half of it is for the rise, the rising speed is 2.50 m/32 min. = 7.8 cm/min.

4.3. Pump Operation Schedule

(1) Flow Rate Pattern

The set flow rate equals $nq + \alpha q$.

n number pumps are running continuously and the (n+1)th pump periodically. The (n+1)th pump runs for T1 time and keeps stopping for T2 time.

Here, $T1 + T2 = T$ T : cycle time

$= T1/T$

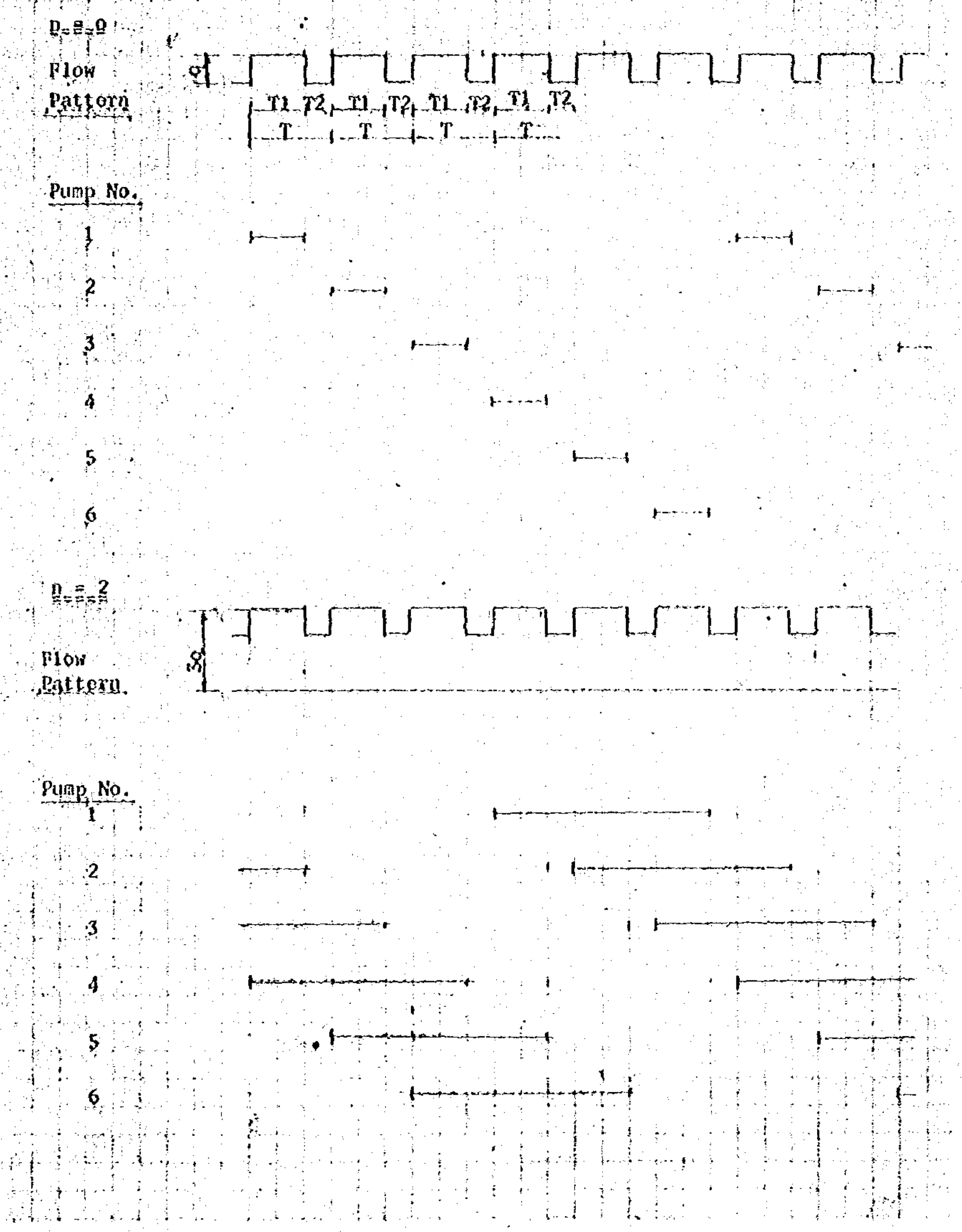
(2) Pumps' Rotation

The pump programmer lets the pumps run in rotation so that every pump is under the same condition.

(3) Pumps' Running Schedule, Rule of Programme

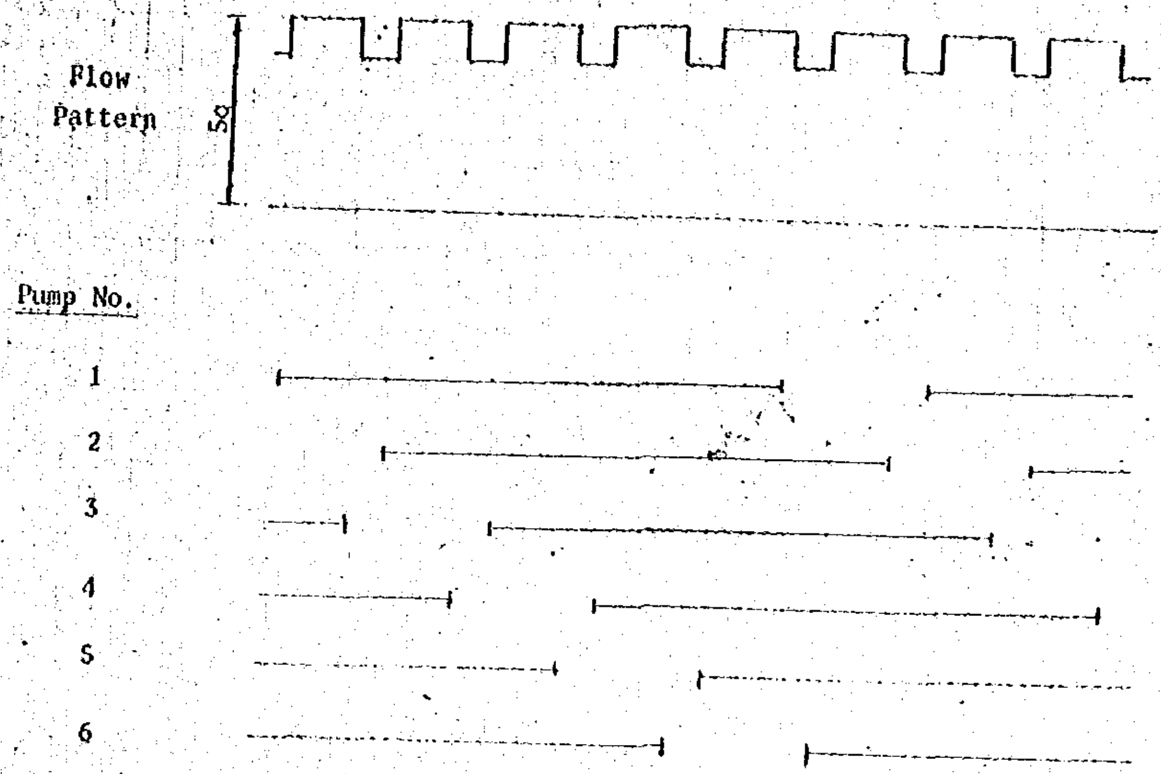
Fig.9 is the schedule of 6 pumps under a certain cycle time T, with n = 0, 2, 4, as shown on the following pages.

Fig-9 Schedule of 6 Pumps



D=4

Fig-9 Schedule of 6 Pumps (continued)



From Fig.9 the following general rules are found :

* Each pump starts and stops every 6 T.
* The running time T, is nT + T1 and the resting (kept stopped) time is (5-n)T + T2.
* The interval of every start and stop is T.

These rules are to be used in designing the pump programmer.

(4) Occurence of Strong Electric Current by Starting Pumps

When a pump is started, a strong instant current flows at the switch and in the motor and too frequent starting is detrimental to the gadgets.

Electricity is supplied through the main switch and the branch switch to each motor.

The branch switch and motor are started every 6T and the main switch is affected by the additional current every T.

As it is discussed previously, T is longer than 60 minutes and is long enough for reasonably normal operation of the motor and switches.

5. CHANGING SET FLOW RATE

5.1. Increase of Demand

Fig-10 Increase of Demand

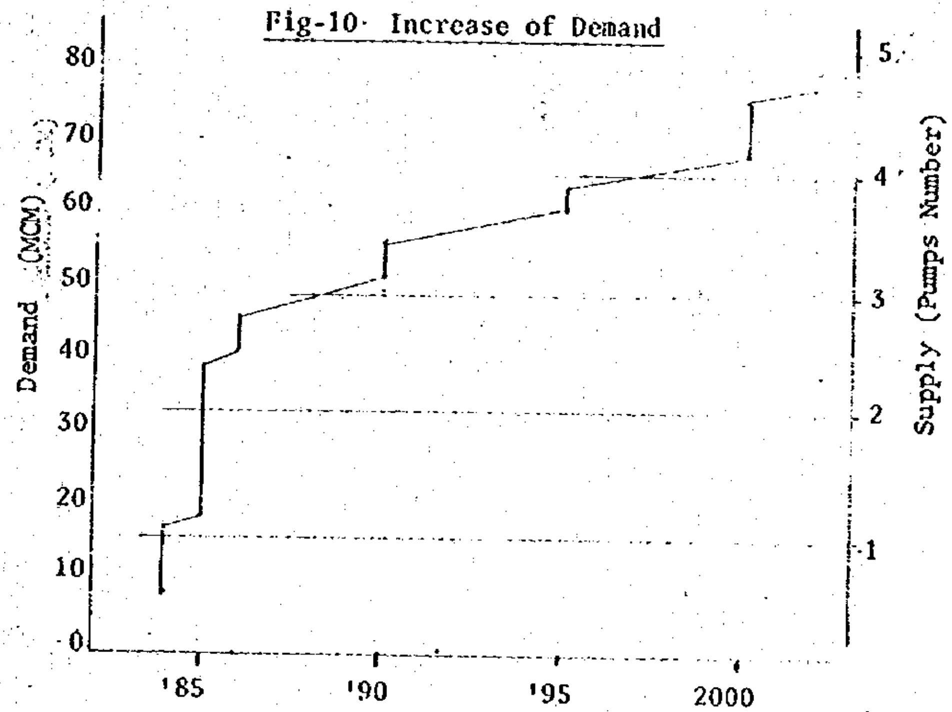


Fig.10 shows the increase of demand from 1984 to 2000. In '84, the first year of operation, initially 1 and very soon 2 pumps will be working. In '85, the 3rd pump must be placed in operation and 3 pumps stage will go on till '88.

5.2. 5 Stages

5 stages, corresponding the increase, can be expected. They are tabulated as shown below, using the same terms quoted before :

Stage	No. of Pumps		Range of Flow
	Continuous	Intermittent	Unit:q
1st	0	1	0 - 1
2nd	1	1	1 - 2
3rd	2	1	2 - 3
4th	3	1	3 - 4
5th	4	1	4 - 5

The pump programmer shall work the pumps rotational operation at each of the 5 stages.
The most possible practice of operation is to move from the existing stage to the next one. For instance, to go to 2nd from 1st, the operator will have to reach an equilibrium at 1 pump's continuous operation and then will set the flow rate at the desired value between 1 and 2.
However, a sudden change like jumping from 1st to 3rd must be also expected probably in case of emergency.

5.3. For Sudden Change of Flow

The pump programmer shall have 5 independent circuit for the 5 stage and shall be able to select any one of them. It will also be used for the gradual move to next stage, needless to say.

6. SUMMERY

The discussion made in this paper will be summerized here :

(1) Setting the flow rate can be worked out with no difficulty.

(2) The flow between the head tank and the receiving well is steady.

(3) The stepping flow pattern of pumps output is changed into the steady flow of the head tanks outflow.

(4) The rise and fall of head tank's water level is cyclic and the cycle time depends on the intermittence of one pump and the tank size. The cycle is rather slow practically.

(5) All the pumps shall be run in turn and the schedule of rotation can be programmed.

(6) The change of cycle time does not cause any problem on the electrical equipment and the motor of pump.

(7) A programme must be prepared for both gradual and sudden change of the set flow rate.

NO.8 REPORT ON DESIGN OF HEAD TANK

MARCH 1982

Prepared by

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1. INTRODUCTION

The Report "Pumping System and Water Hammer" proves that the head tank is absolutely necessary to prevent the occurence of negative pressure in the pipeline, due to water hammer.

The Report "Comparative Study of Flow Control System" concludes that Pump Unit Number Control is best applicable for the flow control of the pipeline. "Hydraulics and Operation of Pump Unit Number Control System" discusses the storage capacity, the size and shape of storage part, of the head tank required for control.

In this paper, another operational issue is raised and discussed. And then, the basic matters of design will be elaborated, reasoned and derived from the papers quoted above.

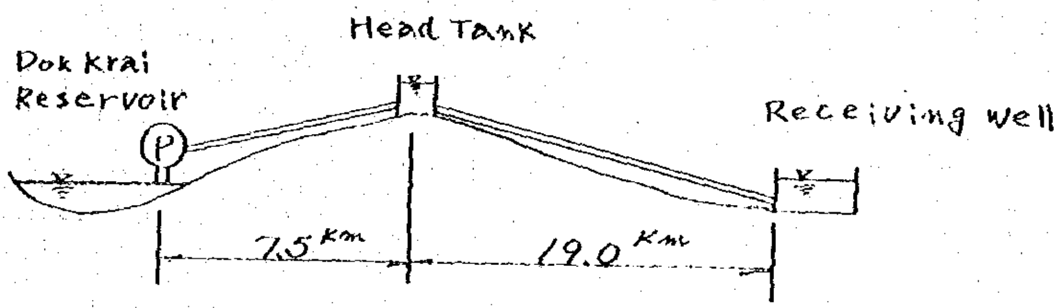
2. DESIGN BACKGROUND

2.1. Topographical, Geological

The head tank will be located at the roadside of Route 3191. The ground level is around 82.00 m and the soil is clayey sand or sandy clay, decomposed and weathered of granite (Feasibility Study). The bearing power seems to be rather high there.
The head tank shall be constructed at the ground level of more or less than 82.00 m.

Study on the pipeline system by the Feasibility Study Team and the Detailed Design Team's review were made with a topographical map which is 1/50,000 scale and without topographical survey. After the D.D. Team's review on the feasibility study the topographical survey has been finished along the pipeline route and now it must be finalized according to the result.
From the result of the longitudinal survey the length of each part of the pipeline is shown in Fig. 1

Fig. 1 Pipeline Length



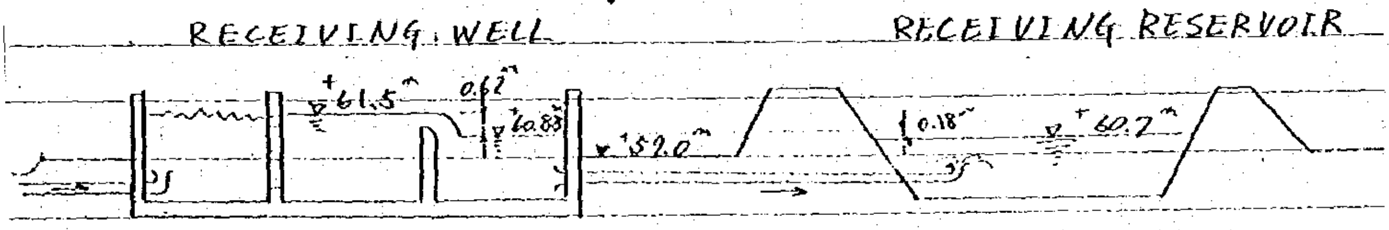
2.2. Hydraulic

2.2.1. Water level of the receiving well at Mab Ta Pud

The altitude of the proposed site is EL 59.0 m, and to minimize the earth work volume such as excavation and banking of receiving reservoir, the altitude EL 60.7 m

is most favorable for the water level of receiving reservoir.
The loss head between the receiving reservoir and receiving well is estimated around 0.18 m and overflow depth at a weir in the receiving well is estimated 0.62 m.
From these conditions the first pond water level of receiving well should be EL 61.5 m. These figures are shown in Fig. 2.

