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10-2-A. STUDY ON INTAKE STRUCTURE

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CONTENT

1. INTRODUCTION

2. DESIGN

2.1. Temporary Bridge

2.2. Intake Structure

(1) Pile Foundation Method

(2) Concrete with Steel-form Method

3. CONSTRUCTION

3.1. Temporary Bridge

3.2. Pile Foundation

3.3. Caisson with Steel-form

4. DESIGN OF STEEL BOX

4.1. Design Condition

4.2. Design of Bottom

4.3. Design of Wall

5. CONSTRUCTION SCHEDULE

6. BILL OF QUANTITY AND CONSTRUCTION COST

6.1. Approximate Bill of Quantity

6.2. Construction Cost

1. INTRODUCTION

The study is on the conceptual design and erection works of the intake in Dok Krai Reservoir.
The following two alternatives have been raised and studied.

1) Pile Fountation
2) Steel and Concrete Caisson

The both of the above follows the order of erection, that is, constructing a temporary bridge at first and using it to build the intake structure.

Hencefore, the design, erection, construction term and construction cost will be explained.

2. DESIGN

2.1. Temporary Bridge

(1) Dimension

Fig. 2-1 Section of Temporary Bridge

Fig.2-1. Sectional View of Temporary Bridge

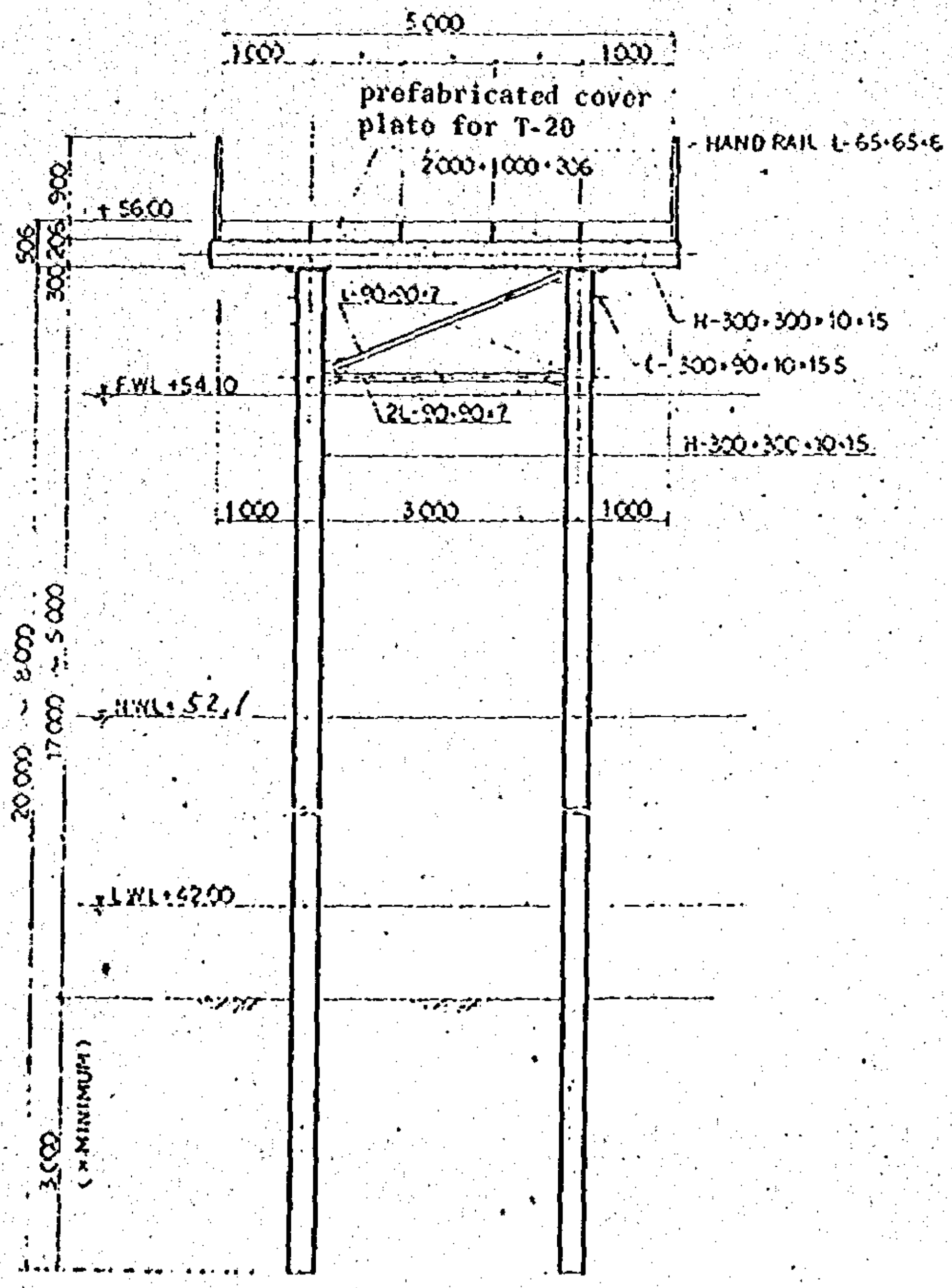
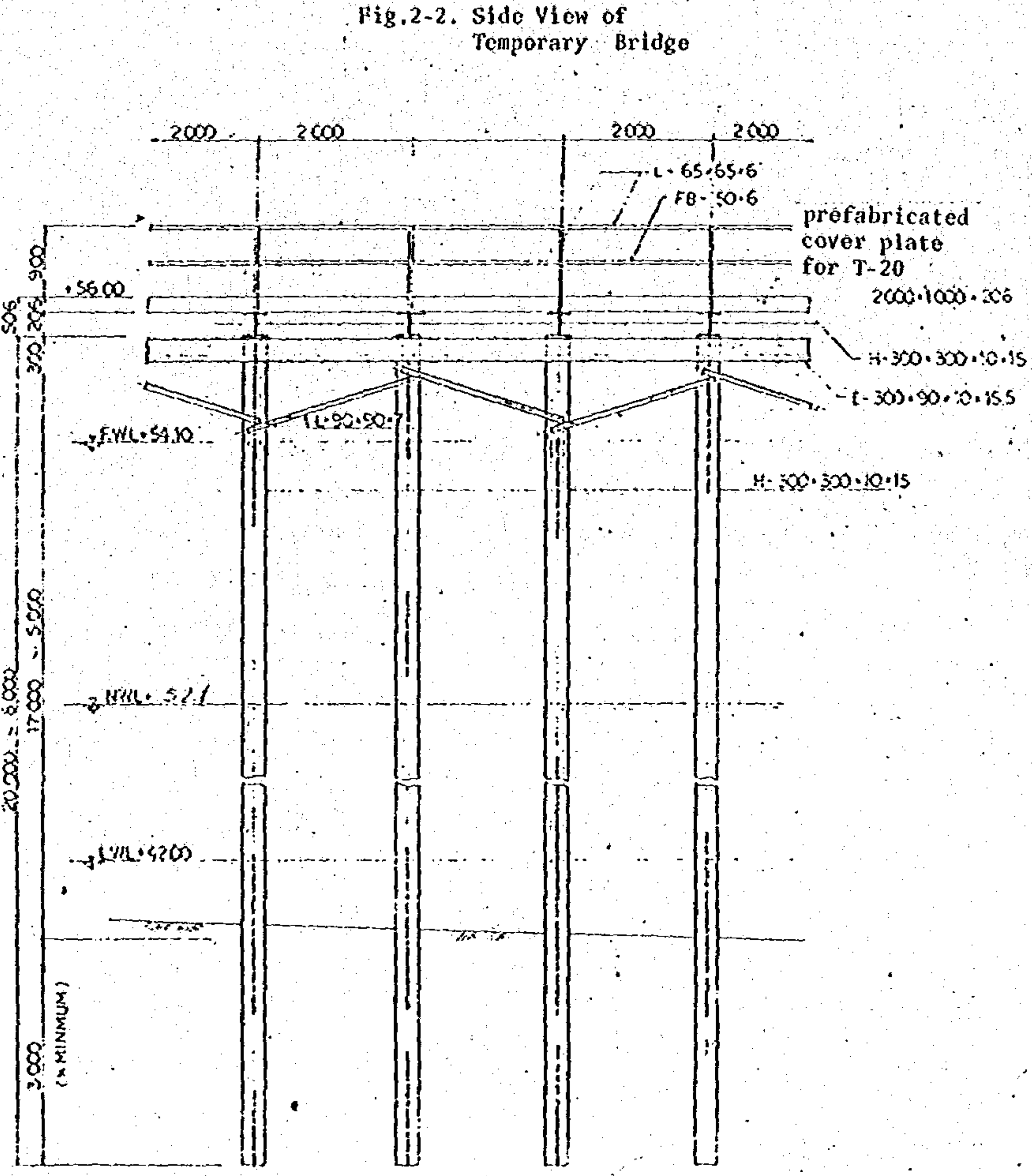


Fig. 2-2 Sido View of Temporary Bridgo



(2) Design Conditions

Load

a) Live Load
JIS T-20 (20 ton truck)

b) Coefficient of Impact

$$i = \frac{20}{50 + l} = \frac{20}{50 + 3} = 0.38$$

i : coeff. of impact
l : Span

c) Dead Load
Steel cover plates 2,000 x 1,000 x 206 for T-20 are used. The dead load w is : $w = 180\ kg/m^2$

Allowable Stress

Because of the temporary role of bridge, 50% increase is tolerated.

(3) Calculation of Slab Plate

As the plates for T-20 are used no calculation is made.

(4) Calculation of Beams with 2.0m interval (Fig.2-1, 2-2)
Assumed as a simple beam of 3.0 m span.

Bending Moment

* Live Load : the rear wheel load 8^t

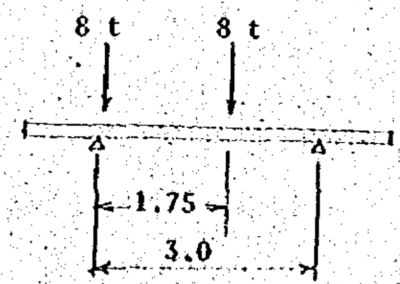
$$Mt = \frac{1}{4} pl = \frac{1}{4} \times 8 \times 3 = 6.0^{\ t\text{-}m}$$

* Impact : $Mi = 6.0 \times 0.38 = 2.28^{\ t\text{-}m}$
* Dead Weight : slab plate $w = 180\ kg/cm^2$ width = 2 m,
beam of H-formed steel 100 kg/m
uniform load = $0.18 \times 2 + 0.1 = 0.46$ t/m
$Md = \frac{1}{8} \times 0.46 \times 3^2 = 0.52^{\ t\text{-}m}$

Total Bending Moment = $6.0 + 2.28 + 0.52 = 8.8^{\ t\text{-}m}$

Shearing Force

* Live Load : $St = 8.0 \times (1 + \frac{1.75}{3.0}) = 12.67^{t}$ as shown below.

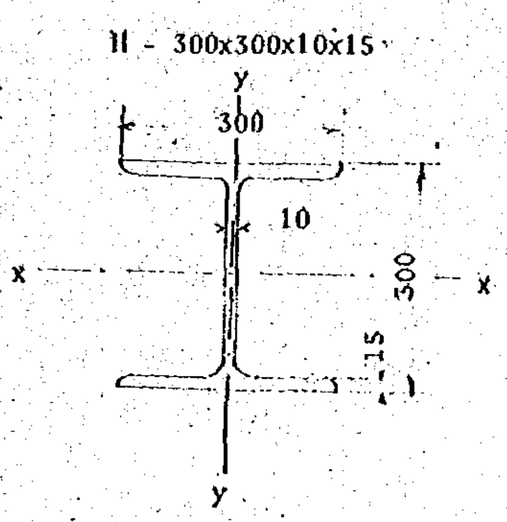


* Impact : $Si = 12.67 \times 0.38 = 4.81^{t}$
* Dead Weight : $Sd = \frac{1}{2} \times 0.46 \times 3.0 = 0.69^{t}$

Total Shearing Force = $12.67 + 4.81 + 0.69 = 18.17^{t}$

Stress

The beam is a H shaped formed steel of H-300x300x10x15 as shown below.



Area: $A = 119.8\ cm^2$ Section Modulus : $Zx = 1,360\ cm^3$

Geometrical Moment of Inertia : $Ix = 20,400\ cm^4$

* bending stress $\sigma = \pm \frac{M}{Zx} = \frac{8.8 \times 10^5}{1,360} = \pm 647\ kg/cm^2$

allowable stress $\sigma_{ba} = (1,300 - 0.6\ (\frac{l}{b})^2) \times \alpha$

$= (1,300 - 0.6\ (\frac{3.0}{0.3})^2 \times 1.3 = 1,612\ kg/cm^2$

bending stress $<$ allowable stress $\alpha = 1.3$

* Shearing stress $I = \dfrac{S}{(D - 2t_2)t_1}$

$$= \frac{18{,}170}{(30-2x1.5)x1.0} = 673 \text{ kg/cm}^2$$

t_1 : thickness of web

t_2 : thickness of flange

allowable stress t_a = 800 x 1.3 = 1,040 kg/cm^2

Shearing Stress < allowable stress

Deflection

$$P = \frac{Pl^3}{48EI} + \frac{5wl^4}{384EI}$$

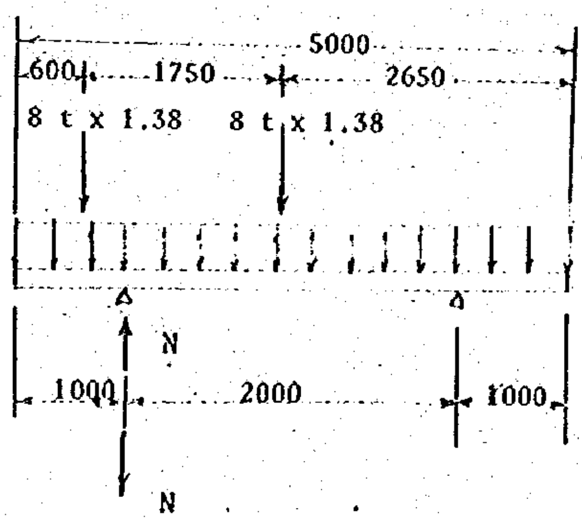
$$= \frac{11.04x10^3x300^3}{48x2.1x10^6x20{,}400} + \frac{5x4.6x300^4}{384x2.1x10^6x20{,}400}$$

$$= 0.145 + 0.011 = 0.156 \text{ cm}$$

S/l = 0.156/300 = 1/1,900 is far less than allowable ratio of deflection/span : 1/400.

(5) Calculation of Column (Pile) with 2m interval

The figure below shows the two rear wheels rest on the beam, causing the maximum force acting on the column (pile).

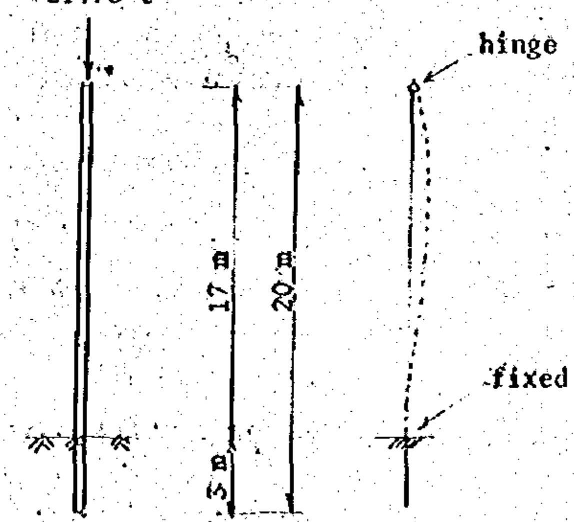


$$N = \frac{1}{3.0} \left(5 \times 0.46 \times \frac{3.0}{2} + 8 \times 1.38 \times (3.4+1.65) \right) = 19.73^{\,t}$$

Column's dead weight : $0.1 \times 20 = 2^{\,t}$

The axial compressive force acting on the column (pile) is 21.73 t.

The column is also H-300 x 300 x 10 x 5 and is assumed in the condition shown below.



For calculation of the allowable stress, 70% of 20 m is taken into account.

axial compressive stress : $21.730/119.8 = 181\ kg/cm^2$

allowable stress : $(7,200,000/(\frac{l}{iy})^2) \times 1.3$

$$= (7,200,000/(\frac{1,400}{7.51})^2) \times 1.3 = 269\ kg/cm^2$$

iy : radius of gyration on y axis

(6) Bearing Strength of Ground

Disregarding the impact, and considering the pile's own weight the axial force becomes :

$$N = \frac{1}{3.0} (5 \times 0.46 \times \frac{3.0}{2} + 8 \times (3.4+1.65)) + 2.0 = 16.62^{\,t}$$

The assumed ground condition :

bearing strength (as blow count)	:	$n = 10$
embeeded length of pile	:	$l = 3.0$ m
bearing strength of embedded part	:	$\bar{n} = 10$ (average)
factor of safety	:	$\mu = 2$

The allowable strength of bearing is :

$$qa = \frac{1}{F}(30^{n} Ap + \frac{\bar{n} As}{5})$$

$$= \frac{1}{2}(30 \times 10 \times 0.3^{2} + \frac{10 \times 0.3 \times 4 \times 3.0}{5})$$

$$= 17.1^{t}$$

Ap : apparent sectional area of pile

As : peripheral area of embedded part

2.2. Intake Structure

Two alternatives, so-called the concrete caisson and the jacket, are now under comparison.
Here, for the jacket a pile foundation method and for the concrete caisson, a concrete with steel-form method, both using the temporary bridge, are discussed.

(1) Pile Foundation Method

Using the temporary bridge, the piles are driven to the pre-determined positions. Followed will be the erection of platform, placing the slabs of platform, installation of the casings and the erection of bridge from the intake to the shore.
Two problems are raised concerning the method.

* The horizontal displacement due to the horizontal force shall be controlled.

* The geological survey shows that the subsoil is hard rock-like one. Possibility of driving the piles must be checked.
For the first, the usage of diagonal piles can control the horizontal displacement, as shown on Table 2-1.
For the second, the newly developed "Gun Pile" seems to solve it.

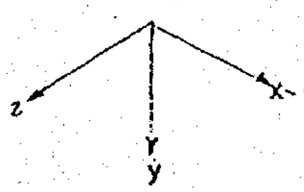
The diagonal piles are to be driven by the diesel hammer and the gun piples by the vibro-hammer. Though it is a kind of limiting requirement, the method will be able to ensure shortening of the construction period.

a) Loads

Vertical Load : 2,630 ^{t}

Horizontal Load : Hx = 276 ^{t} Hz = 276 ^{t}
working independently

Direction of Load : see the figure below



The vertical load is the dead load, excluding that of casings which are supported by the ground.
The horizontal load consists of the inertia of dead weight and casings and the dynamic pressure of water.
Point of Action of Loads : the center of floor slab

b) Dimensions

The elevations are assumed:
Top of piles 54.5 m, ground surface 40.0 m, bottom of piles 35.0 m.
Consequently the length of piles is 19.5 m, 14.5 m of which is above the ground surface.

c) Compared Cases of Calculation

* Two size of pile : 609.6 mm dia. of 16.0 mm thickness and 812.8 mm dia. of 19.0 mm thickness
* Two numbers of pile : 20 no.s and 25 no.s
* Angle of diagonal pile : 0, 15, 18.4 ($\tan^{-1}$ 1/3), 20 deg.
* Earthquake : yes and no

d) Results of Calculation

The results of comparative analysis are shown on Table 2-1.

e) Recommendation

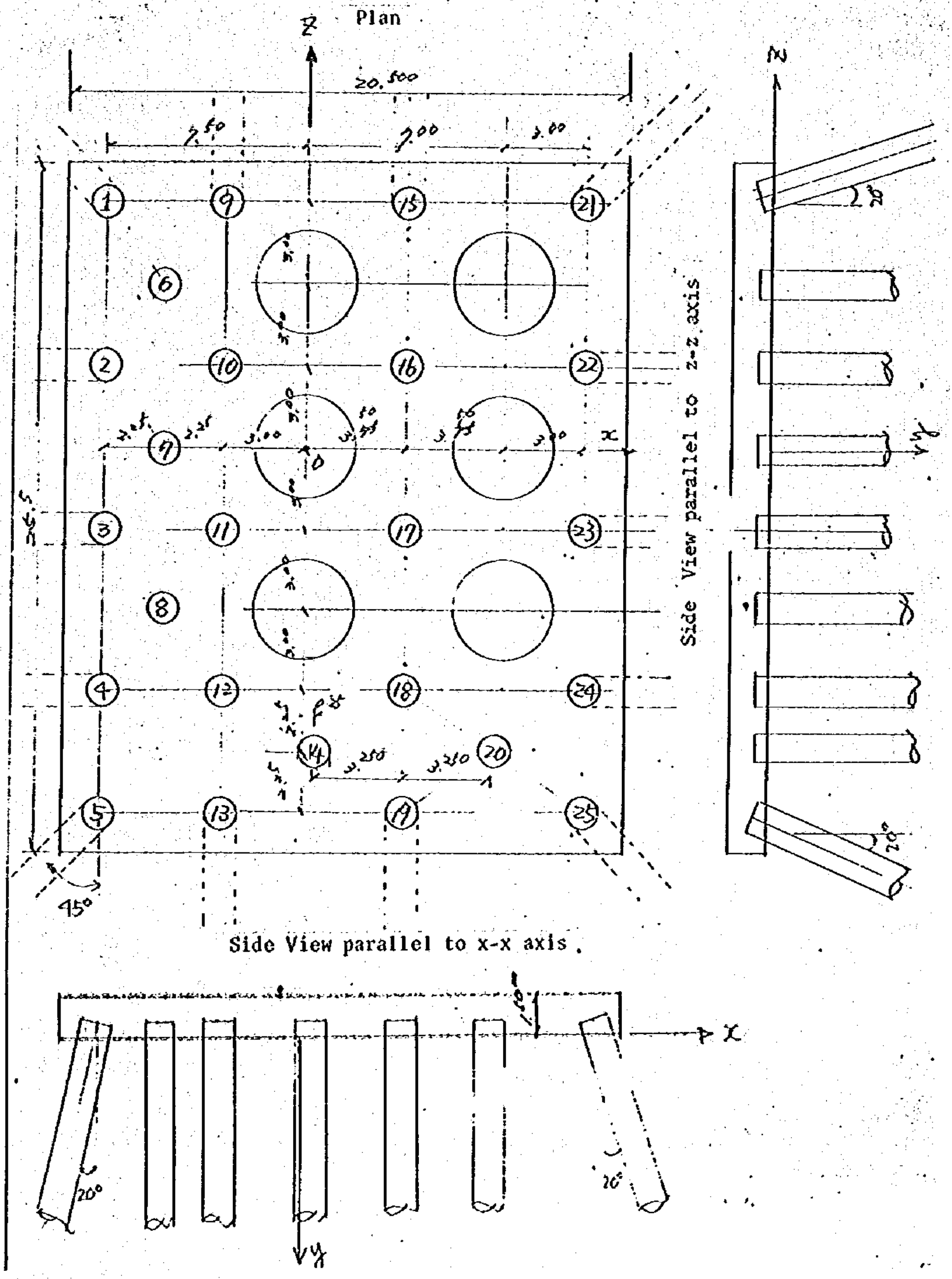
Considering the horizontal displacement, stress in the piles, the most recommendable is the model 125s18, the layout of which is shown on Fig. 2-3.

Table 2-1. Comparative Analysis

	Model Number	225 V	225 S18	220 V	220 S20	220 S15	125 V	125 S18	120 V	120 S20	120 S15
Pile	pile dia. (mm) Thickness (mm)	812.8 x 19.0					609.6 x 16.0				
	number	25		20			25		20		
	slope of diagonals (deg)	0	18	0	20	15	0	18	0	20	15
Normal	horizontal displace. (cm)	0.05	-0.42	0.03	-0.35	-0.31	0.07	-0.81	0.04	-0.60	-0.65
	vertical displace. (cm)	0.21	0.24	0.26	0.29	0.28	0.34	0.39	0.41	0.46	0.44
	tensile force per pile (t)	151	170	156	180	169	151	177	156	185	174
	compressive f. per pile (t)	0	0	0	0	0	0	0	0	0	0
	stress (kg/cm^2)	321	471	331	464	437	509	761	525	746	712
Earthquake *	horizontal displace. (cm)	6.07 6.00	1.13 1.54	7.52 7.47	1.72 2.12	2.56 3.22	15.70 15.61	2.10 2.69	19.49 19.41	3.31 3.69	5.21 5.92
	vertical displace. (cm)	0.22 0.21	0.20 0.21	0.26 0.26	0.26 0.27	0.25 0.25	0.35 0.34	0.31 0.33	0.42 0.41	0.40 0.42	0.38 0.40
	tensile force per pile (t)	175 131	204 241	181 155	256 301	258 312	175 131	208 253	180 155	281 317	293 339
	compressive f. per pile (t)	0 0	7 43	0 0	0 51	0 45	0 0	16 57	0 0	5 50	24 74
	stress (kg/cm^2)	1480 1392	657 726	1718 1661	938 929	1100 1124	2895 2760	1040 1154	3382 3288	1551 1476	1884 1830

* The upper figures are for x-x axis and the lower for z-z axis.

Fig.2-3. Layout of Piles



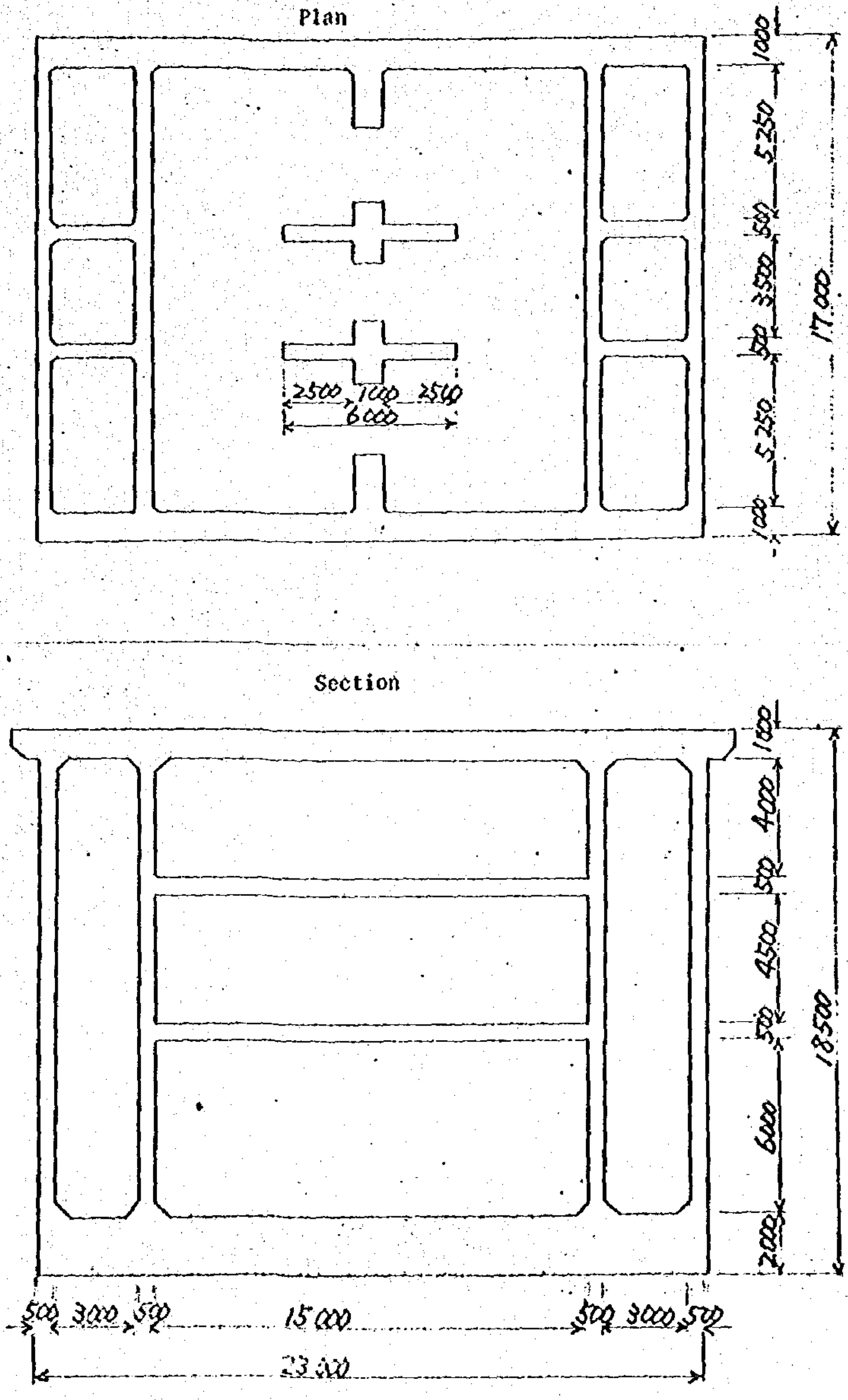
(2) Concrete with Steel-forms Method

The steel caisson mentioned here does not mean a caisson of steel made . but does a concrete caisson using steel forms to facilitate the construction works, as explained below:

* Because of the light weight, the temporary dock can be of smaller depth.
* Because of the same reason, smaller boats can make tugging of the caisson.
* Box-like steel forms can be fabricated in pieces at the factory and as they can replace a part of concrete form and stress-bearing reinforcement, the site works can be saved to shorten the construction period.

The basic structural design of concrete caisson is mostly followed as originally designed and a part of lower portion has been modified because of the change of construction method.
Checking of the steel boxes are made later in the construction planning.
Fig. 2-4 shows the plan and sections of intake structure.

Fig,2-4. Caisson with Steel-form



3. CONSTRUCTION

3.1. Temporary Bridge

The flow (order) of construction steps and the machine required for the steps are shown:

Flow of Steps	Machinery
Embankment of shore	bulldozer
↓ driving columns	truck crane vibro-hammer
↓ fixing longitudinal beams	
↓ fixing diagonal members	truck crane welder
↓ fixing lateral beams	
↓ fixing floor slab	
↓ installation of handrail	
↓	

3.2. Pile Foundation

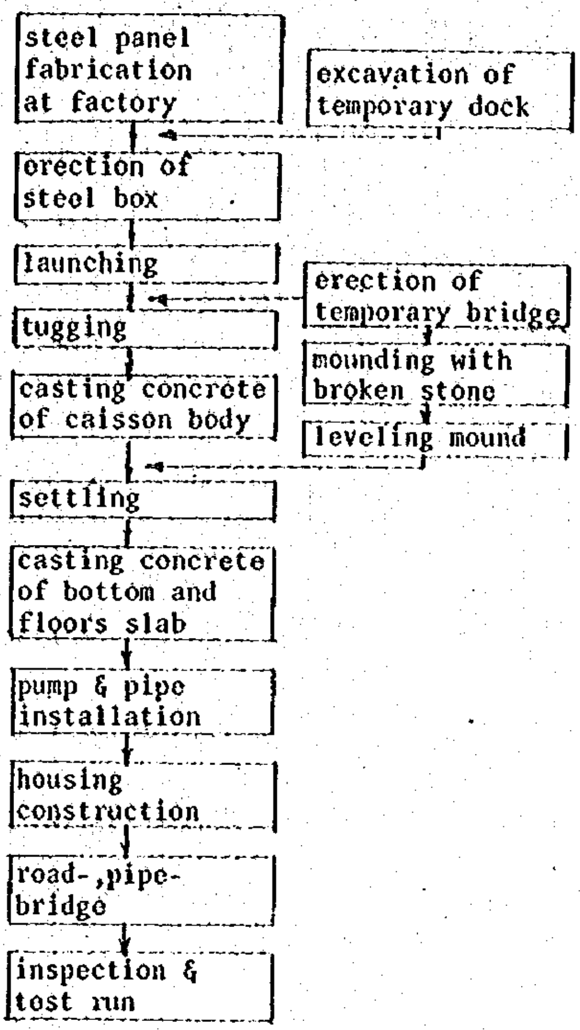
The major and minor flows and the machinery are shown on the next page:

Main Flow	Flow Details	Machinery	Material
temporary bridge			
intake structure	*preparatory works	*set of pile machine	*steel pile 609.6mm dia. x 19.5 m x 25 no.s
	*pile machine set up	*truck crane for transporting	
driving pile	*lead frame		
	*pile driving	*welder	
casting slab	*pile head treatment	*concrete plant	*concrete 1153 m^3
	*bracing of piles		
casing erection	*form of slab concrete	*truck mixer	*steel bar for reinforce.
	*fixing reinforce. bar	*concrete pump car	*concrete form
pump & pipe installation	*metal works	*machines for steelworks	
building house	*cutout works		*steel
	*casting concrete		
road-,pipe-bridge erection	*pier		
	*upper structure		
inspection & test run	*pipeline		
	*instrumentation &electrical equipments		

3.3. Caisson with Steel-form

The flow, machines and materials are shown on the next page:

Flow of steps	Machines, plants	Materials



Machines, plants

*temporary bridge (refer 3.1.)

*temporary dock
- power shovel
- well point

*steel box
- truck crane
- welder

*mounding
- cram shell
- diver

*tugging
- winch
- tug boat

*concrete casting
- concrete plant
- truck mixer
- concrete pump car

*settling
- submersible pump
- winch
- tug boat
- diver

Materials

*steel
- H-700x300
- 20 ton
- I-250x125 -350x150
- 8 ton
- L-200x90 -250x90
- 18 ton
- Plates
- 62 ton

*broken stone
- 2000 m^3

*concrete

The steps of construction are as follows:

(1) Temporary Dock

The depth of temporary dock is decided on the draft, when the steel box is taken out to water. To stabilize the box, a part of the bottom is filled with concrete in the temporary dock. For the existing ground level of 54.0 m and normal water level 50.6 m, the bottom of temporary dock will be 47.5 m as the required draft is 2.2 m. The temporary dock can be excavated without sheet piles. However, the ground water shall be lifted by the well-point method until the completion of steel box.

While the excavation of temporary dock is under way, fabrication of the parts of steel box in the factory and erection of the temporary bridge can be promoted simaltaneously.

(2) Erection of Temporary Bridge

In case of the caisson with steel-form method, the length of bridge will be 30 m longer than in the case of pile foundation.

The concept is that the foundation of caisson is better be made of mounding with broken stone instead of dredging which requires the dredger boat of remarkable size. And it needs the caisson's location be about 30 m further ashore than that of pile foundation. When the steel box is taken to water from the dock, the temporary bridge will have to stand against the horizontal force pulling the box.

(3) Launching of Steel Box

After completing the erection of steel box, the well point method is stopped and the reservoirs water will be introduced to the dock as the earth is dug away.

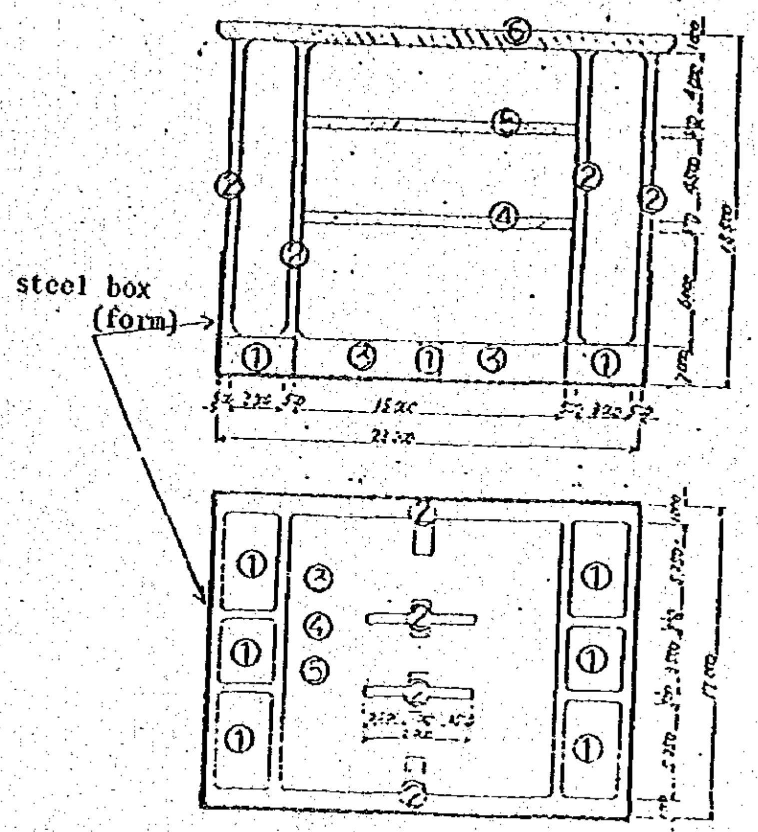
(4) Tugging of Steel Box

The steel box is pulled out of the temporary box by winching from the temporary bridge and tugged, assisted by small boats.

The temporary dock is backfilled then.

(5) Building of Caisson's Body

Concrete is placed into the steel box being moored to the temporary bridge. The order of concrete placing is as shown on the next page with the figures in circles..



Here, to stabilize the steel box 300 m^3 concrete of (1) of the figure is cast in the temporary dock.

