KINGDOM OF THAILAND
CHIANG MAI
WATER SUPPLY EXPANSION PROJECT

VOLUME I
DETAIL DESIGN REPORT

- MARCH 1973 -

PREPARED FOR
OVERSEAS TECHNICAL COOPERATION AGENCY
GOVERNMENT OF JAPAN
BY
TOKYO ENGINEERING CONSULTANT CO., LTD.
TOKYO JAPAN

VOLUME I

DETAIL DESIGN REPORT

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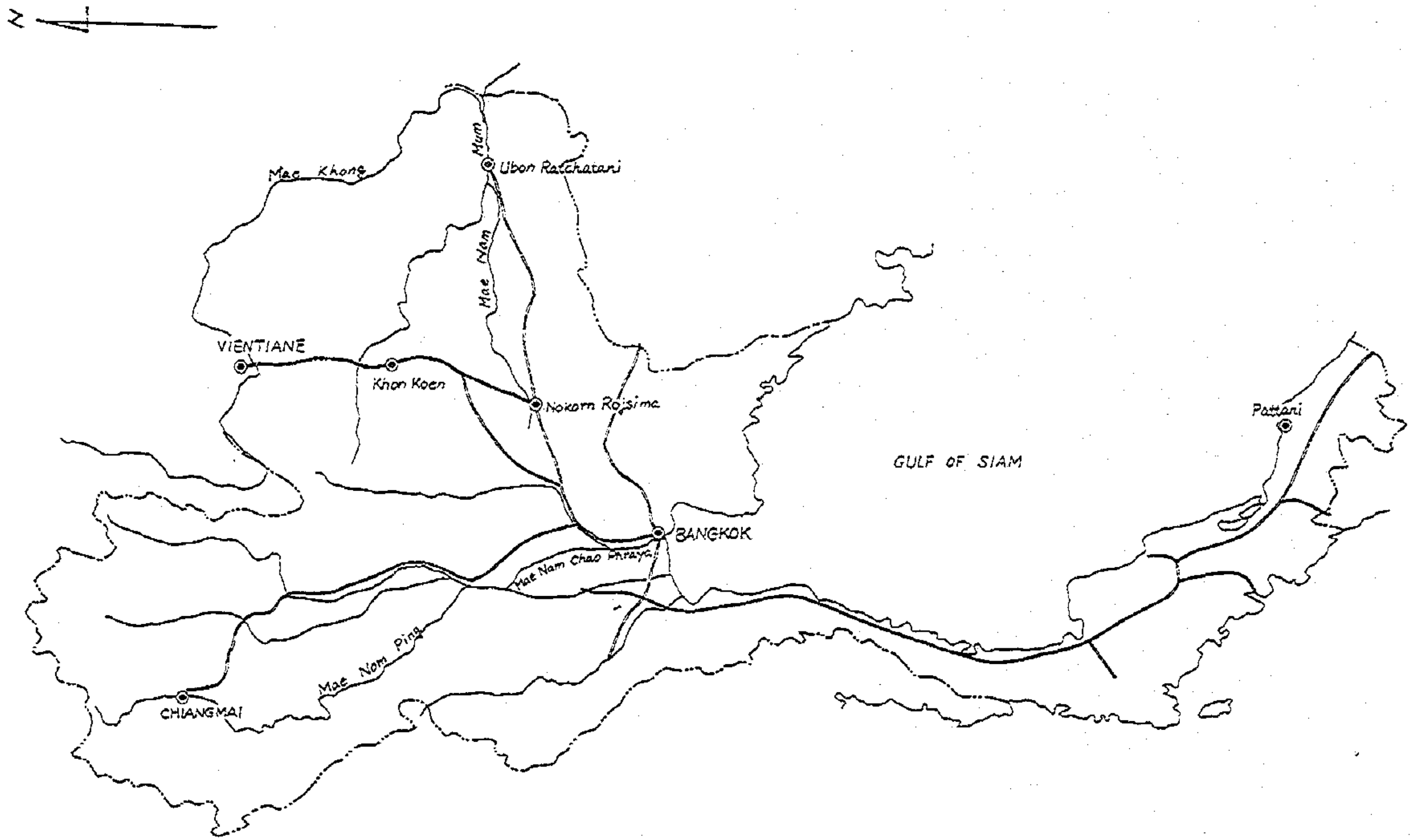
N
Mae Khong
Mum
Ubon Ratchatani
Mae Nam
VIENTIANE
Khon Koen
Nokorn Rajsima
GULF OF SIAM
Pattani
BANGKOK
Mae Nam Chao Phraya
Mae Nam Ping
CHIANGMAI

Chapter 1 INTRODUCTION

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Chapter 1 INTRODUCTION

1-1 Events Leading to the Detail Design for the Chiang Mai Water Works Expansion Project

Chiang Mai City is a provincial capital located in the northwest part of Thailand about 750 kilometers from Bangkok.

Initially, under the Colombo Plan, a Japanese planning and survey team visited Thailand during the two years of 1968 and 1969, followed by visits to Thailand during 1970 and 1971 of the Naito planning and survey group, and others. As a result, the government of Thailand requested in March 1971 that a team be dispatched to make plans and surveys for the two cities of Korat and Chiang Mai. The Overseas Technical Cooperation Agency of Japan then sent to Thailand a group headed by Dr. Tamon Ishibashi of Tokyo University to make a feasibility survey. The survey extended over thirty days in March and two weeks in August, and the report was submitted at the end of September.

Next, in response to another request by the Government of Thailand in June 1972, the Chiang Mai City Water Works Detail Design and Investigation Group, from Tokyo Engineering Consultants Co., Ltd., with Dr. Sachiho Naito as consultant, was dispatched to Thailand. This mission extended 95 days from June 26 to September 28, 1972. The survey included geological investigation of the site, survey of main routes at the site, investigation of domestic unit prices of materials, investigation of existing mains, and investigation of existing main heads and other material necessary for planning the expansion. During the height of this work in July, a combination of 14 Japanese and 6 Thai personnel were involved. At the end of August, work at the site was concluded, and 4 Japanese members made a simplified design plan, so that both PMWD and the Japanese were able to, based on design blueprints, exchange opinions, and reach final concurrence after making several revisions. The Japanese team left Thailand on September 28, 1972.

The present report is a final plan, produced in Japan, based on the reports and records of the above work.

1-2 Present Status of the Chiang Mai Water Works and Its Planning

1-2-1 The Intake Plant

Intake facilities on the Ping River for the Existing Old Water Treatment Plant provide that water be obtained at the right bank by means of a simple channel dug in the river bed from the center of flow near the left bank. Because of pollution of waters in this vicinity, this intake is not desirable, so it has been planned to be abandoned, and an intake point upstream was selected. This location is about 3.2 kilometers upstream from the Super Highway Bridge. Adequate water is obtainable here even in dry seasons although embankment work will be necessary to prevent erosion by flood, because the river bends. There is no fear of pollution.

1-2-2 The Water Treatment Plant

At present, there are two treatment plants for Chiang Mai. One is along the Ping River at Wang Sing Kum, and the other at Umong along the irrigation canal. The official rated capacity of the former is 7000 m^3/day, supplied to the eastern section of the city, and the capacity of the latter is 6000 m^3/day for the western sector.

1-2-3 Capacity of Existing Water Treatment Plants

Old Water Treatment Plant: Wang Sing Kum, Ping River source

New Water Treatment Plant: Umong, Irrigation Canal source

Old Plant (No. 1 Treatment Plant)

Filter	Size (m)	Amount	Capacity (m^3/hr.)	Capacity (m^3/day)	Effective Capacity (m^3/day)
*Filter No.1	2.50 x 3.0	2 (fan shape)	40	960	0
Filter No.2	4.00 x 3.0	2	80	1920	960
Filter No.3	4.00 x 3.0	5	250	6000	6000
Total:			370	8880	6960 (or 7000 in round figures)

New Plant (No. 2 Treatment Plant)

Filter	Size (m)	Amount	Capacity (m^3/hr.)	Capacity (m^3/day)
Filter No.1	4.0 x 3.0	5	250	6000
Total			250	6000
Sum Total:			620	14880 ≒ 13000

Note: *No. 1 Filter is to be abolished. One tank of No. 2 shall be considered a stand-by.

Clear Wells and Elevated Tanks

Old Plant	Size	Capacity (m^3)
E.T. No. 1	in the tower	200
C.W. No. 1	in the tower	300
C.W. No. 2	cylindrical	500
C.W. No. 3	27.0 x 21.0 x 2.65	1800
Total:		2800
New Plant	Size	Capacity (m^3)
C.W. No 1	27.0 x 21.0 x 2.65	1800
Total:		1800
Sum Total:		4600

1-2-4 Distributing Main Network

The system for the existing No. 1 Treatment Plant uses natural gravity flow from an elevated tank, while water from the Existing No. 2 Treatment Plant is sent directly by distributing pumps.

Chiang Mai University now has its own independent water works, using a pond as the source.

A point worthy of mention is that pressure in the delivery mains is very low in the vicinity of the railroad station on the left bank of the Ping River. The reason is that the 300 mm diameter of the delivery main from the No. 1 Treatment Plant is not large enough, as will be made clear in the report on the survey of main heads, included in subsequent pages of this report. Planning of this distributing main network includes making smaller the present supply districts, which must be improved and included in the new planning.

Furthermore, the present plan provides that water be supplied Chiang Mai University from the city water system.

1-2-5 Plan for Water Works Facilities for Chiang Mai

A. Served Population, Water Supply & Purification Plant Capacity in 3 Stages

Item		Stage 1 (1980)	Stage 2 (1990)	Stage 3 (2000)
Served Population		155000	196000	243000
Daily Maximum Supplied Capacity (m^3/day)	General	27000 m^3/d	42000 m^3/d	55000 m^3/d
	University	2000	4000	6000
	TOTAL	29000	46000	61000
Hourly Maximum Supplied Capacity (m^3/day)	General	40500	63000	82500
	University	2000	4000	6000
	TOTAL	42500	67000	88500
Capacity of Each Purification Plant (m^3/day)	Existing No.1	7000	7000	7000
	Existing No.2	6000	6000	6000
	Planned New Construction	16000	32000	48000
	TOTAL	29000	46000	61000
Capacity Of Each Intake Plant (m^3/day)	Existing No.1	abolished	abolished	abolished
	Existing No.2	6600 m^3/d	6600 m^3/d	6600 m^3/d
	Planned New Construction	7700	7700	7700
	TOTAL	31900	49500	67100

1-3 Objectives of the Water Works Expansion Project

The work of the detail design has as its objective producing specifications, design reports (construction cost sheets), blueprints and other related calculation sheets, which are fully adequate for the construction work of the Chiang Mai Water Works Expansion Project. Particularly, care shall be exercised regarding the below-listed points.

(1) Matters determined by consultation with PWD shall be adequately incorporated into the detail design.

(2) Tender for the construction shall be international.

(3) Standards for the Specifications shall give priority to ISO standards.

(4) Construction shall be safe and economical, with planning priority given to Stage 1 (1980) construction among the three stages of Stage 1 (1980), Stage 2 (1990), and Stage 3 (2000), so as to avoid unnecessary investment earlier than actually necessary, but still taking into consideration future planning.

(5) Locally produced materials shall be used to the extent possible.

(6) Maintenance and operation supervision shall be given due consideration, and care taken in the design stage that long stoppages in production do not occur because parts are not available for repairs.

1-3-1 Intake Facilities

1-3-3-1 The pump wells shall have sufficient capacity for Stage 3.

1-3-3-2 The pump room shall have enough space for installing pumps through Stage 3

1-3-3-3 Only pumps for Stage 1 shall be installed.

1-3-3-4 The space of the generator room shall have sufficient space to install the generators through Stage 3, but the generators shall be provided only for Stage 1.

1-3-2 Raw Water Mains

Mains shall be only for Stage 1, two additional parallel mains shall be added in the future.

1-3-3 Water Treatment Plant

1-3-3-1 Sedimentation Basin, Filter Basin, and Clear Water Reservoir.

These shall be designed for Stage 1 as one group. Facilities for Stage 2 and Stage 3 shall be similar type groups, located nearby. However, a connecting gallery shall be provided.

1-3-3-2 Distributing Pump Room

The pump room shall have space adequate for Stage 3, but only pumps for Stage 1 shall be installed.

1-3-3-3 Chemical Design Room and Chlorine Disinfecting Room

a) Room space shall be adequate through Stage 3, Equipment shall be for Stage 1 only.

b) Chlorine disinfecting equipment shall take into consideration neutralizing equipment.

1-3-3-4 Main Control Room

Space shall be adequate for Stage 3

1-3-3-5 Personnel Quarters

A concrete, two-story, long house shall provide for housing 11 families, and one of these units (for the superintendent) shall be especially large.

1-3-3-6 Embankment Work at the Plant Site

Embankments shall be done only for the group of facilities of Phase 1. Required earth shall be used from excavation for the drainage lagoon, and any quantity in deficiency trucked in from outside sources.

1-3-3-7 Elevated Tanks

Direct delivery by pumping shall be used, without building elevated tanks

1-3-3-8 Distribution Mains

a) Water for Chiang Mai University, based on the daily maximum demand, will be delivered from the No.2 Treatment Plant (by the airfield).

b) Where the width of roads is sufficient, mains shall be laid separately for Stage 1, Stage 2, and Stage 3. Where roads are too narrow, one main shall be laid adequate for all stages in the future.

c) Fire hydrants shall be the same type as those already in use.

d) Mains under 300 mm. in diameter shall use Thailand manufacture asboestos-cement pipe.

1-3-4 Power and Electricity

1-3-4-1 Both the intake plant and the treatment plant shall be supplied with 11 KV.

1-3-4-2 Design for electricity supply to the treatment plant shall include only facilities from the first pole at the exit of the transformer station. Design for electric facilities for the intake plant shall include only those within the intake plant site after the power and electric lines have been brought in within the site.

1-3-4-3 Telephone shall be used for liaison between the treatment plant and intake plant, so that communication lines will not be provided.

1-3-4-4 An Application for Electricity will be made to the Chiang Mai City Provincial Electrical Authority (PEA).

1-3-4-5 The cut-off volume will be set at 1000 MVA.

1-3-4-6 A booster generator will be used to limit fluctuations in electricity from the power source to ± 5%, and equipment to stabiliz electric power will not be included in the design.

1-3-4-7 Power supply voltages are to be 11 KV 50 Hz
Low voltage motors 3ϕ4W 380/220V
Electric light voltage 3ϕ3W 200V

1-3-5 Structures (including the main operations building, pump room, chemical dosing room storage, and operational gallery)

1-3-5-1 Generally, construction shall be for facilities through Stage 3. Movable partitions shall be designed to facilitate enlarging or reducing size.

1-3-5-2 Structures shall be reinforced concrete and wood. Steel frame structures are not desirable as materials.

1-3-5-3 Generally, design shall use local, domestic materials.

a) Floors shall be polished, artificial stone.

b) Exteriors shall be a ferrous material, silaraeng.

c) Roofs shall be corrugated asbestos slate.

1-3-5-4 The seasonal climatic conditions should be fully considered in design to provide adequate air circulation and protection from sun (mechanical facilities are not included in the design). Prevailing winds are southwest.

1-3-5-5 Ceiling heights should be 3.00 m or more.

1-3-5-6 Rain water and drain water shall be disposed of by sumps.

1-3-5-7 Toilets shall be Thai-style flush toilets, and waste water disposed of by sump-type facilities.

1-3-5-8 Hard woods shall be used structurally and lauan (mahogany) for finishes.

1-3-5-9 Scale of drawings shall be 1/50, 1/100, 1/250 and 1/500.

1-3-5-10 Design shall exercise care to comply with the building codes and laws of Thailand.

1-4 Phasing of the Detail Design

1-4-1 Plan of Operation for the Detail Design

1-4-1-1 Signatories

Signing of the plan of operation was completed at 2 p.m. on July 4, 1972 at the DTEC conference room by the following officials:

a) Thailand

Xujati Pramoolpol
Deputy Director-General
Department of Technical and Economic Cooperation (DTEC)
Ministry of National Development

Damrong Cholvijarn
Director-General
Dept. of Public and Municipal Works (PMWD)
Ministry of Interior

b) Japan

Toru Hayashi
Head of Japanese Survey Team
Overseas Technical Cooperation Agency (OTCA)
Ministry of Health and Welfare

1-4-1-2 Representatives attending the official signing of the plan of operation were:

Name	Title
Yasuo Tokuoka	Secretary, Japanese Embassy
Moriya Miyamoto	Branch Chief, OTCA
Xujati Pramoolpol	Deputy Director-General, DTEC
Somsak Chowprasert	"
Wanchai Siriratana	"
Damrong Cholvijarn	Director-General, PMWD
Kasien Anambutr	"
Sawasdi Orvichian	"
Toru Hayashi	Head, Investigation Team
Sachiho Naito	" "
Eiichi Mutsuro	" "

PLAN OF OPERATION

FOR

THE DETAIL DESIGN OF

THE CHIANG MAI WATER WORKS EXPANSION PROJECT

CONTENTS

I. Introduction

1. The Government of Thailand requested the Government of Japan to investigate and to design the Expansion Project for Provincial Water Supply System.

2. The Government of Japan sent the Colombo Plan experts, at the request of the Government of Thailand, to cooperate with the Government of Thailand in completing the master plan, starting in 1968.

3. The feasibility study of the water supply system both in Chiang Mai and in Nakhon-Rachasima was completed by the Japanese survey team dispatched in March, 1971. The feasibility reports were submitted to the Government of Thailand in October 1971.

4. The Government of Thailand decided to take up the water supply system in Chiang Mai as the first priority project and has further requested the Government of Japan to draw up the detailed design for its construction.

5. The Government of Japan, in response to this request, had decided to take necessary measures to carry out the detailed design work and has entrusted the Overseas Technical Cooperation Agency (OTCA) of Japan with its implementation.

6. This document sets forth a plan of operation in regard to the detailed design for the project.

II. Scope of Survey

7. The Government of Japan will carry out the detailed design work with particular reference to:

1) raw water intake facilities
2) purification facilities
3) distribution system

8. The detailed design work of this project is confined to the 1st stage of the master plan described in the afore-mentioned feasibility reports. The master plan shows that the estimated served population will be 243,000 and the water supplying capacity will be increased by 48,000 m^3/day in the year 2000.

The master plan also predicts that when the 1st stage is completed, served population will be 155,000 and the capacity of water supply will be increased by 16,000 m^3 per daily maximum.

9. The raw water is drawn from Ping River. The survey of the distribution system covers only the main pipeline system, which is connected with the existing pipe line system.

III. Execution of Survey

10. The team will conduct field survey and data collection as follows for about three (3) months in Thailand.

(1) Soil Investigations

Penetration Test	10 holes
Core Boring	1 hole
(Thin Wall Piston Sampling)	

(2) Surveying
- Leveling
- Plane Table
- Other necessary surveying

(3) Collection of data and information concerning labour conditions, labour cost, construction materials, construction machines, laws and regulations, design standards, existing facilities and so on.

(4) Investigation of existing pipelines.

(5) Water pressure test at the connection point of the proposed distribution main and existing pipelines.

11. The team will conduct the following detailed design work for about three (3) months in Japan.

(1) Intake facilities
- Intake Tower
- Raw Water Main
- Receiving Well

(2) Purification Facilities
- Mixing Basin
- Flocculation Basin
- Sedimentation Basin
- Rapid Sand Filter
- Elevated Tank for Washing

(3) Distribution system

Clean Water Reservoir

Distribution Main

Distribution Pump or Elevated Tank

12. The following documents written in English will be presented in draft form to the Government of Thailand by the end of December, 1972.

The complete documents will be presented by the end of January, 1973.

(1)	Design Report	30 copies
(2)	Specifications	30 copies
(3)	Cost Estimate	1 copy
(4)	Drawings	30 copies
(5)	Bill of Quantities	30 copies
(6)	Tender Document Forms	30 copies

IV. Arrangements to be made by the Government of Thailand

13. The following will be arranged for the survey team by the Government of Thailand.

(1) Appointment of a project manager who is responsible for the Water Supply Expansion Project in Chiang Mai and who accompanies the team when necessary.

(2) Appointment of two counterparts and two technicians.

(3) Provision of a furnished office and two jeeps with chauffeurs in Chiang Mai.

(4) Provision of a furnished office in Bangkok.

(5) Procurement of data and materials related to the Project.

V. Privileges and Exemptions to be Granted to the Survey Team

14. The members of the team engaged in the survey will be entitled to such privileges and exemptions as the Government of Thailand normally extends to Colombo Plan experts in respect of tax and duty exemptions, and immigration facilities.

VI. Signatures

15. The undorsigned agreed on the foregoing on behalf of parties concerned on this date of July 4, 1972.

Mr. Xujati Pramoolpol	Mr. Toru Hayashi	Mr. Damrong Cholvijarn
Deputy Director-General	Head of Japanese	Director-General
DTEC.	Survey Team	Department of Public
Ministry of National	Overseas Technical	and Municipal Works
Development.	Cooperation Agency	Ministry of Interior.
	Japan.	

1-4-2 Site Investigation

1-4-2-1 Participants in the investigation survey were:

Japanese	Thai
Toru Hayashi (MHW)	Sawasdi Orvichian (Chief Engineer)
Eiichi Mutsuro (OTCA)	Aroon Thaichareon (Counterpart)
Sunao Kameda	Tospon Tiparos (Engineer)
Shigeichi Kobayashi	Suttichai Anombuta (Engineer)
Sachiho Naito	Vera Rangkao (Technician)
Hajime Yamada	Suwan Klinbula (Boring Engr.)
Tomio Matsuhashi	Paibun Trithum (Boring Engr.)
Makoto Kaneko	Wiroon Rungrong Thanin (Supt., Chiang Mai Water Works)
Masaharu Takasugi	
Kazufumi Momose	
Koichi Kato	
Minoru Kujima	
Tadashi Akakabe	
Makoto Sato	
Kanasugi Watanabe	
Yoshinori Takada	
Takashi Suzuki	

1-4-2-2

The site investigation was conducted in regard to matters listed below. Work at Chiang Mai was done between July 30 and Aug. 30, 1972, and four Japanese remained in Thailand for consultation work and to produce drawings between August 31st and September 28, before returning to Japan.

a) Walking survey of site included raw water intake points, locations for treatment plant and pipe routes.

b) Site investigation included the intake plant, treatment plant, and distributing main routes.

c) Site geological investigation included standard boring sampling for the intake plant and treatment plant sites, and projected location of the conduit bridge, and production of soil strata maps.

d) Water pressure investigation included installed pressure meters and newly selected locations in the existing mains.

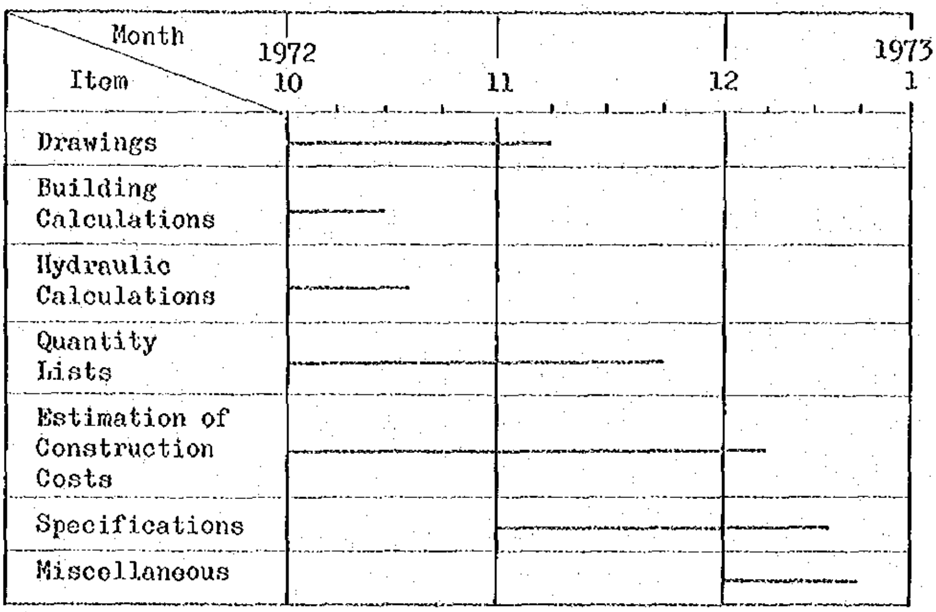
e) Investigation of underground pipes included checking depth, type, and diameter of existing mains running close to newly planned mains.

f) Unit costs for domestic labor, raw material, transport, taxes, and duties, as well as standards for manufactured auxiliary items or parts were checked. Also regulations regarding restoration, etc., of road surfaces. Such unit prices for labor and materials as were not available at Thai government offices were checked with local city merchants.

1-4-3 Consultations to Reach Agreement between Design Engineers and Clients

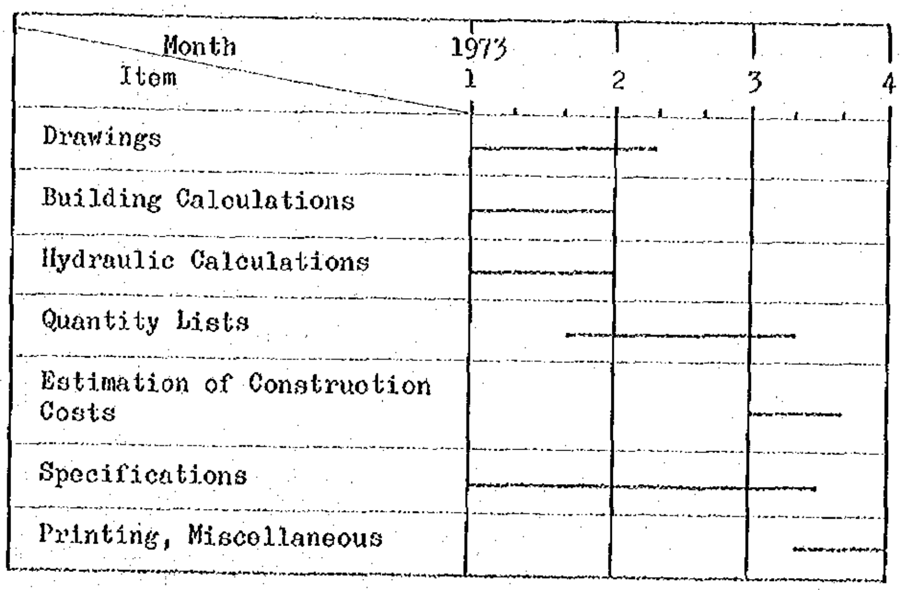
Consultations were held from Sept. 20 to Sept. 22, and further verified by various correspondence after return to Japan of the Japanese engineers.

1-4-4 Design Work in Japan



Note: The schedule was changed for submission of the interim report at the end of December.

After discussions with PWD in early January in 1973, some sections of the Design report were changed. Accordingly, the schedule was changed as follows:



1-4-5 Submission of the Draft Design, and Consultation on Its Contents

a) December 12, 1972

One engineer was sent to Thailand, hand-carrying material

b) January 3, 1973

Two additional personnel were sent to Thailand for consultations, held at a PWD conference room on January 9th. Representing Thailand were the Honorable Mr. Damrong Cholvijarn, Director-General; Mr. Kasien, Division Head, Mr. Sawasdi, Section Head, and Mr. Aroon. Japanese present were Dr. Naito, and Messrs. Yamada and Takasugi.

1-4-6 Interim Report

On January 20, 1973, an interim report was made to a Work Supervision Committee at the Japanese Ministry of Health and Welfare, determining the manner of revising the various items desired to be changed by the Thai government.

1-4-7 Submission of Final Design Report

Three personnel will carry the final design report for submission to Thailand by March 31, 1973.

Chapter 2 <u>SITE INVESTIGATION FOR THE DETAIL DESIGN</u>

Chapter 2 Site Investigation for the Detail Design

2-1 Scope of Investigation

The Feasibility Survey completed in the summer of 1971 was used for reference. In order to assemble the additional design data necessary to begin detail design, surveying was done as indicated below over a 95-day period between June 26 and September 28, 1972.

2-1-1 Consultation and Liaison with the Thai Government

2-1-1-1 Time: June 27, 1972 at 1:30 p.m.

Place: Library, PMWD, Bangkok.

Participants:

Thai ——— Messrs. Kasien, Sawasdi, Aroon, Sutthichoi and Tospon

Japanese ——— Messrs. Hayashi, Kameda, Kobayashi Mutsuro, Yamada, Matsuhashi, Kaneko, Takasugi, and Sato

2-1-1-2 Time: June 28, 1972 at 1:30 pm.

Place: Library, PMWD, Bangkok

Participants:

Thai - Messrs. Kasien, Sawasdi, Aroon, Sutthichoi and Tospon

Japanese - Messrs. Hayashi, Kameda, Kobayashi, Rikuji, Yamada, Matsuhashi, Kaneko, Takasugi, and Sato.

2-1-1-3 Time: June 29, 1972 at 2:00 pm.

Place: Conferance Room, PMWD, Bangkok

Participants:

Thai - The Honorable Damrong Cholvijarn, Messrs. Kasien, Sawasdi, Aroon, Sutthichoi, and Tospon

Japanese - Messrs. Tokuoka (Embassy Secretary) Miyamoto (OTCA Branch Office Chief), Hayashi, Kameda, Kobayashi, Naito, Rikuji, Yamada, Matsuhashi, Kaneko, and Takasugi

Note: As of October 1, 1972, the PMWD changed its designation to PWD (Department of Public Works)

Technical Memorandum

27th June 1972

1. Water Demand for University:

This is defined to be the daily maximum demand.

2. Existing Capacity:

It is confirmed to be 7,000 m^3/day at the old plant and 6,000 m^3/day at the new plant

3. High Water Level of River:

This data will be provided by PMWD.

4. Turbidity:

Maximum turbidity may be around 1,000 ppm or less, but this data will be provided by PMWD.

5. Drainage System:

This system is adopted as comparatively cheaper than using a lagoon.

6. Clarifier:

This shall depend on the turbidity aforementioned. The clarifier adopted is considered to be comparatively less in cost and of high efficiency.

7. Filter:

The conventional type is probably adequate, but the automatic washing system shall be further discussed.

8. Depth of Distribution Pipes:

In general, all pipes shall be laid under the side walk, but this may depend upon conditions at the site.

9. Load of Trucks:

Standard criteria is based on 20 tons maximum.

10. Railway Crossing:

Data is provided by the PMWD from previous data approved by the Railway Department.

11. Connection between New Pipe and Existing Pipe:

The location and depth of existing pipes shall be notified by the PMWD.

2-1-2 Site Investigation at Chiang Mai

(1) Walking Investigation

(2) Instrument Surveying

(3) Geological Investigation

(4) Water Pressure Investigation

(5) Investigation of Underground Pipes

(6) Investigation of Unit Costs for Labor and Materials, of Taxes or Duties, and of Laws and Regulations

2-1-3 Drawings Produced in Bangkok

(1) Drafting

(2) Liaison, Coordination, and Consultation

Personnel and Duration of Investigation Survey

2-2 Names of Personnel in the Site Investigation and Days in Thailand (June 26 - September 28, 1972)

	Name	Date of Arrival	Date of Departure	Days in Thailand	Nature of Duty
1	Hayashi, Toru	June 26	July 5	10	Head of Japanese personnel
2	Kameda, Sunao	" "	" 10	15	Walking survey
3	Kobayashi, Shigeichi	" "	" 10	15	" "
4	Naito, Sachiho	" "	Aug 9	45	Walking survey general investigation
5	Mutsuro, Eiichi	" "	" "	45	" "
6	Yamada, Hajime	" "	Sept 28	95	Assembling, and organizing data, and reports
7	Matsuhashi, Tomio	" "	" "	95	Surveys for underground mains, investigation of water pressures
8	Kaneko, Makoto	" "	" "	95	" "
9	Takasugi, Masaharu	" "	" "	95	" "
10	Kato, Koichi	July 6	Aug 5	31	Instrument survey
11	Momose, Kazufumi	" "	" 5	31	" "
12	Takada, Yoshinori	" "	July 20	15	Structural engineering
13	Suzuki, Takashi	" "	" 20	15	Electrical engineering
14	Sato, Makoto	June 26	" 10	15	Geological survey
15	Watanabe, Kanasugi	July 6	Aug 9	35	" "
16	Kujima, Minoru	" "	" 5	31	Instrument survey
17	Akakabe, Tadashi	" "	" 5	31	" "

2-3 Walking Investigation

2-3-1 Possibilities for the Intake Plant Site

Three or four sites were considered for the following conditions:

2-3-1-1 Assurance that conditions for water intake would not be impaired at a future time.

2-3-1-2 That the site be inexpensive, and easy to acquire.

2-3-1-3 That the site would not be vulnerable to polluting influences.

As a result, the point selected was about 2.9 km. upstream from the highway bridge on the left bank of the Ping River.

2-3-2 Two Site Possibilities for the Treatment Plant

Conditions taken into consideration were:

2-3-2-1 That the site be inexpensive and easy to acquire.

2-3-2-2 That facilities for waste water disposal be not costly.

2-3-2-3 That the location be favorably located with respect to the distributing main network.

2-3-2-4 That the location be favorable with respect to the intake plant.

The location selected is adjacent to the ice plant along the highway.

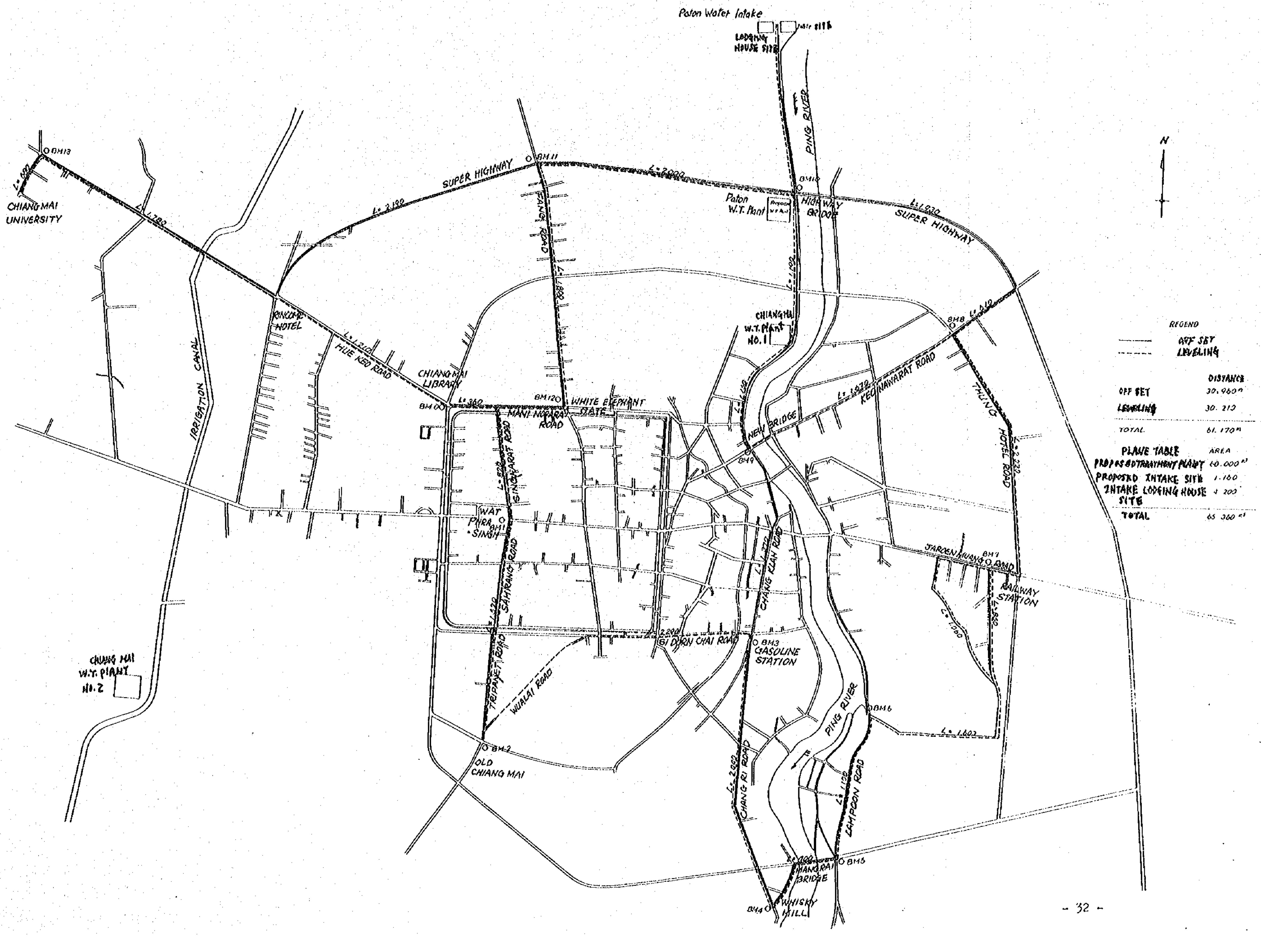
2-4 Surveying

The scope of the surveying included the intake site, the treatment plant site, the raw water canal, and distributing main routes, with plane table and leveling, and surveying of offsets as follows:

Offset	30.96 km.
Leveling	30.21 km.
Total:	61.17 km.

Plane Table

Proposed Treatment Plant site	38.000 Rai (60,800 m^2)
Proposed Intake Plant site	0.725 Rai (1,160 m^2)
Site for Lodging at Intake Plant	2.625 Rai (4,200 m^2)
Total:	41.350 Rai (66,160 m^2)



Paton Water Intake
LODGING HOUSE SITE
PING RIVER
N
SUPER HIGHWAY
CHIANG MAI UNIVERSITY
FANG ROAD
Paton W.T. Plant
HIGHWAY BRIDGE
SUPER HIGHWAY
IRRIGATION CANAL
RINCOME HOTEL
HUE KEO ROAD
CHIANG MAI LIBRARY
CHIANGMAI W.T. PLANT NO. 1
KEO NAWARAT ROAD
WHITE ELEPHANT GATE
MANI NOPARAT ROAD
NEW BRIDGE
HOTEL ROAD
WAT PHRA SINGH
SINGHARAT ROAD
SAMLANG ROAD
CHANG KLAN ROAD
JAROEN MUANG ROAD
RAILWAY STATION
TRIPANET ROAD
GI DURN CHAI ROAD
GASOLINE STATION
CHIANG MAI W.T. PLANT NO. 2
WUALAI ROAD
PING RIVER
OLD CHIANG MAI
CHANG RI ROAD
LAMPOON ROAD
MANGRAI BRIDGE
WHISKY HILL
REGEND
OFF SET
LEVELING
DISTANCE
OFF SET 30.960 m
LEVELING 30.210
TOTAL. 61.170 m
PLANE TABLE AREA
PROPOSED TREATMENT PLANT 60.000
PROPOSED INTAKE SITE 1.160
INTAKE LODGING HOUSE SITE 4 200
TOTAL 65 360

2-5 <u>Geological Investigation</u>

For the geological survey, standard boring tests and geological strata maps were made of the various locations indicated below.

No.	Depth of Boring (m)	Location
No. 1	10.8	Right bank of highway bridge
No. 2	10.5	Existing Water Treatment Plant site
No. 3	15.5	Proposed Water Intake site
No. 4	12.5	Proposed Water Treatment Plant site
No. 5	12.5	" " " " "
No. 6	12.5	" " " " "
No. 7	12.5	" " " " "
No. 8	12.5	" " " " "
No. 9	15.5	Proposed Water Intake site
Total	114.8	

2-5-1 Objectives

At Chiang Mai, a geological survey was made for the water works expansion construction. In main, the investigation consisted of test boring. The area investigated was a flat area along the Ping River, where a fan-shaped area with a strata of conglomerate material was found about 10.00 m. below the surface.

Strata consisted of an upper sand layer, a layer of clay, a middle layer of sand, and the strata of coarse conglomerate material. As a foundation for buildings, it appears that the middle layer of sand, or the strata of conglomerate material below are satisfactory. Sinking does not seem much of a problem in the sandy earth. Since the underlying strata is deep, pile foundations are feasible, while the long-term allowed supporting strength of the conglomerate material strata is slightly less than twice that of the upper sand layer. Choice between the two layers to lay building foundations would depend on the time of the construction, costs, and operating conditions. The upper sand layer and the clay layer are soft and weak, so that there may be negative friction, and there, the N-value may be a comparatively large N = 10. If the areas of the various strata could be clearly defined, the type of foundation work might be varied according to the location, providing a still more rational foundation design. However, at present, the use of pile foundations for all structures seems preferable.

2-5-2 Outline of Investigation

2-5-2-1 Objective of Investigation

This investigation was conducted to obtain design data, rendering clear conditions pertaining to the foundations, mainly of building structures, for the intake plant, treatment plant and related facilities for the water works expansion project of Chiang Mai City.

2-5-2-2 Area of Investigation

Along the Chiang Mai Super Highway and the Ping River of Chiang Mai City, Thailand (Refer to drawings locating boring points and survey locations)

2-5-2-3 Period of Investigation

July 2, 1972 to August 6, 1972

2-5-2-4 Scope of Investigation

The survey determined the distribution and nature of geological strata, and the nature of these soil strata by boring tests, and also obtained N-values as data to estimate earth bearing stresses. The original intent was further to collect undisturbed earth samples to conduct laboratory tests, but it became apparent there was no special necessity for this work, so it was suspended. The number of tests made are shown separately at the end of section 2-5-2-6.

2-5-2-5 Main Equipment and Tools Used.

Item	Model	Capacity	Personnel or Quantities
Drilling Machine	TFP-2H	50.00 m	1
Motor	Yanmar Diesel F-8	8 HP	1
Drilling Tools			1 set
Iron Scaffolding			1
Equipment for Standard Penetration Tests	Items meeting	JIS standards	1 set
Auxiliary Tools			1 set

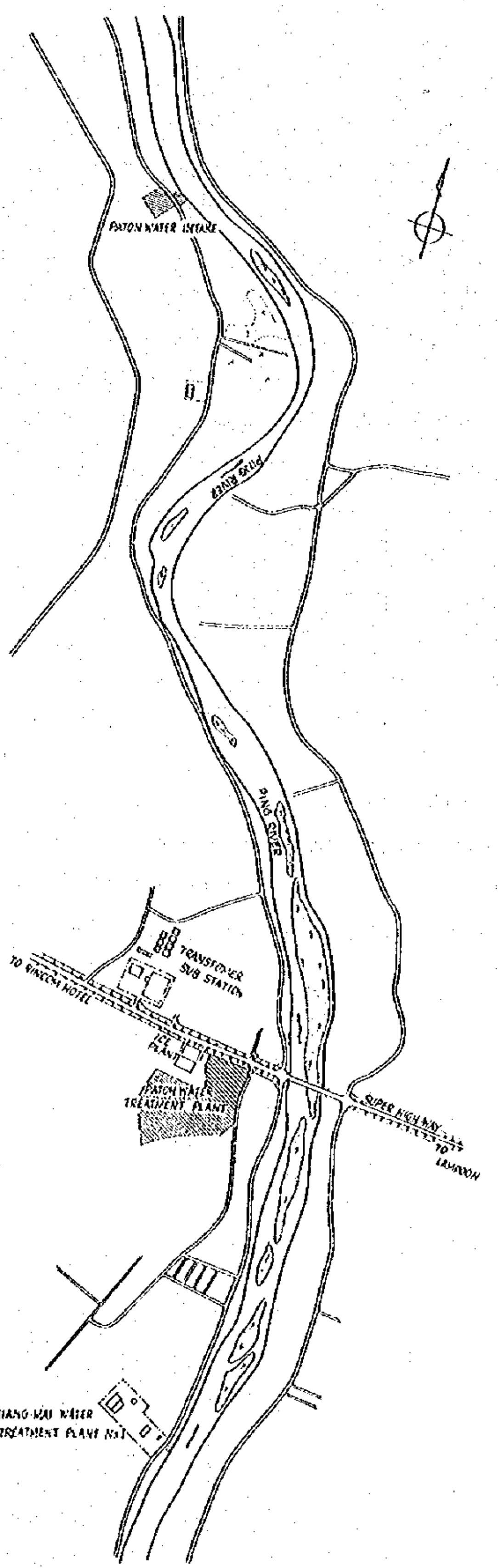
Note: The pump used for drilling was piston type, with a capacity of 30 ℓ/min., and connected to the TFP-2H boring machine.

2-5-2-6 Outline of the Investigation

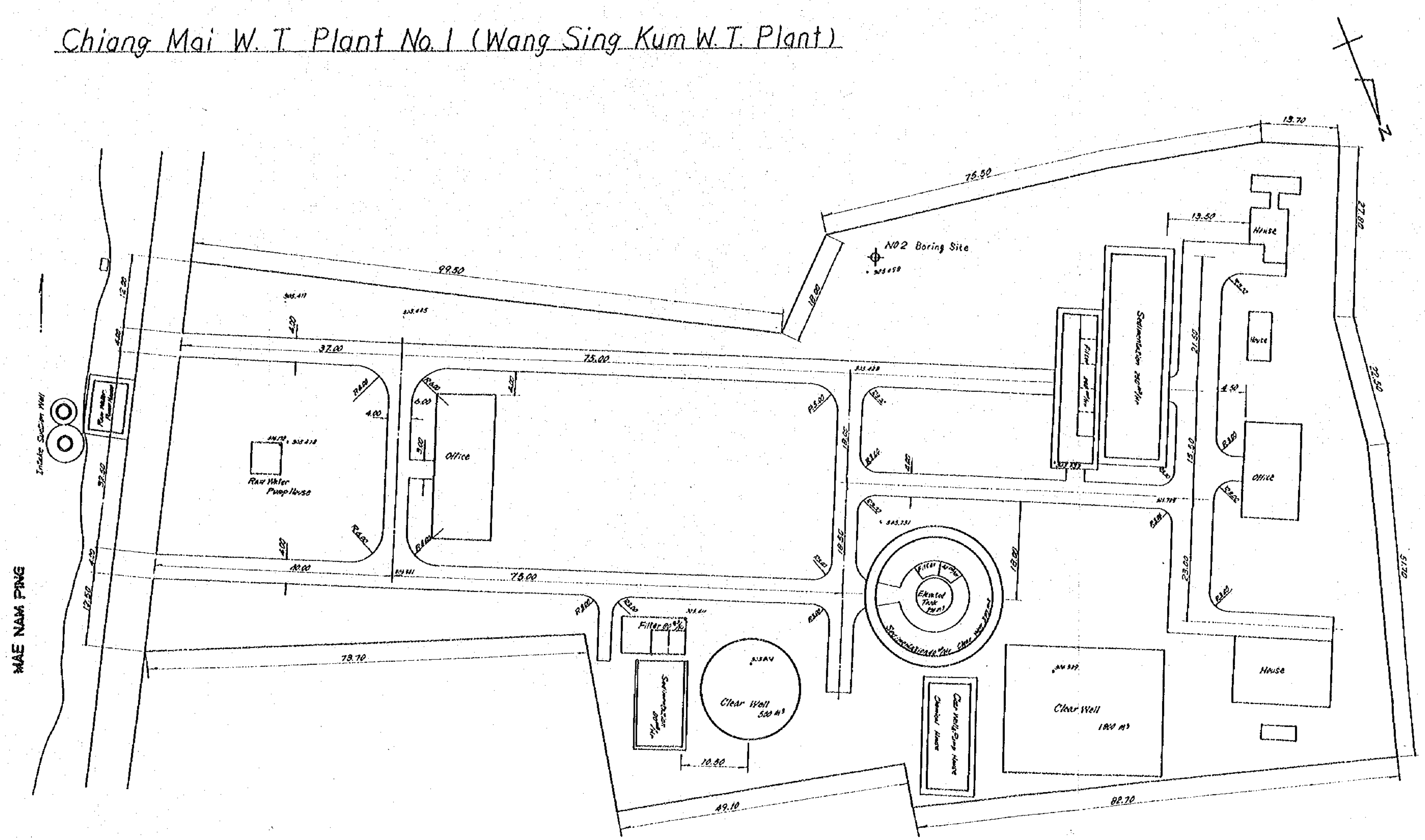
The work was done by a team comprised of Suwan Klinbula of the Provincial Water Supply Division, Department of Public Works, Kanasugi Watanabe and Makoto Sato. Access to the intake site, and to the existing treatment plant was comparatively easy, but much work was necessary for transporting or removing equipment and for temporary access by foot to the rice-paddy area which is to be the site of the proposed treatment plant.

The drilling machine used was the Tone Model TFP-2H, with a rated capacity for 50 m. The sand strata was deep, and it was impossible to completely remove the slime, so that the efficiency of the pump-equipped test drilling apparatus was impaired by slime accumulating in the circulating muddy water. For a while, the boring efficiency was low but by using casing pipe, controlling the sludge, and improving the circulation apparatus, the survey was completed within the original schedule.

As a rule, standard penetrating tests are not used along with underground water test boring. The engineers of the Thai boring team, without experience in this technique, became proficient in a short time, and their help produced accurate data.