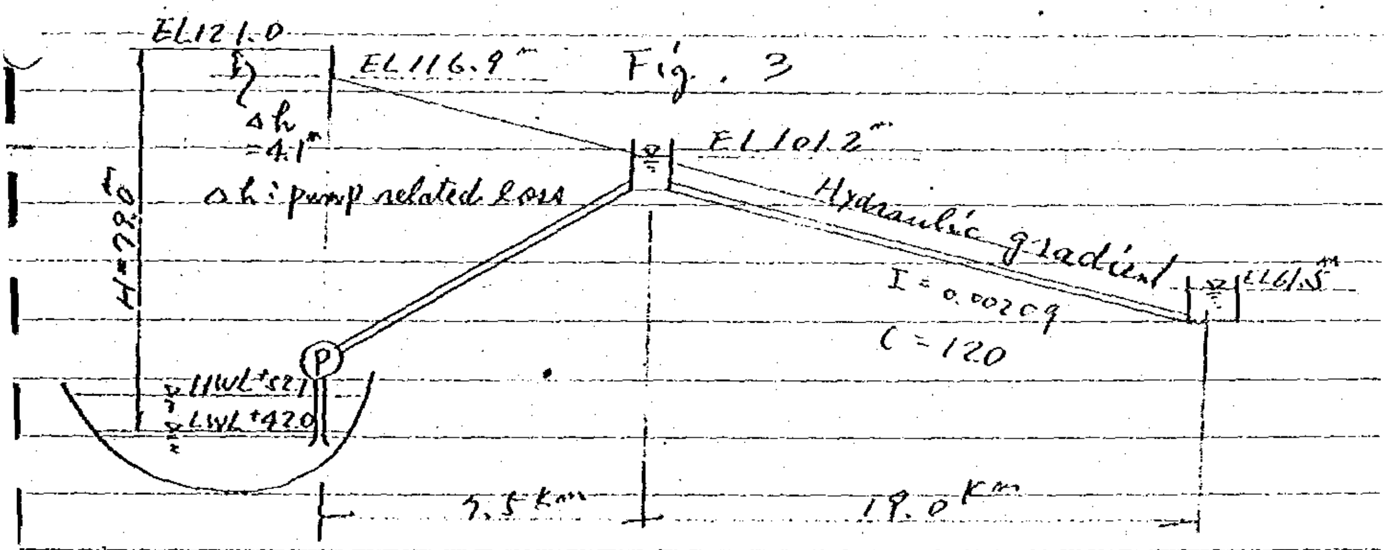
Fig. 2



2.2.2. Water level at the head tank

To attain the design flow rate 2.62 m^3/sec by 1.35 m dia. pipe of c = 120 (roughness coeficient) and 19,000 m long, with the water level 61.50 m at the receiving well, the water level of head tank must be 101.20 m, that is, 101.20 m level must be within the head tank.

Fig. 3



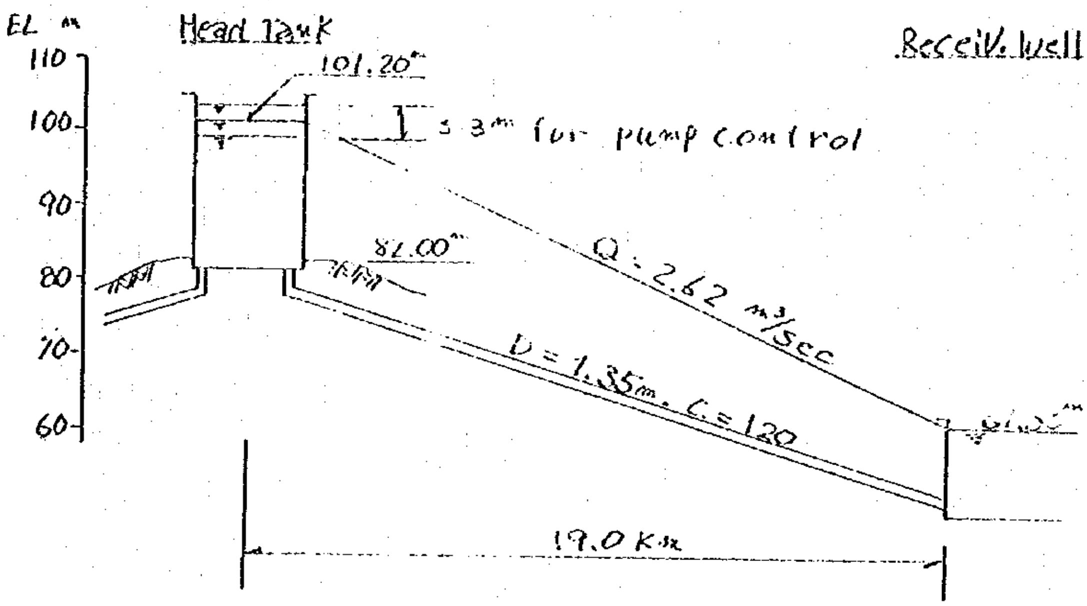
The major hydraulic dimensions are fixed as below:

* Water level at the receiving well : EL +61.5 m
* Mean water level at the head tank : EL+101.2 m
* Total head of the pumps (H) : 79 m
* Pump related loss* is assumed 4.1 m

2.2.3. Height of Head Tank

Before going on to further discussion, a picture shall be shown to see the approximate height of head tank from the foregoing description. (Fig. 4)

Fig. 4 Height of Head Tank



As it is seen on the figure, a tank of about 25 m height from the ground is anticipated.

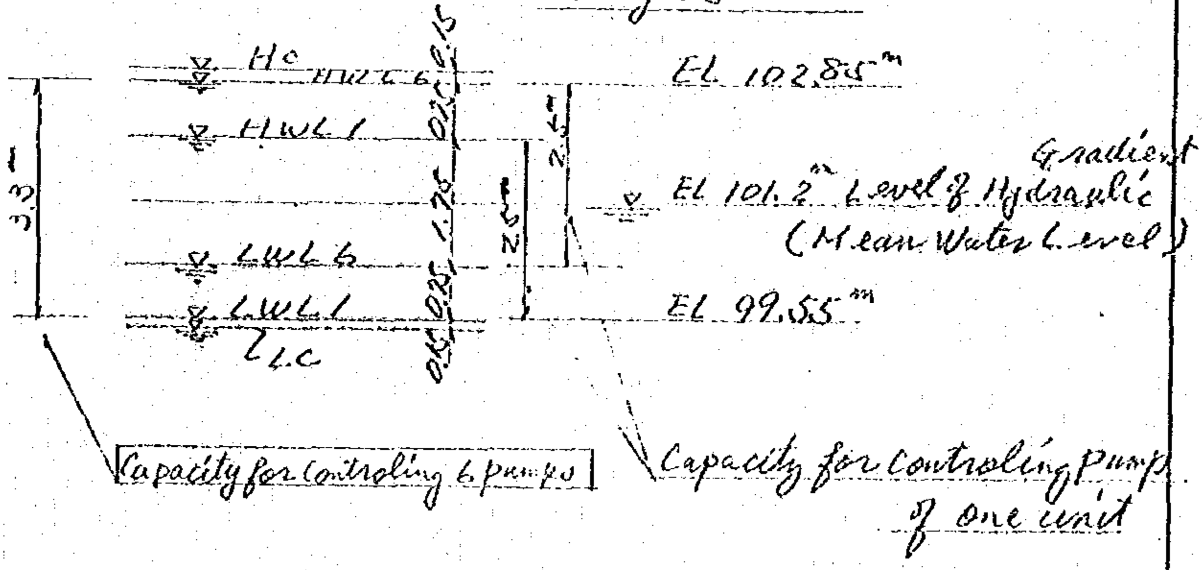
* Pump related loss include a screen loss, an entrance loss at the intake gate and intake pipe, and etc.

3. REQUIREMENTS ON TANK'S CAPACITY

3.1. For Controling Pumps

In"Hydraulics and Operation of Pump Unit Control System",it is found that the 3.3 m deep parts is used for control, as shown in Fig. 5.

Fig. 5

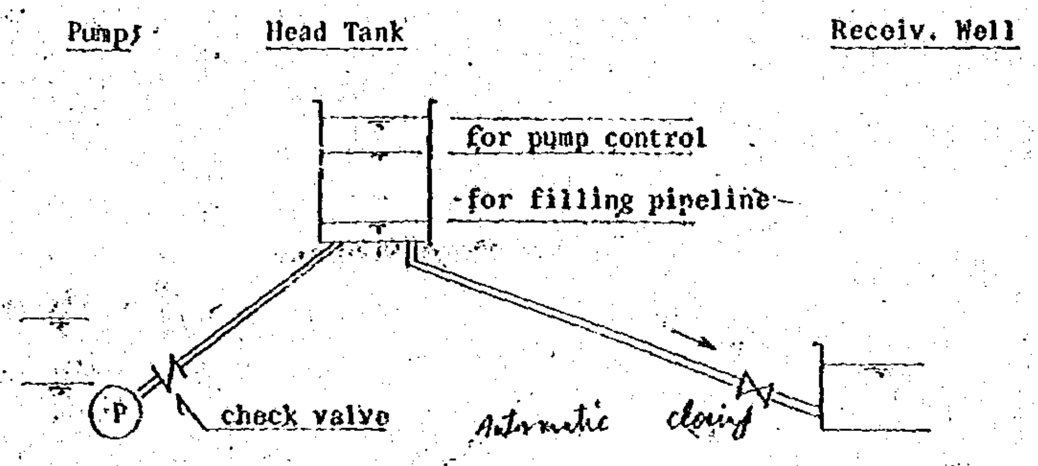


3.2. For Filling Pipeline

It is a well known fact that a pipeline must always be kept filled with water, whether it is working or resting, only except it can be put out of operation for long duration. In case of the power failure and other accidents, the part of pipeline between the head tank and the receiving well must be closed sooner by the valve of receiving well. Otherwise, the water in it will flow-out and make it emptied.

The part between the pump and the head tank, controled by the check valve, does not need any operation. The type of valve suitable for the system is needed at the receiving well, as shown on Fig. 6.

Fig. 6 Filling Pipeline



The capacity of tank for filling shall be discussed in detail in the following chapter.

3.3. For Water Hammer

The occurence of water hammer shall be expected in two cases. The first case is its occurence in the part of pipeline between the pumps and the head tank, due to sudden failure of electric power.
The second case is the occurence in the part of pipeline between the head tank and the receiving well, caused by quick closing of the valve in case of the power failure and other accidents.
In both cases, the water hammer induces the change of water level in the tank.
It must be analysed quantitatively after deciding the details of pumps, air chamber and head tank.

After determination of both capacity for pumps control and filling pipeline, capacity for water hammer is checked with the capacity mentioned above. The result is shown in Fig. 7 under conditions as shown on the next page:

1) Pump

Discharge	31.44 m^3/min x 5 units
Total head	79.0 m

2) Pipeline

Diameter	1,350 mm	
Material	steel pipe	
Length	pump station - air chamber	220 m
	air chamber - head tank	7,280 m
	head tank - receiving well	19,000 m

3) Water level

Dok krai Reservoir	LWL	42.0 m
	NWL	52.1 m
Head Tank	LWL	99.55 m
	MWL	101.20 m
	HWL	102.85 m
Receiving well	NWL	61.5 m

4) Diameter of the Head Tank 16 m

5) Capacity of the Air Chamber

32 m^3 x 3 sets

6) Initial Conditions

* pump suction level LWL 42.0 m
* head tank level LWL 99.55 m
* receiving well level NWL 61.5 m
* flow rate of pipeline 2.7 m^3/s (LWL to LWL)
* value of c = 120

7) Case of study

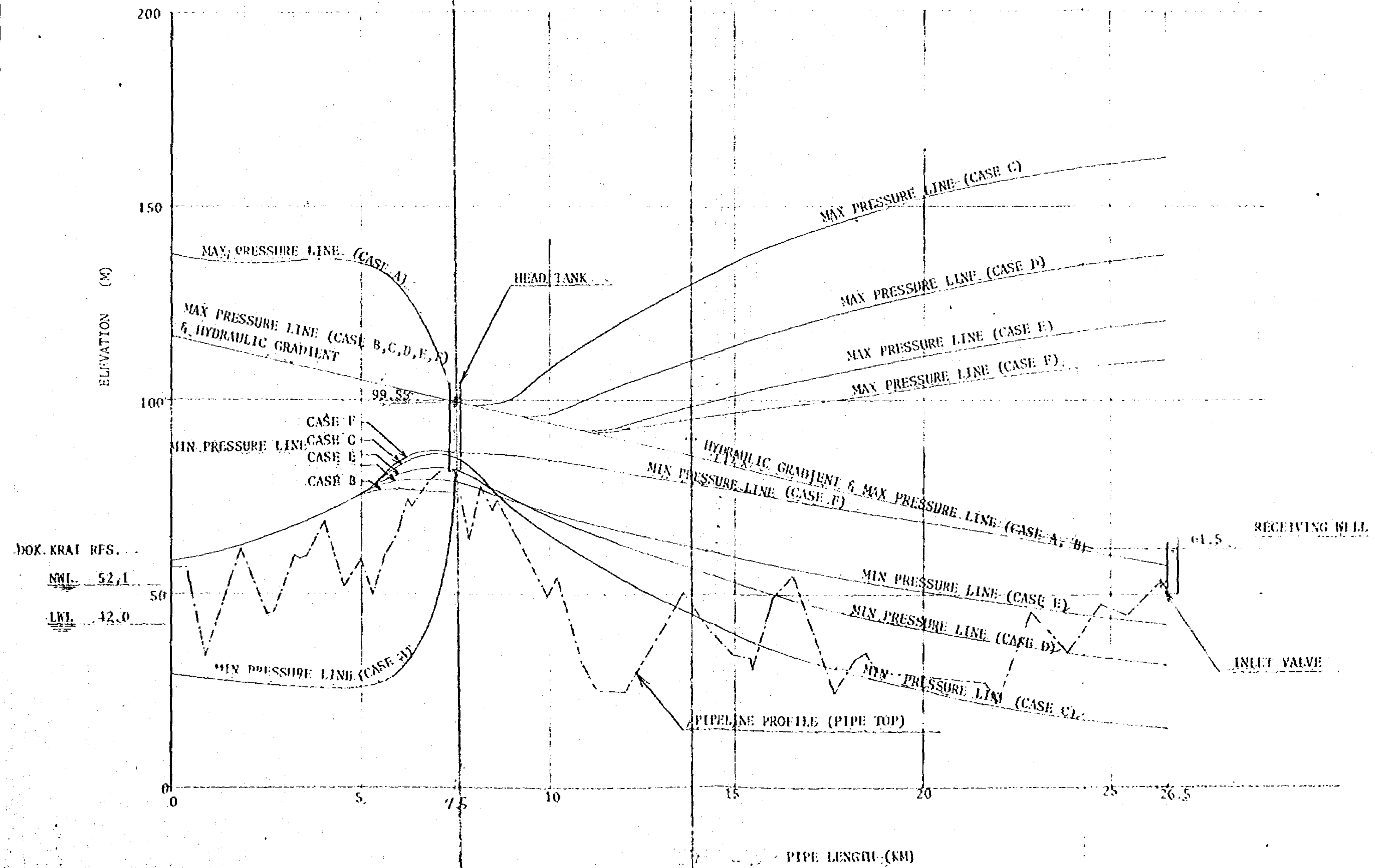
Table - 1

Case	A	B	C	D	E	F
Pump	Trip	Trip	Trip	Trip	Trip	Trip
Air Chamber	without	with	with	with	with	with
Inlet Valve of Rec. Well	batterfly valve not closed	" not closed	" Closed time 10min	" 15min	" 20min	rote valve 10 min

From Fig. 7 it is clear that case F is most suitable for the system. Because the minimum pressure line is above the pipeline at any portion and furthermore the maximum pressure is smallest. Accordingly it can be said that the capacity for water hammer is enough using the capacity of pump control and filling pipeline with a rote valve at the receiving well (case F).

No.

FIG. 7 WATER HAMMER ANALYSIS



4. CAPACITY FOR FILLING PIPELINE

4.1. Flow Condition

Needless to say, a pipeline is emptied at fastest when the end valve is open fully and here the valve at the receiving well is open fully for the largest (design) flow rate. The discussion will be made in the case.

4.2. Automatically - Closing Valve

An automatically - closing valve, by name trigger valve, will be used to close the pipeline automatically at power supply suspension. A rote valve is preferable for the sake of prevention of negative pressure and it is also used for flow control. The valve is closed in about 10 minutes by design and more 10 minutes is taken here for safety.

4.3. Capacity, Depth

The flow rate is 2.62 m^3/sec and the time for closing is 20 minutes. Then, the required capacity becomes:

$$2.62 \times 20 \times 60 = 3,144 \ m^3$$

Actually, here is another margin for safety, because the flow decrease as the valve is closed and more-over as the water level of head tank is lowered by flowing out.
The capacity is for filling and it is stored at the bottom of the tank while the capacity for pump control shall be at the top of it.
In "Hydraulics and Operation of Pump Unit Number Control System", the head tank is assumed to be circular with 16 m diameter.
The capacity 3,144 m^3 in the 16 m diameter tank needs about 15.7 m depth.