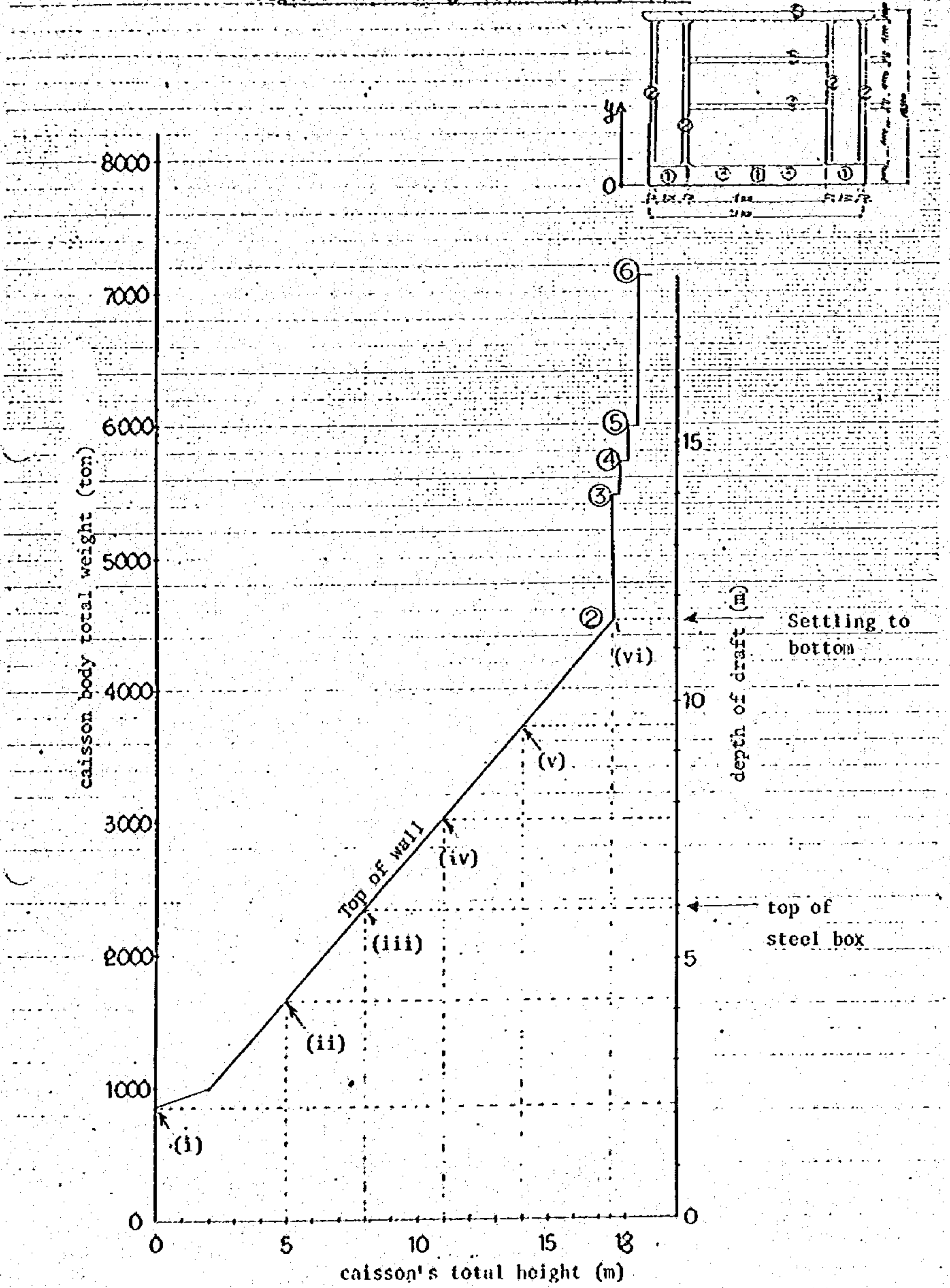
<u>Progress of Caisson and Draft</u>

Shown on Fig. 3-1 is the increase of caisson's weight versus the change of draft, following the progress of caisson by casting concrete.

(i) When the steel box is pull out of the dock and moored to the bridge.
(1) part 300 m^3 already cast and 2.2 m draft.

(ii) When 3 m height of (2) part (wall) is cast. With 2 m thick base slab, the total height y is now 5 m. 4.3 m draft

Fig.3-1 Total height vs. weight & draft



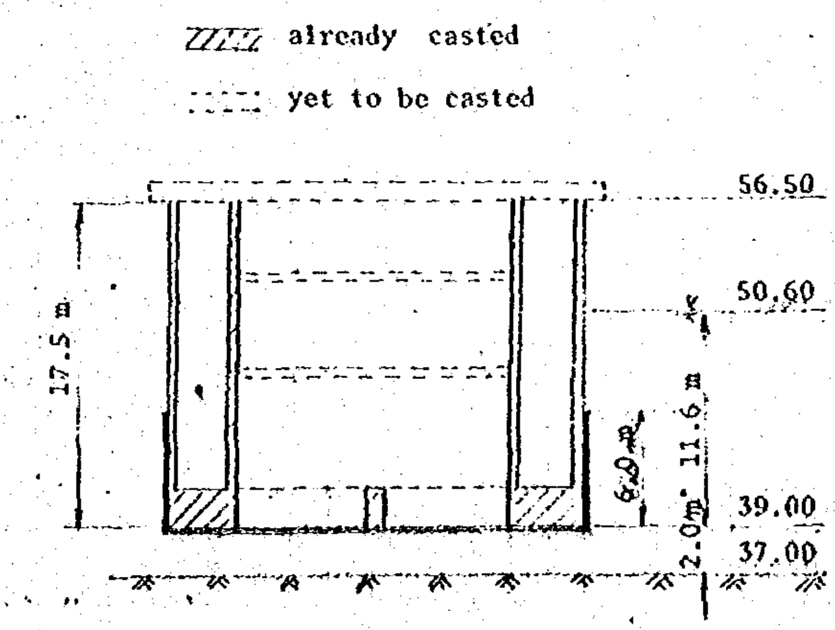
(iii) When 3 m more height of (2) part is cast, making the total height y = 8 m.
The draft reaches 6.0 m and comes to the top of steel box.

(iv) When 3 m more height of (2) part is cast, making the total height height y = 11 m. 7.8 m draft

(v) When 3 m more height of (2) part is cast, making the total height y = 14 m 9.5 m draft

(vi) When 3.5 m more height of (2) part is cast, making the total height y = 17.5 m and completing the wall. 11.6 m draft
Then, the caisson is settled at the bottom. The bottom slab (3), floor slabs (4), (5), (6), will be cast after the settling to complete the intake structure.

(6) Settling of Caisson
As explained before, the draft of caisson is 11.6 m when completed. As the ground surface (bottom surface of the reservoir) is 37.0 m, the caisson's position is like the picture below.



To settle the caisson, about 800 m^3 water must be taken in and six compartments at the both side, having 1,300 m^3 space, can be used for both introducing water and controling the list of caisson.

4. DESIGN OF STEEL BOX

4.1. Design Conditions

(1) Load

As for the bottom plate, the water pressure after settling i.e. $qo = 13.6\ t/m^2 \doteq 14\ t/m^2$ is considered and as for the wall plate, the pressure during construction i.e. $qo = 2.5\ t/m^2$ is considered.

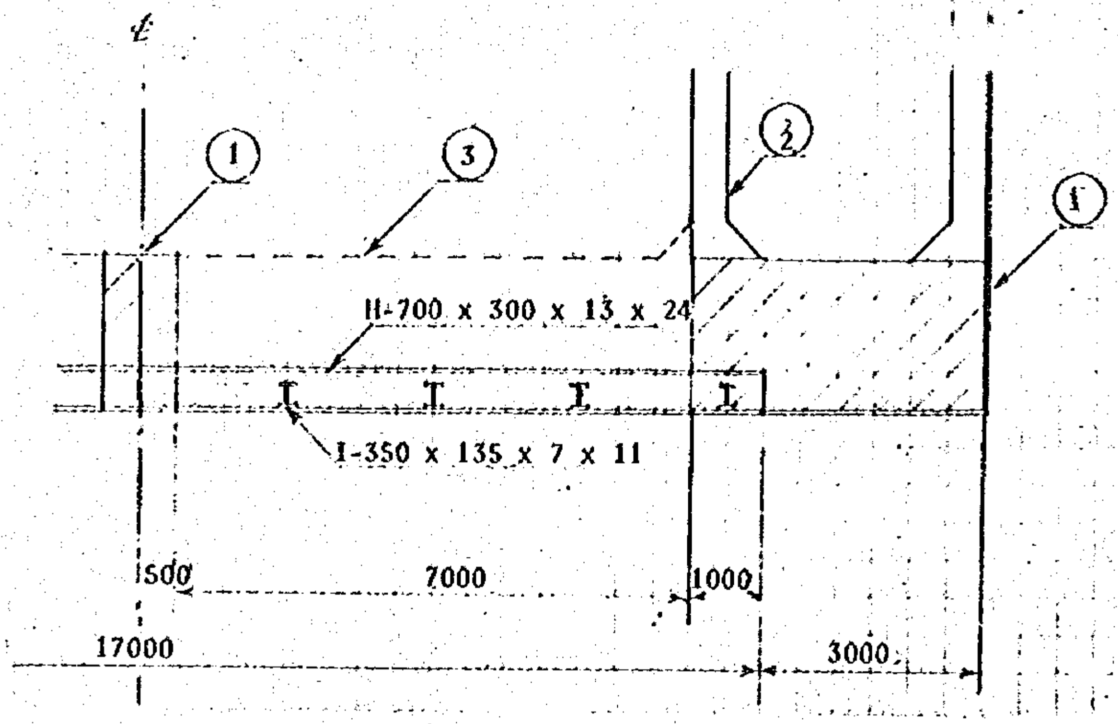
(2) Allowable stress

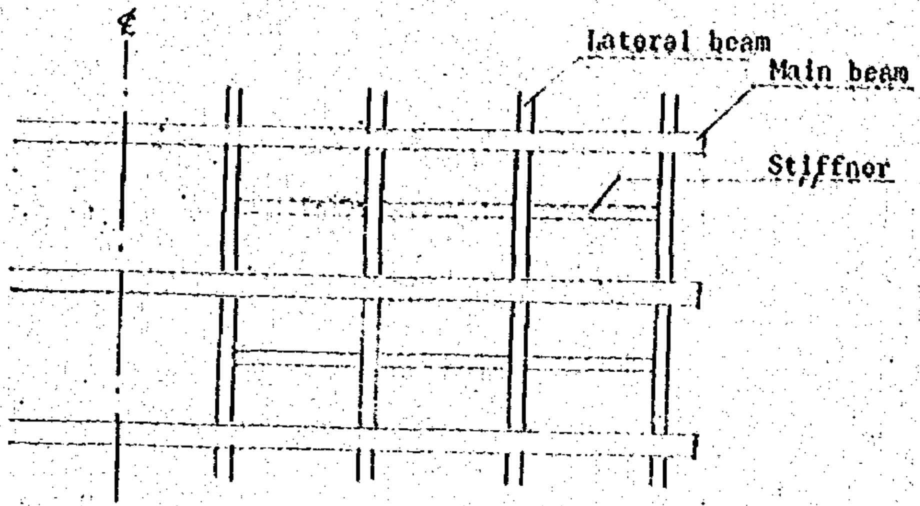
As the loads can be taken as short-term load, the allowable stress is increased by 50% of the permanent one.

$\sigma a = 1{,}400\ kg/cm^2 \times 1.5 = 2{,}100\ kg/cm^2$

4.2. Design of Bottom (see attached drawing No.5)

The bottom plate is under the condition shown on the picture below. Part 1 was cast in the dock and part 2 was cast at first in mooring. Then, the steel bottom is subjected to the water pressure working upwards.





(1) Main beam

A 7 m beam of the both ends fixed. Subject to 2m wide strip of the water pressure 14 t/m^2 as an uniform load.

$M = \frac{1}{12} \times 2 \times 14 \times 7^2 = 115 \text{ t-m} = 11,500,000$ kg-cm

H-700 x 300 x 13 x 24 has the section modulus 5,760 cm^3

$\sigma = 11.5 \times 10^6/5,760 = 1,996 < 2,100 \text{ kg/cm}^2 = \sigma_a$

(2) Lateral beam

A 2 m beam of the both free ends, namely a simple beam is assumed.

$M = \frac{1}{8} \times 2 \times 14 \times 2^2 = 14 \text{ t-m} = 1,400,000$ kg-cm

With I-350 x 175 x 7 x 11 of the section modulus 775 cm^2, the stress is:

$$\sigma = 1,400,000/775 = 1,806 < 2,100 \text{ kg/cm}^2 = \sigma_a$$

(3) Stiffner

The stiffner with 9 mm skin plate makes a composite T beam, and it is under 1 m wide pressure.

$M = \frac{1}{8} \times 14 \times 2^2 = 7 \text{ t-m} = 700,000$ kg-cm

$\sigma = 700,000/360 = 1,944 < 2,100 \text{ kg/cm}^3 = \sigma_a$

360 cm^3 is the section modulus of only the stiffner, L-250 x 90 x 10 x 5.

4.3. Design of Wall (see attached drawing No.5)

The main beams and laterals are under the triangle water pressure.

(1) Main beam

I-250 x 125 x 5 x 8 are arranged with 4 m intervals. As the bottom part is embedded, the beam is a cantilever.

$M = \frac{P}{15} \times 2.5 \times 4 \times 2.5^2 = 4.2^{\text{ t-m}} = 420{,}000$ kg-cm

$\sigma = 420{,}000/285 = 1{,}473 < 2{,}100 \text{ kg/cm}^2 = \sigma_a$

(2) Lateral beam

The intervals are 1 m and the beam is a simple beam.

$M = \frac{1}{8} \times 2.5 \times 4^2 = 5^{\text{ t-m}} = 500{,}000 \text{ kg/cm}^2$

The beam, L-200 x 90 x 9 x 4 , has 260 cm^2 section modulus.

$\sigma = 500{,}000/260 = 1{,}923 < 2{,}100 \text{ kg/cm}^2 = \sigma_a$

5. CONSTRUCTION SCHEDULE

Two construction schedules, the temporary bridge Pile Foundation and the temporary bridge - Concrete Caisson with Steel - form, are shown on Table 5-1, 5-2.

Table S-1. Construction Schedule, Temporary Bridge - Pile Foundation

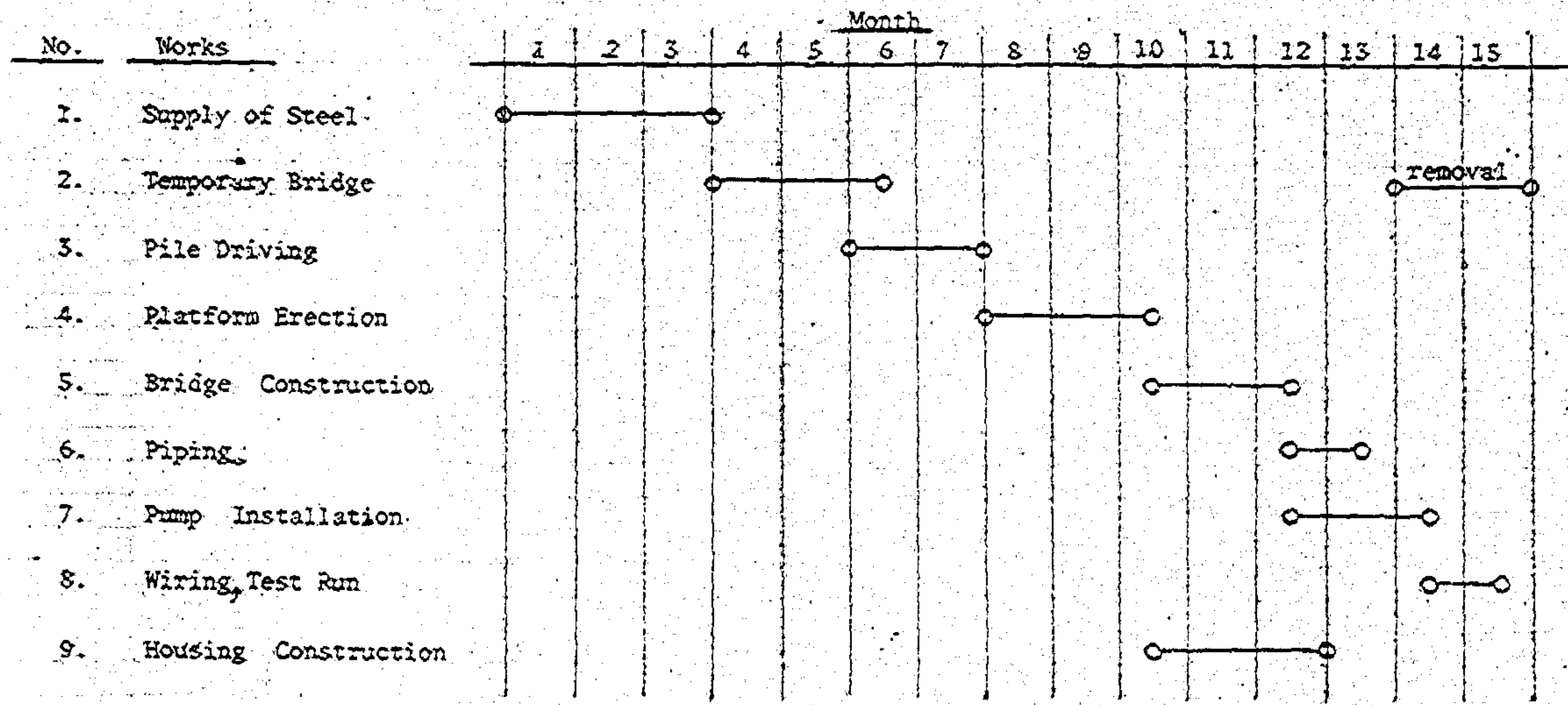
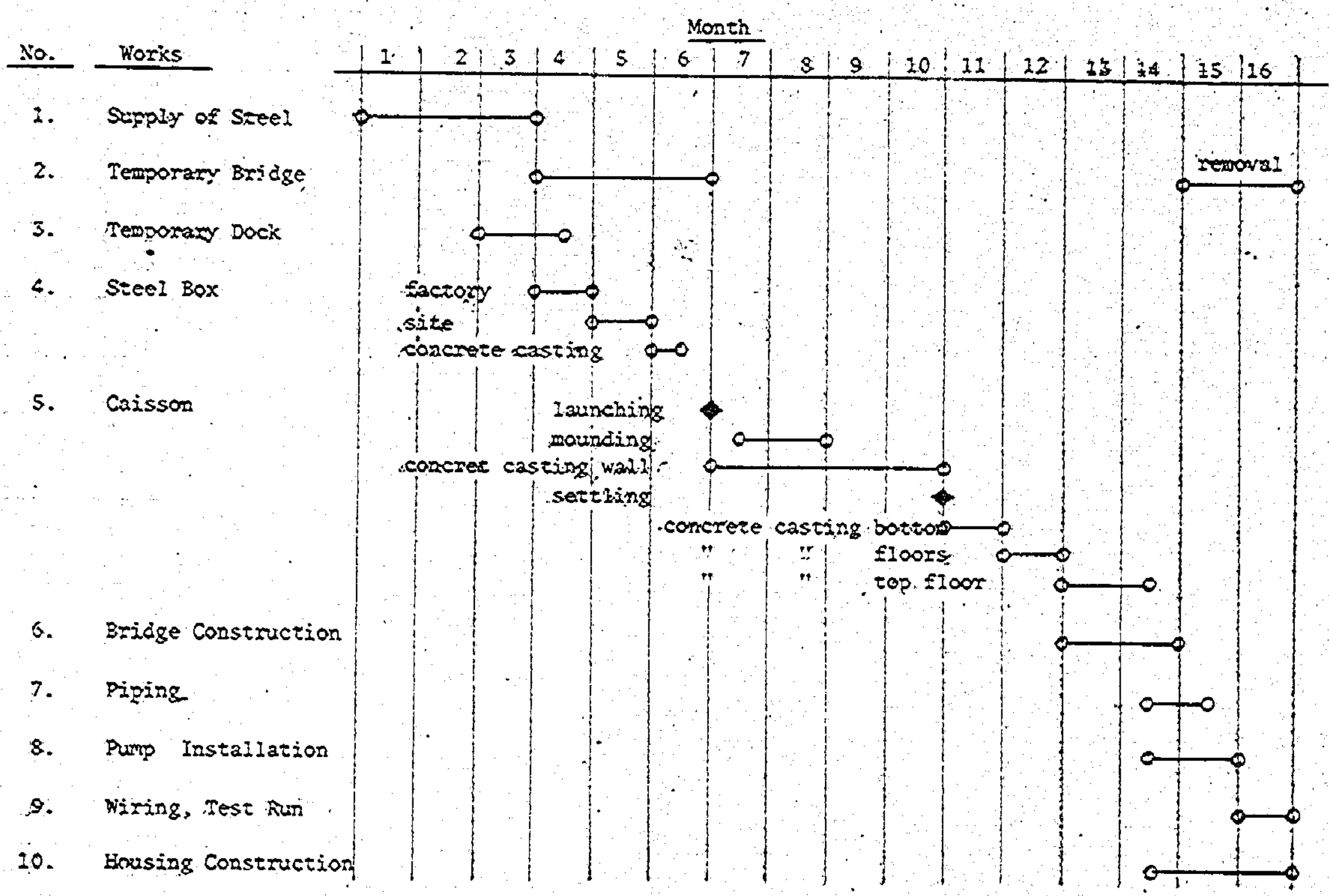


Table 5-2. Construction Schedule. Temporary Bridge - Concrete Caisson with steel - form



6. BILL OF QUANTITY AND CONSTRUCTION COST

6.1. Approximate Bill of Quantity

(1) Temporary Bridge

The table below covers the quantity of steel of the temporary bridge.

Item	Description	Length(m)	Unit Weight(kg/m)	Weight(t)
Slab plate	Steel for T-20			950 m^2
Pile	H-300x300x10x15	2,660.0	94	250.0
Lateral beam	"	499.2	94	46.9
Longitudinal beam	I-300x90x10x15.5	380.6	43.8	16.7
Bracing	L-90x90x7	1,470.1	9.6	14.1
Handrail	L-65x65x6	592.4	5.9	3.5
Handrail	FB-50x6	380.0	2.4	0.9
Total				332.1 ton

(2) Aqueduct Bridge

* Pile 508 mmØ x 14 mmt x 15 m L(ave.), 28 no.s, total weight : 71.8 ton
* Concrete 175 m^3
* Pipe bridge 1,350 mmØ x 210 m, total weight : 176 ton

(3) Pile Foundation

* Pile 609.6 mmØ x 16 mmt x 20 mL, 25 no.s, total weight : 117 ton
* Concrete Platform 20.5 m x 25.5 m x 1.5 m t = 754 m^3

	Casing (precast plate)	283 "
	filling pile	116 "
	Total	1,153 m^3

* Steel Bars assumed 70 kg reinforcement per m^3 concrete for platform and casing, etc. 80 ton
* Concrete form assumed 1 m^2 form per m^3 concrete as the sturcture is massive, 1,153 m^2
* Miscellaneous steel for spacing of piles and bracing, approx. 25 ton

(4) Concrete Caisson with Steel-form

* Steel Box

Item	Q'tity	Unit weight(kg)	Weight (ton)
Plate, 9 t	871 m^2	70.65	61.6
H-700x300	102 m	185.0	18.9
I-350x175	90 m	49.6	4.5
I-250x125	132 m	25.7	3.4
L-250x90	228 m	29.4	6.7
L-200x90	480 m	23.3	11.2
Total			106.3 ton

* Temporary Dock

  The earth volume, excavating and backfilling including it for launching, is approximately 16,000 m^3.

  The well point, 170 m long, shall be worked for 4 months.

* Mounding

  The broken stone for mounding is 2,000 m^3 and the leveling area 650 m^2.

* Concrete Works of Caisson

  Concrete 2,920 m^3, steel bar 150 t, concrete form 4,720 m^2 and timbering for floors 4,570 m^3 (void)

6.2. Construction Cost

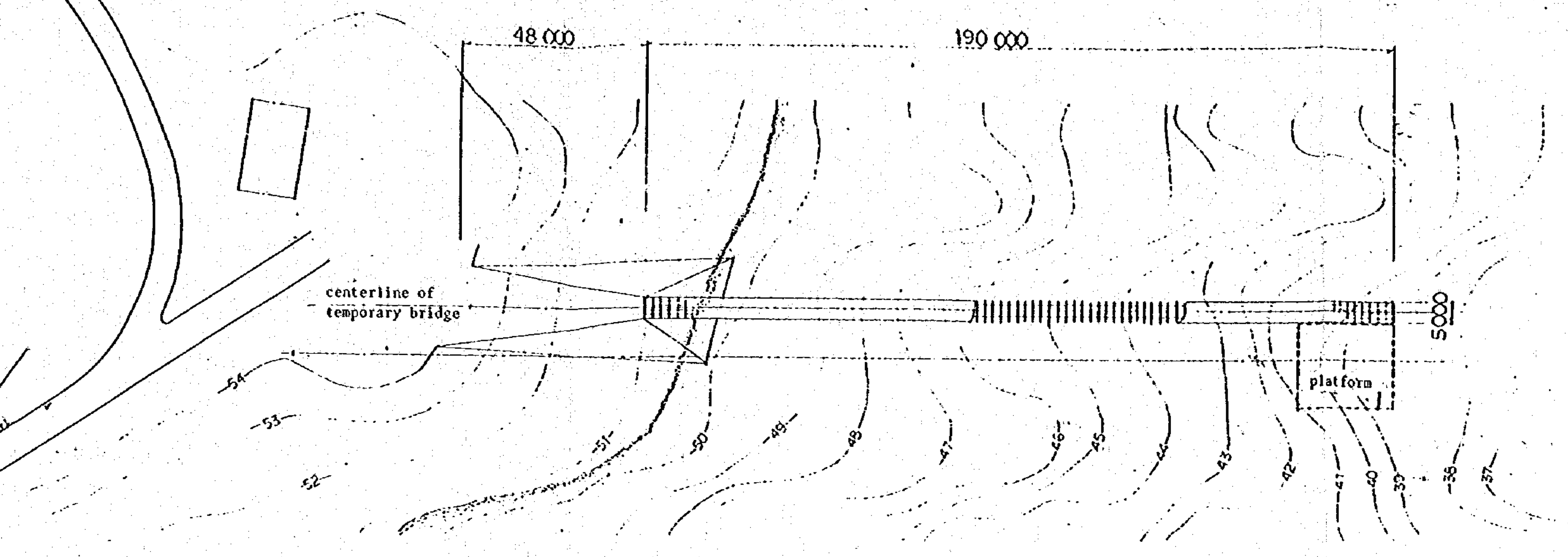
With the preceeding bill of quantity and assumed construction unit cost , the following table shows the construction cost.

Table 5-3 Construction Cost (million Yen)

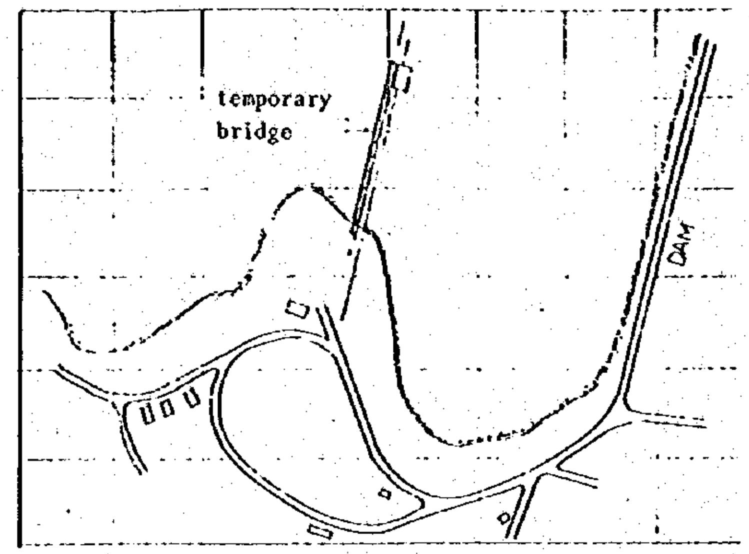
Pile Foundation Method: Works	Cost	Caisson with Steel-form Method: Works	Cost
Temporary bridge	217.5	temporary bridge	252.0
Aqueduct	313.2	Aqueduct	350.9
Upper structure	232.3	upper structure	269.3
Lower structure	80.9	lower structure	81.6
Platform	227.7	Caisson	337.9
Total	758.4		940.8

The cost does not include the housing and the pumps.

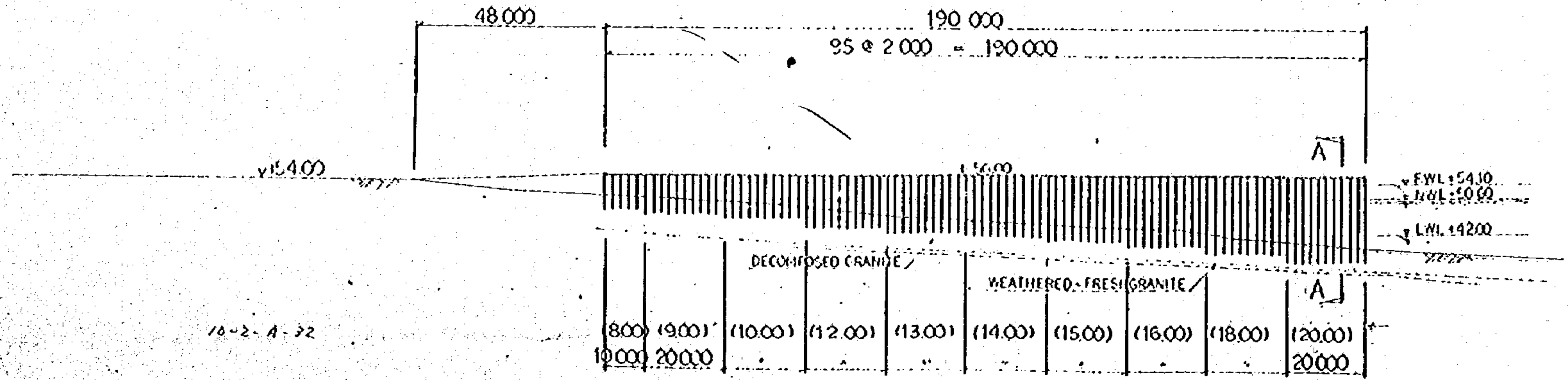
OVERALL LAYOUT S-1/1000



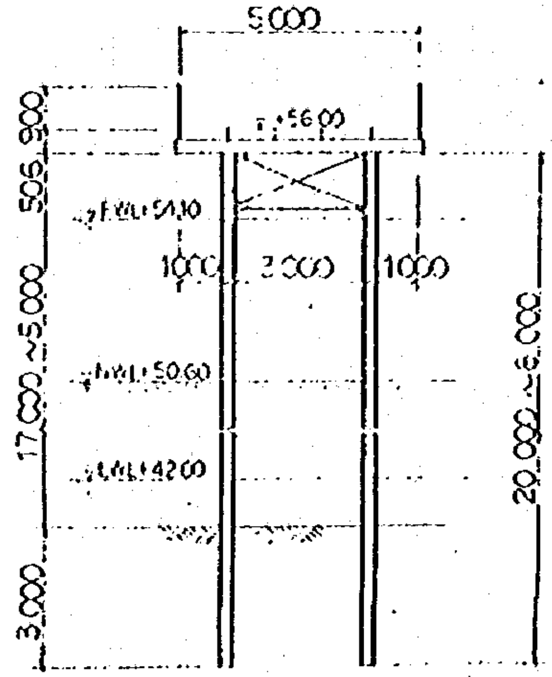
KEY PLAN



SIDE VIEW : S-1/1000



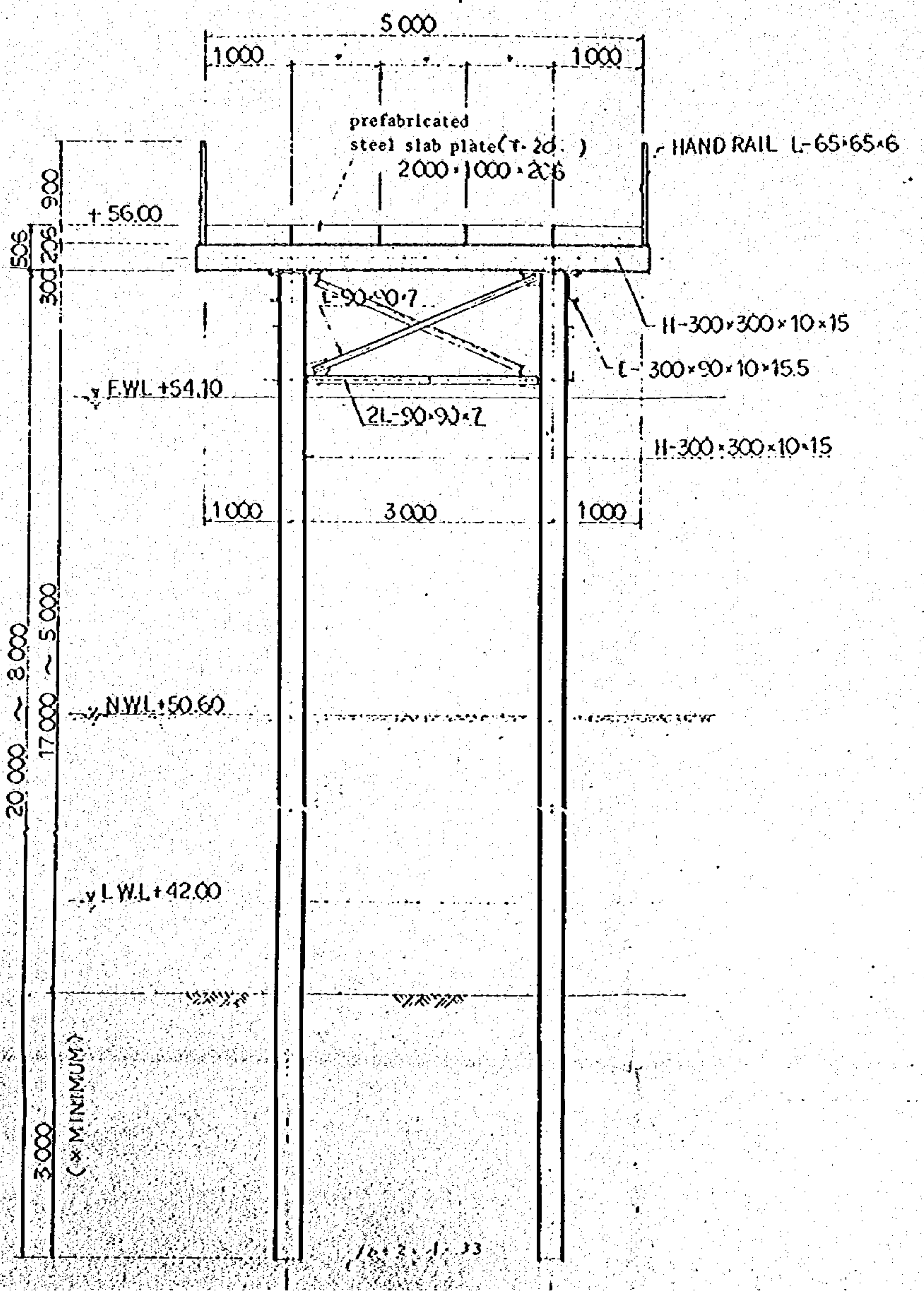
SECTION (A – A) S-1/125



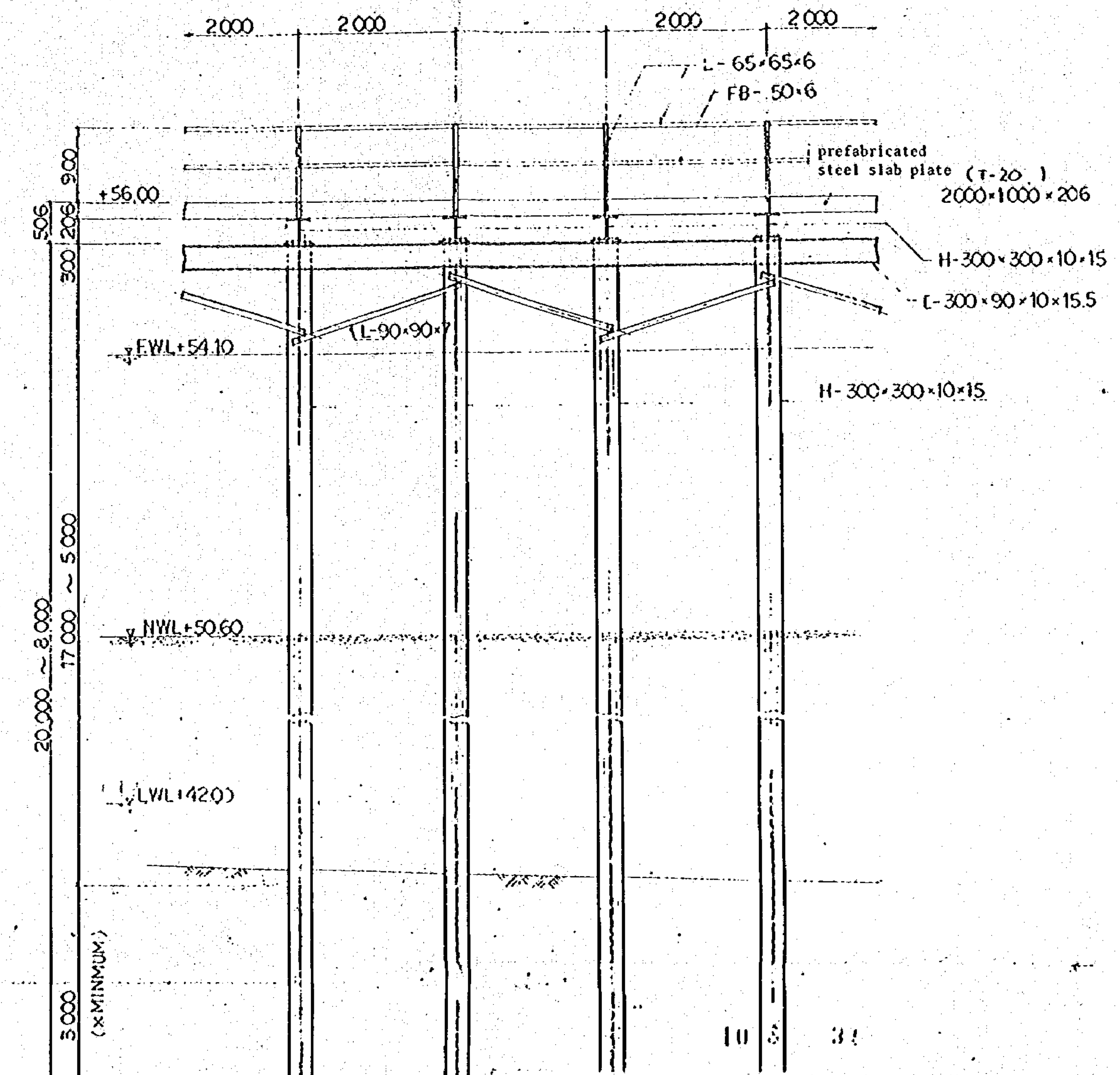
※ ( ) length of pile, H-formed steel

10-2-A-33	DWG NO. 1	TEMPORARY BRIDGE (1) GENERAL

LATERAL SECTION S.1:50
(A—A)