PATON WATER INTAKE
PING RIVER
PING RIVER
SUB STATION
TO RINCOM HOTEL
ICE PLANT
PATON WATER TREATMENT PLANT
SUPER HIGH WAY
TO
CHIANG MAI WATER TREATMENT PLANT No1

Chiang Mai W. T. Plant No. 1 (Wang Sing Kum W. T. Plant)



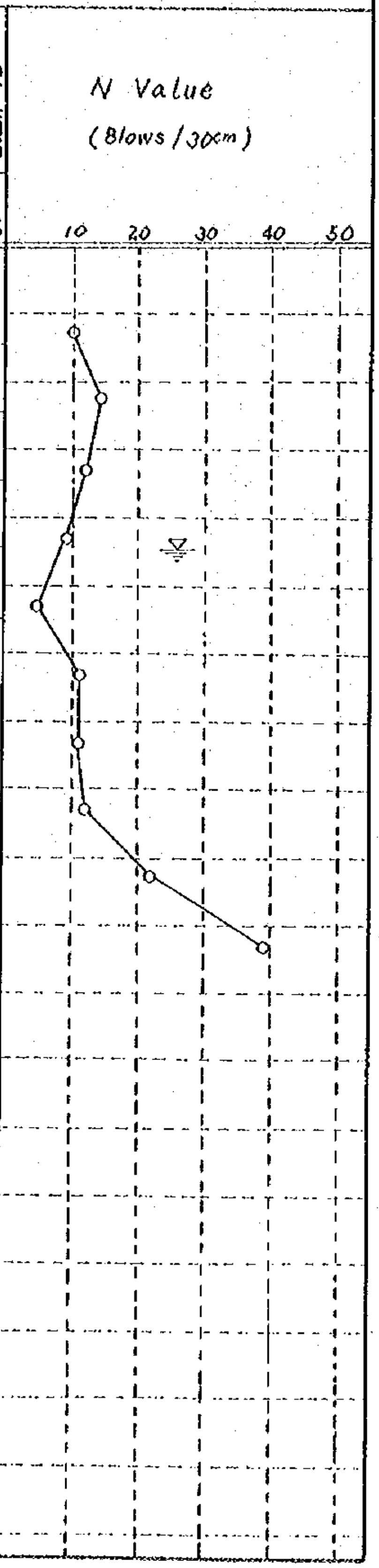
Date of Investigation 8th Jul ~ 9th Jul. '72

Core No. 2

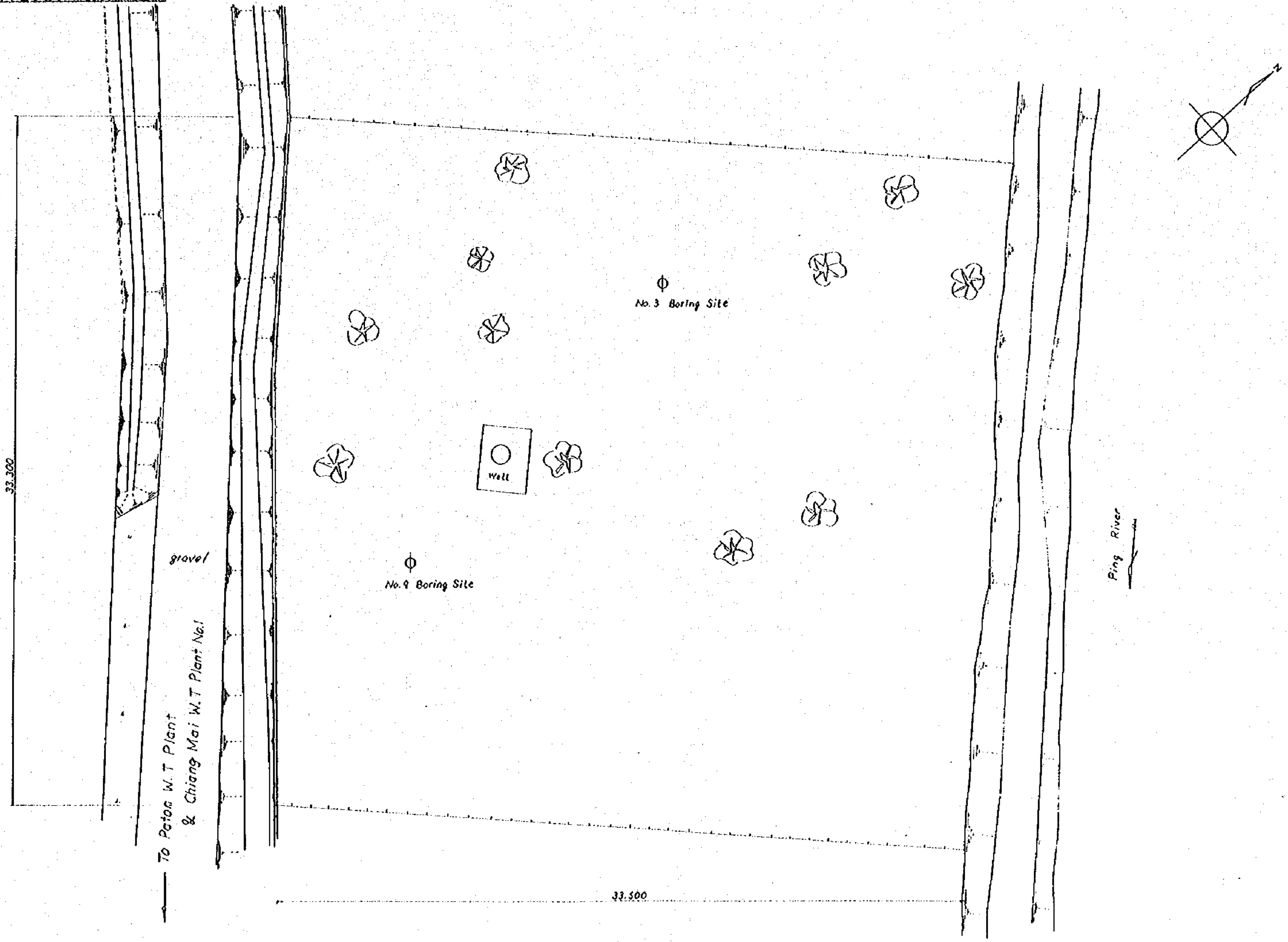
Water Table -4.50m

Depth (m)	Thickness of Stratum (m)	Classification of Soils	Color
0.00			
0.60	0.60	Surface Soil (Sandy Clay)	Light Brown
1.20	0.60	Gravelly Coarse Sand	Brown
5.30	4.10	Fine Sand ~ Coarse Sand	Brown
6.30	1.00	Clay	Black Gray
9.10	2.80	Medium Sand ~ Coarse Sand	Light Gray
10.50	1.40	Gravelly Coarse Sand	Light Gray

Depth of Test (m)	N Value (Blows/30 cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
1.15 – 1.45	10	3	4	3
2.15 – 2.45	14	5	5	4
3.15 – 3.45	12	4	4	4
4.15 – 4.45	9	3	3	3
5.20 – 5.50	5	3	1	1
6.20 – 6.50	11	3	3	5
7.15 – 7.45	11	3	3	5
8.15 – 8.45	12	3	4	5
9.15 – 9.45	22	6	7	9
10.20 – 10.50	39	11	13	15



Paton Water Intake



Date of Investigation

11th Jul. ~ 15th Jul. '72

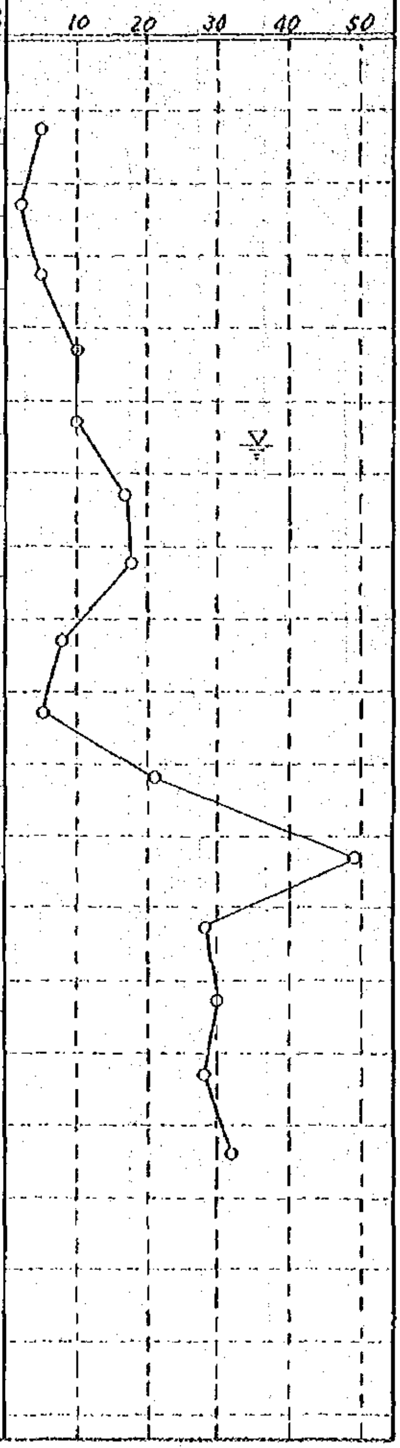
Core No. 3

Water Table -5.60

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color
0.00				
0.40	0.40		Surface Soil (Sandy Clay)	Light Brown
2.80	2.40		Clayey Fine Sand	Brown
9.80	7.00		Clay	Gray ~ Bluish Gray
15.50	5.70		Gravel and Sand	Gray

Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
1.15 – 1.45	5	2/14	2	1/2
2.15 – 2.45	2	1/15	1/15	
3.15 – 3.45	5	1	2	2
4.15 – 4.45	10	2	3	5
5.15 – 5.45	10	2	3	5
6.15 – 6.45	17	4	6	7
7.10 – 7.40	18	6	5	7
8.15 – 8.45	8	2	3	3
9.15 – 9.47	5/32	2/15	2/12	1/5
10.00 – 10.30	21	7	7	7
11.10 – 11.40	49	16	16	17
12.10 – 12.40	28	9	9	10
13.10 – 13.40	30	8	10	12
14.10 – 14.40	28	8	9	11
15.20 – 15.50	32	9	10	13

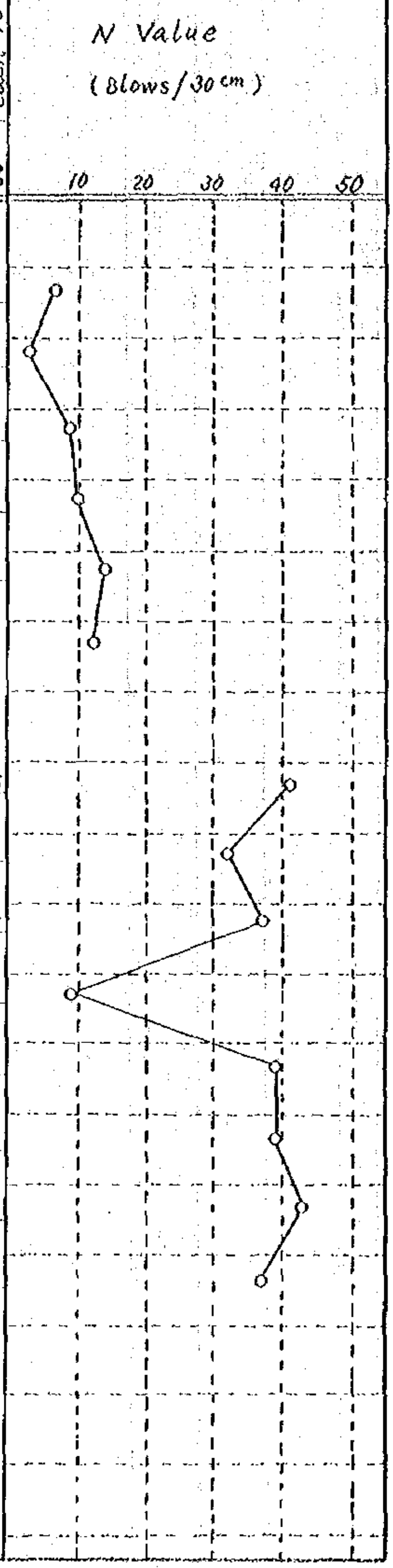
N Value (Blows/30cm)

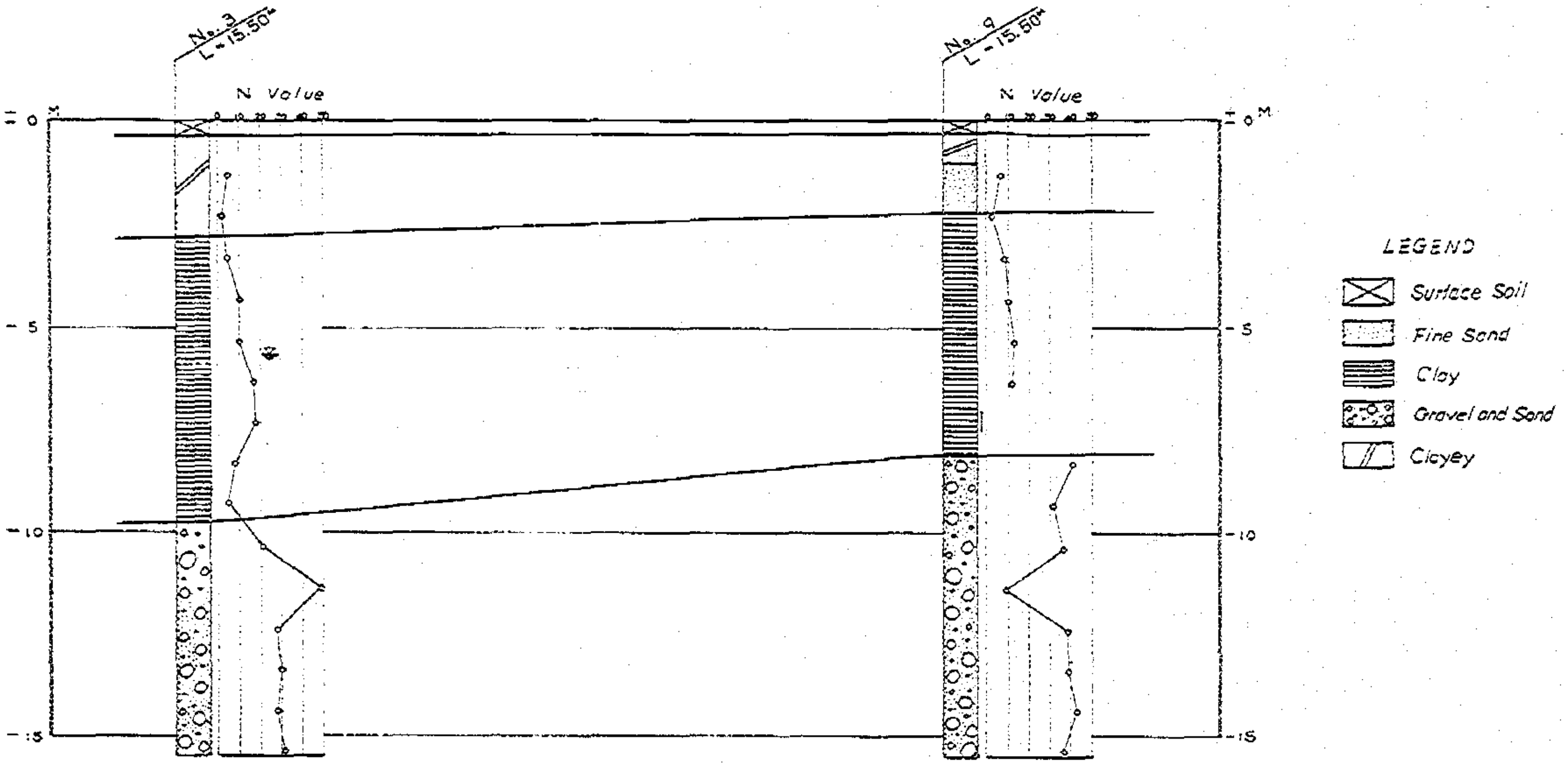


Date of Investigation
4th Aug. ~ 5th Aug. '72

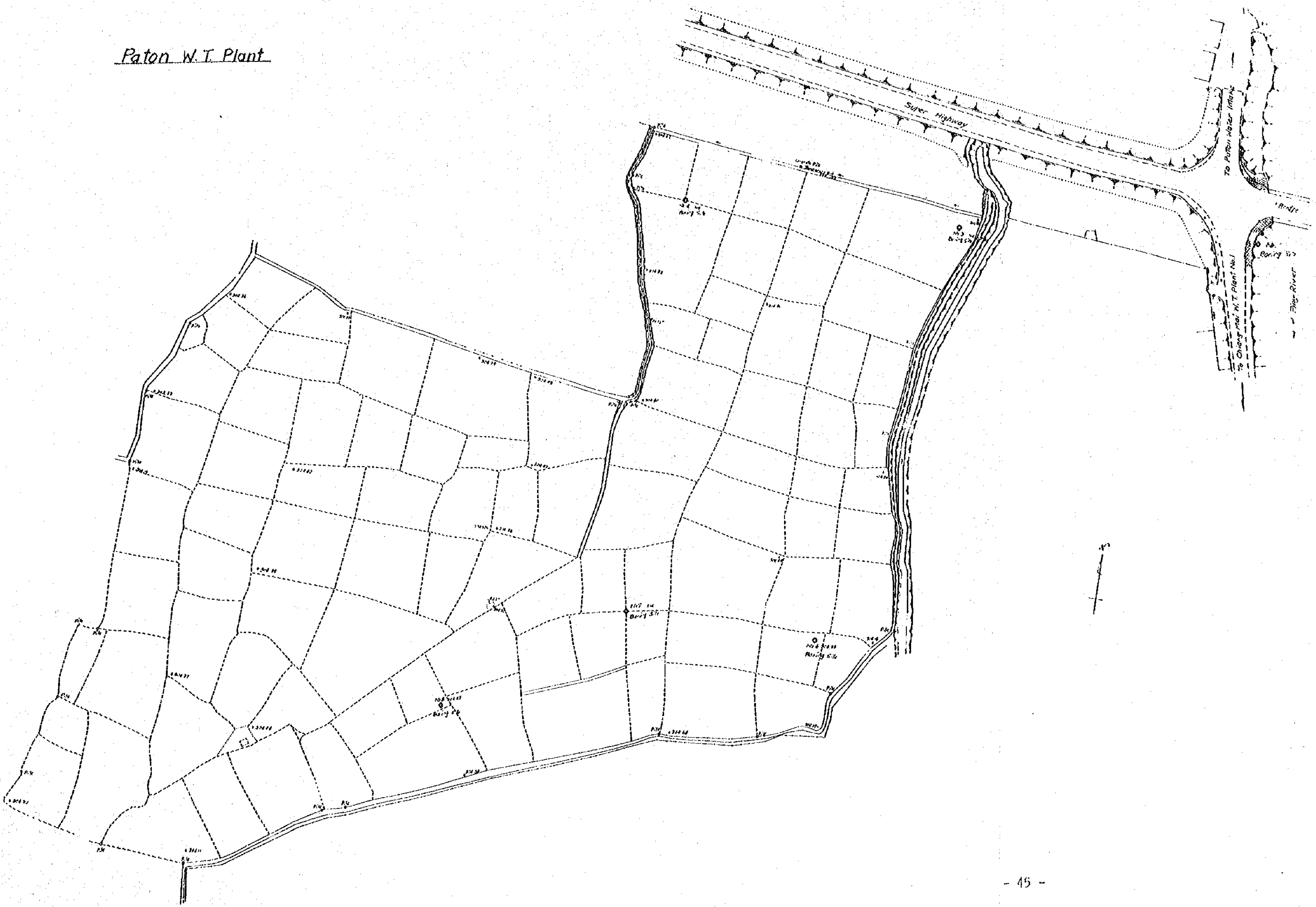
Core No. 9
Water Table -5.10 m

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color	Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
0.30	0.30		Surface Soil (Sandy Clay)	Light Brown					
1.00	0.70		Cleyey Fine Sand	Brown	1.20 – 1.50	7	2	2	3
2.20	1.20		Fine Sand	Brown	2.10 – 2.40	3	1	1	1
					3.15 – 3.45	9	2	3	4
				Blaish Gray	4.15 – 4.45	10	3	3	4
				~	5.15 – 5.45	14	4	4	6
					6.15 – 6.45	12	4	4	4
8.10	5.90		Clay	gray					
					8.15 – 8.45	41	12	14	15
					9.10 – 9.40	32	10	12	10
					10.10 – 10.40	37	12	12	13
					11.10 – 11.40	9	3	3	3
					12.15 – 12.45	39	13	[illegible]	13
					13.20 – 13.50	39	12	12	15
					14.20 – 14.50	43	13	15	15
15.50	7.40		Gravel	Gray	15.20 – 15.50	37	12	12	13




No. 3
L = 15.50m
N Value
No. 9
L = 15.50m
N Value
0 M
5
10
15
LEGEND
Surface Soil
Fine Sand
Clay
Gravel and Sand
Clayey

Paton W.T. Plant



Date of Investigation

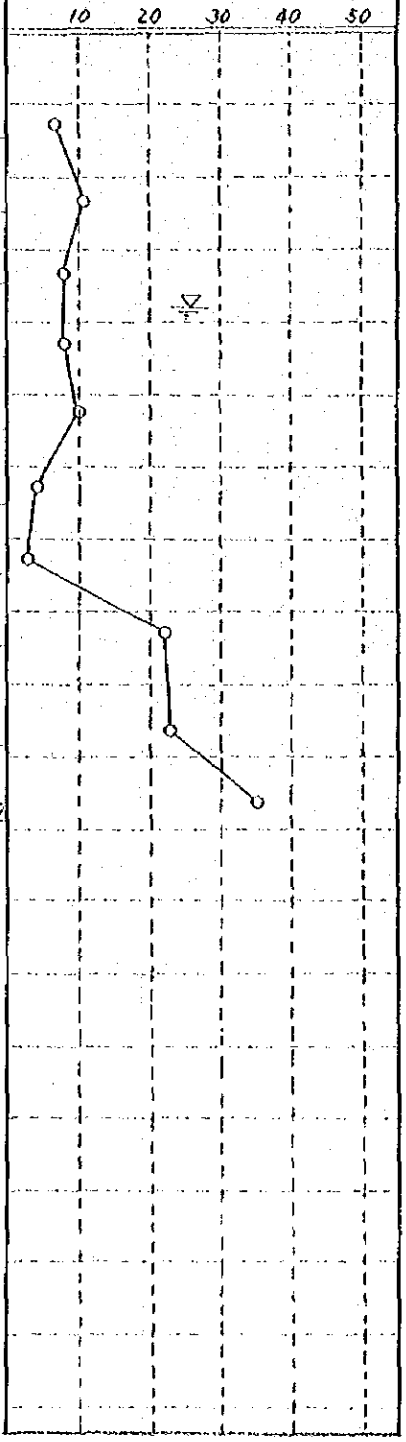
6th Jul. ~ 7th Jul. '72

Core No. 1

Water Table - 3.80m

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color	Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
1.00	1.00		Surface Soil (Gravelly Clay)	Brown	1.15 – 1.45	7	3	2	2
2.00	1.00		Fine Sand	Brown					
2.80	0.80		Coarse Sand ~ Medium Sand	Redish Brown	2.15 – 2.45	11	2	4	5
3.10	0.30		Sandy Clay	Dark Brown	3.15 – 3.45	8	3	3	2
4.80	1.70		Coarse Sand	Light Gray	4.15 – 4.45	8	3	2	3
6.00	1.20		Fine Sand ~ Medium Sand	Dark Gray	5.15 – 5.45	10	3	4	3
7.40	1.40		Clay	Gray	6.00 – 6.50	4/50	1/30	1	2
				Dark Gray	7.15 – 7.45	3	1	1	1
9.00	1.60		Coarse Sand	Light Gray	8.15 – 8.45	22	6	8	8
10.80	1.80		Gravelly Coarse Sand	Light Gray	9.50 – 9.80	23	8	6	9
					10.50 – 10.80	35	14	11	10

N Value (Blows/30cm)



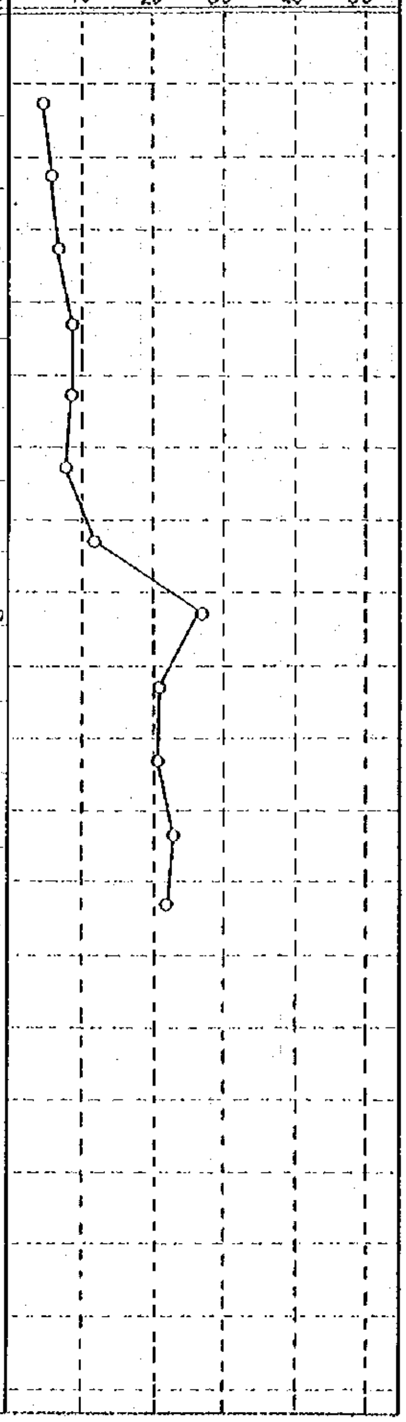
Date of Investigation: 17th Jul. ~ 18th Jul. '72

Core No. 4

Water Table

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color	Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
0.00									
0.40	0.40		Surface Soil (Sandy Clay)	Light Brown					
2.20	1.80		Medium Sand	Brown	1.15 ~ 1.45	5	1	2	2
					2.15 ~ 2.45	6	2	2	2
6.10	3.90		Clay	Blue Gray ~ Gray	3.15 ~ 3.45	7	2	3	2
					4.20 ~ 4.50	9	3	3	3
					5.15 ~ 5.45	9	2	3	4
					6.15 ~ 6.45	8	2	3	3
9.70	3.00		Silty Fine Sand ~ Medium Sand	Gray ~ Light Gray	7.15 ~ 7.45	12	5	4	3
					8.15 ~ 8.45	27	8	9	10
					9.20 ~ 9.50	21	6	6	9
12.50	2.80		Gravel and Sand	Light Gray	10.20 ~ 10.50	21	7	6	8
					11.20 ~ 11.50	23	7	8	8
					12.20 ~ 12.50	22	6	8	8

N Value (Blows/30cm)



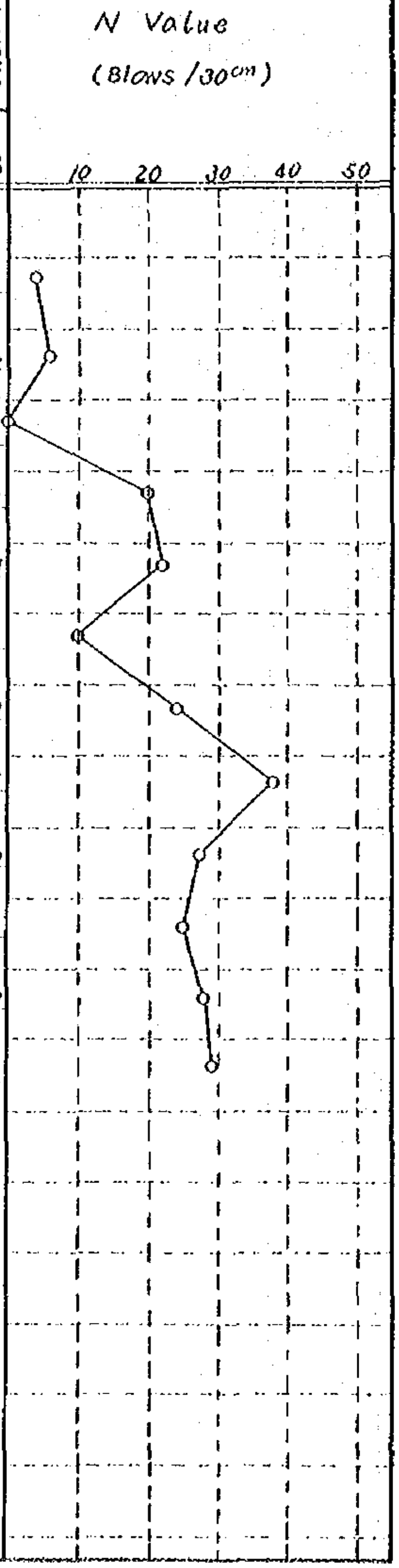
Date of Investigation

7th Jul. ~ 20th Jul. '72

Core No. 5

Water Table

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color	Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
0.00									
0.30	0.30		Surface Soil (Sandy Clay)	Light Brown					
0.70	0.40		Mediume Sand	Brown					
					1.15 / 1.47	4/32	2/12	1	1
					2.20 / 2.52	6/32	2	2	2/12
2.90	2.20		Sandy Clay	Bluish Gray					
					3.00 / 3.60	0/60	"		
					4.15 / 4.45	20	4	7	9
5.00	2.40		Clay	Bluish Gray ~ Blackish Gray					
					5.15 / 5.45	22	4	7	11
6.10	0.80		Silty Fine Sand	Bluish Gray	6.15 / 6.45	10	3	3	4
7.40	1.30		Clay	Bluish Gray	7.15 / 7.45	24	5	8	11
					8.15 / 8.45	38	11	12	15
					9.20 / 9.50	27	8	9	10
			Gravel and Sand	Light Gray	10.20 / 10.50	25	9	8	8
					11.20 / 11.50	28	9	9	10
12.50	5.10			Light Gray	12.20 / 12.50	29	9	9	11



Date of Investigation

22nd. Jul. ~ 25th Jul. '72

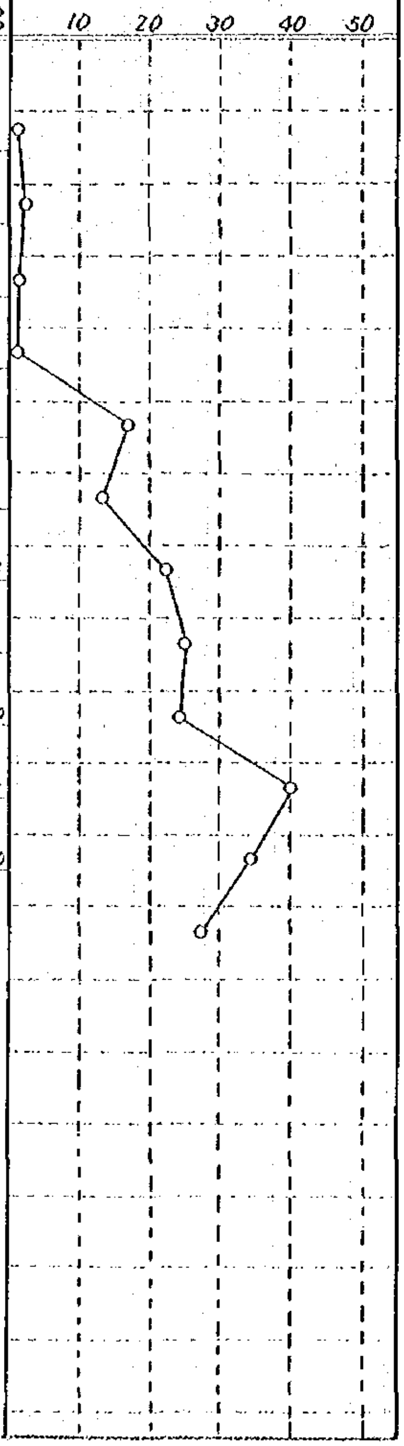
Core No. 6

Water Table

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color
0.00 – 0.30	0.30		Surface Soil (Sandy Clay)	Light Brown
2.70	2.40		Medium Sand	Light Brown
4.80	2.10		Clay	Bluish Gray / Light Gray
9.80	5.00		Medium Sand ~ Coarse Sand	Light Gray
12.50	2.70		Gravel and Sand	Gray

Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10 cm	20 cm	30 cm
1.00 – 1.50	1/50	1/50		
2.00 – 2.50	2/50	2/50		
3.15 – 3.50	1/35	1/35		
4.15 – 4.51	1/36	1/36		
5.20 – 5.50	17	5	5	7
6.15 – 6.45	13	4	4	5
7.15 – 7.45	22	7	7	8
8.15 – 8.45	25	8	8	9
9.15 – 9.45	24	6	8	10
10.15 – 10.45	40	13	14	13
11.15 – 11.45	34	13	11	10
12.20 – 12.50	27	10	10	7

N Value (Blows/30cm)



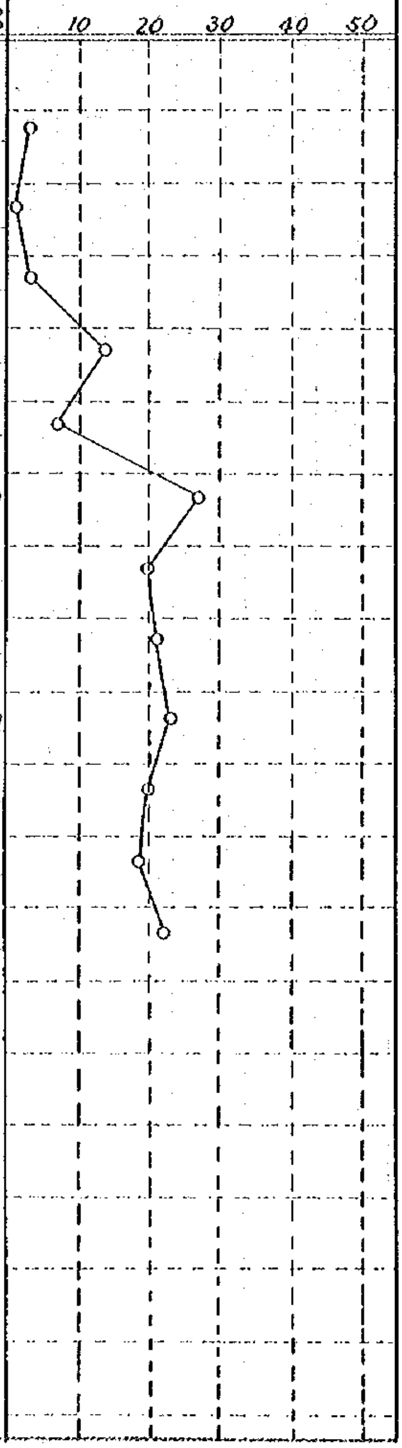
Date of Investigation 27th Jul. ~ 28th Jul '72

Core No. 7

Water Table

Depth (m)	Thickness of Stratum (m)	Legend	Classification of Soils	Color	Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10 cm: 10 cm	20 cm	30 cm
0.00 – 0.30	0.30		Surface Soil (Sandy Clay)	Light Brown					
1.80	1.50		Mediume Sand	Light Brown	1.10 – 1.40	3	1	1	1
				Blueish Gray	2.00 – 2.60	1/60	1/60		
				~	3.15 – 3.55	3/40	1/20	1	1
4.60	2.80		Clay	Light Gray	4.15 – 4.45	14	3	5	6
				Gray	5.15 – 5.45	7	2	2	3
				~	6.15 – 6.45	27	8	9	10
7.50	2.90		Fine Sand	Light Gray	7.15 – 7.45	20	6	6	8
					8.15 – 8.45	21	6	7	8
					9.20 – 9.50	23	6	7	10
			Gravel and Sand		10.20 – 10.50	20	6	7	7
					11.20 – 11.50	19	5	7	7
12.50	5.00			Gray	12.20 – 12.50	22	6	7	9

N Value (Blows/30 cm)



Date of Investigation

31st Jul. ~ 1st Aug. '72

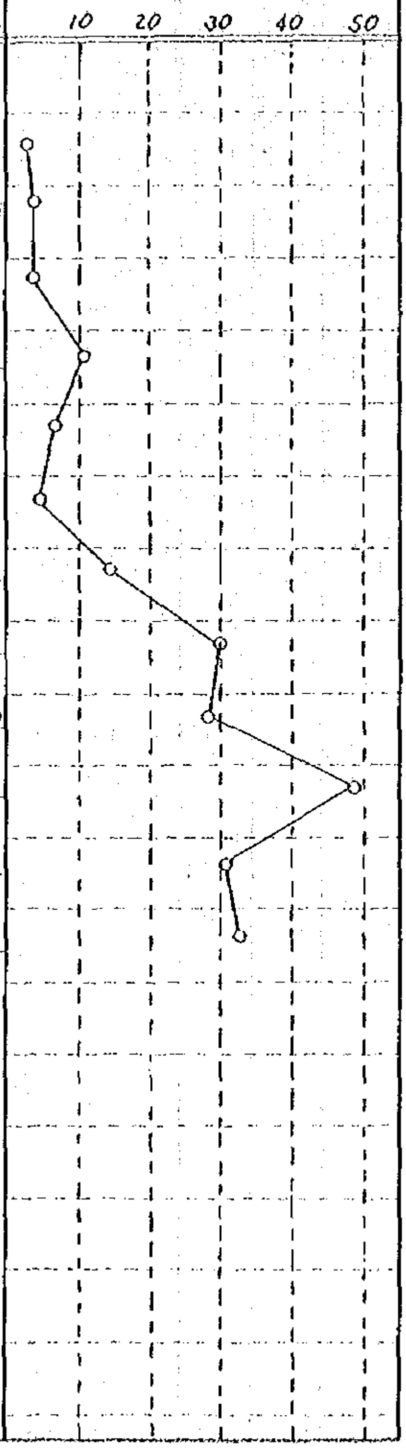
Core No. 8

Water Table

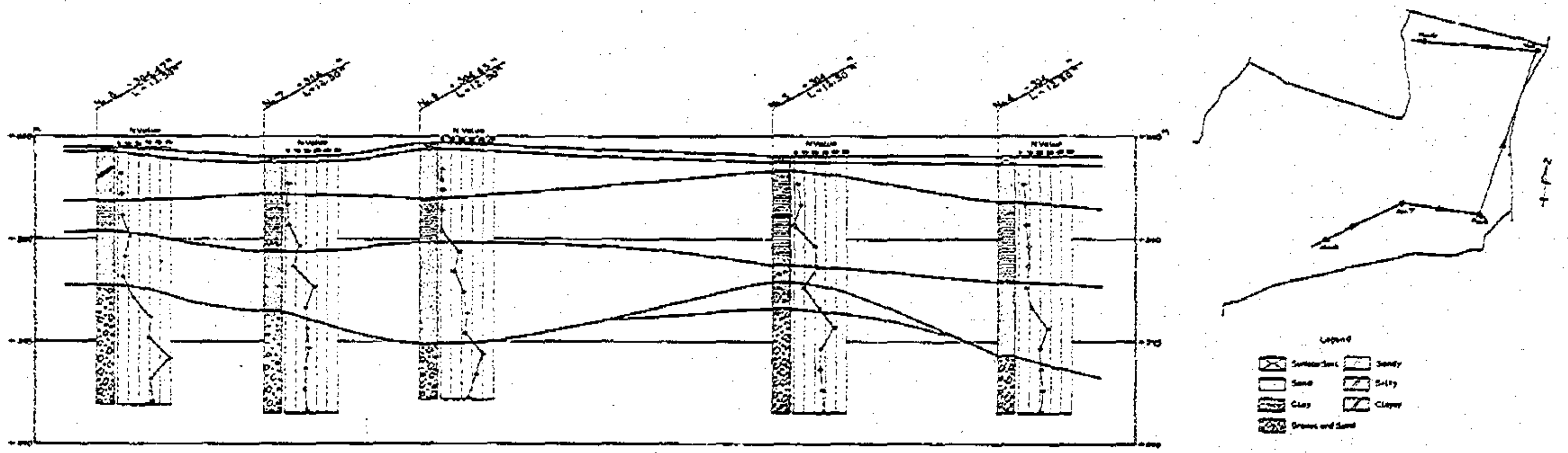
Depth (m)	Thickness of Stratum (m)	Classification of Soils	Color
0.00 – 0.20	0.20	Surface Soil (Sandy Clay)	Light Brown
2.60	2.40	Medium Sand with Clay	Light Brown
4.10	1.50	Clay	Blue Gray / Blackish Gray
6.70	2.60	Fine Sand	Gray ~ Light Gray
12.50	5.80	Gravel and Sand	Gray

Depth of Test (m)	N Value (Blows/30cm)	Number of Blows in each 10cm: 10cm	20cm	30cm
1.36 – 1.66	3	1/20	2	
2.00 – 2.40	4/40	1/15	2	1/15
3.10 – 3.48	4/38	1/18	1	2
4.20 – 4.50	11	4	3	4
5.15 – 5.45	7	2	2	3
6.20 – 6.50	5	1	1	3
7.15 – 7.45	15	4	5	6
8.15 – 8.45	30	9	10	11
9.15 – 9.45	29	10	9	10
10.10 – 10.40	49	15	17	17
11.20 – 11.50	31	9	10	12
12.20 – 12.50	33	10	10	13

N Value (Blows/30cm)



Drawing of Estimated Earth Formation



2-6 Investigation of Water Pressures

2-6-1 Background

There are two water supply (distribution) systems for the Chiang Mai waterworks -- the system for the No. 1 Water Treatment Plant at Wang Sing Kum and the system for the No. 2 Water Treatment Plant at Umong. Water is supplied directly from the No. 1 Water Treatment Plant by gravity flow from an elevated water tank (H.W.L. +328.80; L.W.L. +325.80), and from the No. 2 Water Treatment Plant by distributing pump with a lifting capacity of 44m, and a normal lift of 10~20m, the ground height above sea level being +336.6 ~ 346.6m. Moreover, the two water treatment plants are situated on two flanks of the Chiang Mai City water district and the two distribution systems are connected at their extremities, without being separated by sluice valves.

In order to investigate the condition of the water supply and of the division of the water supply districts between the two systems, an investigation of water pressures in the distribution main network was conducted between July 11 to July 29, 1972.

The method of investigation was to select 28 reference points or locations throughout the entire water supply area, and to install an automatic pressure recorder on faucets of houses as close to each point as was feasible. The water pressure was measured continuously for 24 hours. A certain degree of error was unavoidable in the results of this investigation, because measurements of water pressure was not possible simultaneously throughout the entire supply district, because of errors arising from differences in diameters of pipes and distances between the actual installation point of the various recorders and the selected reference points, and due to errors in the measurements by the recorders. However, the values are considered to be of practical use, save for some measurements which varied.

2-6-2 Evaluations Based on Results of the Investigation of Water Pressures

The following conclusions concern only the period up to the completion of the waterworks expansion for Stage 1.

2-6-2-1 The area division of load between the two distribution systems was measured at points as indicated in the drawing for Water Pressure Investigation.

a) Points surveyed within the system of the No. 1 Water Treatment Plant total 15, and include Nos. 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, and 28.

b) Points included within the system for the No. 2 Water Treatment Plant total 13, and include Nos. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13.

In regard to the extent of the water supply area for the No. 1 Water Treatment Plant System, the above two distribution systems are not separated by valves for the entire area along the left bank of the Ping River nor for the district on the right bank. Thus the supply boundaries tend to shift, although they are locatable approximately between points 11 to 16, 10 to 15, and 9 to 14.

The scope of the supply area for the No. 2 Water Treatment Plant system includes all areas outside the No. 1 system.

2-6-2-2 The hydrostatic pressures and allowable strengths for pipes underground are given below for the 28 surveyed points.

Survey Point	Survey Value for Hydrostatic Pressure (kg/cm^2)	Value Converted To Heights Above Sea Level (m)	Allowable Strength for Underground Pipe (kg/cm^2)	Excess or Deficiency (kg/cm^2)
(14)	2.00	+323.75	4.00	+2.00
(15)	2.10	+327.41	4.00	+1.90
(16)	0.95	+315.66	4.00	+3.05
(17)	2.40	+329.00	4.00	+1.60
(18)	2.50	+328.86	4.00	+1.50
(19)	2.45	+331.00	4.00	+1.55
(20)	2.20	+327.21	4.00	+1.80
(21)	2.60	+322.27	4.00	+1.40
(22)	2.60	+329.00	4.00	+1.40
(23)	2.50	+328.73	4.00	+1.50
(24)	2.50	+329.36	4.00	+1.50
(25)	2.45	+327.62	4.00	+1.55
(26)	1.00	+313.72	4.00	3.00
(27)	2.40	+328.49	4.00	1.60
(28)	2.50	+330.25	4.00	1.50

Survey Point	Survey Value for Hydrostatic Pressure (kg/cm^2)	Value Converted To Heights Above Sea Level (m)	Allowable Strength for Underground Pipe (kg/cm^2)	Excess or Deficiency (kg/cm^2)
(1)	1.40	+332.00	5.00	3.60
(2)	1.10	+329.09	5.00	3.90
(3)	1.20	+326.50	5.00	3.80
(4)	3.15	+342.36	5.00	1.85
(5)	1.42	+329.03	5.00	3.58
(6)	2.00	+323.67	5.00	3.00
(7)	2.20	+329.14	5.00	2.80
(8)	1.80	+327.16	5.00	3.20
(9)	2.20	+328.40	5.00	2.80
(10)	2.15	+329.29	5.00	2.85
(11)	1.70	+327.57	5.00	3.30
(12)	1.80	+328.49	5.00	3.20
(13)	2.20	+330.64	5.00	2.80

The highest hydrostatic pressures measured at any point within the No. 1 Water Treatment Plant system was 2.6 kg/cm^2 at point No. 22 on the right bank of the Ping River, and at point No. 21 on the left bank, also 2.6 kg/cm^2. The existing mains in this area are AC - Class 15 and AC - Class 12. GS pipe, and even the Class 12 (which has the smallest allowable strength against hydrostatic pressures) has a leeway for safety of 1.4 kg/cm^2. The hydrostatic pressures, thus, must not be allowed to exceed 4.0 kg/cm^2.

The highest hydrostatic pressures measured at any one point within the No. 2 Water Treatment Plant system was 3.15 kg/cm^2, found at point No. 4. The existing mains in this area are AC - Class 15 GS pipe, providing a leeway of 1.85 kg/cm^2. Here, hydrostatic pressures must not be allowed to exceed 5.0 kg/cm^2.

2-6-2-3 Minimum Hydraulic Pressures and Appropriate Effective Hydraulic Pressures for the Various Points

The values charted for appropriate hydraulic pressures are the residual pressures when water is sent the distance from the water treatment plant to the specified point, through appropriate distributing mains.

Survey Point	Minimum Hydraulic Pressure (kg/cm^2)	Value Converted To Heights Above Sea Level (m)	Appropriate Minimum Hydraulic Pressures (m)	Excess or Deficiency (kg/cm^2)	Notes
(O_1)		+325.80			
(20)	0.93	+314.51	+323.70	-9.19	
(18)	1.30	+316.86	+319.46	-2.60	
(14)	1.00	+313.75	+318.70	-4.95	
(16)	0.14	+307.56	+322.50	-14.94	Large measurement error
(15)	1.00	+316.41	+320.20	-3.79	
(19)	1.65	+323.00	+321.60	+1.40	
(22)	1.70	+320.00	+318.60	+1.40	
(17)	0.50	+310.00	+320.00	-10.00	Large measurement error
(21)	1.73	+322.27	+323.00	-0.73	
(24)	1.80	+322.36	+321.85	+0.51	
(23)	1.70	+320.73	+320.26	+0.47	
(25)	1.20	+315.12	+318.90	-3.78	
(26)	0.15	+305.22	+317.51	-12.29	Large measurement error
(27)	1.85	+322.99	+320.91	+2.08	
(28)	1.80	+323.25	+320.06	+3.19	
(O_2)		+338.00			
(1)	0.44	+322.40	+333.00	-10.60	Large measurement error
(4)	1.50	+325.86	+328.29	-2.53	
(8)	0.60	+315.16	+326.44	-11.28	Large measurement error
(7)	0.90	+316.14	+323.82	-7.68	
(9)	1.05	+316.40	+321.58	-5.18	
(6)	1.10	+314.67	+321.58	-6.91	
(2)	0.25	+320.59	+331.06	-10.47	Large measurement error
(3)	0.60	+320.50	+329.49	-8.99	
(5)	0.60	+320.83	+327.28	-6.45	
(11)	0.90	+319.57	+326.08	-6.51	
(12)	1.20	+322.49	+324.58	-2.09	
(13)	1.40	+322.64	+323.56	-0.92	
(10)	1.20	+319.79	+324.62	-4.83	

b) Comparison of the surveyed or measured values with the appropriate minimum hydraulic pressures is as follows:

Within the No. 1 Water Treatment Plant system, there are insufficient hydraulic pressures at points 20, 18, and 14 south of the treatment plant, and lying in the city center. Also, there are insufficient hydraulic pressures near the railway station at points No. 25 and No. 26 on the left bank of the Ping River.