4.4. At Lower Flow Rate

When the flow rate is lower than the design flow, the consideration about filling the pipeline becomes far less serious due to the following reasons:

(1) 20 minutes retention time is for the design flow and for the lower flow rate it will increase in inverse proportion.

(2) When the flow rate is low, the control valve is in a position to throttle the flow already.
The automatically-closing valve can be closed in shorter time than 20 minutes.

5. EXTRA DEPTH FOR OVERFLOW AND OUTFLOW

5.1. Overflow

At the top, a morning glory type overflow is needed. Considering an extraordinary case when the design flow spills out of the overflow, the overflow depth can be calculated by the following equation:

$$Q = C(2\pi R)\,Ho^{3/2}$$

Q : design flow, 2.62 m^3/sec

C : coeff. of discharge 2.0

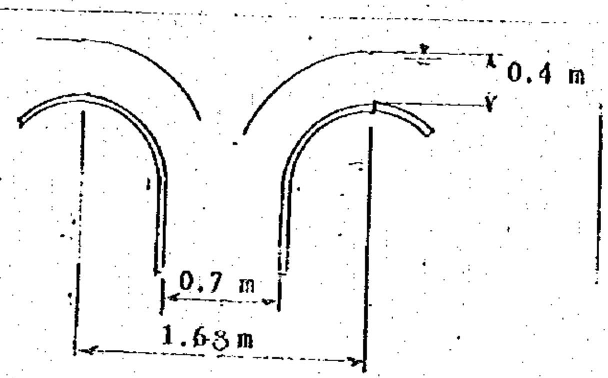
R : radius of morning glory, 0.84 m

Ho : overflow depth, m

Ho is calculated to be o.40 m.

The above description is shown below by Fig. 8

Fig. 8 Overflow



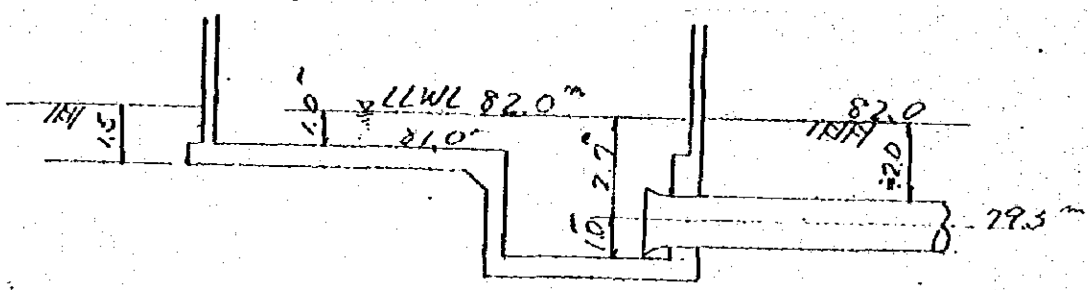
The water spilled out here will be diverted to a ditch, about 400 m apart from the tank on upstream side, which seems to be wide enough for the flow.

5.2. Outflow and Sand Deposit

When the water level in a tanks come down too low, a vortex takes place and air is sucked into the outflow through it. To avoid it, the lowest water level must be at 2.0 times of the pipe diameter high, above the center of outflow pipe.

That is, the LWL must be 2.7 m above the center of the outflow pipe. Fig. 9 shows that sump is set to decrease dead head tank capacity at the bottom.

Fig. 9 Outflow



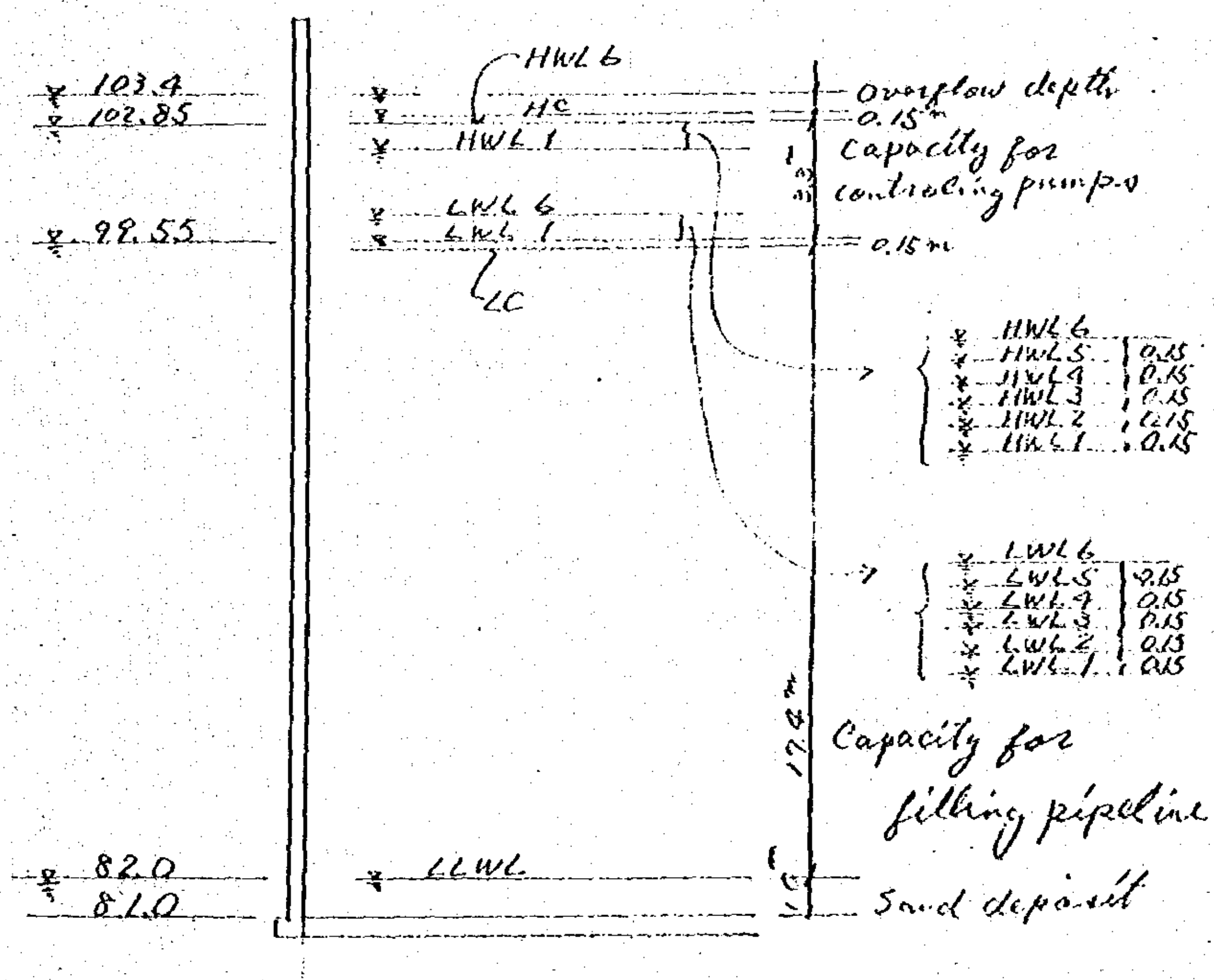
The bottom of the base concrete should be 1.5 m below the ground level so that it is not to be scoured or excavated.
1.0 m depth can be prepared for sand deposit assuming that thickness of base concrete is 50 cm.

6. DIMENSION OF HEAD TANK

6.1. Requirements on Capacity and Depth

Fig. 10 shows the requirements discussed so far.

Fig. 10 Capacity and Depth Requirement

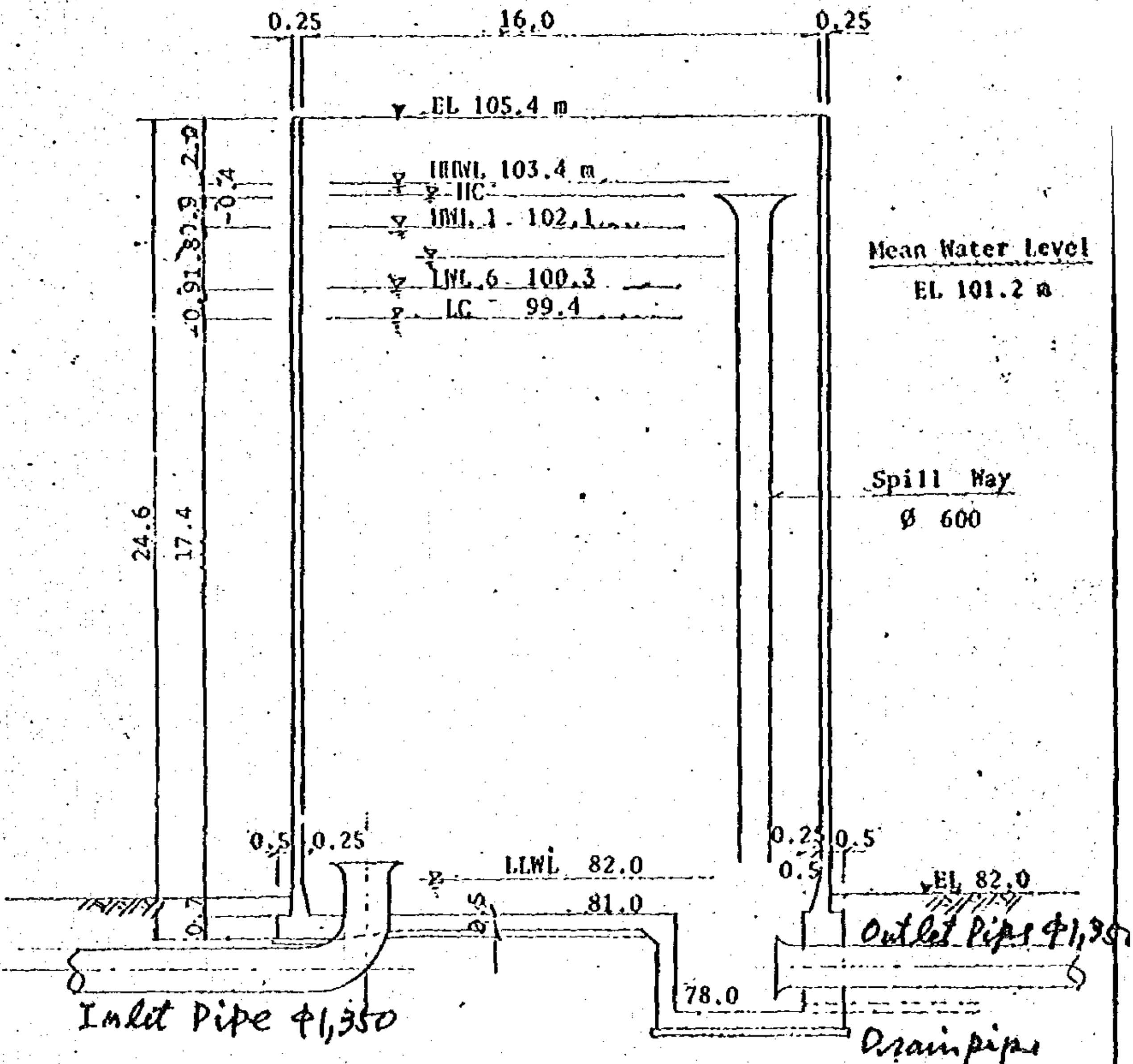


From the bottom upwards :

* The diameter of tank is set as 16 m.
* The level of bottom is set at 81.5 m
* 1.0 m at the bottom is for the sand deposit
* The capacity 3,498 m^3 or 17.4 m depth satisfies the required capacity 3,144 m^3 or 15.7 m depth for filling the pipeline.
* For alarm allowance below LWL 1', 0.15m depth is prepared.
* 3.3 m depth is required for controling the pumps.
* For alarm allowance upper HWL 6, 0.15 m depth is prepared.
* 0.4 m at the top is for the overflow.

The alarms of high and low water levels are set in the operators room of the pump station. The control levels are connected to the pumps programmer. From the overflow water level to the edge of tank, 2.0 m clearance is given.

Fig. 11 Dimensions of Head Tank



HHWL	Overflow Water Level
HC	High Water Alarm Level
HWL 1-6	High Water Control Level of each pump
LWL 1-6	Low Water Control Level of each pump
LC	Low Water Alarm Level
LLWL	Lowest Low Water Level

6.2. Dimensions

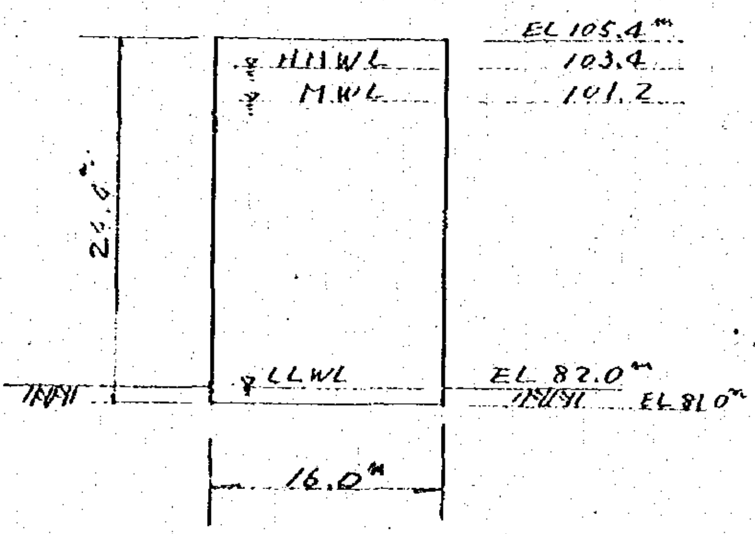
Fig. 11 shows the finalized dimensions and levels of the head tank.

7. COMPARATIVE STUDY ON STURCTURE

7.1. Required dimensions

The Required dimensions for the head tank finalized so far are shown in Fig. 12.

Fig. 12



7.2. Structures to be Compared

As the result of the Feasibility Study Report, prestressed concrete type have been selected for the tank. Now in this study it will be compared with one more type which is re-inforced concrete again. Because PC and RC were cheapest two and a steel tank was highest.

The structure of each type is shown in Fig.13,14.

Fig. 13 RC Type

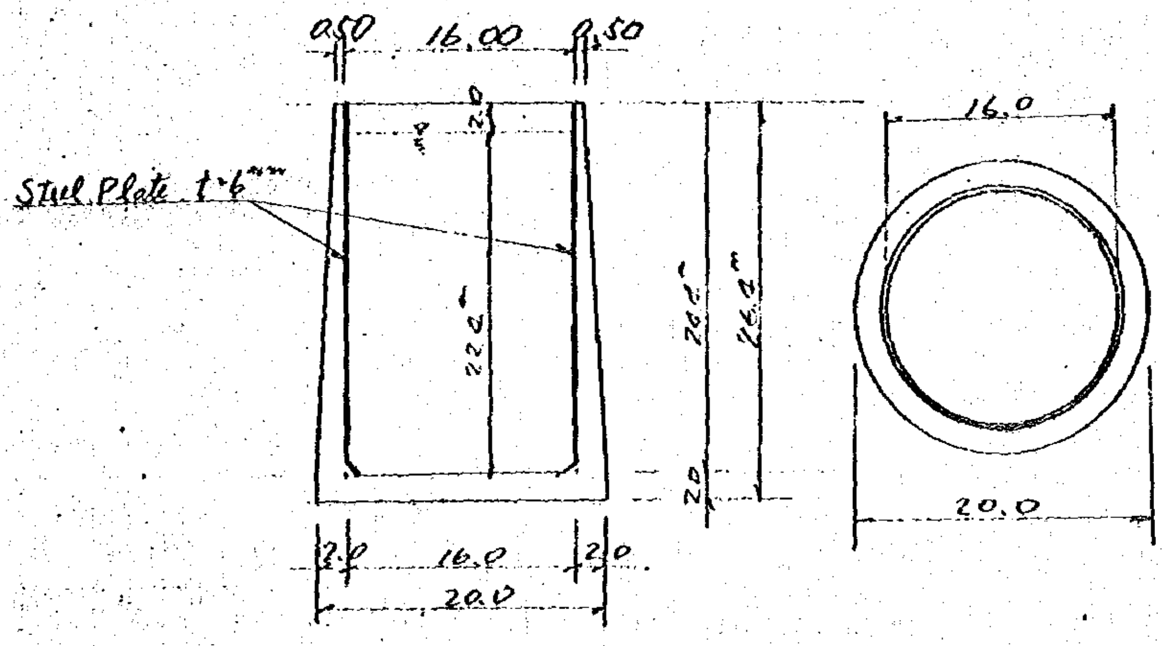
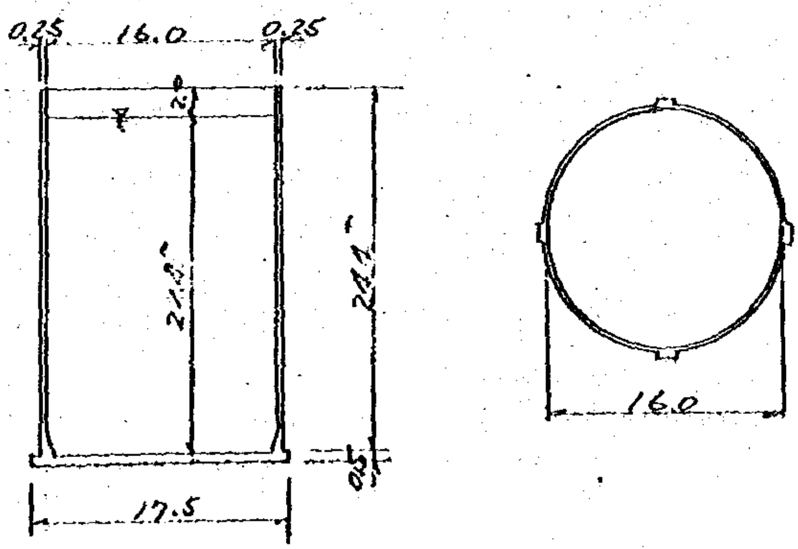


Fig. 14 PC Type



Wall and baseplate thickness of each types are shown in Fig.13, 14 after rough calculation wyich is not reported here and steel plate of 6 mm thickness will be set inside of the RC type tank to prevent leekage of water through tension cracks of wall concrete. RC tank with steel plate inside should be installed 2 tanks for the reason why the steel plate must be painted each some years and during one tank is painted the other can use and the pipeline system is able to operate without suspension.