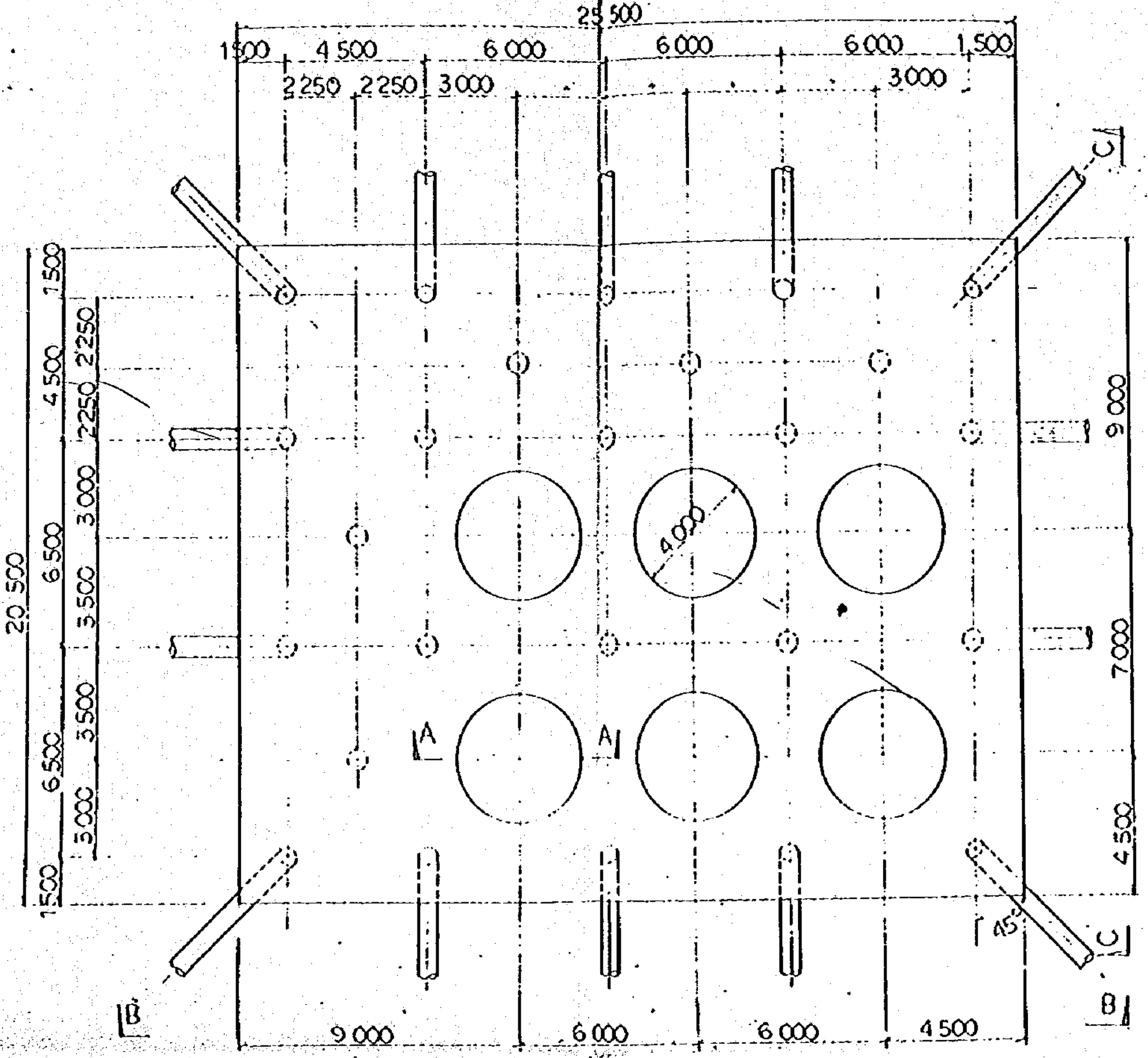
LONGITUDINAL SECTION S.1:50



10-2-A-3

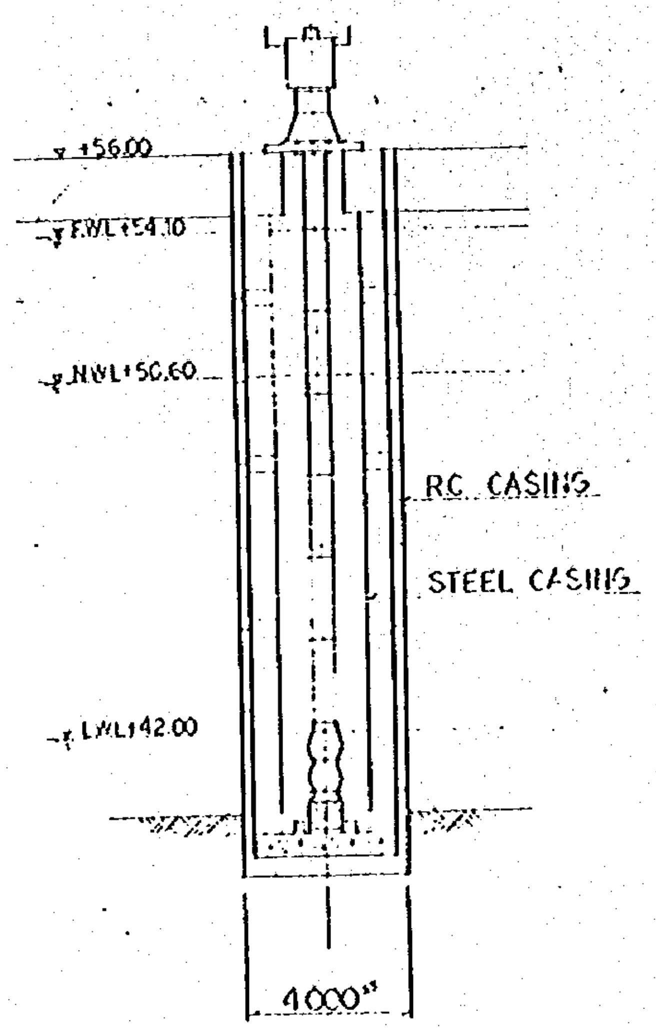
DWG NO. 2 TEMPORARY BRIDGE (2)
DETAILED SECTIONS

PLAN S-1/150



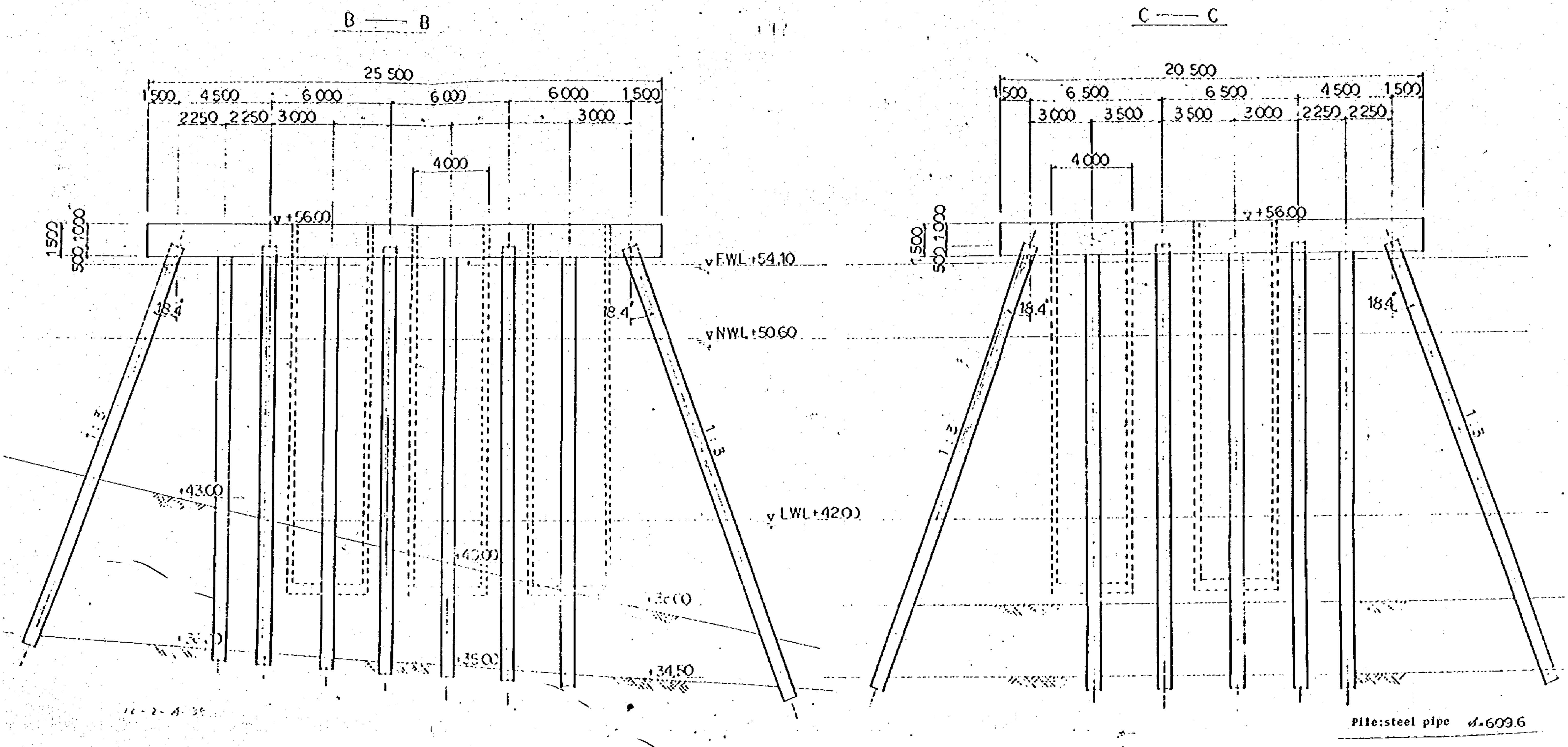
Ø-609.6

A—A SECTION S-1/150



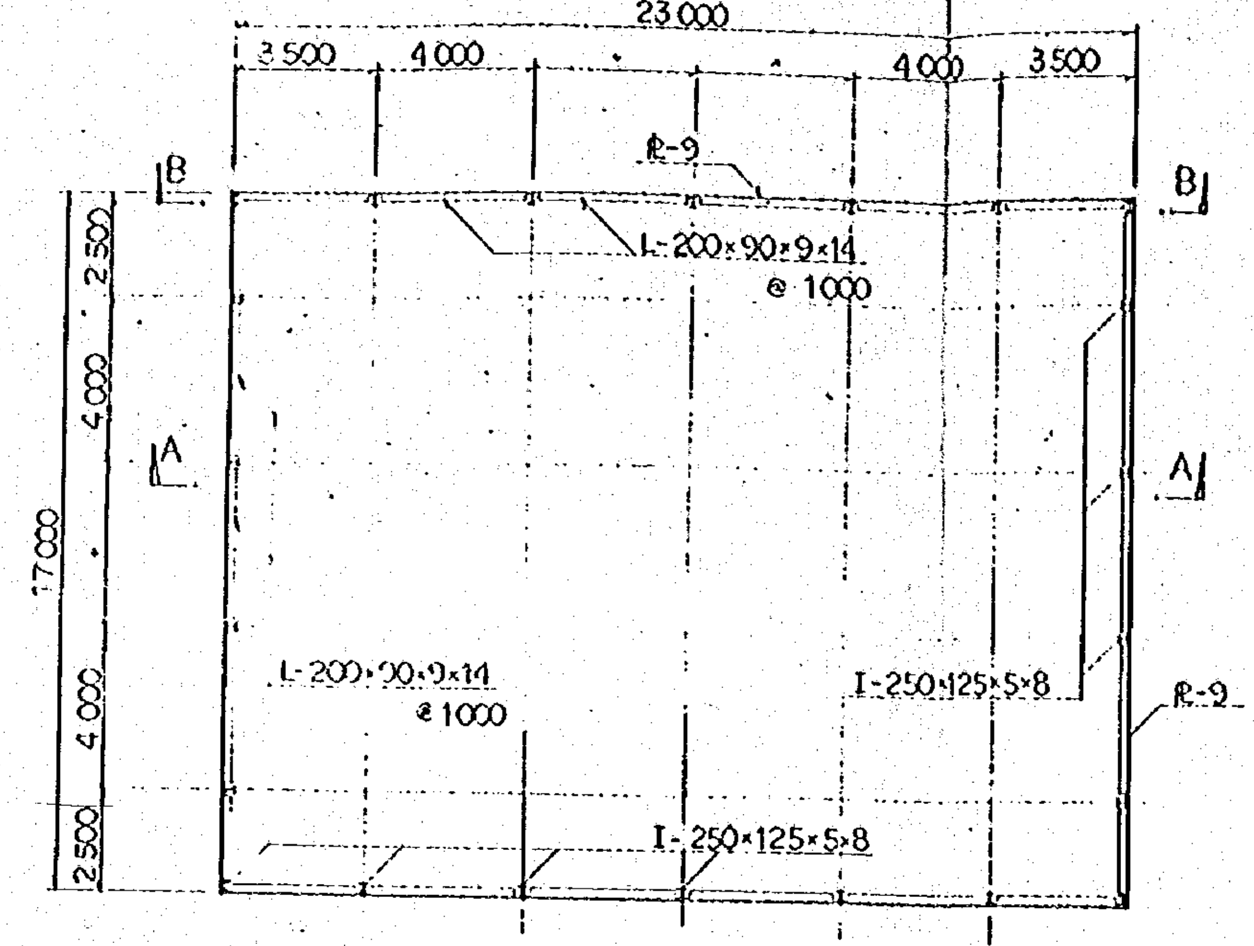
10-2-A-33 DWG NO. 3 PILE FOUNDATION (1)
PLAN , JACKET

SECTIONS S=1/150

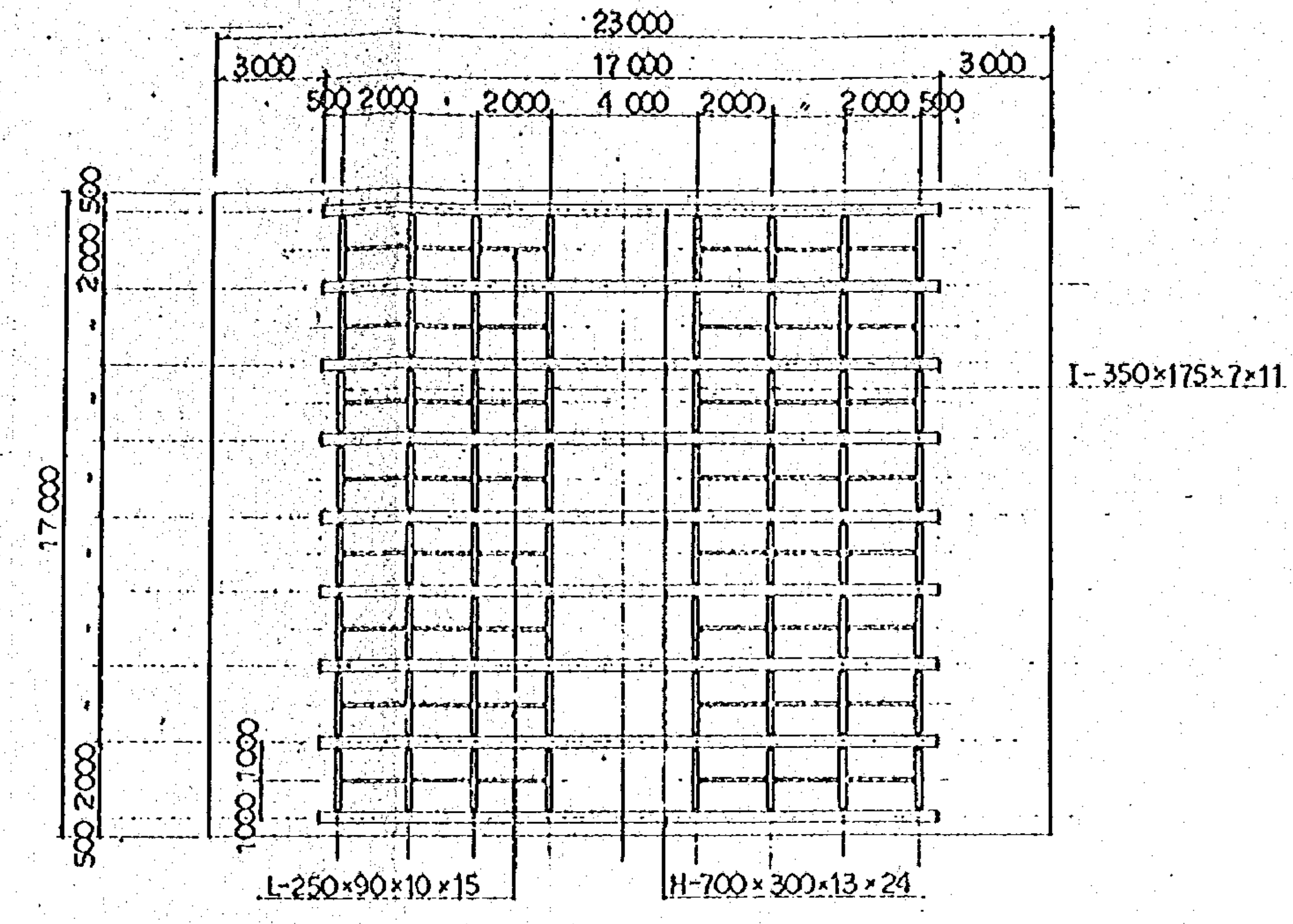


10-2-A-38 DWG NO. 4 PILE FOUNDATION (2) SECTIONS

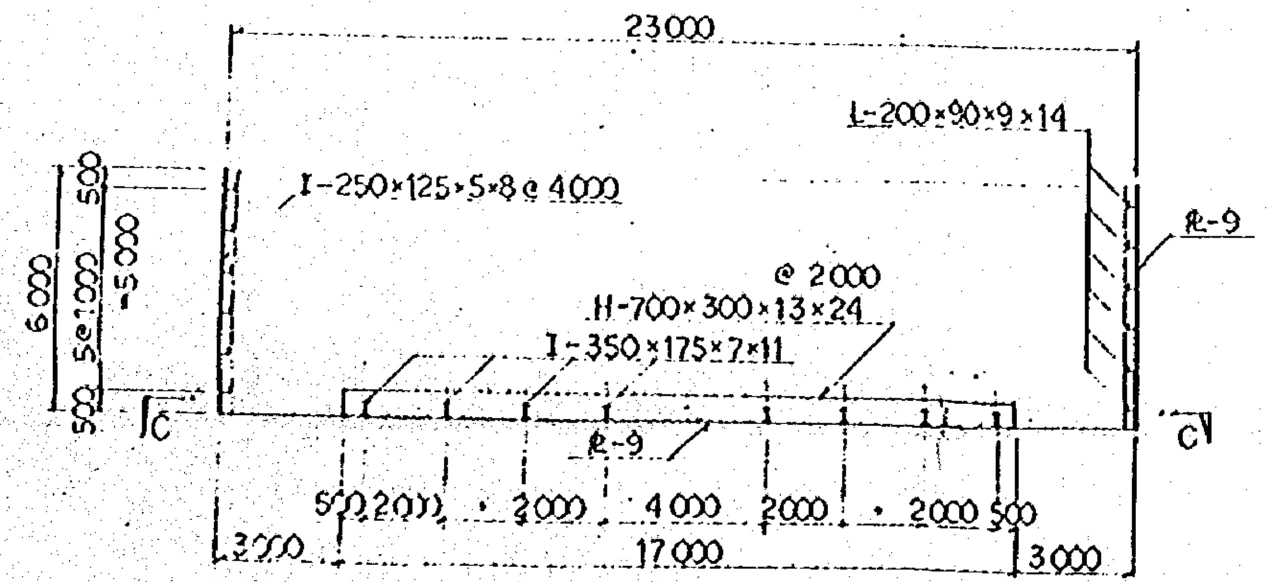
PLAN S=1/150



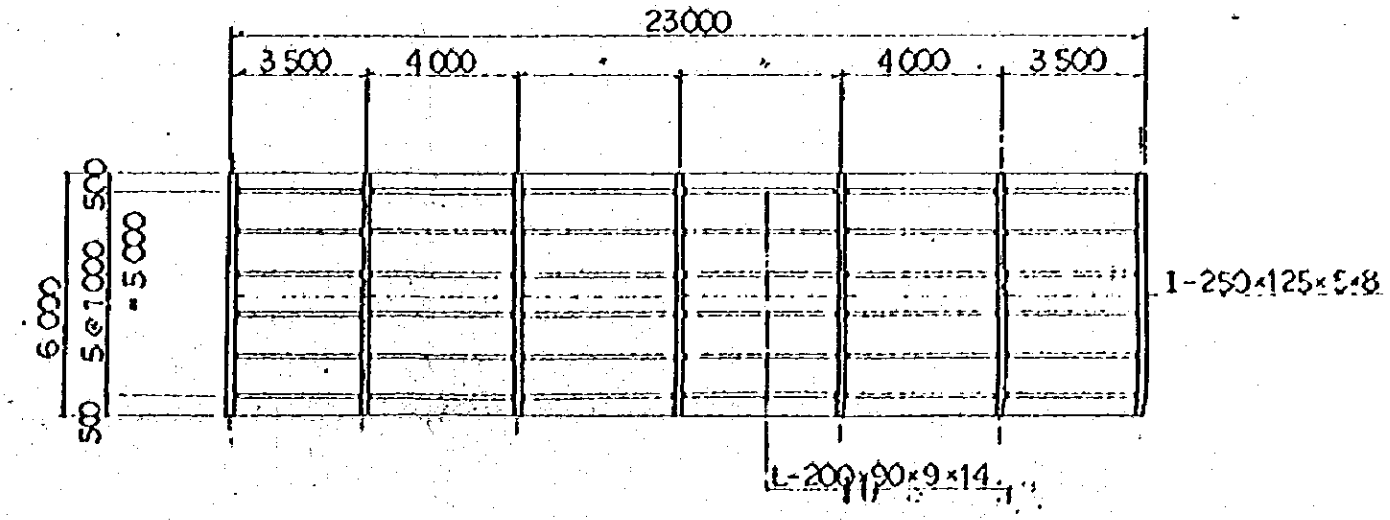
A — A S=1/150



A — A S=1/150



B — B S=1/150



10-2-A-36

10-2-A-37 DWG NO. 5 STEEL BOX FOR CAISSON DETAILS

No.11 Study on Selection of Control Valve and Emergency Shut-off Valve

MARCH 1982

Prepared by

YOSHIKAZU NISHIDA
Pipeline Engineer
Detailed Design Team
JICA

CONTENT

1. INTRODUCTION

2. DESIGN CONDITION

2.1. Flow Rate
2.2. Pipeline
2.3. Water Levels

3. FLOW CONTROL, PRESSURE REDUCTION AND CAVITATION

3.1. Hydraulic Profile
3.2. Cavitation
3.3. Gradual Reduction of Pressure by Splitting
3.4. Selection of Type of Valve

4. EMERGENCY SHUT-OFF VALVE

4.1. Requirements on Closing Time
4.2. Mechanism of Available Type
4.3. Problem of Butterfly Type
4.4. Multiple Sleeveport Type

5. RECOMMENDATION AND POWER SUPPLY

5.1. Selection of Type
5.2. Power Supply

1. INTRODUCTION

At the inlet of receiving well, the pipeline system needs a flow control valve and an emergency shut-off valve. The reasoning for existence and the function of valves are described in Reports No.4, 7-1, 7-2, 8 and 16.

The paper will discuss with the hydraulic characteristics and the operational requirements of those valves, and then will concluded about the selection of type of the valve.

2. DESIGN CONDITIONS

2.1. Flow Rate

The flow rate of pipeline grows from the initial stage of operation year by year until the flow rate reaches the design rate.

Even in the later stage, the flow rate will fluctuate due to the seasonal change of municipal demand and change of production activity in industries.

Here, 3 figures will be considered for discussion:

Q d	design capacity 2.62 m^3/sec
Q max	estimated rate of low demand in the later stage 1.57 m^3/sec
Q min	estimated rate of mean demand in the initial stage 0.30 m^3/sec.

2.2. Pipeline

The paper deals with only the part of pipeline between Head Tank and Receiving Well.

Diameter	1350 mm	
Length	19.0 km	
Coefficient of roughness	c = 120	William Hazen formula's C

2.3. Water Levels

The water level of Receiving Well is selected as 61.50 m and under the pipeline conditions Head Tank's level at Qd becomes+101.20 m

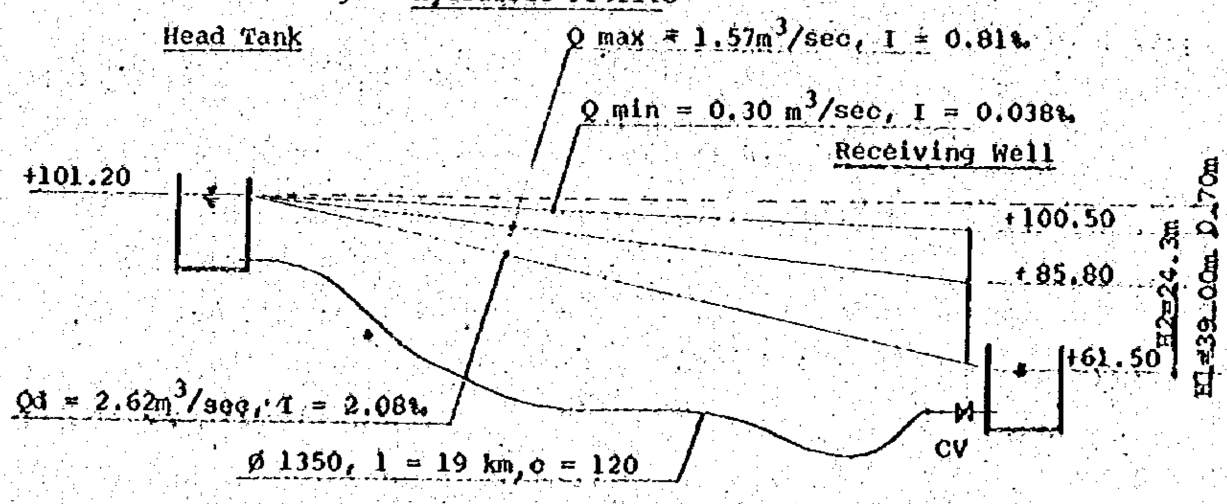
3. FLOW CONTROL AND PRESSURE REDUCTION AND CAVITATION

Two different water levels, when they are connected by a pipeline, causes flow of water. The flow rate can be controled by a vlave which is installed usually at the downstream end for the purpose. Throttling the valve gives an amount of resistance to the piepline, in the form of difference of static pressure between the both sides of valve.
The difference (reduction) of pressure can cause a phenomena called cavitation at the valve which is accompanied by vibration and wearing and it may result in the damage of it, in extreme cases.
Cavitation must be checked by all means.

3.1. Hydraulic Profile

Fig.1 is drawn under the flow rate Q min = 0.30 m^3/sec in William Hazen formula and the two tank levels shown.
From it one can clearly understand the control valve C.V. gives 39 m pressure reduction in the initial stage of operation.
Even in the later stage, Q max = 1.57 m^3/sec can be attained by 24.3 m pressure reduction by C.V., it is apparent.

Fig.1 Hydrauric Profile



3.2. Cavitation

The occurence of cavitation can be discussed on two factors:

(1) Cavitation Coefficient, σ

σ, the cavitation coefficient, is calculated by the following formula.

$$\sigma = \frac{H2 + Ha - Hv}{H1 - H2} = \frac{H2 + 10.03}{H1 - H2}$$

H1 : upstream side static head (m)
H2 : downstream side " " (m)
Ha : atmospheric pressure, 10.33 m
Hv : vapor pressure of water, 0.3 m under normal temperature

(2) Inherent Cavitation Value, σv

Table 1 shows the inherent cavitation value of various types of valve. The value is called "dynamic cavitation value" and hereafter in this paper it will be refered as icv and σv.

Table 1 Type of Valve and icv

Type	icv
gate	3.0
butterfly	2.0
rote	1.5
multiport	0.3

(3) Requirement of preventing Cavitation

The requirement of preventing cavitation is,

$$\sigma > \sigma v$$

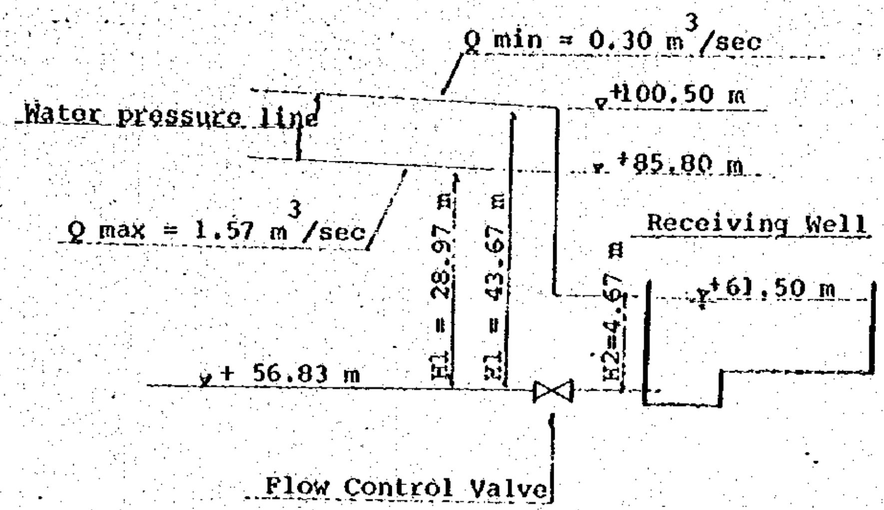
To prevent cavitation means to find methods to decrease σ and they will be explained in the following chapter.

3.3. Gradual Reduction of Pressure by Splitting

(1) Single Step Reduction

Fig.2 shows a single step reduction of head, by using only one valve.

Fig. 2 Reduction of Pressure, single step



For the cases, O will be :

H1 = 43.67 m or Q min = 0.30 m^3/sec σ = 0.38

H1 = 28.97 m or Q max = 1.57 " σ = 0.6

(2) Multiple Steps Reduction

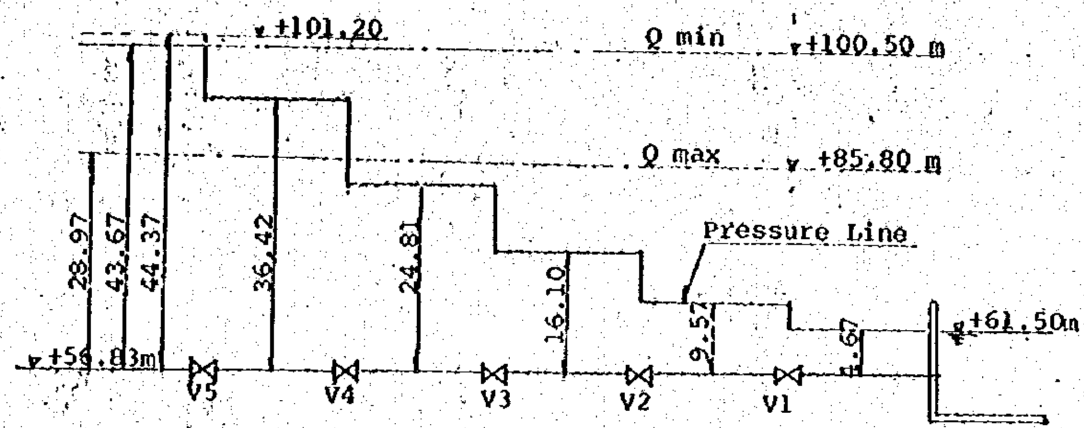
By using several valves and decreasing the pressure at each step, $\sigma > \sigma_V$ can be attained in each of splitted pipeline, thus making prevention of cavitation possible for the whole pipeline.

The number of necessary valves depends on the type employed there, as can be seen on Fig.3

Fig.3-1 - Fig.3-4 Pressure Reduction by Types of Valve

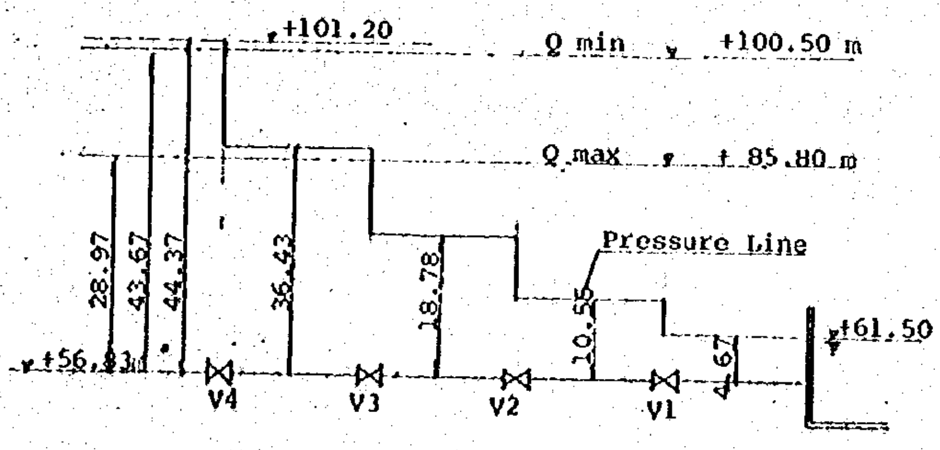
Fig. 3 Reduction of Pressure,5 or 4 steps

(1) Gate Valve δv = 3.0



Q min	H1	43.67	36.42	24.81	16.10	9.57
	H2	36.42	24.81	16.10	9.57	4.67
	δ	6.4 (5.8)	3.0	3.0	3.0	3.0
Q max	H1	-	28.97	24.81	16.10	9.57
	H2	-	24.81	16.10	9.57	4.63
	δ	-	7.3	3.0	3.0	3.0

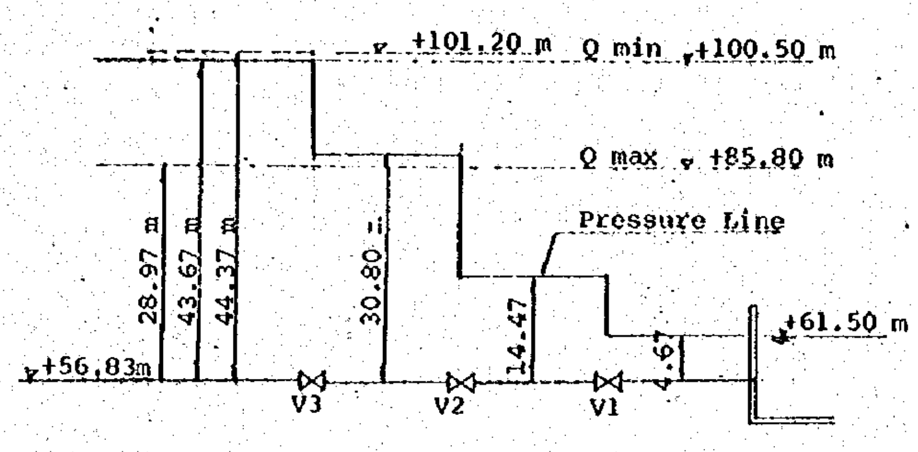
(2) Butterfly Valve δv = 2.5



Q min	H1	43.67	36.43	18.78	10.55
	H2	36.43	18.78	10.55	4.67
	δ	6.4 (5.6)	2.5	2.5	2.5
Q max	H1	-	28.97	18.78	10.55
	H2	-	18.78	10.55	4.67
	δ	-	2.8	2.5	2.5

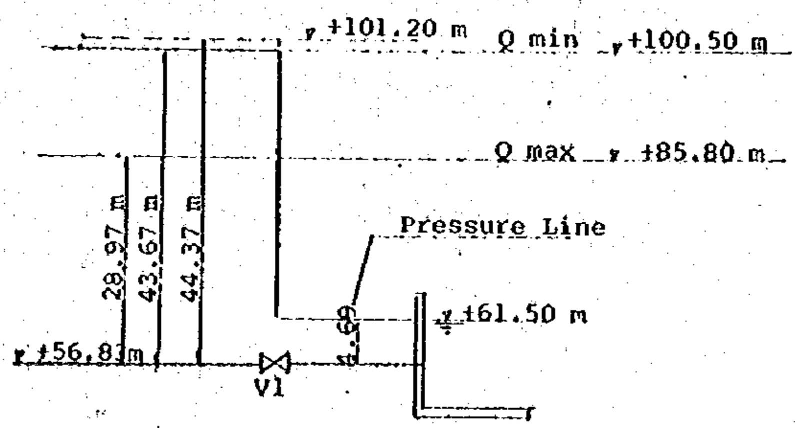
Fig. 3 Reduction of Pressure, 3 & 1 step

(3) Roto Valve $\sigma v = 1.5$



Q min	H1	43.67	30.80	14.47
	H2	30.80	14.47	4.67
	σ	3.2 (3.0)	1.5	1.5
Q max	H1	-	28.97	14.47
	H2	-	14.47	4.67
	σ	-	1.7	1.5

(4) Multiple Sleeveport Valve $\sigma v = 0.3$



Q min	H1	43.67
	H2	4.67
	σ	0.37 (0.37)
Q max	H1	28.97
	H2	4.67
	σ	0.60

(3) Summary of Multiple Steps Reduction

Fig.3-1 to 3-4 lead that more than 2 valves are needed, except in the case of multiple sleeveport valve. It is also shown on Table 2.