Hydraulic pressures are inadequate throughout the entire area served by the No. 2 Water Treatment Plant system. The cause of this deficiency is that the operating lift of the distributing pumps is reduced to 1.2 kg/cm^2 by the lift operation for No. 2 Water Treatment Plant during the set times for measuring water pressures.

2-6-3 Adjustment or Consolidation of the Two Existing Water Supply Systems

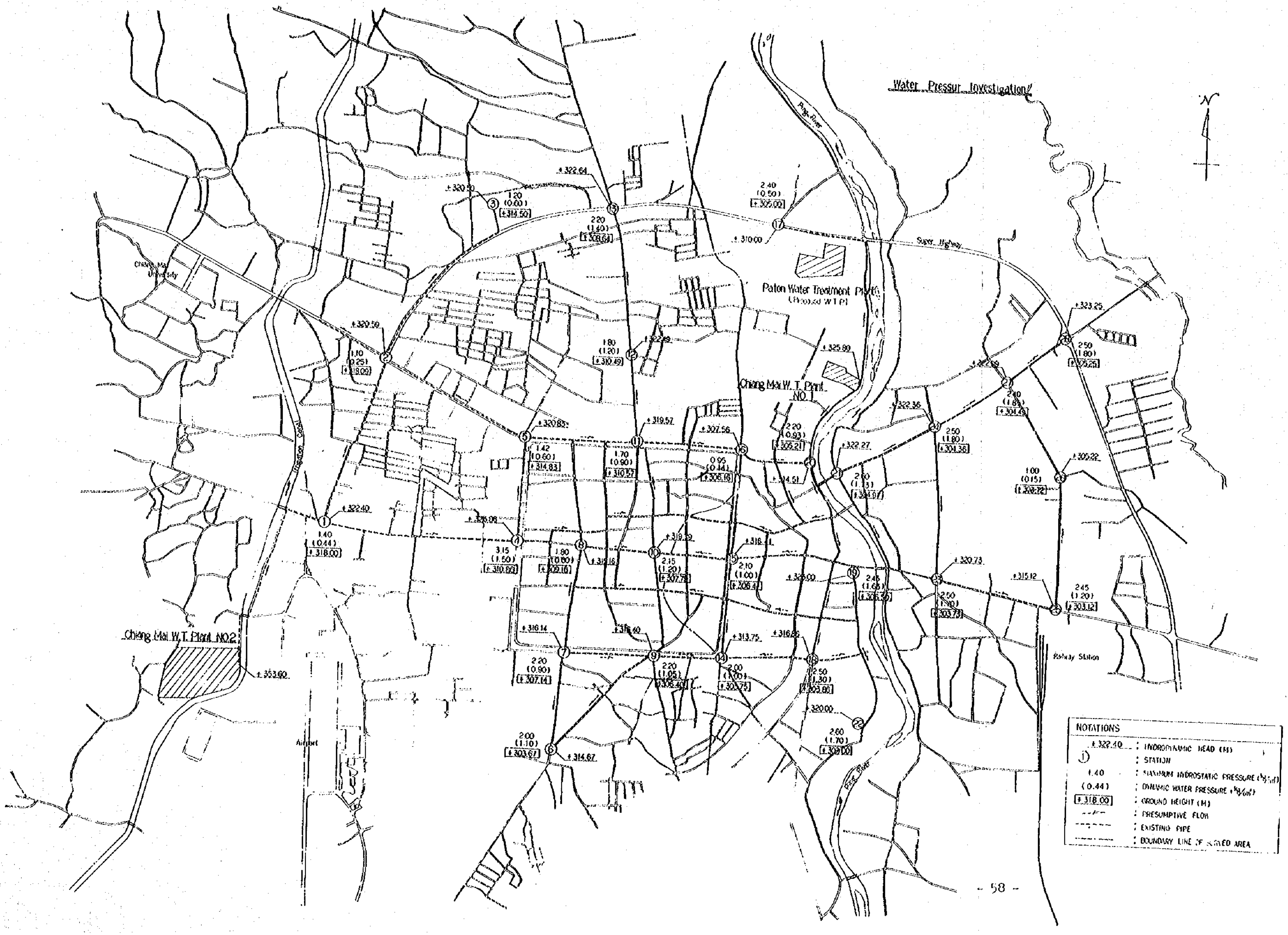
2-6-3-1 Urgently Required Measures to Augment the Distributing Trunk Main

The basic cause of the inadequate hydraulic pressures in the area on the right bank of the Ping River, served by the No. 1 Water Treatment Plant system, is that the existing mains of 300mm, 250mm, and 200mm diameter, between the No. 1 Water Treatment Plant and points No. 18 to 20, are not large enough. Therefore, it is necessary to increase the mains' capacities by laying new pipe.

Also, the deficiency in hydraulic pressures in the vicinity of the railway station on the left bank of the Ping River may be corrected by laying a trunk main between points 27, 26, and 25.

2-6-3-2 Main Networks Requiring Expansion in the Future

There is a deficiency in hydraulic pressures at all points on the No. 2 Water Treatment Plant supply system. But as stated above, this deficiency is caused by limiting to 12m the actual lift of the pumps, the rated lift of which is 44m. Provided the operating lift is increased to about 25m, the deficiency in water pressure will be corrected and also the allowable hydrostatic pressure will not exceed 5.0 kg/cm^2.



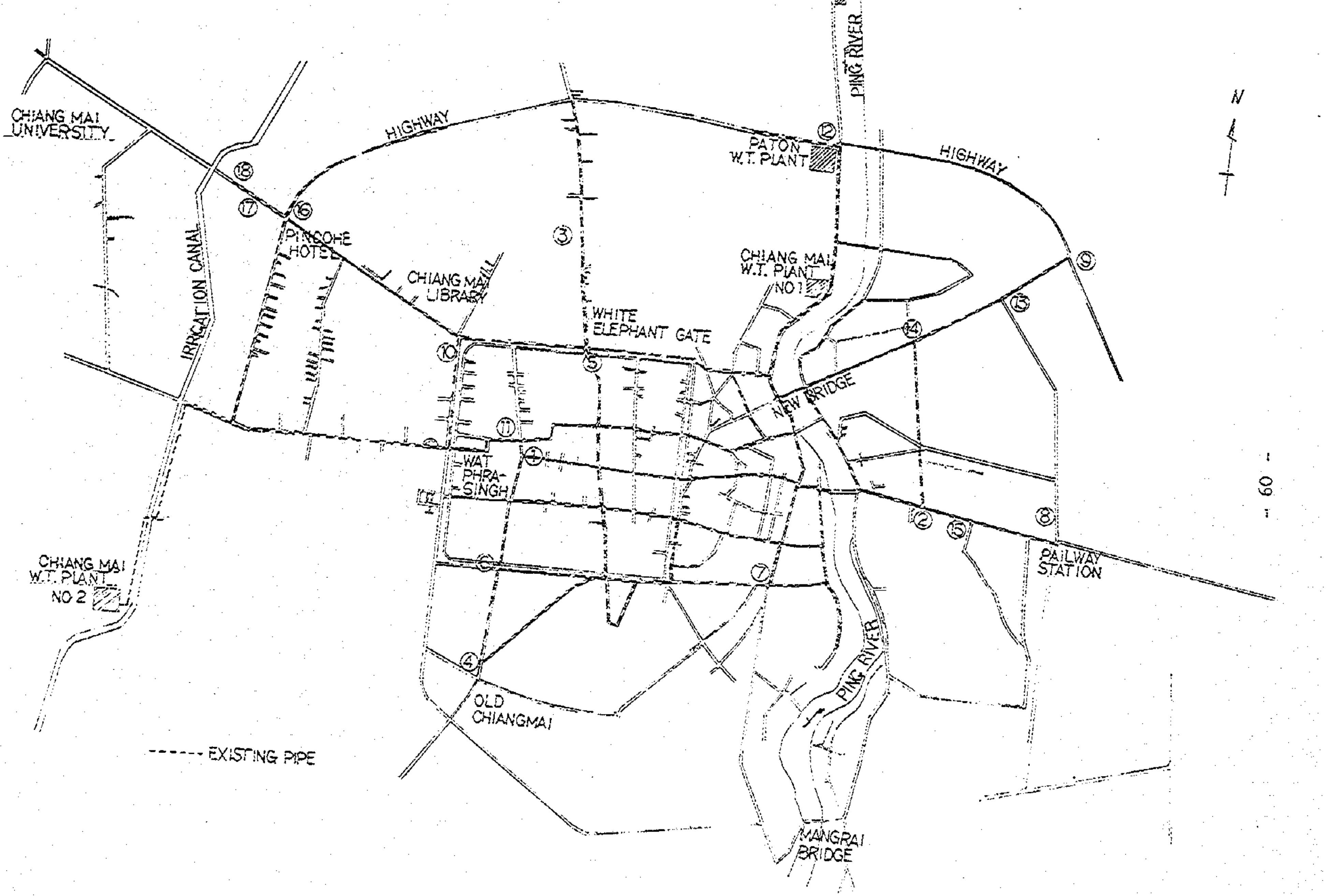
Water Pressur Investigation
N
Ping River
Super Highway
Chiang Mai University
Paton Water Treatment Plant
Chiang Mai W.T. Plant NO.1
Chiang Mai W.T. Plant NO.2
Railway Station
Airport
NOTATIONS
±322.40 : HYDRODYNAMIC HEAD (M)
① : STATION
1.40 : MAXIMUM HYDROSTATIC PRESSURE (kg/cm²)
(0.44) : DYNAMIC WATER PRESSURE (kg/cm²)
±318.00 : GROUND HEIGHT (M)
PRESUMPTIVE FLOW
EXISTING PIPE
BOUNDARY LINE OF SERVED AREA

2-7 Investigation of Underground Pipes

Reference data was assembled for use wherever connections between the newly planned mains and existing mains might be necessary. The survey involved reference to blue prints of distributing main routes remaining in Thai records, and investigation accompanied by personnel at the water works familiar with the locations of underground mains. Digging, using local labor, was done at feasible points where the mains were not below paved roads. Plane view locations, depth, type of pipe, and pipe diameters were ascertained.

The investigation showed that Class 10, Class 12, and Class 15 asbestos-cement pipe were used but it proved difficult to distinguish these classes according to route, and this data is not clearly recorded.

LAYOUT



------ EXISTING PIPE

Chapter 3 CONSTRUCTION SCHEDULE

Facilities	1st Stage	2nd Stage	3rd Stage
Paton Intake Plant			
Grit Chamber	4.5(B) x 1.5(ED) x 15.0(L)m, Cap. 202.5 m^3/2 chambers Retention Time: 11.5 min.	Ret'n.Time: 6.8 min.	Ret'n.Time: 4.8 min.
Raw Water Pump Well	4.5 x 13.0 x 1.5(ED) m, 1 well	----	----
Raw Water Pump Room	4.9(B) x 14.4(L) m, 1 room	----	----
Raw Water Pump	Serving the No. 1 Water Treatment Plant: 5.35 m^3/min. x 48m x 80 Kw 2 sets (1 Stand-by) Type: Vertical Shaft Mixed-Flow Pump	----	----
	Serving the Paton Water Treatment Plant: 12.22 m^3/min. x 43m x 150 Kw 2 sets (1 Stand-by) Type: Vertical Shaft Mixed-Flow Pump	(Same as for 1st Stage, but 1 set)	(Same as for 1st Stage, but 1 set)
Transformer Equipment	500 KVA, 1 set	(Same as for 1st Stage)	----
Generator Room	12.0(B) x 24.0(L) m, 1 room	----	----
Generator Equipment	500 KVA, 1 set	(Same as for 1st Stage)	----
Lodging House	9.0(B) x 16.2(L) m, 1 bldg. for 3 families, 2-story high	----	----
Warehouse	4.0(B) x 10.0(L) m, 1 bldg.	----	----
Raw Water Main	Serving the No. 1 Water Treatment Plant: 300mm dia. x 4190m, ACP	----	----
	Serving the Paton Water Treatment Plant: 400mm dia. x 3110m, DCIP	(Same as for 1st Stage)	(Same as for 1st Stage)

Facilities	1st Stage	2nd Stage	3rd Stage
No. 1 Water Treatment Plant			
Receiving Well	4.4m dia. x 6.2m depth, Cap. 31 m^3, 1 well Ret'n. Time: 6.5 min.	----	----
Paton Water Treatment Plant			
Receiving Well	6.5m dia. x 5.5 m depth Cap. 183 m^3, 1 well Ret'n.Time: 16.5 min.	Ret'n.Time: 11 min.	Ret'n.Time: 5.5 min.
Flush Mixing Basin	1.9 x 1.9 x 2.1(ED) m, 2 basins, Cap. 7.6 m3/basin Ret'n.Time: 1.4 min. Flush Mixer, 0.75 Kw, 2 sets	(Same as for 1st Stage)	(Same as for 1st Stage)
Flocculation Basin	4.6(B) x 9.3(L) x 2.4(ED) m, 4 basins, Cap. 103 m^3/basin, Ret'n.Time: 37.0 min., Flocculator, 0.75 Kw, 8 sets	(Same as for 1st Stage)	(Same as for 1st Stage)
Sedimentation Basin	9.1(B) x 34.0(L) x 3.5(ED) m, 2 basins, Cap. 1080 m3/basin Ret'n.Time: 3.25 hrs. Sludge scraper, 0.75 Kw, 1 set Sludge Pump, 15 Kw, 2 sets (1 Stand-by)	(Same as for 1st Stage)	(Same as for 1st Stage)
Rapid Sand Filter	6.4(B) x 5.0(L) m, 6 chambers (1 Stand-by), Filter Area: 32 m^2/chamber, Filtration Ratio: 100 m/day	(Same as for 1st Stage)	(Same as for 1st Stage)
Operations Rm. (above Filter)	20.0 m x 20.0 m, 1 room	(Same as for 1st Stage)	(Same as for 1st Stage)
Clear Water Reservoir	4.7m x 19.8m x 2.25m(D), 2 basins, Cap. 209 m^3/basin, Ret'n.Time: 0.6 hrs.	(Same as for 1st Stage)	(Same as for 1st Stage)
Chemical Dosing Room	10.0(B) x 30.0(L) m, 1 building, 2-story high	----	----

Facilities	1st Stage	2nd Stage	3rd Stage
Chemical Dosing Facilities	Alum Dosing Equipment: Injection pump, 0.4 Kw, 2 sets (1 Stand-by) Solution tank; Cap. 1745 liter, 2 tanks	Injection pump, 0.4 Kw, 1 set Solution tank (same as for 1st Stage)	Injection pump, 0.4 Kw, 1 set Solution tank (same as for 1st Stage)
	Lime Soda Feeder Equipment: Injection pump, 0.75 Kw, 2 sets (1 Stand-by) Solution tank: Cap. 2500 liter, 2 tanks	---- Solution tank (same as for 1st Stage)	---- ----
Chlorine Dosing Facilities	Chlorine Dosing Equipment: Pressure pump, 0.75 Kw, 2 sets (1 Stand-by) Chlorinator (Injector), q. max = 10 kg/hr., 2 sets (1 Stand-by)	(Same as for 1st Stage, but only 1 set each of pressure pump and chlorinator	(Same as for 1st Stage, but only 1 set each of pressure pump and chlorinator
	Chlorine Neutralization Equipment: Treating Capacity, 100 kg/hr., 1 set	----	----
Elevated Tank	HWL: +318.00, LWL: +315.50 Cap. 156 m^3	----	----
Backwashing Lift Pump	2.0 m3/min. x 19m x 15 Kw, 2 sets (1 Stand-by), Type: Vertical Shaft Volute Pump	(Same as for 1st Stage, but 1 set)	----
Clear Water Reservoir	24.0(B) x 48.0(L) x 4.0(ED) m, 1 basin, Cap. 4610 m^3, Ret'n.Time: 6.9 hrs.	(Same as for 1st Stage)	(Same as for 1st Stage)
Distribution Pump Well	8.0(B) x 24.0(L) x 4.0(ED) m, 1 chamber, Cap. 770 m^3, Ret'n.Time: 1.15 hr.	Ret'n.Time: 0.58 hr.	Ret'n.Time: 0.39 hr.
Distribution Pump Room	8.0(B) x 24.0(L) m, 1 room	----	----

Facilities	1st Stage	2nd Stage	3rd Stage
Distribution Pump	8.4 m^3/min. x 43.0m x 100 Kw, 2 sets (1 Stand-by) Type: Vertical Shaft Mixed-Flow Pump 4.2 m^3/min. x 43.0 m x 55 Kw, 2 sets Type: Vertical Shaft Mixed-Flow Pump	(Same as for 1st Stage, 2 sets) ----	(Same as for 1st Stage, 2 sets) ----
Transformer Equipment	500 KVA, 1 set	(Same as for 1st Stage)	(Same as for 1st Stage)
Generator Room	12.0(B) x 24.0(L) m, 1 room	----	----
Generator	500 KVA, 1 set	(Same as for 1st Stage)	(Same as for 1st Stage)
Office	13.0(B) x 54.0(L) m, 1 room	----	----
Lodging House	9.0(B) x 63.4(L) m, 1 bldg., with 11 family units, 2-story	----	----
Warehouse	4.0(B) x 10.0(L) m, 1 bldg.	----	----
Distribution Facilities			
Distribution Main	DCIP Dia.(mm) / Type / Length (m) 600 / Type 3 / 73.0 500 / " " / 1757.0 450 / " " / 286.0 400 / " " / 1682.5 350 / " " / 3659.0 300 / " " / 25.0 150 / " " / 14.4 100 / " " / 109.0 Effective Length: 7605.9	DCIP Dia.(mm) / Type / Length (m) 600 / Type 3 / 1870.0 500 / " " / 2180.0 450 / " " / 1150.0 350 / " " / 1700.0 Effective Length: 6900.0	

Facilities	1st Stage	2nd Stage	3rd Stage
Distribution Main (cont.)	ACP Dia.(mm) Type Length (m) 300 Class 15 2616.5 250 " " 5296.0 200 " " 7784.5 150 " " 50.0 100 " " 4.0 75 " " 83.2 Effective Length: 15834.2 GSP Dia.(mm) Length (m) 350 10.0 300 42.0 250 153.5 200 407.5 75 17.0 Effective Length: 630.0	ACP Dia.(mm) Type Length (m) 300 Class 15 12830.0 250 " " 15540.0 200 " " 12100.0 150 " " 10500.0 100 " " 6900.0 Effective Length: 57870.0	
Aqueduct Piping or Mains	SP Dia. 450mm, Length 140.10m	---	---
Booster Pump Chamber	2.2 x 3.0 x 1.485(D) m, 1 chamber	---	---
Booster Pump	1.85 m^3/min. x 10.0m x 7.5 KW, 2 sets (1 Stand-by), submerged motor pump	Replace 1st Stage pumps with 2.46 m^3/min. x 20.0m x 15 Kw, 2 sets (1 Stand-by), submerged motor pump	

Chapter 4 **DESIGN**

4-1 Conditions for the Design Work

4-2 Detail Design

Chapter 4 Design

4-1 Conditions for the Design Work

4-1-1 Conditions of the Design Work for the Various Facilities

4-1-1-1 Intake Facilities

Providing against a foreseeable lowering of the bed of the Ping River at the intake point, the intake shall be constructed 1.8 meters below the present elevation of the bed, which is 303.2 meters.

The raw water intake channel shall also serve as a grit chamber. Consequently, the various water levels are set as follows:

Designed high water level .... 308.0 m
Present low water level .... 303.8 m
Future low water level .... 302.0 m

Discharge of settled sand shall be by siphon pump. For convenience, there shall be provided a portable type pump which may be carried to the site. The intake pump shall be a vertical mixed-flow type, Both the priming pump and the intake pump shall be turned on and off by manually-operated push buttons.

4-1-1-2 Facilities of the Raw Water Mains

Velocity coefficient C (Hazen & Williams Formula), used in calculations of distribution pipe, shall be set at 100, in consideration of future corrosion of pipe.

Two raw water mains shall be laid, one to the existing purification plant, and one to the newly designed purification plant.

Where the mains cross the highway, two casing pipe lines shall be installed, each containing one raw water main. Connections for these two lines shall be on the north side of the highway, and connecting valves shall be installed.

The design endeavors to utilize, to the fullest extent, existing piping as connective raw water mains at the existing purification plant. Thus the design links existing raw water delivery pipe with new raw water mains, and existing raw water mains with new receiving wells.

4-1-1-3 Purification Facilities

a) Designated Water Level

The H.W.L. (high water level) of the supply reservoir shall be 306.0 m with a crown at 307.0 m. These values are so determined because the maximum flood high water level at the newly designed purification plant is estimated to be 306.0 m.

b) Location Plan for the Purification Plant

Construction for Stage 1, including water purification facilities with a capacity of 16000 m^3/day and the supply reservoir, shall be located together as one group.

Construction for Stage 2 and for Stage 3 shall be located in similar separate groups.

However, the chemical dosing chamber, the generator room, the control room, and the distribution pump room shall be allotted space sufficient for the facilities at the conclusion of Stage 3, and shall be housed in the same building.

c) Construction of the Lagoon

Waste water from washing the rapid sand filter beds and sludge from the sedimentation basins shall be collected in a lagoon. Clear waters from the upper layer of the lagoon shall be pumped into the Ping River, while the sludge shall be taken out of the plant and disposed of.

d) Foundation Work

The sedimentation basin and the receiving well shall be supported by concrete piling. However, a sand mat at least two meters thick shall be laid down and rendered firm and solid, after which the bottom slab of the sedimentation basin shall be built on top.

For the bottom of the rapid sand filter basin, concrete piles shall penetrate through a layer of clay, and shall be supported by a bottom layer of gravel.

The clear water reservoir shall be constructed with a bottom consisting of a sand mat, spread evenly and rendered firm. Concrete piles shall not be used.

e) Agitator Equipment

Because of breakdowns of the bearing seat for the underwater shafts of horizontal-type flocculators, necessitating frequent replacement, a vertical-type slow action flocculator shall be used instead.

f) Sludge Discharge from the Sedimentation Basin

The sludge shall be assembled by a power-driven scraper into a pit in one corner of the basin. This sludge shall be discharged along with water by a sludge pump into an open channel leading to the lagoon.

g) Washing Equipment for Filter Basins and the Washing Process

Equipment for underdrain system of gravity-type rapid sand filter basins shall employ perforated pipes and be equipped for back washing and surface washing.

Surface washing shall be of the fixed type. Surface washing water shall be distributed from the header pipe for the distribution pumps after reducing the water to proper pressure by means of a reducing valve. Back washing water shall be sent from the elevated tank by gravity. Filter controllers shall be installed, and washing shall begin when these controllers set off an alarm. The on-off operation shall be manual, consisting of opening or closing valves.

h) Housing of Filter Basins

1) Housing shall prevent small insects from getting into the basins.
2) Housing shall shield the filters from direct rays of the sun, which are undesirable because they aid the propagation of bacteria, and cause clogging of the filter surfaces.

i) Chemical Dosing Equipment

Since turbidity and alkalinity are comparatively high, during an initial very short period of time, dosing shall be done using both lime and alum, after which alum only shall be used. Dosing shall be done by dissolving in water solid alum, storing the solution in a storage tank, and pumping it into the raw water by means of a dosing pump. The dosing pump shall be a plunger pump, operated according to readings from the turbidity meter on the meter panel of the dosing chamber as well as readings from the flow meter in the raw water main.

Although dosing equipment shall be designed only for post-chlorination, pre-chlorination can be given future consideration if required, by installing connecting pipes.

j) Connecting Mains at the Plant Site

Class 3 DCIP shall be used for connecting mains at the purification plant, and Class 15 ACP pipe for the waste water discharge mains. For 400 mm or larger, butterfly-type valves shall be used instead of sluice valves.

4-1-1-4 Distribution Facilities

a) Distribution Pump

Distribution pumps shall be manually-operated, vertical mixed-flow type. Pumps shall be operated according to the distribution pressure and clear water volume indicators at the control center.

b) Distribution Mains

ϕ 300 mm or smaller mains shall be Class 15 ACP; ϕ 350 mm or larger mains shall be class 3 DCIP.

Moreover, the earth cover shall conform with standards, and in case the earth cover should be inadequate at places where valves are installed, special shape pipes shall be used, and the head of the spindle adjusted so that it is not exposed. Control valves over 500 mm in diameter shall be butterfly-type valves.

4-1-1-5 Electric Metering Equipment

a) Power Supply and Type of Electricity

Type of Power Supply

3ϕ3W11KW, 1 phase overhead line (50 Hz)

Type of Electricity

Low Voltage: 380V/230V 3ϕ 4W

Illumination: 380V/230V 1ϕ 3W

b) Auxiliary Source of Electricity

Power supply shall be a one-phase line, with a private plant generator installed as an auxiliary source.

Capacity of Plant Generator:	Capacity to operate at full load
Plant Generator:	Diesel engine alternating current generator
Fuel:	A-type heavy oil

c) Method of Operation

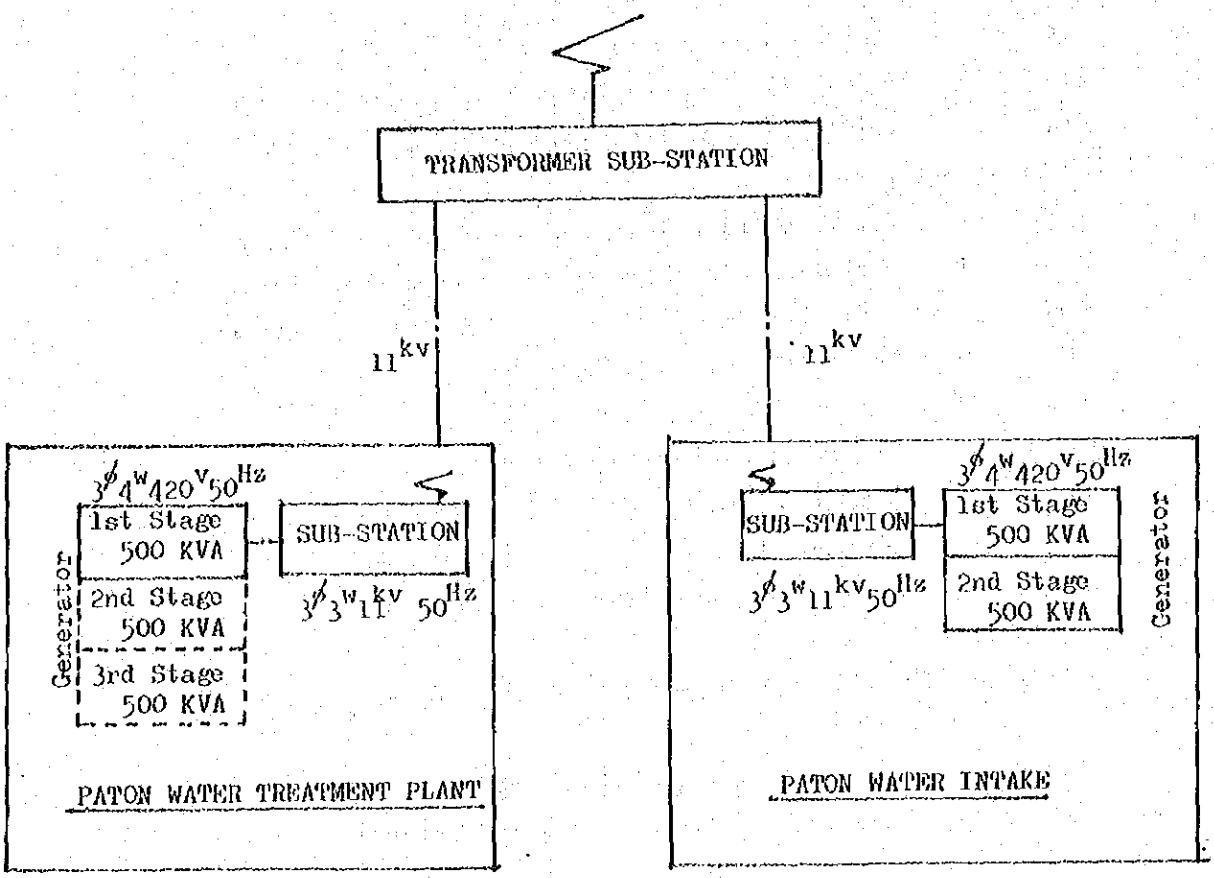
All equipment shall be operable as a unit by turning on or off by hand but the sedimentation basin sludge pump shall be capable of automatic operation by means of a timer.

d) Flow Sheet

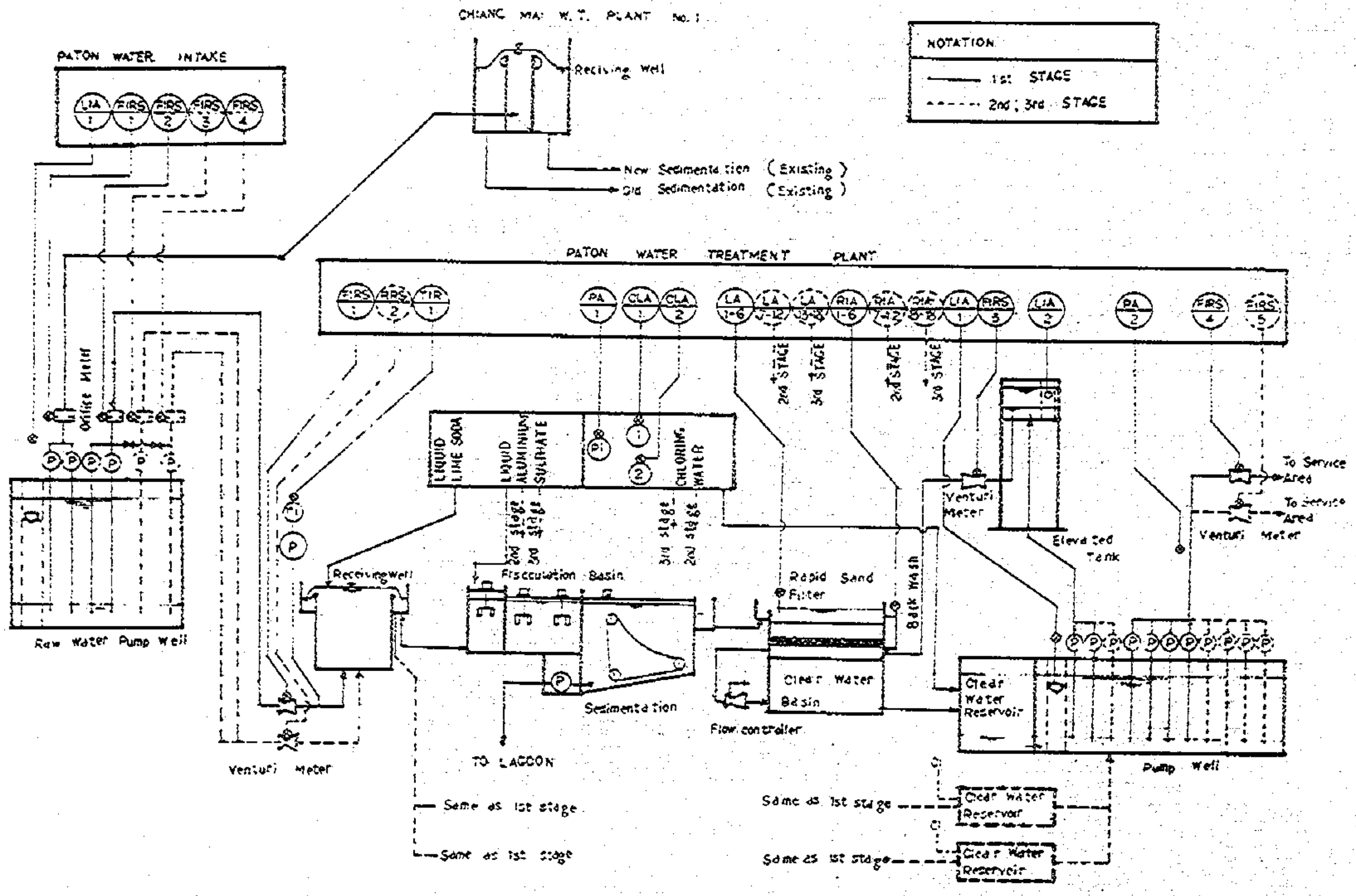
1) Meter flow sheets for this plan are as follows:

Intake Area		
LIA	1	Raw Water Pump Well Level Indicator Alarm
FIRS	1	Raw Water Flow Indicator/Recorder/Integrator
FIRS	2-4	Raw Water Flow Indicator/Recorder/Integrator
Water Treatment Plant		
FIRS	1-2	Raw Water Flow Indicator/Recorder/Integrator
FIRS	3	Backwash Water Flow Indicator/Recorder/Integrator
FIRS	4-5	Distributing Water Flow Indicator/Recorder/Integrator
TIR	1	Turbidity Indicator/Recorder
PA	1	Chlorine Gas Pressure Alarm
PA	2	Distribution Pressure Alarm
CLA	1	Leakage Alarm for Chlorine Gas Cylinder Storage Room
CLA	2	Leakage Alarm for Chlorine Dosing Room
LA	1-6, 7-12 13-18	Water Level Meter for Rapid Sand Filter
RIA	1-6, 7-12 13-18	Rapid Sand Filter Resistance Indicator Alarm
LIA	1	Distribution Pump Well Water Level Indicator Alarm
LIA	2	Elevated Tank Water Level Indicator Alarm

2) Flow Sheet for Electric Power from the Sub-Station



3) Electric Control System Flow Sheet



4-1-2 Investigation of the Water Quality

4-1-2-1 Place of Sampling: Ping River

Date	1st August 1969		1st August 1966	
	Max.	Min.	Max.	Min.
True Color	205	Non	Non	Non
Odor	unobserved	unobserved	unobserved	unobserved
Taste	unobserved	unobserved	unobserved	unobserved
Turbidity	380	0.5	25	0.2
pH-Value	9.1	6.6	9.0	6.0
Electrical Conductivity at 20°C	890	135	450	200
Total Hardness	162	72	202	62
Carbonate Hardness	162	38	140	3.0
Non Carbonate Hardness	112	nil	183	1.0
Residual Alkalinity	94	nil	46	nil
Oxygen Consumed	10.3	nil	1.8	nil
Nitrite Expressed as Nitrogen	0.1	nil	0.02	nil
Chloride	8.0	2.5	37.5	5.0
Fe	6.0	trace	0.3	nil
Pb	0.3 / nil	nil		nil
Zn	3.5	nil	2.3	1.0
Mg	13.9	1.0	14.9	nil
SO_4	nil	nil	13.4	nil
As	nil	nil		nil

Place of Sampling:	Ping River	Ping River Near the Nauran Temple
Date	13th March 1971	14th March 1971
Time	A.M. 9:50	A.M. 10:30
Temperature	26.5°C	26°C
Water Temperature	25.0°C	24.5°C
Conductivity	1.65×10^{2} µʊ/cm	1.6×10^{2} µʊ/cm
pH-Value	7.9	7.9
Color	13.4	
Turbidity	46.9	50
Ammonia Nitrogen	0.05ppm, less than	0.05 ppm, less than
Nitrite Nitrogen	0.002 ppm, less than	0.002 ppm, less than
Nitrate Nitrogen	0.22 ppm	0.15 ppm
M-Alkalinity	95 ppm	92 ppm
Chlorine	1.25 ppm	1.2 ppm
COD	3.60 ppm	3.6 ppm
Total Hardness	89.6 ppm	95.1 ppm

4-1-2-2 TURBIDITY

TURBIDITY YEAR	JAN.		FEB.		MAR.		APR.		MAY		JUNE		JULY		AUG.		SEP.		OCT.		NOV.		DEC.		
	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	
1966	7	5.8											11.0	9.6			112	11.5			111.0	25	47.0	11.0	Remarks:-
1967	25.0	10.8			11.0	9.6	11.0	9.6	36.0	11.0			42.0	5.2	380.0	4.5		2.8	99.0	4.8	52.0	3.8	26.0	4.6	Turbidity in Silica Units.
1968	11.0	5.6	25.0	4.7	25.0	4.5	9.6	3.8	60.0	3.9		6.6	133.1	5.1	29.7	7.3	88.7	5.4	97.5	5.5	25.0	4.5	11.0	4.5	R.W.→ RAW WATER
1969	11.0	5.8	8.0	5.2	33.0	4.8	11.0	5.8	11.0	5.8	80.0	0.5			150	0.5			0.5				16	1.0	
1970	19	1.5	16	6.0			23	1.0			210	5.5	190	4.2	135	4.4	355	4.6			37.0	3.7	40	3.7	T.W.→ TREATED WATER
1971			56.0	0.2	26	4.0	52.0	7.0	28	4.4	134		37	2.9	120	3.7			94.0	0.9	102	1.2	45	1.0	
1972			20.0	5.5	45.0	3.3	30.0	4.4			29.0	8.4	54.0	4.8	208	6.5	67.0	13.0							

4-1-2-3 Dosing Standards for the No.1 Water Treatment Plant

Tabulation of Alum for Corresponding Turbidity in Silica Unit.

Turbidity (ppm)	Alum (gm/m^3)	Turbidity (ppm)	Alum (gm/m^3)	Turbidity (ppm)	Alum (gm/m^3)	Turbidity (ppm)	Alum (gm/m^3)
0	0.0	15	11.2	50	24.0	150	36.0
1	0.7	16	12.0	55	25.0	160	37.0
2	1.5	17	12.7	60	26.0	180	39.0
3	2.2	18	13.5	65	28.0	200	42.0
4	3.0	19	14.2	70	29.0	250	45.0
5	3.7	20	14.4	75	30.0	300	48.0
6	4.5	21	18.0	80	31.0	350	54.0
7	5.2	22	18.0	85	31.0	400	60.0
8	6.0	24	19.0	90	32.0	500	72.0
9	6.7	26	19.0	90	32.0	600	84.0
10	7.5	28	20.0	100	33.0	800	108.0
11	8.2	30	20.0	110	34.0	1000	132.0
12	9.0	35	21.0	120	34.0	1500	192.0
13	9.7	40	22.0	130	35.0	2000	252.0
14	10.5	45	23.0	140	35.0	3000	372.0

Tabulated above are the quantities of alum required in grams per cu.m. of raw water for the corresponding turbidity. This chart is used primarily for the jar test in which a number of tests with varying amounts of alum are tried for optimum flocculation. Then the amount is used as a feeding rate in water treatment plant.

4-1-3 Survey of Water Levels

Survey data for water levels on the Ping River is as follows.

Sand is being dug commercially from many places on the main river bed, so that special care is required to prevent lowering of the river bed by random digging. Also, it seems that a dam is planned to be built up river in the future, but data on this is lacking.

MINISTRY OF NATIONAL DEVELOPMENT

ROYAL IRRIGATION DEPARTMENT

SURVEY DIVISION, HYDROLOGY SECTION

(NAVARAT BRIDGE)

Mean Daily Gage Height – m (   )

And Discharge – m³/s

For Water Year 1970 Ending March 31, 1971

River System [illegible] Name of Stream [illegible] Station [illegible] Code P.1 Province CHIANG MAI Region [illegible] Drainage Area ________ km²

Day	April 1970 Gage Height (m)	April 1970 Discharge (m³/sec.)	May 1970 Gage Height (m)	May 1970 Discharge (m³/sec.)	June 1970 Gage Height (m)	June 1970 Discharge (m³/sec.)	July 1970 Gage Height (m)	July 1970 Discharge (m³/sec.)	August 1970 Gage Height (m)	August 1970 Discharge (m³/sec.)	September 1970 Gage Height (m)	September 1970 Discharge (m³/sec.)
1	[illegible]	10.6	[illegible]	23.6	301.40	90.0	302.30	176.6	302.45	275.0	302.78	241.2
2	300.77	14.5	[illegible]	20.6	301.72	103.2	302.18	157.5	303.04	277.6	302.67	225.8
3	300.74	13.4	300.84	17.2	301.60	92.2	301.86	116.5	302.62	218.8	302.43	193.2
4	300.75	16.6	300.80	23.0	301.63	93.0	301.67	90.2	302.27	156.4	302.29	172.6
5	300.80	16.6	300.80	25.0	301.77	102.1	301.57	85.7	301.93	127.6	302.20	160.0
6	300.82	16.6	300.81	15.8	301.66	96.6	302.55	330.0	302.61	213.2	302.33	251.6
7	300.86	19.8	300.78	17.2	301.47	75.7	302.42	150.4	302.27	169.8	302.36	255.2
8	300.90	21.4	300.86	15.8	301.35	62.0	302.19	157.8	302.79	242.6	302.93	262.2
9	300.95	27.0	300.96	72.4	301.33	67.0	301.93	127.6	303.13	290.2	303.44	356.0
10	300.95	25.2	301.00	31.0	301.52	71.2	301.71	113.7	303.46	277.6	303.56	352.0
11	300.90	21.4	300.97	24.6	301.51	80.2	301.52	114.4	302.23	206.2	303.47	338.5
12	300.88	19.8	300.90	20.2	301.42	74.6	301.66	96.6	302.54	179.6	303.25	307.0
13	300.87	20.6	300.89	22.2	301.44	72.4	301.53	92.3	302.49	200.6	303.52	392.0
14	300.86	19.6	300.87	29.4	302.02	188.4	301.57	86.5	302.45	295.0	304.32	478.9
15	300.77	20.4	301.26	54.4	302.56	182.4	301.62	94.6	303.48	283.2	303.93	408.1
16	300.78	16.5	301.60	90.9	302.29	272.6	301.80	117.9	303.41	359.5	303.45	335.5
17	300.75	17.4	301.68	94.4	301.91	130.9	301.75	106.5	303.65	367.0	303.22	301.4
18	300.73	17.4	301.70	101.0	302.60	70.0	301.65	95.5	303.49	325.0	303.13	290.2
19	300.73	17.4	301.92	122.0	301.56	85.6	301.63	93.3	303.64	305.6	303.02	274.8
20	300.83	17.4	301.62	124.4	301.65	95.5	302.90	124.0	303.37	295.8	303.11	287.4
21	300.86	19.8	302.55	200.0	301.58	87.8	302.27	154.4	303.77	383.5	303.16	291.6
22	300.85	19.0	302.27	307.8	301.40	68.0	302.28	172.9	303.97	414.9	303.05	279.0
23	300.82	16.6	303.23	299.2	301.42	70.2	302.35	182.0	303.69	371.5	302.93	262.2
24	300.76	19.8	302.99	214.6	301.68	98.8	302.12	152.8	303.57	355.3	302.73	234.2
25	300.76	19.3	302.76	165.6	302.17	156.4	301.99	135.8	303.25	259.5	302.70	230.0
26	300.95	27.8	302.12	190.4	302.54	207.6	301.99	134.8	304.07	432.6	303.43	332.5
27	301.12	61.8	301.93	171.6	302.03	146.8	302.19	168.9	304.24	464.5	302.75	360.5
28	301.12	41.8	301.77	107.1	301.77	108.7	302.49	209.6	303.79	384.5	303.42	331.0
29	301.07	37.3	301.60	90.0	301.70	101.0	302.29	172.6	303.49	342.5	303.25	293.0
30	301.06	36.6	301.69	99.2	302.08	145.6	301.97	132.4	303.25	307.0	303.24	291.6
31			301.65	95.5			301.95	127.6	302.93	255.0		
Total		650.9		2732.2		3155.7		4041.7		8860.6		8592.2
Mean		21.69		88.1		105.2		130.5		285.8		286.4
Max.		61.8		307.8		272.4		209.6		464.6		478.9
Min.		13.4		15.0		61.8		86.7		113.2		152.6
Runoff		57.015		236.053		272.592		392.202		765.538		742.366

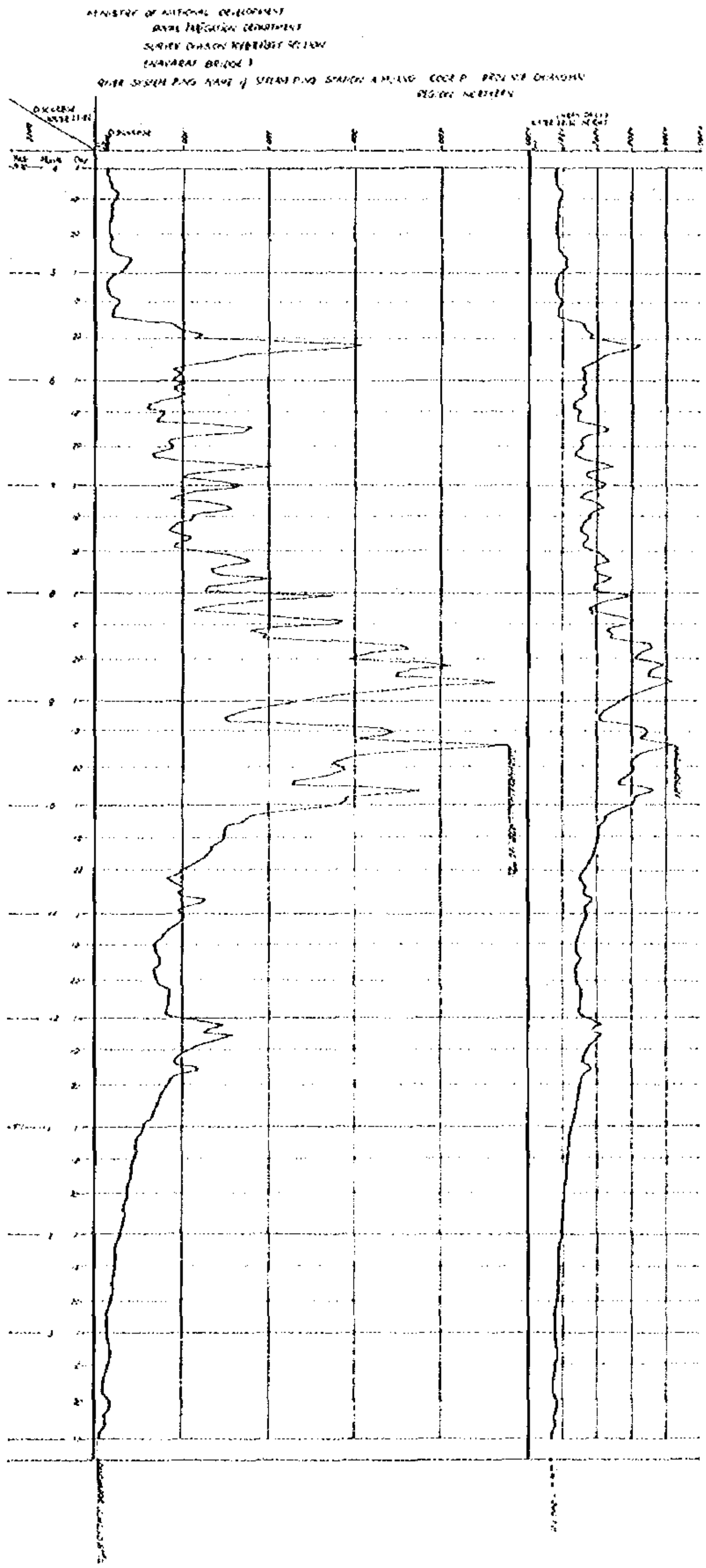
Day	October 1970 Gage Height (m)	October 1970 Discharge (m³/sec.)	November 1970 Gage Height (m)	November 1970 Discharge (m³/sec.)	December 1970 Gage Height (m)	December 1970 Discharge (m³/sec.)	January 1971 Gage Height (m)	January 1971 Discharge (m³/sec.)	February 1971 Gage Height (m)	February 1971 Discharge (m³/sec.)	March 1971 Gage Height (m)	March 1971 Discharge (m³/sec.)	Day
1	303.10	286.0	301.70	101.0	301.89	99.9	301.10	58.0	301.00	31.0	300.81	15.8	1
2	302.85	252.0	301.73	104.3	301.92	126.4	[illegible]	77.1	300.98	30.4	300.81	15.8	2
3	302.59	214.6	301.70	101.0	302.11	149.2	301.26	56.2	300.96	27.8	300.82	16.6	3
4	302.38	185.2	301.66	96.6	301.99	134.8	301.23	51.7	300.94	26.2	300.83	17.4	4
5	302.31	175.4	301.61	92.1	301.92	126.4	301.22	50.8	300.92	24.6	300.84	18.2	5
6	302.27	169.8	301.56	85.6	302.20	160.0	301.22	50.8	300.92	24.6	300.85	19.0	6
7	302.13	151.6	301.52	81.2	302.05	142.0	301.21	49.9	300.95	25.4	300.86	19.8	7
8	302.11	149.2	301.49	79.9	301.96	131.2	301.22	50.8	300.93	25.4	300.86	19.8	8
9	302.10	148.0	301.46	74.6	301.85	118.0	301.20	49.0	300.93	25.4	300.85	19.0	9
10	302.10	148.0	301.43	71.3	301.81	113.2	301.19	48.1	300.93	25.4	300.79	18.2	10
11	302.07	144.4	301.44	72.4	301.72	103.2	301.17	46.3	300.92	24.6	300.81	15.8	11
12	302.04	140.8	301.45	73.5	301.67	97.7	301.15	44.5	300.91	23.8	300.80	15.0	12
13	301.99	134.8	301.49	75.7	301.64	94.4	301.14	43.6	300.91	23.8	300.77	22.6	13
14	301.99	134.8	301.60	74.8	301.64	94.4	301.15	44.5	300.90	23.0	300.77	22.6	14
15	301.96	131.2	301.56	77.9	301.84	116.8	301.14	43.6	300.88	21.4	300.76	23.8	15
16	301.93	127.6	301.48	76.8	301.86	119.2	301.14	43.6	300.91	23.8	300.75	11.0	16
17	301.87	120.4	301.42	70.2	301.68	98.8	301.12	41.8	300.91	23.8	300.75	11.0	17
18	301.82	114.4	301.43	71.3	301.60	90.0	301.10	40.0	300.90	22.2	300.75	11.0	18
19	301.74	105.4	301.45	73.5	301.58	87.8	301.09	39.1	300.89	21.4	300.79	14.2	19
20	301.70	101.0	301.46	74.6	301.55	84.5	301.09	39.1	300.88	21.4	300.85	19.0	20
21	301.66	96.6	301.47	75.7	301.53	82.3	301.10	40.0	300.86	19.3	300.85	19.0	21
22	301.55	84.5	301.53	82.3	301.51	80.1	301.08	38.2	300.84	18.2	300.85	19.0	22
23	301.62	92.2	301.58	87.8	301.49	77.9	301.07	37.3	300.82	16.6	300.83	17.4	23
24	301.68	98.8	301.58	87.8	301.48	76.8	301.05	35.5	300.80	15.0	300.77	12.6	24
25	301.73	104.3	301.57	86.7	301.46	74.6	301.05	35.5	300.80	15.0	300.77	12.6	25
26	301.67	97.7	301.57	86.7	301.45	73.5	301.06	36.4	300.80	15.0	300.77	12.6	26
27	301.70	101.0	301.57	86.7	301.43	71.3	301.06	36.4	300.80	15.0	300.79	14.2	27
28	301.94	128.8	301.55	84.5	301.42	70.2	301.05	35.5	300.82	15.0	300.76	11.8	28
29	301.90	124.0	301.54	83.4	301.40	68.0	301.04	34.6			300.72	8.5	29
30	301.75	106.5	301.55	84.5	301.39	67.0	301.03	33.7			300.70	7.0	30
31	301.67	97.7			301.32	60.0	301.02	32.8			300.68	5.6	31
Total		4265.7		2675.5		3089.6		1344.2		604.2		454.0	
Mean		137.6		82.5		99.6		43.5		21.7		14.6	
Max.		286.0		104.3		160.0		58.0		31.0		19.6	
Min.		84.5		70.2		60.0		32.8		15.0		5.6	
Runoff		368.556		231.883		266.941		116.312		52.203		39.226	