7.4. Economical Comparison

7.4.1. Reinforced Concrete (RC) type

Table 2 Cost of RC Type

Item	Quantity	Unit Price	Cost (10^3US$)
Concrete	2,353 m^3	260 US$/$m^3$(*)	612
Steel Plate	58 t	1,300 US$/t(*)	75
Painting	4,900 m^2(*)	20 US$/$m^2$	98
Sub-total			785
Total	785 10^3US$ x 2 tanks	= 1,570 10^3US$	

* Including reinforcement bar and forming price

* Including welding price

* Including 3 times repainting

7.4.2. Prestressed Concrete (PC) Type

Item	Quantity	Unit Price	Cost (10^3US$)
Tank Volume	4,900 m^3	150 US$/$m^3$(*)	735

* Including concrete, PC wire and forming etc.

7.5. Conclusion

Cost of both type are shown below;

RC type head tank	1,570	10^3US$
PC type head tank	735	

Apparently PC type has economical advantage and for RC type, tension crack of concrete by static water pressure is inevitable and it reduce tank's life-span though steel plate is attached inside.
From these reasons mentioned above PC type is preferable for the head tank.

8. CHANGE OF LOCATION

Feasibility Study proposed the location of head tank at the top of a hill, about 1 km apart from Route 3191.

The detailed design team, after examining every condition which have been cleared after F.S., has decided to change the location to the road side, thus shortening the pipeline length by 2 km and making construction of the tank easier and cheaper.
Also, the running cost will be remarkably saved due to the decreased head of pumps.

The saving will amount by about 2 million US dollars.

9. NUMBER OF THE HEAD TANK

Number of the head tnak has decided to be one at the meeting held between JICA DD Team and RID on 26th February 1982.

10. SPILLWAY

10-1. PURPOSB

The maximum overflow rate from the head tank, 2.62 m^3/s, has been decided in the section 4, main dimention of the head tank, so a spillway have to be designed to allow the overflow water to discharge safely.

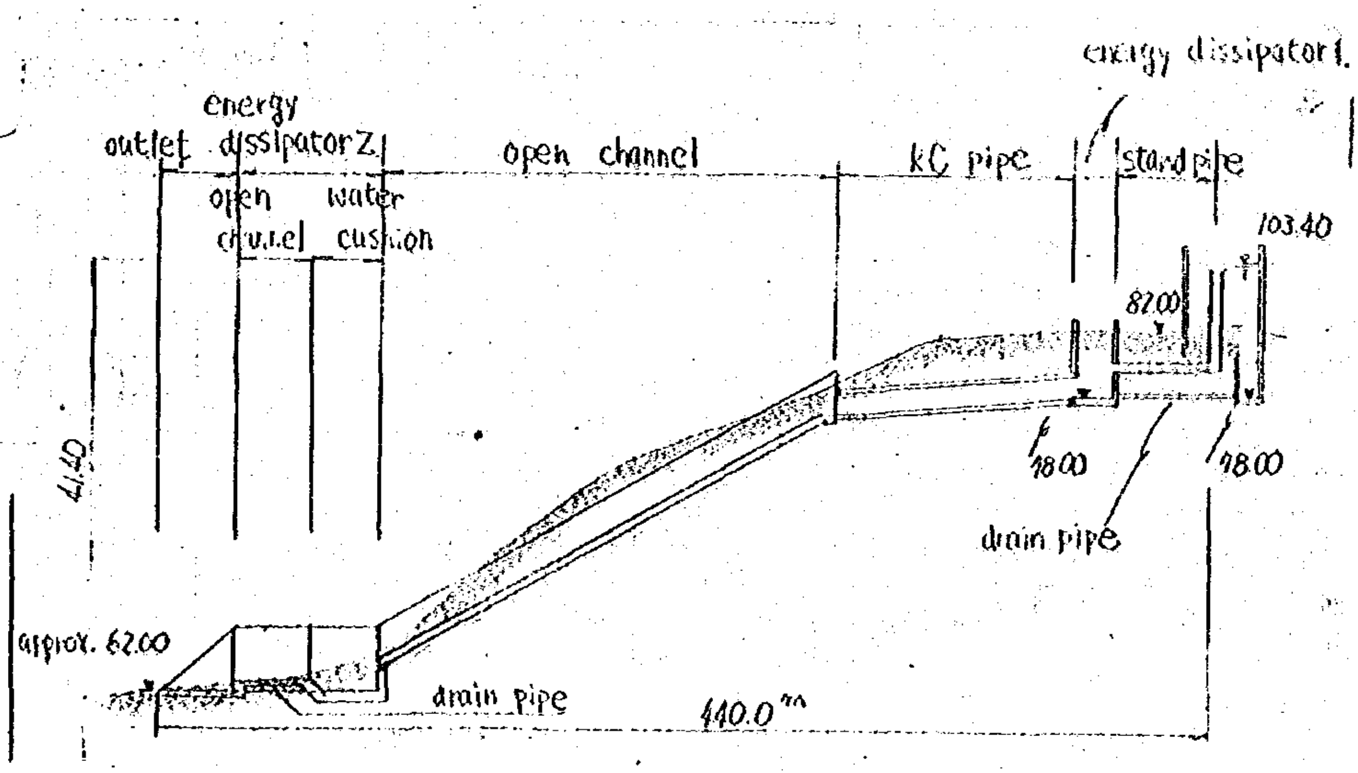
10-2. LOCATION

The overflow is finally discharged into the creek, which the pipeline crosses about 440 m away towards Dok Krai Dam from the head tank. The spillway connects the head tank and the creek, running along the highway No. 3191. The center line of the spillway is 2.5 m out of the right of way which is parallel to the national road. Please refer to the Plan H-1.

10-3. FACILITIES

The general concept of the facilities needed for the spillway is summarized in Fig. 14.

Fig. 14 Spillway Facilities



In the following section, hydraulic designs are made for the facilities; stand pipe, energy dissipator 1, channel (RC pipe & open channel), energy dissipator 2 (water cushion & open channel), and an outlet.

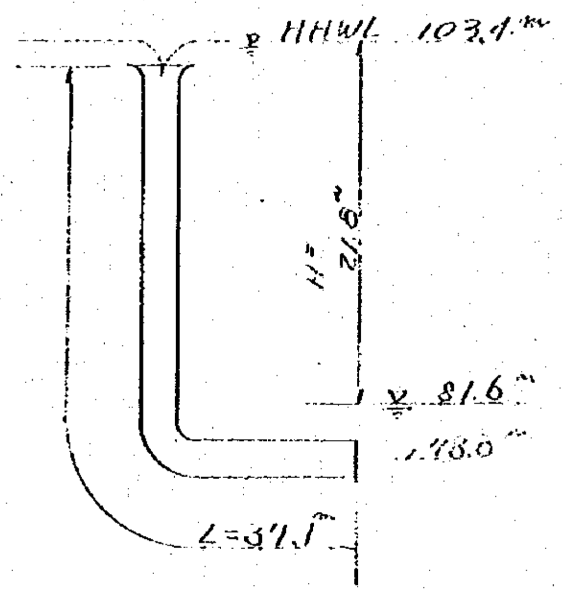
10-4. HYDRAULIC DESIGN

The profile of the whole spillway system proposed by the study described below is shown in Fig. 15.

10-4-1. Energy Dissipator 1

(1) Stand pipe diameter

Fig. 16 Hydraulic Model for Stand Pipe



The diameter of the stand pipe for spill way is calculated as below with the condition of the design flow rate 2.62 m^3/sec.

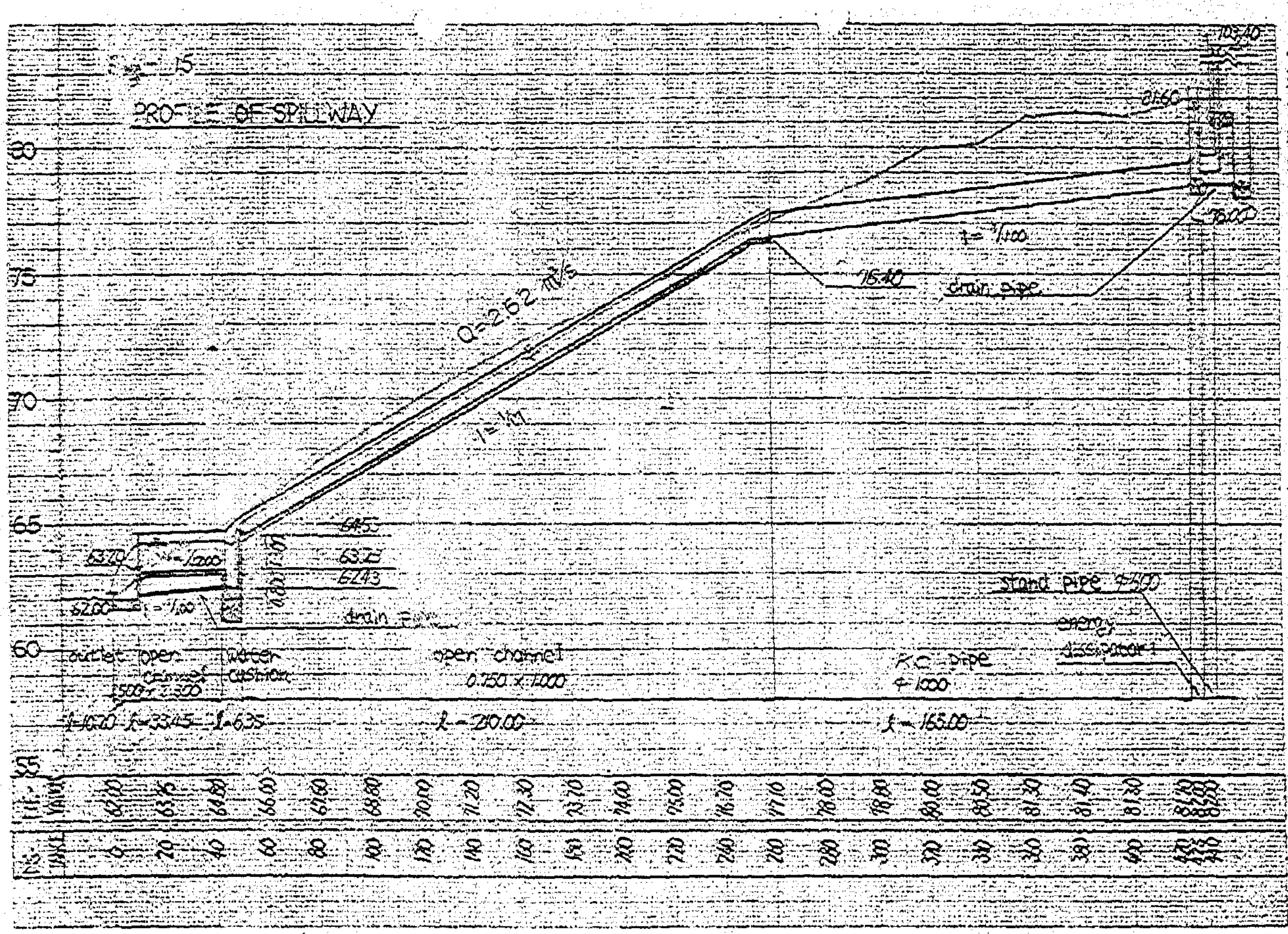
$$H = hf + he + hb + hd \quad \text{............(1)}$$

$$hf = \frac{n^2 \cdot L \cdot V^2}{R^{4/3}} \quad \text{...................(2)}$$

$$he = fe \cdot \frac{V^2}{2g} \quad \text{......................(3)}$$

$$hb = fb \cdot \frac{V^2}{2g} \quad \text{......................(4)}$$

$$hd = fd \cdot \frac{V^2}{2g} \quad \text{......................(5)}$$



PROFILE OF SPILLWAY
80
75
70
65
60
55
81.60
76.00
$i = 1/100$
76.40
drain pipe
Q=2.62 m³/s
$i = 1/1.7$
64.50
63.23
62.43
63.70
62.00
$i = 1/200$
$i = 1/100$
drain
stand pipe φ300
energy dissipator
outlet
open channel 1.500×1.300
water cushion
open channel 0.750×1.000
R.C. pipe φ1000
$l = 10.70$
$l = 33.45$
$l = 6.35$
$l = 210.00$
$l = 165.00$
62.00
63.75
64.80
66.00
67.60
68.80
70.00
71.20
72.30
73.70
74.00
75.00
76.70
77.10
78.00
78.00
78.80
80.00
80.50
81.30
81.40
81.80
81.70
82.00
82.00
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
423
440

H : Total head (m)

hf : Friction loss head (m)

R : Hydraulic mean depth (m)

n : Coefficient of roughness = 0.014

V : Flow velocity (m/sec)

he : Entrance loss head (m)

fe : Coefficient of entrance loss = 0.1

hb : Bend loss head (m)

fb : Coefficient of bend loss = 0.1

From the formula (1) to (5) ;

$$V = \left(\frac{2gH}{\frac{2g.n^2.L}{R^{4/3}} + fe + fb + fd} \right)^{1/2}$$

Table 3. Calculation of discharge for each diameter

D (m)	$R=\frac{D}{4}$ (m)	$R^{4/3}$	$\frac{2gn^2L}{R^{4/3}}$	V (m/sec)	A (m^2)	Q (m^3/sec)
0.5	0.125	0.0625	2.15	11.3	0.196	2.22
0.6	0.150	0.0797	1.76	12.0	0.283	3.39
0.7	0.175	0.0979	1.43	12.8	0.385	4.93

From Table 3, a pipe of 0.6 m diameter will satisfy the design flow rate 2.62 m^3/sec.

(2) Energy dissipator 1

The general plan of the energy dissipator 1 is shown in Fig.16. This type of energy dissipator is effective to reduce the energy of the flow flashing out of a pipeline and is generally used when flow velocity is more than 10 m/s with discharge less than 10 m^3/s. The flow velocity in the proposed stand pipe is approximately 12 m/s $>$ 10 m/s and the discharge is 2.62 $m^3/s < 10$ m^3/s. The merit is that this type of dissipator occupies less ground area than other types. The dimensions of each part of this structure was decided refering to Table 4.