Table 2 Number of Valve

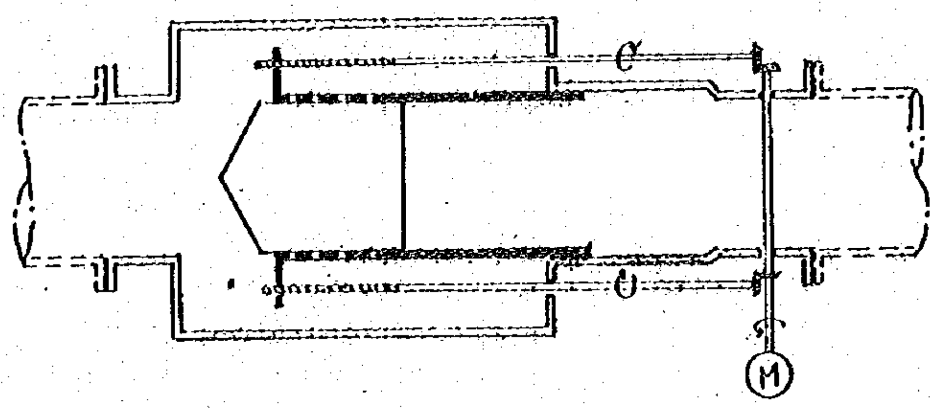
Type	Initial Stage of Operation	Final Stage of Operation
Gate	5	3
Butterfly	4	3
Rote	3	2
Multiple Sleeveport	1	1

(4) Sketch and Mechanism of Multiple Sleeveport Valve

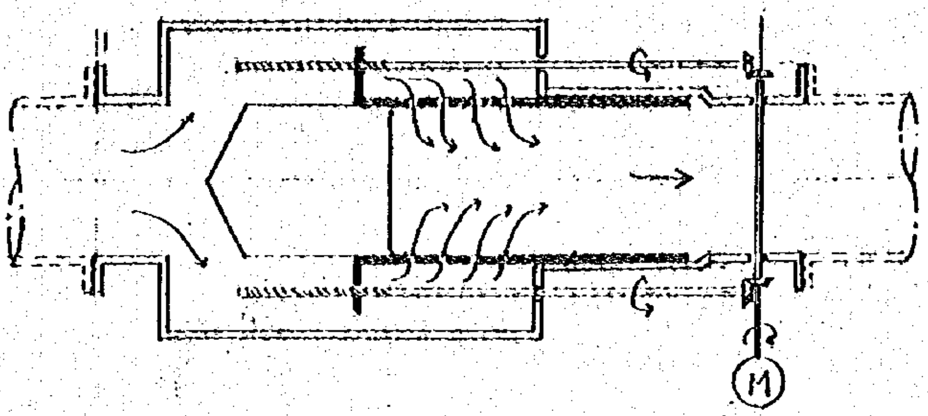
The valve's sketch will be self-evident of the low characteristic Cv:

Fig.4 multiple Sleeveport Valve

(1) Closed Position



(2) Open Position



(5) Study on Economy and Easiness of Operation

With the 4 types of valve, comparison on economy and easiness of operation shall be discussed.
Table-3 shows the price of comparison.

Table 3 Comparison of Value Price*

Type	Necessary Number	Unit price (1,000¥)	Amount (1,000 ¥)
Gate	5	700	3,500
Butterfly	4	400	1,600
Rote	3	3,500	10,500
Multiple Sleeveport	1	3,700	3,700

* ex-factory in Japan

About the easiness of operation, the less number is always better because in case of several valves, each one must satisfy the condition (the share of pressure reduction). It needs delicate adjustment accompanied by measurement of pressure and once the flow rate is changed, rechecking and readjustment are needed.

3.4. Selection of Type of Valve

In selecting the type the economy, easiness of operation, control capability and necessary land area will be the factors of evaluation.

The table below is :

* Each of the 5 factors is given the priority points, the total of which is 100.
* The priority points are shared by 4 types of valve.
* The sum of each type's point on 5 factors is compared.

Table 4. Comparison of Type

Type	Economy	Factors easy Operation	Control Capability	Land Area	Sum
Gate	9	3	3	1	18
Butterfly	20	4	4	2	30
Rote	3	5	8	3	19
Multiple Sleeveport	8	13	10	4	35
Total (priority point)	40	25	25	10	100

From the table, the decision is on the multiple sleeveport type.

4. EMERGENCY SHUT-OFF VALVE

In case of unexpected accidents including the power failure, an emorgency shut-off valve must close the pipeline in order to prevent it from being emptied.

4.1. Requirements on Closing Time

However, short closing time is not necessarily preferable as it tends to cause the water hammer effect. Too long closing time is clearly meaningless.
For the project, about 10 minutes seem suitable, considering the capacity of head tank.
Another desirable characteristic is that the flow rate be decreased in proportion to the lapse of closing time.

4.2. Mechanism of Available Type

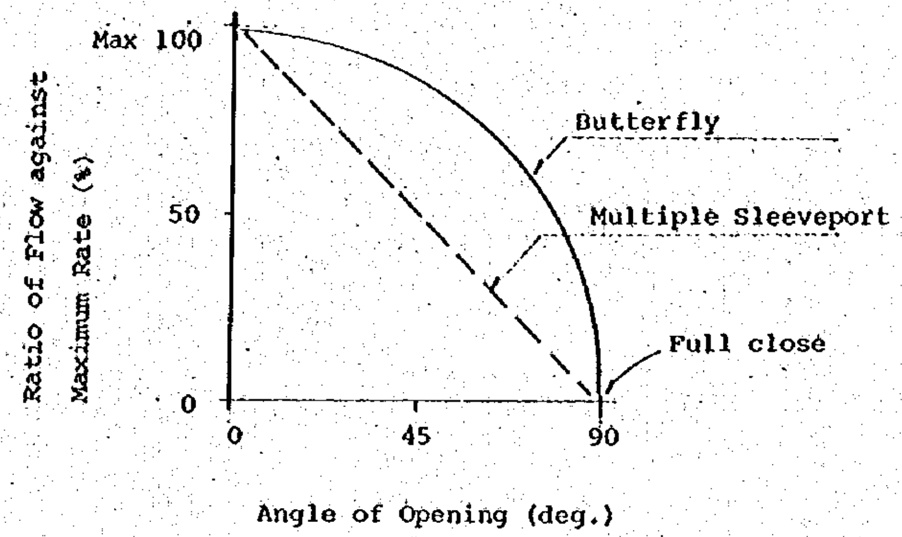
The most widely used one is butterfly type with mechanism in which a counterweight closes the opening by its own weight, in place of motor.
To restore the valve in open position, the counterweight shall be lifted by manually and/or by a small motor which can be driven with battery, even in case of the power failure.
The problem with the type using a counterweight is that it closes too fast and the possible longest closing time will be about 90 seconds due to the gear mechanism.

4.3. Problem of Butterfly Type

The foregoing discussion denies the use of available type. Moreover, the butterfly type is not suitable because of the following reasons with computer analysis.

* The butterfly type, when it is closed at a constant spead, throttles the flow rapidely nearing the full-closed position, as shown on Fig.5.

Fig.5 Butterfly Valve Characteristics



It certainly causes the water hammer and does not agree with the characteristics described in 4.1.

* In order to avoid the water hammer problem in using the type, the closing time shall be at least 25 minutes which contradicts the condition in 4.1.

4.4. Multiple Sleeveport Type

The characteristics of the type, as shown on Fig.5, does agree with the condition in 4.1.

5. RECOMMENDATION AND ADDITIONAL POWER SUPPLY

5.1. Selection of Type

The multiple Sleeveport valve shall be choosen as 1 valve can do the both roles of flow control and emergency shut-off, without causing the water hammer problem.

5.2. Power Supply

The valve shall be closed both by manually and by electricity in about 10 minutes.
The estimated motor rating is 3.7 kw and the generator at Mab Ta Pud shall be able to supply the power and the battery shall supply extra current in starting the generator.

NO.12 REPORT ON PIPE WALL THICKNESS

FEBRUARY 1982

Prepared by YOSHIKAZU NISHIDA
Pipeline Engineer
Detailed Design Team
JICA

CONTENT

1. INTRODUCTION

2. PLERIMINARY CONSIDERATION

3. FOUNDATION AND BACKFILLING

4. LOAD
   - 4.1. Classification
   - 4.2. Earth Pressure
   - 4.3. Load by Truck
   - 4.4. Load by Railroad
   - 4.5. Load during Construction

5. TOLERENCE IN DEFLECTION AND STRESS
   - 5.1. Tolerence in Deflection and Stress
   - 5.2. Tolerence in case of Short Term Load

6. DESIGN CALCULATION
   - 6.1. Selection of Loads
   - 6.2. Circumferential Stress by Internal Pressure
   - 6.3. Deflection and Stress by External Load

7. STANDARD THICKNESS

8. COMPARATIVE STUDY
   - 8.1. Conditions
   - 8.2. Calculation of Loads
   - 8.3. Calculation of Stress and Deflection

9. DISCUSSION

10. CONCLUSION

1. INTRODUCTION

The wall thickness of steel pipeline is an important subject which needs serious study in the project.
Dok Krai - Mab Ta Pud pipeline is a vital artery on which the modern industrial complex must live and grow. Also it is a precedent in the size of pipeline and the distance of transmission in Thailand.
The feasibility study team proposed 11.9 mm thickness on the basic consideration of internal pressure and external load.
The detailed design team likes to study it more widely, taking account of the informations collected here since it started working in Nov. 1981.

2. PRELIMINARY CONSIDERATION

(1) Cost

A rough estimate on 3 thickness will be made, 11.9 mm, 11.1 mm and 10.3 mm.
The assumption is :

* The diameter is 1.35 m I.D. (inside diameter).
* The specific gravity of steel is 7,840 kg/m^3.
* The pipeline length is 27,000 m.

For 3 different thickness, the total material weight is :

thickness (mm)	11.9	11.1	10.3
weight (ton)	10,778	9,957	9,318

Assuming the cost is roughly 1,100 US$/MT (Metric Ton), the cost is :

thickness (mm)	11.9	11.1	10.3
cost (10^6\$)	11.86	10.95	10.25
cost* (10^6฿)	266.76	246.44	230.62

* 1 \$ = 22.5 ฿

36 million Baht difference between 11.9 and 10.3 mm thickness seems to be worth making further study.

(2) Corrosion Allowance and Protective Coating

A quotation from "Steel Pipe, Design and Installation", American Waterworks Association Manuall 11, says :
Adding a fixed rule-of-thumb thickness to the pipe wall as a corrosion allowance is not a rational solution of a corrosion problem. This is especially true in the water works field, where approved coating materials and coating procedures prevail. It is preferable to use the required nominal wall thickness pipe and then apply the proper protective coating for the condition encountered.

An understanding that the thickness shall be studied on strictly technical requirements is needed here.

(3) Local Road and Traffic Conditions

The pipeline runs along the RID road between Dok Krai and Route 3191, Route 3191 and 3.
The road is wide in its right of way and only the center part of it is paved and used. The traffic is not heavy and the road pavement condition is well kept, probably of good subsurface preparation, general soil condition, low underground water table and not-so-big traffic.
There is no possibility of heavily loaded trucks running along on the water pipe but,across it trucks will have to go to private properties and small roads.

(4) Hydraulic Condition

The pipeline is subjected to about 7.0 kg/cm^2 under normal operation and to about 10.0 kg/cm^2 when water hammer occurs.

(5) Backfilling and Compaction

The excavated earth is suitable for backfilling after laying pipes.
In Thailand, compaction after backfilling is well practiced and as the backfill earth is rather good, careful and skilled compaction will make proper conditions to distribute the external load evenly on the pipe.

(6) Textbook for Study

The author uses "Criteria in Calculating Pipe Thickness of Buried Waterworks Steel Pipe" published by Japan Waterworks Steel Pipe Association and revised in Sept. 1980.
The criteria refered to not only.Japanese technical literatures but also the recent literatures published by American Waterworks Association and American Society of Civil Engineers.*

* A.K. Howard : Modulus of Soil Reaction Values of Buried Flexible Pipe, Jan. 1977, Proc. of ASCE
AWWA : Steel Pipe Manual, 1979

3. FOUNDATION AND BACKFILLING

3.1. Foundation

The bottom of the trench should be clean, flat and free from stones and hard lumps so that the pipe will lie directly on earth in the bottom of the trench. (AWWA Manual 11) Here the soil is sandy clay, possibly free from stones and hard lump.

The trench bottom under large steel pipe may be shaped advantageously for arc contact instead of line contact. (AWWA Manual 11)

For the project, about 0.3 m thick sand bed at the bottom of the trench will be specified for the most part. The sand bed, under the load of pipe, will settle and an arc contact will be made between the earth and the pipe.

3.2. Backfilling

After the pipe being laid, backfilling both side of the pipe and compacting must be done with good care.
AWWA Manual 11 says :

* The depth of backfill must be more than 6 in. above the horizontal centerline of pipe, with a diameter greater than 12 in.

* The backfilled earth should be tamped in layers whose thickness may vary 4 in. to 12 in., depending on soil properties, degree of compaction required, and method of compaction.

* The fill should rise the same on each side of and coincidentally be tamped in layers.

JICA team will specity about the matters in following terms, so that the above requirements are satisfied.

* About 2/3 of pipe diameter or about 9 in. above the centerline of pipe backfill.
* Each Layer, about 12 in. thickness, will be tamped by a tamper or compactor weighing 80km, three times on all over the backfilled surface.
* The filling and tamping of the same level layer at the both side is a "must" in pipelaying works.

The grade of compaction after backfilling is expressed as coefficient E' which is used in calculating the deflection and bending stress of the pipe.

4. LOADS

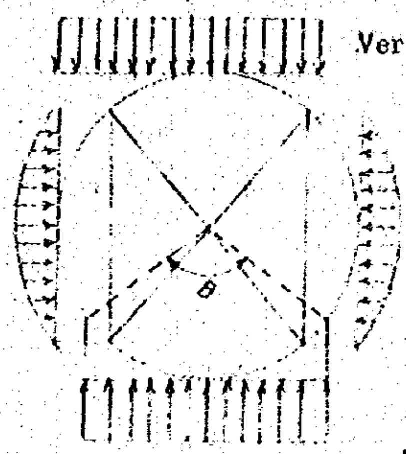
4.1. Classification

The main loads include the internal pressure, earth load, automobile load, railway load and other load acting on the pipe.
The subordinate loads include the load during construction and the water hammer.

4.2. Earth Pressure

The model of earth pressure's distribution follows Iowa formula proposed by Spangler, as shown on Fig. 1.

Fig.1 Earth Pressure Distribution



(1) Vertical Load

Ordinary Ditch

The ordinary ditch means a ditch excavated without the use of sheet pile.
Here, two cases are conceivable. One case where the cover depth (the depth between the surface of backfilled ditch and the top of pipe) is less than 2m and another case where the cover depth is more than 2 m.

In the former case, the earth load of the unit weight multiplied by the cover depth works upon the pipe but in the latter case, the earth load, being reduced by internal friction of the backfilled earth, acts with less strength, in term of unit area, on the pipe.

Former case is formularized :

$$Wv = \gamma s \cdot H \quad \text{(Cover depth } H = 2\text{m)}$$

Latter case is formularized :

$$Wv = Cd^{*} \cdot \gamma s \cdot B \quad \text{Marston's formula}$$

With the two formular :

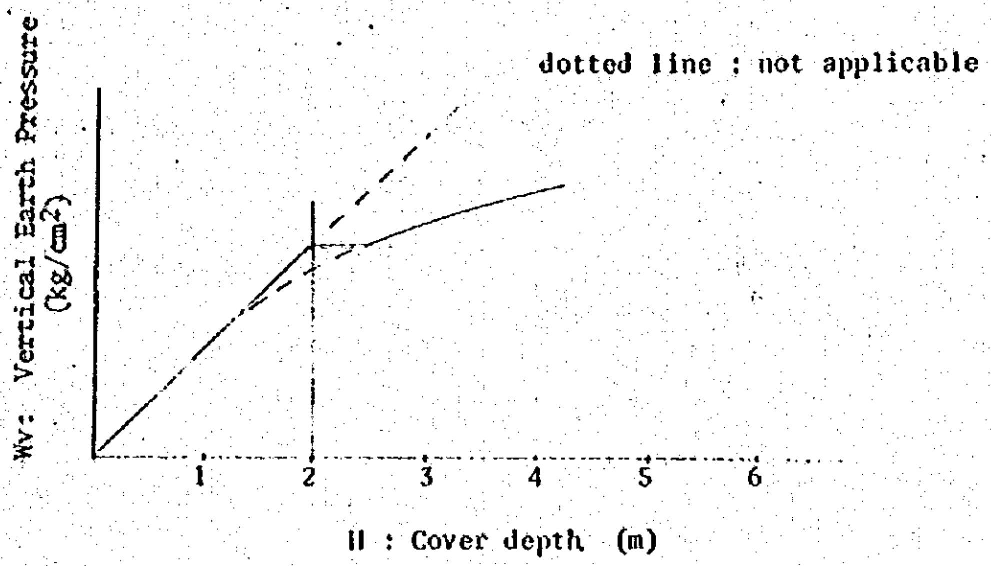
- Wv : vertical earth pressure, kg/cm^2
- γs : earth's unit volume weight, normally 0.0017 kg/cm^3
- H : cover depth, cm
- Cd : coefficient applicable to the ditch
- k : Rankin's coefficient of the earth pressure
- ϕ : angle of internal friction of the backfilled earth
- ϕ' : angle of friction between the backfilled earth and the surface of ditch wall (usually $\phi' = \phi$ is assumed)
- μ' : coefficient of friction between the backfilled earth and the surface of ditch wall
- B : width of ditch at the top of pipe

In case where the cover depth is in between 2 m and 3 m and the value of earth pressure, calculated by Marston's formula, is found to be less than the case of $H = 2^m$, the earth pressure at $H = 2^m$ shall be used in the calculation.

$$^{*}\ Cd = \frac{1 - C^{-2K\mu' \frac{H}{B}}}{2K\mu'} \qquad K = \frac{1 - \sin\phi}{1 + \sin\phi} \qquad \mu' = \tan\phi$$

The quotation is pictured on Fig. 2.

Fig.2 Application of Earth Pressure Formula.



Ditch with Sheet Pile

use the formula for H = 2m and extrapolate linearly..

(2) Passive Earth Pressure

(a) Coefficient E'

The passive earth pressure on Fig.1 is caused by the deformation of pipe and it is a reaction force working horizontally on the side of pipe. E', the coefficient of reaction by the earth, can be considered as a kind of spring coefficient (The spring elongates proportionally to the weight working on it) and it changes on the degree of compaction and the nature of back-filled earth.

In practice, Table 1. is applicable to find E'.

Table 1 B' Value (kg/cm^2)

(Translation of U.S.B. of Reclamation data, 1977)

Classification of Earth	B' depending on Compaction		
	No Compaction	lightly compacted P.D. < 85% R.D. < 40%	properly compacted P.D. ≧ 85-90% R.D. ≧ 40-70%
fine - grained, plasticity : middle to high	No data available, need consultation, otherwise B' = 0		
fine - grained, plasticity : nil to middle	3.5	14	28
fine - grained W. more than 25% coarse grain, plasticity : nil to middle / coarse - grained W. more than 12% fine grain,	7	28	70
coarse - grained W. less than 12% fine grain,	14	70	140

P.D. Proctor Density   W. with

R.R. Relative Density

(b) Application of E'

As E' of the undisturbed earth before excavation is generally larger than E' of the backfilled earth, except in the case of soft ground. Accordingly E' of the backfilled earth is to be used for calculation. Where the ground is soft, E' of both the ground earth and the backfilled earth is compared and the smaller value is used while where the width of ditch is sufficiently wide (for instance 5 times of diameter), E' of the backfilled earth is used disregarding the ground condition.

(c) Standard of E'

Where the sand is used without compaction and/or the excavated earth is good and lightly compacted after backfilling, E' = 14 kg/cm^2 can be a standard.

(3) Reaction Force at Foundation

(a) Distribution of Reaction Force

The distribution of reaction force at the foundation, shown on Fig.1, is shaped complicatedlly depending on the foundation material, thickness, workability and the deflection of pipe, etc.
Here, an assumption is given that the force is evenly distributed on the width of foundation, designated by the angle of contact between the pipe and foundation bed. (Fig.1)

(b) Angle of Contact

The angle of contact is assumed as 90 deg. here, as a standard.
The designer can decide the angle when he has sufficient reasons to do so, needless to say.

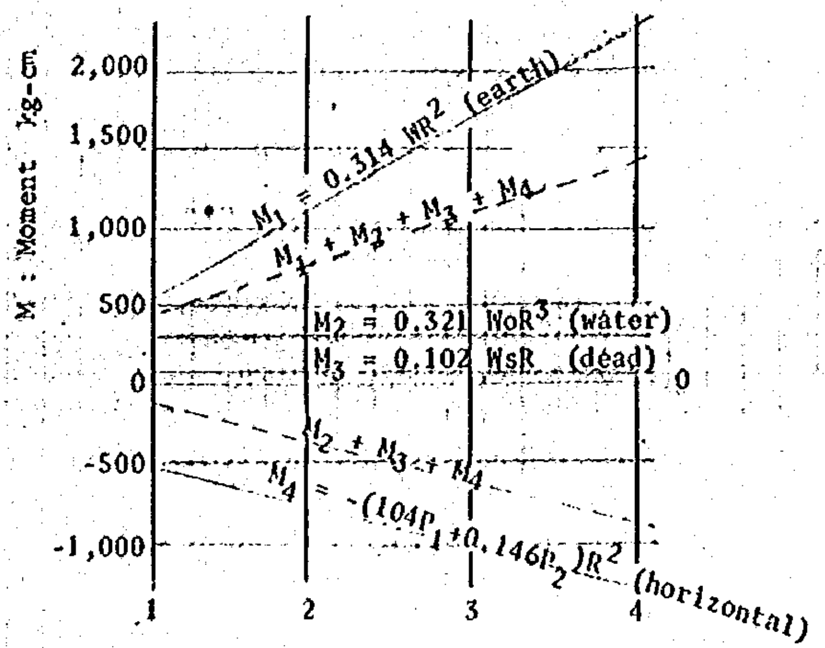
(4) Dead Weight of Pipe, Water Weight and Active Earth Pressure

In the calculation of pipe thickness buried underground, the bending moment caused by the pipe's dead weight and water weight acts against the bending moment caused by the active earth pressure from the side.
Consequently, the effect of these forces can be disregarded, as they are canceled out.
Paraphrasing the above :
The bending moment at the bottom of pipe is taken as M' and assume that M' is the sum of M1, M2, M3, M4, where M1 is caused by the vertical load, M2 by the pipe's dead weight, M3 by the water weight and M4 by the active earth pressure.
Then,

$$M' = M1 + M2 + M3 - M4$$

Generally proved is $M2 + M3 < M4$
Accordingly $M' < M1$
For calculation, only M1 must be taken into consideration and it results in the safer side of design.
An example is shown on Fig.3.

Fig.3 Various Loads and Moments
$R = 100$ cm $W_v = \gamma_s H$ kg/cm^2 $\gamma_s = 0.0017$ kg/cm^2
$\phi = 17$ deg.



4.3. Load by Truck

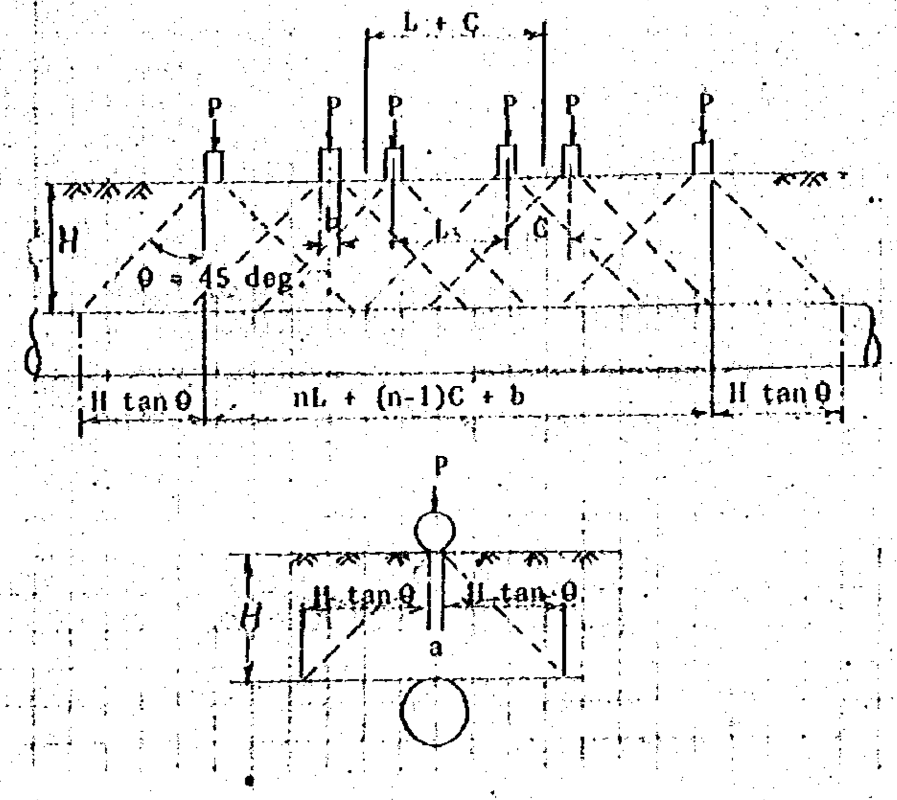
(1) Angle of Load Distribution

The concentrated load of truck's wheel at the surface of road decreases by the depth as it is distributed wider. Here, Koglor's method is followed with the angle of distribution as $\theta = 45°$. (Fig.4)

(2) Calculation of Load Distribution

Fig. 4 shows how to reason the distribution :

Fig.4 Distribution of Load



Two Classes

In Japanese Standards, two classes T-14, T-20 are used, representing the total weight of a truck as 14 and 20 tons.

Split of Load

The weight is split into 20% on the front wheels and 80% on the rear wheels.

Each rear wheel 8 ton (P in Fig.4) in the case of T-20.

Spreading over Surface of Road

The weight is spread over the road surface by the contact area of a tyre.

For the rear wheel tyre, the width (b in Fig.4) is 50 cm and the length (a in Fig.4) is 20 cm.

Distance of Wheel

The distance of rear wheels is designated as L in Fig.4 and is specified as 175 cm.

Width of Occupancy

A truck's width of occupancy is designated as L+C in Fig.4, thus making C as the distance of wheels of adjacent two trucks. C is specified as 100 cm and L+C becomes 275 cm.

Depth of Pipe

The earth's cover depth of pipe is H in Fig. 4.

Angle of Distribution

The load is distributed downwards evenly with the angle of 45 deg.

As seen in Fig.4, the distribution of the load on the top of pipe is not uniform in an exact sense but it is considered uniform pracitically.

With the mentioned reasoning and designation, the uniform strength of load can be expressed :

$$Wt = \frac{2n\,P(1+i)}{(nL + (n-1)C + b + 2H \tan\theta)\,(a + 2H \tan\theta)}$$

Wt : strength of load, kg/cm

n : number of trucks

i : coefficient of impact

The coefficient of impact, varying according to the depth, is given as Table 2.

Table 2 i, Coefficient of Impact

H : cover depth (m)	i : coeff. of impact
$H < 1.5$	0.5
$1.5 \leqq H < 6.5$	$0.65 - 0.1\ H$
$H \geqq 6.5$	0

Simplified Formula

Substituting the specified and given values to P, L, C, b, a and θ, the formula becomes :

$$Wt = \frac{8000(1+i)n}{(275.n-50+2H)\ (10+H)}$$

n is the number of trucks which run over across the pipe simaltaneously. Even if the road is wide enough, n cannot be larger in accordance with the width. Here, n = 1 or 2 at most is sufficient.

$$n = 1 \qquad Wt = \frac{8000(1+i)}{(225+2H)\ (10+H)}$$

$$n = 2 \qquad Wt = \frac{8000(1+i)}{(250+H)(10+H)}$$

4.4. Load by Railroad

It is omitted here.

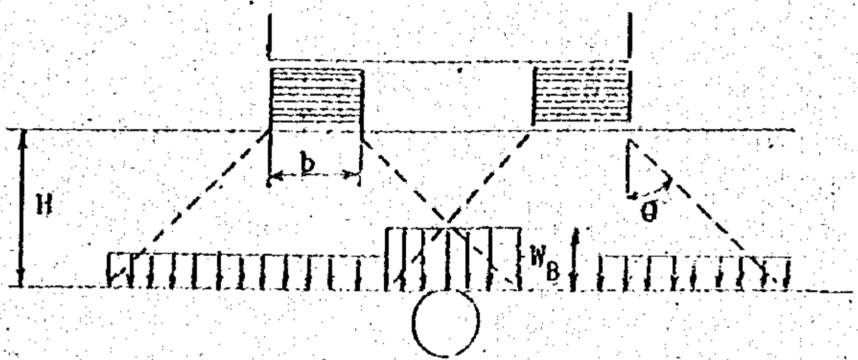
4.5. Load during construction

During the construction stage, a bulldozer may have to run over across buried pipes. This kind of work must be avoided as much as possible.

Here the concept will be explained but the condition will be excluded in the later calculation.

Distribution of Load

Fig.5 Load by Bulldozer



One difference here is that the load by two caterpillers is overlapped to become bigger here. It comes from the assumed insufficient compaction of earth.

The formula is : $W_B = n \cdot g_B\ (1+i)\ \left(\frac{b}{b + 2H \tan\theta}\right)$.

W_B : strength of load, kg/cm

i : coefficient of impact, normal ground $i = 0$

soft ground $i = 0.2$

g_B : contact pressure of caterpiller, kg/cm^2

b : width of caterpiller, cm

n : number of caterpiller

H : cover depth of earth, cm

θ : angle of distribution

4.6. Other Load

After laying of pipe, other loads like more covering of earth, crossing culverts and retaining walls may be placed on the pipe. For each, consideration must be paid.

4.7. Internal Pressure

Normally the internal pressure for design includes the maximum pressure under normal operation and the water hammer pressure.

5. TOLERANCE IN DEFLECTION AND STRESS

5.1. Tolerence in Deflection and Stress

(1) Deflection

The ratio of deflection is expressed as the ratio of deflection to nominal diameter.
The tolerance is different on the inside protection of coating or lining.
With tar-epoxy coating it is 3% while with cement mortar lining it is 5%.
A steel pipe, in spite of capability for elongation and ductility, should be cautiously protected against rust and corrosion. Accordingly, the deflection must be specified so that the inside protection shall not be damaged due to over-deflection.

(2) Stress

The allowable stress depends on the characteristics of steel used for the purpose.
In case of STPY 41 of JIS, Carbon Steel Pipe for General Piping by Arc Welding, 1,400 kg/cm^2 is specified.

(3) Plate Thickness, Mill Tolerance

JIS specifies the tolerence of plate thickness and the tolerence of deflection and stress of the pipe has taken account of it. Calculation shall be made on the nominal plate thickness.

(4) Corrosion Allowance

Already discussed in 2.

(5) Effectiveness of Welding

Welding, whether done in the shop or on the site, does not reduce the strength of material. The Welded part's strength can be equal to that of mother material.

5.2. Tolerence in case of Short Term Load

Quite different from the case of permanent load, an instanteneous load or a short term load can be met with the increase of tolerence.

Internal Pressure

For the sum of normal pressure and water hammer pressure, the tolerence is 150% of the specified one.

External Load

For the sum of earth pressure and overlying loads as the main load, the tolerence is 150% of the specified one.

6. DESIGN CALCULATION

6.1. Selection of Loads

The following combination of the loads shall be considered in design calculation.

Combination of Loads

Case	Main Load	Subordinate Load
1	Normal pressure	
2	" "	water hammer pressure
3	earth pressure plus overlying loads as main load	
4	earth pressure plus overlying loads as main load	load during construction

6.2. Circumferential Stress by Internal Pressure

The following equation shall be used in calculating the circumferential stress.

$$\sigma t = \frac{PD}{2t}$$

σt : stress, kg/cm^2

P : internal pressure, kg/cm^2

D : average diameter of pipe, cm

t : thickness of pipe wall, cm

Instead of the average diameter, the nominal diameter can be used as the error is negligibly small.

6.3. Deflection and Stress by External Load

The deflection and Stress of steel pipe under the earth pressure and overlying loads is calculated by the following formula :

Deflection

$$\Delta x = \frac{2kx \cdot (Wv + Wt)R^4}{EI + 0.061\, E'R^3}$$

Bending Stress

$$b = \frac{2}{fZ}(Wv+Wt)\,\frac{Kb\cdot R^2EI + (0.061\ Kb - 0.083\ Kx)E'R^5}{EI + 0.061\ E'R^3}$$

- Δx : horizontal deflection, cm
- b : bending stress at pipe bottom, kg/cm^2
- f : coefficient by shape, 1.5
- Z : section modulus, cm^3
- Wv : vertical earth pressure, kg/cm^2
- Wt : load by truck, kg/cm^2
- R : average radius of pipe, cm
- E : modulus of elasticity of steel, 21000000 kg/cm^2
- I : geometrical moment of inertia, cm^4
- E' : coefficient of passive earth pressure, kg/cm^2
- Kb : coefficient of bending moment at pipe bottom
- Kx : coefficient of horizontal displacement

(1) Distribution of External Load and Earth Pressure

Already shown on Fig. 1

Δx is the deflection at the center of pipe and σ_b is the bending stress at the bottom, where the stress is the largest along the ring.

(2) Coefficient Kb, Kx

Kb and Kx depends on the angle of contact between the pipe and foundation bed, as shown below :

Table of Kb and Kx

Angle of Contact (deg.)	Kb	Kx	(0.061Kb-0.083Kx)
60	0.189	0.103	0.00307
90	0.157	0.096	0.00161
120	0.138	0.089	0.00107
150	0.128	0.085	0.00082

(3) f

f is a coefficient to convert the bending stress in the wall into the axial stress. With the rectangular section of pipe wall, f equals 1.5.

(4) Special Case

With a steel pipeline, the design is made without considering the internal pressure, except in the case of extraordinary high pressure. Then the result of calculation is on the safer side of the existing condition.
Accordingly here, the calculation is made independently on the internal pressure and on the external loads.
Some special case requires the calculation to be made on a simaltaneous and combined consideration which shall be refered to other literatures.

7. STANDARD THICKNESS

Table 3, 4 shows the nominal pipe size and standard thickness in relation to the cover depth of earth.

The tables are calculated under the following conditions.

earth weight per unit volume : $\gamma s = 1.7 \times 10^{-3}$ kg/cm^3

coefficient horizontal displacement : Kx = 0.096

coefficient of bending moment at pipe bottom : Kb = 0.157

Load of truck : T - 20 2 number

coefficient of passive earth pressure : E' = 14 kg/cm^2

angle of internal friction of the backfilled earth : ϕ = 20 deg.

width of excavated ditch : shown on the table below :

Pipe dia. (mm)	ditch width (mm)	Pipe dia. (mm)	ditch width (mm)
350	700	1000	2000
400	800	1100	2000
450	900	1200	2500
500	1000	1350	2500
600	1000	1500	3000
700	1300	1600	3000
800	1500	1800	3000
900	2000	2000	3500

Table 3 Cover Depth with Sheet Pile

nominal dia. (mm)	Wall thick (mm)	Cover depth (m): 0.5 1 1.5 2 2.5 3 3.5
350	6.0	
400	"	
450	"	
500	"	
600	"	
700	"	
800	7.1	
900	7.9	
1000	8.7	
1100	10.3	
1200	11.1	
1350	11.9	
1500	12.7	
1600	14.0	
1800	16.0	
2000	18.0	

Note : ══════ Range of applicability

{══ Limit of reality

Table 4 Cover Depth without Sheet Pile

nominal dia, (mm)	Wall thick (mm)	Cover depth (m) 0.5 1 1.5 2 2.5 3 3.5 4 4.5
350	6.0	
400	"	
450	"	
500	"	
600	"	
700	"	
800	7.1	
900	7.9	
1000	8.7	
1100	10.3	
1200	11.1	
1350	11.9	
1500	12.7	
1600	14.0	
1800	16.0	
2000	18.0	

Note : ══════ Range of applicability

⊱══ Limit of reality

8. COMPARATIVE STUDY

Previously in 2. of this report, a comparative estimation was made on 10.3, 11.1 and 11.9 mm thickness.
Hereafter, a technical checking of the 3 different thickness will be made following the steps and the methods explained in Chap. 3 - Chap. 7.

8.1. Conditions

(1) Internal Pressure

The normal pressure is 7 kg/cm^2 and the normal plus water hammer pressure is 10 kg/cm^2.

(2) Pipe Material and Manufacturing

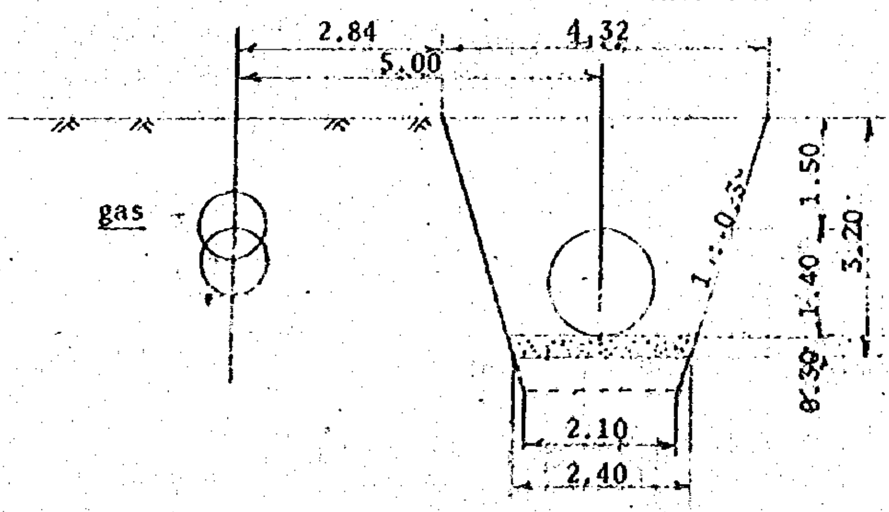
Material : JIS STPY 41, allowable stress 1,400 kg/cm^2
Manufacturing : arc welding of seams, straight and/or spiral in factory
the same of butt joints at site
Coating : tar-epoxy inside of pipe

(3) Pipe laying and Earth Load Pressure

Fig.6 shows the standard section of pipelaying.

Fig.6 Section of Pipelaying

Unit : m



Remarks :
* The gas pipes cover depth is 1 and 1.5 m.
* The dotted line shows the section for joint welding.
* The width at the top of pipe is 3.42 m.

With the figure, the cover depth can change from 0.8 m to 3.4 m for calculation.