Remarks

1) Instantaneous peak ______ cms at ______ m (______) at ______ hours on ______
2) Lowest flow ______ cms at ______ m ( ) at ______ hours on ______
3) Overflow begins at ______ hours on ______ [ ] estimated, [ ] computed with hm³ ______

Processed ______
Date ______
Eng. Checked ______
Date ______
Approved ______
Date ______

Water Year 1970	
40,276.5	m³/s
110.3	m³/s
478.9	m³/s
5.6	m³/s
3,522.887	(m³) x 10⁶



4-1-4 Unit Weights, Allowable Stresses and Allowable Bearing Capacities for the Design

4-1-4-1 Unit weights for the design are determined as follows:

Item	Unit	Weight
Weight of Earth	m^3	1,6 t
Weight of Earth (Below underground water level)	m^3	1.0 t
Reinforced Steel Concrete	m^3	2.4 t
Concrete	m^3	2.3 t

4-1-4-2 Intensity of various allowable stress are determined as follows:

Compressive strength of concrete (4 weeks)

$\sigma o28 = 200\ Kg/cm^2$

Compressive strength of concrete (allowable stress)

$\sigma ca = 50\ Kg/cm^2$

Shearing strength (allowable stress) (beams)

$\tau a = 5\ Kg/cm^2$

Shearing strength (allowable stress) (slab)

$\tau a = 7\ Kg/cm^2$

Reinforcing steel tensive strength

$\sigma s = 1200\ Kg/cm^2$

Tensive strength of deformed reinforcing steel

$\sigma s = 1400\ Kg/cm^2$

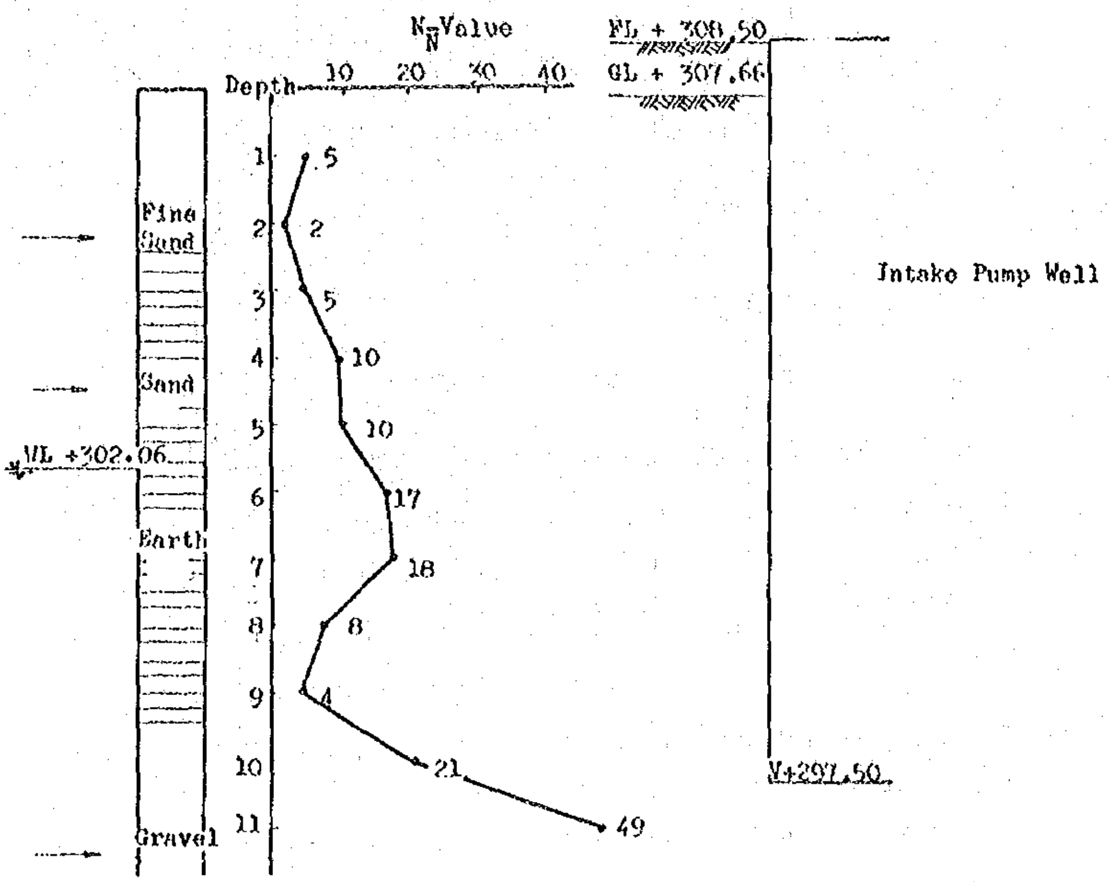
Allowable bond strength of concrete

$\sigma oa = 14\ Kg/cm^2$

4-1-4-3 Allowable Bearing Stress for Foundation Work

a) Intake Site

Earth Pressure Coefficients (Boring Data No.3)



The N-value at 8 to 9m, where earth pressures have considerable effects, is set at 4-10. Allowing a safety factor, the earth pressure coefficient has been determined with an N-value of 5.

Using Pec. Meyerhof values, the interior friction angle is set at ϕ=25, When ϕ=25° the earth pressure coefficient is calculated as follows:

---- Employing the Coulon earth pressure coefficient graph, the value is 0.4.

b) <u>Bearing Power of Ground</u> (Sedimentation Basin and Intake Well)

When $\phi = 25$

$Nc = 26,\ Nq = 14,\ Nr = 8.5$

$$qa = \frac{1}{3}\left\{\left(1 + 0.3\frac{B}{L}\right) CNc + \left(0.5 - 0.1\frac{B}{L}\right)\right\} \gamma BN_r + DfNq$$

$$= \frac{1}{3}\left\{\left(1 + 0.3 \times \frac{11.4}{28.75}\right) \times 0 \times 26 + \left(0.5 - 0.1 \times \frac{11.4}{28.75}\right) \times\right.$$

$$\left. 1.0 \times 11.4 \times 8.5 + 1.0 \times 11.0 \times 14\right\}$$

$$= 66.19 \doteqdot 66\ t/m^2$$

$B = 11.4\ m$

$L = 28.75\ m$

$= 1.0\ t/m^3$

$Df = 11.0\ m$

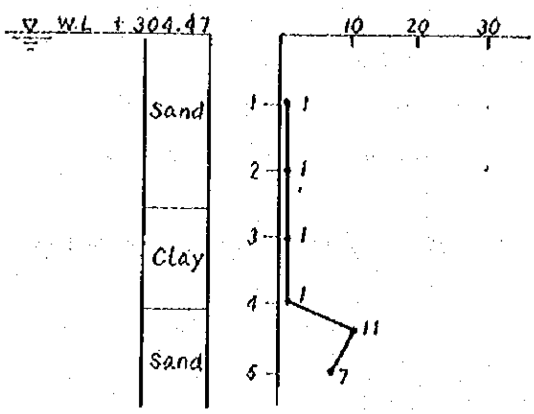
c) <u>Weights Involving the Grit Chamber and Intake Pump Well</u>

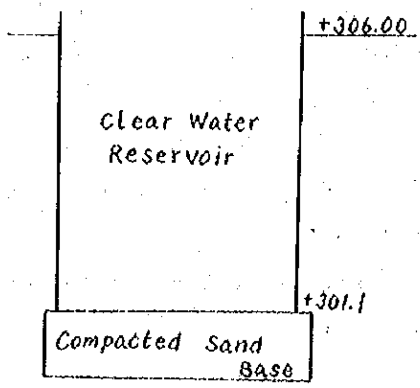
Slab	0.3 x 2.4 = 0.72
Water	1.0 x 9.5 = 9.5
Bottom slab	1.0 x 2.4 = 2.4

$W = 12.62 \doteqdot 12.6\ t/m^2$

d) Earth Pressure Coefficients (Boring Data No.8) for the Purification Plant

Calculations are based on an N-value 5 m from the surface of the earth.





N-value is set at 1 for 1 m to 4 m, where earth pressures have the most effect.

Using Pec. Meyerhof values, $\phi = 25^{o}$.

When $\phi = 25^{o}$, the earth pressure coefficient is calculated as follows:

Employing the Coulon earth pressure coefficient graph, the value is 0.4.

e) <u>Bearing Capacity of Ground (Clear Water Reservoir)</u>

When $\phi = 25$

$Nc = 26,\ Nq = 14,\ N\ = 8.5$

$$qa = \frac{1}{3}\left\{(1.0 + 0.3\frac{B}{L})\ cNc + \gamma DfNq + (0.5 - 0.1\frac{B}{L})\ \gamma BN\gamma\right\}$$

$$= \frac{1}{3}\left\{(1.0 + 0.3\frac{26.0}{26.8}) \times 0 \times 26 + 1.0 \times 4.9 \times 14 + (0.5 - 0.1\frac{26.0}{26.8}) \times 1.0 \times 26.0 \times 8.5\right\}$$

$$= 33\ t/m^2$$

$B = 26.0$

$L = 26.8$

$\gamma = 1.0\ t/m^3$

(weight in water)

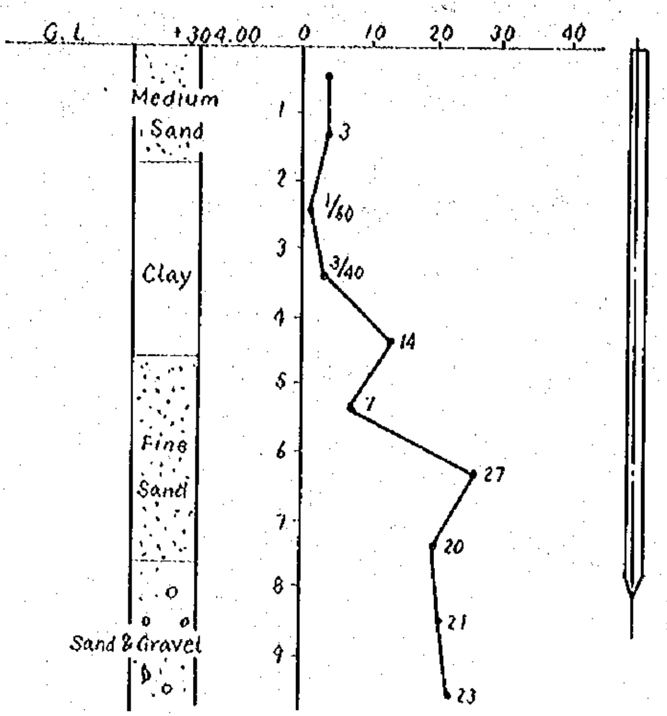
$Df = 306.0 - 301.1 = 4.9$

f) Weight of Clear Water Reservoir

Sand	0.5 x 1.6 = 0.8
Slab	0.3 x 2.4 = 0.72
Water	1.0 x 4.3 = 4.3
Bottom	0.7 x 2.4 = 1.68
	$\Sigma W = 7.5 \doteq 8\ t/m^2$

$qa = 33\ t/m^2 > 8\ t/m^2$

g) Allowable Bearing Capacity for RC Piles (Boring Data No. 7)
(for receiving wells, sedimentation basins, and filter basins)



N-value for end of pile (80% assumed) $21 \times 0.8 = 17$

N-value around pile

$$Nm = \frac{14 + 7 + 27 + 20}{4} = 17$$

$$Na = \sqrt{\frac{(14 - 17)^2 + (7 - 17)^2 + (27 - 17)^2 + (20 - 17)^2}{4 - 1}} \doteqdot 5$$

$$\bar{N} = Nm - \frac{1}{2} Na = 17 - \frac{1}{2} \times 5 \doteqdot 15$$

Allowable Bearing Capacity per pile

$$Rp = \frac{1}{3} \left(40N.Ap + \frac{\bar{N}.Af}{5}\right)$$

$\phi 300$

$Ap = 0.3 \times 0.3 = 0.09\ m^2$

$Af = 0.3 \times 4 \times 4.0 = 4.8$

$Rp = \frac{1}{3}(40 \times 17 \times 0.09 + \frac{15 \times 4.8}{5} \doteqdot 25\ t/pile$

$\phi 250$

$Ap = 0.25 \times 0.25 = 0.0625$

$Af = 0.25 \times 4 \times 4.0 = 4.00\ m^2$

$Rp = \frac{1}{3}(40 \times 17 \times 0.0625 + \frac{15 \times 4.00}{5}) \doteqdot 18\ t/Pile$

4-1-5 Operation and Maintenance

4-1-5-1 Intake Facilities

a) Intake Site

Removal of debris from the intake screen shall be by hand. Sand shall be disposed of in the lower reaches of the river with water by use of a portable sand pump. Operation of the intake pumps shall be in 3 shifts per day, and the selection of pumps for the work made by telephone consultations with the water purification plant.

The plant generator shall be put into operation during electrical stoppages with press buttons, located in the generator room.

b) Connecting Valves in the Raw Water Main

Water will be conveyed to the original old water purification plant and the proposed new plant. A connecting valve shall be placed where the two raw water mains for the two plants cross the Super Highway. By operating this valve, distribution of raw water for the two plants can be regulated in event of an emergency failure of one of the two mains.

4-1-5-2 Water Purification Facilities

a) Chemical Dosing

Flow volume is taken from readings of the flow meter in the raw water mains, and turbidity is obtained by means of a turbidity meter. Then, using a table kept at hand on dosing volume, the dosing is done by operation of a chemical injection pump.

b) Operation of Slow Speed Agitators and Sludge Scraper

One set of slow speed agitators may be used at times instead of two, depending on changes in the turbidity of the raw water. The sludge scraper also may be stopped at times instead of being operated for the entire day. Judgement based on experience shall guide these decisions.

c) Operation of the Sludge Pump

Sludge accumulated in the pit shall be checked by the operators, and when necessary shall be cleaned out by pump. Also, a timer shall be installed, making possible timer-controlled operation.

d) Washing of the Sand Filter Basin

The control panel of the operations room shall include a warning signal for critical filter resistance, and red and green operational signal lamps. When the warning signal indicates the necessity for washing, the worker-in-charge shall operate the valves for this operation by hand, and carry out the entire washing process.

e) Level of the Clear Water Reservoir

The operating personnel will operate the plant with reference to the water level of the supply reservoir reducing the filter rate of the water treatment when the reservoir approaches being completely filled. The water treatment operation shall be so regulated as to meet basic demands under conditions of continuous operation. To this purpose, the supply of raw water shall be regulated to meet the changing demands of the seasons.

f) Operation of the Distribution Pumps

The two types of pump with larger and smaller capacities shall be started and stopped manually in accordance with readings of the pressure meters and flow volume meters for the distribution mains. Moreover, at times when demand is small, as during the night, allowance should be made in operating the pumps for changes reducing the flow volume by the partial closure of the valves on the delivery side of the distribution pumps.

g) Quarters for Personnel

The proposed water purification plant will require three shifts each with three operating personnel, or a total staff of nine personnel. One person will be required to repair breakdowns of equipment at the plant site and in the mains. Finally, a plant superintendent is necessary. Thus, suitable quarters for a staff of eleven personnel and their families shall be built within the plant.

4-2 Detail Design

4-2-1 Calculations of Volumes

4-2-1-1 Planned Volume for Water Works System

Stage	Proposed Plan	Intake Volume	Discharge Volume of Treated Water		Distribution Volume (Hourly Max. expressed in m^3/day)	
			Proposed Plant	Old Plant (No.1)	Proposed Plant	Old Plant (No.1)
1st (1980)	16000+7000 =23000	x1.1=25300	16000	7000	1.5x14000=21000 2000 1000	x1.5=10500
2nd (1990)	32000+7000 =39000	x1.1=42900	32000	7000	1.5x28000=42000 4000 2000	x1.5=10500
3rd (2000)	48000+7000 =55000	x1.1=60500	48000	7000	1.5x42000=63000 6000 3000	x1.5=10500

Note: Existing new water supply system is not listed herein.

4-2-1-2 Intake Site

a) Grit Chamber (to the 3rd Stage)

No. of chambers: $N = 2$

Av. flow velocity: $V = 5.0$ cm/sec

Settling velocity of particles: $U = 0.8$ cm/sec (settling velocity of particles 0.1 mm in dia.)

Effective water depth: $H = 1.5$ m

Safety ratio: $K = 1.5$

Volume of treated water: $Q = 60500\ m^3/day = 42.0\ m^3/min$
$0.7\ m^3/sec = 0.35\ m^3/sec/chamber$

Effective length of grit chamber: $L = K\left(\frac{H}{U}\ V\right)$

$$= 1.5\left(\frac{1.5}{0.008} \times 0.05\right) \doteqdot 15.0m$$

Width of grit chamber: $B = 4.5$ m

Actual average flow velocity in grit chamber:

$$V_o = \frac{Q}{B \cdot H} = \frac{0.35}{4.5 \times 1.5} = 0.052 \text{ m/sec}$$

Retention time: $T = \frac{L}{V} = \frac{15.0}{0.052} = 288.5 \text{ sec}$

$= 4 \text{ min. } 48 \text{ sec.} \doteqdot 5 \text{ min/chamber}$

Raw Water Pump Head

(c=100)

	Existing Plant	Proposed Plant 1st Stage	Proposed Plant 2nd Stage	Proposed Plant 3rd Stage
L (m)	4,190		3,110	
⌀ (mm)	300	400	400, 400 } 520	400, 400, 400 } 607
$Q(m^3/D)$	7,700	17,600	35,200	52,800
I(%)	8.4	9.6	9.6	9.6
V(m/sec)	1.25	1.6	1.6	1.6
Hf(m)	35.2	29.9	29.9	29.9
R.W. WL	+311.20		+311.5	
Intake P.W. WL		+300.5		
Actual(m) Head	10.7	11.0	11.0	11.0
Pump Loss	2.1	2.1	2.1	2.1
Total Head(m)	48.0	43.0	43.0	43.0

Pump Power to Chiang Mai Water Treatment Plant No.1

$Q = 7700 \text{ m}^3/\text{day} = 5.35 \text{ m}^3/\text{min}$

$r = 1.0 \qquad q = 5.35\ m^3/min \qquad H = 48\ m$

Shaft Power: $P = \frac{0.163 rqH}{n}$

$= \frac{0.163 \times 1.0 \times 5.35 \times 48}{0.68} = 61.6\ kw$

Motor Power: $R = \frac{P(1+\alpha)}{7t}$

$= \frac{61.6\ (1+0.15)}{10} = 70.8\ kw = 80\ kw$

Pump Diameter

$$D = 146 \sqrt{\frac{q}{v}}$$

$$= 146 \sqrt{\frac{5.35}{2.0}} = 250\ mm$$

Raw Water Pump Specification:

$\phi 250mm \times 5.35\ m^3/min \times 48^m \times \quad 1450\ rpm \times 80\ kw$

To the Proposed Plant

$Q = 17.600 m^3/day = 12.22 m^3/min$

$r = 1.0 \qquad q = 12.22\ m^3/min \qquad H = 43\ m$

Shaft Power: $P = \frac{0.163 rqH}{n}$

$= \frac{0.163 \times 1.0 \times 12.22 \times 43}{0.72} = 119.0\ kw$

Motor Power: $R = \frac{P(1 \alpha)}{nt}$

$= \frac{119.0 \times (1+0.15)}{1.0} = 136.9 \doteqdot 150\ kw$

Pump Diameter $D = 146 \sqrt{\frac{q}{v}}$

$= 146 \sqrt{\frac{12.22}{3.0}} \doteqdot 300\ mm$

Raw Water Pump Specification:

$\phi 300\ mm \times 12.22\ m^3/min \times 43\ m \times 1450^{rpm} \times 150\ kw$

Raw Water Pump Installation Schedule

(Vertical Shaft Type)

	Existing Plant	Proposed Plant		
		1st Stage	2nd Stage	3rd Stage
Discharge	7,700m^3/d	17,600m^3/d	35,200m^3/d	52,800m^3/d
Set	(1 stand-by)	2(1 stand-by)	3(1 stand-by)	4(1 stand-by)
q(m^3/min.set)	5.35	12.22	12.22	12.22

4-2-1-3 Electric Equipment for Intake Plant

a) Capacity of Transformer

1) Load Capacity for Power Equipment

Type of Equipment	Unit Capacity	Quantity	Actual No. in Operation	Effective Capacity
Raw Water Pump	80 kw	2	1	80 kw
" " "	150 kw	2	1	150 kw
Cooling Water Pump	1.5 kw	1	1	1.5 kw
Water Lubricating Pump	0.75kw	2	1	0.75kw
Generator Accessory	10 kw			10 kw
			TOTAL	242.25kw

$$242.25^{W} \times 1.25 = 303^{kVA}$$

2) Load Capacity for Lighting Equipment

Room	Capacity
Intake Pump Room	1560 w
Generator Room	7280 w
Warehouse	480 w
Lodging House	9600 w
Outdoor	500 w
TOTAL	19420 w

$$19,420^{W} x 1.25 x 10^{-3} \doteqdot 24 \text{ KVA}$$

3) Total Load Capacity

$$303^{KVA} + 24^{KVA} = 327^{KVA}$$

b) <u>Starter Capacity</u>

1) <u>Cable Impedance</u>

i) Connecting Cable for Plant Generator

$325^{\square}$ 35 m R = 0.0745 Ω/km x = 0.0715 Ω/km

$$* \; Pux = \frac{0.0715 \times 1000}{(0.38)^2 \times 10^3} \times \frac{35}{1000} = 0.0172$$

$$* \; Pur = \frac{0.0745 \times 1000}{(0.38)^2 \times 10^3} \times \frac{35}{1000} = 0.018$$

ii) 80 KW (153A) $100^{\square}$ 10m R=0.228 Ω/km X=0.762 Ω/km

$$Pux = \frac{0.0762 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.0052$$

$$Pur = \frac{0.228 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.0157$$

iii) 150 KW (285A) $250^{\square}$ 10 m R = 0.0937 Ω/km X=0.0728 Ω/km

$$Pux = \frac{0.0728 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.005$$

$$Pur = \frac{0.0937 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.0064$$

iv) F-2 $250^{\square}$ 50 m R = 0.0937 Ω/km X = 0.0728 Ω/km

$$Pux = \frac{0.0728 \times 1000}{(0.38)^2 \times 10^3} \times \frac{50}{1000} = 0.0252$$

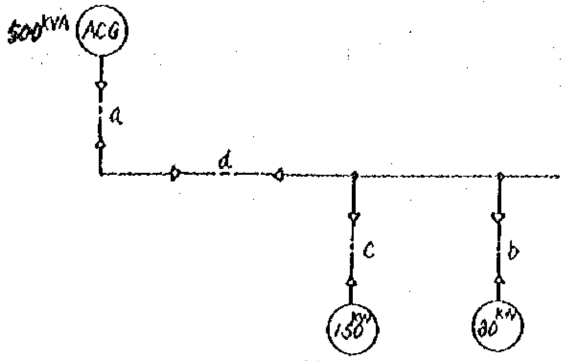
$$Pur = \frac{0.0937 \times 1000}{(0.38)^2 \times 10^3} \times \frac{50}{1000} = 0.0324$$

$$Zadc = \sqrt{0.0474^2 + 0.568^2} = 0.074$$

$$Zadb = \sqrt{0.0476^2 + 0.0661^2} = 0.082$$

* Note: Pux = Per Unit Reactor

Pur = Per Unit Resistor



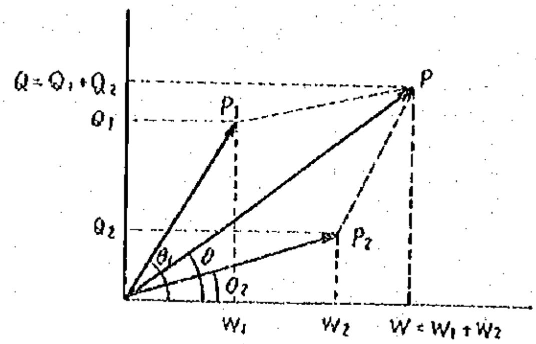
2) 150 KW Generator Capacity at Starting Time

$$Z_1 = \frac{1000 \times 100}{1.88^{KVA} \times 2} = 266\ \%$$ (Starter KVA 250 %)

$$\text{Generator Capacity} = \frac{0.8 \times 25 \times 1000}{0.2 \times 266 - 0.8 \times 0.074} = \frac{0.8 \times 25 \times 1000}{53.14} = 376\text{KVA}$$

(Assuming Generator 25%)

3) In Case of Base Load



P_1 = Generator Capacity when Starting Motor (with no load)

P_2 = Base Load Capacity

P = Generator Capacity to Enable Starting Generator During Operation under Base Load

$CosQ_1$ = Starting Ratio

$CosQ_2$ = Base Load Ratio

$$P = \sqrt{Q^2 + W^2} = \sqrt{(Q_1 + Q_2)^2 + (W_1 + W_2)^2}$$

$$Q_1 = P_1 \sin Q_1 \qquad W_1 = P_1 \cos Q_1$$

$$Q_2 = P_2 \sin Q_2 \qquad W_2 = P_2 \cos Q_2$$

P_1 = 376 KVA

P_2 = 115 KVA + 24 KVA = 139 KVA

W_1 = 376 x 0.2 = 75 KW

W_2 = 111 KW

$Q_1 = 376 \times \sqrt{1-0.2^2} = 368$ KVar

Q_2 = 84 KVar

$P = \sqrt{(368 + 84)^2 + (75 + 111)^2} = 488$ KVA ⟶ 500 KVA

4-2-1-4 <u>Chiang Mai Water Treatment Plant No.1 (Old W.T. P.)</u>

a) <u>Receiving Well</u>

Q = 7000 m^3/ = 292 m^3/H = 4.86 m^3/min = 0.081 m^3/sec

Specification: D = 4.4 m H = 6.2 x 1 Basin

Capacity: 94.5 x 1/3 ≑ 31 m^3

Retention Time: 6.5 min

4-2-1-5 Paton Water Treatment Plant

a) Receiving Well

$Q = 48,000\ m^3/D = 2,000\ m^3/H = 33.3\ m^3/min = 0.555\ m^3/sec$

Specification: $D = 6.5^m$ $H = 5.5 \times 1$ Basin

Capacity: $183\ m^3$

Retention Time: 5.5 min(3rd Stage)

b) Rapid Mixing Basin

1 unit (Vertical-Shaft type)

Specification: 1.9^m $He = 2.1\ m$

$P_s = 0.75$ kw x 2 basin

Capacity: $7.6\ m^3$/Basin

Retention Time: 1.4 min

	1st Stage	2nd Stage	3rd Stage
Unit	1	2	3

c) Flocculation Basin

1 unit (Vertical-Shaft type)

Specification $B = 4.6^m$ $L = 9.3\ m^3$ $H = 2.4m \times 4$ Basin

$P_s = 0.75$ kw x 2/Basin

Capacity: $103\ m^3$/Basin

Retention Time: 37 min

	1st Stage	2nd Stage	3rd Stage
Unit	1	2	3

d) Sedimentation Basin

1 Unit

Specification: $B = 9.1m$ $L = 34.0m$ $H = 3.5^m x2$ basins

Capacity: $1,080\ m^3$/basin

Retention Time: 3.25 hour

Velocity: $\frac{8000}{9.1 \times 3.5 \times 1440} = 0.175m/min$

$(< Va = 0.4m/min)$

Water Surface Area: $9.1 \times 34.0 = 309m^2$/Basin

Water Surface Ratio: $8000 \times \frac{1}{309} = 25.9m^3/m^2D$

Trough Length: $3.1^m \times 6 \times 2 = 37.2/m$/Basin

Overflow Ratio: $8000/37.2 = 215 m^3/m \cdot D$

Unit	1st Stage	2nd Stage	3rd Stage
	1	2	3

o) <u>Sludge Removal Equipment</u>

Calculation of Sludge Volume

Designed Waste Water Volume

1st Stage	16,000 m^3/day
2nd "	32,000 m^3/day
3rd "	48,000 m^3/day

Designed Turbidity, Ratio of Chemical Dosing and $Al_2(OH)_3$

	Raw Water Turbidity (PPM) (A)	$Al_2(SO_4)_3 \cdot 18H_2O$ Dosing Ratio (PPM) (B)	Volume (C) of $Al_2(OH)_3$ $\frac{156}{666} = 0.234$ (PPM)
Maximum	500	72	17
Mean	200	42	10
Minimum	10	7.5	2

[For values of raw water turbidity and of $Al_2(SO_4)_3 \cdot 18H_2O$ refer to Section 4-2-3 on Chemical Dosing Equipment.

$Al_2(OH)_3 = Al_2(SO_4)_3 \cdot 18H_2O \times \frac{156}{666}$]

Sludge Volume

Volume of Solidified Sludge

$$Qs(max.) = 16,000 \times (500 + 17) \times \frac{1}{10^6} = 8.3\ m^3/D$$

$$Qs(mean) = 16,000 \times (200 + 10) \times \frac{1}{10^6} = 3.4\ m^3/D$$

$$Qs(min.) = 16,000 \times (10 + 2) \times \frac{1}{10^6} = 0.2\ m^3/D$$

1) Sludge Scrapers (Double Chain Conveyor)

Sludge Volume (Sludge Concentration of kg/m^3)

$$Q(max) = (8.3\ m^3/D \times \frac{1}{10} \times 1000) = 830\ m^3/D$$

$$Q(mean) = (3.4\ m^3/D \times \frac{1}{10} \times 1000) = 340\ m^3/D$$

$$Q(min) = (0.2\ m^3/D \times \frac{1}{10} \times 1000) = 20\ m^3/D$$

1 Unit B = 4.4 m L = 34.0 m x 2 lanes

Sludge Scraper (Density 1.06 when sludge concentration is 10 kg/m^3

Operating time for sludge scraper is 24 hrs.)

$$Qw(max) = 830\ m^3/D \times (1.06 - 1) \times \frac{1}{24} \times 10^3 \times \frac{1}{2}$$

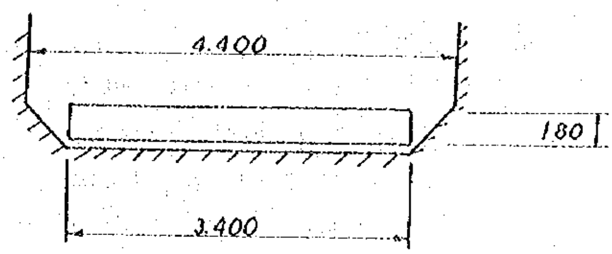
$$= 1{,}040\ kg/hr./unit$$

$$Qw(mean) = 340\ m^3/D \times (1.06 - 1) \times \frac{1}{24} \times 10^3 \times \frac{1}{2}$$

$$= 425\ kg/hr./unit$$

$$Qw(min) = 20\ m^3/D \times (1.06 - 1) \times \frac{1}{24} \times 10^3 \times \frac{1}{2} = 25\ kg/hr./unit$$

Scraper Capacity



Scraper speed: 0.2 m/min - 0.4 m/min

Limits for adjustment of speed: 1 = 2

Shape of scraper: B = 3.4 m H = 0.18 m

Efficiency ratio of scraper: 0.6

$$Q_F = B(m) \times H(m) \times V\ (m/min) \times \eta \times 60\ min \times 2\ lanes$$

(Q_F (m^3/hr./unit) : (Volume capacity of scraper per unit)

$$Q_W' = Q_F \times (1.06 - 1.0) \times 10^3$$

(Q_W' (kg/hr/unit): Q_F (Sludge volume included)

Turbidity Range	Yearly Fre-quency (%)	Scraper Speed (m/min)	Q_F (m^3/hr./unit)	Q_W'(Capacity) (kg/hr./unit)	Q_W (kg/hr./unit)	Q_W' Q_W (OK)
Maximum to Mean	4	0.4	17.6	1,056	1,040	OK
Mean or Lower	96	0.2	8.8	528	425	OK

Scraper Power Output: 0.75 KW/Unit

Plan for Sludge Scraper Equipment

	1st Stage	2nd Stage	3rd Stage
Unit	1	2	3

2) Sludge Pump

1 Unit (volume of treated water 16,000 m^3/day)

Allowing a safety factor, the capacity for sludge disposal of the pump (assuming sludge density of 15 kg/m^3) shall be 1.5 times the volume of sludge.

Sludge disposal volume (sludge density = 15 kg/m^3)

$$Q\ max = (8.3m^3/D \times \frac{1}{15} \times 1{,}000) \times 1.5 = 553m^3/D \times 1.5 = 830m^3/D$$

$$Q\ mean = (3.4m^3/D \times \frac{1}{15} \times 1{,}000) \times 1.5 = 216m^3/D \times 1.5 = 340m^3/D$$

$$Q\ min = (0.2m^3/D \times \frac{1}{15} \times 1{,}000) \times 1.5 = 13m^3/D \times 1.5 = 20m^3/D$$

Ideally, the sludge pump should be operated constantly for 24 hours but the pump capacity is designed so that sludge can be handled even by an 8-hour operation during the daytime. When the operation hours are calculated, data is as follows:

Pump Specification

(During low water level periods in the sedimentation basin)

	Open Channel initial water level	+305.65
-)	Water level for sedimentation basin sludge pit:	+304.85
	Actual lift =	0.80

$\phi 200^{mm} \times 3.5m^3/min \times 5m \times 860\ rpm \times 15kw \times 2$ units (including 1 unit as stand-by)

Turbidity	Volume of Sludge Disposal	Operating Time per Hour (minutes)	
		24-Hour Operation	8-Hour Operation
Maximum	830	10	30
Mean	340	4	12

Unit	1st Stage	2nd Stage	3rd Stage
	1	2	3

Operation of sludge pumps shall not be simultaneous for the 1st, 2nd, and 3rd stages, but shall be operated on a staggered time schedule.

f) Rapid Sand Filter

1 Unit

Specification: B = 6.4m L = 5.0m $A = 32m^2 \times 6$ chambers (including 1 stand-by)

Filtration Ratio: 100m/day

Filtration volume: $3200m^3$/D/chamber x 5 chambers = $16000m^3$/day

Surface Wash Pipe

$Q = 32m^2 \times 0.15m^3/min\text{-}m^2 = 4.8m^3/min = 0.08m^3/sec$

Main: $\phi 250$ V = 1.63m/sec

Branch: $q = 0.04m^3/sec$ $\phi 150$ V = 2.06m/sec

Surface Wash Pressure:

Surface Wash Spray Height: +308.60

Necessary Pressure for Surface Wash: 15.00 (at time of low water level)

+) Pipe Line Head Loss (from the hydraulic motor, etc.):

3.40

Surface Wash Water Head +327^{M}00

(Clear Water Reservoir at High Water Level has a surface wash head of 19.0m)

Reduction of Head

Distribution Pump Head +342.00

(when Clear Water Reservoir is at low water level)

-) Surface Wash Water Head +327.00

15.00

Auxiliary pressure of pressure reducing valves is assumed to be 15.00m · +327.00

Backwash Pipe

$Q = 32m^2 \times 0.5m^3/min/m^2 = 16m^3/min = 0.267m^3/sec$

Main: ∅500 V = 1.35m/sec

Branch: ∅400 V = 2.10m/sec

Backwash Pressure

High water mark for filter tank collection apparatus:	+307.70
Required Head for Backwash (2.5 - 5.0m):	5.00
Head Loss for Perforated Pipe:	1.55
+) Pipe Route Head Loss (from hydraulic calculations):	3.75
	+318.00

Water Level of Elevated Tank: HWL +318.00

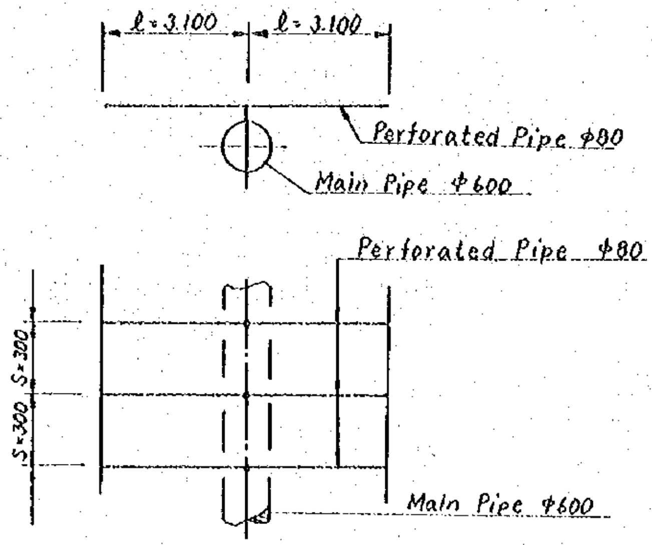
LWL +315.50 (Backwashing Water Pressure at Low Water Level 2.0m)

Drain Pipe

Q = Backwash Water = $0.267m^3/sec$

∅500 V = 1.35m/sec

Perforated Pipe



Assuming values as: $\phi 80$ $\ell = 3.10$m S = 0.3m

Then:

N = 5.0 ÷ 0.3 x 2 systems = 32

Total Sectional Area =

$$\frac{0.08^2}{4} \times \pi \times 32 = 0.16\text{m}^2$$

The total sectional area of pipe perforations for water collection is calculated as follows:

The combined area of the orifices is designed to be 0.2% of the total planned area of each filter bed or $32\text{m}^2 \times 0.002 = 0.064\text{m}^2$ approximately.

The orifice area should be 20 - 50% of the total sectional area (A) of the perforated pipes themselves. Using 40% as an appropriate figure,

$$\text{Orifice Area} = \text{total area} \times \frac{40}{100} = 0.16 \times \frac{40}{100} = 0.064\text{m}^2$$

Main Pipe: 1.75 times the total sectional area of the perforated pipe

$$D = \sqrt{\frac{0.16 \times 1.75 \times 4}{3.14}} \doteqdot 0.6 \qquad \phi 600\text{mm}$$

	1st Stage	2nd Stage	3rd Stage
Unit	1	2	3

g) Elevated Tank

The required volume of water for backwashing is that necessary fo: one filter, as follows:

$32\ m^2 \times 0.5\ m^3/min/m^2 \times 6.0\ m = 96\ m^3$

The volume of water to be lifted to the elevated tank during back-washing is: $2.0\ m^3/min \times 6.0\ min = 12\ m^3$

Thus, the volume necessary to be stored in the elevated tank will be the difference between 96 m^3 and 12 m^3 or 84 m^3.

Capacity of the elevated tank shall be:

39 m^2 (horizontal section) x 4 m (water depth) or 156 m^3

The amount of water available for service water in the plant area will be 156 m^3 - 84 m^3 or 72 m^3.

h) Lift Pump For Elevated Tank

Operating Time: 1 Hour

$q = 104m^3/60min \doteqdot 2m^3/min/set$

Pump Head

	Elevated Tank HWL	318.00
-)	Distribution Pump Well LWL	302.00
	Ha =	16.00
	Pipe Head Loss	0.50
+)	Head Loss around Pump	2.50
	HT =	19.00m

Pump Power:

$q = 2.0m^3/min \qquad \gamma = 1.0 \qquad H = 19m$

Shaft Power $P = \dfrac{0.163\ \gamma qH}{\eta}$

$= \dfrac{0.163 \times 1.0 \times 2.0 \times 19}{0.6} = 10.3kw$

Motor Power $R = \dfrac{P(1+\alpha)}{\eta\ t}$

$= \dfrac{10.3 \times (1 + 0.15)}{1.0} \doteqdot 15kw$

Pump Diameter $D = 146\sqrt{\dfrac{q}{V}} = 146\sqrt{\dfrac{2.0}{3.0}} = 125mm$

Left Pump Specification:

$\phi 125 \times 20m^3 \times 19m \times 1{,}450rpm \times 15\ kw$

D = 125mm   q = 2m^3/min   H = 19.00m   Ps = 15kw

	1st Stage	2nd Stage	3rd Stage
Set	2 (1 stand-by)	3 (1 stand-by)	3 (1 stand-by)
Operating Time	1.0 hour	0.5 hour	0.5 hour

Note: Filter washing cycle 40 - 60 min.

i) Clear Water Basin

1 unit

Specification: B = 4.7^m L = 19.8^m H = 2.25^m x 2

Capacity: 209m^3/basin

Retention Time: 0.6 hour

	1st Stage	2nd Stage	3rd Stage
Unit	1	2	3

j) Clear Water Reservoir

1 unit

Specification: B = 24.0^m L = 48.0^m H = 4.0^m x 1

Capacity: 4610m^3/chamber

Retention Time: $4610 \div \frac{16,000}{24} = 6.9$

	1st Stage	2nd Stage	3rd Stage
Unit	1	2	3

k) Distributing Pump Well

Specification: B = 8.0^m L = 24.0^m H = 4.0^m x 1

Capacity: 770m^3

Retention Time:

	Retention Time
1st	1.15 hour
2nd	0.58 hour
3rd	0.39 hour

1) Distributing Pump

1) Pump Power:

i) $q = 8.4m^3/min \quad \gamma = 1.0 \quad H = 43^m$

Shaft Power: $P = \frac{0.163rqH}{\eta}$

$= \frac{0.163 \times 1.0 \times 8.4 \times 43}{0.72} = 81.8kw$

Motor Power: $R = \frac{P(1 + \alpha)}{\eta t}$

$= \frac{81.8 \times (1 + 0.15)}{1.0} \doteq 100\ kw$

Pump Diameter: $D = 146\sqrt{\frac{q}{V}} = 146\sqrt{\frac{8.4}{2.5}} \doteq 300mm$

Distributing Pump Specification:

ϕ300mm x 8.4m^3/min x 43m x 1,450rpm x 100kw

ii) $q = 4.2m^3/min \quad \gamma = 1.0 \quad H = 43m$

Shaft Power: $P = \frac{0.163\gamma qH}{\eta}$

$= \frac{0.163 \times 1.0 \times 4.2 \times 43}{0.72} = 40.9kw$

Motor Power: $R = \frac{P(1 + \quad)}{t}$

$= \frac{40.9 \times (1 + 0.15)}{1.0} \doteq 55kw$

Pump Diameter: $D = 146\sqrt{\frac{q}{V}} = 146\sqrt{\frac{4.2}{2.5}} \doteq 200mm$

Distributing Pump Specification:

ϕ200mm x 4.2m^3/min x 43m x 1,450rpm x 55kw

2) <u>Distributing Pump Capacity</u>

(Vertical-Shaft type)

Stage	Discharge(Q)	Number of Pumps	Q(m^3/min/pump)
1st	24,000m^3/day	0.5Q x 2 0.25Q x 2 (0.5Q, one Stand-by)	8.4 x 2 pumps 4.2 x 2 pumps
2nd	48,000m^3/day	0.5Q x 4 0.25Q x 2 (0.5Q, one Stand-by)	8.4 x 4 pumps 4.2 x 2 pumps
3rd	72,000m^3/day	0.5Q x 6 0.25Q x 2 (0.5Q, one Stand-by)	8.4 x 6 pumps

4-2-1-6 Proposed Water Treatment Plant Electric Equipment

a) Generator Capacity

1) Power Equipment Load

Equipment	Unit Load (KW)	No.of Units	Effective No. of Units	Effective Load (KW)
Flush Mixer	0.75	2	2	1.5
Agitator	0.75	8	8	6
Link Belt Sludge Scraper	0.75	2	2	1.5
Sludge Suction Pump	15	2	1	15
No.1 Drain Pump (For Clear Water Reservoir)	5.5	1	1	5.5
No. 3 Drain Pump(For waste water and drain of filter)	2.2	1	1	2.2
No. 2 Drain Pump (For Lagoon)	3.7	1	1	3.7
Distribution Pump	55	2	2	110
" "	100	2	1	100
Lift Pump (For Elevated Tank)	15	2	1	15
Water Lubricating Pump	0.75	2	1	0.75
Chemical Dosing System (Alum)	7.5			7.5
Chemical Dosing System (Chlorine & Soda Ash)	16.55			16.55
Generator Accessory	10			10
Total				295

295.0 x 1.25 = 370 KVA

2) Illumination Equipment Load

Room	Load (W)
Chemical Dosing Room	8000
Rapid Sand Filter	4300
Sedimentation Basin	1800
Distribution Pump Room	1240
Warehouse	240
Generator Room	4280
Lodging House	17600
Office	5900
Outdoor	120
Total	43480

$$86969^{W} \times 1.25 \times 10^{-3} = 110^{KVA}$$

3) Total Load

$$370^{KVA} + 54^{KVA} = 424\ ^{KVA}$$

b) Starting Loads

1) Cable Impedance

i) Plant Generator Connecting Cable

$600^{\square}$ 35 m $R = 0.0518\ \Omega/km$ $X = 0.0712\ \Omega/km$

$$Pux = \frac{0.0712 \times 1000}{(0.38)^2 \times 10^3} \times \frac{35}{1000} = 0.0172$$

$$Pur = \frac{0.0518 \times 1000}{(0.38)^2 \times 10^3} \times \frac{35}{1000} = 0.0125$$

ii) F-3 $250^{\square}$ 200 m $R = 0.0937\ \Omega/km$ $X = 0.0728\ \Omega/km$

$$Pux = \frac{0.0728 \times 1000}{(0.38)^2 \times 10^3} \times \frac{200}{1000} = 0.1008$$

$$Pur = \frac{0.0937 \times 1000}{(0.38)^2 \times 10^3} \times \frac{200}{1000} = 0.1297$$

iii) 55 KW (105A) $50^{\square}$ 10 m $R = 0.468\ \Omega/km$ $X = 0.0774\ \Omega/km$

$$Pux = \frac{0.0774 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.0053$$

$$Pur = \frac{0.468 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.0324$$

iv) 100 KW (190A) 150^{0} 10 m $R = 0.15\ \Omega/km$ $X = 0.0733\ \Omega/km$

$$Pux = \frac{0.0733 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.005$$

$$Pur = \frac{0.15 \times 1000}{(0.38)^2 \times 10^3} \times \frac{10}{1000} = 0.0103$$

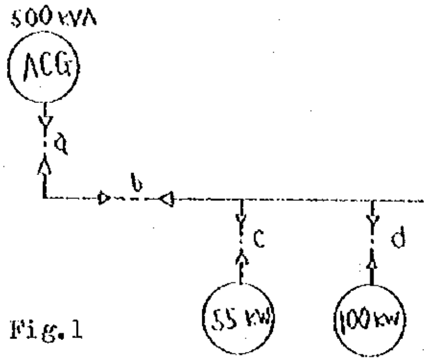


Fig.1

$$Zabc = \sqrt{0.1233^2 + 0.1746^2} = 0.214$$

$$Zabd = \sqrt{0.123^2 + 0.1525^2} = 0.196$$

2) 100 KW Generator Capacity at starting time

$$ZL = \frac{1000 \times 100}{125^{KVA} \times 2} = 400$$ (starting KVA = 200 %)

Generator capacity: $\geq \frac{0.8 \times 25 \times 1000}{0.2 \times 400 - 0.8 \times 0.196} = \frac{0.8 \times 25 \times 1000}{79.84} = 250^{KVA}$

(assuming generator 25%)

3) When there is a base load:

$P_1 = 250$ KVA $\qquad P_2 = 245 + 54 = 299$ KVA

$W_1 = 250 \times 0.2 = 50$ KW $\qquad W_2 = 236$ KW

$Q_1 = 250 \times \sqrt{1 - 0.2^2} = 245$ kvar $\qquad Q_2 = 177$ kvar

$$P = \sqrt{(245 + 177)^2 + (50 + 236)^2} \doteq 500 \text{ KVA}$$

4) Battery Capacity

1) Type of Battery: Alkali pocket type Rapid Discharge Model

2) Electricity Stoppage Time: 30 min.