Fig.17 Hydraulic Model for Energy Dissipator 1

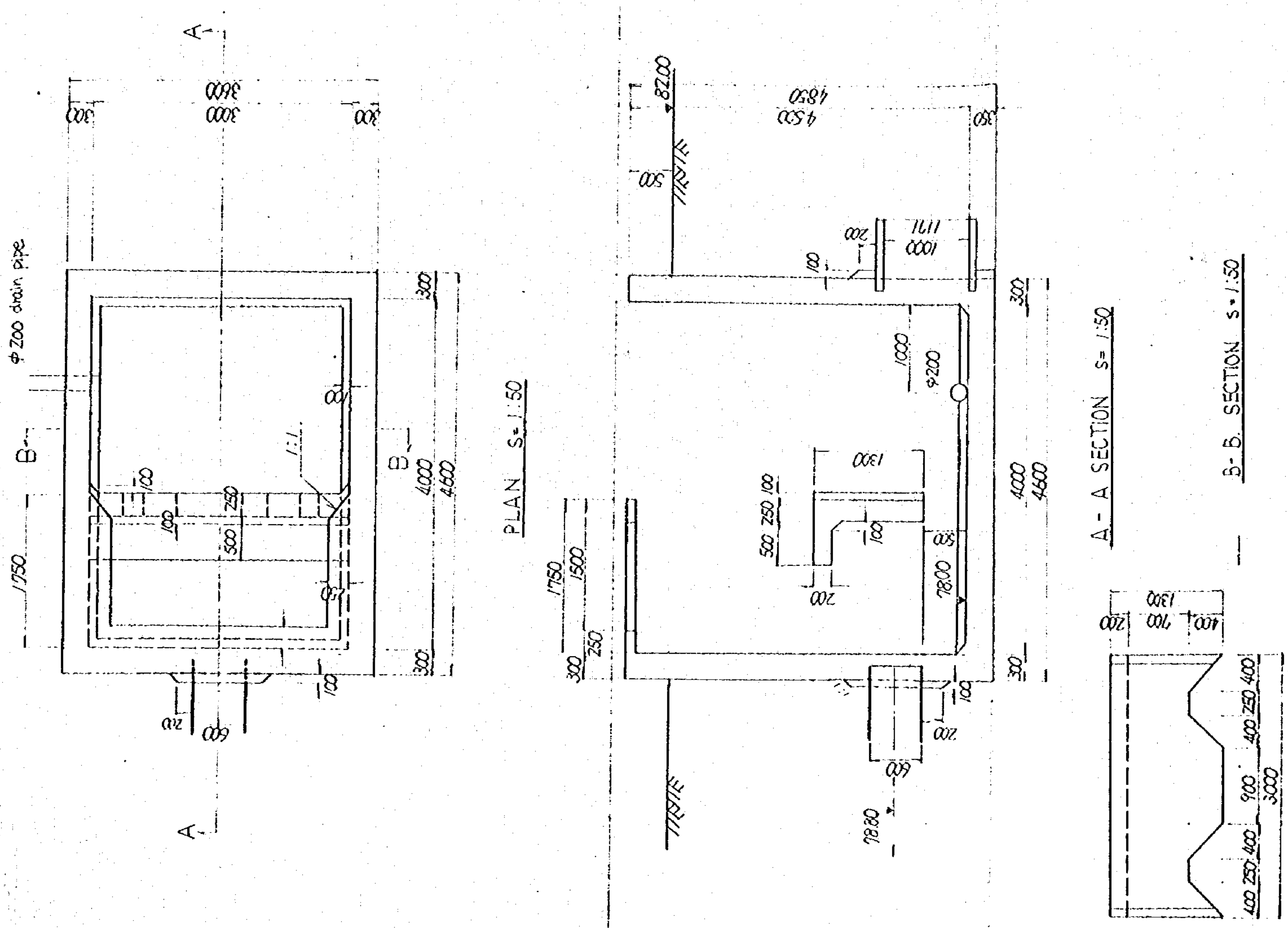


Fig 16 Energy Dissipater

Fig.17 Hydraulic Model for Energy Dissipator 1

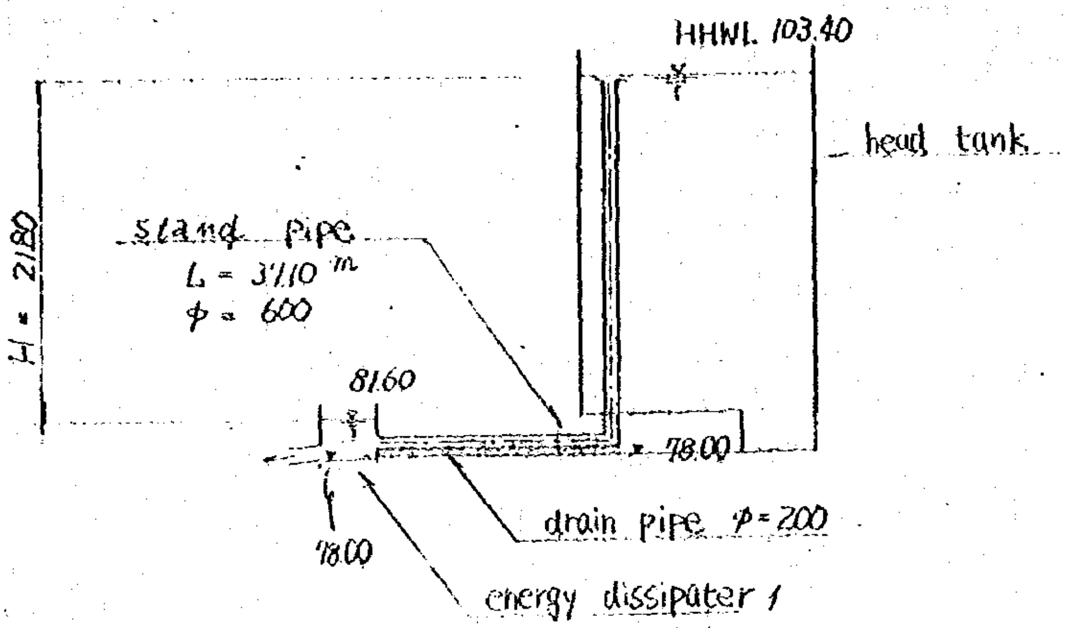
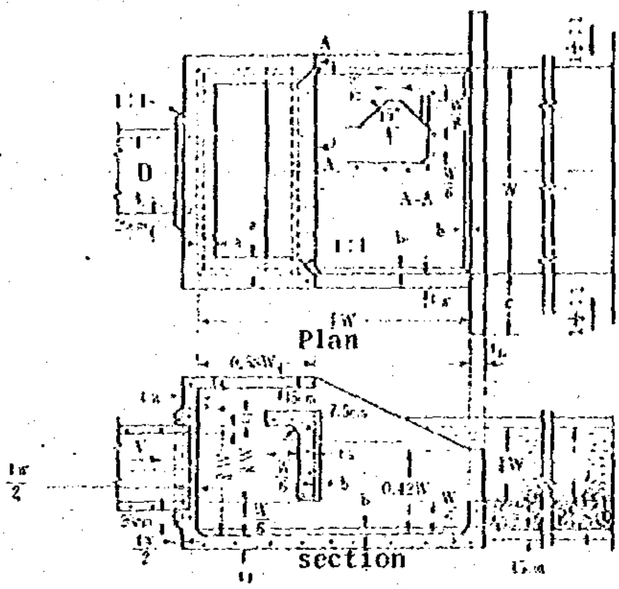


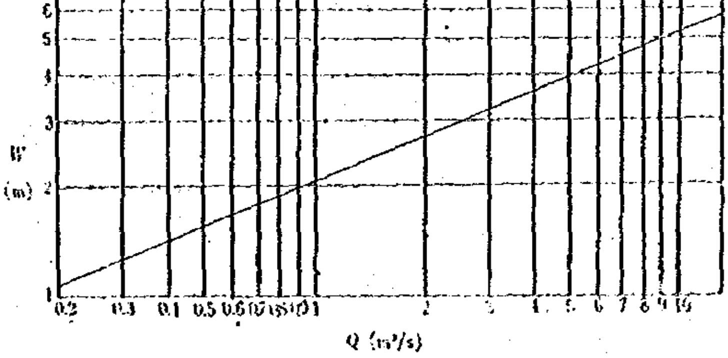
Table 4 Dimensions of Energy Dissipator 1



Concrete thickness (cm)

Q (m³/s)	a	b	c	t_a	t_c	t_s	t_p
2.8	23	6	30	20	20	23	20
5.7	[illegible]	10	[illegible]	25	[illegible]	25	20
8.5	35	15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11.3	[illegible]	15	[illegible]	[illegible]	31	[illegible]	20

Width of the Structure & Q.

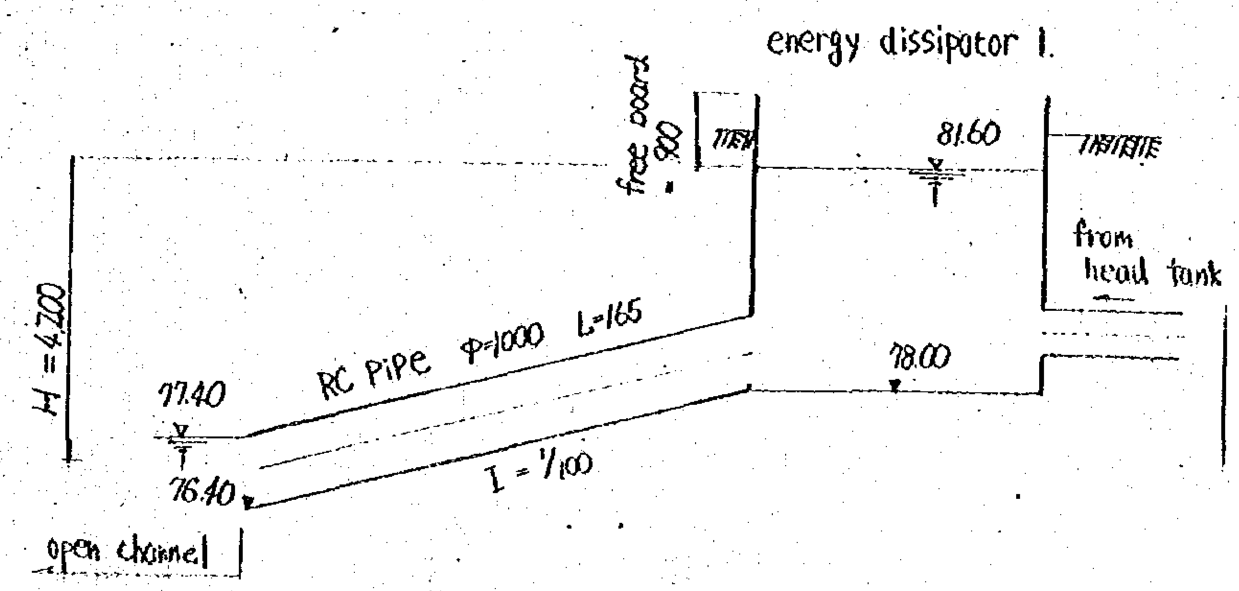


10-4-2. Channel

(1) RC Pipe

The energy dissipator 1 and the open channel are connected with a RC pipe (D = 1,000 mm L = 165 m) buried underground. This is given I of 1/100 to wash out scourings inside the RC pipe.

Fig. 18 Hydraulic Model for Rocla Pipe



Condition:

n: Manning's coefficient of roughness = 0.017

l: Length of the pipe = 165 m

Elevation of water surface at the outlet of RC pipe = eleve 77.40
(This is decided in the following study (2) Open Channel)

hydraulic computation for the decided section

V: $\sqrt{\frac{2gH}{\Sigma f}}$ = 3.33m/S

Q: A x V = 2.63 m^3/S 2.62 m^3/S

A: Section area $D^2\pi$ = 0.79 m^2

H: total head = 4.20^m

Σf: total coefficient of head loss = fe + $f\frac{L}{D}$ + fo = 7.44

f: coefficient of friction loss = $\frac{124.5n^2}{D^{1/3}}$ = 0.036

fe: coefficient of entrance loss = 0.5

fo: coefficient of out loss = 1.0

D: diameter of the pipe = 1.00 m

The diameter was decided to be 1,000 mm according to the D-H table shown below. The table means that the discharge of 2.62 m^3/S flows down through the pipe of a diameter (D^{mm}) when its corresponding water head (H^m) is given.

Table 5 D-H table

Q = 2.62 m^3/s

D^{mm}	H^m
600	57.92
800	13.16
1,000	4.23
1,200	1.69

$$H = \Sigma f \frac{V^2}{2g} \text{ (m)}$$

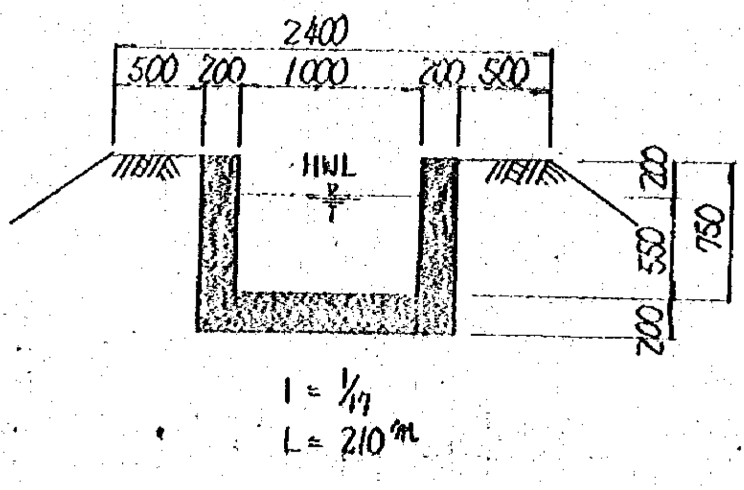
$$V = \frac{2.62}{A} \text{ (m/s)}$$

$$A = \frac{\pi \times D^2}{4} \text{ (m}^2\text{)}$$

(2) Open Channel

A rectangle section with reinforced concrete is adopted to occupy less ground area and to be safe enough against the super-critical flow dew to steep gradient.

Fig.19 Section of Open Channel S=1:40



Condition:

n = 0.020

I: Channel floor gradient = $^1/_{17}$

Q = 2.62 m^3/s

Hydraulic computation for the decided section

V: flow velocity $= \frac{1}{n} \times R^{2/3} \times I^{1/2} = 4.97$ m/s

Q: discharge = AxV = 2.73 m^3/s > 2.62 m^3/s

F: Froud number $= \frac{V}{\sqrt{gh}} = 2.147 > 1.00$ (super-critical flow)

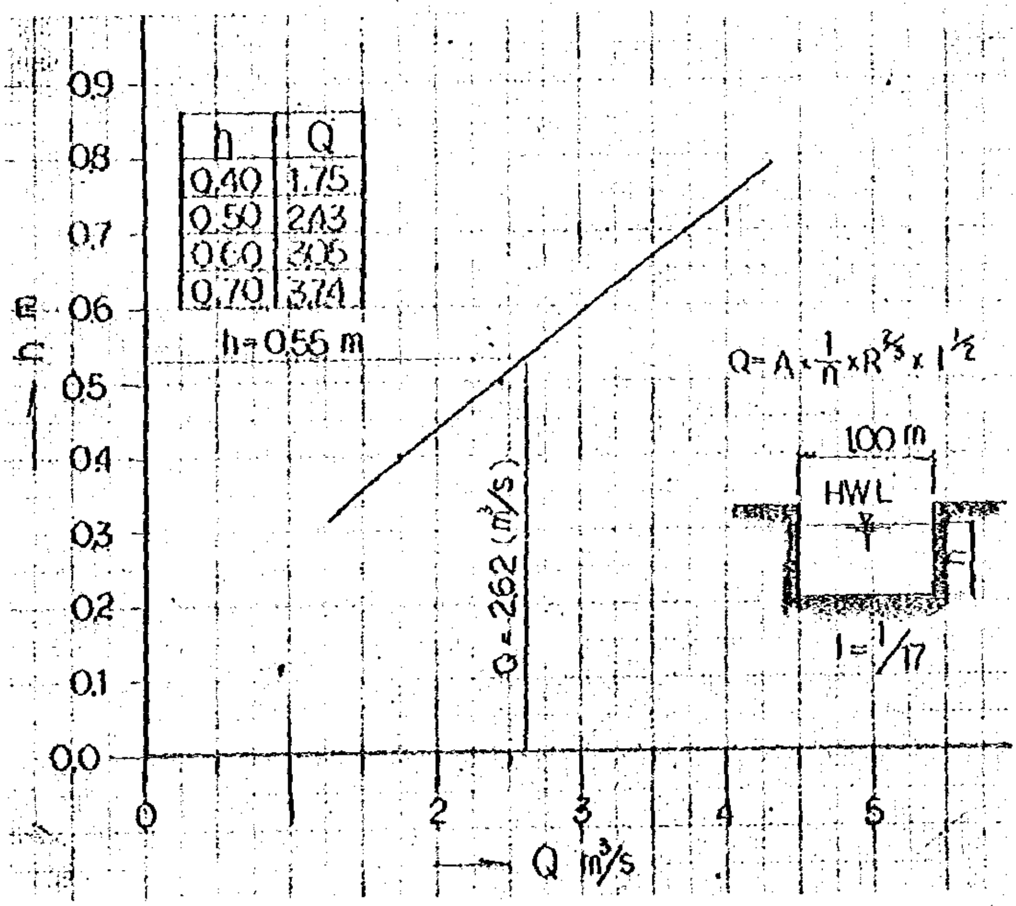
A: section area = 0.55 m^2

S: wetted perimeter = 2.10 m

R: hydraulic mean depth = 0.26 m

The section was decided according to the rating curve shown in Fig.20.

