APPENDIX A-4. IRON AND MANGANESE REMOVAL OF GROUNDWATER

1. Introduction

The United Arab Report of Ministry of Housing and Public Utilities compiled in 1964 says "A general survey of our underground water source was indicated that about 10% of these sources contain iron and manganese salts in their water at fairly high concentration." The General Organization for Greater Cairo Water Supply does not employ groundwater containing Fe and Mn more than the standard values, and has a policy of abandoning artesian wells which may come to produce water containing more Fe and Mn than the standard values.

Considering the fact that the Nile water is not easy to treat owing to high concentration of algae, it may be desirable to use groundwater even if Fe and Mn must be removed. Their removal is technically easy, hygienically it is safer, and it is probably more economical. In the case of the Greater Cairo, there are two significant reasons, namely, (1) groundwater not used only because of Fe and Mn, becomes possible for use as water source, and (2) colouring troubles occuring due to the present excessively-high permissible values of Fe and Mn of the Standards (reported in another paper "Water Quality Standards") can be remedied by some appropriate measures.

2. Methods of Removing Fe and Mn

Generally speaking, manganese is more difficult to remove than iron, and methods for manganese include those for iron too. When Fe and Mn contents are low, they can be removed by oxidation and filtration; if their contents are high, their larger portion must firstly be removed by a combination of oxidation, coagulation and sedimentation, and then the remainder by filtration. In Japan, there are many examples of using groundwater from both shallow and deep wells containing Fe and

Mn after their removal. Considering the above experience and water supply conditions in the Greater Cairo, the following methods will be advisable.

2.1 Fe plus Mn less than 1.5 ppm

For this case there are the following two adequate methods; (1) Contact Oxidation (Surface Adsorption) Method.

Mn is hardly oxidized by aeration, but it is readily oxidized by contacting with manganese-oxides (for example MnO_2). Utilizing this property Mn can be removed; the method is, first coat the filter sand with manganese-oxides, then filter water containing Mn, and manganese iron will be oxidized contacting the sand surface and absorbed. Such sand as the above is called manganese sand. And the related chemical reaction is as follows;

$$mn^{2+} + MnO_2H_2O + H_2O \rightarrow MnO_2\ MnO\ H_2O + 2H^+$$

The compound produced as $MnO_2\ MnO\ H_2O$ does not have contact oxidation ability, it must be oxidized and made as active oxides. For this purpose chlorine is usually used. And the reaction is as follows:

$$MnO_2\ MnO\ H_2O + Cl_2 + H_2O \rightarrow 2MnO_2H_2O + 2H^+ + 2Cl^-$$

For practical application, prechlorination must be so much that free residual chlorine in the filtrate be 0.5 to 1.0 ppm. By doing so, manganese oxides will continue catalistic action. Experience shows that a retention time of 15 to 20 minutes after prechlorination is very effective for removal of Mn.

Method to make manganese sand is as follows;

i) Add 3% solution 150 l of manganeous chloride ($MnCl_2\ 4H_2O$) to 1 cu m sand, mix thoroughly, and then heat it to dry.

ii) Add 3% solution 75 l of potasium permanganate ($KMnO_4$) to the above sand, mix thoroughly, and then dry by heat.

In the above processes, the two chemicals react as shown below, and form brown to black brown film coating the surface of sand.

$$3Mn^{2+} + 2KMnO_4 + 7H_2O \rightarrow 5MnO_2\ H_2O + 4H^+$$

There is a simpler method which does not use manganese sand at first. Water containing manganese oxidized by prechlorination will be filtered in this case through the ordinary sand filter, then the surface of sand will gradually be coated with manganese oxides. It is economical but takes 0.5 to 1.0 month until it works perfectly.

Oxidation of manganese takes more readily place if pH is higher. As the Nile water has high pH and alkalinity, no lime treatment will be needed.

In Japan this contact Oxidation method is in wide use and very effective. Pressure filter with sand layer more than 2 m would be able to treat at a rate of more than 300 m/day.