3) Temperature in Proximity: $5^{o}C$

4) Breakdown on Loads

i) Load for Normal Operating Periods

Signal Lamps (about 10 locations)

1 location, about 0.1A 10 x 0.1 = 1A

ii) Short Period Loads

Cut-off Operation Current about 100A

5) Battery Capacity

$$C = \frac{1}{0.8}\left[1 \times 1^{A} + 0.3 \times 100^{A}\right]$$

$$= 38.8 \text{ AH/5H} \longrightarrow 40 \text{ AH/5H}$$

6) Output of Battery Charger

Normal Load Current: 1A

$$1^{A} + 40/5 = 9^{A} \longrightarrow 10 \text{ A}$$

4-2-2 Hydrographic Calculations

4-2-2-1 Intake of Raw Water

$$V = \frac{23 + 1/n + 0.00155/I}{1 + (23 + 0.00155/I)\frac{n}{\sqrt{R}}} \sqrt{RI} \;\ldots\ldots \quad (1)$$

But, $I = 0.1$, 0/00 $\quad n = 0.013$

A simplification of formula (1) above is:

$$V = \frac{NR}{\sqrt{R} + D} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

But, $N = (23 + 1/n + \frac{0.00155}{I}) \sqrt{I}$

$D = (23 + \frac{0.00155}{I})\, n$

$$N = (23 + 1/0.013 + \frac{0.00155}{\frac{0.1}{1000}}) \sqrt{\frac{0.1}{1000}} = 1.1542$$

$$D = (23 + \frac{0.00155}{\frac{0.1}{1000}})\, 0.013 = 0.5005$$

$$V = \frac{N\,R}{\sqrt{R} + D} = \frac{11542 \times 0.292}{\sqrt{0.292} + 0.5005} = \frac{0.3375}{1.0405} = 0.324 \text{ m/sec.}$$

$Q = B.h.V = (2.15 \times 0.4 \times 0.324) \times 2 = 0.557\ m^3/sec \quad 0.35 m^3/sec$

Width of Inflow Mouth

Inflow volume: $Q = 0.7\ m^3/sec$

$= Q/2 = 0.35\ m^3/sec$ 1 basin

Inflow speed: $v = 0.3$ m/sec, estimated

Inflow depth: $h = 0.2$ m, estimated

Inflow area: $A = \frac{Q}{v} = \frac{0.35}{0.3} = 1.167\ m^2$

Inflow width: $B = \frac{A}{h} = \frac{1.167}{0.2} = 5.84 = 6.0$ m (per basin)

Average Velocity at Inflow Mouth

$Q = 60500\ m^3/day = 0.7\ m^3/sec.$

$Q/2 = 30250\ m^3/day = 0.35\ m^3/sec./per$ basin

Width of open channel:

B = 2.15 x 2 gates (per basin)

Water depth: h = 0.4 m, estimated

$A = 2.15\ m \times 0.4\ m \times 2\ gates = 1.72\ m^2$ (per basin)

$P = (0.4 + 2.15 + 0.4) \times 2 = 5.9\ m$

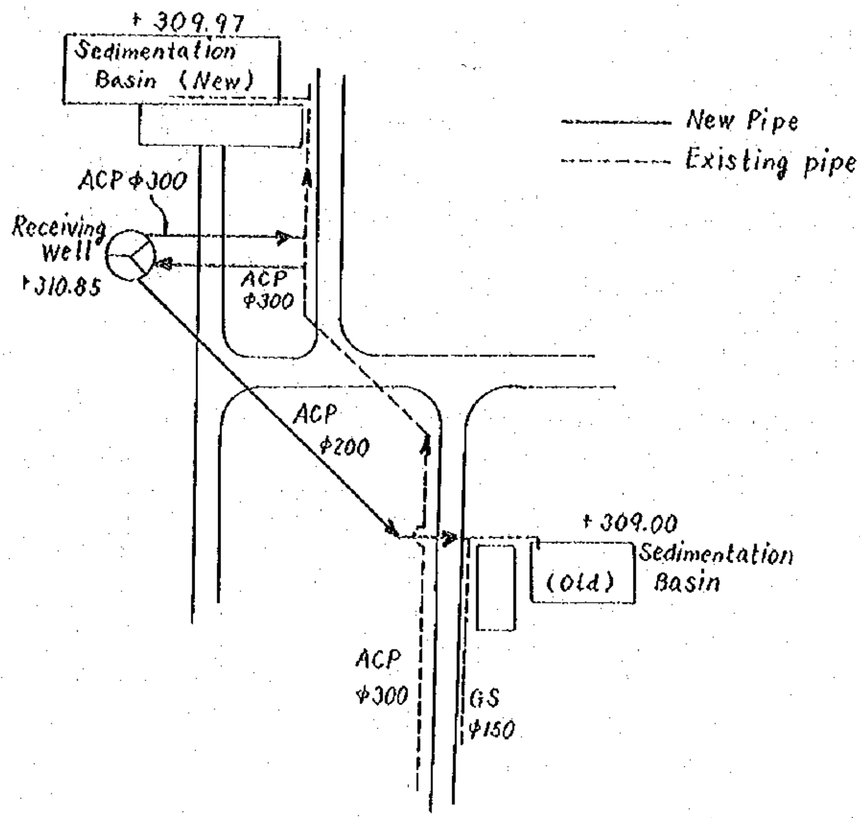
$R = A/P = 1.72/5.9 = 0.292$

4-2-2-2 Delivery of Raw Water to Purification Plant

(C = 100)

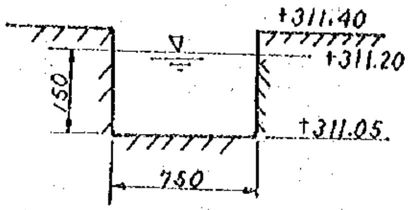
	For Existing Plant	For Proposed Plant 1st Stage	2nd Stage	3rd Stage
Q (m^3/D)	7700	17600	35200	52800
I (m)	4190	3110		
D (mm)	300	400	520 (400, 400)	607 (400, 400, 400)
I (°/oo)	8.4	9.6	9.6	9.6
V (m/sec)	1.25	1.6	1.6	1.6
Hf (m)	35.2	29.9	29.9	29.9
Receiving Well Water Level	+311.2	+311.5		
Intake Plant W.W.L.	+300.5			
Actual Head (m)	10.7	11.0	11.0	11.0
Pump Loss (m)	2.1	2.1	2.1	2.1
Total Head (m)	48.0	43.0	43.0	43.0

4-2-2-3 Connecting Mains for Water Treatment Plant No.1



a) Receiving Well

1) For New Sedimentation Basin



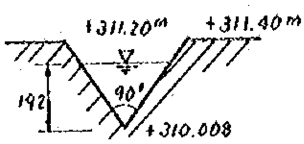
Q = 250 m^3/hr 4.17 m^3/min

Rectangular Weir

Width at Weir: B = 750 mm

Overflow Depth: H = 150 mm

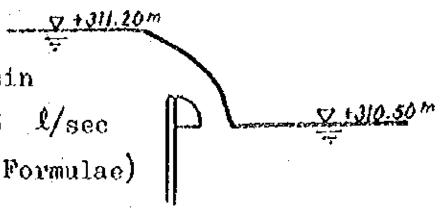
2) For Old Sedimentation Basin



Q = 80 m^3/hr = 1.34 m^3/min

Triangular Weir

Overflow water depth: H = 192 mm



b) Receiving Well - New Sedimentation Basin

Q = 250 m^3/hr = 41.7 m^3/min = 69.5 ℓ/sec

1) Friction loss (by Hazen William's Formulae)

L = 40 m   D = 300 mm   C = 100   I = 5.5 $^{0}/_{00}$ (V=1.0$^{m}/$sec)

$hf = \frac{5.5}{1000} \times 40 = 0.22$ m

2) Bend loss: 4 Places

$hb = 1.0 \times \frac{1.0^2}{2g} \times 4 = 0.20$ m

3) Head loss due to inflow

$he = 0.2\frac{1.0^2}{2g} = 0.01$ m

4) Head loss due to outflow

$ho = 1.0 \times \frac{1.0^2}{2g} = 0.05$

Sum of loss

$\sum h = 0.22 + 0.20 + 0.01 + 0.05 = 0.48$ m

Water Level of New Sedimentation Basin

H = (Level of Receiving Well) - (Pipe loss)

= 310.85 - 0.48

= 31037 m   309.97 m

C) Receiving Well - Old Sedimentation Basin

$Q = 80\ m^3/hr = 1.34\ m^3/min = 22.3\ \ell/sec$

1) Friction loss (by Hazen-William's Formulae)

L_1=130m D_1=200mm   C=100 I_1=4.6$^{0}/_{00}$ V_1=0.70m/sec

L_2= 20m D_2=150mm   C=100 I_2=19 $^{0}/_{00}$ V_2=1.26m/sec

$hf = \frac{4.6}{1000} \times 130 + \frac{19}{1000} \times 20 = 0.98$ m

2) For inflow

$he = 0.2 \times \frac{0.7^2}{2g} = 0.05$ m

3) For sudden decreases

$hsc = 0.25 \times \frac{1.26^2}{2 \times 9.8} = 0.02$ m

4) Head loss due to bends

$hb = 3 \times \frac{1.26^2}{19.6} = 0.24$ m

5) Head loss due to outflow

$$ho = 1.0 \times \frac{1.26^2}{2g} = 0.08 \text{ m}$$

Sum of loss

$$\sum h = 0.98 + 0.05 + 0.02 + 0.24 + 0.08 = 1.37 \text{ m}$$

Water Level of Old Sedimentation Basin

$$H = 310.50 - 1.37 = +309.13 \text{ m}$$

4-2-2-4 <u>Connecting Mains at Paton Water Treatment Plant</u>

a) <u>Receiving Well</u>

Flow volume: $Q = 16000 \times 1.08 = 17280 \text{ m}^3/\text{d}$

$= 11.1 \text{ m}^3/\text{min} = 0.185 \text{ m}^3/\text{sec}$

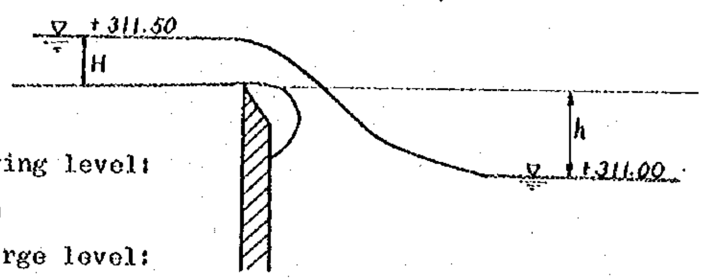
Type: Rectangular

Width: $B = 2000$ mm

Overflow water depth: $Q = 1838\ B.H^{\frac{3}{2}}$ Using this equation;

$H = 150$ mm

Head: $h = 350$ mm



Water receiving level:

+311.50 m

Water discharge level:

+311.00 m

b) <u>Receiving Well</u> - <u>Head loss between the receiving well and the mixing basin</u>

$Q = 16000 \text{ m}^3/\text{d} = 11.1 \text{ m}^3/\text{min} = 0.185 \text{ m}^3/\text{sec}$

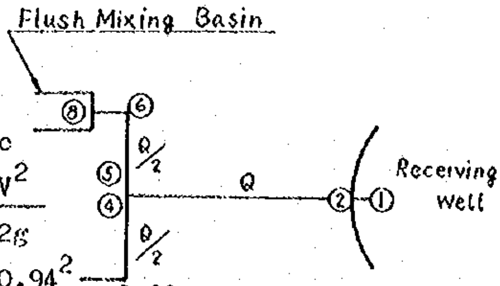
1) Head loss due to inflow

Pipe diameter: $D = 500$ mm

Flow velocity: $V = Q/A = \dfrac{0.185}{\frac{3.14}{4} \times 0.5^2} = 0.94 \text{ m/sec}$

Head loss: $h_1 = 0.2 \times \dfrac{0.94^2}{2 \times 9.8} = 0.01 \text{ m}$

2) Head loss at bends



Pipe diameter: $D = 500$ mm

Flow velocity: $V = 1.02$ m/sec

Head loss: $h_2 = f_{b1} \times f_{b2} \times \frac{V^2}{2g}$

$= 0.2 \times 1.0 \times \frac{0.94^2}{19.6} = 0.01$ m

3) Head loss due to friction

Pipe diameter: $D = 500$ mm

Coarseness coefficient: $n = 0.013$

Distance: $L = 14.442$ m

Flow velocity: $V = 0.94$ m/sec

Head loss: $h_3 = f \cdot \frac{L}{D} \cdot \frac{V^2}{2g} = 0.0265 \times \frac{14.442}{0.5} \times \frac{0.94^2}{19.6} = 0.03$

$$f = \frac{124.5n^2}{D^{\frac{1}{3}}} = \frac{124.5 \times 0.013^2}{0.5^{\frac{1}{3}}} = 0.0265$$

4) Head loss at T-branches, etc.

Pipe diameter: $D = 500$ mm

Flow velocity: $V = 1.02$ m/sec

Head loss: $h_4 = f_{be} \cdot \frac{V^2}{2g} = 0.99 \times \frac{0.94^2}{19.6} = 0.04$ m

5) Head loss due to gradual decrease in pipe diameter

Pipe diameter before decrease: $D_1 = 500$ mm

Pipe diameter after decrease: $D_2 = 400$ mm

Area ratio: $A_2/A_1 = \frac{D2^2}{D_1^2} = 0.64$

Flow velocity: $V = 0.94$ m/sec

Head loss: $h_5 = f_{ge} \cdot \frac{V^2}{2g} = 0.02 \times \frac{0.94^2}{19.6} = 0.00$ mm

6) Head loss at bends

Pipe diameter: $D = 400$ mm

Flow velocity: $V = (Q/2)/A = \dfrac{0.0926}{\frac{3.14}{4} \times 0.4^2} = 0.74$ m/sec

Head loss: $h_6 = f_{b1} \times f_{b2} \times \dfrac{V^2}{2g} = 0.18 \times 1.0 \times \dfrac{0.74^2}{19.6} = 0.01$ m

7) Head loss due to friction

Pipe diameter: D = 400 mm

Coarseness coefficient: n = 0.013

Distance: L = 8.50 m

Flow velocity: V = 0.80 m/sec

Head loss: $h_7 = f.\dfrac{L}{D}\dfrac{V^2}{2g} \times 2$ places $= 0.0285 \times \dfrac{8.50}{0.40} \times$

$$\frac{0.74^2}{19.6} \times 2 = 0.03$$

$$f = \frac{124.5n^2}{D^{\frac{1}{3}}} = \frac{124.5 \times 0.013^2}{0.4^{\frac{1}{3}}} = 0.0285$$

8) Head loss due to outflow

Pipe diameter: D = 400 mm

Flow velocity: V = 0.80 m/sec

Head loss: $h_8 = f_{se} \cdot \dfrac{V^2}{2g} = 1.0 \times \dfrac{0.74^2}{19.6} = 0.03$ m

Following calculations assume a total head loss of 0.14 m in the connecting main between the receiving well and the mixing basin.

0.01 + 0.01 + 0.03 + 0.04 + 0.00 + 0.01 + 0.03 + 0.03 + 0.14 = 0.30 m

Water level of the rapid flocculator basin is +310.70 m.

c) Head losses for the mixing basin, flocculation basin, and sedimentation basins

Treated water: $Q = 8{,}000$ m^3/d

$= 0.0925$ m^3/sec

Rapid mixing shall be of the type using a flush mixer.

9) Upward orifice head loss

Area: $A\ \phi 700 = \frac{3.14}{4} \times 0.7^2 = 0.385\ m^2$

Flow velocity: $V = \frac{Q}{A} = \frac{0.0926}{0.385} = 0.240\ m/sec$

Sectional area ratio: 0.385/1.9 x 1.9 = 0.11

Head loss: $h_9 = f_{ec} \cdot \frac{V^2}{2g} = 3.6 \times \frac{0.24^2}{19.6} = 0.01\ m$

10) Head loss of submerged weir

Width of weir: B = 1.90 m

Flow volume: $Q = 0.0925\ m^3/s$

$$Q = \frac{2}{3}\sqrt{2gU_1}Bh_1^{\frac{3}{2}}\left\{\frac{3}{2}\frac{\mu_2}{\mu_1} + \left(1-\frac{3}{2}\frac{\mu_1}{\mu_1}\right)\left(1-\frac{h_2}{h_1}\right)\right\}\left(1-\frac{h_2}{h_1}\right)^{0.28}$$

$$0.10 = \frac{2}{3}\sqrt{2 \times 9.8} \times 0.66 \times 1.90 \times h_1^{\frac{3}{2}}\left\{\frac{3}{2} \times \frac{0.586}{0.660} + \left(1-\frac{3}{2} \times \frac{0.586}{0.660}\right)\left(1-\frac{h_2}{h_1}\right)\right\}\left(1-\frac{h_2}{h_1}\right)^{0.28}$$

$h_1 = 0.04m$, $h_2 = 0.02\ m$

11) Head loss at inlet mouth of floc coagulation basin

Flow volume: $Q = 4.000\ m^3/d = 2.8\ m^3/min$

$= 0.05\ m^3/sec$

Sectional area: $A = 0.4 \times 0.4 = 0.16\ m^2$

Flow velocity: $V = Q/A = 0.05/0.16 = 0.31\ m/sec$

Head loss: $h_{11} = 1.0 \times \frac{V^2}{2g} = 1.0 \times \frac{0.31^2}{19.6} = 0.00\ m$

12) Head loss at outlet of flocculation basin

(same as directly above)

Head loss: $h_{12} = 0.00\ m$

13) Head loss at 90^o bends

Sectional area: $A = 2.4 \times 0.8 = 1.92\ m^2$

Flow velocity: $V = Q/A = 0.05/1.92 = 0.026\ m/sec$

Head loss: $h_{13} = f_{bo} \frac{V^2}{2g} = 0.99 \times \frac{0.026^2}{2 \times 9.8} = 0.00$ m

14) Head loss at flow control orifice:

$$A = \frac{3.14}{4} \times 0.15^2 \times (17 \times 7) = 2.10 \ m^2$$

Flow volume: $Q = 0.0926 \ m^3/sec$

Flow velocity past flow control orifice:

$V = 0.0926/2.10 = 0.044$ m/sec

Head loss: $h_{14} = \frac{1}{0.6^2} \times \frac{0.044^2}{2g} = 0.00$ m

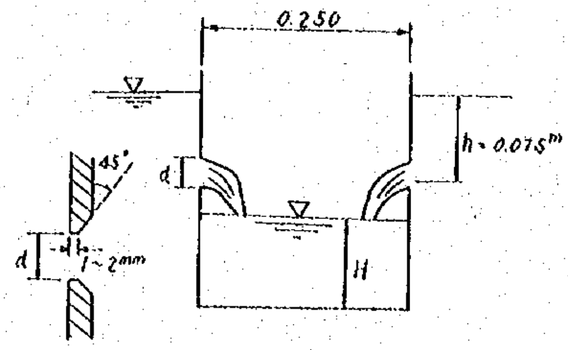
$$0.01 + 0.04 + 0.02 \doteq 0.07$$

Thus, the designed water level, with a tolerance included, of the sedimentation basin is +310.50 m.

15) <u>Calculation of Capacities for Trough</u>

i) Orifice

$Q = CA\sqrt{2gh}$



Here, $C = 0.592 + 0.000677 \left(\frac{1}{d\sqrt{h}} \right)^{0.7636}$

(Mawson's formula)

When a volume of 8,000 m^3/day of water is collected by six troughs

$Q = 8000 \div 6 = 1333$ m^3/day per Trough = 0.0154 m^3/sec

Temporarily determining that d = 26 mm

$$C = 0.592 + 0.000677 \left(\frac{1}{0.026\sqrt{0.075}} \right)^{0.7636}$$

Furthermore, the value of X becomes,

$$X = \left(\frac{1}{0.026\sqrt{0.075}} \right)^{0.7636} = 140^{\ 0.7636}$$

$\log X = 0.7636 \log 140$

$= 2.1461 \times 0.7636 = 1.639$

$X = 43.55$, which may be inserted in the formula for the value of C

$$C = 0.592 + 0.000677 \times 43.55 = 0.621$$

$$Q = 0.621 \quad A\sqrt{2 \times 9.8 \times 0.075} = 0.0154$$

$$A = \frac{0.0154}{0.621\sqrt{2 \times 9.8 \times 0.075}} = \frac{0.0154}{0.75} = 0.02226$$

If there are 42 holes of 26 mm diameter,

$$A = \frac{\pi \times 0.026^2}{4} \times 42 = 0.022 > 0.021$$

ii) <u>Calculations for Depth of Trough</u>

ho (water depth at upper flow edge of trough) > hc (water depth at lower flow edge of trough)

First to be determined is hc, the critical depth

$$hc = \sqrt[3]{\frac{\alpha Q^2}{gb^2}} \qquad \alpha = 1.1, \quad g = 9.80\ m/sec^2 \quad b = 0.25$$

$$hc = \sqrt[3]{\frac{1.1 \times 0.0\ 54}{9.8 \times 0.25^2}}$$

$$= \sqrt[3]{0.000425} = 0.076$$

$$Vc = \sqrt[3]{\frac{gQ}{\alpha b}} \qquad \alpha = 1.1 \quad g = 9.80\ m/sec^2$$

$$b = 0.25\ m$$

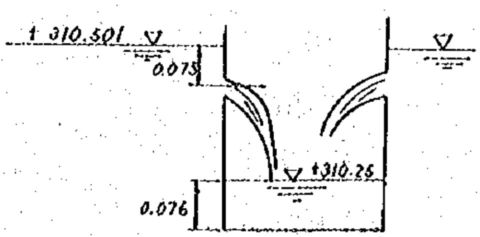
$$= \sqrt[3]{\frac{9.8 \times 0.0154}{1.1 \times 0.25}} = 0.818\ m/sec$$

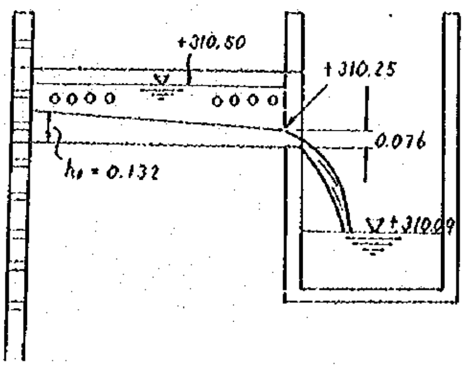
$$ho = \sqrt{2h_c^2 + \left(hc - \frac{il}{3}\right)^2} - \frac{2}{3}\, il \quad \text{(Thomas Camp's formula)}$$

$$ho = \sqrt{3hc^2} \qquad \because i = 0$$

$$= \sqrt{3 \times 0.076^2} = 0.132$$

Thus the value ho for the upper edge of the trough is ho = 0.132 m





d) <u>Chemical Sedimentation Basin - Rapid Sand Filter</u>

16) Head loss for inflow

Flow volume: $Q = 16000\ m^3/d = 11.11\ m^3/min$

$= 0.185\ m^3/sec$

Pipe diameter: $D = 500$ mm

Flow velocity: $V = \dfrac{0.185}{\dfrac{3.14}{4} \times 0.5^2} = 0.94$ m/sec

Head loss: $h_{16} = fe\dfrac{V^2}{2g} = 0.2 \times \dfrac{0.94^2}{19.6} = 0.01$ m

17) Head loss at bends

Flow volume: $Q = 0.185\ m^3/sec$

Pipe diameter: $D = 500$ mm

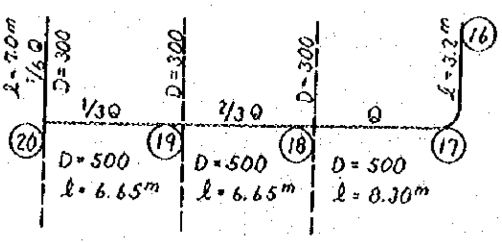
Flow velocity: $V = 0.94$ m/sec

Head loss: $h_{17} = f_{b1} \times f_{b2} \times \dfrac{V^2}{2g}$

$= 0.15 \times 1.00 \times \dfrac{0.94^2}{19.6}$

$= 0.01$ m

From Sedimentation Basin



18) Head loss at branch, etc.

4-way cross shall be treated as 2 T-flanges.

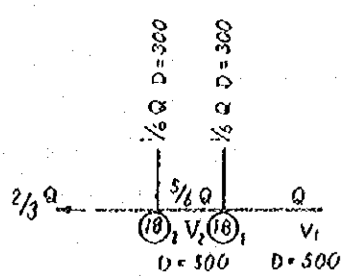
Flow volume: $Q = 0.185\ m^3/sec$

Pipe diameter: $D = 500$ mm

Flow velocity: $V_1 = 0.94$ m/sec

Head loss : $h_{18-1} = f_r \times \dfrac{V_1^2}{2g}$

$= 0.00 \times \dfrac{1.0^2}{19.6} = 0.00$ m



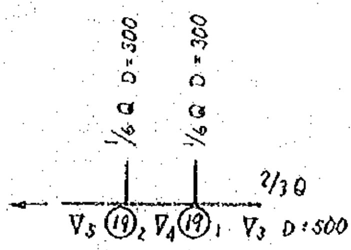
Flow volume: $\frac{5}{6}Q = 0.154\ m^3/sec$

Pipe diameter: $D = 500$ mm

Flow volume: $V_2 = 0.79$ m/sec

Head loss: $h_{18-2} = f_r \frac{V_2^2}{2g} =$

$$0.00 \times \frac{0.79^2}{19.6} = 0.00 \text{ m}$$



Flow volume: $\frac{2}{3}Q = 0.123\ m^3/sec$

Pipe diameter: $D = 500$ mm

Flow velocity: $V_3 = 0.628$ m/sec

Head loss: $h_{19-1} = f_r \frac{V_3^2}{2g} =$

$$0.00 \times \frac{0.628^2}{19.6} = 0.000 \text{ m}$$

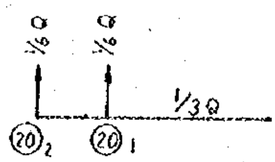
Flow volume: $\frac{1}{2}Q = 0.093\ m^3/sec$

Pipe diameter: $D = 500$ mm

Flow velocity: $V_4 = 0.47$ m/sec

Head loss: $h_{19-2} = f_r \frac{V_4}{2g} =$

$$0.01 \times \frac{0.47^2}{19.6} = 0.000 \text{ m}$$



Flow volume: $\frac{1}{3}Q = 0.061\ m^3/sec$

Pipe diameter: $D = 500$ mm

Flow velocity: $V_5 = 0.31$ m/sec

Head loss: $h_{20-1} = f_r \frac{V_5^2}{2g} =$

$$0.05 \times \frac{0.31^2}{19.6} = 0.000 \text{ m}$$

The final T-flange is assumed to have 2 head loss equal to that of a 90° bend.

Flow volume: $\frac{1}{6}Q = 0.031\ m^3/sec$

Pipe diameter: $D = 300$ mm

Flow velocity: $V = 0.031/0.071 = 0.437$ m/sec

Head loss: $h_{20-2} = f_{be} \times \frac{V^2}{2g} = 0.99 \times \frac{0.437^2}{19.6} = 0.01$ m

19) Head loss due to friction

Flow volume and flow velocity:

$Q_1 = Q = 0.185\ m^3/sec$

$V_1 = Q_1/A = 0.94\ m/sec$

$Q_2 = 2/3Q = 0.123\ m^3/sec$

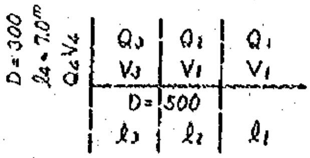
$V_2 = Q_2/A = 0.628\ m/sec$

$Q_3 = 1/3Q = 0.061\ m^3/sec$

$V_3 = Q_3/A = 0.311\ m/sec$

$Q_4 = 1/6Q = 0.0311\ m^3/sec$

$V_5 = Q_4/A = 0.437\ m/sec$



Coarseness coefficient:

$n = 0.013$

$$f_1 = \frac{124.5 \times n^2}{D^{1/3}} \times n^2 = \frac{124.5 \times 0.013^2}{0.5^{1/3}} = 0.026 \quad (D{=}500mm)$$

$$f_2 = \frac{124.5 \times 0.013^2}{0.3^{1/3}} = 0.031 \quad (D{=}300mm)$$

Head loss: $h_{19} = \sum_{i=1}^{n} f_1 \frac{i}{Di} \frac{V_1^2}{2g}$

$$= \frac{f_1}{0.5 \times 19.6} \times (11.50 \times 0.94^2 + 6.65 \times 0.628^2$$

$$+ 6.65 \times 0.311^2) + f_2 X \frac{7.0}{0.3} X \frac{0.437^2}{19.6}$$

$$= 0.04$$

20) Head loss due to flow loss

Flow volume: $\frac{1}{6}Q = 0.031\ m^3/sec$

Flow velocity: $V = Q/A = 0.031/0.071 = 0.437\ m/sec$

Head loss: $h_{20} = f_{se}\ \frac{V^2}{2g} = 1.0 \times \frac{0.437^2}{19.6} = 0.01\ m$

Head loss between the sedimentation basin, and the rapid sand filter: 0.09 m.

The water level for the sand filter basin is estimated as +310.00 m.

e) Rapid sand filter

21) The permissible head loss within the sand filter basin is 3.75 m.

f) In connecting mains between the sand filter basin and clean water reservoir.

22) Head loss due to inflow

Flow volume: $Q = 8000 \times 1.0 = 8000\ m^3/d = 5.56\ m^3/min$

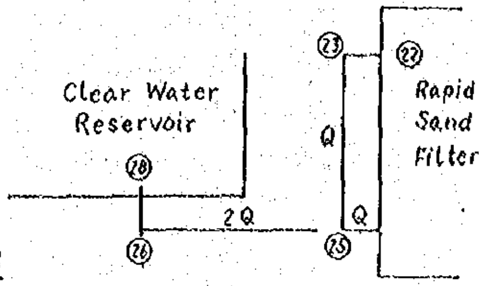
$= 0.0926\ m^3/sec$

Pipe diameter: $D = 400$ mm

Flow velocity : $V = Q/A = 0.0923/0.126 = 0.733$ m/sec

Head loss: $h_{22} = f_e \frac{V^2}{2g} = 0.2 \times \frac{0.733^2}{19.6} = 0.01$ m

23) Head loss due to bends



Flow volume:

$Q = 0.0926\ m^3/sec$

Pipe diameter:

$D = 400$ mm

Flow velocity: $V = 0.733$ m/sec

Head loss: $h_{23} = f_{b_1} \times f_{b_2} \times \frac{V2}{2g}$

$= 0.15 \times 1.0 \times \frac{0.733^2}{19.6}$

$= 0.004$ m

24) Head loss due to friction

Flow volume: $Q = 0.0926\ m^3/sec$

Pipe diameter: $D = 400$ mm

Flow velocity: $V = 0.733$ m/sec

Distance: $L = 18.44$ m

Coarseness coefficient: $n = 0.013$

Head loss from friction:

$$h_{24} = f \frac{L}{D} \frac{V^2}{2g} = 0.03 \times \frac{18.44}{0.4} \times \frac{0.733^2}{19.6} = 0.04$$

$$f = \frac{124.5\ n^2}{D^{1/3}} = \frac{124.5 \times 0.013^2}{0.4^{1/3}} = 0.03$$

25) Head loss due to conjoining currents

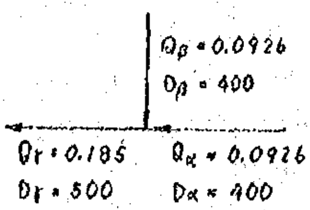
Sectional area ratio of trunk main to branch main:

$$\varphi = \frac{A}{A} = \frac{0.4^2}{0.4^2} = 1.0$$

Flow volume ratio:

$$q_s = \frac{Q}{Q} = 1.0$$

Flow velocity after joining:



$$Vr = \frac{Qr}{Ar} = \frac{0.185}{\frac{3.14}{4} \times 0.5^2}$$

$= 0.943$ m/sec

Head loss: from Gardel formula

$$h_{25} = 0.9 \times \frac{Vr^2}{2g} = 0.9 \times \frac{0.943^2}{19.6} = 0.041 \text{ m}$$

26) Head loss due to bends

Flow volume: $Q = 0.185$ m^3/sec

Pipe diameter: $D = 500$ mm

Flow velocity: $V = 0.943$ m/sec

Head loss: $h_{26} = f_{b_1} \times f_{b_2} \times \frac{V^2}{2g} = 0.15 \times 1.0 \times \frac{0.943^2}{19.6}$

$= 0.007$ m

27) Head loss from friction

Flow volume: $Q = 0.185$ m^3/sec

Pipe diameter: $D = 500$ mm

Flow velocity: $V = 0.943$ m/sec

Distance: $L = 26.1$ m

Coarseness coefficient: $n = 0.013$

Head loss from friction: $h_{27} = f \frac{L}{D} \frac{V^2}{2g}$

$$= 0.03 \times \frac{26.1}{0.5} \times \frac{0.943^2}{19.6} = 0.07 \text{ m}$$

$$f = \frac{124.5n^2}{D^{1/3}} = \frac{124.5 \times 0.013^2}{0.5^{1/3}} = 0.03$$

28) Head loss from outflow

Flow volume: $Q = 0.185$ m^3/sec

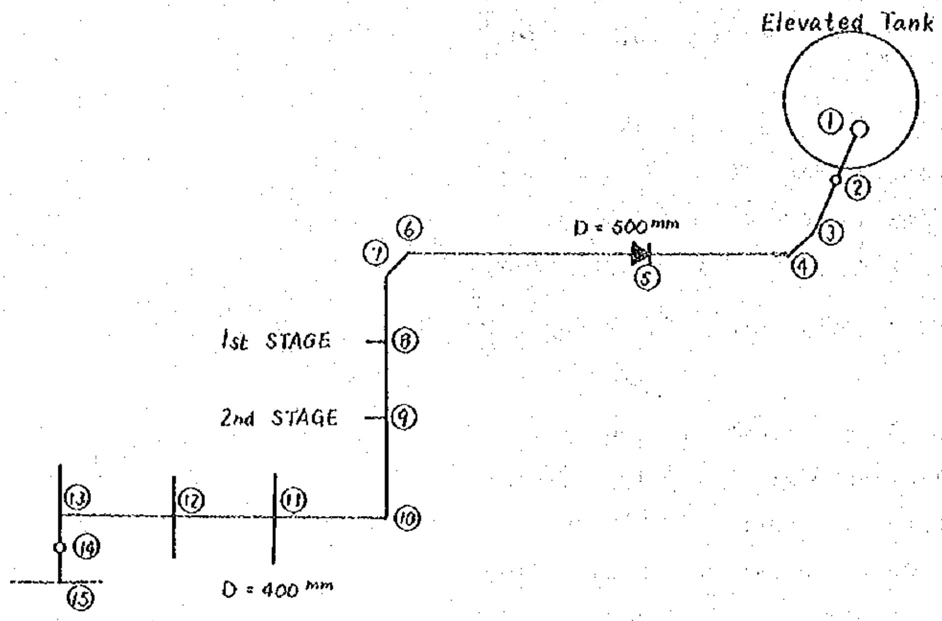
Pipe diameter: $D = 500$ mm

Flow velocity: $V = 0.943$ m/sec

Head loss: $h_{28} = f_{se} \frac{V^2}{2g} = 1.0 \times \frac{0.943^2}{19.6} = 0.045$ m

Thus, the H.W.L. of the clean water reservoir shall be 306.00 m.

g) Elevated Tank - Rapid Sand Filter (Backwash Water)



$Q = 32\ m^2 \times 0.5 m^3/m^2.min = 16\ m^3/min = 0.267 m^3/sec$

Head loss due to inflow(1 place: ①)

Pipe diameter: $D = 500$ mm $\quad A = 0.196\ m^2$

Flow velocity: $V = \dfrac{0.267}{\frac{3.14}{4} \times 0.5^2} = 1.36 m/sec$

Head loss: $h_1 = fe \dfrac{V^2}{2g} = 0.2 \times \dfrac{1.36^2}{2 \times 9.8} = 0.02$ m

90° bend head loss (3 places: ① , ② x 2)

Pipe diameter: $D = 500$ mm $\quad A = 0.196\ m^2$

Flow velocity: $V = 1.36$ m/sec

Head loss: $H_B = fb1 \times fb2 \times \dfrac{V^2}{2g} \times 3$ places $= 0.15 \times 1.0 \times \dfrac{1.36^2}{2 \times 9.8}$
$\times 3 = 0.04$ m

Head loss for 22 $1/2^\circ$ bend (1 place: ③)

Pipe diameter: $D = 500$ mm $\quad A = 0.196\ m^2 \quad \dfrac{P}{D} = \dfrac{2.2}{0.5} = 4.4$

Flow velocity: $V = 1.36$m/sec

Head loss: $H_B = 0.075 \times 0.4 \times \dfrac{1.36^2}{2 \times 9.8} = 0.003$ m

Head loss for 45° bend (3 places: ④ , ⑥ , ⑦)

Pipe diameter: $D = 500$ mm $\quad A = 0.196\ m^2 \quad \dfrac{P}{D} = \dfrac{1.10}{0.5} = 2.2$

Flow velocity: $V = 1.36$ m/sec

Head loss: $H_B = 0.15 \times 0.68 \times \frac{1.36^2}{2 \times 9.8} \times 3 = 0.03$ m

Head loss for 90° bend (3 places: ⑩ , ⑬ , ⑭)

Pipe diameter: $D = 500$ mm $A = 0.196$ m²

Flow velocity: $V = 1.36$ m/sec

Head loss: $H_{Be} = 0.99 \times \frac{1.36^2}{2 \times 9.8} \times 3 = 0.28$ m

Head loss for 90° joint (1 place: ⑭)

Pipe diameter: $D = 400$ mm $A = 0.126$ m²

Flow velocity: $V = \frac{0.267}{0.126} = 2.12$m/sec

Head loss: $H_{Be} = 0.99 \times \frac{2.12^2}{2 \times 9.8} = 0.23$ m

Friction head loss (D = 500 mm)

Pipe diameter: $D = 500$ m $A = 0.196$ m²

Flow velocity: $V = 1.36$m/sec

Roughness coefficient: N: 0.013

$$f = \frac{124.5 \times n^2}{D^{1/3}} = \frac{124.5 \times 0.013^2}{0.5^{1/3}} = 0.026$$

Head loss: $H_f = \frac{f}{D\ 2\ g}\ L$

$$= \frac{0.026}{0.5 \times 19.6} (7.446 + 1.704 + 7.473 + 37.955 + 12.2 + 40.8 + 1.5 \times 2 + 2.5 + 20.4 - 3.2 + 2.3 + 0.96) = 0.36\text{m}$$

Friction head loss (D = 400 mm)

Pipe diameter: $D = 400$ mm $A = 0.126$ m²

Flow velocity: $V = 2.12$ m/sec

Roughness coefficient: $n = 0.013$

$$f = \frac{124.5 \times 0.013^2}{0.4^{1/3}} = 0.03$$

Head loss: $H_f = \frac{f}{D\ 2\ g}\ L$

$$= \frac{0.03}{0.4 \times 19.6} \times 1.66 = 0.06\text{m}$$

Head loss due to outflow

Pipe diameter: $D = 400$ mm $A = 0.126$ m²

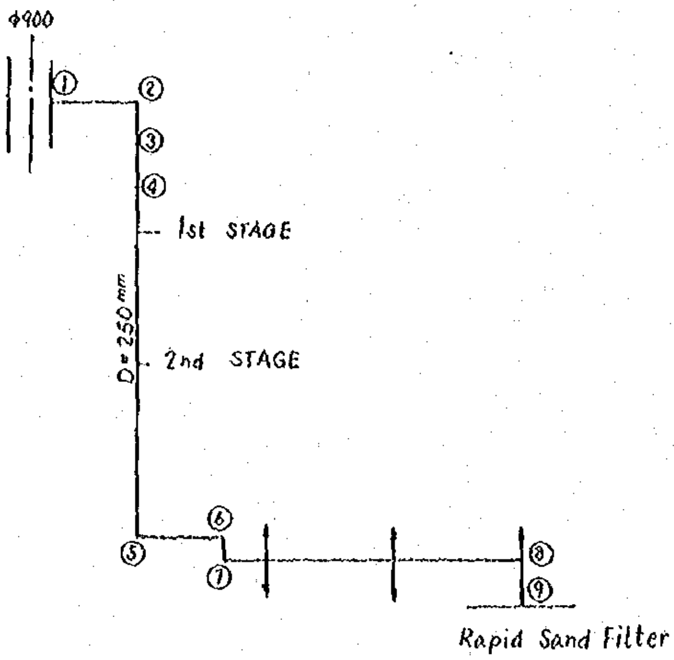
Flow velocity: $V = 2.12$m/sec

Head loss: $H_o = f_o \frac{V^2}{2g} = 1.0 \times \frac{2.12^2}{19.6} = 0.23$ m

Head loss due to Venturi meter 2.5 m

$\sum H = 0.02 + 0.04 + 0.003 + 0.03 + 0.28 + 0.23 + 0.36 + 0.06 + 0.23 + 2.5 \doteq 3.75$ m

h) Distribution Pump Heads - Rapid Sand Filter (Surface Wash)



$Q = 32m^2 \times 0.15m^3/m^2\ min = 4.8m^3/min = 0.08m^3/sec$

$V = \dfrac{0.08}{\frac{\pi}{4} \times 0.25^2} = 1.63m/sec$

Head loss due to inflow (1 place; ①)

$he = fe \dfrac{V^2}{2g} = 0.5 \times \dfrac{1.63^2}{19.6} = 0.07m$

90° bend head loss (5 places; ② , ⑤ , ⑥ x 2, ⑦)

$\dfrac{P}{D} = \dfrac{0.4}{0.25} = 1.6$

$hB_o = 0.15 \times 1.0 \times \dfrac{1.63^2}{19.6} \times 5 = 0.10\ m$

$20^{1/2}$ bend head loss (2 places; ③, ④)

$\dfrac{P}{D} = \dfrac{1.2}{0.25} = 4.8$

$h_B = 0.08 \times 0.44 \times \dfrac{1.63^2}{19.6} \times 2 = 0.10\ m$

90° branch connections (2 places; ⑧, ⑨)

$h_{Bo} = 0.99 \times \dfrac{1.63^2}{19.6} \times 2 = 0.27\ m$

Head loss due to friction

Roughness coefficient: $n = 0.013$

$$f = \frac{124.5 \times 0.013^2}{0.25^{1/3}} = 0.0314$$

$$hf = \frac{0.0314}{0.25 \times 19.6} \times (1.45 + 6.614 + 40.8 + 1.5 \times 2 + 7.136 + 20.4 - 2.5 + 4.9 + 2.3 + 4.4) = 0.57 \text{ m}$$

Friction in surface washing equipment 2.29 m

$$\Sigma H = 0.07 + 0.10 + 0.10 + 0.27 + 0.57 + 2.29 = 3.4 \text{ m}$$

4-2-3 Chemical Dosing and Chlorine Neutralization Equipment

4-2-3-1 Chemical Dosing Facilities

a) Quality of Water from the Ping River (from data at the Existing Old Plant)

Turbidity of Raw Water

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
1966	7.0	-	-	-	-	-	11.0	-	112.0	-	111.0	47.0	57.6
1967	25.0	-	11.0	11.0	36.0	-	42.0	380.0	-	99.0	52.0	26.0	75.8
1968	11.0	25.0	25.0	9.6	160.6		133.1	29.7	88.7	97.5	25.0	11.0	56.0
1969	11.0	8.0	33.0	11.0	11.0	180.0		150.0	-	0.5	-	16.0	52.5
1970	19.0	16.0	-	23.0	-	210.0	190.0	135.0	355.0	-	37.0	40.0	113.9
1971	-	56.0	26.0	52.0	28.0	134.0	37.0	120.0	-	94.0	102.0	45.0	69.4
1972	-	20.0	45.0	30.0		29.0	54.0	108.0	67.0	-	-	-	50.4
													679
Mean	14.6	25.0	28.0	22.8	58.6	138.3	77.9	153.8	155.7	96.8	65.4	30.8	72.3

Alkalinity (yearly average) 60 - 80 ppm

pH (yearly average) 8 - 75

b) Design Turbidity

The Turbidity Frequency Chart, compiled from the above data, is as follows. Where data was not available, the average for that particular month has been calculated and used as the estimated value.

Two peaks appear in the foregoing Turbidity Frequency Chart. The first appears at about 60 ppm and corresponds to seven months. The other peak is at about the 160 ppm level and corresponds to 94% of the year or about eleven months.

The role of chemical dosing in water treatment is very important. In the present designs, preparation to treat turbidity is set as follows.

Design minimum turbidity: 10 ppm

Mean turbidity: 200 ppm

Maximum turbidity: 500 ppm

c) Installation of Alkaline Dosing Equipment

Alkalinity runs high through the year, between 60 to 80 ppm, so that alkaline dosing equipment seems unnecessary on the basis of this data. But observation of the actual operation of Chiang Mai Water Treatment Plant No. 1, using raw water from the Ping River, shows that alkaline dosing is done for about ten to 15 days each month or three months during the operational year, at peak times when maturation of floc is unsatisfactory. Therefore, alkaline dosing equipment has been planned in this design.

d) Design Facilities for Alum

A total of three units each with a capacity to treat 16,000 m^3/day shall be provided, one each for stages, 1, 2 and 3.

Design Turbidity and Chemical Dosing Ratio

	Raw Water Design Turbidity (ppm)	$A_2(SO_4)_3 \cdot 18H_2O$ Dosing Ratio (ppm)
Maximum	500	72
Mean	200	42
Minimum	10	7.5

1 Unit

Quantity of solid alum ($Al_2(SO_4)_3 \cdot 18H_2O$)

$$\text{Max.} = 16{,}000\ m^3/D \times \frac{1}{1000} \times 72 = 1{,}152\ kg/D = 0.8\ kg/min$$

$$\text{Mean} = 16{,}000\ m^3/D \times \frac{1}{1000} \times 42 = 672\ kg/D = 0.467\ kg/min$$

$$\text{Min.} = 16{,}000\ m^3/D \times \frac{1}{1000} \times 7.5 = 120\ kg/D = 0.083\ kg/min$$

Volume of dissolved alum (as a 5% solution of Al_2O_3)

$$V = G + \left(WXP. \frac{Al_2(SO_4)_3 \cdot 18H_2O \quad (666)}{Al_2O_3 \quad (102)} \times \frac{1}{100}\right)$$

V = Volume of alum (as a 5% solution of Al_2O_3)

G = Quantity of solid alum ($Al_2(SO_4)_3 \cdot 18H_2O$)

W = Specific Gravity of V (Al_2O_3 5% solution - 1.184)

P = % coefficient of V (5%)

Max. $1152kg/D + (1.184 \times 5 \times \frac{666}{102} \times \frac{1}{100}) = 2980\ l/D = 2.07\ l/min$

Mean $672kg/D + (1.184 \times 5 \times \frac{666}{102} \times \frac{1}{100}) = 1738\ l/D = 1.21\ l/min$

Min. $120kg/D + (1.184 \times 5 \times \frac{666}{102} \times \frac{1}{100}) = 311\ l/D = 0.22\ l/min$

Injection Pump (Plunger Type)

q_{max} = 2.1 l/min H = 20 m Ps = 0.4 KW

Pump stroke range 10 : 1

Solution Tank

Two solution tanks each with one day's capacity for the mean operating shall be installed, and used alternately.