Fig.20 Rating Curve of Open Channel

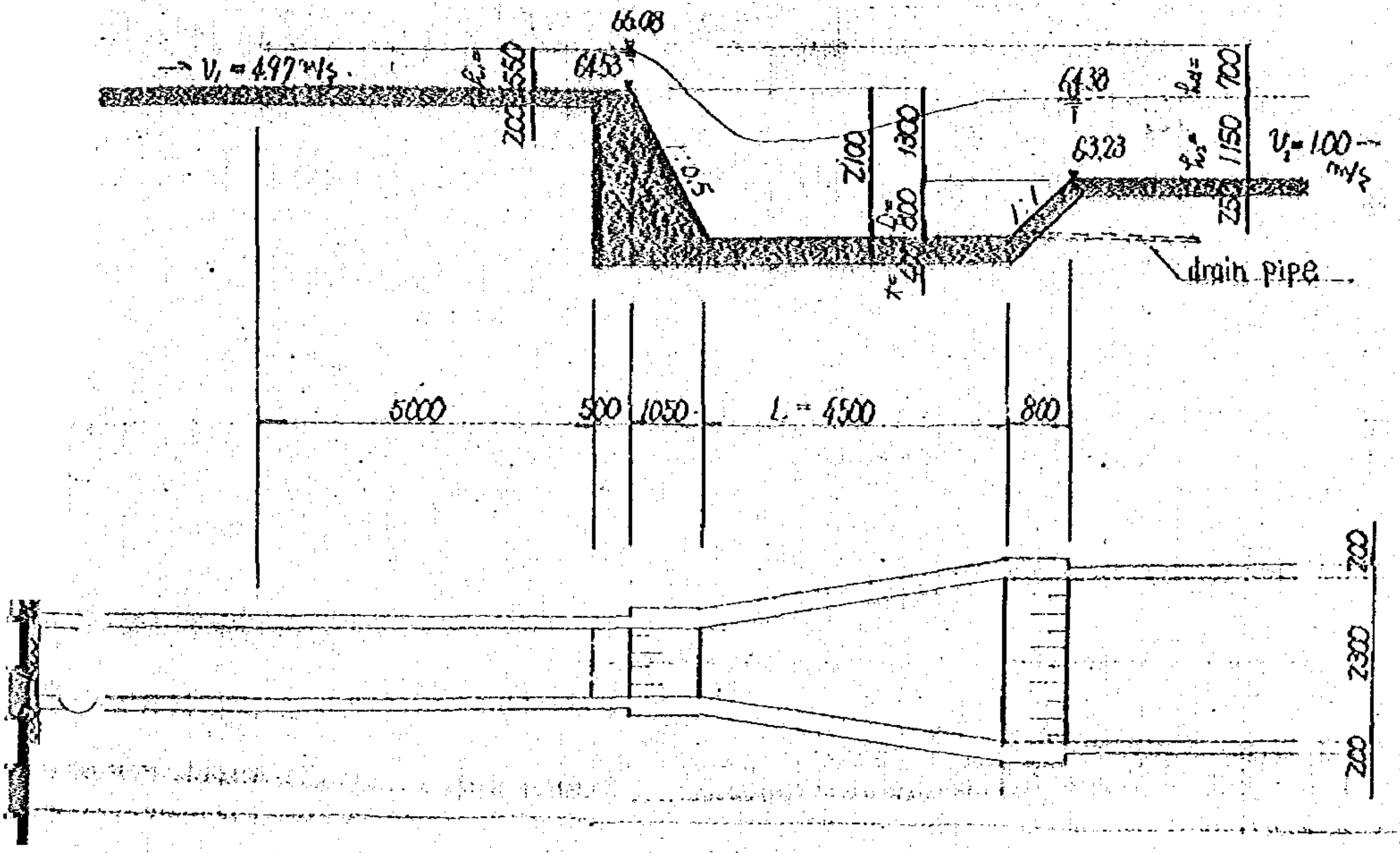


10-4-3. Energy Dissipator 2

(1) Water Cushion

In order to decrease the energy held by the super-critical flow coming down through the open channel with 1/17 of gradient, water cushion is set up as an energy dissipator.

Fig.21 Water Cushion S=1:10



Condition:

V: flow velocity of upper channel = 4.97 m/s

h: water depth of upper channel = 0.55 m

elevation of the floor at the end of the upper channel 64.53 m

V: flow velocity of lower channel = 1.00 m/s

h: water depth of lower channel = 1.15^{m}

elevation of the floor at the end of the lower channel = 63.23^{m}

(V_2 x h_2 were decided in the following study, (2) Open Channel)

1) Depth of water cushion (D)

$D = (^1/2.4) \times F$ (m)

$F = H_1 - H_2 = 1.91$m

H_1: total head of upper channel $= 64.53 + h_1 + \frac{V_1^2}{2g}$ = Eleve.66.34m

H_2: total head of lower channel $= 63.23 + h_2 + \frac{V_2^2}{2g}$ = Eleve.64.43m

$D = 0.80^m$

2) Length of water cushion (L)

$L = 2\sqrt{\alpha HF}$ (m)

α : coefficient of water nap ≒ 1.33

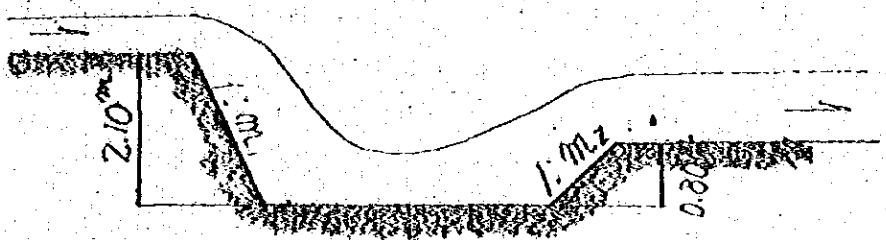
H : specific energy of upper channel $= h_1 + \frac{V_1^2}{2g} = 1.81^2$m

L : 4.50^m

3) Floor thickness (t)

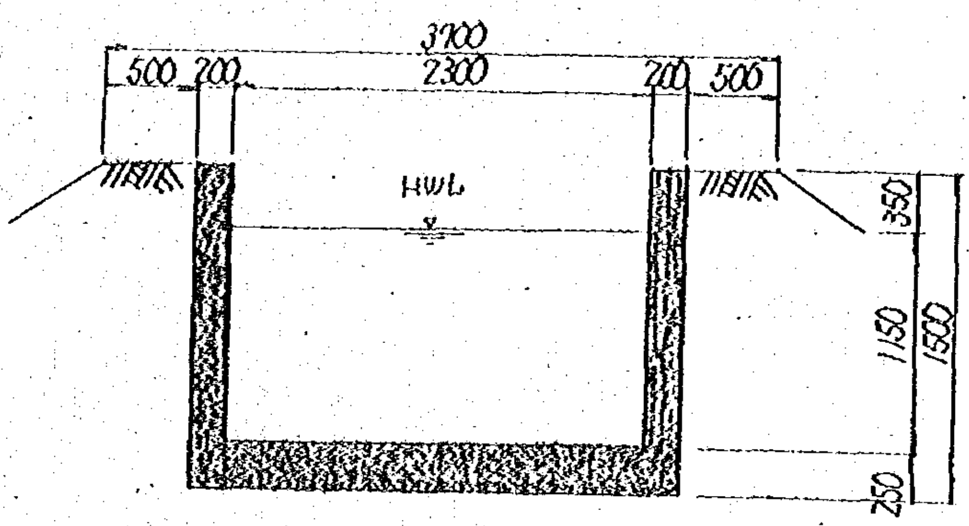
The floor thickness is decided to be 40 cm so that it is firm enough against impact of water and internal stress caused by earth pressure.

4) Slope (m_1 & m_2)



Slope shown above, m_1 & m_2, were decided depending on their height. Generally, slope is 1:0.5 when its height is 2-3 m and it is 1:1 when the height is less than 2 m. So, m_1 is 0.5 and m_2 is 1.0.

Fig.22 Open Channel for Energy Dissipator 2 S = 1:40



I = 1/1200
L = 3345m

Condition:

$n = 0.020$

I : channel floor gradient = $^1/_{1200}$

$Q = 2.62 m^3/s$

hydraulic computation for the decided section :

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times I^{\frac{1}{2}} = 1.00 \ m/s$$

$$Q = A \times V = 2.65 \quad 2.62 m^3/s$$

$$F = \frac{V}{\sqrt{gh}} = 0.30 < 1.00 \text{ (sub-critical flow)}$$

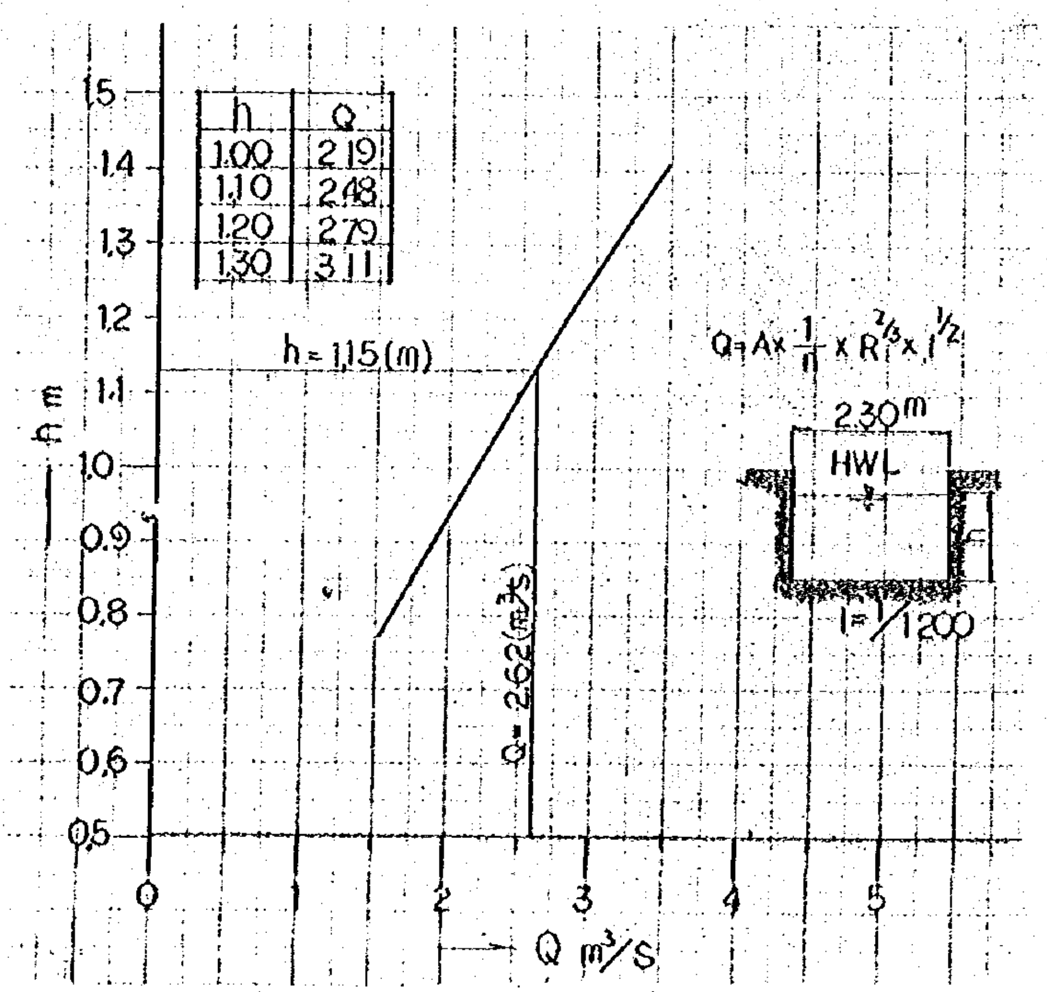
$A = 2.65 \ m^2$

$S = 4.60 \ m$

$R = 0.58 \ m$

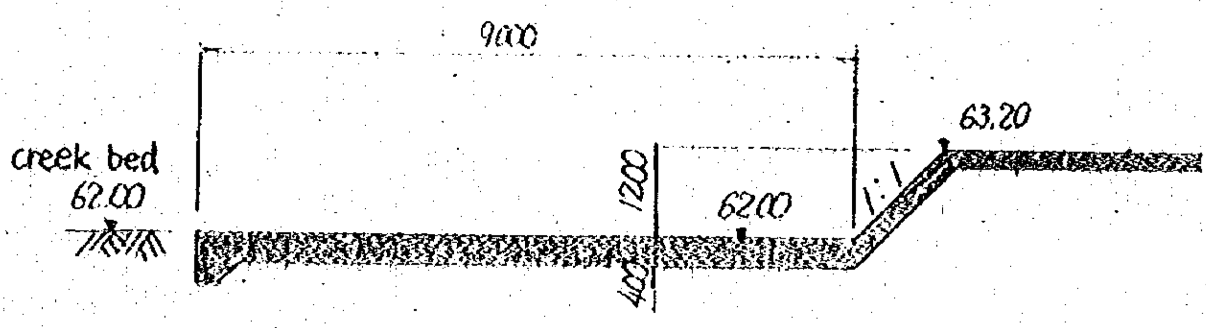
The section was decided according to the rating curve shown in Fig. 23

Fig.23 Rating Curve of Open Channel for Energy Dissipator 2.



10-4-4. OUTLET

At the outlet of this spillway system,a drop is set up to adjust the difference of elevation between the creek bed and the open channel floor.



NO.9 COMPARATIVE STUDY OF PIPELINE SYSTEM DOK KRAI - MAB TA PUD - SATTAHIP

January 1982

Prepared by YOSHIYUKI TOMIOKA
AKIHIRO MIYAKE
Detailed Design Team, JICA

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2. Conditions, Given and Assumed

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2.2. Assumed Conditions

3. Parameters for Comparative Study

4. Hydraulic Conditions by Calculation

5. Cost Estimate

5.1. Conditions
5.2. Discussion of Cost

6. Field Reconnaissance

7. Conclusion

1. Introduction

The diameter of pipeline between Dok Krai and Mab Ta Pud was decided at the conference held at the beginning of November 1981. Consequently, 1.5 m in the Feasibility Study was changed to 1.35 m for the Detailed Design. The reason for the change was due to the modified supply capacity of 57.8 MCM/yr, resulting from the review of demand.

Though the detailed design work is undertaken now for Dok Krai - Mab Ta Pud, its extention to Sattahip in an early future is very certain.

When Mab Ta Pud - Sattahip line is completed, the whole system of Dok Krai - Mab Ta Pud - Sattahip shall be operated as a whole.

Considering the above situation, the detailed design team has worked a comparative study to see;

(1) If the decision of 1.35 m diameter for Dok Krai - Mab Ta Pud was reasonable.

(2) What size will be most economical for the Mab Ta Pud - Sattahip extention.

(3) How the selection of the elevation of receiving well will affect the whole pipeline system.

2. Conditions, Given and Assumed

2.1. Given Conditions

(1) Flow Rate, Water Demand

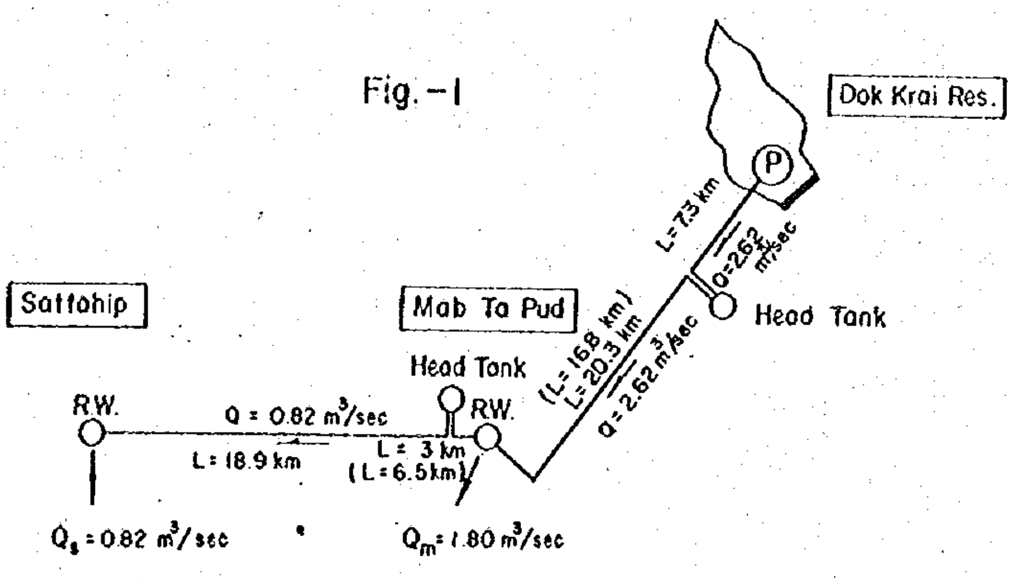
	Flow Rate(m^3/sec)	Water Demand(MCM/yr)
Dok Krai & Mab Ta Pud	# 2.62	# 57.8
Mab Ta Pud & Sattahip	0.82	* 18.1

agreed figure, * Feasibility Study,Table 4-5,1992-1983 value,
demand 0.82 = 2.62 x 18.1/57.8

(2) Pipeline System

As shown below, the pipeline system includes pumps, head tanks, receiving wells and pipes. The pipeline length with and without the bracket corresponds to Model II and Model I which will be explained later. The difference comes from the location of Receiving Well at Mab Ta Pud.