The conditions used in Chap. 7 are used here. They are :

* Weight of earth's unit volume : $s = 1.7 \times 10^{-3}$ kg/cm^2
* Kx and Kb are 0.096 and 0.157 respectively. The case is for 90 deg. of contact.
* $E' = 14$ kg/cm^2, $\phi = 20°$

(4) Load by Truck

T-20 of JIS is used. The number of trucks is 1 and 2 for calculation.

(5) Load by Railroad and During Construction

The loads are omitted for calculation.

8.2. Calculation of Load

(1) Vertical Earth Load (Pressure)

$H \leq 2$ m

The earth pressure increases proportionally to the cover depth.

$H > 2$ m

Cd in Marston's formula is a function of H. Cd and Wv are calculated accordingly.

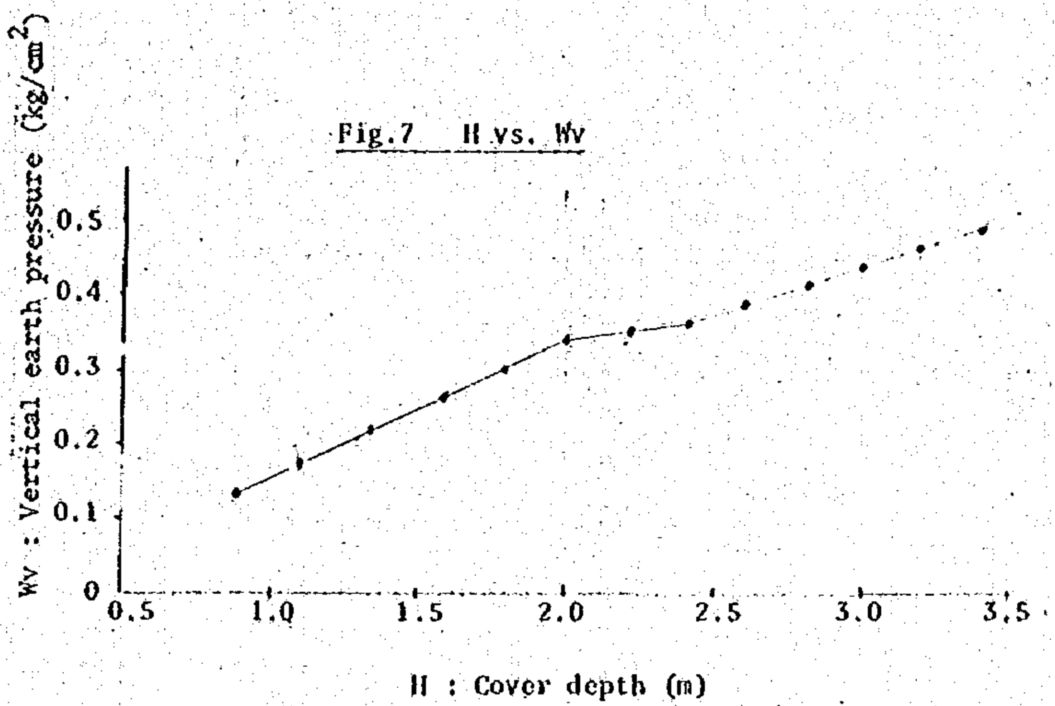
$0.8 \text{ m} < H < 3.4 \text{ m}$

With two equations mentioned above, Wv from 0.8 m to 3.2 m is tabulated on Table 5. Noticable is Wv for H = 2.2 m is lower than that for H = 2.0 m and it is corrected the mean value of Wv for H = 2.0 m and 2.2 m.

Table 5 H vs. Wv

H (m)	Wv (kg/cm^2)	H (m)	Wv (kg/cm^2)
0.8	0.136	2.2	~~0.344~~ 0.350
1.0	0.170	2.4	0.361
1.2	0.204	2.6	0.387
1.4	0.238	2.8	0.413
1.6	0.272	3.0	0.438
1.8	0.306	3.2	0.463
2.0	0.340	3.4	0.487

Fig. 7 is shown as a graph for Table 5 values.



(2) Load (pressure) by Truck

Coefficient of impact

The coefficient of impact varies linearly with the cover depth.

For the range of calculation, it is :

H (m)	i	H (m)	i
0.8	0.5	2.2	0.43
1.0	0.5	2.4	0.41
1.2	0.5	2.6	0.39
1.4	0.5	2.8	0.37
1.6	0.49	3.0	0.35
1.8	0.47	3.2	0.33
2.0	0.45	3.4	0.31

Wt for 1 truck

Substituting H and i into the formula, Wt for 1 track is calculated and tabulated as shown on Table 6.

Table 6 H vs. Wt for 1 Truck

H (m)	Wt (kg/cm^2)	H (m)	Wt (kg/cm^2)
0.8	0.346	2.2	0.075
1.0	0.257	2.4	0.064
1.2	0.199	2.6	0.055
1.4	0.158	2.8	0.048
1.6	0.129	3.0	0.042
1.8	0.106	3.2	0.037
2.0	0.088	3.4	0.033

Wt for 2 trucks

The same steps make Table 7.

Table 7 H vs. Wt for 2 Turcks

H (m)	Wt (kg/cm^2)	H (m)	Wt (kg/cm^2)
0.8	0.404	2.2	0.105
1.0	0.312	2.4	0.092
1.2	0.249	2.6	0.081
1.4	0.205	2.8	0.071
1.6	0.171	3.0	0.063
1.8	0.144	3.2	0.057
2.0	0.123	3.4	0.051

(3) Sum of Loads

From the forgoing results, the depth versus the sum of loads can be tabulated for the case of 1 truck and 2 trucks. It is shown on Table 8 and 9.

Table 8 H vs. (Wv + Wt), for 1 truck

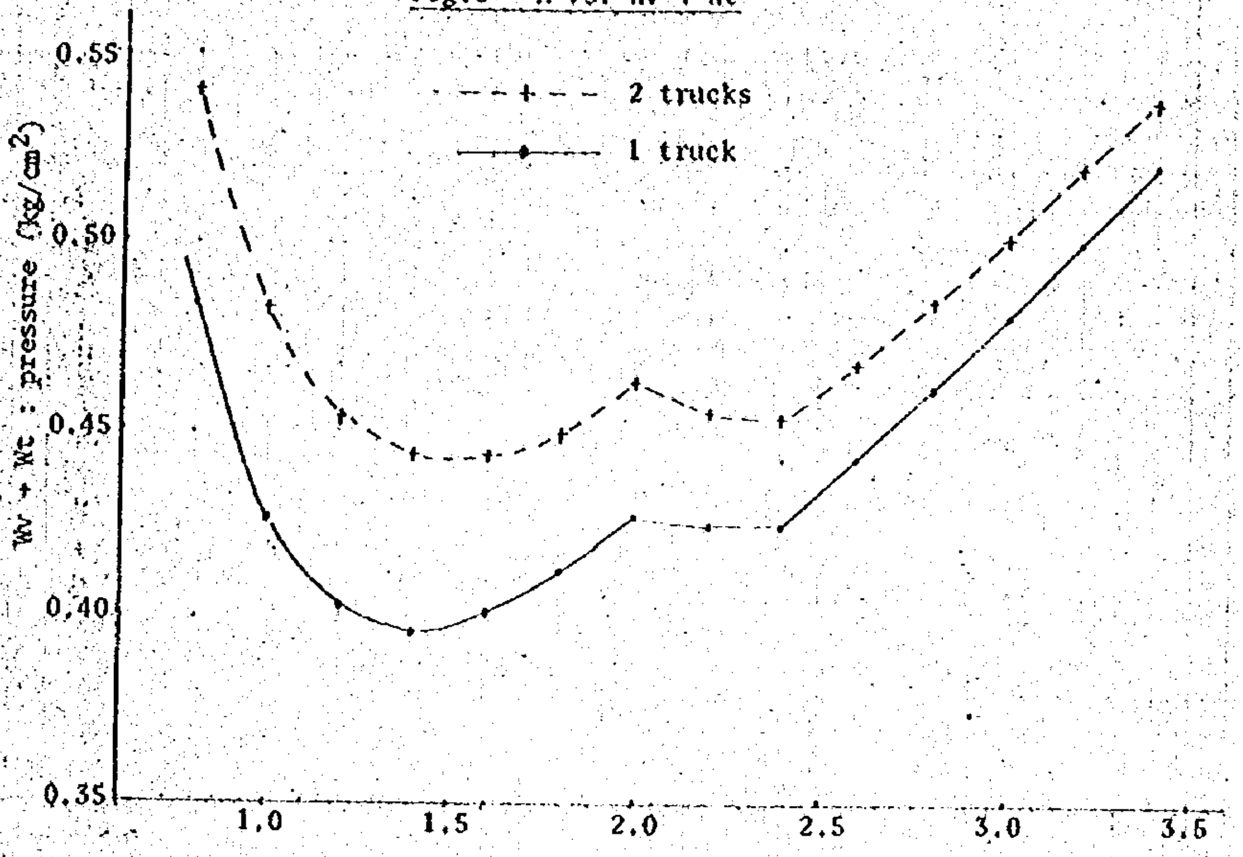
H (m)	Wv + Wt (kg/cm^2)	H (m)	Wv + Wt (kg/cm^2)
0.8	0.482	2.2	0.425
1.0	0.427	2.4	0.425
1.2	0.403	2.6	0.442
1.4	0.396	2.8	0.461
1.6	0.401	3.0	0.480
1.8	0.412	3.2	0.500
2.0	0.428	3.4	0.520

Table 9 H vs. (Wv + Wt), for 2 trucks

H (m)	Wv + Wt (kg/cm^2)	H (m)	Wv + Wt (kg/cm^2)
0.8	0.540	2.2	0.455
1.0	0.482	2.4	0.453
1.2	0.453	2.6	0.468
1.4	0.443	2.8	0.484
1.6	0.443	3.0	0.501
1.8	0.450	3.2	0.520
2.0	0.463	3.4	0.538

Table 8 and 9 are pictured as Fig. 8

Fig.8 H vs. Wv + Wt



Pay attention to the following features :

* In case of 2 trucks, the minimum pressure, about 0.45 kg/cm^2, covers a wide range of H from 1.2 to 1.4 m
* The minimum pressure of about 0.40 kg/cm^2 covers a narrow range of H, 1.2 to 1.6 m, for 1 truck.
* The cover depth less than 1 m is not safe.
* Though the pressure increase linearly after 2.4 m with the cover depth, the curve is not steep as that in the shallow range less than 1 m.

8.3. Calculation of Stress and Deflection

(1) Internal Pressure

The stress caused by the internal pressure is tensile one and for 7.0 kg/cm^2 of the normal pressure and 10 kg/cm^2 of the normal plus water hammer pressure, the stress can be compared to the allowable value as follows :

Normal pressure

Wall Thickness (mm)	Stress (kg/cm^2)	Allowable Stress (kg/cm^2)
10.3	458	1,400
11.1	426	1,400
11.9	397	1,400

Normal plus Water Hammer Pressure

Wall Thickness (mm)	Stress (kg/cm^2)	Allowable Stress (kg/cm^2)
10.3	655	2,100
11.1	608	2,100
11.9	567	2,100

For the internal pressure, the pipe is quite safe.

(2) External Loads

Deflection

The deflection is calculated by the formula :

$$\Delta x = \frac{2Kx(Wv + Wt)R^4}{EI + 0.061E'R^3}$$

When the values Kx = 0.096, R = 135/2 = 67.5, E = 2.1 x 10^6, E' = 14 are substituted in the formula, it is transformed to :

$$x = \frac{2 \times 0.096 \times 67.5^4}{2.1 \times 10^6 \times \frac{1}{12} \times t^3 + 0.061 \times 14 \times 67.5^3} (Wv+Wt)$$

$$= \frac{22.78}{t^3 + 1.50} (Wv + Wt) = \alpha (Wv+Wt)$$

Bending Stress

Similarly, the original formular is turned into :

$$\sigma_b = \frac{5.72\ t^3 + 1.44}{t^5 + 1.50\ .\ t^2} \times 10^3\ (Wv + Wt) = \beta . (Wv + Wt)$$

Table of

In the above two formular, α and β are the functions of t. For t = 1.03, 1.11, 1.19 cm they are :

Table of α and β

t	α	β
1.03	8.786	2.796×10^3
1.11	7.944	2.621 x "
1.19	7.152	2.457 x "

(3) Calculation of Deflection

Two tables of Wv + Wt, Table 8 and 9, for 1 and 2 trucks are already prepared.
Substituting the values into the formula of deflection, the deflections can be calculated.
Here, the deflections at H = 0.8, 3.4 m and at the minimum (1.4 m) are shown and compared to the tolerance, as shown below :

Deflections (cm)

Thickness	for 1 truck			for 2 trucks			Tolerance
(mm)	0.8m	1.4m	3.4m	0.8m	1.4m	3.4m	[illegible]
10.3	4.2	3.5	4.6	4.7	3.9	4.7	6.75
11.1	3.8	3.1	4.1	4.3	3.5	4.3	6.75
11.9	3.4	2.8	3.7	3.9	3.2	3.8	6.75

For all cases, the deflections are well within the tolerance.

(4) Calculation of bending stress

The allowable stress is given as 1,400 kg/cm^2 here.
As quoted before, $\sigma t = \beta$ (Wv+Wt) and β are already calculated.
Then substituting σa = 1,400 into σt, the limit of (Wv+Wt) can be calculated for the three sizes of thickness . That is :

$$Wv + Wt \leqq 1,400/\beta$$

thickness (mm)	β* x 10^3	$1,400/\beta$ (kg/cm^2)
10.3	2.796	0.501
11.1	2.621	0.534
11.9	2.457	0.569

* is non-dimentional.

The results are to be consulted with Table 8 and 9, and the two tables are incorporated to be made into Table 10 for covenience.

Table 10 H vs. Wv + Wt

H (m)	for 1 truck	for 2 trucks
0.8	0.482	0.540
1.0	0.427	0.482
1.2	0.403	0.453
1.4	0.396	0.443
1.6	0.401	0.443
1.8	0.412	0.450
2.0	0.428	0.463
2.2	0.425	0.455
2.4	0.425	0.453
2.6	0.442	0.468
2.8	0.461	0.484
3.0	0.480	0.501
3.2	0.500	0.520
3.4	0.520	0.538

Findings

* 10.3 mm exceed the allowable stress in the case of H = 3.4 m for 1 truck and H = 0.8, 3.0, 3.2, 3.4 m for 2 trucks.
* 11.1 mm exceed the allowable stress in the case of H = 0.8, 3.4m for 2 trucks.
* 11.9 mm are aceptable for all H of both 1 and 2 trucks.

9. DISCUSSION

So far it was found that :

* For the internal pressure, the three sizes are all acceptable.
* For the deflection, the three sizes are all acceptable.
* For the bending stress, 11.9 mm is all acceptable.
* With H = 0.8 m and for 2 trucks, 10.3 and 11.1 mm raises problem. The cover depth less than 1 m shall be clearly prohibited in the specification, despite the thickness.
* With H = 3.4 m and for 2 trucks, 11.1 mm exceed the allowable stress by 0.7% or 51 kg/cm^2. Practically it is tolerable.
* To examine about 10.3 mm, Table 11 is prepared, putting the figure of stress in it.

Table 11 H vs. Wv + Wt, for 10.3 mm(kg/cm^2)

H (m)	for 1 truck	for 2 trucks
0.8	1.348	1.510
1.0	1.194	1.348
1.2	1.127	1.267
1.4	1.107	1.239
1.6	1.121	1.239
1.8	1.152	1.258
2.0	1.197	1.295
2.2	1.188	1.272
2.4	1.188	1.267
2.6	1.236	1.309
2.8	1.289	1.353
3.0	1.342	1.401
3.2	1.398	1.454
3.4	1.454	1.504

As in the case of 11.1 mm, if 50 kg/cm^2 excess of the allowable stress is tolerated, there remains only the case of H = 3.4 m and 2 trucks.

A solution is that the cover depth is to be clearly specified less than 3.2 m.

Conclusively, 10.3 mm can be used by specifying the cover depth between 1.0 m and 3.2 m and by allowing the stress up to 1,450 kg/cm^2.
Needless to say, 11.1 mm can be usable with more reliability than with 10.3 mm.

E', compaction

E', the coefficient coming from the nature of earth and the degree of compaction, means very much in the bending stress. The formula to calculate σt is transformed into :

$$b = \frac{5.72\ t^3 + 0.103\ E'}{t^5 + 0.107\ E'.t^2} \times 10^3\ (Wv + Wt)$$

The calculation so far was made on E' = 14 kg/cm^2.
For comparison, the case of E' = 7, 14, 28 kg/cm^2 will be shown as below :

Table of on E' Values

t (cm)	E' (kg/cm^2) 7	14	28
1.03	3.568 x 10^3	2.796 x 10^3	2.106 x 10^3
1.11	3.275	2.621	1.991
1.19	3.005	2.457	1.890

As it can be understood, β is a coefficient which affects the value of bending stress proportionaly. If instead of E' = 14 kg/cm^2 E' = 28 kg/cm^2 is applied, the stress will decrease by about 24%. The importance of thorough compaction cannot be over-emphasized. With 10.3 and 11.1 mm pipe, the specification of compaction shall be fully observed and supervised during the construction.

10. CONCLUSION

With the forgoing calculation and discussion, the author concludes:

* 10.3 and 11.1 mm wall thickness can be used for the project. The feasibility study's recommendation is 11.9 mm.

* Good workmanship is necessary when the two sizes are used.

NO.13 REPORT ON RECEIVING WELL AND RECEIVING RESERVOIR

January, 1982

Prepared by A. MIYAKE
Structural Engineer
(Water Facilities)
Detailed Design Team

CONTENT

1. Introduction

2. Object of Receiving Well and Receiving Reservoir

2.1. Receiving Well
2.2. Receiving Reservoir

3. Receiving Well

3.1. Steadiness of Divided Flow

(1) Transmission System to Sattahip, Industrial and Urban Water for Sattahip

(2) Urban Water for Mab Ta Pud

(3) Industrial Water for Mab Ta Pud

(4) Steadiness of Downstream Flow

3.2. Size and Structure of Receiving Well

4. Receiving Reservoir

4.1. Size

1. Introduction

This paper is to determine the size and structure of the Receiving Well and the Receiving Reservoir in Mab Ta Pud.

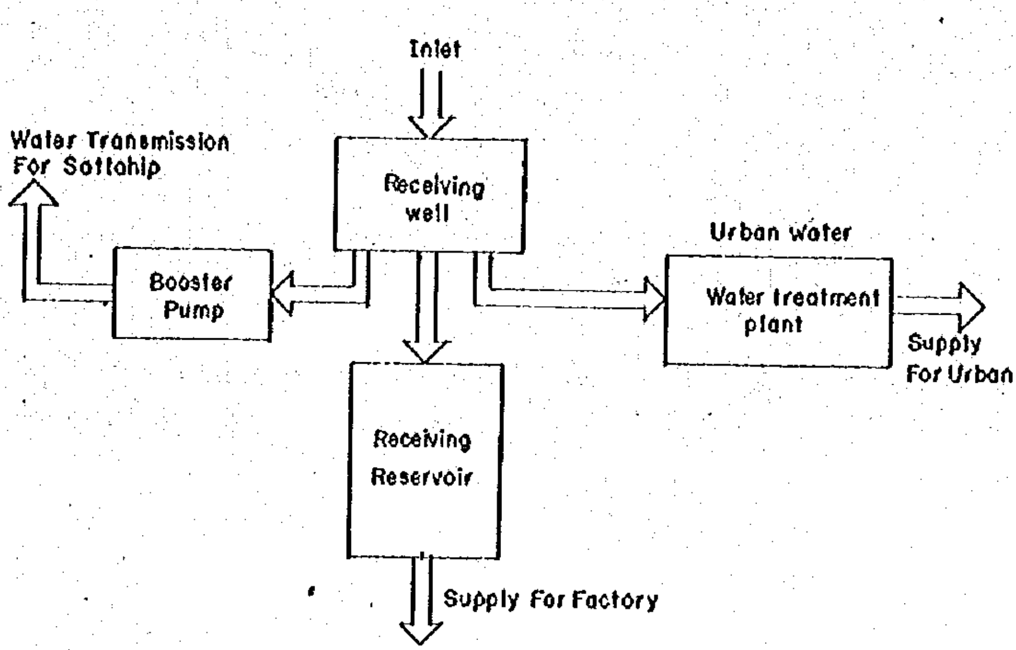
2. Object of Receiving Well & Receiving Reservoir

2.1. Receiving Well

The receiving well's existence is to stabilize the flow's turbulance which comes in from the transmission line and make metering, controlling and dividing the flow easy and accurate.

The receiving well in Mab Ta Pud is to devide the flow into 3 usages, that is, the industrial and urban water for Sattahip area, the industrial water for Mab Ta Pud area and the urban water for the same area, as shown in Fig. 1.

Fig. 1. Division of Flow



2.2. Receiving Reservoir

Two major purposes are quoted here. The first is storing water to meet the needs in the cases of power failure, erroneous operation. The second is clearing water in case of extraordinary high turbidity.

3. Receiving Well

3.1. Steadiness of Divided Flow

To decide the structure of receiving well, it is necessary to study about the downstream conditions under which the divided flow is to be drawn from the well, the steadiness .

(1) Transmission System to Sattahip, Industrial & Urban Water for Sattahip.

The flow to be branched off from the receiving well and be sent to Sattahip, can be either steady or unsteady.

Steady

The pumps which lift water to the head tank are run so that the water level of the head tank is kept constant at a certain height. The flow rate is controlled by the flow control valve at the pumps delivery and by changing the number of running pumps. It is shown on Fig. 2. on the following page.

Unsteady

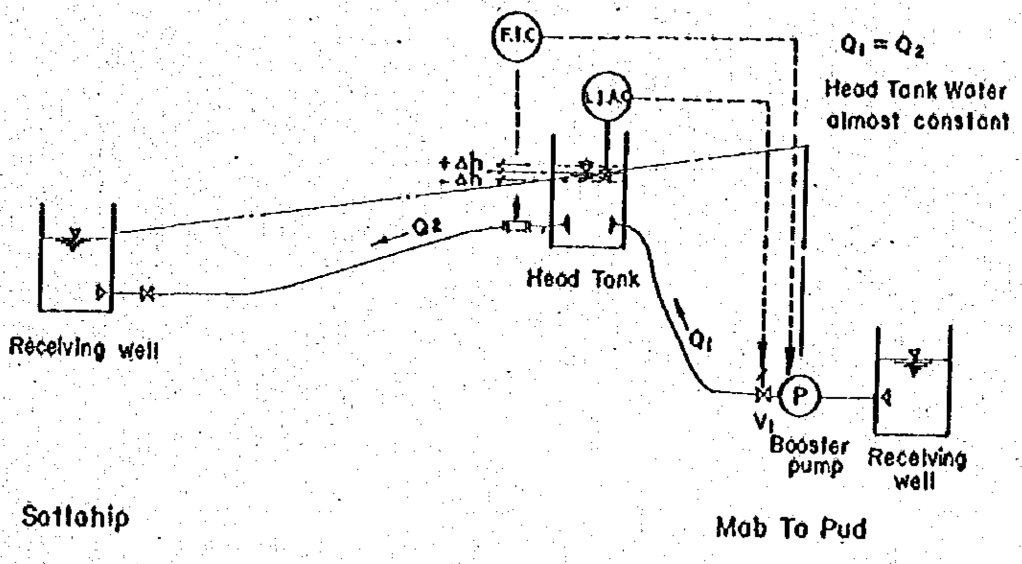
The head tank shall have suitable capacity. Then the pumps are controlled by the high and low water level of the head tank. Another requirement is that the valve at Sattahip is in the position of approximate rate of the set flow.
It is shown on Fig. 3. on the next to the following page.

The control system by unsteady flow is the one which is to be adopted between Dok Krai pumps and Mab Ta Pud receiving well. For operational convenience, the same system will be preferable. Moreover, the system is safer in case of erroneous operation at the downstream end (Sattahip here) as it is discussed in the other report.

Hereafter, the discussion will be along the control system by unsteady flow as the flow out of the receiving well is unsteady.

(to be continued to 2 pages after)

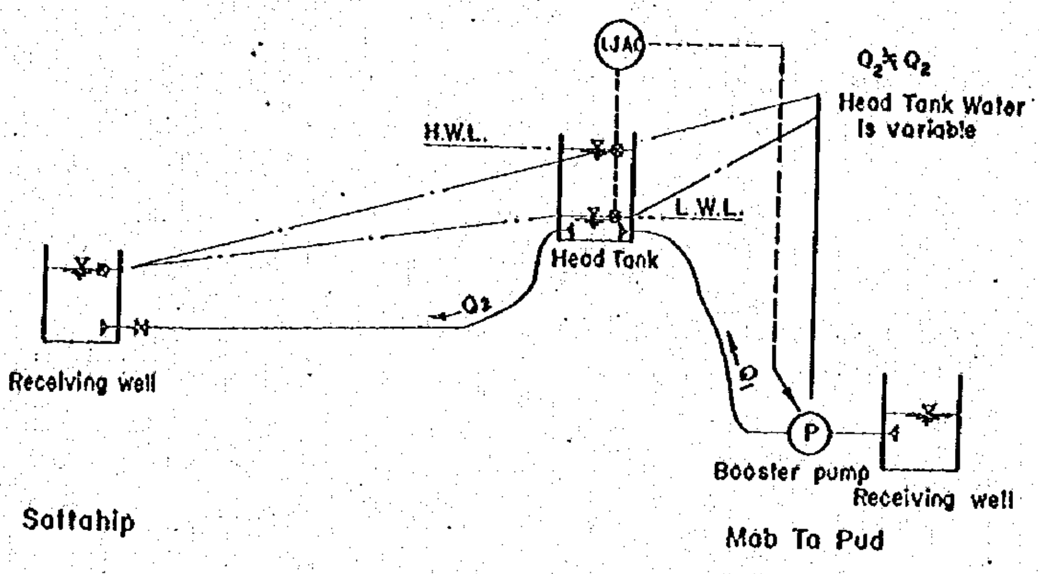
Fig. 2 Steady Flow



Operation of this system is feasible by both manually and electrically. The manual operation will start from an approximate control of the delivery valve and watching the level in the head tank, the operator adjusts the valve until an equilibrium is attained and both the level and the flow become steady. In case a change occurs in the system and the level takes extra-high or low position, the alarm shall be activated and correction be made.

The figure shows that by coordinating the control valve and the pump with the flow and the level meter electrically, the said manual operation can be turned into the automatic operation.

Fig. 3 Unsteady Flow



The system also can be operated both by manually and electrically. When the level in the head tank rises and reaches to the high water level because of the excess flow over the set rate at the end, an alarm is given to the operator to stop one pump of the running pumps. Then the level falls down to the low water level, one pump is started by alarm.
The system also can be worked automatically by connecting the pumps and the high and low level with electric circuit.

(continued from 2 pages before)

(2) Urban Water for Mab Ta Pud

The water for urban use must be treated in filtration plant. The filtration plant is usually operated with steady rate as it requires proper and exact feeding of chemicals in the process. To cover the remarkable hourly fluctuation of treated water demand, clean water tank (reservoir) of sufficient capacity must be prepared.
The flow out of the receiving well, therefore, is steady.

(3) Industrial Water for Mab Ta Pud

The receiving reservoir's first purpose mentioned in 2.2. can be translated to that it can meet changing condition of the downstream demand. The supply to industrial plants are preferably to be steady but is assumed to be unsteady for the time being.

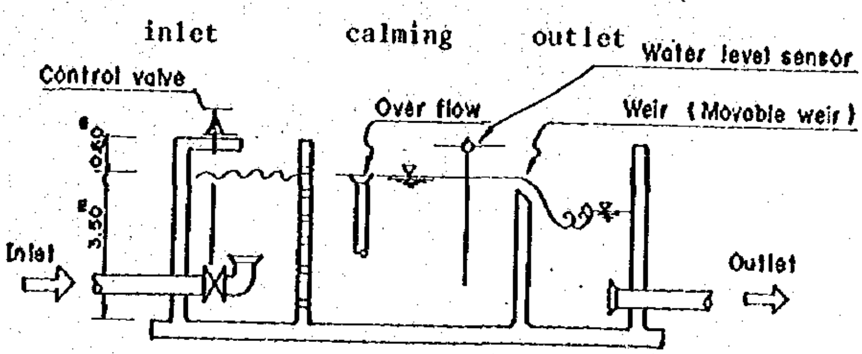
(4) Steadiness of Downstream Flow

Summarizing (1) to (3), the two flows to Sattahip and Mab Ta Pud industries are found unsteady and the one to Mab Ta Pud filtration plant is found steady.

3.2. Size and Structure of Receiving Well

The receiving well in general consists of 3 parts, the inlet, the calming and the outlet.
The inlet and the calming is divided by a perforated wall and the calming and the outlet is divided by a wall with a weir on the top.
The inlet and the following calming parts have a constant level, being controlled by the inlet valve. When the measurement of flow is necessary at the receiving well, it can be done by reading the overflow depth on the weir and by calculating the flow, after introducing the depth into a specific formula.
If the flow is to be controlled, it can be done by adjusting the overflow depth with a movable weir.
A typical profile is shown on Fig. 4.

Fig. 4 Profile of Receiving Well

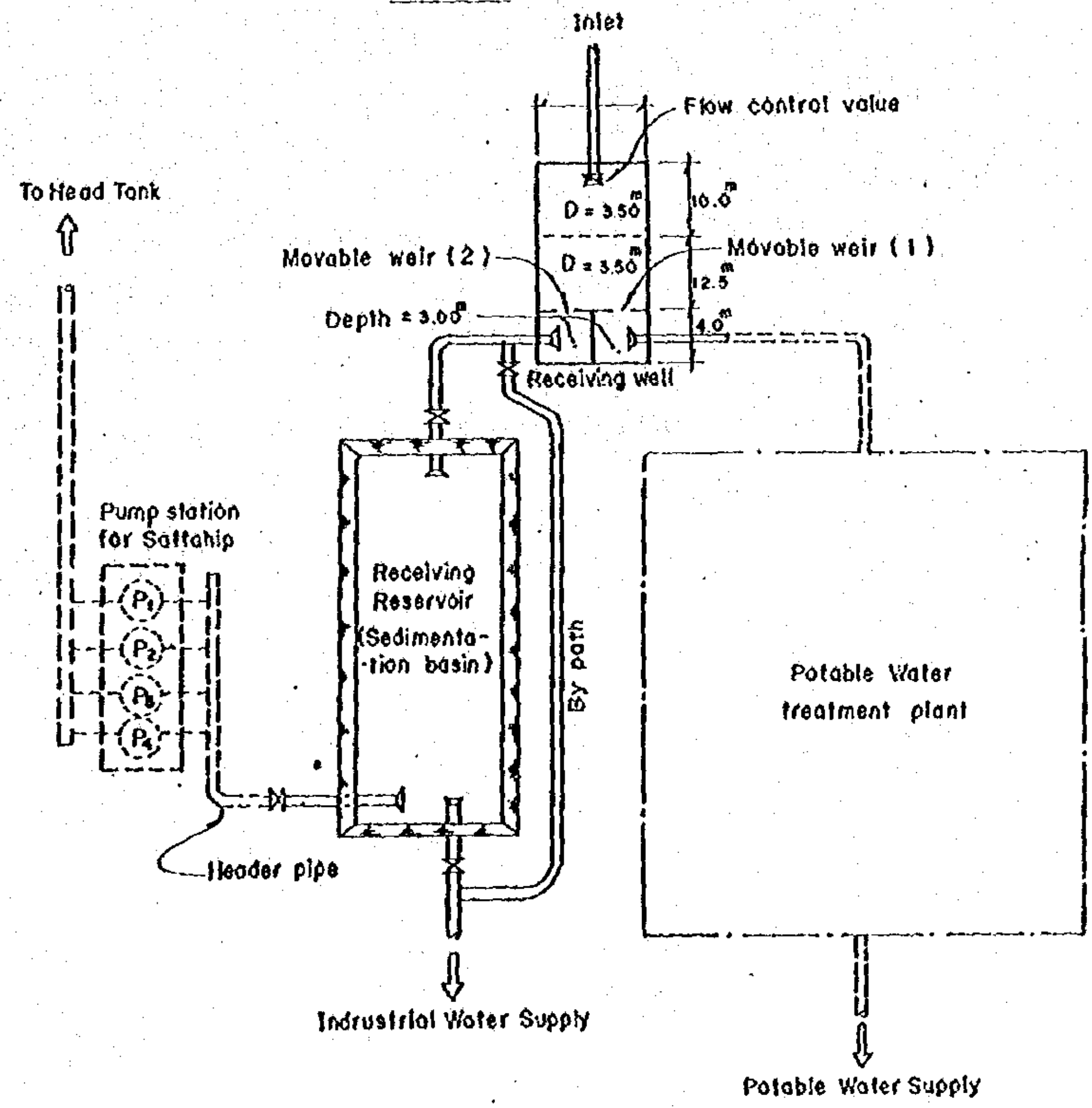


The capacity expressed in terms of the retention time is mostly more than 1.5 minutes with the inlet and the calming parts put together, although the downstream condition is to be considered in determining the capacity.

Here, the existence of receiving reservoir can meet worries due to accidents and the receiving well does not need any extra capacity. The capacity is decided for 5 minutes' retention time.
The dimentions are to be 10 m Breadth x 22.5 m Length x 3.5 m Depth with 787.5 m^3 capacity.

As explained before in 3.1. the two flows to Sattahip and Mab Ta Pud industries are unsteady while that to Mab Ta Pud filtration plant is steady. If these flows of different nature is taken out of the same part of receiving well, the inlet valve must be controlled ceaselessly and delicately so that the level is kept constant.
To remove the difficulty, the flow shall be divided as shown on Fig. 5.

Fig. 5



4. Receiving Reservoir

4.1. Size

The roles of receiving reservoir, storage and treatment, are discussed already.

To determine the capacity from the standpoint of storage is difficult as it is the bigger the better. From the standpoint of treatment, it will need at least 3 hrs retention time.

3 hrs is the minimum in designing the sedimentation basin in a filtration plant.

The capacity will be calculated;

Capacity = 3 hrs. x (Sattahip plus Mab Ta Pud industries)

= 21,385 m^3 or approximately 21,000 m^3

Sattahip ; 0.82 m^3/sec Mab Ta Pud industries ; 1.16 m^3/sec

4.2. Structure

The receiving reservoir can be built of either reinforced concrete or earth material. An earth basin has long been considered not suitable in case where remarkable leakage of water is foreseen. In recent years however, various methods have been invented and carried out in many cases to line the inside of dam and the bottom of basin with rubber sheet or plastic film to prevent leakage, the said defect.

Having the above in mind, a comparative study of using reinforced concrete and earth material with rubber sheet has been made about the cost of constructing the receiving reservoir. The finding is the reinforced concrete is rather costly than the earth material. *

(1) Plastic film is not resistent to ultra-violet ray and must be embedded deep enough below the surface of earth.

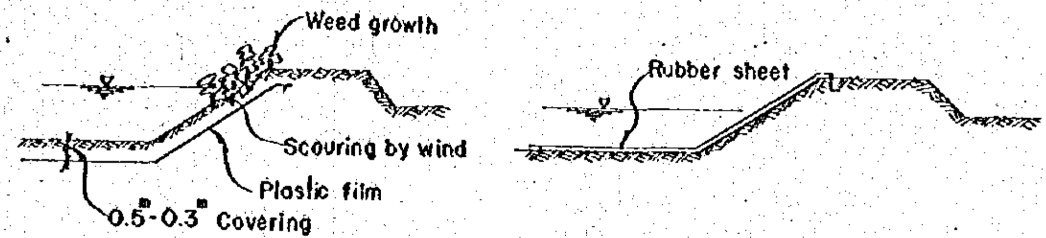
(2) The surface earth covering the plastic film, especially near the water level, is continuously scoured off by the ripple and wave coming from the wind and flow turbulance.

* See attached sheet

(3) The earth above the plastic film is wet and help weed's growth. It is not favourable from the maintenance points.

With the above discussion, the choice is an earth basin, covered with rubber sheet on the surface.

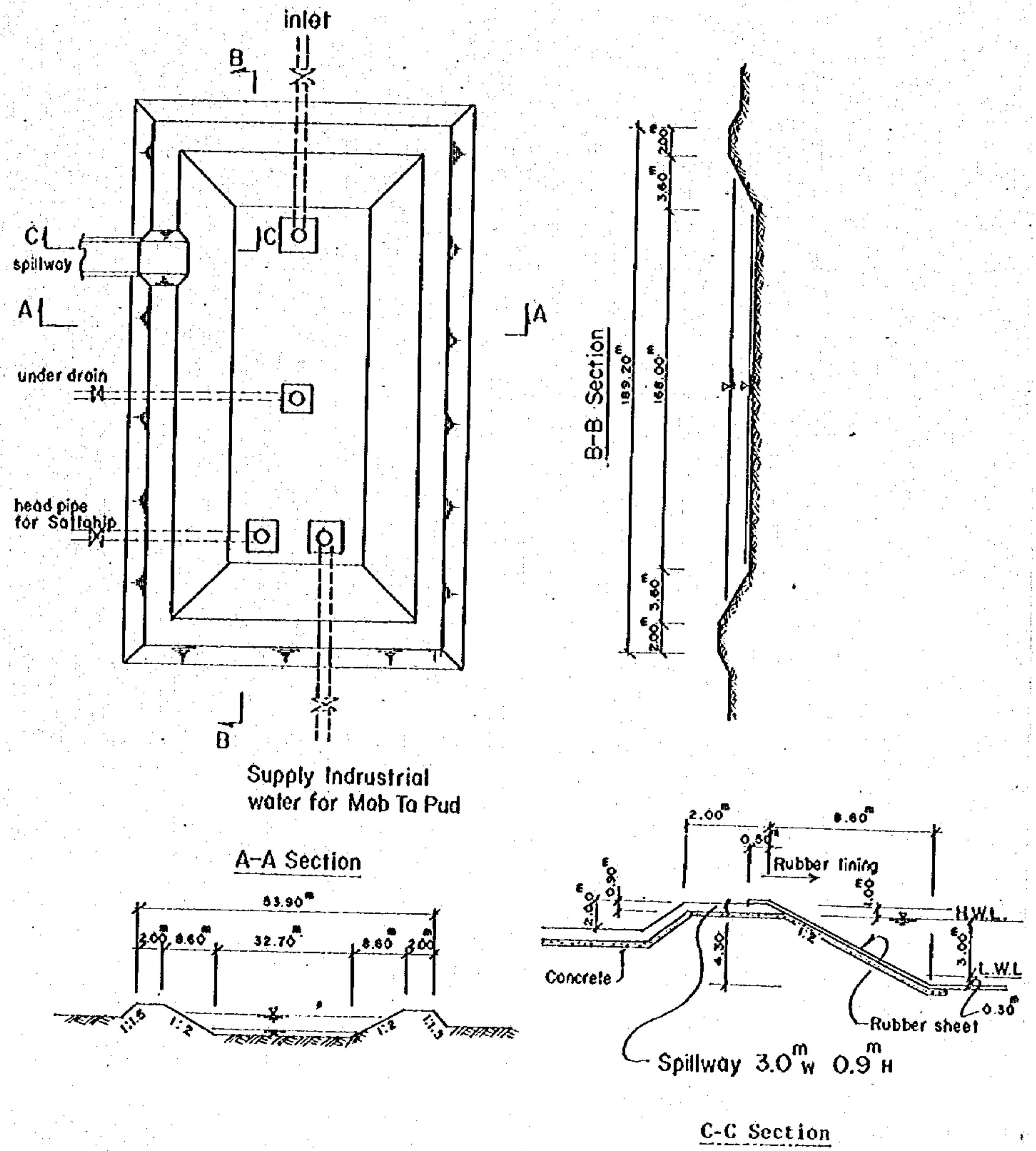
Fig. 6 Plastic Film and Rubber Sheet



4.3. Shape and Dimensions

The shape and dimensions are shown on Fig. 7.