(2) Iron Bacteria Method

Iron bacteria is a kind of microorganisms which oxidize Fe and Mn in water and accumulates it around its cell. It grows largely in water containing Fe and Mn. This property can be used for removal of dissolved Fe and Mn. The method is to filtrate such water through a slow sand filter with iron bacteria grown on the sand surface and in the sand layer. Filtration rate is about 30 to 40 m/day, so required filter area is about 1/10 of slow sand filter. Removal efficiency for both Fe is 90 to 95% depending on the kind of iron bacteria and filtration rate. Iron bacteria is easily grown; when groundwater containing Fe and Mn filtered continuously, iron bacteria will spontaneously grow. Inoculation of iron bacteria will accelerate its growth.

After some period of filtration, the film of bacteria will grow too thick that the filter is clogged similarly to the ordinary slow sand filter. Then, the bacteria's layer must be scraped to a large extent, or backwashed not completely. Backwash rate will be 0.3 m/min and backwash time 5 min. Removal efficiency is low immediately after backwash, but it will recover after 2 hour filtration.

This Iron Bacteria method is practiced at more than ten plants in Japan, with satisfactory results. The merits of this method are that it is economical with no chemicals used, it requires no difficult technique, and it removes not only Fe and Mn but also odour, ammonia and hydrogen sulphate leading to an overall improvement of water quality.

2.2 Fe Plus Mn more than 1.5 ppm

Contact oxidation method produces too much sediments for such water that the filter will soon be clogged. So most part of Fe and Mn must be removed before filtration by coagulation and sedimentation. There are two methods depending on which element is more included.

(1) Fe Much and Mn Little

Such water must be treated in the following order;

i) By aeration Fe to be oxidized and separated from water

ii) By chlorination Mn to be oxidized and separated.
In this case dosage of chlorine should be so much that 0.5 to 1.0 ppm free chlorine remains in the filtrate

iii) To coagulate with alum

iv) To precipitate by settling.
Slurry contact clarifier is preferable because of contact oxidation effect

v) To filtrate
Manganese sand for filter media will remove from the start.

Lime treatment before i) and ii) accelerates oxidization. The Nile water, however, does not require it as it contains alkalinity in sufficiency.

(2) Fe Little and Mn Much

It must be treated in the following order

i) To chlorinate as much as for free residual chlorine to be 0.5 to 1.0 ppm in the filtrate

ii) To coagulate with alum

iii) To remove most of oxides by sedimentation.
Slurry contact clarifier would be more effective because of contact oxidation

iv) To remove the remainder by filtration.
Manganese sand bed would be more effective.

Potasium permanganate, if fed between i) and ii) will accelerate oxidation. The dosage of it is equivalent to Mn. If ammonia nitrogen exists it weakens ability of oxidation of chlorine. Therefore, potasium permanganate should be used.

3. Conclusion

According to the United Arab Report, waters from both shallow wells and deep wells contain iron 0.3 to 0.45 ppm and manganese 0.3 to 0.6 ppm, nearly same concentration. Later investigations show the ratio of the two is not always similar but depends on cases. However in any case where present they are always present together. And the selection of treatment method will be made in accordance with the following considerations:

i) Contact oxidation method will be employed for water containing Fe and Mn below the values of the Standards for the existing wells.

ii) Iron bacteria method will be studied for water containing ammonia nitrogen or hydrogen sulphate together with Fe and Mn of the above values.

iii) In case Fe and Mn are over 1.5 ppm in total, the combined method must be adopted, in principle, of chlorination, then coagulation plus sedimentation, then filtration. Aeration will not be employed because of its dear cost.

iv) Of the above water, additional use of $KMnO_4$ will be checked if much ammonia nitrogen (over 0.5 ppm) is contained; aeration if hydrogen sulphate contained.

v) In connection with the above cases, slurry contact clarifier will be used for sedimentation, and manganese sand for filtration.

APPENDIX A-5. IMPROVEMENT OF SEDIMENTATION BASINS FOR BETTER EFFICIENCY

1. General

Settled water with good quality will ensure not only treated water of good quality but also high efficiency of production, leading to lengthening the filter run and decreasing the frequency of filter washing, and further saving use of treated water for washing. It results in increase of production of the plant. A study is therefore made for improvement of the sedimentation basin.