$1.3^m \times 1.3^m$ $H = 1.5^m$ ($He = 1.04^m$) x 2 tanks

(Steel Structure with Agitator)

Capacity : 1745 l/tank

Agitator: 0.75^{KW}/unit

	1st Stage	2nd Stage	3rd Stage
Dosing Pump	2 sets (1 stand-by)	3 sets (1 stand-by)	4 sets (1 stand-by)
Solution Tank	2 tanks	4 tanks	6 tanks

Method of Alum Dosing

Solid Alum (equivalent to JIS-K-1450) shall be dissolved in a solution tank to form an alum solution (Al_2O_3 5%). The dosing ratio shall be determined by means of jar tests from turbidity meter and raw water flow volume data obtained at the control room, and also by use of the table of Tabulation of Alum for Corresponding Turbidity in Silica Units. Then, the plunger stroke shall be correspondingly adjusted by manual operation, and the dose injected into the head of the flocculator tank.

e) <u>Lime Soda Feeder Equipment</u>

The consumption rate of alkali at time of maximum turbidity (500 ppm)

72 ppm (alum dosing rate) x 0.45 (consumption rate) = 32.5 ppm

Lime Soda Dosing Rate

32.5 ppm x 1.06 (Lime Soda required to increase alkalinity 1^{o}) = 33.92 ppm

Lime Soda Quantities

1st Stage $16,000\ m^3/day \times 33.92^{ppm} \times \frac{1}{1000} = 542.72 kg/D$

2nd Stage $32,000\ m^3/day \times 33.92^{ppm} \times \frac{1}{1000} = 1,085.44 kg/D$

3rd Stage $48,000\ m^3/day \times 33.92^{ppm} \times \frac{1}{1000} = 1,628.16 kg/D$

Lime Soda Solution Dosing Volume (at a strength of 20% W/V %)

1st Stage $542.72 kg/day \times \frac{100}{20} = 2.713 l/day = 1.88 l/mo.$

2nd Stage $1,085.44 kg/day \times \frac{100}{20} = 5,427 l/day = 3.77 l/mo.$

3rd Stage $1,628.16 kg/day \times \frac{100}{20} = 8,139 l/day = 5.65 l/mo.$

Injection Pump (gyrating process pump)

The pump volume is set so that flow velocity in the pipes is 1.0m/sec in order to avoid sediment accumulating in the dosing pipe.

Sectional area of pipe 25 mm dia. $A = 0.00049\ m^2$

$V = 1.0 m/sec$ $Q \doteq 0.5 l/sec = 30\ l/min$

$q = 30\ l/min$ $H = 20\ m$ $Ps = 0.75\ KW \times 2$ pumps (1 stand-by)

Solution Tanks

1.3m x 1.3m H = 2.0m (He = 1.5m) x 2 tanks x 2 units

(Steel Structure with Agitator)

Capacity: 2500 l/tank

Agitator: 0.75 KW/unit

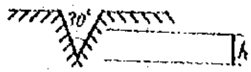
Gauging Well

B = 0.5 m L = 1.0 m H = 0.8 x 1 unit(steel structure)

Calculations for overflow notch (30°)

$$Q(m^3/sec = Ch^{5/2}$$

$$C = 0.3109\ (0.5769 + \frac{0.00394}{h})$$



When h = 2.9 cm

$$Q = 0.3109\ (0.5769 + \frac{0.00394}{0.029}) \times 0.029^{5/2} \times 86400 \times \frac{1}{10^3}$$

$$= 5470\ l/D$$

When h = 3.9 cm

$$Q = 0.3109\ (0.5769 + \frac{0.00394}{0.039}) \times 0.039^{5/2} \times 86400 \times \frac{1}{10^3}$$

$$= 2{,}742\ l/D$$

When h = 4.7

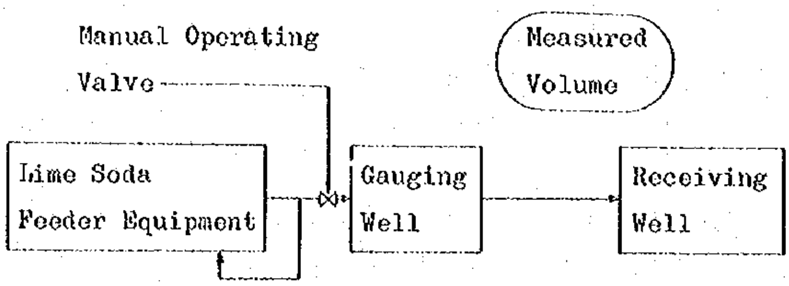
$$Q = 0.3109\ (0.5769 + \frac{0.00394}{0.047}) \times 0.047^{5/2} \times 86400 \times \frac{1}{10^3}$$

$$= 8500\ l/D$$

	1st Stage	2nd Stage	3rd Stage
Dosing Pump	2 sets (1 stand-by)	2 sets (1 stand-by)	2 sets (1 stand-by)
Solution Tank	2 tanks	4 tanks	4 tanks
Gauging Well	1 tank	1 tank	1 tank

Lime Soda Dosing Method

Flow of dosing for Lime soda is as follows:



Lime soda (JWWA-K-108 or equivalent) is dissolved in a solution tank to make a 20 % (W/V % strength) solution. The volume required for dosing is less than the flow volume in the dosing pipes necessary to prevent lime soda sedimentation. Therefore, cycling is used between the lime soda equipment and the receiving well, and the calculated volume of flow obtained by manual operation of a valve installed just above the entrance to the gauging well. The measured volume is injected into the receiving well from the 30° notch of the gauging well.

4-2-3-2 <u>Chlorine Dosing Facilities</u>

Facilities for water treatment for Stage I shall have a capacity of 16000 m^3/day, and additional units of the same capacity shall be built for Stage 2, and for Stage 3, so that the total completed capacity shall be 48000 m^3/day.

a) <u>Chlorine Dosing Equipment</u>

Volume of water treated: 16000 m^3/day

Setting the maximum dosing ratio at 5 ppm, the maximum dose volume will be:

$$16000 \ m^3/day \times 5 \ ppm = 80 \ kg/day$$

Calculating the maximum chlorine storage as ten days' supply, the total storage volume shall be:

$$80 \ kg/day \times 10 \ days = 800 \ kg$$

Selecting the 100 kg cylinder as best for storage, inspection and transporting, the number of cylinders to be stored are:

$$800 \ kg \div 100 \ kg/cylinder = 8 \ cylinders$$

Dosing shall be done at one location with a single dosing machine, with a calculated capacity as follows:

800 kg/day ÷ 24 hrs. = 3.33 kg/hr.

Thus, the dosing equipment shall be:

2 units (1 spare) ---- Capacity, 4 kg/hr.

The chlorine shall be introduced by a chlorine ejector.

Assuming the liquid chlorine to have a strength of 3000 ppm, the volume of water required will be:

$$3.31 \text{ kg/hr.} \times 1/3000 \text{ ppm} = 1.11 \text{ m}^3\text{/hr.}$$
$$= 0.0185 \text{ m}^3\text{/min.}$$

Thus, requirement for pressure pumps will be:

20 /Min. x 40m x 0.75Kw, using 2 pumps (1 Stand-by)

Type: Vacuum-type mechanical diaphragm pump

Installation: Attached to floor

Capacity: Maximum of 10 kg per hour

Meter limit: 10 : 1

Meter method: Flow meter calibrated

Operation method: Manual operation (raw water flow volume comparison possible by installing adapter)

Injector supply water: 4.0 kg/cm^2

Operational breakdown indicator included.

Pressure gauges include chlorine pressure gauge and water pressure gauge.

Equipment for safety, prevention of flow reversal, excess pressures, excess vacuum, or double flow reversals.

Gist of Shape: 400 W x 300 L x 1450 H

	1st Stage	2nd Stage	3rd Stage
Pressure Pump	2 sets (1 Stand-by)	3 sets (1 Stand-by)	4 sets (1 Stand-by)
Chlorinator	2 sets (1 Stand-by)	3 sets (1 Stand-by)	4 sets (1 Stand-by)

b) <u>Method of Chlorine Dosing</u>

Chlorine gas from its cylinder shall be accurately measured in the injector under controlled vacuum pressure, then dissolved in small amounts of water, and injected at the inlet of the clean water reservoir. So that prechlorination is possible in the future, a branch pipe shall be installed, and closed off by a valve.

c) Method of Operation

1) The injector is operated by opening a valve introducing water pressure.

2) Chlorine gas is supplied to the chlorinator.

3) The amount of chlorine injected is calculated from the dosing ratio and the volume of water. Then, the volume of the dosing is adjusted to coincide with dosing volume graduation readings by operation of a flow control valve.

4-2-3-3 Equipment for Chlorine Neutralization

a) Calculation of Capacity for Neutralization

Capacity shall suffice for neutralizing vaporized chlorine from one 100 kg cylinder.

By the following formulas, the amount of heat transmitted from air in the room to the liquid chlorine shall be determined.

$Q = K, A, \Delta t$

K : heat transmission rate from air to liquid chlorine and heated chlorine 20 $Kcal/m^2 \cdot h^\circ C$

A : cylinder surface area 2.75 m^2

t : difference between room temperature and temperature of liquid chlorine $60^\circ C$

$= 20 \times 2.75 \times 60 = 3300$ Kcal/H

The evaporation heat of chlorine is 59 Kcal/kg so if all the above heat is used for evaporation, the heat from evaporation would be as follows:

$W = 3300/59 \doteqdot 56$ kg/hr.

Thus, allowing for safety, the neutralizer capacity shall be 100 kg/hr.

b) Neutralizing Equipment

Calculation of blower capacity

$$V = W \times \frac{\text{volume of 1 mol. of chlorine gas}}{\text{weight of 1 mol. of chlorine gas}} \times \frac{273 + \Delta t}{273}$$

W : volume of heat from liquid evaporation 56 kg/hr.

Δt : difference between air temperature and temperature of liquid chlorine $60^\circ C$

$V = 56$ kg/hr. $\times$ 22.4 l/71 g $\times$ 333/273

$= 56$ kg/hr. $\times$ 0.0224 m^3/0.071 kg $\times$ 333/273 $=$ 21.6 m^3/hr.

Provided the blower capacity is 21.6 m^3/hr. or more, pressure inside the room will not exceed exterior pressure (expelling outside the room will not occur). However, actually, the concentration of chlorine gas which can be treated by the neutralizer is 10% or less.

Blower capacity:

21.6 m^3/hr. ÷ 0.1 = 216 m^3/hr. = 36 m^3/min.

So, taking into consideration the blower efficiency

250$^{\square}$ x 240 ∅ x 20 m^3/min. x 40 mmAq x 0.75 kw x 1 blower

c) Method of Chlorine Neutralization

Leakage from the chlorine cylinder room and the injection chamber shall be detected by a leakage detector, activating a warning lamp in the control room.

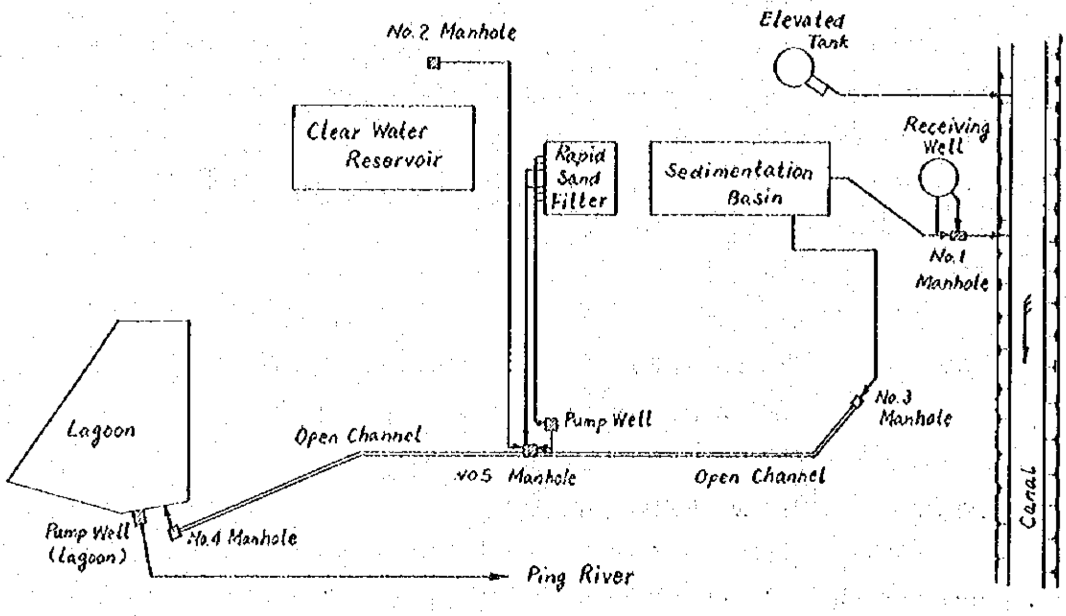
After verifying the warning lamp, neutralization shall be effected by use of the neutralizer, by the activated soda circulating pump, and by blowers operated from the control panel of the chemical dosing room.

4-2-4 Drainage System in the Plant Area

4-2-4-1 Summary of Waste Water Drainage Plan

Raw water brought from the intake plant on the Ping River will be purified in the treatment facilities of the water treatment plant and sent to the supply districts. Waste water from the various stages of the purification process is of many types, ranging in quality from close to that of raw water, to waste water containing various chemicals, to water close to treated clean water, and ranging in quantity from small volumes at infrequent intervals to volumes of several hundred cubic meters per day. In order to control the quality and volume of this waste water, not only to ensure smooth disposal but to prevent environmental pollution arising from wastes, a lagoon (excavated 2m. in depth, and 4000 m^2 in area) has been planned within the treatment plant site.

Waste waters small in volume and presenting no problems as to quantity are to be discharged into a water canal flowing along the east side of the treatment plant. Other waste waters are disposed of by pump into the Ping River by way of the lagoon.



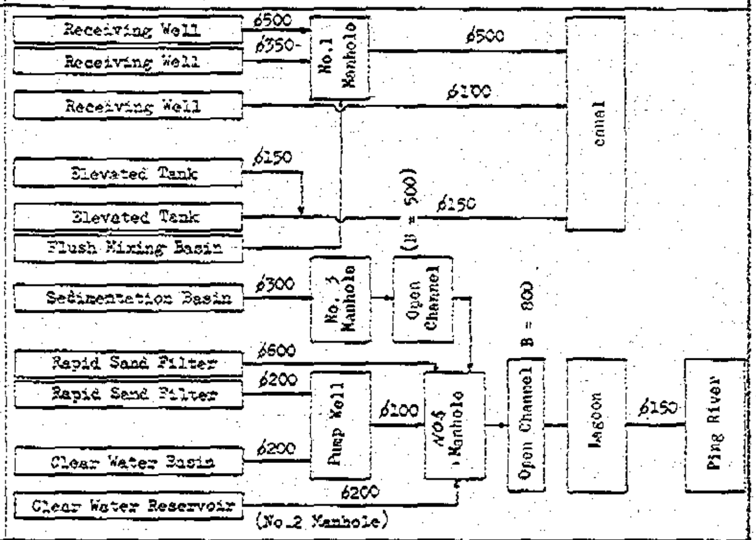
(Summary Drawing of Plan)

Diagram of Waste Water Drainage Plan

4-2-4-2 Drainage System

Facility		Name of Pipe	Standard Design Elevation of Drainage Pipe: Fixed Location	Elevation (m)	Pipe Diameter (mm)
Receiving Well	i)	Overflow Pipe	W. L.	+ 311.50	500
	ii)	Drain Pipe	Bottom elevation	+ 306.00	350
	iii)	"	Elevation of bottom of Receiving Well drain pipe	+ 310.00	100
Elevated Tank	iv)	Overflow Pipe	W. L.	+ 318.00	150
	v)	Drain Pipe	Bottom elevation	+ 314.00	150
Flush Mixing Basin	vi)	"	"	– 307.70	150
Sedimentation Basin	vii)	Sludge drain Pipe	(Pump drain water)		300
Rapid Sand Filter	viii)	Wash Water Drain Pipe	Height below bottom section of under drain	+ 307.40	600
	ix)	Drain Pipe (Waste Water Overflow)	Bottom elevation	+ 304.00	200
Clear Water Basin	x)	Drain Pipe		– 304.00	200
Clear Water Reservoir	xi)	" "	(Pump drain water)		200

Drainage System



4-2-4-3 Determining the Waste Water Pipes

a) Receiving Well Overflow Pipe

Based on graphs showing the relation between overflow volume(Q) and overflow water depth(H) from the overflow pipe(with bell-mouth), values are as follows:

Overflow volume Q = 48,000 m^3/day = 0.555m^3/sec. (Estimated for 3rd Stage)

Overflow depth H = 0.20 m

Assuming the above, pipe diameter will be D = 500 mm

When pipe of this diameter is laid horizontally for 84 m to the No. 1. manhole, the difference in water levels (h=6.55m) between the receiving well (+311.56) and the elevation of the bottom of the pipe (+304.45) will ensure a fully adequate flow.

b) Receiving Well Drain Pipe

Assuming the entire capacity of the receiving well to the weir level is the volume of waste water, waste water volume:

Q = 0.785 x 6.5 m^2 x 5.32m = 176.5 m^3

Pipe diameter: D = 350 mm

In determining pipe diameters, when the flow velocity within the pipe is set at v = 1.0 m/sec, volume in the pipe will be Q = 0.1 m^3/sec. This is on condition that the drainage time be about 30 minutes.

Because the drain pipe from the receiving well to No.1 manhole is laid level (horizontal distance is 11.65 m), when the pipes are full, the bottom elevation (+306.00m) and elevation of the top of the pipe (+305.40m) afford a difference in level of 0.60m.

With this, the water must flow. To investigate this matter; Setting the flow volume in the pipe at Q = 0.1 m^3/sec. we obtain an approximate calculation of head loss (allowing for head loss at pipe bends, etc.), making C = 100, we calculate by the Hazen-Williams formula:

n = i, l = 4.9 x 001165 = 0.057m 0.600m

Thus water will flow adequately even with level drain pipes. The reason level pipes are used is that it was unavoidable due to the comparatively high water level of the canal (+305.00m) at the drainage site. If the water level of the canal were lower it would have been desirable to have a slope to prevent earth or sand accumulating inside the pipe.

Furthermore, the reason for increasing pipe diameter from 350 mm to 400 mm is that 350 mm ACP pipe is not available.

c) Receiving Well Drain Pipe for Outflow Channel

This drain is designed to handle that volume of waste water which will not flow through the overflow pipe.

Volume of Drain Water: $Q = 3.14 \times 8.1 m \times 10m \times 0.3m = 7.6m^3$

Pipe Diameter: D = 100 mm

In determining pipe diameter, flow velocity is temporarily set at v = 0.8m/sec, in which case flow volume in the pipe is $Q = 0.006 m^3$, and the drainage time becomes about 20 minutes.

This pipe diameter is sufficient for adequate flow even if the pipe is laid level to the canal (horizontal distance ℓ= 14.50m), due to the difference in water levels between the bottom of the receiving well drain (+310.00 m and the bottom of the pipe, (+ 305.35m)

d) Elevated Tank Overflow Pipe

Based on graphs showing the relation between overflow volume (Q) and overflow water depth(H) from the bugle-mouth overflow pipe, values are:

Overflow volume Q = 5.760 m^3/day (Estimated for 3rd Stage.)

Overflow water depth H = 0.15 m

Assuming the above, we find pipe diameter to be

D = 150 mm

With this diameter, flow in the pipe will be adequate due to the difference in water levels of the elevated tank (HWL + 318.00m) and the level of the bottom of the pipe (+305.025 m), even if the pipe is laid level to the canal. (Level distance to canal, ℓ = 5.10m. Actually, slope is 152.1%)

e) Elevated Tank Drain Pipe

The volume of drain water is defined to be that water below the low water line of the elevated tank, for determination of this drain pipe diameter.

Volume of drain water: $Q = 32m^2 \times 1.0 m = 32 m^3$

Pipe Diameter: D = 150 mm

In determining pipe diameter, flow velocity in the pipe is set at a trial figure of V = 0.85m/sec. Under this condition, flow volume Q = 0.015m^3/sec, and drainage time will be about 35 minutes. Water flow will be adequate, because of the difference in water level between the tank bottom (+314.00m) and the top of the pipe (+305.175m), even though the pipe be laid level to the canal, a distance of ℓ = 51.0m (actually the gradient is 2.1%).

f) Flush Mixing Basin Drain Pipe

The entire volume of the flush mixing basin is assumed to be the volume of the drain water.

Drain water volume: Q = 1.9m x 1.9m x 3.0m = 10.8m^3

Pipe diameter: D = 150mm

An assumed value for flow velocity in the pipe is set at V = 0.85m/sec in calculating the pipe diameter. In that case, flow volume will be Q = 0.0015m^3/sec, and drainage time will be about 12 minutes.

Not only is there adequate difference in water level between the flush mixing basin and No. 1 manhole, but water will flow by gravity, the slope being 11.5%.

By the Kutter formula, if n = 0.011,

when the slope is 11.5%

V_o = 1.025m/sec

Q_o = 0.0182m^3/sec > Q (= 0.015m^3/sec)

g) Sedimentation Basin Drain Pipe

Sludge disposal from sedimentation is effected by drainage pumps, and sent to No. 3 manhole by pressure pipes. Calculation of volume of sludge is done with design work for the sedimentation basin, and only determinations for the pipe are shown here.

Sludge Volume: Q = 200m^3/hr = 0.0556m^3/sec. Diameter of pipe for the pump delivery side is D = 200mm. Flow velocity in the pipe thus would be more than 2m/sec, with excessive head loss. Therefore, pipe diameter for the sedimentation basin drain pipe shall be D = 300mm.

h) Rapid Sand Filter Wash Water Drainage Pipe

The filter washing cycle is 40 to 60 minutes, and two filters are not washed at the same time. Drainage pipe diameter is determined as that required to drain that volume of waste water produced by washing one filter at any one time.

Waste water volume $Q = 104m^3/5min/basin = 20.8m^3/min/basin$
$= 0.347m^3/sec/basin$

Using gravity flow for draining, and applying the Kutter formula: $n = 0.011$ $D = 600mm$ $I = 2.20\%$

$V = 1.245m/sec$, $Q_0 = 0.3520m/sec > Q$

Therefore, pipe diameter is designed at D = 600mm and the pipe to be laid on a 2.20% gradient.

i) Rapid Sand Filter Drain Pipe (Waste and Overflow)

There is an overflow pipe for the receiving well so the overflow volume may be excluded from calculations, and the pipe diameter determined to carry the volume of waste water alone.

Disposal or waste water (per basin) volume = Q and is calculated as follows:

From upper edge of trough to top of filter	$5.0m \times 6.4m \times 0.7m = 22.4m^3$
Filter sand (porosity 40%)	$5.0 \times 6.4 \times 0.6 \times 0.4 = 7.68m^3$
Filter gravel (porosity 35%)	$5.0 \times 6.4 \times 0.5 \times 0.35 = 5.60m^3$
Leopold block (porosity 30%)	$5.0 \times 6.4 \times 0.3 \times 0.3 = 2.88m^3$
	$Q_0 = 38.56m^3$

Thus, waste water volume per basin is set at $Q = Q_0 + X = 37m^3$

If washing time is forty minutes per unit, then capacity must be to drain Q ($37m^3$) of water in forty minutes. (But, the volume retainable in the drainage channel shall be for one side a maximum value of

$$Q_1 = \underset{B}{1.0} \times \underset{H}{2.0} \times \underset{L}{19.8} = 39.6m^3$$

Setting the outflow volume as $Q' = Q/40 = 0.925m^3/min.$, and using gravity flow, the Kutter formula gives the below values:

$n = 0.011$ $D = 200mm$ $I = 2.2\%$, thus $V = 0.554m/sec$, and

$Q = 0.0174m^3/sec = 1.044m^3/min$ Q'

Therefore, pipe diameter is determined as 200mm, and the gradient on which the pipe shall be laid as I = 2.2%.

j) Clear Water Basin Drain Pipe

The clear water basin connects to the clear water reservoir. The volume of water drained is estimated at $Q = 1.0m^3/sec$ and pipe diameter set at 200mm. This pipe shall be linked with the rapid sand filter drain pipe, and drainage conducted so as to avoid overlapping.

k) <u>Clear Water Reservoir Drainage System</u>

Overflow pipe is omitted, for reasons given below.

1) Water levels rise in the Ping River at flood times (H.W.L. = +306.00m) and would cause a backflow if an overflow pipe were installed, posing danger of polluting the clear water.

2) Overflow pipes are provided both in the receiving well and the rapid sand filter, and a safety factor of 70cm is allowed in designing the height of the clear water reservoir.

The method of disposal is to send waste water to a manhole using a portable-type submerged pump, then to use natural gravity flow. The volume of waste water is considered to be that capacity below the low water level in the reservoir.

Volume below the L.W.L.

Clear Water Reservoir	0.3m x 48.0m x 24.0m	= $345.6m^3$
Distributing Pump Well	1.4m x 8.0m x 24.0m	= $268.8m^3$
		$\Sigma V 614.4m^3$

If a pump is used capable of pumping a volume of $Q = 1.4m^3/min$, drainage time will be about seven hours.

Pipe diameter is designed to obtain a flow of $1.4m^3/min$ (= $0.023m^3/sec$). According to Kutter's formula:

n = 0.011   D = 200mm   I = 4.0%   V = $0.752^m/sec$

$Q_0 = 0.0236m^3/sec$ Q

Thus, pipe diameter shall be 200mm, laid on a gradient of 4.0%.

l) <u>Pump Well (Rapid Sand Filter Drain Pipe) and Pump Specifications</u>

As stated in paragraph (i), provided the washing time per unit is determined as forty minutes, the volume of drain water will be $37m^3$. The amount of $_B$water$_L$accumu$_H$latable in the drainage channel per basin is $13m^3$ (1.0 x 6.55 x 2.0) so that the volume of drainage water which must be disposed of during the washing process is $Q' = 37 - 13 = 24m^3$. Calculating the required pump capacity $Q = 24/40 = 0.6m^3/min$, and allowing a safety factor the value will be $Q = 0.7m^3/min$. Water accumulated in the drainage channel shall be drained in periods between washing.

The pump lift head is calculated as the actual lift from the minimum operable water line of the pump well to the water line of the connecting manhole, to which is added head loss around the pump. This lift head value is determined to be $H = 6m$. Thus specifications for the pump are as follows:

$0.7m^3/min \times 80mm$ Dia. $\times 6m \times 2.2kw \times 1$ pump

The size of the pump well was determined by first fixing the base height, which is set by the gradient of the pipe leading from the rapid sand filter. If volume is calculated with the high water level of +306.00m.:

$V = 2.0m \times 2.0m \times 3.4m$ height $= 13.6m^3$

Filling time is:

$$T = \frac{13.6}{0.7} = 20min.$$

m) Open Channel (concrete)

The width of the open channel is determined so that when clogged by sludge, it can be cleaned out by shovel.

1) Open Channel Breadth

$B = 500mm$ (Between the No. 3 manhole and connecting manhole.)

Flow volume: Hydraulic mean depth shall be sufficient that

$Q = 0.058m^3/sec$ (Drain pump capacity for rapid sand filter)

Water depth; $H = 0.25m$ Gradient, $I = 0.8\%$

Roughness coefficient, $n = 0.013$

Assuming the above, the Manning formula may be used as follows:

Hydraulic Mean Depth: $R = \frac{BH}{B+2H} = \frac{0.125}{1.0} = 0.125m$

Flow velocity: $V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2} =$

$$\frac{1}{0.013} \times 0.125^{2/3} \times 0.0008^{1/2} = 0.54m/sec.$$

Flow Volume: $Q' = AV = 0.125 \times 0.54 = 0.0675m^2/sec > Q$

Therefore, the channel is to be built as detailed in the design drawings, with a gradient, $I = 0.8\%$.

2) Open Channel (concrete)

$B = 800mm$ (between the connecting manhole and No. 4 manhole)

Flow Volume: $Q = 0.058m^3/sec$ (Sludge pump capacity for sedimentation tank) $+0.347m^3/sec$ (Washing waste water volume for rapid sand filter) $+ X = 0.43m^3/sec.$

Regarding X ,

Disposal of waste water from the clear water basin is infrequent and may be done at other times than that for disposal of washing waste water. However, drainage from the distributing reservoir shall be considered together with washing waste water.

The Manning formula is used setting values as follows:

Water depth $H = 0.50m$ Gradient, $I = 1.6‰$

Roughness coefficient $n = 0.013$.

Then,

Hydraulic Mean Depth: $R = \frac{BH}{B+2H} = \frac{0.40}{1.8} = 0.22m$

Flow velocity: $V = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot I^{1/2} =$

$$\frac{1}{0.013} \times 0.22^{\frac{2}{3}} \cdot 0.0016^{\frac{1}{2}} = 1.12m/sec.$$

Flow volume: $Q' = A.V = 0.40 \times 1.12 = 0.448m^3/sec > Q$

Thus, the channel is to be built as shown in the design drawings, with a gradient of 1.6‰.

n) Lagoon

This facility is provided to gather waste water from the various processes, and enable control of the quality and volume. The lagoon is designed to be made by simple excavation, with a bottom area of $3,265m^2$, the sides sloped to a gradient of 2:1 and with a depth of 2m.

Inflow volume: $Q_o = 400m^3/day$ (sludge volume from sedimentation tanks for 1st Stage) $+ 624m^3/day$ (washing waste water volume from filters for 1st Stage, washing one filter per day) $+ \alpha$ $= 1,050m^3/day$ or 12.15 ℓ/sec.

Designed water level: +304.00m

Estimated underground

water level in rainy season: +304.00

Capacity of lagoon (if water depth = 1.5m):

$$V = \frac{h}{3}\left(A + \sqrt{A.B} + B\right) =$$

$$\frac{1.5}{3} x \left(3265 + \sqrt{3265 \times 4033} + 4033\right) = 5,463m^3$$

o) Determination of the Lagoon Pump and Pump Well

Pump capacity shall include a 20% safety factor for rain water falling into the open channel to increase the inflow volume, and other factors.

$$Q = 1,050m^3/day \times 1.2 = 1,250m^3/day = 0.87m^3/min$$

The lift load of the pump is estimated at 10m, which includes the lift from the lowest operating water level to the water level of the Ping River, plus head loss due to friction in the pipes, and head loss of the pump.

$0.87m^3/min$ x 100mm dia. x 10m x 3.7kw x 1 set

The pump well has a stop log, so constructed that the upper-level clear water only may be drained off. Capacity is as follows:

$$V = 2.05m \times 2.00m \times \underset{(H)}{1.4m} = 5.74m^3$$

Filling time will be:

$$T = \frac{5.74}{0.87} = 6.6 \text{ min.}$$

p) Drainage Pipe for Clear Surface Water of Lagoon

As stated in paragraph (o), the design was made with waste water volume set at $Q = 1,250m^3/day$, to be discharged into the Ping River by pressure pipes. Even though the waste be lagoon surface water, it has been assumed that the waters will be muddy, and pipes were determined so that flow velocity would not be too slow ($D = 150mm$, $V = 0.82m/sec$).

4-2-5 Calculations of Diameters of Distributing Mains

4-2-5-1 Policy

Distribution System	AD 1900	AD 2000
Existing Old (No.1 W.T.P. System)	7000 x 1.5 = 10500m^3/d (121.5 ℓ/sec)	10500m^3/d (121.5 ℓ/s)
Existing New (No.2 Water Treatment Plant System)	4000 x 1.5 = 6000m^3/d (69.5 ℓ/sec) 2000 x 1.0 = 2000m^3/d (23.4 ℓ/sec)	1900 x 1.5 = 2850m^3/d (32.5 ℓ/s) 4100 x 1.0 = 4100m^3/d (47.8 ℓ/sec) } for Univ.
Proposed (Paton Water Treatment Plant System)	16000 x 1.5 = 24000m^3/d (277.1 ℓ/sec)	1900 x 1.0 = 1900 (21.7 ℓ/sec) } for Univ. 46100 x 1.5 = 69150m^3/d (800.3 ℓ/s)
Total	42500m^3/d	88500m^3/d

AD 1980

The supply district shall be divided into two systems, the Existing Old System and a second system combining the Existing New System with the Proposed New System.

The Existing Old System lies on the right bank of the Ping River, its boundaries being the purification plant to the north, the vicinity of the Mengray bridge in the south, the Ping River on the east, and a line between the Southern Gate and the North Gate of the city walls on the west. This area is comparatively low, and is served by natural gravity flow.

At present the trunk main along the Ping River is not adequate in capacity so that it will be necessary to enlarge the trunk main's capacity as far as the vicinity of the Suriwong Theatre.

The Existing New System, besides supplying water to Chiang Mai University, will have as its supply area a belt extending from the vicinity of the Existing New Plant in the western part of Chiang Mai City to the vicinity of the city walls on the west side. However, there is a difference in water level with the proposed new system so that the two systems cannot be operated in unison. Consequently, the distribution mains to the southern sector consist of double distribution pipes. The supply district for the proposed new system is the area roughly circumscribed by the Super Highway along the left bank of the Ping River, and the area south of the Super Highway on the right bank, running in a north-and-south direction through the center of the walled city, with the Existing Old System and Existing New System on either side.

AD 2000

All increase in water supply capacity between 1980 and 2000 AD will depend on the Proposed Water Treatment Plant. Therefore the distribution areas for the Existing Old System and the Existing New System will be smaller in 2000 AD than in 1980.

The Existing Old System will yield to the Proposed New System the eastern half of that part of its distribution area within the city walls, and the southern section near the Mengray bridge. The 2000 AD east-west limits will be the eastern wall of the city, and the Ping River, while the north-south limits will be the purification plant and the vicinity of (60).

The trunk mains of the Existing New System shall run north along the Irrigation Canal, then turn west toward the University water treatment plant at the south part of Chiang Mai University. In addition to supplying water to the University, this system shall also supply water to the areas enroute, so that it will not be adequate for the entire requirements of the University. The deficient quantity will be supplied from the system of the Proposed Water Treatment Plant.

The Proposed System shall cover most of the area of the right bank of the Ping River and all the left bank area. To fully utilize the Existing New System's 300 mm trunk main, a sub-main shall be installed, extending to the Rincome Hotel from the fork at the Super Highway mentioned above.

Existing Old W.T.P (No.1 W.T.P. System) AD 1980 (C= 130)

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydralic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
16' (Old plant)					325.80		
16-16	121.5	390	120	2.6	325.49	305.43	20.06
16-17	121.5	390	630	2.6	324.11	305.32	18.79
17-18	114.7	390	430	2.3	323.12	305.21	17.91
18-15	87.4	390	190	1.4	322.85	306.70	16.15
18-19	26.2	200	270	3.9	322.06	306.16	15.90
15-39	86.9	336	260	2.9	322.11	305.60	16.51
39-42	78.4	336	350	2.4	321.28	304.23	17.05
39-38	6.9	200	210	0.3	322.04	305.65	16.39
42-65	49.8	260	430	3.6	319.76	303.70	16.06
42-41	14.8	300	770	0.2	321.14	306.41	14.73
42-43	10.1	200	160	0.7	321.23	306.50	14.68
65-73	47.2	260	330	3.2	318.69	303.86	14.83
65-66	1.7	150	740	0.1	319.68	305.59	14.09
64-65	2.7	100	280	1.7	319.76	303.70	16.06
73-60	39.3	250	650	2.8	316.90	303.61	13.29
73-72	2.7	100	700	1.7	317.47	303.75	13.72
73-63	0.8	100	390	0.2	318.63	304.90	13.73
60-58	20.7	250	1230	0.8	315.85	302.65	13.20
19-38	2.4	200	510	0	322.04	305.65	16.39
19-20	14.5	200	290	1.3	321.69	306.16	15.53
38-37	9.3	200	550	0.6	321.73	305.66	16.07
20-22	8.1	200	790	0.4	321.34	310.57	10.77
37-36	5.2	150	620	0.8	321.24	308.69	12.55
41-66	0.8	50	300	4.9	319.68	305.59	14.09
41-40	9.4	300	580	0.1	321.09	307.79	13.30
66-72	4.8	100	460	4.8	317.47	303.75	13.72
67-66	6.7	150	520	1.2	320.33	307.01	13.32
63-62	2.9	100	680	1.9	318.63	304.90	13.73

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradiant (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
40-67	10.2	150	280	2.7	321.09	307.79	13.30
36-40	4.3	150	280	0.5	321.24	308.69	12.55
22-36	2.1	150	610	0.1	321.34	310.57	10.77
43-64	9.3	150	410	2.3	320.22	300.31	19.91
64-63	5.4	100	260	6.1	318.63	304.90	13.73

Existing New W.T.P. AD 1980 (C = 130)
(No.2 W.T.P. System)

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
90' (New plant)					353.60		
90'-90	93.2	300	150	5.6	352.76	326.23	26.54
90-89	87.5	300	550	5.0	350.01	321.92	28.09
89-30	86.5	300	850	4.9	345.84	321.96	23.88
30-31	79.3	300	300	4.2	344.58	318.60	25.98
31-33	39.3	300	700	1.1	343.81	312.61	31.20
31-32	40.0	300	650	1.2	343.80	318.35	25.45
33-34	37.6	300	770	1.1	342.96	310.86	32.10
34-24	1.1	300	780	0	342.96	314.83	28.13
32-26	39.0	300	690	1.1	343.04 (351.25)	318.09	24.95
25-24	2.6	232	580	0.2	342.96	314.83	28.13
26-25	5.0	232	630	0.1	342.98	318.09	24.89
26-27	30.9	250	960	1.8	349.52	322.82	26.70
27-28	25.6	200	820	3.7	346.49	331.49	15.00
28-29 (University)	23.1	200	680	3.1	344.38	338.62	5.76
34-88	30.8	200	820	5.2	338.70	310.52	28.18
88-69	24.4	200	420	3.4	337.27	307.14	30.13
69-70	24.4	200	700	3.4	334.89	303.67	31.22

Proposed W.T.P. AD 1980 (C = 130)
(Paton W.T.P.System)

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
(PING RIVER RIGHT REGION SYSTEM)							
4(proposed plant)							
4-2	277.1	600	200	1.5	342.40	304.78	37.32
2-1	92.3	350	1,870	2.6	337.24	308.64	28.60
1-21	72.6	515	730	0.3	337.05	310.49	26.56
21-22	67.1	515	1,070	0.2	336.81	310.57	26.24
22-23	59.9	365	500	1.0	336.33	312.21	24.12
23-35	54.3	350	820	1.0	335.53	309.16	26.37
35-40	1.5	300	570	0.0	335.53	307.79	27.74
35-68	46.8	212	310	8.5	332.89	308.30	24.59
68-67	1.6	150	650	0.1	332.82	307.01	25.81
68-69	41.5	232	510	4.4	330.65	307.14	23.51
69-71	11.7	150	700	3.5	328.20	306.40	21.80
69-70	28.2	200	700	4.5	327.50	303.67	23.83
71-70	1.5	100	1,080	0.6	327.55	303.67	23.88
(PING RIVER LEFT REGION SYSTEM)							
2-3	184.8	450	130	2.8	341.74	305.70	36.04
3-5	178.6	450	200	2.6	341.22	305.50	35.72
5-6	162.8	400	880	3.9	337.79	304.50	33.29
6-8	160.6	400	850	3.8	334.56	305.25	29.31
8-9	131.0	350	610	5.1	331.45	304.49	26.96
9-52	59.5	300	880	2.5	329.25	303.72	25.53
9-10	59.7	300	670	2.5	329.77	304.36	25.41
52-53	50.6	300	990	1.8	327.47	303.12	24.35
53-51	39.7	250	150	2.8	327.05	303.44	23.61
50-51	2.6	150	390	0.2	327.02	303.38	23.64
51-54	38.1	250	850	2.6	324.81	303.04	21.77
49-50	6.7	150	430	1.3	327.10	303.73	23.37

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
54−55	34.7	250	500	2.2	323.71	302.40	21.31
55−56	30.6	250	1,100	1.7	321.84	303.42	18.42
56−57	16.9	200	1,100	1.7	319.97	302.94	17.03
13−12	3.2	100	390	2.3	326.12	304.90	21.22
10−47	21.0	200	390	2.6	328.76	304.02	24.74
10−11	29.5	300	460	0.7	329.45	304.07	25.38
47−48	17.5	200	390	1.8	328.06	303.88	24.18
48−49	12.9	200	360	1.1	327.66	303.73	23.93
49−46	2.2	150	380	0.2	327.58	306.61	20.97
11−14	27.2	300	360	0.6	329.23	304.97	24.26
14−44	20.6	200	380	2.5	328.28	305.79	22.49
14−13	4.6	100	490	4.5	327.02	304.90	22.12
44−46	16.4	200	480	1.6	327.51	306.61	20.90

Existing Old W.T.P. (No.1 W.T.P. System) AD 2000 (C = 130)

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
16' (Old plant)					325.80		
16'-16	121.5	390	120	2.6	325.49	305.43	20.06
16-17	121.5	390	530	2.6	324.14	305.32	18.84
17-18	108.9	390	430	2.1	323.24	305.21	18.03
18-15	83.7	390	190	1.3	323.00	306.70	16.30
18-19	23.5	200	270	3.2	322.39	304.90	17.49
15-39	82.9	336	260	2.6	322.32	305.60	16.72
39-42	81.7	336	350	2.3	321.51	304.23	17.28
39-38	1.8	200	210	0	324.59	305.65	18.94
42-65	60.3	260	430	5.0	319.36	303.70	15.66
42-43	13.5	200	160	1.1	321.33	306.50	14.83
42-41	1.1	300	770	0	321.51	306.41	15.10
65-73	55.2	260	330	4.2	317.97	303.86	14.11
65-64	2.8	100	280	1.8	319.85	304.04	15.81
65-66	1.3	150	740	0.1	319.32	305.59	13.73
73-60	35.9	250	650	2.3	316.47	303.61	12.86
73-63	2.7	100	390	1.6	317.34	303.49	13.85
60-61	4.1	100	230	3.7	315.62	303.41	12.21
61-62	1.7	100	300	0.7	315.40	303.00	12.40
43-64	11.8	150	410	3.6	319.85	304.04	15.81
64-63	6.9	100	260	9.7	317.34	303.49	13.85
63-62	3.6	100	680	2.8	315.40	303.00	12.40
19-20	2.1	200	290	0	322.38	306.16	16.22
19-38	3.9	200	510	0.1	322.33	305.65	16.68
38-37	2.1	200	550	0	322.31	305.66	16.65

Existing New W.T.P. AD 2000 (C= 130)

(No. 2 W.T.P. System)

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Level (m)
90' (New plant)					368.50		
90'-90	80.3	300	150	4.3	367.85	326.23	41.62
90-91	13.0	200	4,000	1.1	363.46	340.00	23.45
90-89	65.3	311	550	2.9	366.25	321.92	44.33
89-30	62.8	300	850	2.7	363.95	321.96	41.99
30-92	60.3	250	920	6.2	358.25	333.75	24.50
92-94	47.8	250	1,010	4.0	354.21	327.47	26.74
94-95 (University)	47.8	250	850	4.0	343.81	338.62	12.19

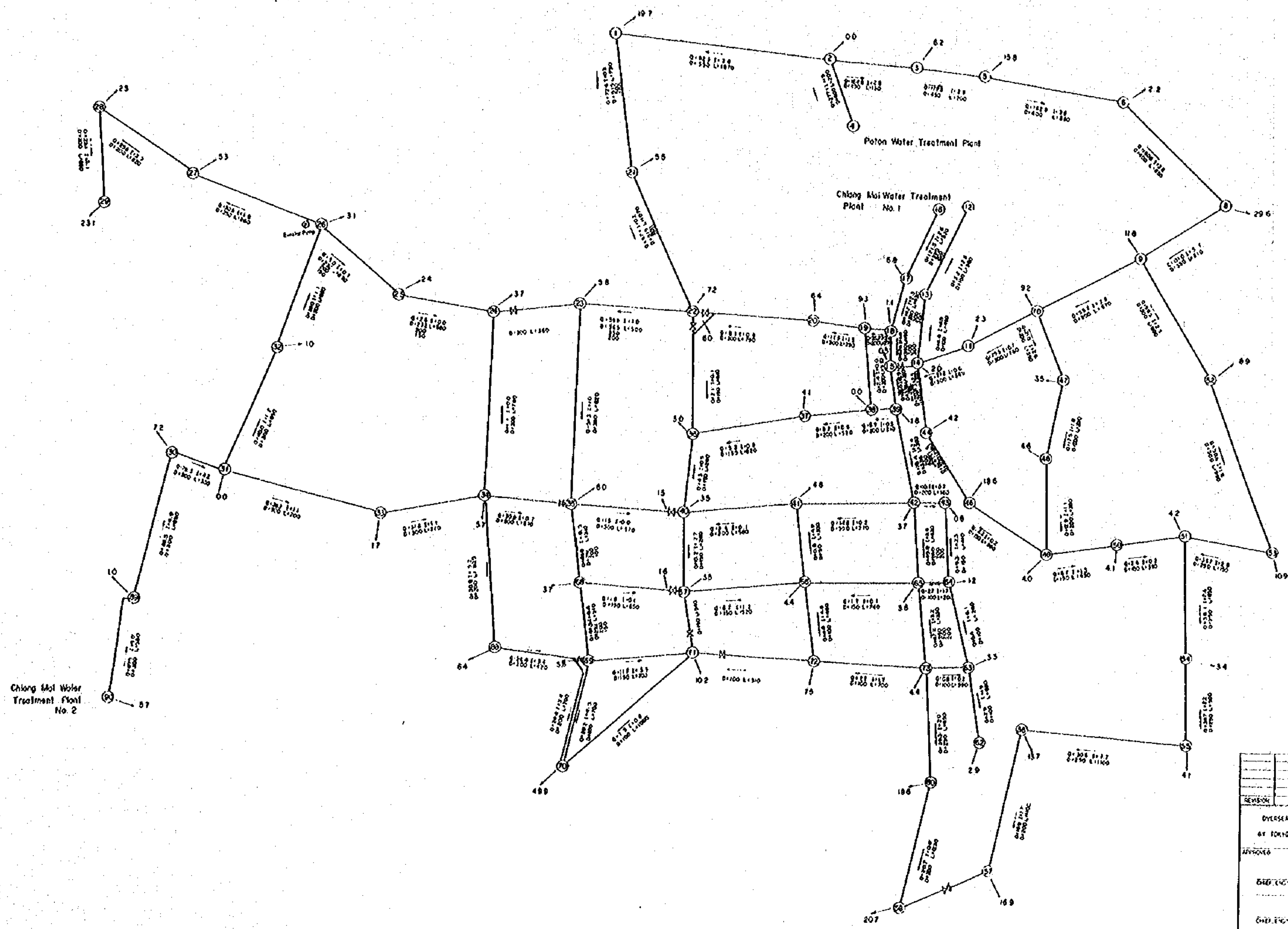
Proposed W.T.P. AD 2000 (C = 130)
(Paton W.T.P. System)

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
(PING RIVER LEFT REGION SYSTEM)							
4(proposed plant)					342.40		
4－2	816.9	780	200	3.0	341.80	304.78	37.02
2－3	379.1	450	130	10.5	340.43	305.70	34.73
3－5	363.5	450	200	9.7	338.49	305.50	32.99
3－76	10.4	200	5,300	0.7	336.72	311.00	25.72
5－12	185.8	450	1,150	2.8	335.26	305.43	29.83
5－6	144.1	400	880	3.1	335.74	304.50	31.24
5－77	28.1	300	8,600	0.6	333.22	314.00	19.22
12－13	99.4	353	390	2.9	334.13	304.90	29.23
12－84	80.6	300	320	4.3	333.88	305.41	28.47
13－14	96.7	353	490	2.7	332.79	304.97	27.82
14－44	73.8	296	380	3.9	331.31	305.79	25.52
14－11	19.1	300	360	0.3	332.68	304.07	28.61
44－46	65.9	296	440	3.2	329.91	306.61	23.30
44－45	4.0	100	200	3.5	330.61	304.33	26.28
46－85	66.2	250	640	7.3	325.26	304.21	21.05
49－46	6.5	232	380	0.1	329.91	306.61	23.30
85－56	37.3	250	730	2.5	323.43 (330.71)	303.42	20.01
56－57	36.5	200	1,100	7.2	322.79	302.94	19.85
55－56	24.6	250	1,100	1.2	323.43	303.42	20.01
57－86	11.7	160	800	3.5	320.00	305.00	15.00
6－8	133.2	400	850	2.7	333.46	305.25	28.21
6－7	5.4	100	1,200	6.1	328.44	298.00	27.44
8－79	42.7	250	1,230	3.2	329.49	304.60	24.89
8－9	72.1	350	610	1.7	332.45	304.49	27.96
8－78	6.2	100	1,000	7.9	325.59	298.00	27.59
79－81	29.2	250	1,200	1.6	327.58	302.80	24.78