Fig. 7 Shape and Dimension of Receiving Reservoir



APPROXIMATE CONSTRUCTION COST
OF RECEIVING RESERVOIR

Material	Description	Cost 1000 $	Rank
Concrete	wall, bottom	423	7
Concrete and Earth	concrete wall, rubber sheet bottom	379	6
Earth and Rubber Sheet	outside slope with grass planting	269	3
Earth and Rubber Sheet	outside slope with rock filling	311	5
Earth and Plastic Film	outside slope with grass planting	98	1
Earth and Plastic Film	outside slope with rock filling	186	2
Earth and Plastic Film	inside and outside slope with rock filling	302	4

NO.14 REPORT ON SURVEYING

FEBRUARY 1982

Prepared by KAZUHIRO ISHIZUKA
Surveyor
Detailed Design Team
JICA

CONTENT

1. INTRODUCTION

2. RESULTS

2.1. Maps for Practical Use

2.2. Maps for Reference Use

3. SURVEYER'S COMMENT

3.1. Point of Reference for Level Survey

3.2. Point of Reference for Coordinates and Plan Survey

3.3. Error of Survey

3.4. Contour

3.5. For Design of River Crossing

4. REFLECTION

1. INTRODUCTION

On Novmber 26th 1981, a letter of request was submitted to RID to conduct the topographic survey.
The letter was attached with the specification difining the purpose, survey sites, period of work, content of work, etc.

Responding the request, RID provided the counterparts, a survey team, to work with the JICA surveyer.
The survey work, starting from December 2nd 1981, continued until the middle of February 1982 as it had been scheduled initially.
The surveyer's report hereby contains the results of survey and his comments on them.

2. RESULTS

Two list of maps resulted from the survey is shown below. They are divided on the practical use and the reference use. All the maps are by A1 size.

2.1. Maps for Practical Use

No.	Title	Scale	No. of map	Original	Master Print	Remarks
1	Pipeline Route.	1/1000	23	RID	DD team	
2	Dok Krai Area	1/1000	1	"		
3	D.K. Res. Sounding	1/1000	2	"	Tokyo	6 more in 2.2
4	Head Tank Site	1/500	1	"	"	
5	Receiv. Well Site	1/500	1	"	"	
6	Crossing 36, 3	1/500	1	"	"	Highway 36. 3
		Total	30			

2.2. Maps for Reference Use

No.	Title	Scale	No. of maps	Original	Master Copy	Remarks
7	Dok Krai Res. Sounding	1/1000	6	RID	Tokyo	2 more in 2.1.
8	Borehole Point	1/1000	1	Tokyo		for locating points from land
9	Longitudinal Section of Pipeline	1/1000	33	RID		
10	Cross Section of Pipeline			RID		100m interval at Feas. study 50 m interval at D.D.
11	Rivers along Pipeline	1/500	23	RID		

3. SURVEYER'S COMMENT

3.1. Point of Reference for Level Survey (No. 1 & 2)

The point of reference for the level survey is PTT. A-8 Bench Mark, the elevation 54,776.
The point is closely located to RID 222393-OK-(136-48-31)

3.2. Point of Reference for Coordinates and Plan Survey (No.1 & 2)

RID 22393-OK-(136-48-32), the elevation 55,063.

3.3. Error of Survey

It was done by the traverse survey.
The error is 1/10000.
The error of the level survey is negligibly small.

3.4. Contour

No. 4, 5, 6 are with 0.25 m contour line and the others are with 1.0 m contour line.

3.5. For Design of River Crossing

No. 11 shall be refered.

4. REFLECTION

Frankly admitting, the survey work has a disadvantage. In ordinary way, the pipeline alignment preceeds the survey work but here it is reversed. And ordinarily the longitudinal section (profile) shall be surveyed along the pipeline alignment but here it is impossible.

In Japan, the alignment will be set on the maps and the longitudinal section will be made from maps No. 1.

For practical purpose the materials thus worked will be useful, however the surveyer must comment about it.

NO.16 REPORT ON ELECTRICAL FACILITIES

FEBRUARY 1982

Prepared by HIDEAKI NAKAO
Electrical Engineers
Detailed Design Team
JICA

CONTENT

1. INTRODUCTION

2. POWER SUPPLY AND DEMAND

3. RECEPTION AND TRANSFORMATION AT DOK KRAI

3.1. Demand
3.2. Voltage
3.3. 1 System Reception
3.4. Transformation

4. ENGINE-DRIVEN GENERATOR FOR EMERGENCY USE

4.1. Background
4.2. Requirements and Conditions
4.3. Generated Voltage and Engine Cooling
4.4. Sturting up of Engine, Restoration of Power Supply

5. UNINTERUPTABLE POWER SOURCE (U.P.S.)

5.1. Role of U.P.S.
5.2. Components and Function
5.3. Loads
5.4. Battery and Rectifier

6. FLOW METER

7. LEVEL METER

7.1. Measuring Points and Purpose
7.2. Selection of Type

8. INSTRUMENTATION AND CONTROL

8.1. Flow Sheet
8.2. Emergency

9. COLLECTION OF INFORMATIONS IN THAILAND

9.1. Manufacturing of Electrical Equipments

9.2. Standards in Thailand

9.3. Consideration

9.4. Flow and Level Meter

9.5. Spare Parts

9.6. Transportation

10. CONTACTS WITH OTHER AGENCIES

10.1. Provincial Electricity Authority

10.2. Ministry of Interior

10.3. Telephone Organization of Thailand

10.4. Ministry of Telecommunication

1. INTRODUCTION

The electrical equipments in the project can be divided into three parts, the power reception and supply, the control and instrumentation and the communication.
The power reception and supply in Dok Krai is most critical in the whole system and needs an intensive and extensive study. Consequently it occupies the largest part of this report.
The control and instrumentation here is, so to say, simple as the objects of sensing are few, the feed back and sequence control is rather simple.
The communication, use of telephone and radio to convey informations vocally, is important. Not only the hardware requires reliability but also the software, mostly the capability of operators, must be upgraded.

Translator's (WAKAMOTO) note:
As the translator's ability for both understanding and expressing is limited, major items of the original report will be taken up here.

2. POWER SUPPLY AND DEMAND

(1) Dok Krai

A 22 kv transmission line from Aophai Substation runs close to Dok Krai Reservoir.

The power demand of 3,300 kw may be supplied from the line.

(2) Head Tank

A low voltage line runs along Route 3191 and it can feed the head tank's consumption of 10 kw.

(3) Receiving Well and Reservoir

A low voltage line runs along Route 3 and it can feed the receiving well and reservoir's consumption of 20 kw.

(4) During Construction

The locations (1) to (3) needs 20-30 kw supply during construction stage.

(5) Confirmation

The detailed design team shall confirm Provincial Electricity Authority of Thailand about the above said matters.

3. RECEPTION AND TRANSFORMATION AT DOK KRAI

3.1. Demand

The maximum power demand, being assumed that individual loads occur simaltaneously, is shown below :

Table of Loads

Item	Capacity (kw)
Intake pumps 5 running, 1 stand-by	600 x 5
Delivery Valves 5 no.s	2.2 x 5
Floor drainage pump	3.7
Exhaust fans, air conditioner	30
Other anxiliaries	30
Lightings	45
Total	3,119.7

3.2. Voltage

As mentioned in 2., the size of demand may be fed from the 22 kv, 50 Hz transmission line.

3.3. 1 System Reception

When plural supply systems coming from different sub-stations exist and where the supply is of importance, the plural supply lines must be utilized.

Though the pumps at Dok Krai are apparently important, the area is covered by only one supply system of Ao-Phai Substation.

Inevitably here, the reception is on 1 system.

3.4. Transformation

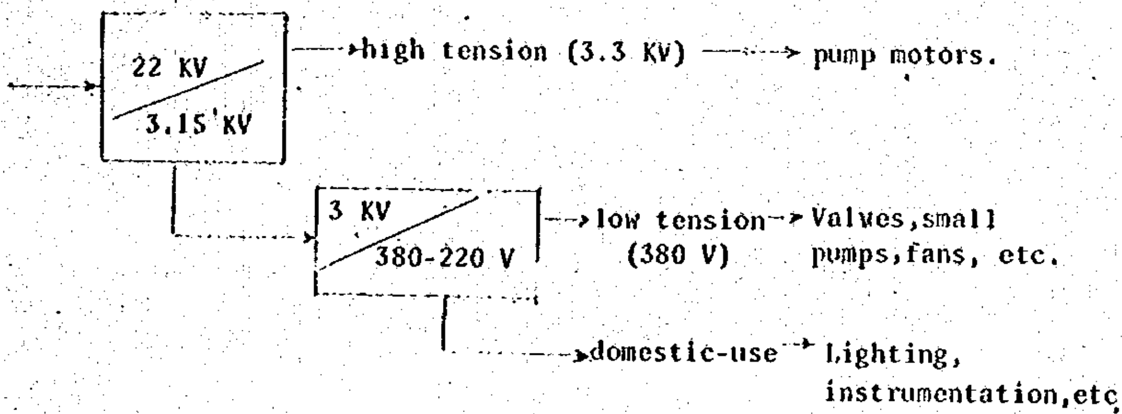
(1) Number of Main Transformer

Nowadays a transformer is reliable and practically free from failure. However, to make the maintenance easier two transformers in parallel use are decided.

(2) Transformation of Voltage

The use at Dok Krai can be classified into, the high tension for pump motors, the low tension for pump's valves, flow control valves, floor drainage pumps, exaust fans and other power loads, the domestic-use voltage for lighting, sockets, instrumentation, etc. It is done as on Fig.1.

Fig.1 Transformation and Uses.



The Voltages are 3.3 kv, 380 v and 220 v in accordance with Thai standards. The former two are 3 phases and the last single phase.

(3) Type of Transformer Station

Three types are concidered, Outdoor and Open, Outdoor and Closed, Indoor and Closed.

The open type is the one which is constructed by using outdoor type gadgets and steel material at the site and the closed type is the one enclosed in a steel cubicle (cabinet) with indoor type gadget and manufactured at the factory.
Comparing the three, the Indoor and closed type is chosen on the safety, easiness of operation and maintenance, although it is most expensive of the three as it needs housing.

(4) Use of Two Transformers, 22 kv/3.15 kv

The requirement assumed here is that a transformer can stand the load consisting of 3 pump motors, low tension uses and lighting, etc.
When 5 pumps are run, two transformers shall be used, and then if a transformer fails, 3 pumps can be run. Each transformer's capacity is 3,000 KVA with some allowance.

(5) Capacity of Low Voltage Transformer, 3,000 v/380-220 v

With the transformer, both 380 v x 3 phase and 220 v x single phase can be taken out.
With some allowance, the capacity is 200 KVA.

4. ENGINE-DRIVEN GENERATOR FOR EMERGENCY USE

4.1. Background

The situation of electricity supply in Dok Krai area is relatively good, with only 2 to 3 times' failure in a year. For the power failure, a reservoir for 3 hrs' (at the design flow rate) storage capacity is already prepared.

For further precaution, the detailed design team studied about the generator and found that to operate the pumps the generator's capacity must be in 1,000 to 2,000 kw, which is obviously unacceptable for reasons of the cost, difficulties in operation and maintenance.

After discussing the matters with Mr. Boonthai, RID, it was decided that an engine-driven generator should be prepared for the limited uses of demand.

4.2. Requirements and Conditions

Of the demands shown on the table in 3.1., the whole of lighting, floor drainage pump and exhaust fans and air conditioner and the half of other auxiliaries shall be supplied.

From the requirements, a generator of 125 KVA with an engine of 150.9 PS (French horse-power) shall be used.

The fuel is diesel gas oil by name, called in Thailand.

The estimated fuel consumption is 38.4 litter per hour and a 390 l fuel tank will meet about 10 hrs'run, which seems satisfactory for the purpose.

4.3. Generated Voltage and Engine Cooling

As the generator must be designed in relation with the public power supply, two alternatives can be conceived.

The first is the one of 3,000 V generated voltage.

The second is of 380 V with a 380 V/380-220 V transformer.

From anticipated cost and the operation and maintenance viewpoints, the second shall be the choice.

Of the two alternatives in cooling the engine, the air cooling shall be better than the water cooling.
The former is noisier than the latter and it can be tolerated at Dok Krai. Also the former needs no care about the quality of cooling water which is rather important with the latter.

4.4. Starting up of Engine, Restoration of Power Supply

Two methods, using compressed air and battery, can be conceived.
Generally the air method is easier in maintenance and may be preferable where the frequency of using generator is low in a case like this.
However in Thailand, the battery starting is widely used in 100-150 KV range.
The two will be studied further before making decision.

Usually the engine starts automatically when the public supply fails and after the power is restored, the operator stops the engine and switches to the public supply manually. Needless to say, the automatic operation can be also worked by the manual one.

When the public supply is restored, it shall be announced by the signal lamp on the control panel, together with the sound of buzzer or bell.

5. UNINTERRUPTABLE POWER SOURCE (U.P.S.)

5.1. Role of U.P.S.

In case of the power failure, automatically the diesel engine-driven generator starts up.
However, it normally takes 1-2 minutes till the generator reaches the full capacity and it may take more time in case of unexpected malfunctions.
U.P.S. is to supply the electricity without interruption for both the normal and abnormal start-up operation of generator.
Once the generator works fully U.P.S. stops automatically.

5.2. Components and Function

U.P.S. is composed of battery, inverter, sensor and automatic switches.
The battery is automatically charged by the public supply and by the generator when it is working.
The inverter is to invert direct current of the battery to alternative current for the use of wirless, instrumentation and control panel.
The sensors and automatic switches are for sensing the power failure, generators' start of supply, power restoration and responding the conditions, switching to the necessary reactions.

5.3. Loads

(1) Alternative Current

The sum of consumption by alternative current is estimated about 2,000 VA and it leads to the imput of inverter as 27 A.

(2) Direct Current

The sum of direct current, including operating the high voltage circuit breaker, the inverter input and emergency lighting will amount about 117 A.

5.4. Battery and Rectifier

Comparing the characteristics of battery and application for the requirements here, the alkaline battery is better than the lead battery.
The choice is the alkaline battery with Nickel and Cadmium electrodes.
The rectifier is of 130 V - 70 A.

6. FLOW METER

Three types of flow meter are widely used in the waterworks facilities. They are the differential pressure, electromagnetic and supersonic, classified by the physical principle they employ in measuring.
After comparing the three on various factors, the decision is on the supersonic type because of the following reasons:

(1) It is not affected by the changes of water quality and turbidity.

(2) The pipe arrangement for it does not need a by-pass line.

(3) Easily maintained

(4) The cost is relatively low.

(5) The accuracy is relatively high.

7. LEVEL METER

7.1. Measuring Points and Purpose

The depth of water at the intake tower must be refered for the operation of pumps. As the depth can be converted to the dam's water level, it can be used as an information for wide use also.
The head tank's level shall be used for the pump operation.
The receiving well's level is for reading the quantity of stored water there and it can be used for the operation of pipeline system.

7.2. Selection of Type

(1) Intake Tower

If the sensing part is installed outdoors, the maintenance becomes apparently difficult. It must be indoors.
The diaphragm type of relatively low cost and high accuracy fits the condition.

(2) Head Tank

Due to the operation of pump, the level repeats ups and downs periodically.
Other requirements are:

* Outdoors installation
* Relatively high accuracy and low cost
* Less energy consumption. As the sensing of level is electrically connected to the pump's switch, the power for the level meter shall be transmitted from Dok Krai.

The float type is most suitable here.

(3) Receiving Reservoir

Approximate measuring is satisfactory and a simple float and reversely graduated gauge seems the choice.

8. INSTRUMENTATION AND CONTROL

8.1. Flow Sheet

The flow sheet of whole pipeline system instrumentation is shown on Fig. 2 of next page.
The control system and operation is discussed in other reports.

8.2. Emergency

(1) Communication

In case of emergency, the communication between stations is most urgent and important.
The public telephone must be installed at the three stations, Dok Krai, head tank and the receiving well.
Anticipating that emergency will jam the telephone and the wireless communication between Dok Krai and the receiving well becomes a requirement.

(2) U.P.S. at Receiving Well

U.P.S. shall be prepared at the receiving well to supply currency to the wireless transmitter-receiver and the emergency lighting.
Duration of the failure of power supply may be 48 hrs. at most and U.P.S must be designed accordingly.

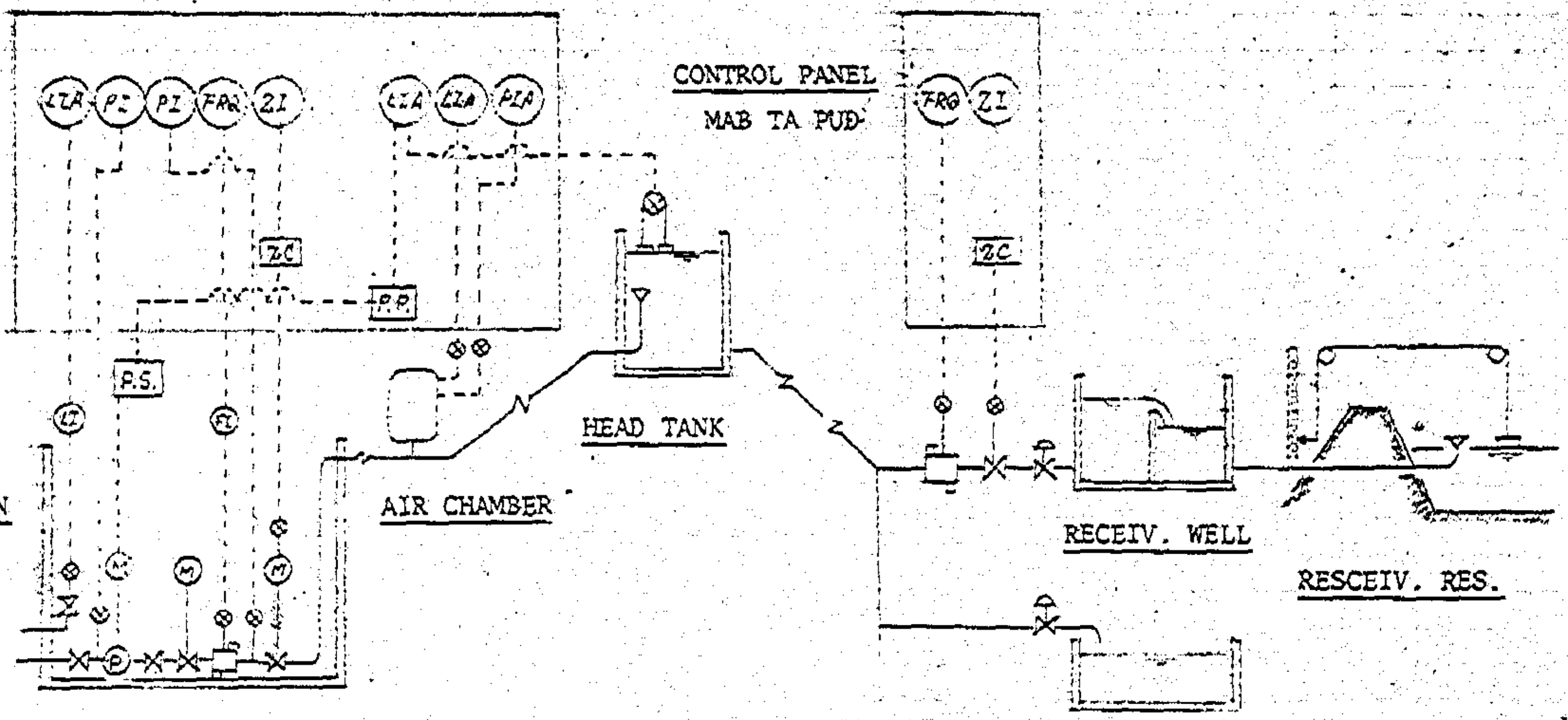
(3) Automatic - Closing Valve

Automatic - closing valve, in case of the power failure, closes automatically by a counter weight. The time for closing is 10 minutes due to the reduction of pressure and the time is long enough not to cause water-hammer.
Consequently, U.P.S. capacity does not cover for closing the valve.

(4) Walkie-Talkie

As is explained in other report, the walkie-talkie shall be prepared.

FIG.2 FLOW SHEET



SYMBOL

METER ON PANEL
METER ON SITE
TRANSMITTER
CONTROLLER
FAST-CLOSING VALVE
VALVE
SUPERSONIC WAVE SENSOR
MOTOR
PUMP

ABBREVIATION

LI	LEVEL INDICATER
LIA	" " WITH ALARM
PI	PRESSURE INDICATER
PIA	" " WITH ALARM
ZI	VALVE OPENING INDICATER
ZC	VLAVE OPEINING CONTROLLER
PP	PUMP PROGRAMMER
PS	PUMP STARTER

9. COLLECTION OF INFORMATIONS IN THAILAND

9.1. Manufacturing of Electrical Equipments

(1) Switchboard

In Thailand, the switchboard up to 20,000 Volt can be manufactured with the components, mostly imported from overseas. The appearance and workmanship are a little inferior to those of the imported.

(2) Price

Assuming that a switchboard's cost is 100 in overseas countries, the selling price in Thailand will be 180 to 200.
As the most components are imposed the import duty, the domestic product will be sold at 140 to 160.
About the exemption of import duty, No.13/2522 of Board of Investment Act must be studied.

(3) Decision

The decision shall be on import of the switchboard.

9.2. Standards in Thailand

(1) Standards
The standards of electrical equipments are set up by the Ministry of Interior.

(2) Cable
The standards about cable generally follows JIS, however the CVV cables which are widely used in Japan are excluded. Further study is necessitated on the matter. The control cables here seem to be of single wire type instead of strand type.

9.3. Consideration

A well experienced engineer has advised the team to consider about the following matters:

(1) Floor Space

The existing facilities of waterworks are generally arranged in too narrow space, causing much incovenience in operation and maintenance.

More spacious design is preferable.

(2) Protection

The complete protection of main circuit is desirable.

(3) Operation

Where automatic operation is involved, manual operation also must be made possible.

(4) Vermin & Rat

The protection from vermin and rat's harm must be considered.

(5) Ventilation

Because of the sensitiveness of control equipments, ventilation must be regarded.

(6) Reliability, Safety, Easiness of Maintenance

The above is the first priority, needless to say.

(7) Innovation

New technology and products must be adopted as far as possible.

(8) Low Voltage

Lower voltage is preferable for a low capacity generator.

(9) Start up of Generator

Starting up of a low voltage generator is usually worked by battery. As it is infrequently used, the operator tends to forget charging of battery and resulting malfunction has been experienced often. It needs studying.

(10) Saline Air

Where the saline air may cause harm on electrical equipments, housing for protection must be considered.

9.4. Flow and Level Meter

At Samsen Filtration Plant, the venturi tube type is used in measuring the raw water flow. Due to sticking of slime to the differential pressure tube, the accuracy has remarkably decreased.

The type of level meter is the air purge and float. The condition of installation and the price shall be studied in selecting the type.

9.5. Spare Parts

The quantity of spare parts must possibly be large enough for several years' use.

(1) Power Equipments

Sets of the control and protection relays, classified on usage are needed. Lamps and fuses shall be prepared in abundance.

(2) Instrumentation

The parts of electronic equipment including semi-conductors and the wearable parts of recorders shall be prepared in sufficient quantity.

9.6. Transportation

Of the equipments used at Dok Krai, the tranformer is the heaviest and largest. The weight is 17 ton and the size is approximately 4 m x 4 m x 4 m as a package. The road condition from Bangkok to Dok Krai will not cause the transportation any inconvenience.

10. CONTACTS WITH OTHER AGENCIES

10.1. Provincial Electricity Authority, PEA

During construction and after the completion of pipeline, electricity is needed at Dok Krai, the head tank, the receiving well.
Confirmation about the supply capacity of existing PEA line and application for the above must be prepared.

The demand and prospective supply line of PEA has previously been explained in this paper's 2. Power Supply and Demand.
One additional explanation here is that during the construction stage, the contractors shall provide diesel engine-driven generators for welding works.

10.2. Ministry of Interior

As the pump station at Dok Krai uses electrical equipments of high tension and large capacity, an application of the regulation of fire protection can be expected.
A study is needed about the jurisdiction, details of regulation, process of application and other procedures.

10.3. Telephone Organization of Thailand , TOT

The availability of public telephone system in the area before August 1984 must be studied and then the application of telephone at the three stations shall be submitted.
In case of scarce possibility of the above condition, the importance of wireless communication increases remarkably.

10.4. Ministry of Telecommunication, MOT

Two wireless stations, Dok Krai and the receiving well, must be prepared.

A study is needed about the process of application, possible allocation of frequency, possible specification of equipments, prospective location, structure and type of antenna, qualification of wireless operater, etc.

NO. 17-1. COMPARISON OF INTAKE-PUMP

FEBRUARY 1982

Prepared by

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CONTENT

1. INTRODUCTION

2. COMPARISON

3. SUMMERY OF COMPARISON

4. CONCLUSION OF COMPARISON

1. INTRODUCTION

In the feasibility study report, several types of combination of pump-intake structure was compared and it concluded B-3 as the choice.

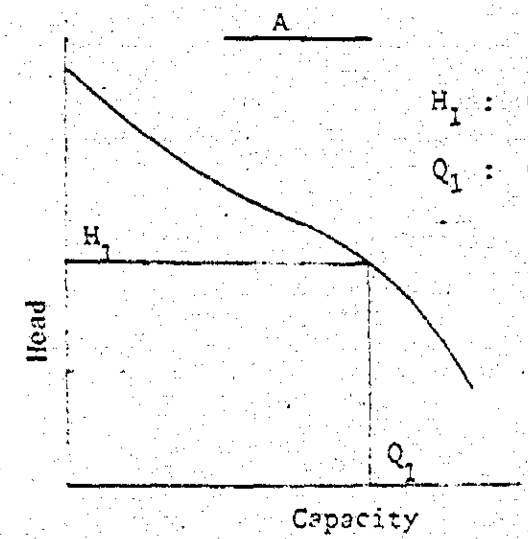
To recheck the soundness of conclusion, the detailed design team will take up an alternative and compare it with B-3. The alternative is the one named B-1 in the feasibility study.

2. COMPARISON

The two alternatives, named B-1 and B-3 in the feasibility study, will be re-maned here as A and B respectively.
Each of A and B is shown on the figures A-1 to A-5 and B-1 to B-5.
The comparison of A and B is summerized and tabulated on Table 1 and understanding the table shall be helped much by the drawings A-1 to A-5 and B-1 to B-5.

TABLE I. COMPARISON OF PUMP

No.	Point	A	B
1.	Type	Vertical shaft, mixed flow	Vertical shaft, double suction volute
2.	Order of assembling	Motor, water rising pipe, shaft pump	Motor, Shaft, Pump
	Order of disassembling	Reversing of above	Reversing of above
3.	Load	All loads are beared by platform	Loads are spread on each floor
4.	Maintenance	Can be done on platform only	Shall be done on 4 floors
	Staircase Space	Unnecessary platform is crowded by all equipments	Necessary Less crowded than A
5.	Lubrication	Needs clear water for bearing	Ordinary lubrication, oil and grease
6.	Pump characteristics		
	Efficiency Motor Capacity	Two stages cause 2% less than B Slightly larger than B	Single stage causes 2% more than A Slightly smaller than A
	Characteristics Curve	as shown below	as shown below



H_1 : Optimum point of head in operation

Q_1 : Optimum point of capacity in operation

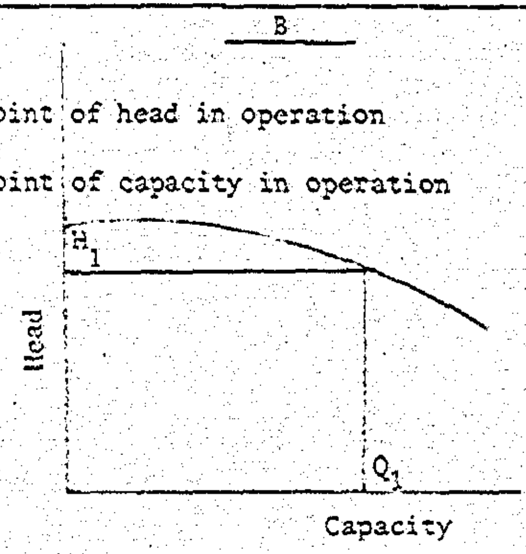


TABLE 1 (CONTINUED)

6.	Shut-off Head	Higher than *O.P.H. as H-Q curve's gradient is sharp	Close to O.P.H. as H-Q curve's gradient is gentle
	Change of Q	Affects on H much by the above reason	Does not affect on H much
	Suitable Q range	Above *O.P.C.	Below O.P.C.
7.	Weight	Because of the long water rising pipe the total weight is about 7.5 ton, excluding the water weight in the pipe. It acts on the platform.	The total weight is about 4.0 ton the water weight acts on the basement floor.
8.	Auxiliaries needed for starting	Lubrication water piping, electro-magnetic valve, flow relay	none
9.	Workmanship Alignment readjusting after installation	Because of long shaft, it needs the skill troublesome and difficult as the shaft is underwater	same as A easy as the shaft is in the room
10.	Changing parts	To change the vane, the motor and rising pipe must be removed To change the bearing, the same as above	To change the vane, only the upper casing must be removed To change the bearing, the work is limited to the spot
11.	Inspection	Regular inspection is indispensable as many parts are underwater	Regular inspection is less needed than for A

* O.P.H.---- Optimum point of head in operation

* O.P.C.---- Optimum point of capacity in operation

TABLE 1 (CONTINUED)

12.	Vibration, noise	Noise of motor hits the control room on the same floor Vibration can be cut off	The same as A The same as A
13.	Ventilation	Only the platform needs it	All 4 floors need it
14.	Floor area	1 floor 480 m^2	4 floors 1,594 m^2
15.	Cost of pumps	higher than B	lower than A
16.	Electricity cost for full operation (1.2 Baht/kw)	2.61 million Baht/month 0.384 Baht/m^3 - water	2.35 million Baht/month 0.426 Baht/m^3 - water

3. SUMMERY OF COMPARISON

As it is seen on the foregoing table of comparison and the attached drawings, Fig A-1 to A-5 and B-1 to B-5, a table of advantages and disadvantages can be the summery.
It is shown on the next page.

As is easily understood, the advantage and disadvantage is relative, that is, A's advantage is B's disadvantage and vice versa.

Table 2 Summery of Comparison

Advantage of A	Disadvantage of B
* all operations can be made on one floor, platform accordingly :	* 4 floor building and equipments are installed on every floor accordingly :
* no need of moving up and down	* more moving up and down
* pipings are simple and easy arrangement	* pipings are complexed
* smaller floor space	* larger floor space
* smaller ventilation capacity	* larger ventilation capacity
* no need of platform for maintenance	* platform for maintenance is needed around pumps

Disadvantage of A	Advantage of B
* rusting and wearing of underwater parts are possible	* malfunction is less frequent due to rusting, wearing
* auxiliary's trouble affects pump's start-up	* no auxiliary and higher reliability of pump's start-up
* less efficiency of energy	* more efficiency of energy

Table 2 Summery of Comparison (cont'ed)

Disadvantage of A	Advantage of B
* because of difficulties of inspection and maintenance for underwater parts, installation requires skill and accuracy	* installation, inspection, repairing and replacement are easy
* higher cost	* lower cost
* crowded arrangement of equipments on a single floor makes operation of gates, disassembling of pumps difficult	* operation of gates, disassembling of pumps are easy because of ample space
* horizontal displacement of structure possibly causes bending of water rising pipe, resulting in bending of shaft and dislodging of bearing	

4. CONCLUSION ON COMPARISON

The requirements on pumps are the low cost, reliability and easiness of maintenance.
B shall be the choice from the aspects.
Paraphrasing the above:

(a) less breakdown and malfunction is certain as the equipments are kept dry.