Fig.-I



R.W. = Receiving well

(3) Low Water Level at Dok Krai

42.00 is given

2.2. Assumed Conditions

30.00 for the level of receiving well at Sattahip is assumed, considering that the industrial supply be fed by gravity and the municipal by pumping.
The industries will consume about 73% of the supply and they will be in low land of 5 to 10 m above sea level.
The supply by gravity to them will save much energy in the long run.

3. Parameters for Comparative Study

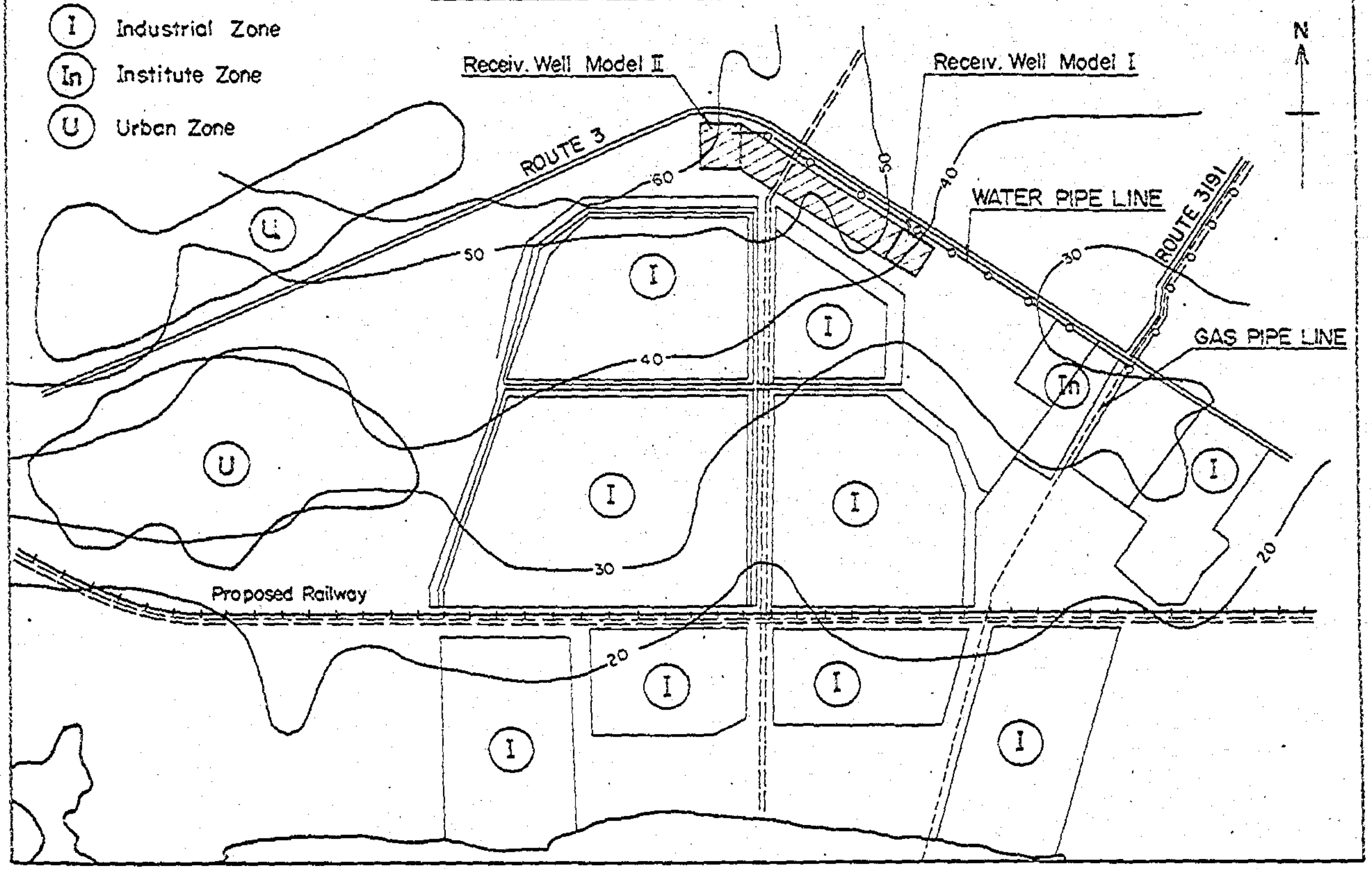
3 parameters, the level of receiving well at Mab Ta Pud, the diameter between Dok Krai and Mab Ta Pud, the same between Mab Ta Pud and Sattahip are the parameters for comparative study.
These parameters are combined to make 18 cases of combination on the next page.
Fig.-2 on the following page to the next shows two locations, meaning two water level and the difference of pipe length in Fig.-1.

<u>18 Cases for Comparison</u>

Water Level of Receiving Well	Case	Pipe Diameter, Dok Krai-Mab Ta Pud	Pipe Diameter Mab Ta Pud-Sattahip
	1	Ø 1,200^{mm}	Ø 800
	2	"	Ø 900
Model I	3	"	Ø 1,000
EL 45^{m}	4	Ø 1,350	Ø 800
	5	"	Ø 900
	6	"	Ø 1,000
	7*	D → H → M Ø 1,500 Ø 1,350	Ø 800
	8	" "	Ø 900
	9	" "	Ø 1,000
	10	Ø 1,200	Ø 800
	11	"	Ø 900
	12	"	Ø 1,000
Model 2	13	Ø 1,350	Ø 800
EL 60^{m}	14	"	Ø 900
	15	"	Ø 1,000
	16	Ø 1,500	Ø 800
	17	"	Ø 900
	18	"	Ø 1,000

* Dok Krai → Head Tank → Mab Ta Pud

Fig. - 2 LOCATION OF RECEIVING WELL



4. Hydraulic Conditions by Calculation

Calculation was made by using Hazen-Williams formula as shown below;

$$I = 10.666 \times C^{-1.85} \times D^{-4.87} \times Q^{1.85}$$

I : hydraulic gradient

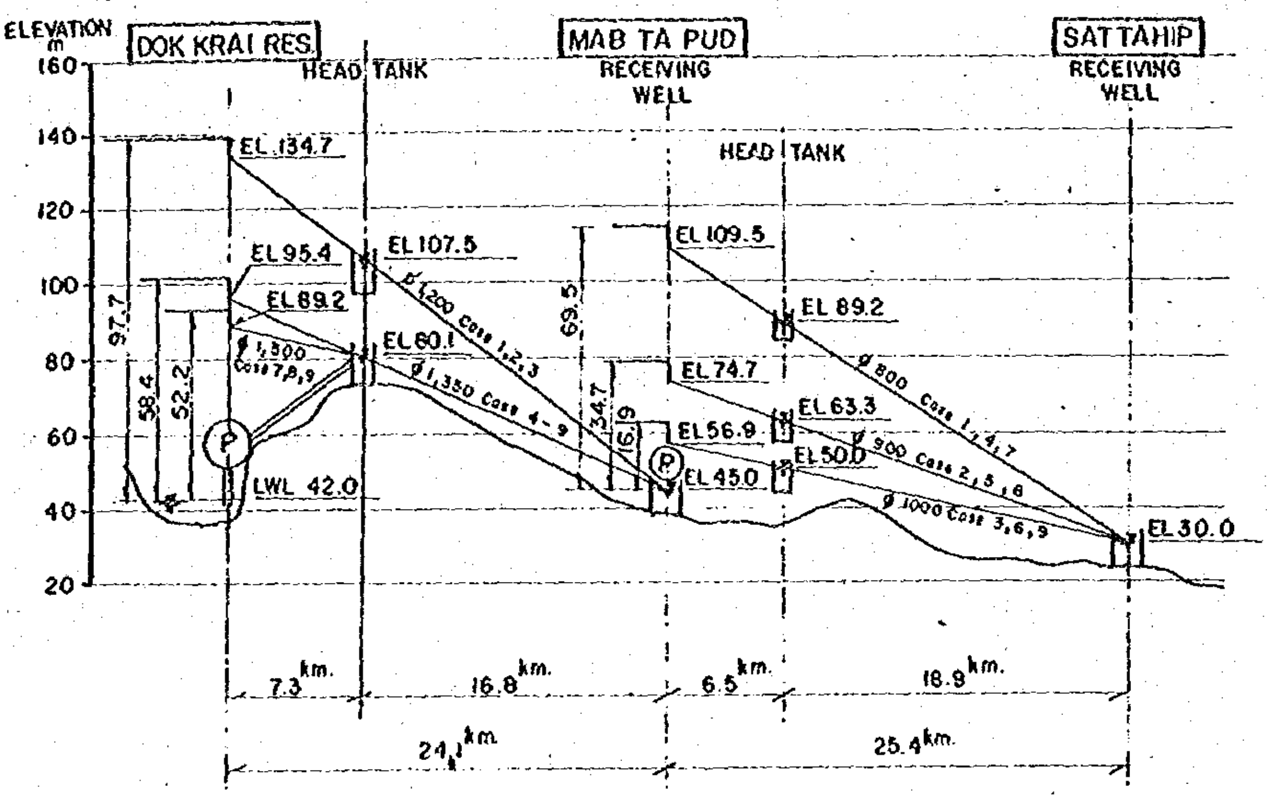
C : coefficient of roughness = 120

D : pipe diameter, m

Q : flow rate, M^3/sec

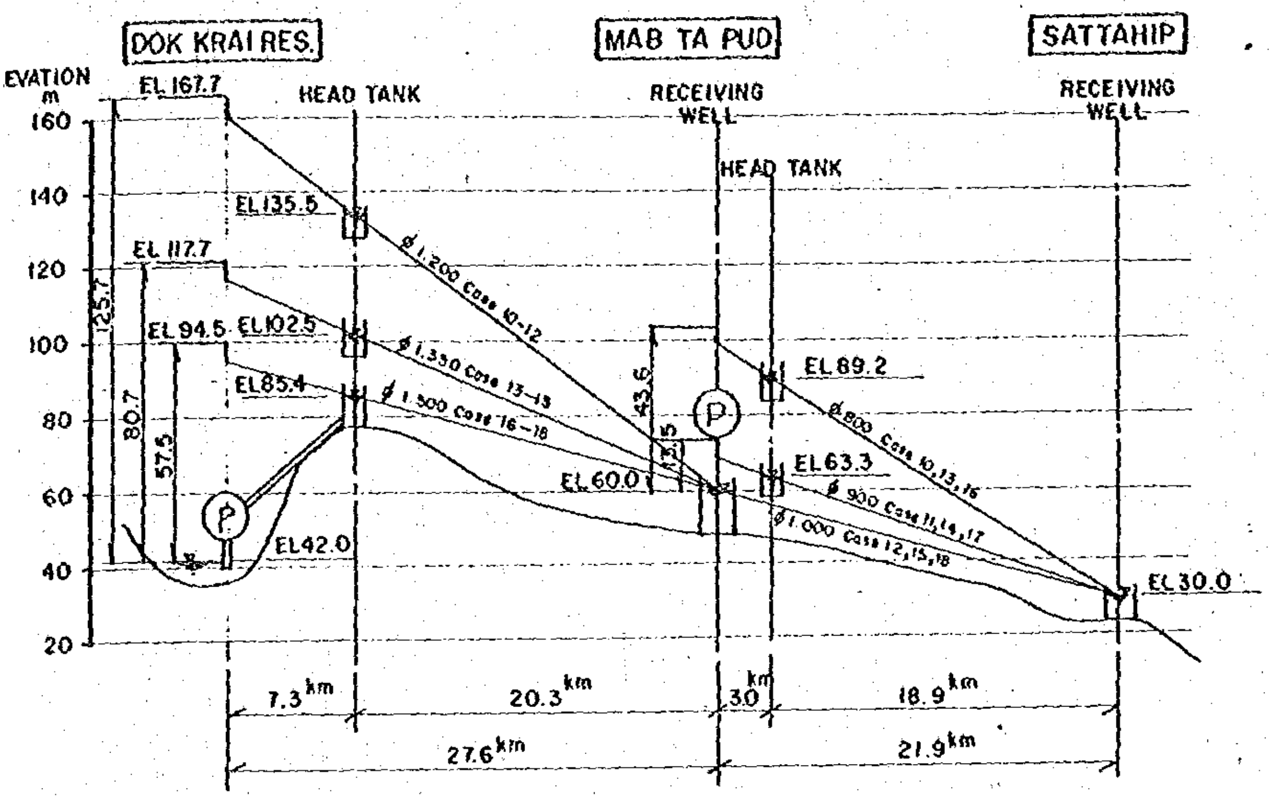
The results are shown graphically on Fig-3 & -4.

Fig.-3 Model I



* marginal loss head at pump is 5.0 m.

Fig.-4 . Model II



* marginal loss head at pump is 5.0 m.

5. Cost Estimate

5.1. Conditions

The cost estimate was made under the following conditions,

(1) The construction cost includes the pipeline, pumps and motors, electric facilities, pumping station, head tanks and the filtration plant, and excludes the intake tower and pipe bridge.

(2) The running cost is the electricity for 3 pumping stations, Dok Krai, Mab Ta Pud and the potable supply in Mab Ta Pud area.
The unit cost of electricity is assumed as 51.7 US$ per 1000 KWH or 1.17 ฿ per KWH.

(3) The maintenance cost is assumed individually for each case on yearly basis.

(4) The construction cost is discounted from 1983 to 2007 at 10% rate and is converted to the Net Present value.

(5) The running cost and the maintenance cost, calculated as a constant figure on yearly basis, are also converted to the Net Present Value.
Here, the running cost is calculated assuming as if the pipeline will work fully from the beginning of operation.

(6) The Net Present Value of construction and maintenance is named the initial cost. The initial cost and the running cost for 18 cases are tabulated on the next page.

(7) The filtration plant occupies about 40% of the initial cost and 5 to 10% of the running cost.

Table of Cost Comparison

Model	Case	Pipe Diameter D - M	Pipe Diameter M - S	Initial Cost 10^6 US$	Running Cost 10^6 US$	Total Cost 10^6 US$	Ranking
Model I EL 45^m	1	Ø1,200	Ø 800	41.6	6.0	47.6	4
	2	"	Ø 900	43.0	5.5	48.5	10
	3	"	Ø1,000	44.9	5.3	50.2	17
	4	Ø1,350	Ø 800	43.1	4.3	47.4	1
	5	"	Ø 900	44.5	3.9	48.4	7
	6	"	Ø1,000	46.4	3.6	50.0	16
	7	Ø1,500 Ø1,350	Ø 800	44.1	4.1	48.2	5
	8	"	Ø 900	45.5	3.6	49.1	13
	9	"	Ø1,000	47.4	3.4	50.8	18
Model II EL 60^m	10	Ø1,200	Ø 800	41.2	6.3	47.5	3
	11	"	Ø 900	42.5	5.9	48.4	7
	12	"	Ø1,000	42.9	5.6	48.5	10
	13	Ø1,350	Ø 800	43.0	4.4	47.4	1
	14	"	Ø 900	44.3	4.0	48.3	6
	15	"	Ø1,000	44.7	3.7	48.4	7
	16	Ø1,500	Ø 800	45.3	3.4	48.7	12
	17	"	Ø 900	46.5	3.0	49.5	14
	18	"	Ø1,500	47.0	2.7	49.7	15

The same amount is ranked as the same.

5.2. Discussion of Cost

(1) Initial and Running

Except cases 1, 2, 3, 10, 11, 12 where the pipe diameter between Dok Krai and Mab Ta Pud is 1.20 m, the running cost is less than 10% of the initial cost. The initial cost is far more influential than the running cost which is assumed for the full flow rate.

(2) Pipe Diameter

To evaluate the pipe diameters' effect on the cost, two table shown on the next page can be prepared. Each table, on different water level in the receiving well, shows the ranking.