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Grand Level (m)	Residual Head (m)
79-80	6.7	100	1,200	9.1	318.60	298.00	20.60
81-83	3.1	100	3,500	2.2	319.96	299.00	20.96
53-81	8.9	200	900	0.5	327.58	302.80	24.78
81-82	17.5	150	1,500	7.5	316.39	298.00	18.39
84-10	76.2	300	330	3.9	332.60	304.36	28.24
10-47	62.0	296	390	2.8	331.49	304.02	27.47
10-9	16.3	300	670	0.2	332.45	304.49	27.96
11-10	14.9	300	460	0.2	332.60	304.36	28.24
47-48	55.6	296	390	2.3	330.59	303.88	26.71
48-49	47.4	296	360	1.7	329.97	303.73	26.24
49-50	33.5	232	430	3.0	328.66	303.38	25.28
9-52	66.4	300	880	3.0	329.80	303.72	26.08
52-53	49.8	300	990	1.8	328.05	303.12	24.93
53-51	20.4	250	150	0.8	327.93	303.44	24.49
51-54	38.4	250	850	2.7	325.67	303.04	22.63
50-51	25.9	232	390	1.9	327.93	303.44	24.49
54-55	32.2	250	500	1.9	324.71	302.40	22.31
(PING RIVER RIGHT REGION SYSTEM)							
4-2	816.9	780	200	3.0	341.80	304.78	37.02
2-1	437.8	652	1,870	2.3	337.58 (346.47)	308.64	28.94
1-21	229.1	515	730	2.1	336.01	310.49	25.52
1-74	185.8	500	1,100	1.7	335.73	314.50	21.23
1-75	11.4	150	3,700	3.4	334.00	319.00	15.00
21-22	218.7	515	1,070	2.0	333.90	310.57	23.33
22-20	83.5	336	790	2.7	331.81	306.16	25.65
22-23	96.6	365	500	2.3	332.74	312.21	20.53
22-36	14.0	150	610	4.9	330.89	308.69	22.20
20-37	73.8	350	540	1.7	330.88	305.66	25.22

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
37--41	69.1	350	280	1.5	330.45	306.41	24.04
36--37	0.9	150	620	0.0	330.88	305.66	25.22
41--66	64.9	300	300	2.9	329.58	305.59	23.99
40--41	3.3	300	580	0.0	330.45	306.41	24.04
66--72	53.3	306	460	1.8	328.75	303.75	25.00
66--67	4.8	150	520	0.7	329.23	307.01	22.22
72--58	42.0	250	2,250	3.1	321.70 (328.30)	302.65	19.05
72--71	3.9	100	510	3.3	327.09	306.40	20.69
58--59	11.5	150	4,500	3.4	313.00	298.00	15.00
74--26	160.3	500	1,080	1.3	334.35 (353.01)	318.09	16.26
26--32	47.0	360	690	0.7	333.90	318.25	15.65
26--27	41.0	250	960	3.0	350.13	322.82	27.31
26--25	64.8	350	630	1.4	333.49	315.37	18.12
24--23	5.9	200	360	0.2	332.74	312.21	20.53
24--34	46.3	300	780	1.5	331.63	310.86	20.77
25--24	58.9	350	580	1.1	332.83	314.83	18.00
23--35	92.0	350	820	2.6	330.61	309.16	21.45
35--68	110.7	341	310	4.2	329.33	308.30	21.03
34--35	48.2	300	610	1.7	330.61	309.16	21.45
35--40	18.3	300	570	0.3	330.46	307.79	22.67
68--69	101.8	350	510	3.1	327.73	307.14	20.59
68--67	2.1	150	650	0.1	329.23	307.01	22.22
69--70	100.3	335	700	3.8	325.09 (345.40)	303.67	21.42
88--69	14.9	200	420	1.4	327.73	307.14	20.59
69--71	5.6	150	700	0.9	327.09	306.40	20.69
70--87	33.6	250	4,000	2.1	337.00	322.00	15.00
71--70	2.8	100	1,080	1.9	325.09	303.67	21.42
32--31	44.5	360	650	0.6	333.51	318.00	15.51
31--33	44.5	300	700	1.4	332.51	312.61	19.90

From-To	Quantity (ℓ/sec)	Diameter (mm)	Length (m)	Hydraulic Gradient (‰)	Water Level (m)	Ground Level (m)	Residual Head (m)
33—34	39.5	300	770	1.1	331.63	310.86	20.77
34—88	26.8	200	820	4.1	328.30	310.52	17.78
36—40	7.5	150	280	1.6	330.46	307.79	22.67
40—67	13.1	150	280	4.4	329.23	307.01	22.22
67—71	12.5	150	640	4.0	327.09	306.40	20.69
27—28	28.2	200	820	4.4	346.49	331.49	15.00
28—29	21.7	200	680	2.7	344.63	338.62	6.01



LEGEND

○	:	Amount of Water Consumption (l/sec)
Q	:	Quantity (l/sec)
I	:	Hydraulic Gradient (‰)
D	:	Diameter of Pipe (mm)
L	:	Length (m)

REVISION	DATE	DESCRIPTION	APPROVAL

PREPARED FOR
OVERSEAS TECHNICAL COOPERATION AGENCY
GOVERNMENT OF JAPAN
BY TOKYO ENGINEERING CONSULTANTS CO., LTD
TOKYO, JAPAN

PROVINCIAL WATER SUPPLY DIVISION
DEPARTMENT OF PUBLIC AND MUNICIPAL WORKS
MINISTRY OF INTERIOR
KINGDOM OF THAILAND

APPROVED

CHIEF, ENGINEERING SECTION .... DATE

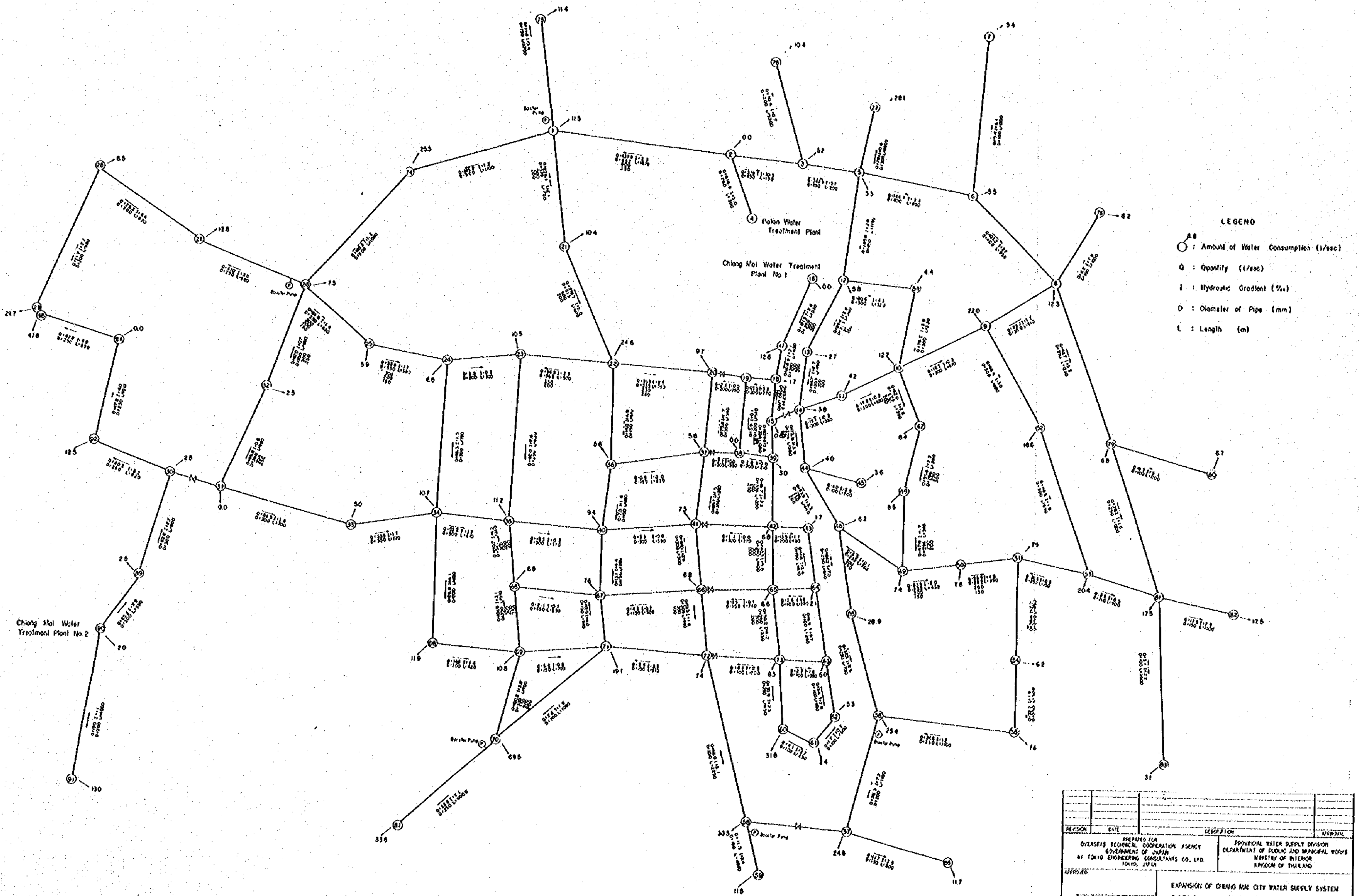
CHIEF, ENGINEERING DIVISION .... DATE

EXPANSION OF CHIANG MAI CITY WATER SUPPLY SYSTEM

DISTRIBUTION MAIN
DIAGRAM OF FLOW VOLUMES IN
DISTRIBUTION MAIN (A.D. 1980)

APPROVED .... DATE

SCALE 1/ | DRAWING NUMBER /



REVISION	DATE	DESCRIPTION	APPROVAL

PREPARED FOR
OVERSEAS TECHNICAL COOPERATION AGENCY
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PROVINCIAL WATER SUPPLY DIVISION
DEPARTMENT OF PUBLIC AND MUNICIPAL WORKS
MINISTRY OF INTERIOR
KINGDOM OF THAILAND

APPROVED

CHIEF, ENGINEERING SECTION .... DATE

CHIEF, ENGINEERING DIVISION .... DATE

EXPANSION OF CHIANG MAI CITY WATER SUPPLY SYSTEM

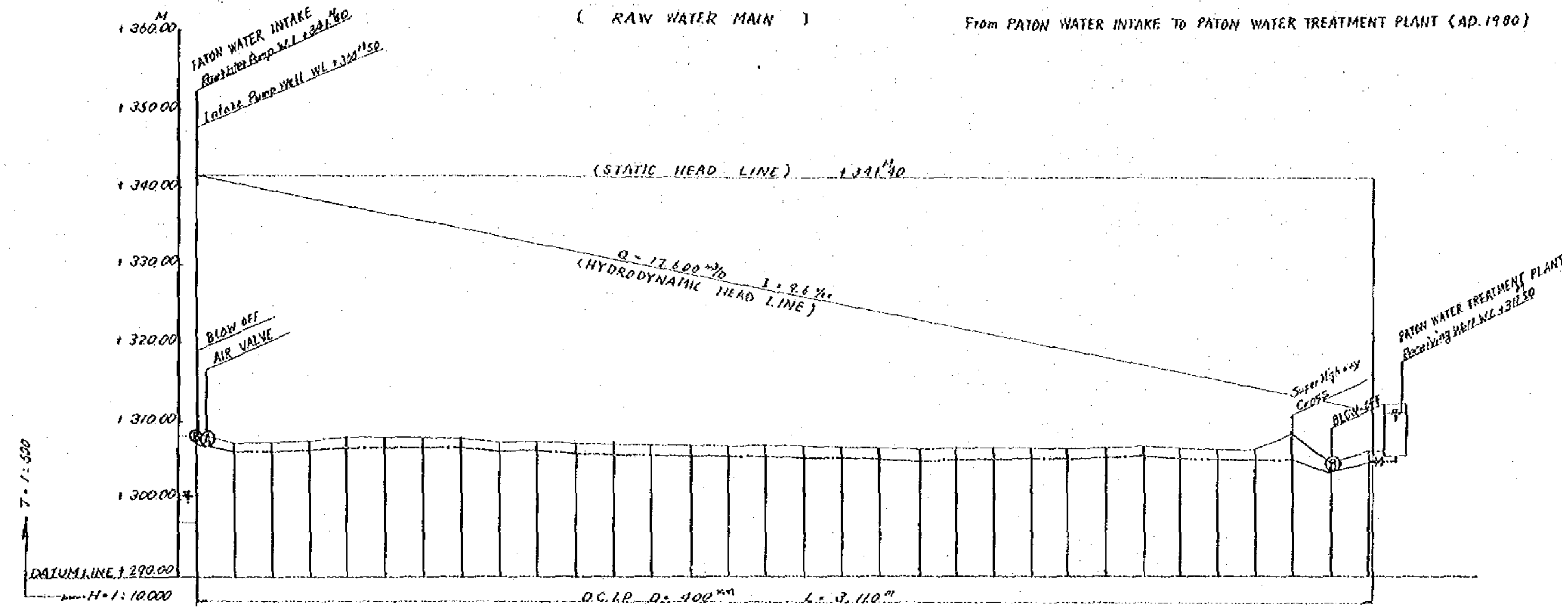
DISTRIBUTION MAIN
DIAGRAM OF FLOW VOLUMES IN
DISTRIBUTION MAIN (A.D. 2000)

APPROVED
DATE:

SCALE: / DRAWING NUMBER: /

(RAW WATER MAIN)

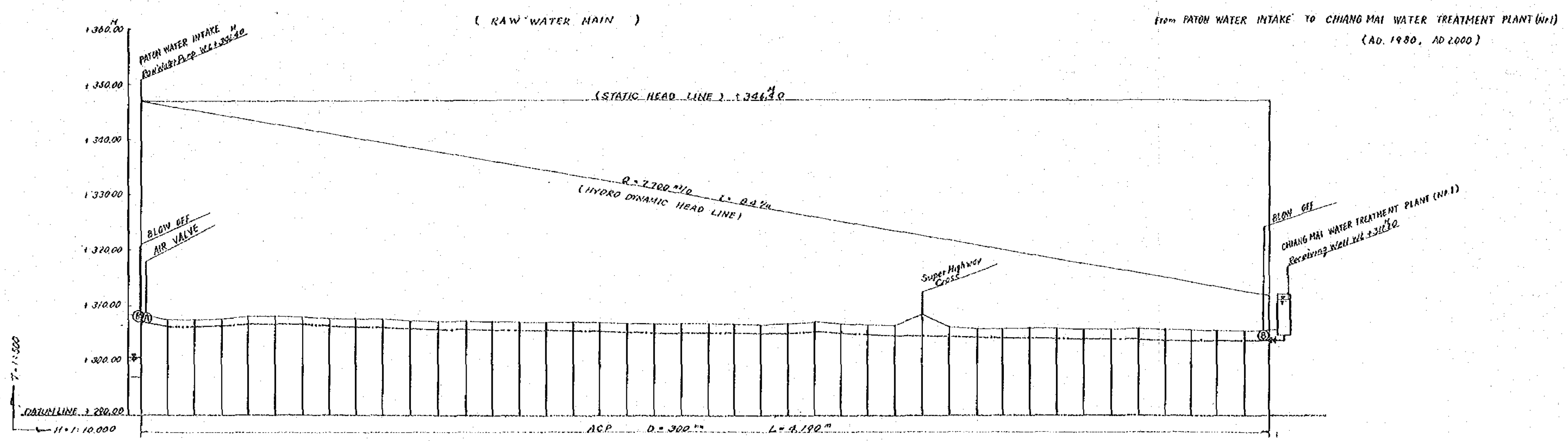
From PATON WATER INTAKE To PATON WATER TREATMENT PLANT (AD.1980)



STATION	SECTIONAL DISTANCE	TOTAL DISTANCE	GROUND HEIGHT	CENTRAL HEIGHT OF PIPE	EFFECTIVE HEAD	HYDRO-DYNAMIC HEAD	STATIC HEAD
0	0	0	308.50	307.08	32.90	341.40	341.40
	100	100	307.38				
	100	200	307.35				
	100	300	307.46				
	100	400	307.97				
	100	500	307.90				
	100	600	307.01				
	100	700	307.79				
	100	800	307.35				
	100	900	307.61				
1	100	1,000	307.16	305.06	26.44	331.80	
	100	1,100	306.97				
	100	1,200	306.90				
	100	1,300	306.81				
	100	1,400	306.72				
	100	1,500	306.77				
	100	1,600	306.73				
	100	1,700	306.65				
	100	1,800	306.55				
	100	1,900	306.39				
2	100	2,000	306.49	306.00	15.71	322.20	
	100	2,100	306.55				
	100	2,200	306.51				
	100	2,300	306.43				
	100	2,400	306.56				
	100	2,500	306.72				
	100	2,600	306.46				
	100	2,700	306.28				
	100	2,800	306.24				
	100	2,900	308.28				
3	100	3,000	306.60	302.10	8.00	312.60	
	100 / 10	3,100 / 3,110	306.00 / 308.00	306.50	5.50	311.50	341.40

(RAW WATER MAIN)

from PATON WATER INTAKE TO CHIANG MAI WATER TREATMENT PLANT (No.1)
(AD. 1980, AD 2000)



STATIC HEAD	346.40										
HYDRO-DYNAMIC HEAD	346.40										338.00
EFFECTIVE HEAD	37.90										30.84
CENTRAL HEIGHT OF PIPE	307.15										305.81
GROUND HEIGHT	308.50	307.38	307.35	307.40	307.67	307.90	307.91	307.79	307.55	307.41	307.16
TOTAL DISTANCE	0	100	200	300	400	500	600	700	800	900	1,000
SECTIONAL DISTANCE	0	100	100	100	100	100	100	100	100	100	100
STATION	0										1

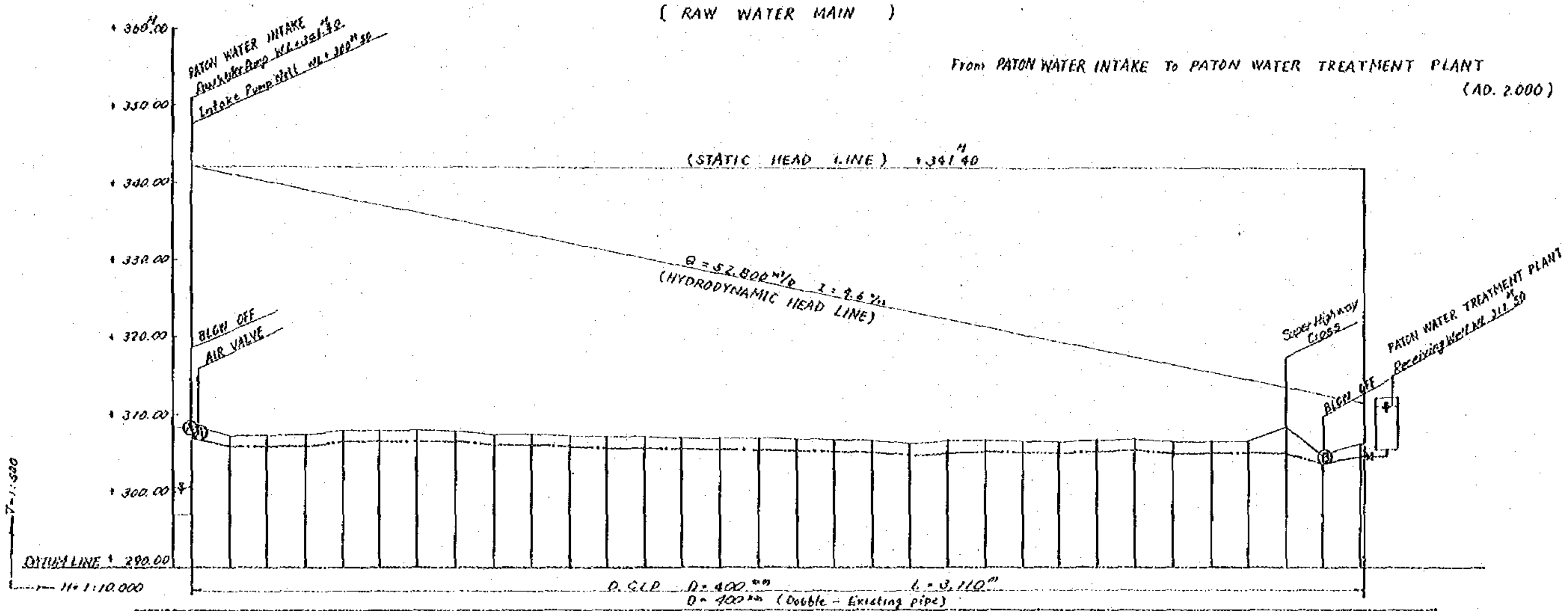
STATIC HEAD											
HYDRO-DYNAMIC HEAD										329.60	
EFFECTIVE HEAD										23.11	
CENTRAL HEIGHT OF PIPE										305.14	
GROUND HEIGHT	306.97	306.90	306.81	306.72	306.77	306.73	306.65	306.55	306.39	306.49	306.55
TOTAL DISTANCE	1,100	1,200	1,300	1,400	1,500	1,600	1,700	1,800	1,900	2,000	2,100
SECTIONAL DISTANCE	100	100	100	100	100	100	100	100	100	100	100
STATION										2	

STATIC HEAD											
HYDRO-DYNAMIC HEAD									321.20		
EFFECTIVE HEAD									15.26		
CENTRAL HEIGHT OF PIPE									304.59		
GROUND HEIGHT	306.51	306.45	306.56	306.72	306.66	306.28	306.24	308.26	305.94	305.68	305.55
TOTAL DISTANCE	2,200	2,300	2,400	2,500	2,600	2,700	2,800	2,900	3,000	3,100	3,200
SECTIONAL DISTANCE	100	100	100	100	100	100	100	100	100	100	100
STATION									3		

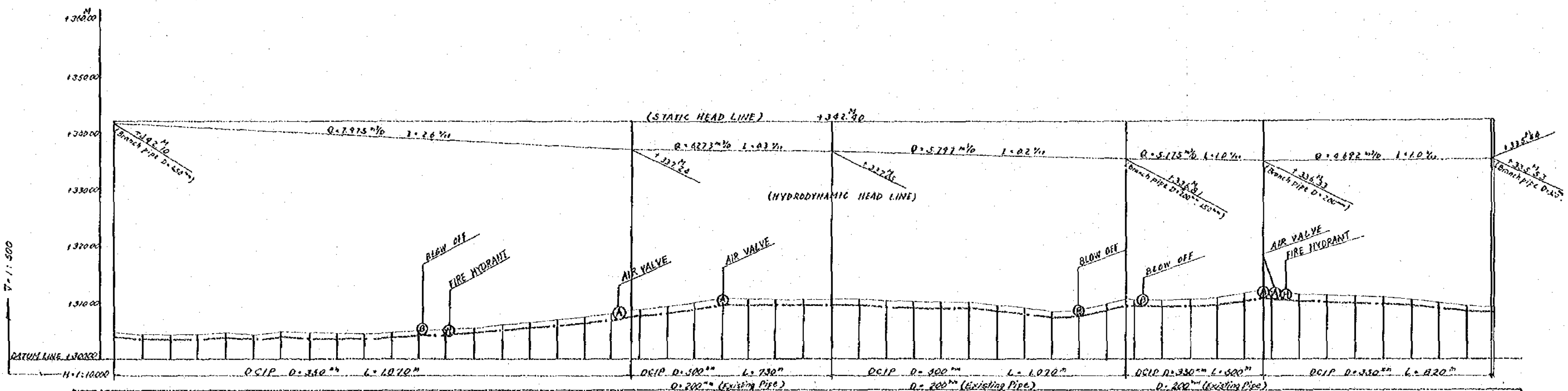
STATIC HEAD										346.40
HYDRO-DYNAMIC HEAD								312.80		311.20
EFFECTIVE HEAD								7.40		5.80
CENTRAL HEIGHT OF PIPE								304.05		306.05
GROUND HEIGHT	305.71	305.69	305.63	305.59	305.71	305.51	305.50	305.60	305.36	305.60
TOTAL DISTANCE	3,300	3,400	3,500	3,600	3,700	3,800	3,900	4,000	4,100	4,190
SECTIONAL DISTANCE	100	100	100	100	100	100	100	100	100	90
STATION								4		1

(RAW WATER MAIN)

From PATON WATER INTAKE To PATON WATER TREATMENT PLANT
(AD. 2,000)

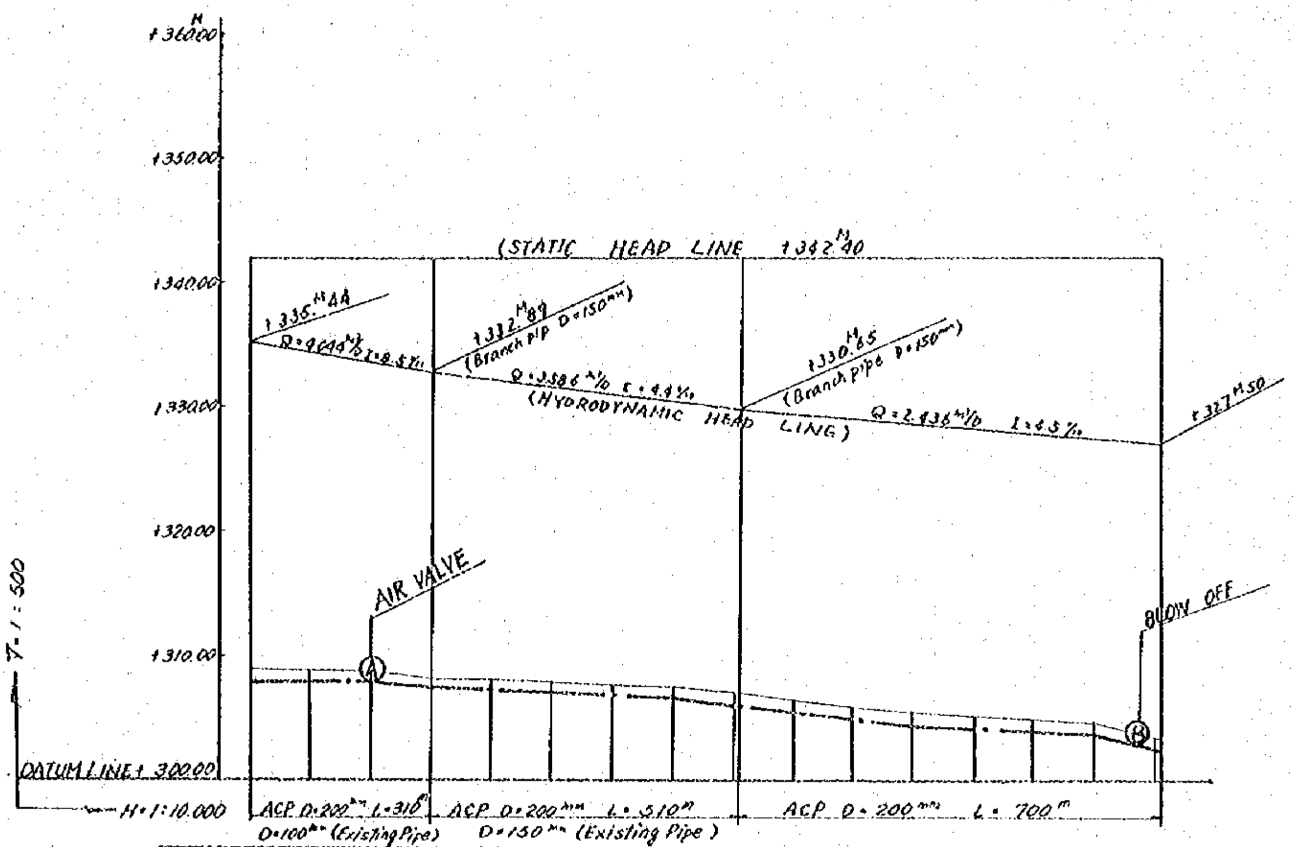


STATION	SECTIONAL DISTANCE	TOTAL DISTANCE	GROUND HEIGHT	CENTRAL HEIGHT OF PIPE	EFFECTIVE HEAD	HYDRO-DYNAMIC HEAD	STATIC HEAD
0	0	0	308.50	307.00	32.90	341.40	341.40
	100	100	307.38				
	100	200	307.35				
	100	300	307.46				
	100	400	307.97				
	100	500	307.90				
	100	600	307.91				
	100	700	307.79				
	100	800	307.35				
	100	900	307.41				
1	100	1,000	307.16	305.66	24.64	331.20	
	100	1100	306.97				
	100	1200	306.90				
	100	1300	306.81				
	100	1400	306.72				
	100	1500	306.77				
	100	1600	306.73				
	100	1700	306.65				
	100	1,800	306.55				
	100	1,900	306.39				
2	100	2,000	306.49	306.99	15.71	322.20	
	100	2,100	306.55				
	100	2,200	306.51				
	100	2,300	306.63				
	100	2,400	306.56				
	100	2,500	306.72				
	100	2,600	306.46				
	100	2,700	306.28				
	100	2,800	306.74				
	100	2,900	308.28				
3	100	3,000	306.60	303.10	8.00	312.60	
	108 10	3,108 3,110	306.00 306.00	306.50	5.50	311.50	341.40

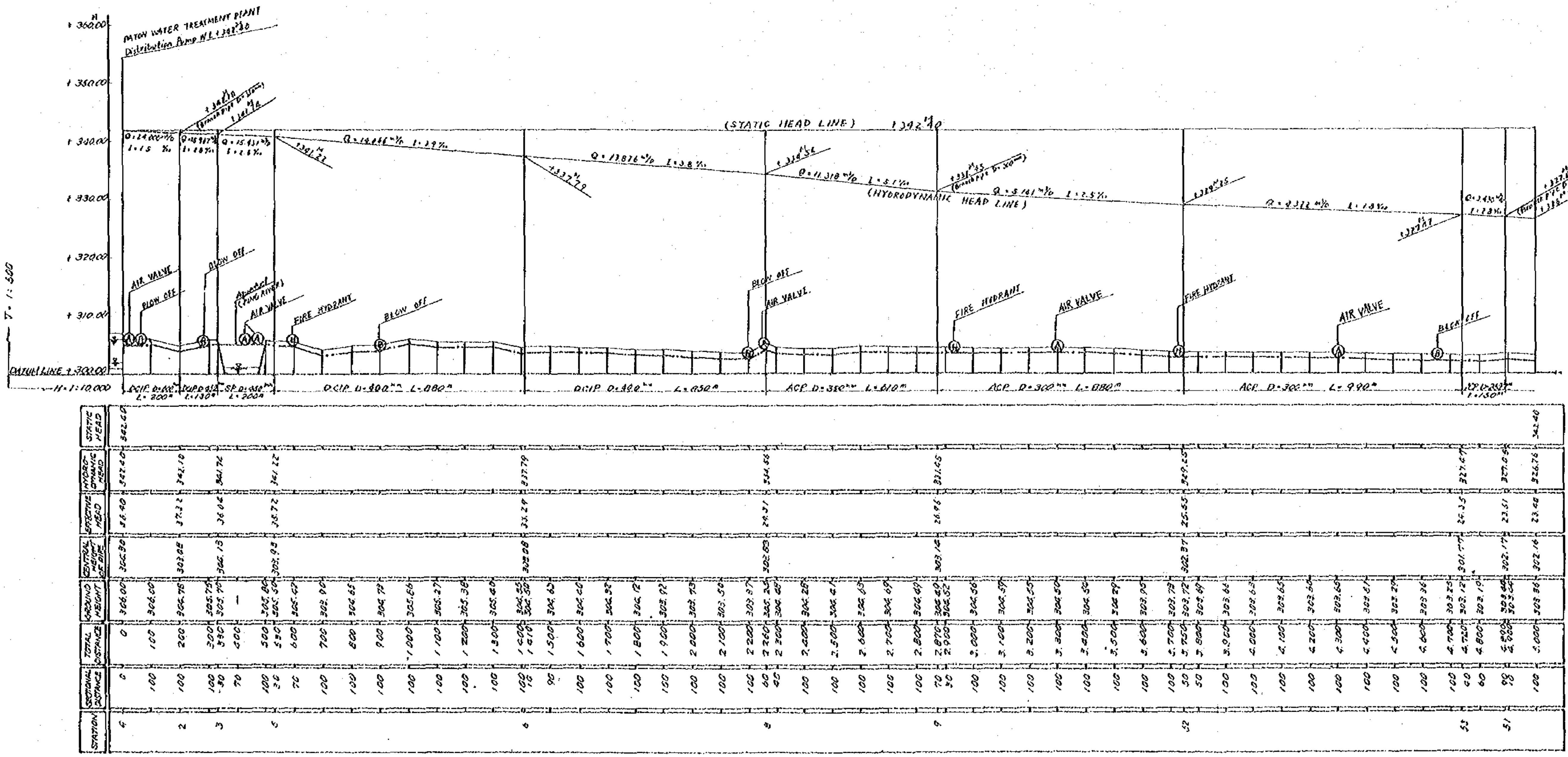


STATION	SECTIONAL DISTANCE	TOTAL DISTANCE	GROUND HEIGHT	CENTRAL HEIGHT OF PIPE	EFFECTIVE HEAD	HYDRODYNAMIC HEAD	STATIC HEAD
2	0	0	304.78	303.36	37.32	342.10	342.40
	100	100	304.65				
	100	200	304.41				
	100	300	304.36				
	100	400	304.71				
	100	500	304.38				
	100	600	304.78				
	100	700	304.56				
	100	800	304.02				
	100	900	304.33				
	100	1,000	304.69				
	100	1,100	304.94				
	100	1,200	305.10				
	100	1,300	305.62				
	100	1,400	305.93				
	100	1,500	306.20				
	100	1,600	306.81				
	100	1,700	307.20				
	100	1,800	307.41				
1	70	1,870	308.66	307.09	28.60	337.24	
	30	1,900	308.68				
	100	2,000	309.05				
	100	2,100	309.85				
	100	2,200	310.48				
	100	2,300	310.57				
	100	2,400	310.51				
	100	2,500	310.63				
21	100	2,600	310.60	308.96	26.56	337.05	
	100	2,700	310.55				
	100	2,800	310.26				
	100	2,900	310.09				
	100	3,000	310.45				
	100	3,100	310.07				
	100	3,200	309.61				
	100	3,300	309.12				
	100	3,400	308.45				
	100	3,500	308.66				
	100	3,600	309.60				
22	70	3,670	310.57	309.02	26.24	336.81	
	30	3,700	310.51				
	100	3,800	310.39				
	100	3,900	310.53				
	100	4,000	310.07				
	100	4,100	311.63				
23	70	4,170	312.21	310.79	26.12	336.33	
	30	4,200	311.96				
	100	4,300	311.43				
	100	4,400	311.28				
	100	4,500	311.20				
	100	4,600	311.06				
	100	4,700	310.57				
	100	4,800	309.96				
	100	4,900	309.47				
53	90	4,990	309.16	307.74	26.37	335.83	342.40
	10	5,000	309.16	307.06	22.28	335.44	

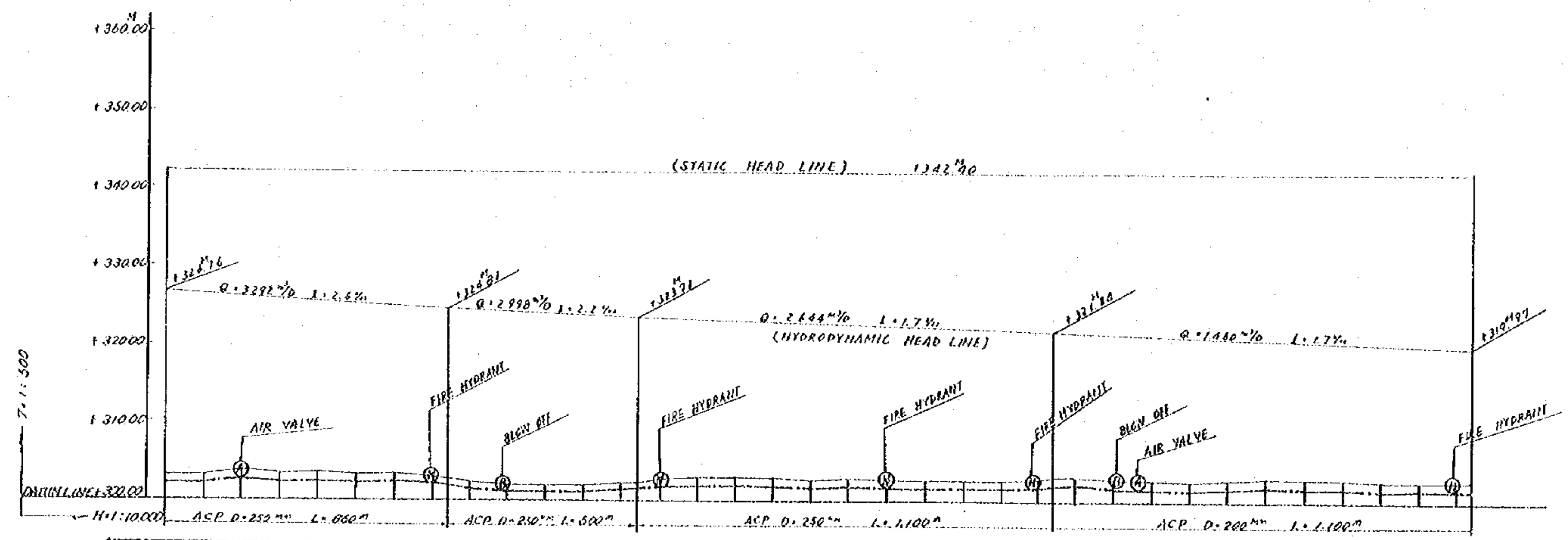
From PATON WATER TREATMENT PLANT To MOE HIA (AD 1980) 2/2



STATIC HEAD	342.40															342.40
HYDRO-DYNAMIC HEAD	335.44			332.89					330.65							327.50
EFFECTIVE HEAD	26.28			24.59					23.51							23.83
CENTRAL HEIGHT OF PIPE	307.96			307.10					305.04							302.47
GROUND HEIGHT	309.15	308.99	308.89	308.30	308.28	308.01	307.67	307.63	307.18 307.14	306.54	306.19	305.72	305.27	304.93	304.63	304.55 303.67
TOTAL DISTANCE	5.000	5.100	5.200	5.300	5.400	5.500	6.600	5.700	5.800 5.810	5.900	6.000	6.100	6.200	6.300	6.400	6.500 6.510
SECTIONAL DISTANCE	10	100	100	100	100	100	100	100	100 10	90	100	100	100	100	100	100 10
STATION				68					69							70

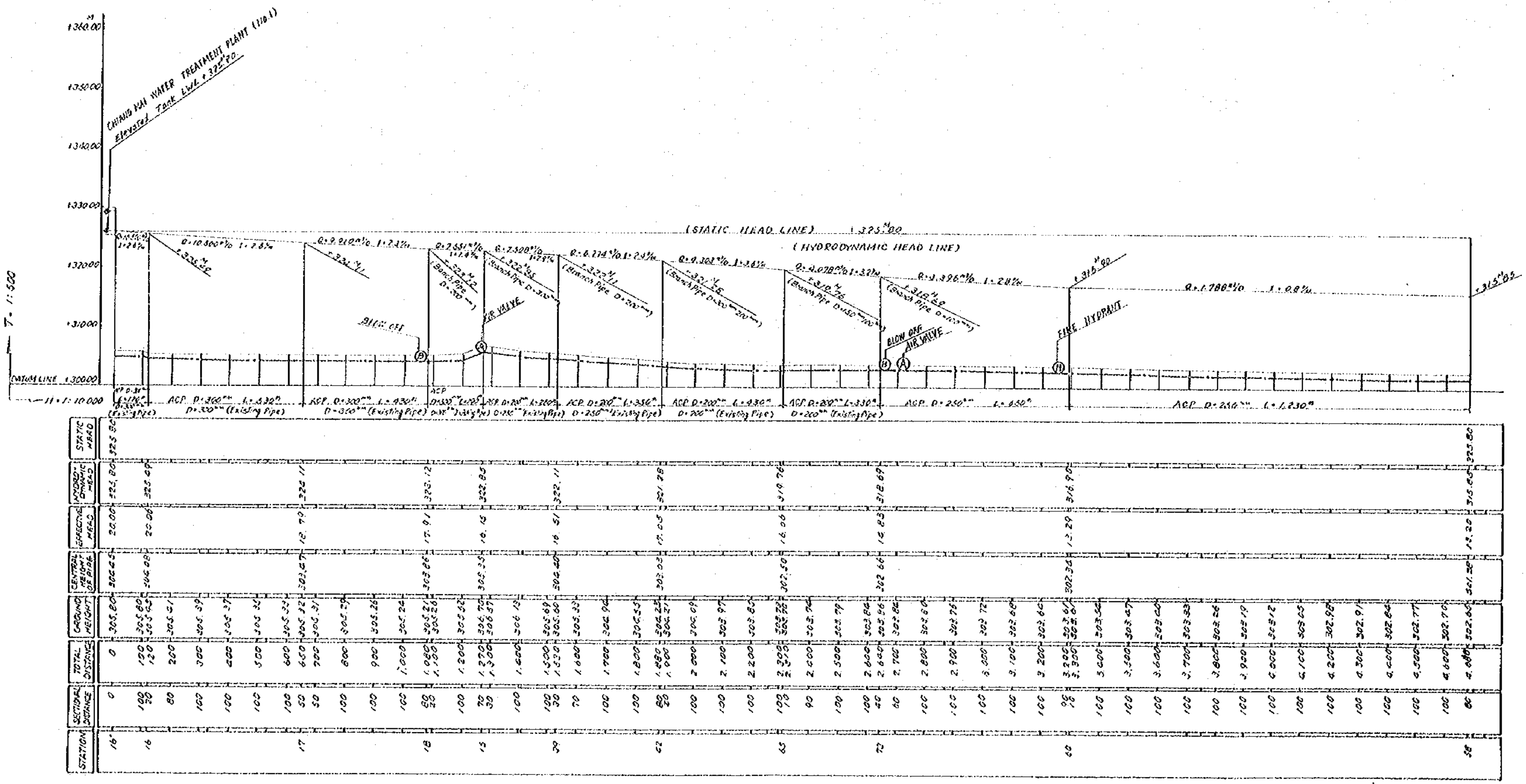

PATON WATER TREATMENT PLANT
Distribution Pump W.L +342.40
(STATIC HEAD LINE) +342.40
(HYDRODYNAMIC HEAD LINE)
AIR VALVE
BLOW OFF
FIRE HYDRANT
DATUM LINE +300.00
H = 1:10,000
V = 1:500
DCIP D=400mm L=880m
DCIP D=400mm L=850m
ACP D=350mm L=610m
ACP D=300mm L=880m
ACP D=300mm L=990m
STATIC HEAD
HYDRODYNAMIC HEAD
EFFECTIVE HEAD
CENTRAL HEIGHT OF PIPE
GROUND HEIGHT
TOTAL DISTANCE
SECTIONAL DISTANCE
STATION

From PATON WATER TREATMENT PLANT To LAMPOON (AD 1980) 2/2

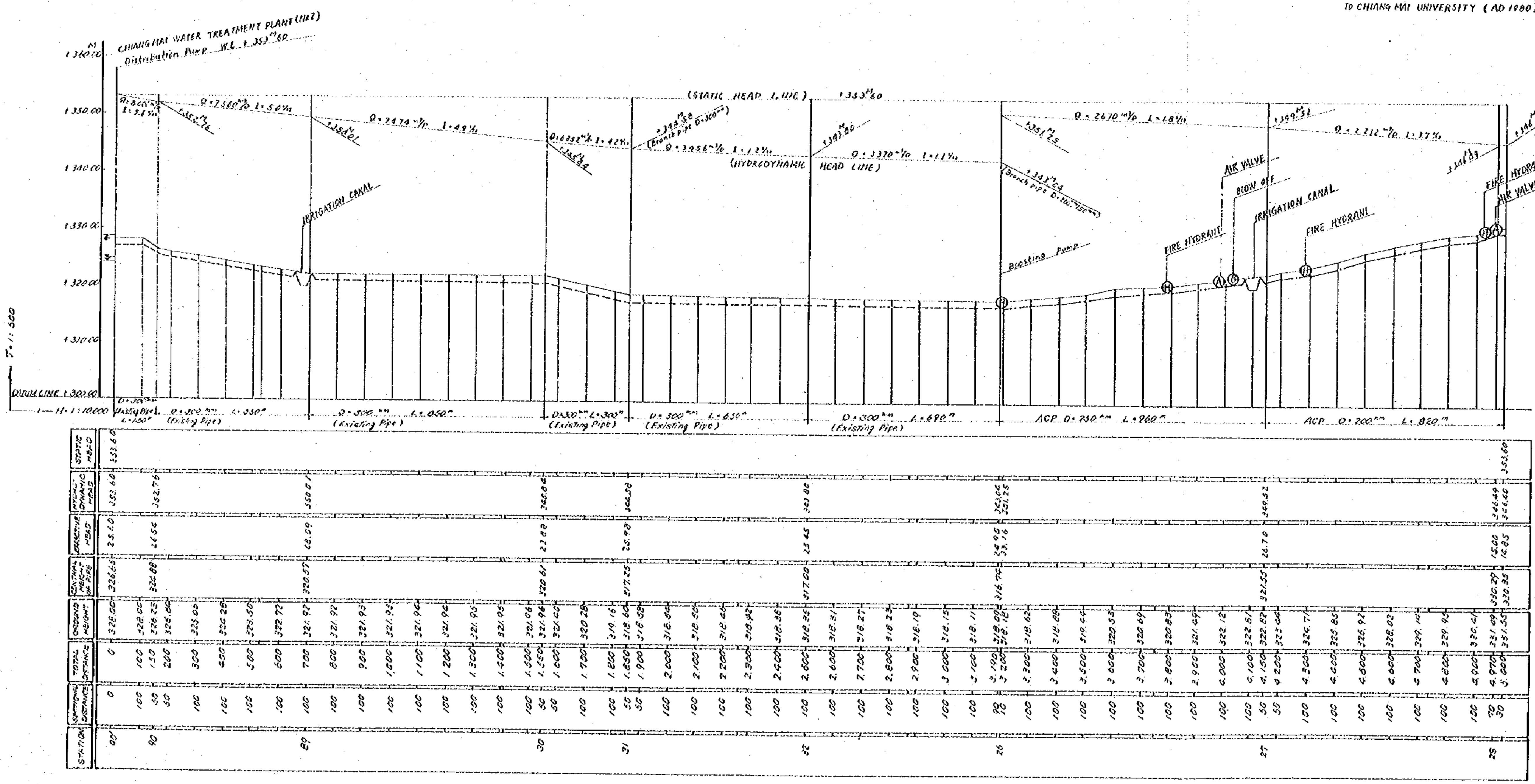


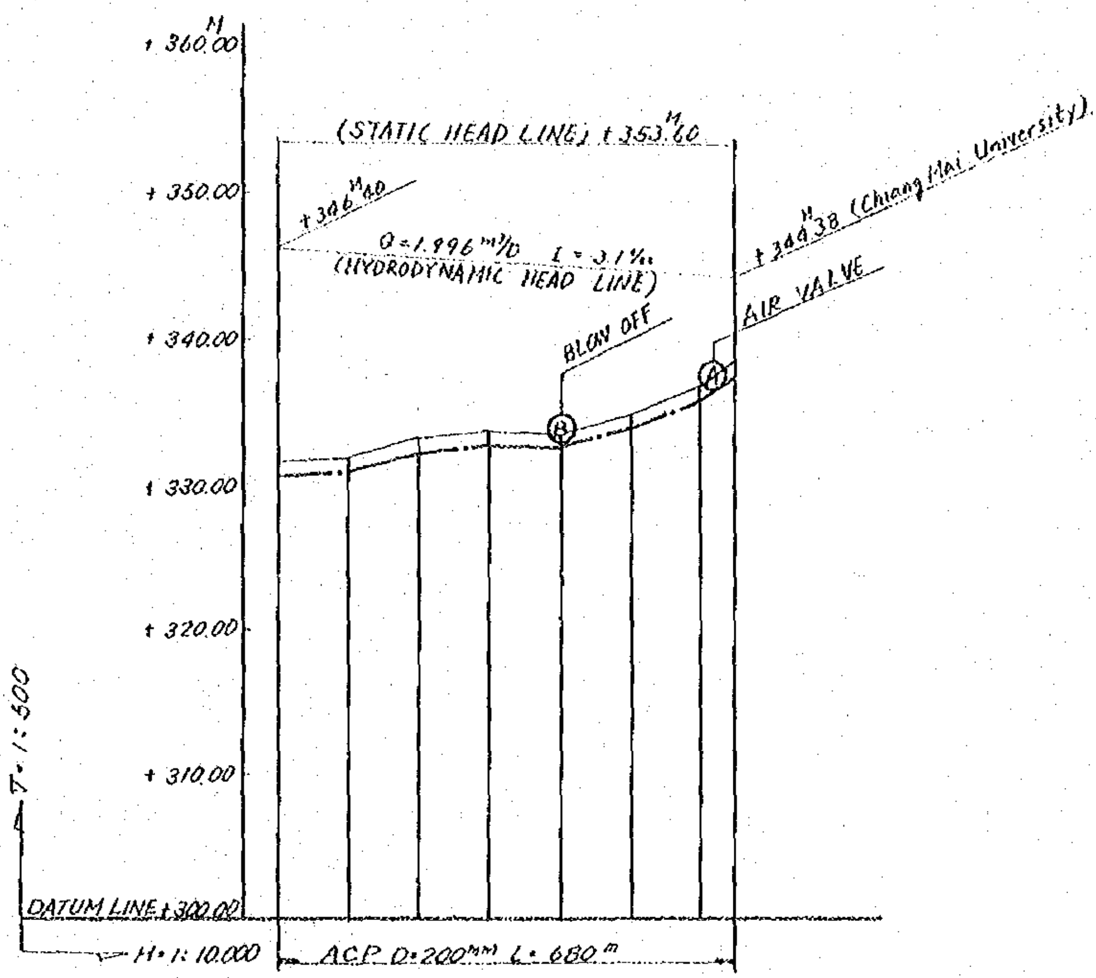
STATION	SECTIONAL DISTANCE	TOTAL DISTANCE	GROUND HEIGHT	CENTRAL HEIGHT OF PIPE	EFFECTIVE HEAD	HYDRO-DYNAMIC HEAD	STATIC HEAD
	100	5.000	303.36	302.16	23.60	326.76	342.40
	100	5.100	303.36				
	100	5.200	303.51				
	100	5.300	303.63				
	100	5.400	303.56				
	100	5.500	303.32				
	100	5.600	303.66				
	100	5.700	303.30				
54	40	5.740	303.06	301.84	21.77	324.81	
	60	5.800	302.67				
	100	5.900	302.16				
	100	6.000	302.26				
	100	6.100	302.07				
	100	6.200	302.58				
55	40	6.240	302.40	301.20	21.31	323.71	
	60	6.300	302.70				
	100	6.400	302.11				
	100	6.500	302.06				
	100	6.600	302.93				
	100	6.700	302.65				
	100	6.800	302.63				
	100	6.900	302.81				
	100	7.000	302.85				
	100	7.100	302.84				
	100	7.200	302.86				
	100	7.300	303.13				
56	40	7.340	303.42	302.22	18.42	321.64	
	60	7.400	303.56				
	100	7.500	303.09				
	100	7.600	302.76				
	100	7.700	302.62				
	100	7.800	302.65				
	100	7.900	303.32				
	100	8.000	303.18				
	100	8.100	303.13				
	100	8.200	302.83				
	100	8.300	302.85				
	100	8.400	302.88				
57	40	8.440	302.96	301.76	17.03	319.97	342.40