(b) in case of repairing, it can be easier

(c) no auxiliary equipment which may cause difficulty is used

(d) lower cost

(e) higher efficiency

Fig.A-1 Section

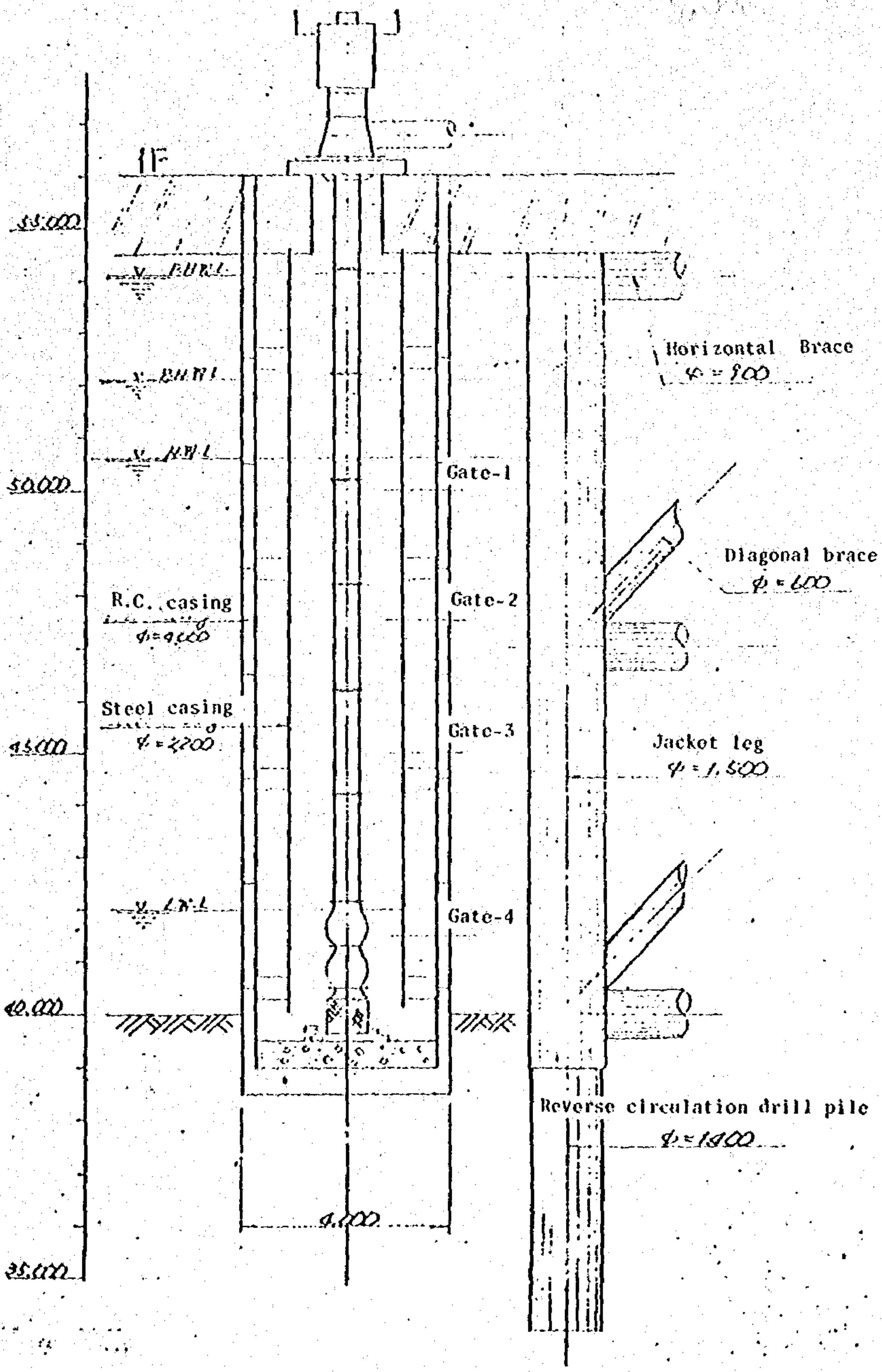


Fig.A-2 Plan

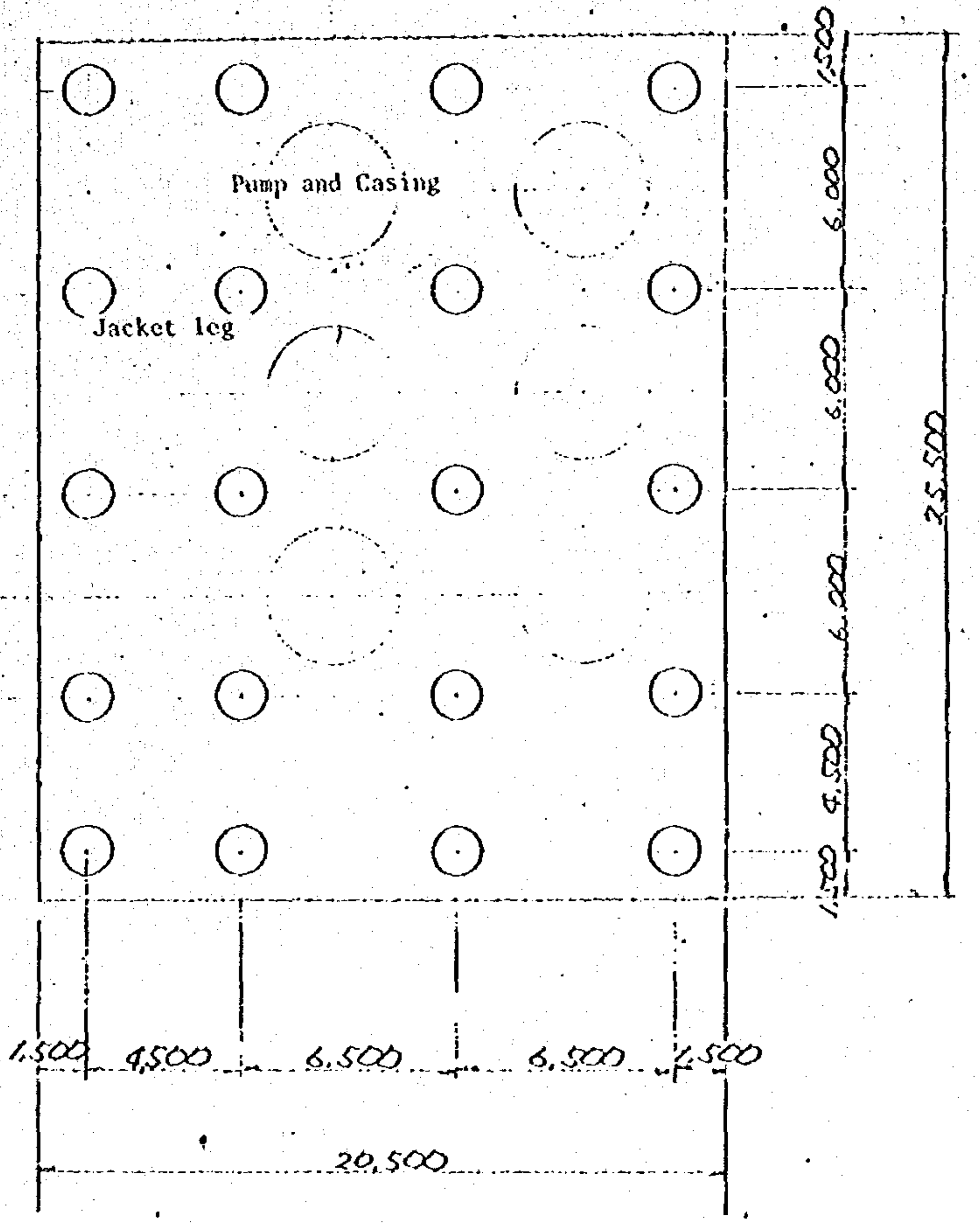

Pump and Casing
Jacket leg
1,500
6,000
6,000
6,000
4,500
1,500
25,500
1,500
4,500
6,500
6,500
1,500
20,500

Fig.A-3 Layout

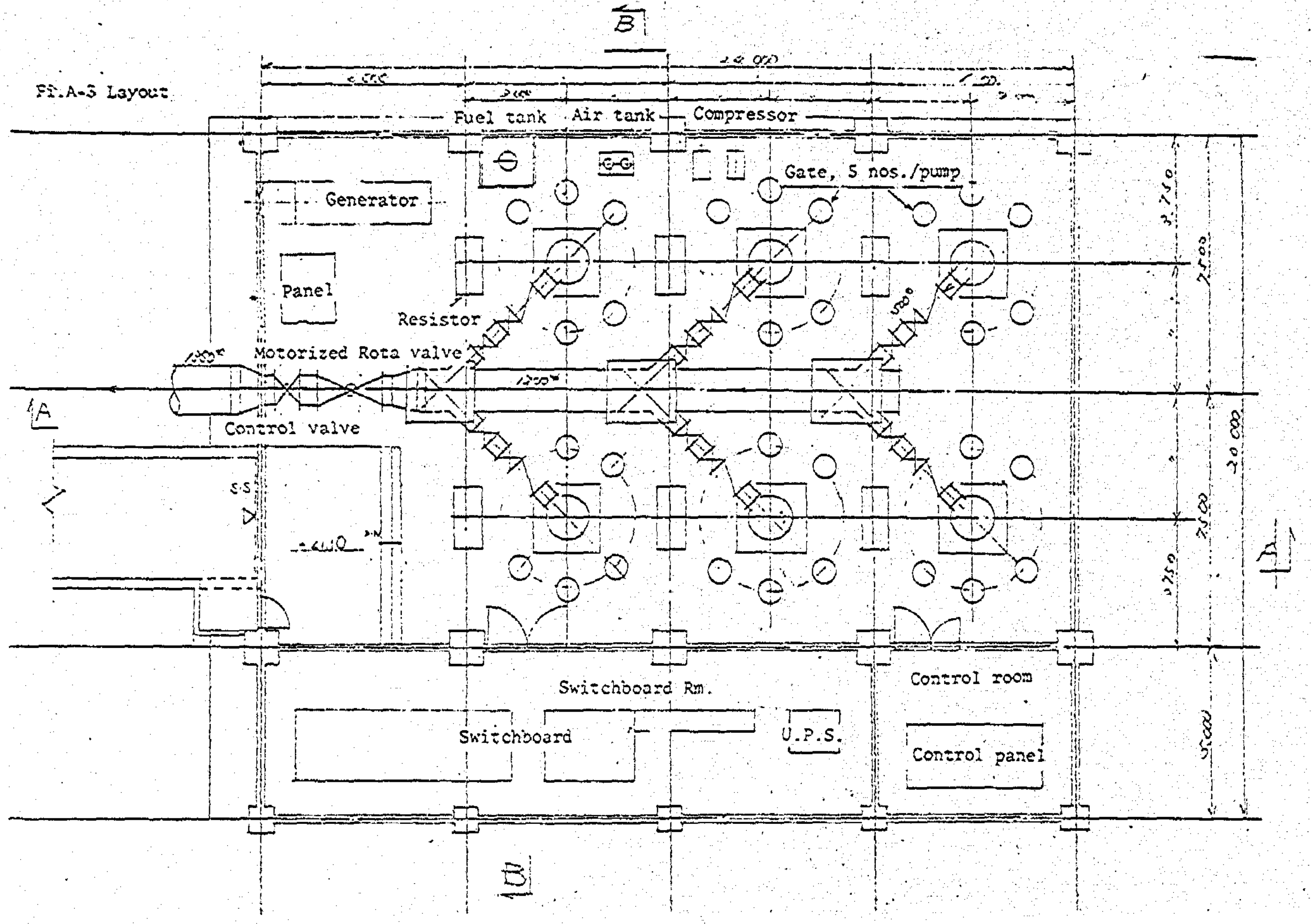


Fig.A-4
A-A
Section, view

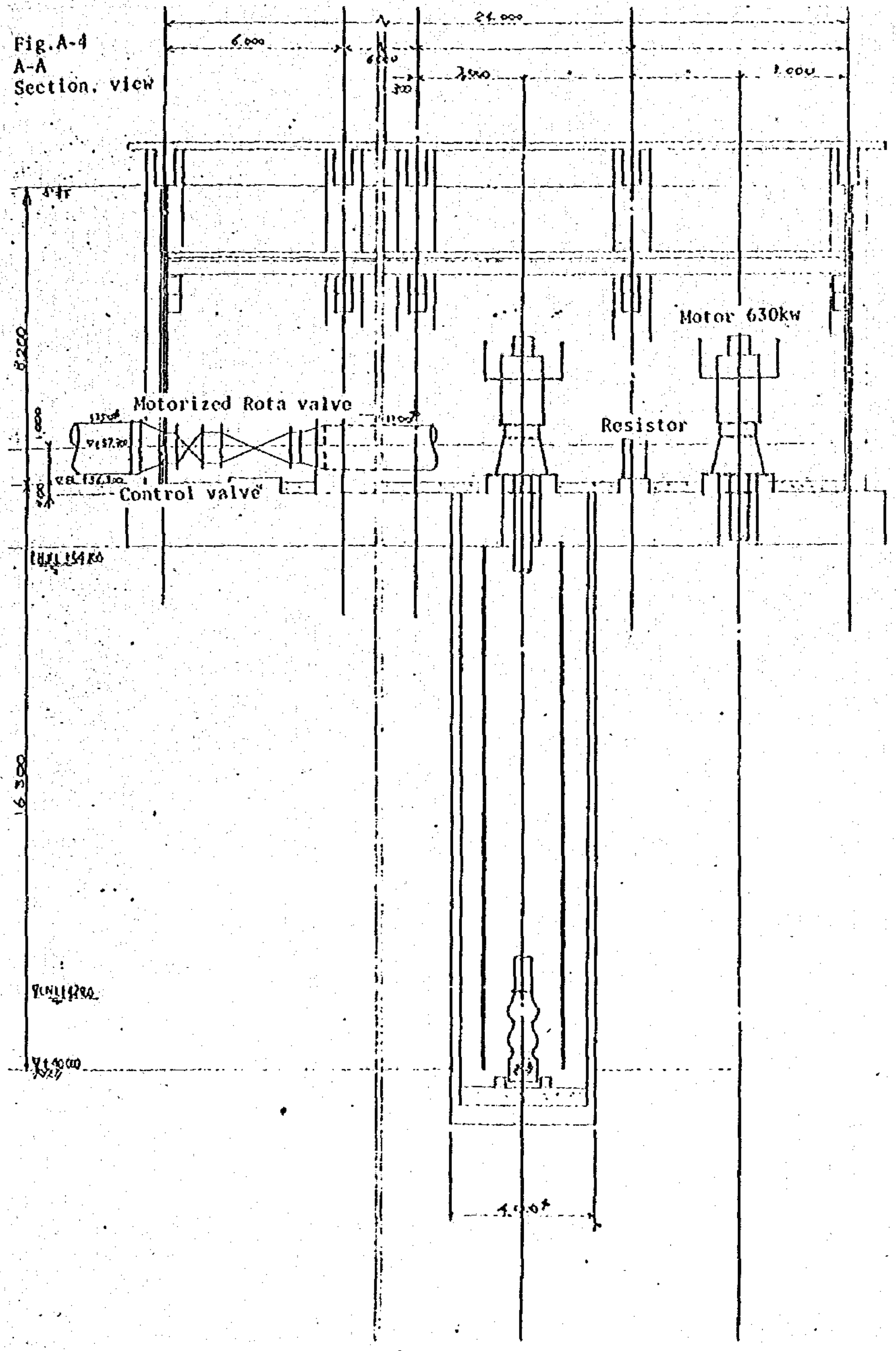


Fig.A-5
B-B
Section, view

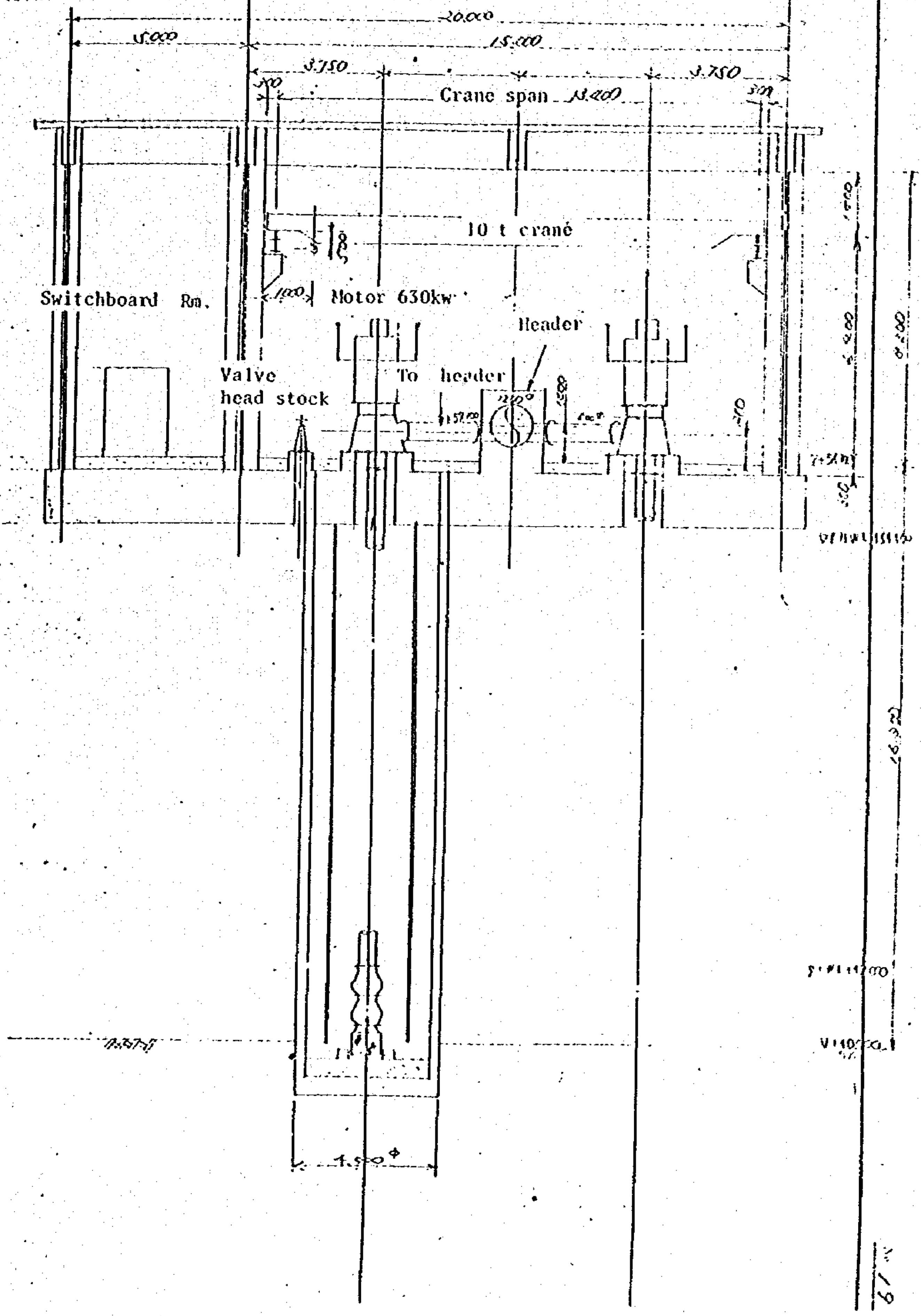


Fig.B-1 Layout,1st floor

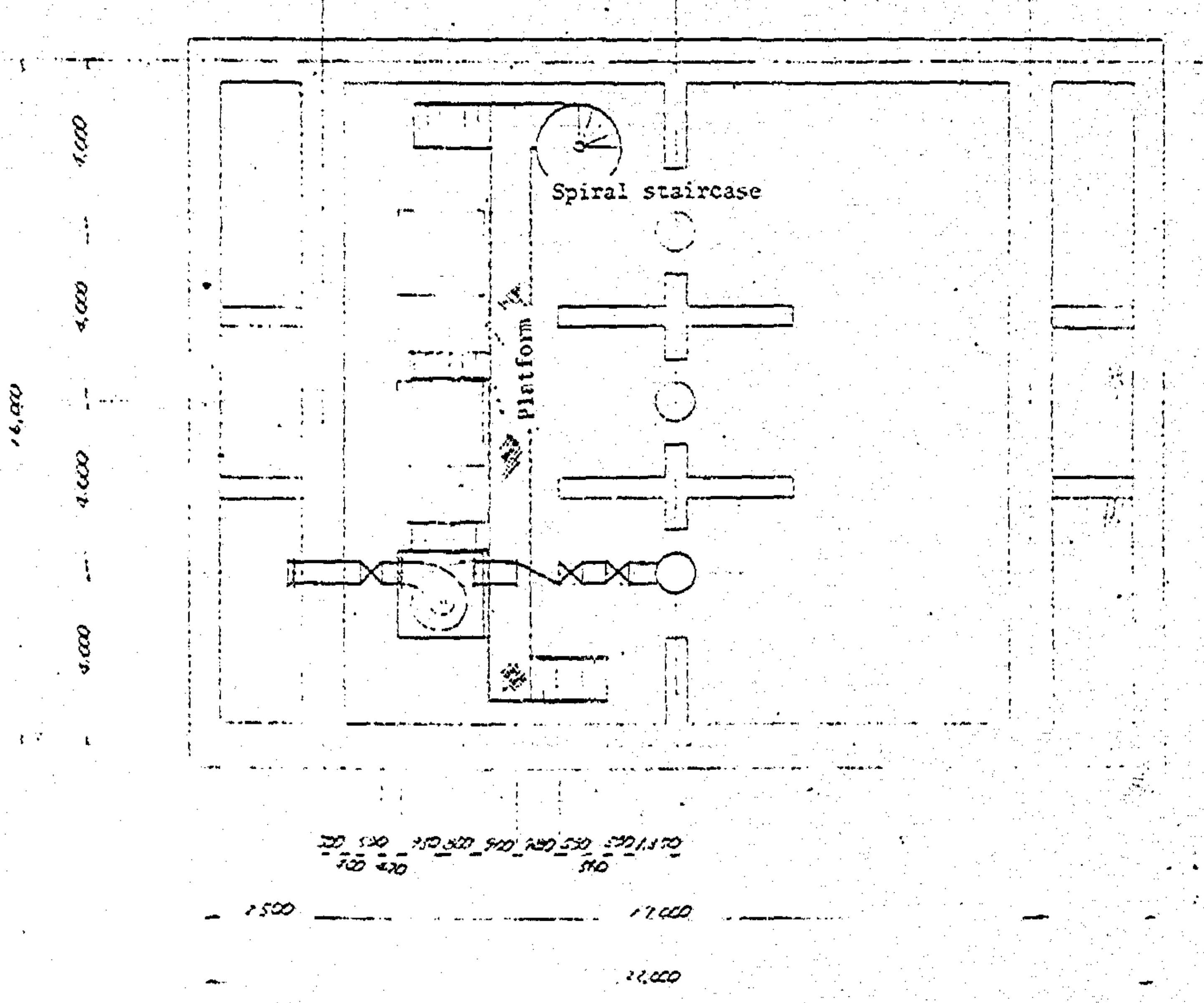


Fig.B-2 Layout,4th floor

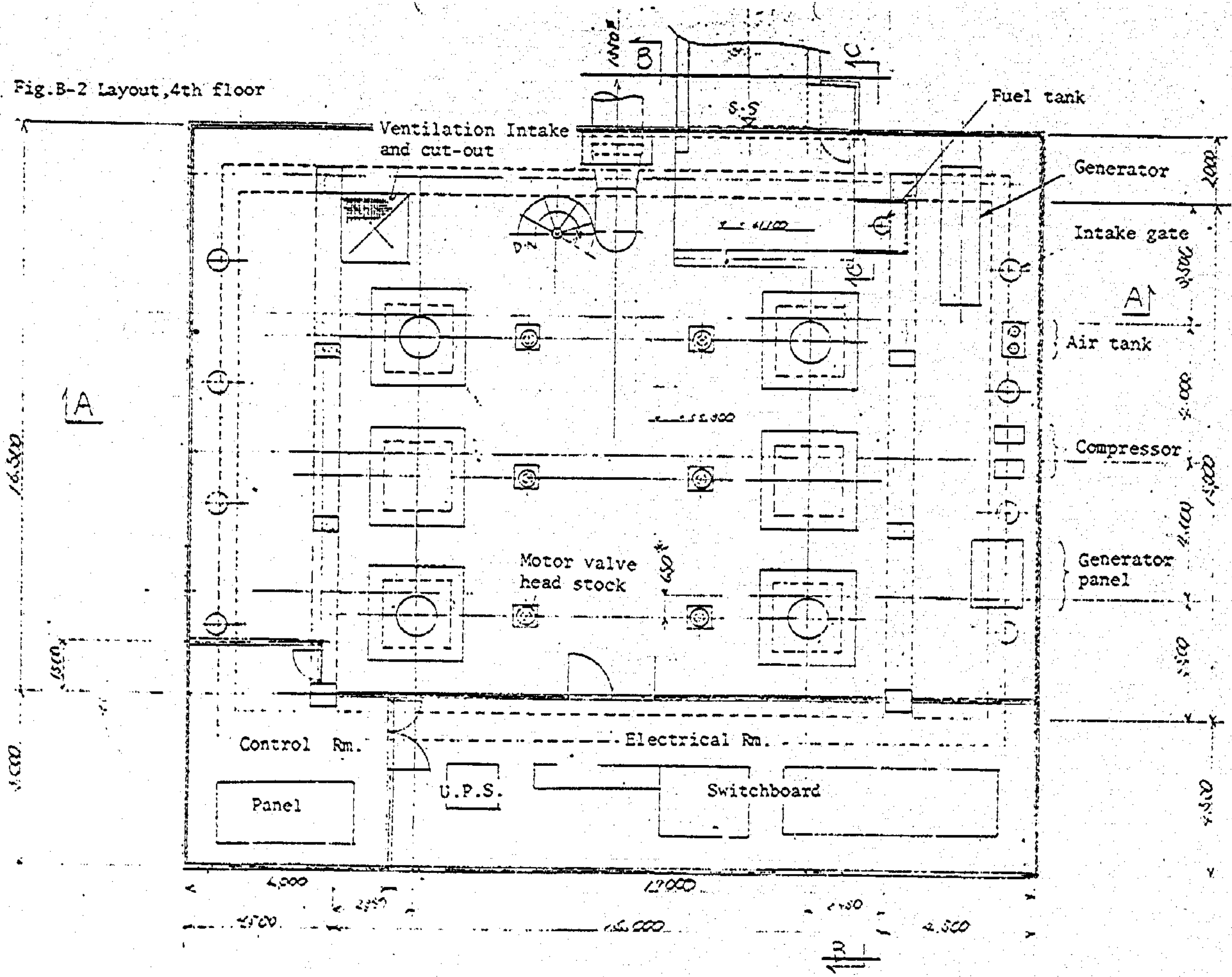


Fig.B-3 Sectional view

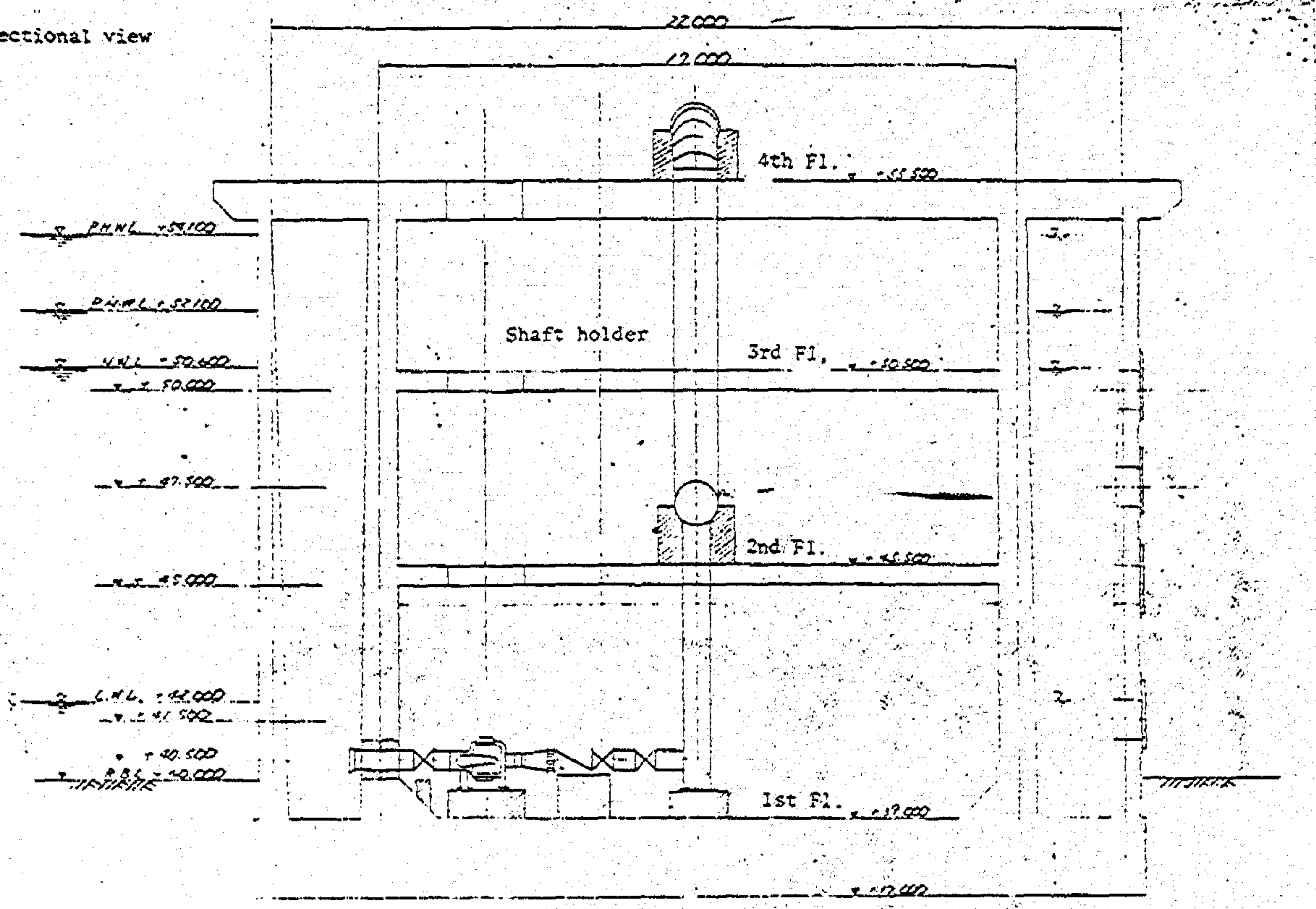


Fig.B-2 A-A Section. view

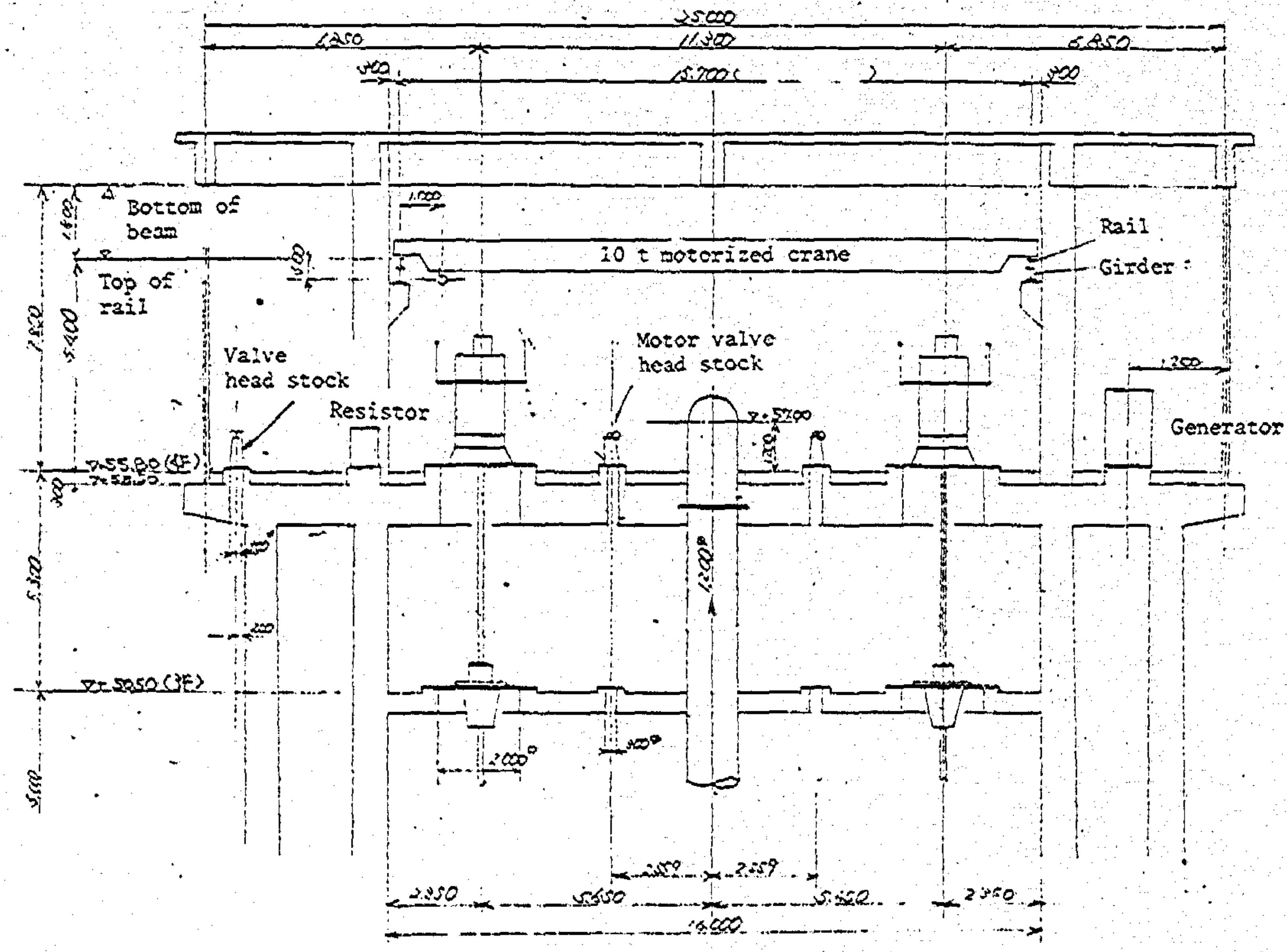
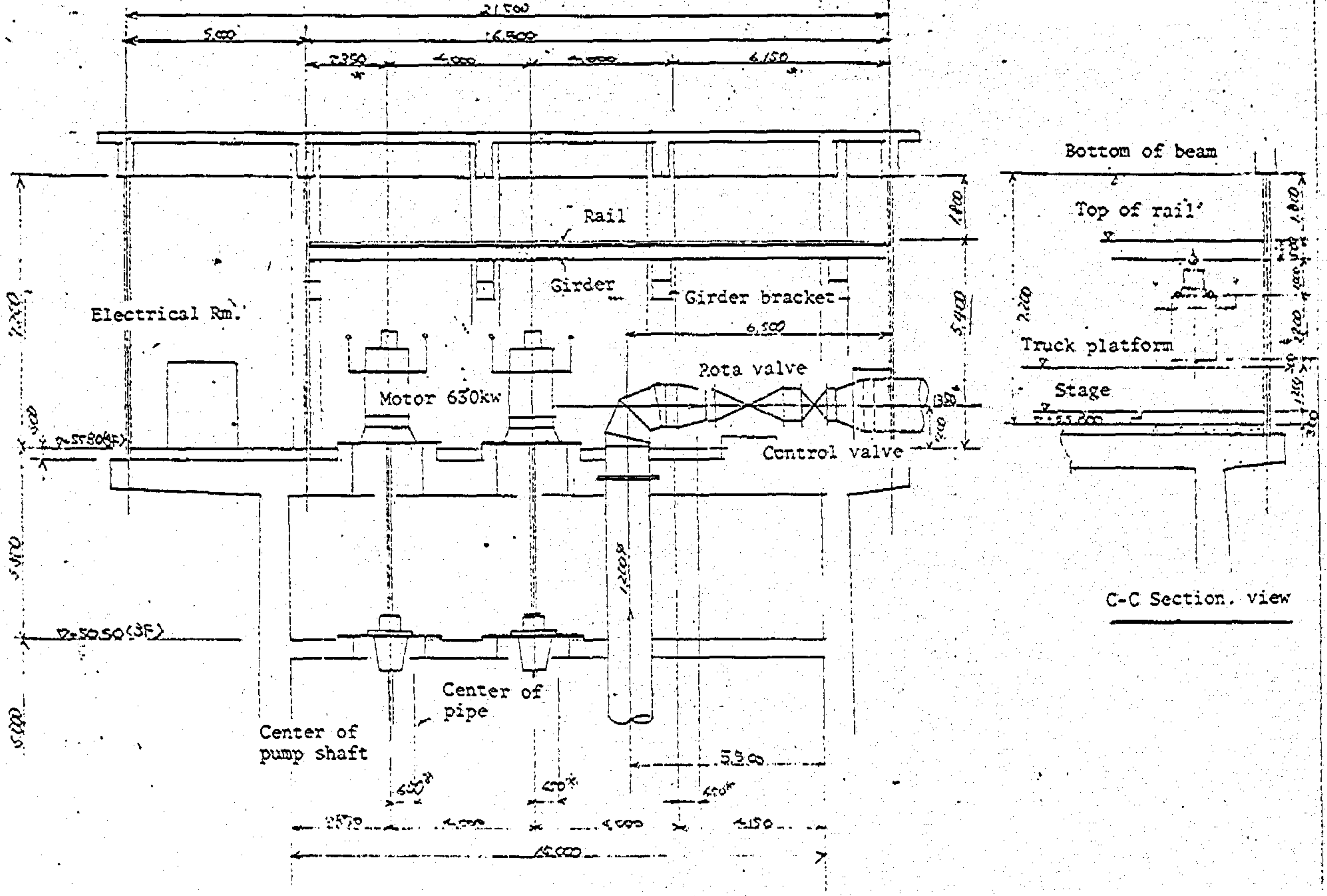


Fig.B-5 B-B Section. view.



NO. 17-2 OPTIMUM RANGE OF PUMP OPERATION

FEBRUARY 1982

Prepared by

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JICA

CONTENT

1. INTRODUCTION AND HYDRAULIC CONDITIONS

1.1. Introduction
1.2. Hydraulic Conditions

2. PUMP CHARACTERISTICS AND PARALLEL RUNNING OF PUMPS

2.1. Pump Characteristics
2.2. Parallel Running of Pumps
2.3. Essential Difficulty

3. PUMP UNIT NUMBER CONTROL AND APPLICATION

3.1. Concept
3.2. Actual Flow Pattern

4. PUMP CHARACTERISTICS, VALVE CONTROL, CAVITATION

4.1. Pump Characteristics
4.2. Valve Control

5. ALTERNATIVE

5.1. Review and Points of Alternative
5.2. Improvement
5.3. Efficiency

1. INTRODUCTION AND HYDRAULIC CONDITIONS

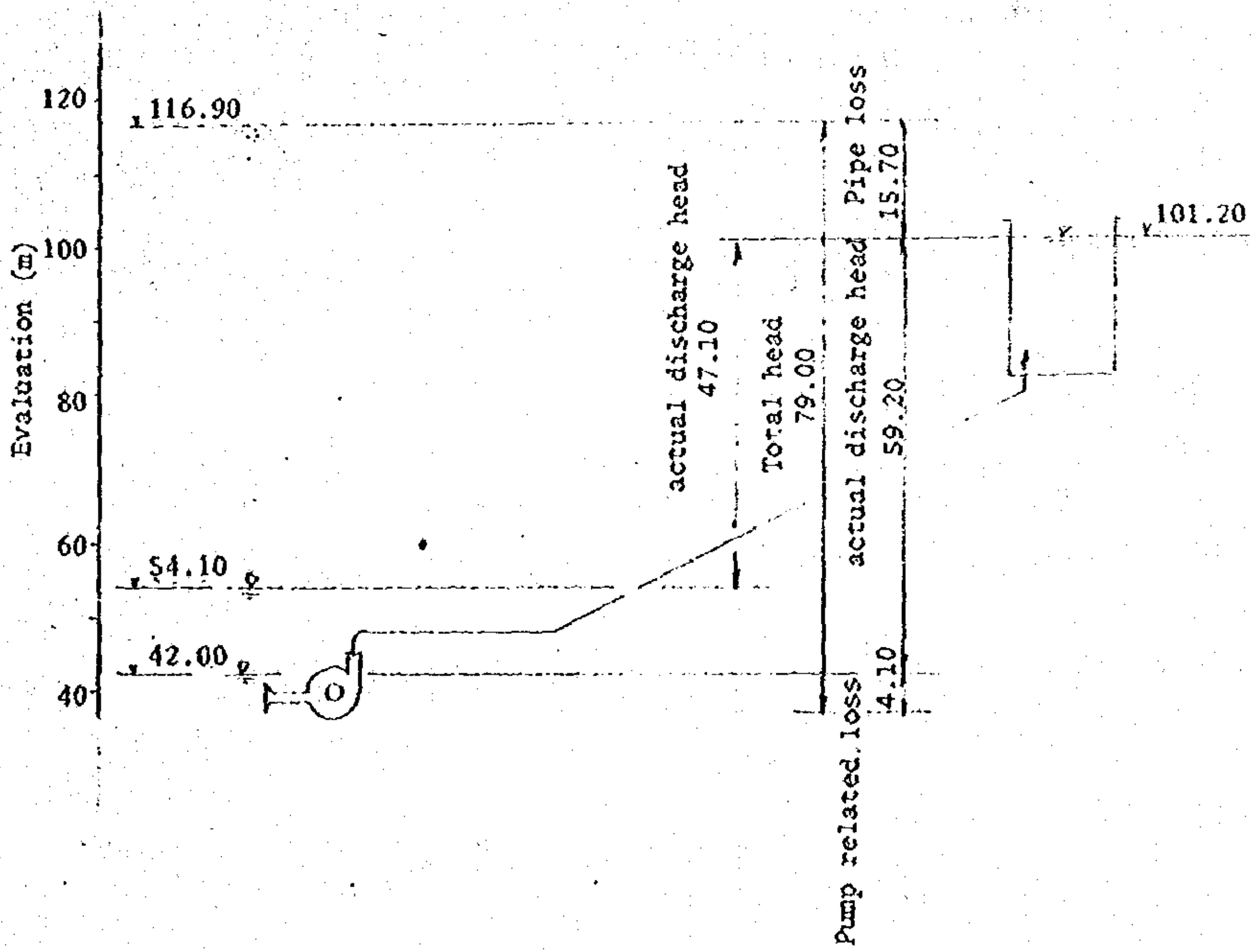
1.1. Introduction

The papers, 7-1 Comparative Study of Flow Control System and 7-2 Hydraulics and Operation of Pump Unit Number Control, have lead to the decision on flow control system. Under the decision, the requirements about the pumps characteristics and control valve shall be studied.
This paper, starting from reviewing the pump characteristics, pumps behaviour in the parallel run, suggests modification of the concept of Pump Unit Number Control and it also examines the soundness of valve control.