The hatched part is for single digit or below 10.

It covers 5 cases for both Ø 1,350 and Ø 800 and moreover the combination Ø 1,350 - Ø 800 ranks 1 in the both tables.

(3) Water Level in Receiving Well

From the table, which of the two levels is better cannot be decided.

Table of Ranking vs. Pipe Diameter (1)

Model I, 45.-0 Water level

M-S \ D-M	Ø 1,200	Ø 1,350	Ø 1,500
Ø 800	4	1	5
Ø 900	10	7	13
Ø1,000	17	16	18

Model II, 60.00 Water level

M-S \ D-M	Ø 1,200	Ø 1,350	Ø 1,500
Ø 800	3	1	12
Ø 900	7	6	14
Ø1,000	10	7	15

6. Field reconnaisance

From the field reconnaisance around the two lots in Fig.-2 for the receiving well and receiving basin, the next matters became clear.

(1) The geological conditions of Model I (EL 45^m) and Model II (EL 60^m) are almost same and the both have enough bearing capacity of ground.

(2) The topographical conditions of Model I and II are different. Near the area of Model II, there is a ditch about 100 m away from the site which have a enough cross section area to divert the spill flow.
On the other hand, no ditch or river exist for the purpose around Model I area.

(3) The surrounding area of Model II lot is wide and better suited for the future extention.

7. Conclusion

(1) Pipe Diameter

Under the conditions given and reasonably assumed, the pipe diameter of 1.35 m between Dok Krai and Mab Ta Pud is better than 1.50 m and 1.20 m.
The choice is proved as right.
For the extention to Sattahip from Mab Ta Pud, 0.80 m shall be recommended as the best, though The Feasibility Study chose 1.0 m.

(2) Sattahip Pipeline

Pumping to Sattahip must be determined now.

(3) Receiving Well and Reservoir

Model II spot on Fig.-2 shall be decided.

NO.10-1 REPORT ON PUMPING SYSTEM AND WATER HAMMER

January 1982

Prepared by FUMIO ENOMOTO
Pumping Engineer
Detailed Design Team
JICA

CONTENT

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2. Definition of "Open Type" and "Closed Type" Pumping System

   2.1. Closed Type

   2.2. Partially Open Type

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3. Tanks for Water Hammer

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6. Conclusion

1. Introduction

In Feasibility Study Report, it is stated that the "open type pumping system" is preferable to the "closed type pumping system" without defining the words.

In Progress Report 1, the detailed design team defines the words and roughly explains that the preference is made on consideration of water hammer.

A report titled "Report on Facilities", in reviewing Feasibility Study Report, discusses the water hammer problem in principle and in actual application of this pipeline.

This report aims to summerize the matters of pumping system and water hammer discussed in the reports mentioned above.

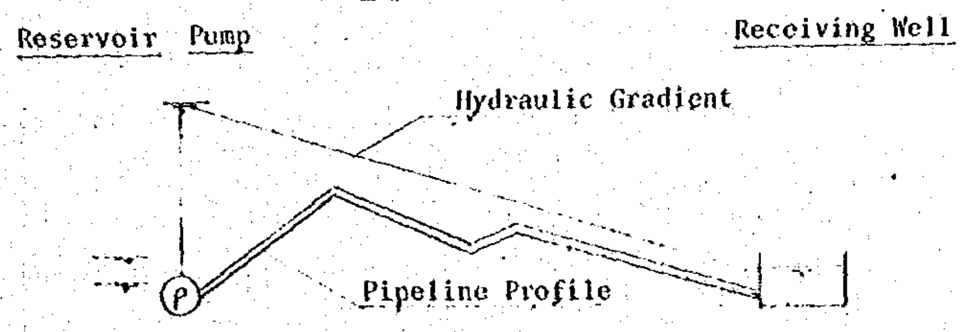
2. Definition of "Open Type" and "Closed Type" Pumping System

"Open Type" and "Closed Type" used in Feasibility Study Report are the technical terms used in classifying the pipeline. Here it will be used in wider meaning including pumping system.

2.1. Closed Type

In this project, the receiving well is set up at the end of pipeline.
The closed type, here, will mean that no free water surface exists on the pipeline between the delivery of pump to the receiving well like shown on Fig.1.

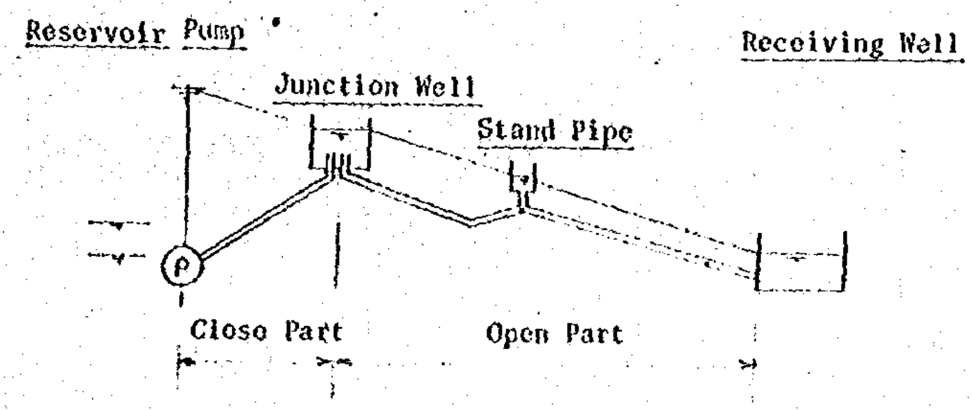
Fig.1 Closed Type



2.2. Partially Open Type

The open type, here, will mean that one or more free surface exists on the pipeline, in the form of stand pipe and/or junction well like shown on Fig.2.

Fig.2 Partially Open Type



Exactly speaking, with Fig.2, the most upstream part is similar to Fig.1 by nature, namely of the close type. However, the downstream part from the junction well downwards is "open".

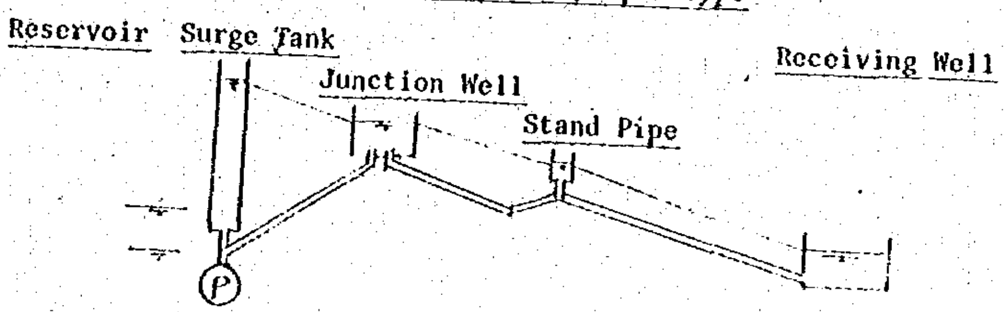
On Fig.2, two different types of tank are pictured, a junction well and a stand pipe. The former is connected to the pipeline with two pipes, the inlet and outlet, while the latter is connected with a T (tee).

For the water hammer effect, the connection between a tank and a pipeline shall be so made that the energy caused by water hammer can be well dissipated in the tank.

2.3. Fully Open Type

As shown on Fig.3, a surge tank can be built at the delivery side of pump. When the tank can absorb the water hammer energy, the whole pipeline is freed from the adverse effect.

Fig.3 Fully Open Type



3. Tanks for Water Hammer

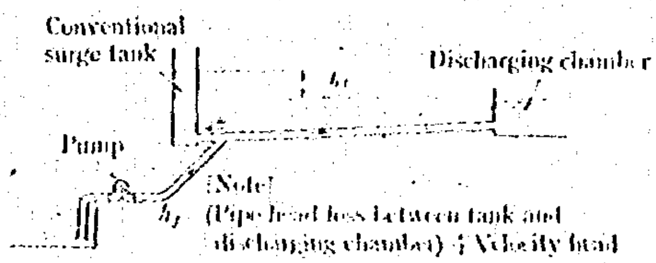
3.1. Junction Well

A junction well is built in the conveyance pipeline, principally to reduce the water pressure in the pipe. The water hammer energy is dissipated to the water in the well, causing the rise and fall of the level.

3.2. Surge Tank (Fig.4)

"Conventional" means having the free surface and having no check valve in the connecting pipe with the main.
The tank shall be built closely to the delivery pipe of pump.
The function for the water hammer is same as the junction well.

Fig.4 Conventional Surge Tank

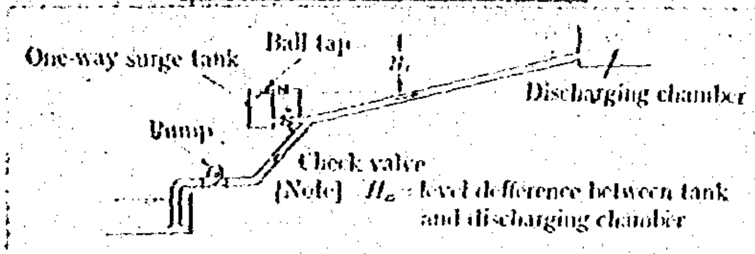


3.3. One Way Surge Tank (Fig.5)

It has a seemingly free surface but the level is so controlled by a ball tap that it cannot rise above the designed height.
A check valve is set up in the connecting pipe of the tank and the main. It ensures one way flow from the tank to the main.
With the gadget, the one way surge tank can be installed in and connected to the pressurized pipeline.

It is effective for the negative pressure in the pipeline as water can be fed from the tank, but it is ineffective for the positive pressure as the tank's level is controlled.

Fig.5 One-Way Surge Tank



4. Project Pipeline

On Fig.6, the profile of the project pipeline is shown. Some features are noticable:

* As the topography is rolling land, many gentle rises and falls make the profile.
* In case of water hammer, the magnitude of negative pressure tends to be high at the crests and that of positive pressure tends to be also high at the hollows.
* Especially the highest crest must be dealt with,as for the negative pressure by water hammer.

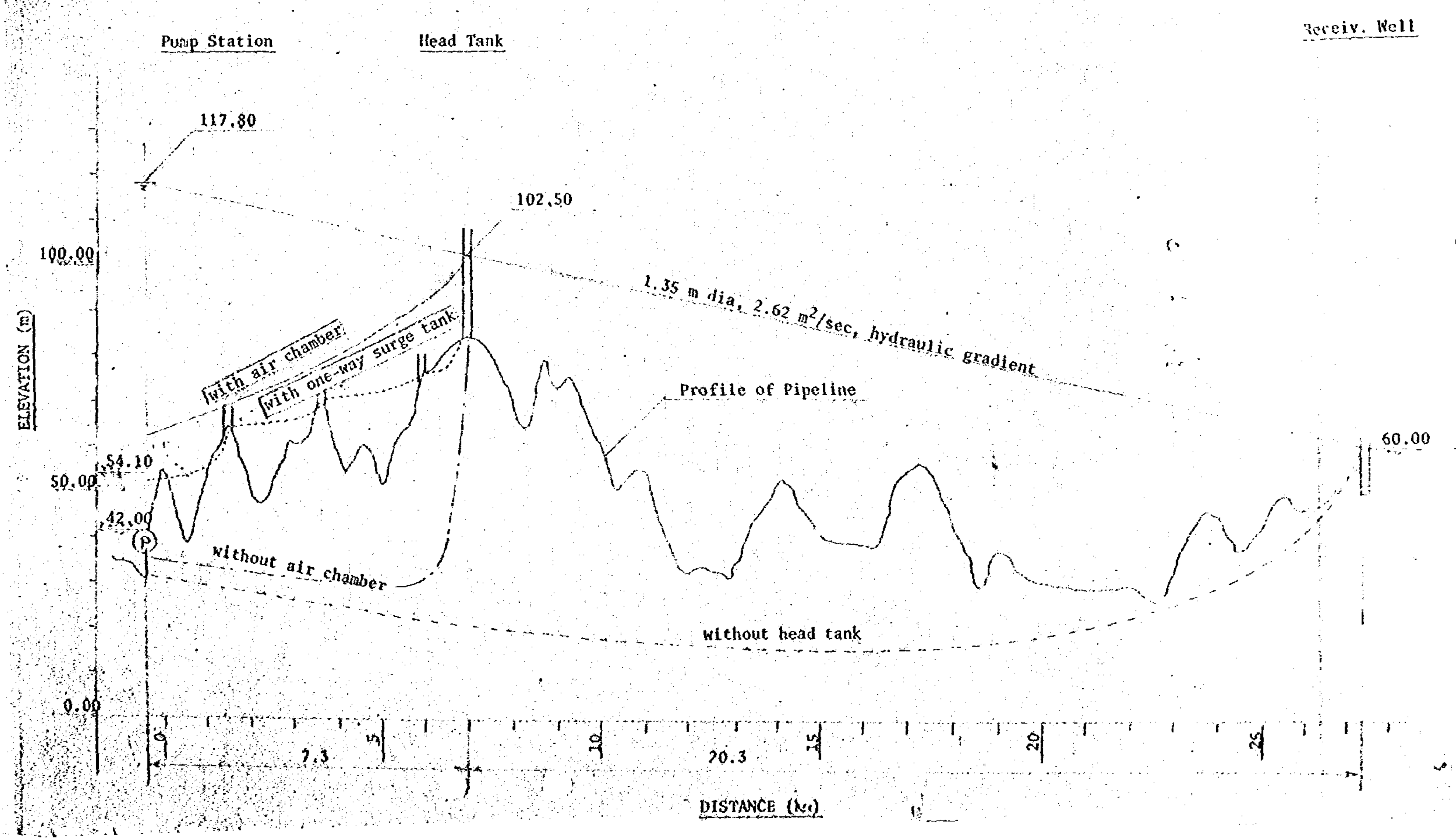
4.1. Conditions

Given is the design flow rate and assumed is the water level of the receiving well, in order to calculate the hydraulic gradient.
The gradient line, in case of water hammer, changes quite remarkably.

4.2. Without Head Tank and With Conventional Surge Tank

The minimum pressure line with a dotted line, is shown at the bottom of Fig.6.
It is the case when the whole pipeline is of the "closed" type explained in 2.1.
The largest negative pressure will amount 60 m water column and the pipeline will be in a great danger.
To make the system "open", the sole means is to build a high tank, higher enough to contain 117.80 water level, at the pump station.
The ground height of Dok Krai is about 56.00 and building such a high tank is quite questionable.
The case must be taken out of consideration.

Fig.6 MINIMUM PRESSURE LINE BY WATER HAMMER



4.3. With Head Tank

The case is of the "partially open" type explained in 2.2. The part between the head tank and the receiving well, the "open" part is free of the water hammer effect.
Only the part between the pump station and the head tank must be taken care of.

4.4. With Head Tank and One-Way Surge Tanks

The case is to build the one-way surge tank on each of three crests between the pump station and the head tank. The one-way surge tank, as is explained in 3.3 and shown on Fig.5, can be used for the topographical conditions under the project. With the head tank and three one-way surge tank, the minimum pressure line, as shown with a dotted line on Fig.6, will stay above thb profile of pipeline, nearly for the most of the part. Theoretically the method is feasible.
However, it needs too much care in maintenance to be used practically. The ball tap and the check valve shall be well kept in working condition and the regular, periodical inspection of three tanks along the road will certainly be a burden to the maintenance people.
Moreover, in case when the ball tap malfunctions, a large flow of spilling water must be dealt with.
For the size of flow, the rivers and water channels there seem too small.

4.5. With Air Chamber and Head Tank

With reasoning discussed in 4.1. to 4.4., the possibilities to deal with the water hammer are narrowed.

There remains the following methods still;

* To decrease the water hammer effect by using slow-closing check valves and rotating parts (motor and pump vanes) having bigger moment of inertia.

* To dissipate the water hammer energy by using air chamber

* Combination of the above two methods

Of the three methods quoted above, the most reliable and probably the cheapest is the use of air chamber and the air chamber is rather easily maintained, adding to its closeness to the pumping station.
The minimum pressure line, in the case of air chamber and head working in combination, will be well above the profile of pipeline, as shown on Fig.6. It means that the negative pressure can be prevented from occurence.
For comparison, another minimum pressure line in case of lacking the air chamber, is shown also on Fig.6 with a chained line.

5. Other Factors Considered

As is discussed in Report on Comparative Study of Flow Control System, the existance of head tank makes the control of system easy, in running pumps and in steadying the flow.

Also the head tank is necessitated to fill the pipeline with water following possible power failure.

In this paper, the discussion has been made to generalize the water hammer problem in great length.

6. Conclusion

The best method to prevent water hammer for the pipeline is induced tousing the air chamber and the head tank.