From CHIANGMAI WATER TREATMENT PLANT (No.1) To CHIANG MAI CITY.
(AD. 1980)

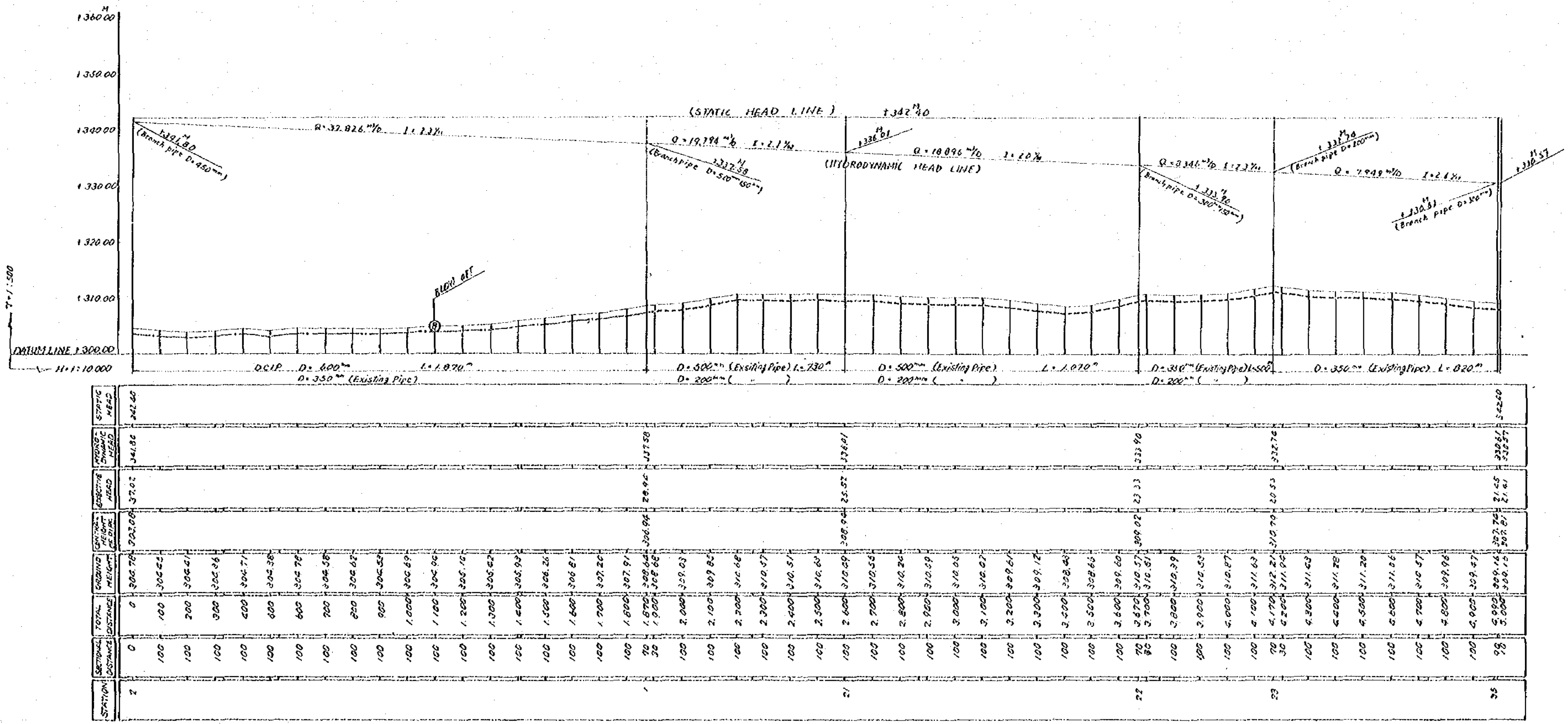


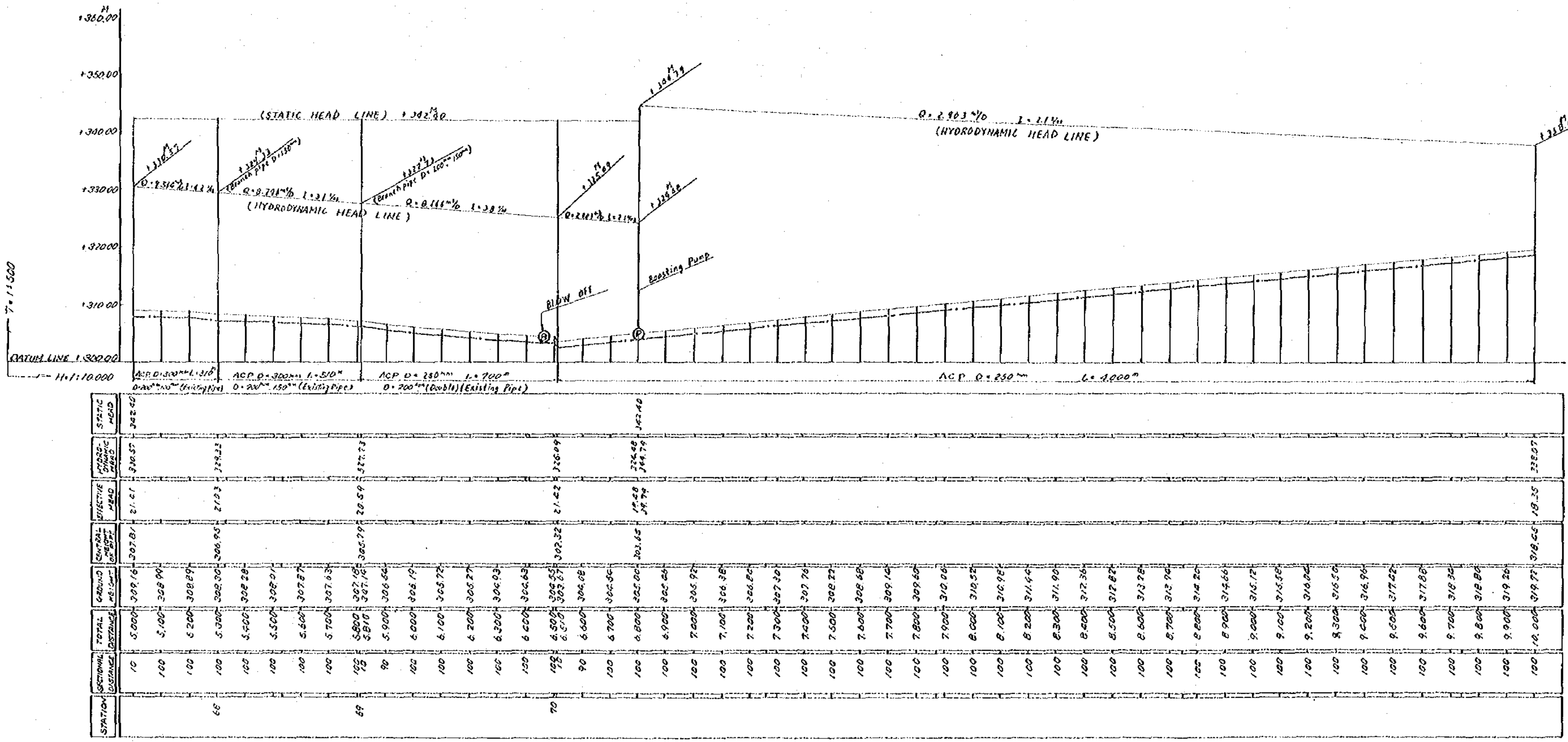
From CHIANGMAI WATER TREATMENT PLANT (No.2)
To CHIANG MAI UNIVERSITY (AD 1980) 1/2



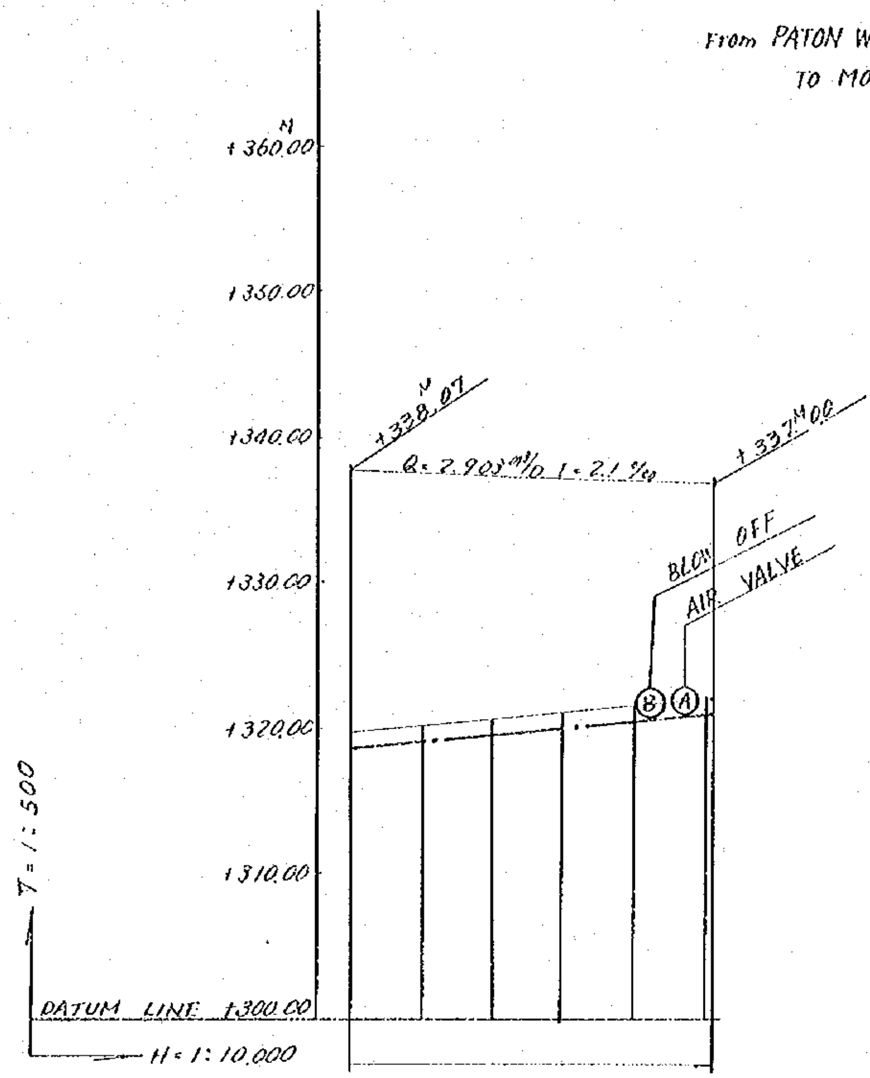


STATIC HEAD	353.60							353.60
HYDRO-DYNAMIC HEAD	346.40							344.38
EFFECTIVE HEAD	14.85							5.76
CENTRAL HEIGHT OF PIPE	330.35							337.42
GROUND HEIGHT	331.55	331.85	333.16	333.63	333.31	334.67	336.71	338.62
TOTAL DISTANCE	5,000	5,100	5,200	5,300	5,400	5,500	5,600	5,650
SECTIONAL DISTANCE	30	100	100	100	100	100	100	50
STATION								27

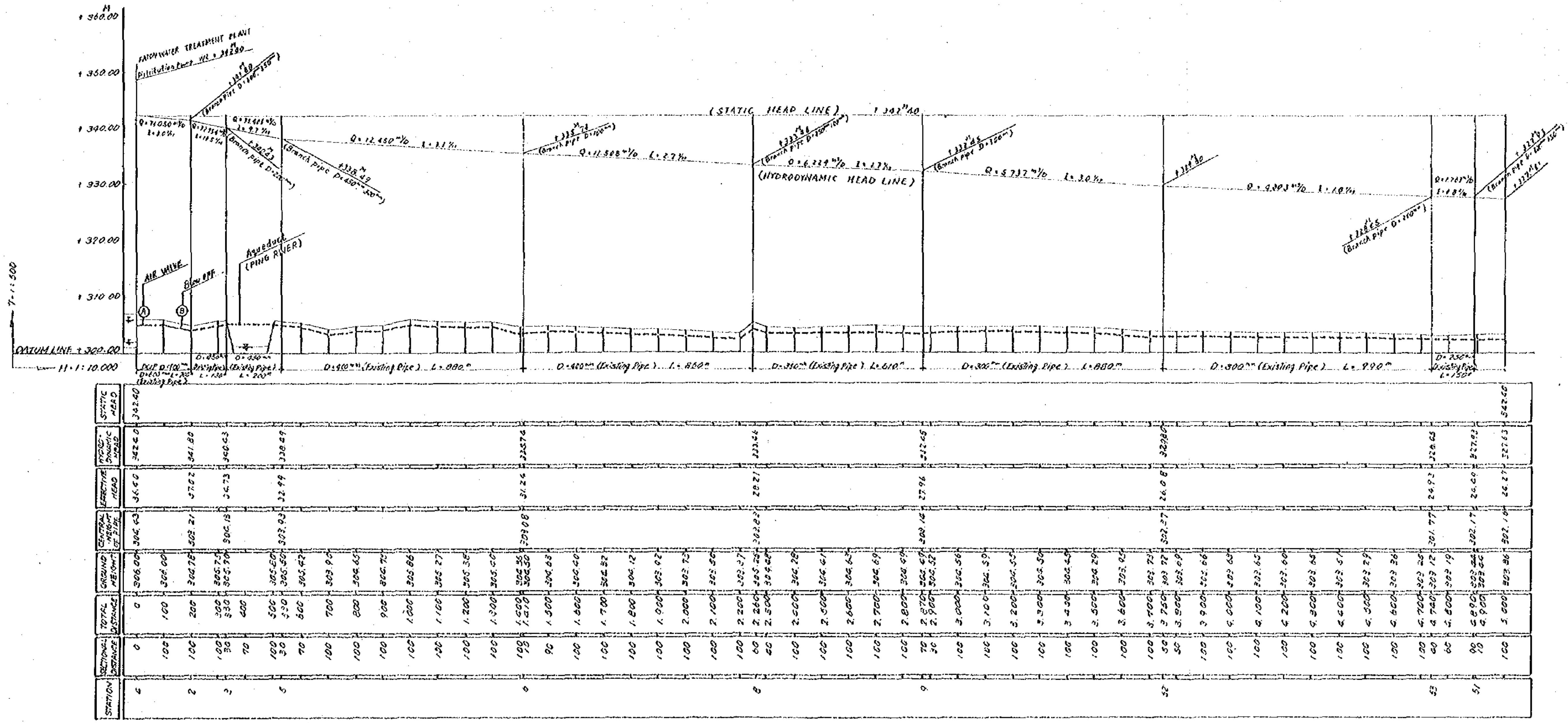

STATIC HEAD LINE
HYDRODYNAMIC HEAD LINE
BLOW OFF
DATUM LINE
H=1:10 000
V=1:500
DCIP D=400mm L=1.870m
D=350mm (Existing Pipe)
D=500mm (Existing Pipe) L=730m
D=200mm
D=500mm (Existing Pipe) L=1.070m
D=350mm (Existing Pipe) L=500
D=350mm (Existing Pipe) L=820m
STATIC HEAD
HYDRODYNAMIC HEAD
EFFECTIVE HEAD
CENTRAL HEIGHT OF PIPE
GROUND HEIGHT
TOTAL DISTANCE
SECTIONAL DISTANCE
STATION


(STATIC HEAD LINE) +342.40
(HYDRODYNAMIC HEAD LINE)
BLOW OFF
Boosting Pump
DATUM LINE +300.00
H=1:10,000
V=1:500
ACP D=250mm L=1,000m
STATIC HEAD
HYDRODYNAMIC HEAD
EFFECTIVE HEAD
CENTRAL HEIGHT OF PIPE
GROUND HEIGHT
TOTAL DISTANCE
SECTIONAL DISTANCE
STATION

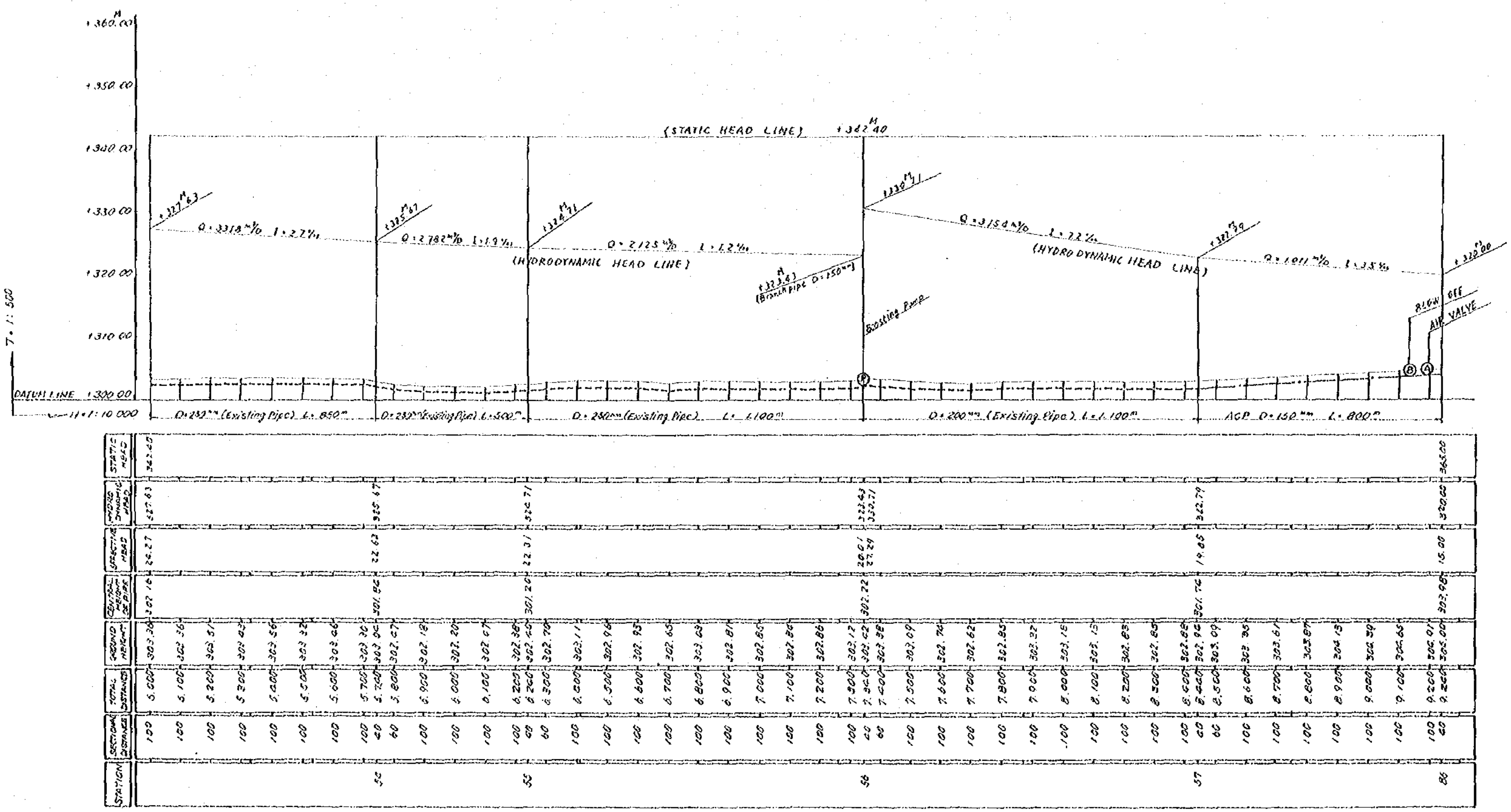
From PATON WATER TREAMENT PLANT
TO MOE H/A (AD 2.000) 3/3



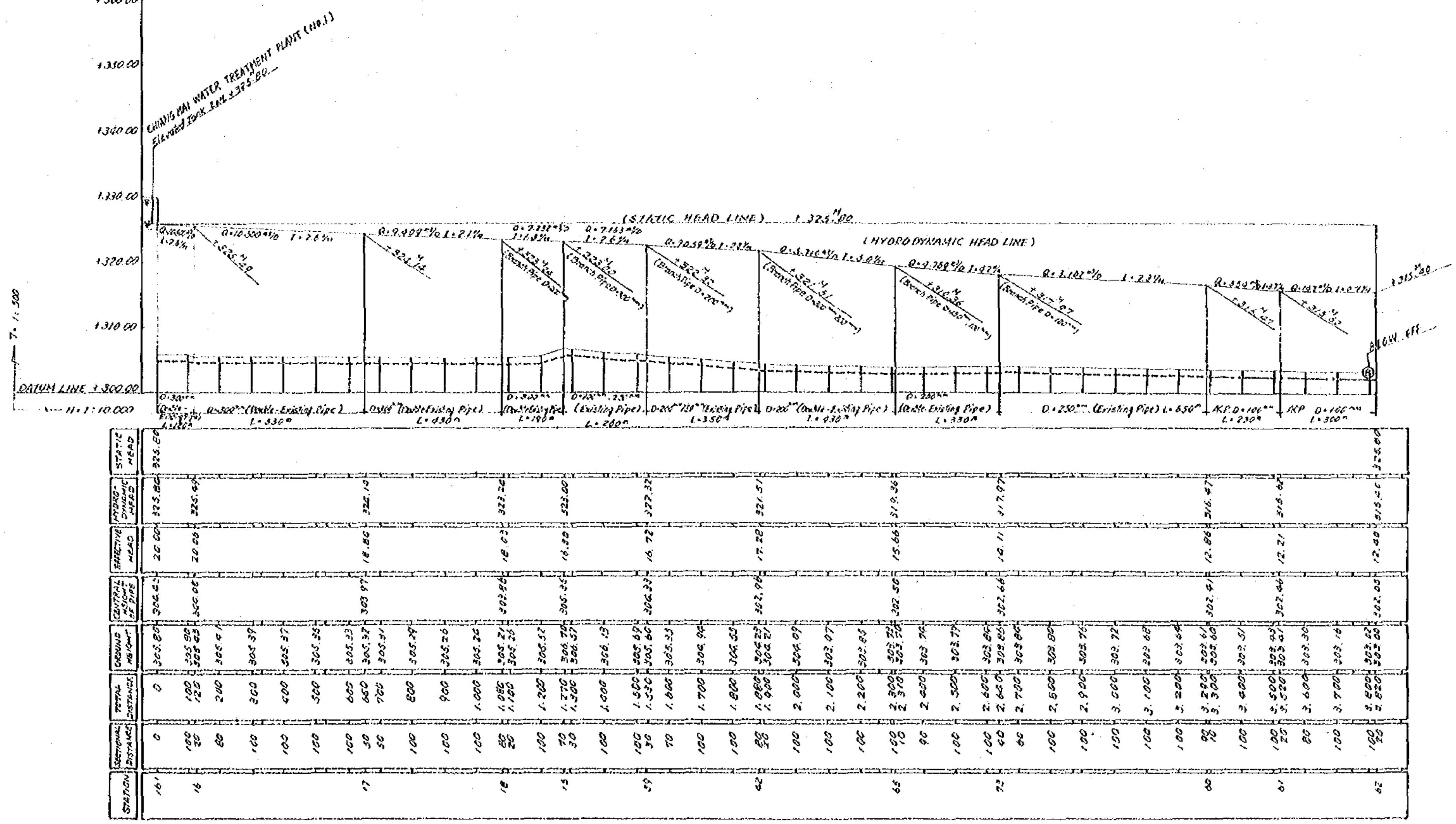
STATIC HEAD						
HYDRO-DYNAMIC HEAD	338.07					337.00
EFFECTIVE HEAD	18.35					15.00
CENTRAL HEIGHT OF PIPE	318.43					320.73
GROUND HEIGHT	319.72	320.18	320.64	321.10	321.56	321.95 322.00
TOTAL DISTANCE	10.000	10.100	10.200	10.300	10.400	10.500 10.510
SECTIONAL DISTANCE	100	100	100	100	100	100 10
STATION						57

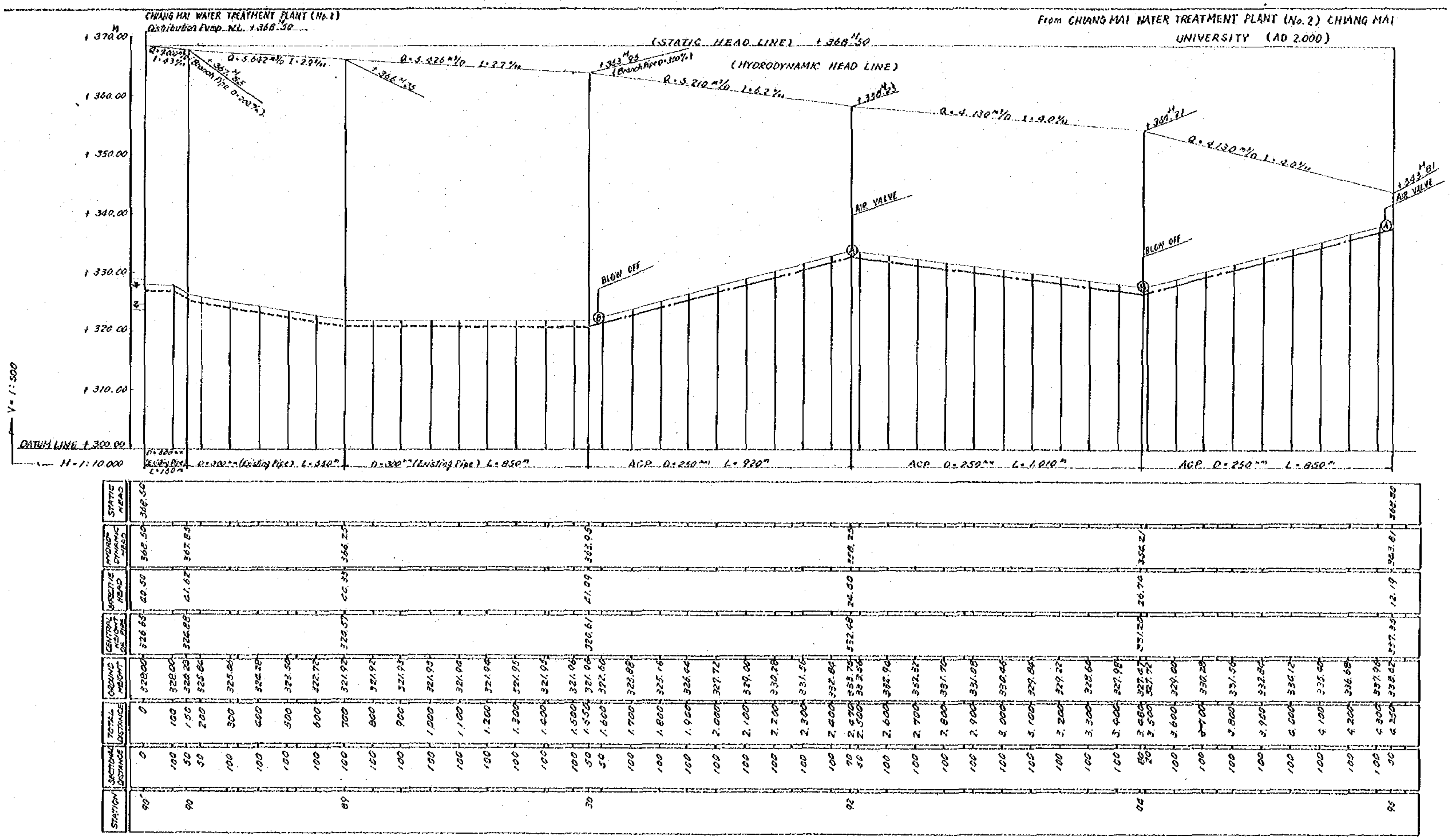


PATON WATER TREATMENT PLANT
(STATIC HEAD LINE)
(HYDRODYNAMIC HEAD LINE)
AIR VALVE
Blow off
Aqueduct (PING RIVER)
DATUM LINE
V : 1 : 500
H : 1 : 10.000
STATIC HEAD
HYDRODYNAMIC HEAD
EFFECTIVE HEAD
CENTRAL HEIGHT OF PIPE
GROUND HEIGHT
TOTAL DISTANCE
SECTIONAL DISTANCE
STATION


(STATIC HEAD LINE) +342.40
(HYDRODYNAMIC HEAD LINE)
(HYDRO DYNAMIC HEAD LINE)
Existing Pump
BLOW OFF
AIR VALVE
DATUM LINE +300.00
V: 1: 500
H: 1: 10 000
D=250mm (Existing Pipe) L=850m
D=250mm (Existing Pipe) L=500m
D=250mm (Existing Pipe) L=1100m
D=200mm (Existing Pipe) L=1100m
ACP D=150mm L=800m
STATIC HEAD
HYDRO DYNAMIC HEAD
EFFECTIVE HEAD
CENTRAL HEIGHT OF PIPE
GROUND HEIGHT
TOTAL DISTANCE
SECTIONAL DISTANCE
STATION

From CHIANGMAI WATER TREATMENT PLANT (No.1) TO CHIANG MAI CITY
(AD 2.000)





CHIANG MAI WATER TREATMENT PLANT (No.2)
Distribution Pump W.L. +368.50
From CHIANG MAI WATER TREATMENT PLANT (No.2) CHIANG MAI UNIVERSITY (AD 2.000)
(STATIC HEAD LINE) +368.50
(HYDRODYNAMIC HEAD LINE)
+370.00
+360.00
+350.00
+340.00
+330.00
+320.00
+310.00
DATUM LINE +300.00
V=1:500
H=1:10.000
AIR VALVE
BLOW OFF
ACP D=250mm L=920m
ACP D=250mm L=1.010m
ACP D=250mm L=850m
STATIC HEAD
HYDRODYNAMIC HEAD
EFFECTIVE HEAD
CENTRAL HEIGHT OF PIPE
GROUND HEIGHT
TOTAL DISTANCE
SECTIONAL DISTANCE
STATION

4-2-6 Calculations of Pipe Thicknesses

4-2-6-1 Loads Considered

a) Interior Pressure

Still water pressure: Ps

Moving water pressure: Pd

b) Exterior pressure

Earth covering

Truck weights and shock pressures

4-2-6-2 Method of Calculation

a) Induction

Tensile stress due to interior pressure, at

$$\sigma_t = \frac{(P_s + P_d)\ d}{2t} \quad \cdots\cdots\cdots\cdots \quad (1)$$

also, d = inner diameter of pipe, t = pipe thickness.

Torsion stress due to exterior pressure, σ_b

$$\sigma_b = \frac{6(M_f + M_t)_{max}}{t^2} = \frac{6(K_f W_f + K_t W_t) R^2}{t^2} \quad \cdots\cdots\cdots \quad (2)$$

$$M_f = K_f . W_f . R^2 \quad \cdots\cdots\cdots\cdots \quad (3)$$

$$M_t = K_t . W_t . R^2 \quad \cdots\cdots\cdots\cdots \quad (4)$$

also,

M_f: torsion moment caused by the earth cover

M_t: torsion moment caused by truck loads

W_f: earth pressure due to earth cover

Wt: earth pressure due to truck loads

R : width of ditch

Multiply torsion stress σ_b by 0.7 to convert to tensile stress. Pipe thicknesses shall be determined so as to satisfy the following equation.

$$\sigma_t + 0.7\sigma_b = \sigma_a \quad \cdots\cdots\cdots\cdots \quad (5)$$

b) Safety Factor

For still water pressure: 2.5

For moving water, earth cover and truck weights: 2.0

c) Method

When the drag tension stress of the pipe material is designated s, formula (5) becomes as follows.

$$2.5\sigma_{ts} + 2.0\sigma_{td} + 1.4\sigma_{b} = S \quad \text{.......................} \quad (6)$$

σ_{ts} : stress due to still water pressure

σ_{td} : stress due to moving water pressure

Leaving R = d/2, t may be formulated as follows:

$$t = \frac{1.25P_s + P_d + \sqrt{(1.25P_s + P_d)^2 + 8.4(K_f W_f + K_t W_t)S}}{2S} d$$

$W_f = \gamma . H$

: unit weight of earth

H : depth of earth cover

$$W_t = \frac{2P(1+i)}{(2H+0.2)(2H+2.25)}$$

i : shock factor

P : rear wheel road

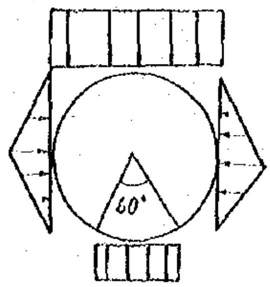
4-2-6-3 Design Conditions

Still water pressure, Ps: 4.0 Kg/cm^2

Moving water pressure, P_d: 5.5 Kg/cm^2

Assuming the distribution of pressure due to the earth cover to be as shown in Drawing A,

$K_f = 0.223$

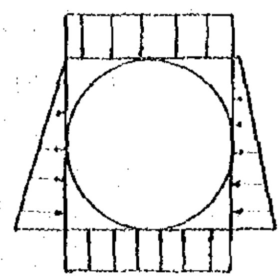


Drawing A

Assuming the distribution of earth pressure due to truck loads,

$K_t = 0.011$

Earth's unit weight, γ: 1.6 g/cm^3



Drawing B

Truck weight: T - 20

Shock factor due to truck weight: 0.5

Drag tension stress for pipe material:

Steel : 4100 kg/cm^2

Cast iron: 3800 kg/cm^2

4-2-6-4 Calculations

a) Steel Pipe for Casings (hydrostatic pressures and shock pressures not present)

Diameter (mm)	Earth Cover (cm)	Earth Load (Wf) (Kg/cm^2)	Load Due to 20-Ton Truck (Wt) (Kg/cm^2)	Calculated Pipe Thickness (t) (cm)	Pipe Thickness To Be Used (cm)
800	300	0.48	0.048	0.59	0.71
600	300	0.48	0.048	0.35	0.60

b) Ductile Cast Iron Pipe

Diameter (mm)	Earth Cover (cm)	Earth Load (Wf) (Kg/cm^2)	Load Due to 20-Ton-Truck (Wt) (Kg/cm^2)	Calculated Pipe Thickness (t) (cm)	Pipe Thickness To Be Used (cm)
					(Type 3)
600	140	0.225	0.159	0.64	0.9
500	130	0.208	0.176	0.51	0.8
450	135	0.216	0.16	0.47	0.75
400	130	0.208	0.176	0.41	0.7
350	125	0.20	0.187	0.35	0.65

Chapter 5 Construction Work Plans

5-1 Basic Conditions for Construction

5-2 Preparatory Work for Which Contractor is Responsible

5-3 Construction Working Plans for the Various Works

Chapter 5 Construction Work Plans

The following construction work plans are outline plans. The successful bidders shall, before beginning the construction, make a Construction Work Plan, submit it to the Engineer for approval, and execute the Works according to the approved plan.

5-1 Basic Conditions for Construction

The following points shall be taken into consideration in planning the work. The Contractor shall make a feasible economical and efficient construction work plan based on consideration of these points.

5-1-1 Natural Conditions Pertaining at the Work Site

5-1-1-1 Rainy Season

During the seven months from December through June, there is almost no rain. From the Kao Pun Sa in July and on, there is a heavy rainfall.

5-1-1-2 Nam Ping River

The river water is turbid at all times, and the water level is low during the dry season, but remains about one meter deep at the intake.

5-1-1-3 Planned Location of the Purification Plant

Because the land is a low paddy field along the highway, filling is necessary.

5-1-1-4 Drainage Channels

Since the intake plant is along the river, it will not be necessary to build special drainage channels. There are irrigation channels, carrying irrigation water during the irrigation season, located around the intended location of the purification plant.

5-1-2 Construction Machinery and Materials Procurable in Thailand

5-1-2-1 Materials

Materials procurable in Thailand include aggregate for concrete, sand, cement, reinforcing steel, brick, a hard stone of the region called silaraeng, wood for forms, crushed stone, and fuel for construction machinery.

5-1-2-2 Construction Equipment and Tools

Items procurable in Thailand include tractors, bulldozers, dump cars, etc. Also available are drop-hammer type pile drivers. There seem to be almost no construction firms possessing boring equipment or steel forms used for concrete.

5-1-2-3 Experienced Skilled Workers

Truck drivers are available but very few operators of special construction equipment.

5-2 Preparatory Work for Which Contractor is Responsible

Before beginning the construction work, the Contractor shall carry out the following.

5-2-1 Investigation of the Vicinity of the Site

5-2-1-1 During construction of the intake plant, a near-by area of land across the road from the intake plant site will be used as a material storage area and for fabrication. This area is the planned site of the housing facilities for the staff at the raw water intake plant.

5-2-1-2 The condition of the road from the highway to the intake plant shall be investigated, as repair of certain sections is necessary.

5-2-1-3 The proposed location of the new water purification plant is close to the highway. During the height of construction, the traffic of vehicles going in and out of the site will be heavy so measures will be necessary to prevent accidents with general traffic vehicles on the highway. A temporary auxiliary road will be necessary at two locations.

5-2-1-4 Preparations shall be made for provision of electricity, and water supply needed for the construction work. Sources of power need not always be electric.

5-2-2 Procedures and Applications to Concerned Government Offices

The Contractor shall make all applications, and effect all procedures necessary, including import and transportation of construction materials, liaison regarding exclusive use of locations on the highway, clearances for digging up roads, and clearances for work affecting river channels.

5-2-3 Offices and Warehouse

The Contractor shall build an Engineer's office, a Contractor's office and a warehouse on or in the vicinity of the site.

5-2-4 Arrangements for Tools and Materials for the Work

The contractor shall make adequate advance arrangements for imported items not domestically procurable such as steel sheet, etc., as well as for domestically produced materials such as sand, gravel, cement, crushed stone, lumber, reinforcing steel, and stone. Particularly, large amounts of earth and sand will be required for fills, so that sources, amounts available, the available capacity to transport, etc., shall be ascertained before commencing work.

5-2-5 Arrangements for Labor

The successful bidder will find it necessary to recruit special skilled labor from non-local sources, so arrangements for their housing must be made in advance. Ordinary laborers shall be recruited locally. But, the general rule shall be that location of housing will not be allowed at or on the construction site.

5-3 Construction Working Plans for the Various Works

5-3-1 Temporary Coffering for the Intake Facilities

Construction work shall be done during the dry season when the water level is comparatively low. To protect the area around the intake mouth, a temporary coffer dam shall be made using wood piling and sand-bags, with earth packed in between.

5-3-2 Embankment Work for the Grit Chamber

A large area is affected by the excavation of the raw water channel to a depth of 1.0 m below ground level. To lessen the affected area and prevent crumbling of the face of the cut, embanking work shall be done, using sheet piles.

Drainage shall be done by pumps operated by diesel engine, and the waste water shall be put into the river.

Earth from digging shall be taken to the site of the new proposed purification plant.

5-3-3 Refills of Earth at the Intake Plant

Underground water tends to cause rising of filled-in earth. Thus careful planning is necessary as to method and time for refilling in order to prevent structures built over refilled ground from tilting or collapsing.

5-3-4 Laying the Raw Water Main

The raw water main shall be laid at the same time as the 300 mm connecting main to the existing water purification plant and the 400 mm connecting main to the new proposed water purification plant. Excavation and refilling also will be done at the same time.

Where crossing the highway, a 800 mm diameter steel pipe shall be used as casing for the 400 mm main and a 600 mm diameter steel pipe as casing for the 300 mm diameter main for protection.

5-3-5 Land Preparation for the New Proposed Water Purification Plant

The construction site and the connecting road from the highway to the construction site for the 1st stage shall be built up to a height of +306 m, after the construction for the Receiving Well, Chemical Sedimentation Basin, Rapid Sand Filter and the Distribution Basin. Construction of facilities other than the foregoing shall be done after completing filling of the construction site.

5-3-6 Work for the Sand Mat of the Sedimentation Basin and Receiving Well

The bottom of the sedimentation basin and the receiving well will be built higher than the existing ground level. Therefore, sand shall be spread evenly and tamped down firmly before driving the piles.

5-3-7 Foundation Work for the Clear Water Reservoir

The bottom of the clear water reservoir will be about 3.5 m lower than the existing ground level. Boring tests have shown that about 4.5m will be the depth at which the over layer of clay gives way to the sand layer below.

Using this sand as a foundation, a 1 meter layer of sand shall be laid on top.

For this site, excavation may be done without filling first. Sheet piling shall be driven prior to beginning work to prevent crumbling of the faces of cuts due to water draining in, or due to excavation.

5-3-8 Foundation Work for the Elevated Water Tank, Receiving Well, Sedimentation Basin and Rapid Sand Filter

Because these structures are heavy, care shall be taken that the concrete piling is driven deep enough to adequately reach the underlayer of sandy soil.

5-3-9 Laying the Distribution Mains

5-3-9-1 Location of Underground Installations

Before excavating, investigation of the area shall be made with concerned parties present, even though locations of the installations may be indicated on maps.

5-3-9-2 Paved Roads

Cutting for mains on paved roads shall include 1/3 the width of the main ditch additional on each side, or a total of 2/3 more than the actual main channel. The Contractor shall pay the concerned agency the cost of restoring paved road crossings wherever dug up

5-3-9-3 Covering Up Mains

The earth covering is specified on drawings and is determined for each pipe diameter.

5-3-9-4 Aqueducts

The section of the main crossing the Ping River parallel to the highway shall have a special viaduct. Concrete pillars shall be erected, aligned with the span of the piers of the adjacent road bridge, and on the up-river side of this bridge.

Chapter 6 Contract Documents for the Works

6-1 Tender Documents

6-2 Instructions to Tenderers

6-3 Conditions of Contract

6-4 Specifications

6-5 Quantity Lists

6-6 Bid Schedule

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6-8 Investigating and Evaluating the Qualifications of Prospective Tenderers

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6-10 The Tender

6-11 Report of Evaluation of Tender Documents

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6-13 Concluding the Contract

Chapter 6 CONTRACT DOCUMENTS FOR THE WORKS

6-1 Tender Documents

Tendering will be based on the following documents:

1 Invitation to Tender
2 Instructions to Tenderers
3 Tender
4 Tender Guarantee
5 Performance Bond
6 Agreement
7 Specifications
   (a) General Conditions
   (b) Special Conditions
8 Bill of Quantities
9 Forms for Material Costs
10 Drawings

The above documents shall be made for the purpose of international tender.

6-2 Instructions to Tenderers

Instructions to Tenderers are instructions and admonitions concerning tender procedures. The major matters referred to are:

1 Instructions on filling in Tender Documents
2 Submitting of program for construction work
3 Submitting the Tender Guarantee
4 Submitting data on Foreign Currency requirements
5 Instructions on submission of substitute plans
6 Confidential treatment of Documants
7 Non-responsibility of PWD for expenses incurred by Tenderer
8 Method of submitting Tenders
9 Retraction of Tender
10 Return of Documents

6-3 Conditions of Contract

The Conditions of Contract shall be those included as a part of the Contract Document for this Project. However, any Conditions of Contract which are in standard use by the PWD may supplement those contained in the Contract Document, and the Contractor must familiarize himself with them.

6-4 Specifications

The Specifications include, in addition to detailed specifications as to workmanship, materials, temporary works, such other documents as Description of the Works and List of Drawings.

The standards described in the Specifications and the detailed Quantity Lists are mainly ISO Standards, or equal standards, because international bids are to be held.

Dimensions, weights, and strengths are expressed in the metric system presently in use by the Government of Thailand.

6-5 Quantity Lists

All sums contained in the detailed Quantity Lists are expressed in Baht.

All quantities are estimates, and the completed quantity of work shall be measured by methods described in the Quantity Lists, so that quantities may be determined for the purpose of payment.

The unit prices entered by the Contractor in the Quantity Lists shall be multiplied by the measured determined quantities to calculate the amount of payments.

6-6 Bid Schedule

For this project, international bidding shall be conducted, the best qualified participants being selected and designated approved tenderers.

The following procedures shall result in the selection of a Contractor to carry out the Works, and the concluding of a Contract between the Contractor and the PWD.

i Advertising
ii Investigating and Evaluating Qualifications of Prospective Tenderers
iii Selecting Tenderers
iv The Tender
v Evaluation of Tender Documents

vi Selecting the Successful Tenderer

vii Concluding the Contract

6-7 Advertising

Advertising informs contracting firms of an invitation to tender. The advertisement should include the name of the PWD, name of the consultant, type of construction, details, and location, the documents required to be submitted, and the closing date to submit a tender. Included among documents required to be submitted by a prospective tenderer should be a form to be filled in, listing name of firm, location, paid-in capitalization, bank, recent projects which the firm completed or participated in and the number of its technical and office staff.

The closing tender date should be 60 to 90 days after advertising.

6-8 Investigating and Evaluating the Qualifications of Prospective Tenderers

The documents submitted by prospective tenderers should be investigated and an evaluation made of their suitability to do the work. The work may best be done by a collaboration between the authority ordering the work, and the Consultant. Thus, prospective contractors should be selected who are well qualified to do the work. Thirty to forty days is necessary for this selection.

6-9 Selecting Tenderers

A Central Tender Board shall hold meetings and discussions to select eight to ten firms for designation as approved tenderers. This work may require about thirty days.

6-10 The Tender

Selected tenderers are sent a set of documents for the tender. Closing date for submitting a tender should be about 30 days after receiving these documents.

6-11 Report of Evaluation of Tender Documents

The Consultant shall investigate each set of tender documents submitted and make a report of his observations to the PWD. Important points in the Consultant's report shall be the completeness of the tender documents, the prices set forth in the tender, methods of effecting the work, time required to complete the work, capacity of the tenderer, and the guarantee. The evaluation of the various tenderers' documents will require about 30 days.

6-12 Selecting the Successful Tenderer

The PWD shall add its opinions to the report of evaluation of the tender documents, and forward all material to the Tender Board. The Tender Board shall contact those tenderers who are rated highest and conduct negotiations. Finally, the Board shall determine the successful tenderer and the tender price. This process shall require about 30 days.

6-13 Concluding the Contract

Signing of the Contract shall take place between the PWD and the successful tenderer after his selection.