1.2. Hydraulic Conditions

The pumps total head shall be 79 m, as is shown on the picture below:

Fig.1 Pump Head



The design flow rate is 2.62 m^3/sec employing five pumps and the capacity per pump is $2.62 \times 60 \div 5 = 31.44 \doteq 31.5$ m^3/min.

2. PUMP CHARACTERISTICS AND PARALLEL RUNNING OF PUMPS

2.1. Pump Characteristics

The original pump's characteristics is assumed as shown on Fig.2.

With the optimum operation point of 79 m head, 84% efficiency and 31.5 m^3/min. capacity, the calculated shaft horsepower is 483 kw.

The pump and motor shall be :

0.5 m diameter, 31.5 m^3/min capacity, 79 m total head, 540 kw motor rating of 3,000 volt and 6 poles

2.2. Parallel Running of Pumps

When 1 to 5 of the said pumps are run in parallel, the head-capacity curve will be the one shown on Fig. 2.

On the figure, two kinds of curves are drawn.

The head-capacity curves, five with numbered from 1 to 5, shows the relation of head vs. capacity under the parallel operation of 1 to 5 pumps. It shall be noted that the five curves converge at the left hand end where the capacity is zero. The loss of head curves show that the increase of loss is parabolic and at the left hand end the curves show the actual discharge head (the difference of water levels, Dok Krai and head tank) and at the right hand end the curve for LWL, the head-capacity curve numbered 5, the vertical line of 157.5 m^3/min. (31.5 x 5) and the horizontal line of 79 m (design head) meet.

The operation point of pump(s) is a cross point of the head-capacity curve and the loss of head curve.

A part of Fig. 2 is taken and shown on the next page.

Fig-2 Pump Characteristics

volute type
0.5 m dia. x 31.5 m^3/min x 79 m
x 540 kw x 3,000 V x 6 P

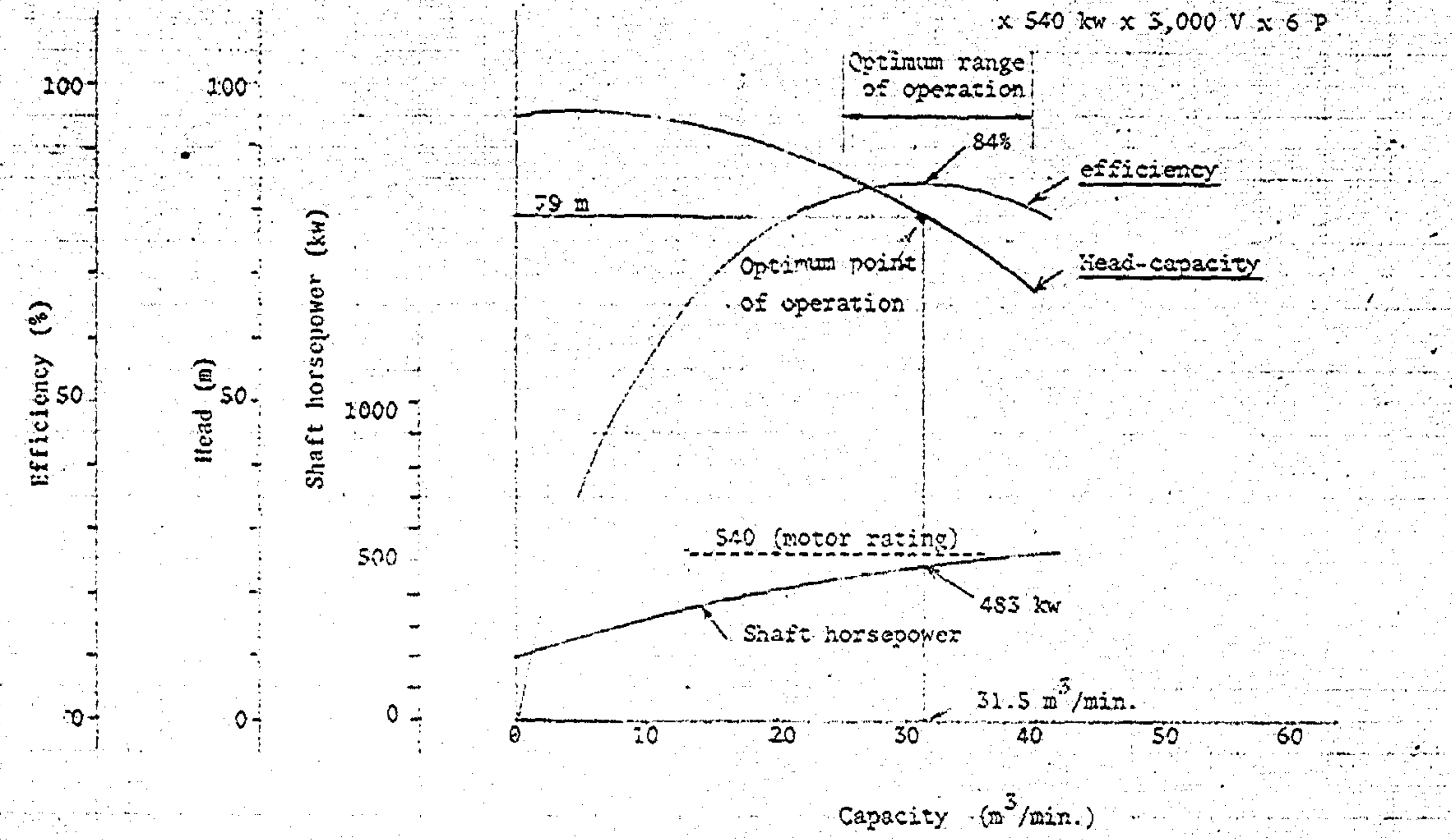


Fig.3) Parallel Running of Pumps

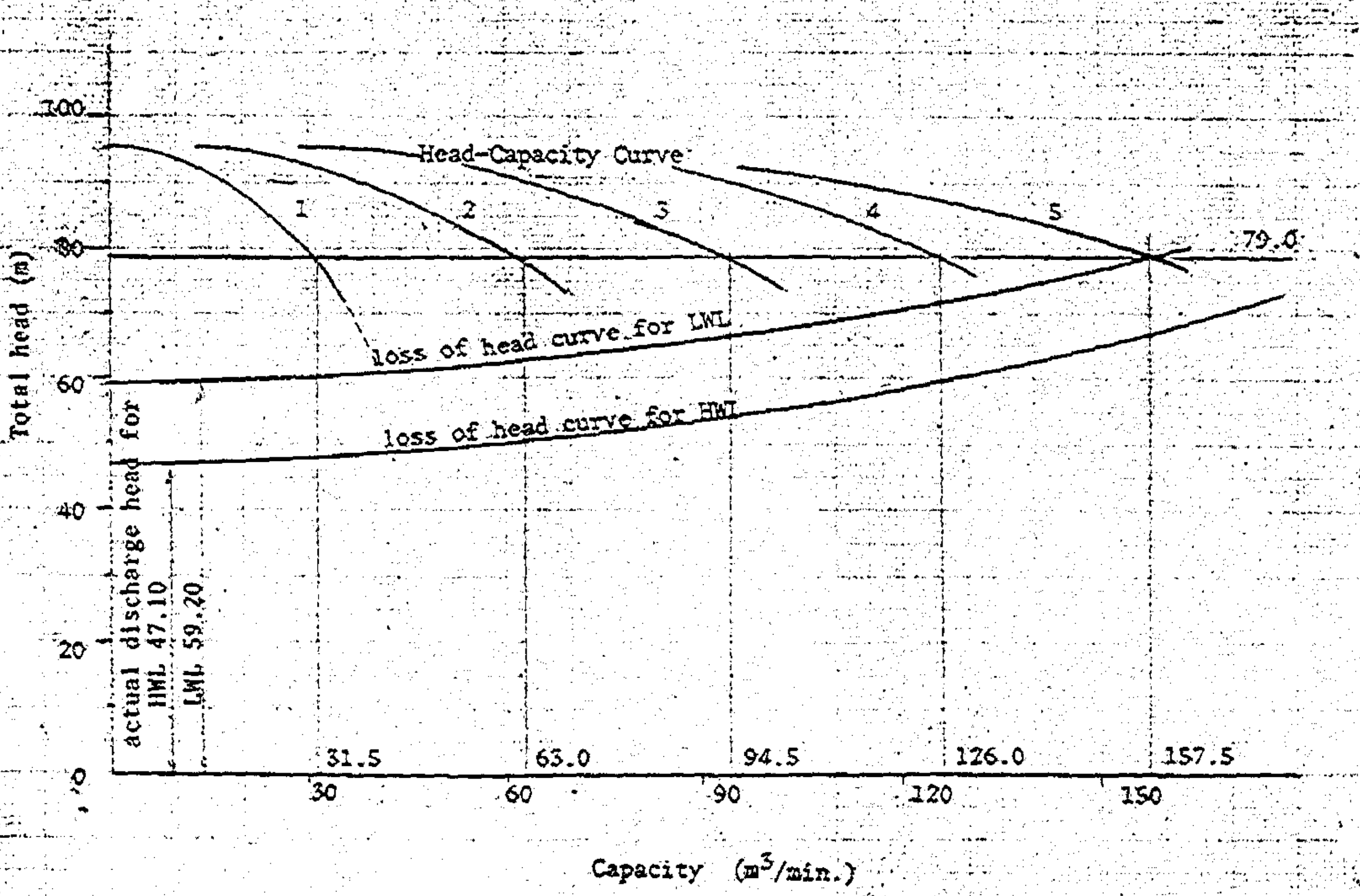
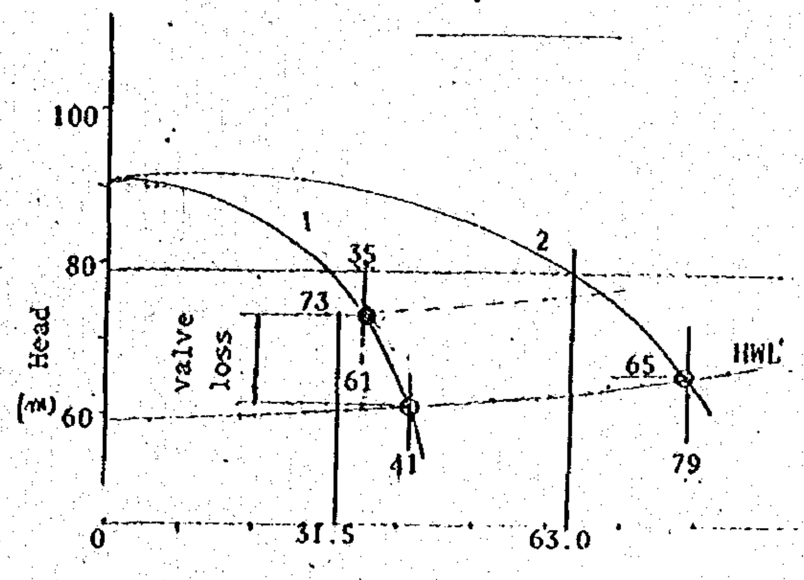


Fig. 4



If no control is given to the valve, in case of 1 and 2 pumps, the flow will be 41 and 79 m^3/sec with the delivery head of 61 and 65 m respectively.

In case of 1 pump, 30% more than 31.5 m^3/min. expected flow and in case of 2 pumps, 25% more than 63.0 m^3/min. will be naturally settled down in the actual discharge.

If a specified flow rate, for instance 35 m^3/min, shall be obtained, the control of valve (throttling is the case) will change the loss of head curve artificailly to meet the two curves.

The cross point moves from the point, 41 m^3/min and 61 m, to the point, 35 m^3/min. and 73 m, by throttling the valve to give 73 - 61 = 12 m loss artificially. Under the condition, the loss of head moves to the new position shown with the dotted line.

2.3. Essential Difficulty

As shown on Fig.2 , a pump's characteristics is decided tentatively on the head and capacity, considering the conditions of full capacity. Also the decision on the optimum head and capacity is usually made independently from the pipeline's conditions,that is, considering the behaviour of single unit of pump.

When the pumps are run in parallel, however, the natural point of operation is not always the optimum one.
For example on the foregoing picture, the optimum point, 31.5 m^3/min.and 79 m, can be obtained by throttling and giving about 19 m loss to the valve.
The apparent efficiency of pump is 84% as seen on Fig.2 but the actual efficiency is approximately 84 x 60/79 = 64% instead.
Moreover, the change of reservoir level causes movement of the point of operation, thus affecting pump's efficiency.

As is seen on Fig.3., the most efficient point of operation can only be attained, with 5 pumps running and LWL of the reservoir.

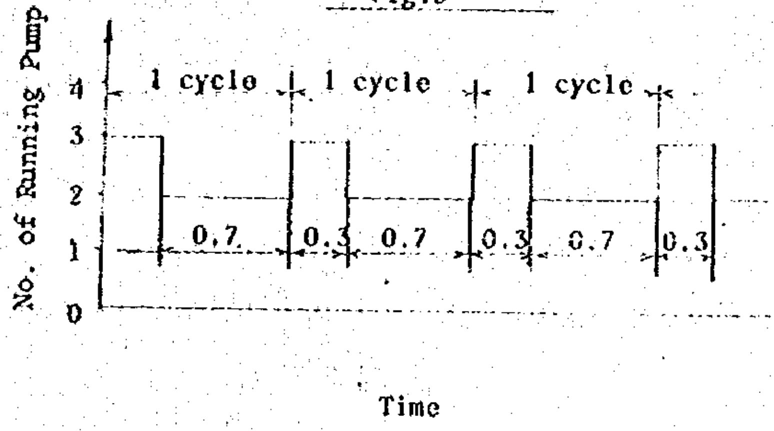
3. PUMP UNIT NUMBER CONTROL AND APPLICATION

3.1. Concept

As is discussed in the report "Hydraulics and Operations of Pump Unit Number Control", the concept is to change the number of running pumps periodically in order to obtain the desired flow rate.

Suppose, for instance, the desired rate is 2.3 times of a pumps capacity, 31.5 m^3/min here. Then the number of running pumps will follow the pattern shown below:

Fig.5

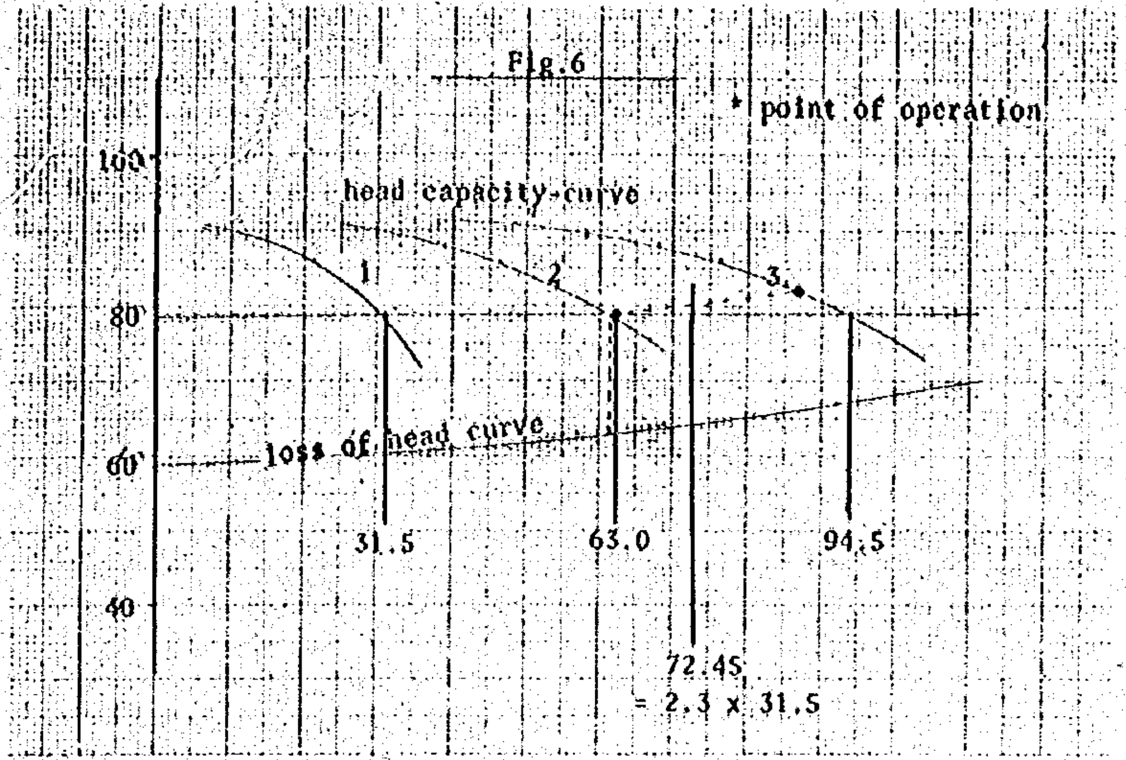


As it is periodical, the time can be divided into cycles. The physical meaning of cycle is explained in the quoted report.

Within every cycle, 2 pumps runs steadily and 1 more pump runs for 0.3 parts of a cycle and rests for 0.7 parts. Thus, 2.3 times of a pumps capacity can be obtained. However in practice, it does not behave in the way.

3.2. Actual Flow Pattern

Again a part of Fig. 3 is taken and shown on the next page.



The two running pumps are expected to discharge 63 m^3/min. and accordingly the loss of head curve shall be moved to the position of dotted line, by controling the valve.
Then, the 3rd pumps starts and the point of operation goes to the cross point of the head-capacity curve and the dotted loss of head curve.
As is seen on the picture, the 3rd pump cannot discharge the expected value of 31.5 m^3/min and consequently has to run longer time than 0.3 parts of a cycle.

The flow pattern has come from that the valve is controled so that 2 running pumps discharge 63 m^3/min. Instead, when the valve is controled in the way where 3 running pumps discharge 94.5 m^3/min, then 2 running pumps will discharge more than 63 m^3/min. The actual flow pattern differs from the conceived (pre-supposed) one.

It is shown below.

Fig.7 63 m^3/min. by 2 pumps

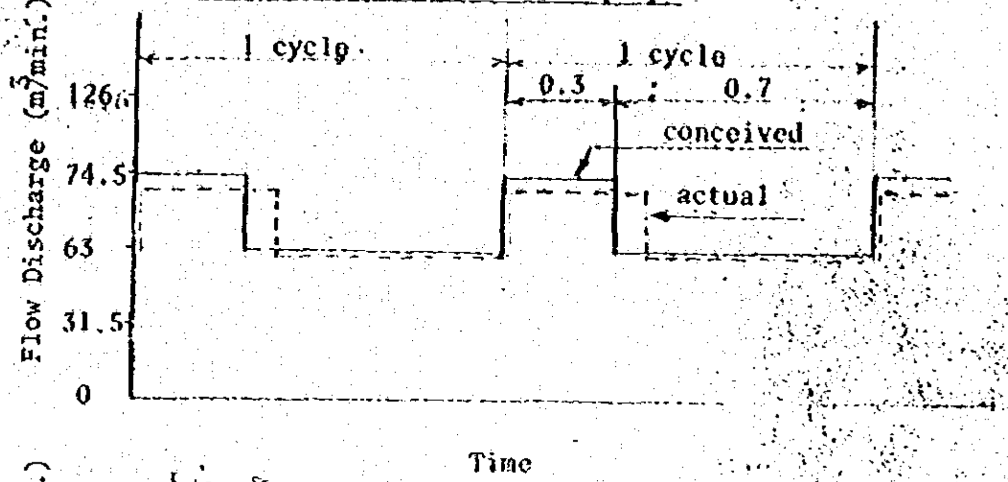
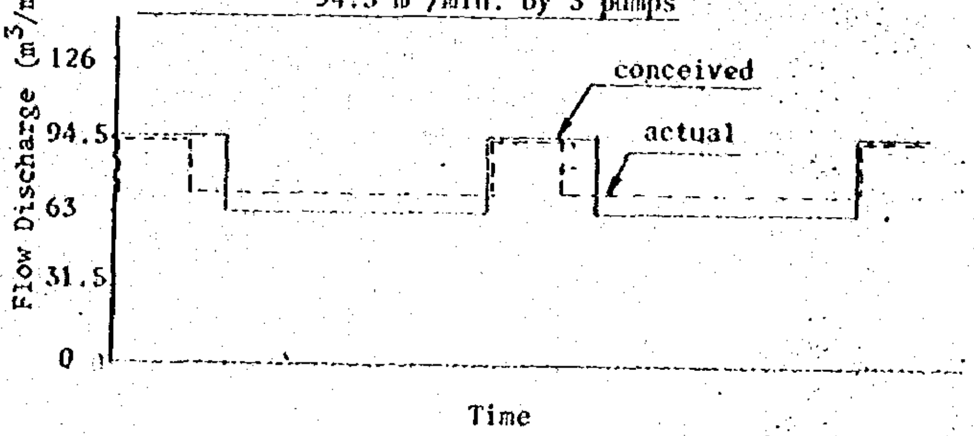


Fig. 8

94.5 m^3/min. by 3 pumps



Though the actual pattern does not follow the conceived one exactly, it does not cause any problem in operation.
The problem does exist in the pumps characteristics and the control of flow by the valve.

4. PUMP CHARACTERISTICS, VALVE CONTROL, CAVITATION

4.1. Flow Characteristics

The pump is usually expected to work around the optimum point of operation. It means that it has a limited range of operation.

As is seen on Fig. 2 , the lower limit is at 0 capacity, or at the shut-off operation.

The upper limit, as Fig. 2 shows also, can be extended only at the sacrifice of efficiency and by the increase of motor rating.

4.2. Valve Control

As it has been discussed in the previous chapter, operating pumps at the optimum point of operation needs delicate control of the valve, in order to adjust the loss of head curve and especially in Pump Unit Number Control, periodical control of the valve is impractical.

4.3. Cavitation

On Fig. 3 when one pump is running at LWL and the optimum point of operation (79 m, 31.5 m^3/min) is to be kept, the control valve has to make 79-48 = 31 m head loss.

The loss, reducing the pressure, may possibly cause cavitation which will result in noise, vibration and corrosion in the extreme case.

To avoid the troublesome control of valve and cavitation, an alternative can be presented and studied.

5. ALTERNATIVE

5.1. Review and Points of Alternative

Using 1 to 5 pumps of which characteristics are shown on Fig. 2 and running them in parallel for Pump Unit Number Control system to obtain the desired flow rate, is the approach made so far. In the foregoing chapters it was seen that :

* The optimum point of operation can be decided in combination of the characteristics of both pump and pipeline. In case of the parallel running of pumps, rather the optimum range of operation, instead of point, must be considered, as the optimum point changes position.

* Especially in Pump Unit Number Control System, the number of running pumps changes periodically. Because of the pipeline characteristics, the actuall flow pattern cannot exactly follow the theoreticalone, even though the two patterns are practically similar and close.

* In order to converge the two flow patterns, continuous and delicate control of valve is required and it is not practical.

* The existance of control valve may cause cavitation.

* Keeping the pump(s) at the optimum point of operation by controling the valve seems efficient apparently but actually the head loss of valve decrease the apparent efficiency.

From these findings, an alternative is conceived :

* Eliminating the control valve. Consideration on control and cavitation becomes unnecessary then..

* Using the pumps which have a wider practical range of operation, although the motor rating must be raised.

The alternative can be shown on Fig. 9 and 10, concerning the pump characteristics and parallel running of pumps.

Fig.9 Pump Characteristics of Alternative

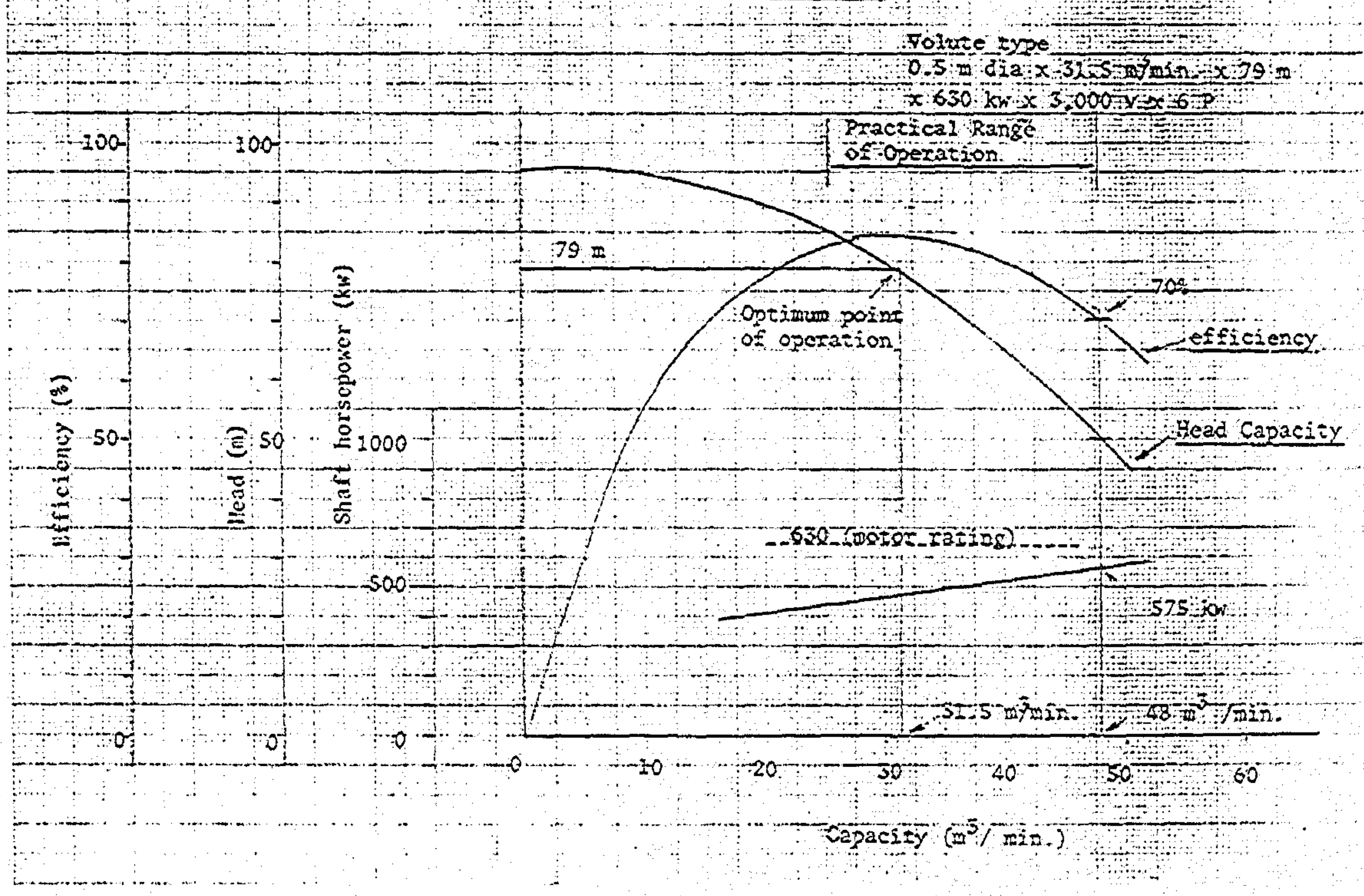
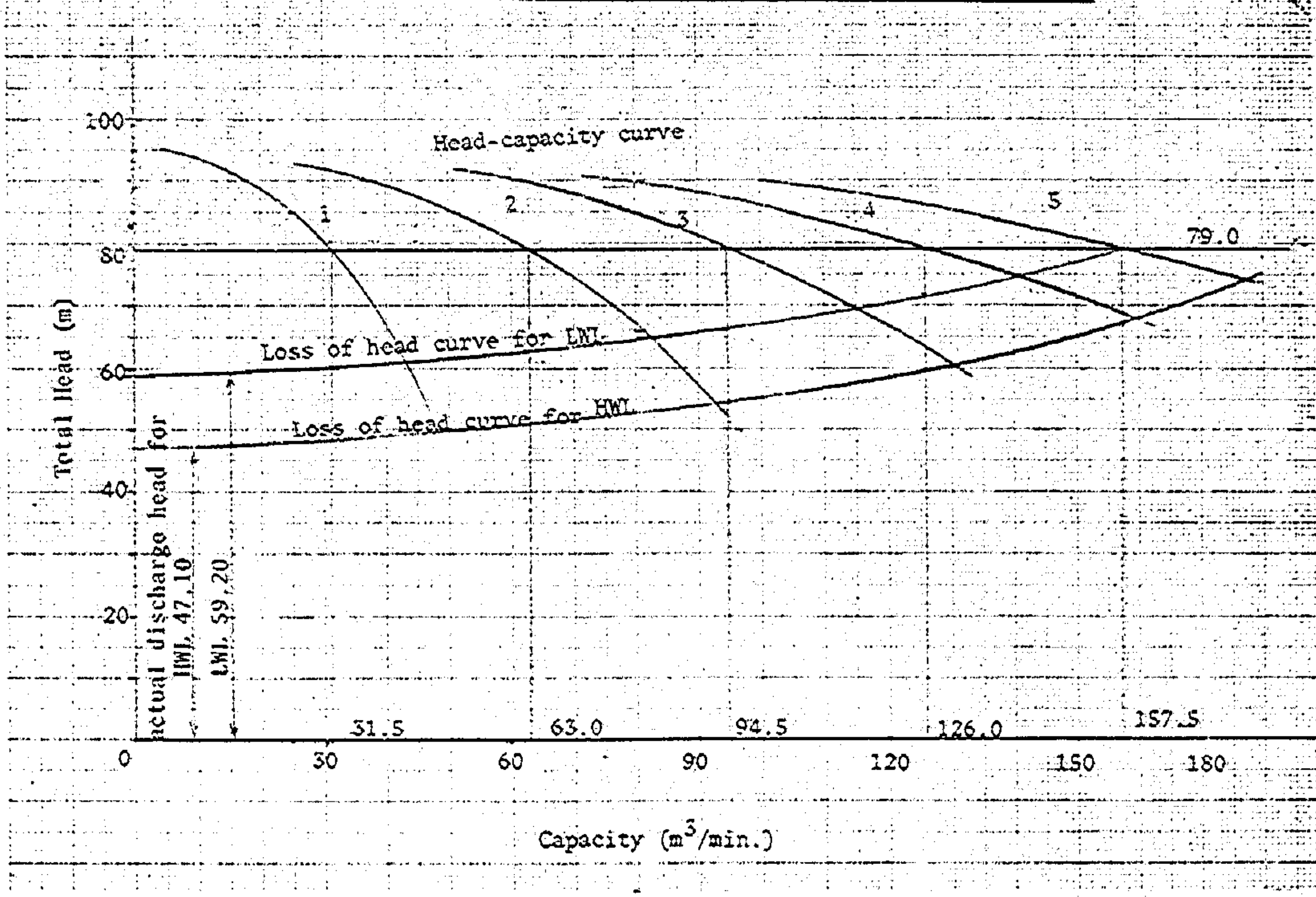


Fig. 10 Parallel Running of Pumps of Alternative



5.2. Improvement

The alternative is actually an improved one.
The difference is :

* Instead of 540 kw motor, 630 kw motor is used for pumpset.
* The control valve is eliminated and the controling operation becomes unnecessary.

Table 1 shows the comparison of the original and alternative.

5.3. Efficiency

Original

As is discussed in 2.3., the pumps apparent efficiency is remakably decreased in the pipeline.
The difference of apparent efficiency and actual one is shown, in case of LML:

No. of pump	A: total head (%)	control B: valve loss	C: $\frac{A-B}{A}$	*actual efficiency (%)
1	79	19	0.76	64
2	79	16	0.80	67
3	79	13	0.84	70
4	79	8	0.90	76
5	79	0	1.00	84

* 84% x C

Alternative

Against the original, the alternative's efficiency is always actual, 70-75%.
In the earlier stage of pipeline's operation, the efficiency is rather better than the original, though when the flow rate becomes high in the later stage, it will be reversed.

Table 1 Comparison

No.	Features	Original	Alternative
1.	Pump Characteristics	0.5 m dia. x 31.5 m^3/min x 79 m	0.5 m dia. x 31.5 m^3/min x 79 m
2.	Motor "	630 kw x 3,000 V x 6 P	540 kw x 3,000 V x 6 P
3.	Pump Operation efficiency range of operation	70-75% without vlave control wider	84% with valve control narrower
4.	Discharge (m^3/min) at HWL		
	pumps no. 1	45	31.5
	2	95	63.0
	3	135	94.5
	4	160	126.0
	5	184	157.5
5.	Control valve	not needed	needed
6.	Operation and maintenance	Simple and easy	less than original

NO.18 REPORT ON PIPELAYING

JANUARY 1982

Prepared by YOSHIKAZU NISHIDA
Pipeline Engineer
Detailed Design Team
JICA

CONTENT

1. INTRODUCTION

2. ACTIVITIES

2.1. Works in Field
2.2. Works in Office

3. FIELD STUDY

3.1. Objects
3.2. Recording

4. DISCUSSION AND DICISION MADE ON FIELD STUDY

4.1. Which Side of Road
4.2. Other Matters
4.3. Agencies to be Contacted with

5. OFFICE WORKS

5.1. Pipe Thickness
5.2. Method of Pressure Reduction

6. WORKS IN JAPAN

1. INTRODUCTION

The author stayed and worked in Thailand from the middle of December 1981 to the end of January 1982.
His assignment schedule there-after is that he works for one and half months in Japan before his second visit here for another month.

His report will include his activities and findings here, his intention about the works in Japan and his thinking about coordination.

2. ACTIVITIES

2.1. Works in Field

From Jan. 13 to Jan. 15 1982, 3 days were spent in the investigation of field conditions.
The findings are described later in the report.

2.2. Works in Office

The latter half of December 1981 and the beginning of January 1982 were used in studying on the matters about:

* the burying depth of pipe and the clearance from the gas pipe, the riverbed crossing in relation to the gas pipe
* the pipe thickness in relation to the burying depth
* the control of flow in relation to pressure reduction

3. FIELD STUDY

3.1. Objects

(1) Distance

Instead of measuring, the meter of automobile which can be read by 0.1 km was used to check the distance. It is only for confirmation.

(2) Undulation of Pipeline Route

Mainly for locating the drain and air valves.

(3) Details of Crossing River and Highway

(4) Location of Right of Way, Electricity Pole, Building, etc.

(5) Checking Underground Facilities

3.2. Recording

(1) Photography

All the route and the locations of importance were shot by the camera.

The pictures are placed in order in an album with necessary remarks like the distance and noticable features.

(2) Sketches

At the places where special consideration about the construction method and/or the authorization by relevant agency is needed, sketches were made on the spot.

(3) Maps

A map was used to put in the locations of importance where the pictures were shot and the sketches were made.

4. DISCUSSION AND DICISION MADE ON FIELD STUDY

On Jan. 22 1982, a meeting was held at Mr. Boonthai's office attended by Mr. Boonthai, Mr. Wakamoto and Mr. Nishida, pipeline engineer.
The following is the discussion and decision made there and then.

4.1. Which Side of Road

(1) Dok Krai to Route 3191
The both side of road (of RID) can be used.
Wait the completion of survey map and make decision.

(2) Along Route 3191, before reaching Route 36
The east side of Route 3191 shall be used as the same side must be used after the crossing of Route 36.

(3) Along Route 3191, parallel to Gas Pipeline
The west side is scheduled for use of PTT and the east side must be used.

(4) Along Route 3
The sea side has more undulation and less working space for construction than the hill side and moreover the electricity line's voltage is 115 kv on the seaside and 22 kv on the hill side.
Under the said condition, the choice will be the sea side, however another consideration must be taken into.
Route 3, because of its importance on nation-wide road network, is going to be paved wider in the foreseeable future and the sea side is apparently cannot be used for it. If the pipeline is laid on the hill side, the protection against possible heavy traffic in the future must be seriously considered now.
The decision is the sea side.
Consequently, the pipeline must cross Route 3 at the T intersection of Route 3 and 3191.

4.2. Other Matters

(1) Private Property concerning Living

After crossing Route 36, the pipeline will have to run on a private land now used as the yard. The purchase of 5 m width will solve it but compensation for a brick fence and a small hut is needed here. Moving the house further seems unnecessary. In other place of the prospective 5 m width, the land is mostly farming and tree plantation.

(2) River Crossing

Except one case, the river crossing is to be made by burying the pipe under the river bed.

At the point R-19 (about 19.7 km from Dok Krai and R designated for river), a river runs parallel to Route 3191 and the gas pipeline is existing, buried in the road bank.

Mr. Boonthai ordered that the water pipe shall be put on the saddle supported by the pile. The saddles and piles placed in a row will support the pipe so that it will make a continuous equal-span bridge.

4.3. Agencies to be Contacted with

The pipe-laying must be executed under the understanding, authorization and cooperation by the relevant agencies. They are : PTT, DOH, IEAT, PEAT (Provincial Electricity Authority of Thailand?) TAT (Telecommunication Authority of Thailand?)

The matters to be raised between the agencies and the detailed design team are not elaborated here, as the assistant leader will do so in the* other papers.

* Mr. Wakamoto's Comment.

It is taken up in Chapter 4 of Memorandum at Beginning of Feb. 1982.

5. OFFICE WORKS

5.1. Pipe Thickness

A report titled "Thickness of Pipe" is being prepared. So far the thickness 11.9 mm has been taken and 10.3 mm shall be checked newly.

5.2. Method of Pressure Reduction

As was reported in "Memorandum for Meeting with Minutes" used for January 14th meeting, technical and economical study of the subject is important and these matters are better studied in Japan on the requirements which have been decided here.

6. WORKS IN JAPAN

The maps of pipeline route will be completed here in the middle of February and the geological study will be available at the end of February 1982.
The pipeline alignment will be made in Japan in the latter half of February and if possible, the maps filled with alignment will be sent to Thailand. Otherwise, the author will bring them to Thailand on his next visit in the middle of March 1982. In the meantime, the assistant leader will contact with the relevant agencies for public relation purpose and when the materials are prepared, he will start talking with them officially.

The author, after determining the pipeline alignment, will have to prepare the materials to be used for the meetings with the relevant agencies, as mentioned above.
Then he can proceed to make the bill of quantity for estimating the cost of construction.