The existing sedimentation basins are classified into two major types: up-flow and horizontal-flow basins. Out of the two types, the horizontal-flow sedimentation basins are treated and analysed for improvement works, which will not require much time and cost to execute. Treatment plants with sedimentation basins of this type are the South Giza, the Rod El Farag and the Kafr El Elw plants. The basins of the South Giza plant are selected for study considering that this plant has been designed recently based on the current practice. The basin consists of a flocculation basin and a settling basin, and in this study both of them will be dealt with. In addition to the improvement works, a typical design of the sedimentation basin will be presented for future treatment plants.

2. Capacity and Dimension of Sedimentation Basins

Design capacities, dimensions and important values concerning flocculation and settling basins are summarized in the following paragraphs.

2.1 Capacity of the Treatment Plant

Production capacity 69,100 cu.m/day = 0.800 cu.m/sec
Intake capacity 76,000 cu.m/day = 0.880 cu.m/day,
which is equivalent to 110% of production capacity.

2.2 Flocculation Basin

a. Number of basins	6
b. Flow for each basin	12,670 cu.m/day = 8.80 cu.m/min = 0.1467 cu.m/sec
c. Volume of each basin	157.2 cu.m
d. Retention time	17.9 min = 1,072 sec
e. Depth	3.00 m
f. Width of channel	0.60 m
g. Cross area of channel	1.80 sq.m
h. Velocity in channel	0.082 m/sec
i. Head loss in basin	0.060 m in total, out of which 0.050 m is by inlet weir
j. G value	23.3 sec^{-1} (including weir) as calculated 9.5 sec^{-1} (basin only) as calculated
k. GT value	25,000 (including weir) as calculated 10,200 (basin only) as calculated
l. Ratio of operations of perforated wall	4.6%
m. Velocity at opening	0.148 m/sec

2.3 Settling Basin

a. Number of basins	Same as flocculation basin
b. Flow for each basin	Same as flocculation basin
c. Volume of each basin	1,397 cu.m
d. Retention time	2.65 hr
e. Depth	4.00 m
f. Width	7.125 m
g. Length	49.0 m
h. Mean flow velocity	0.309 m/min
i. Overlfow rate	36.3 cu.m/day/sq.m
j. Weir loading	1,770 cu.m/day/m

3. Present Conditions and Actions Required

3.1. Present Conditions of Sedimentation Basins

The present flow of sedimentation basins, 8 numbers in service, is 105,000 cu.m/day, and it is 13,300 cu.m/day per basin, 3.5% over the designed flow. It may safely be said that the basins are now being operated on the design basins.

Most of the values in 2.2, such as retention time of the flocculation basins, and retention time and mean velocity of the settling basins, etc., as they have been properly established, may ensure good flocculation and settling, but it is thought a few values may be insufficient for the purpose. The following is conditions of floc formation and settling which must be noted.

Fine flocs form at about 1/3 from the entrance in the flocculation basin and grow a little, but still small and fragile, until the exit of the basin. One of the reasons for this poor flocculation may be no dosing of coagulant aids. Main reason however may lie, despite generally proper design, on the constant-intensity flocculation and its insufficiency.

The turbidity of flocculated water is about 20 ppm, and that of settled water is about 10 ppm. From this the settling efficiency is calculated as about 50%. A major factor which lowers the efficiency may be the light and fragile floc currently formed, and besides there may be another factor related to the design of the settling basin. It was observed that the turbidity of water is high at the inlet zone, comparatively low in the middle part and becomes high again at the outlet zone of the settling basin. The reasons for this may be:

(1) Development of density flow with high algal concentration of flocs passing along the bottom of the basin, and

(2) Growth of big approach velocity by weir overflow, dragging flocs from the bottom to the outlet of the basin.

The density flow of the item (1) may be exaggerated by the big change in depth between the flocculation and the settling basins. When the flow which contains flocs comes to the latter basin from the former it may change its direction downward, and then change again horizontally to downstream and runs along the bottom as illustrated in Fig. A-5-1.

As for the approach velocity, it is produced by the flow over the outlet weir of the basin. The weir loading of 1,770 cu.m/day/m is far bigger than that of the usual case, and the large overflow quantity per unit width of the weir will encourage growing of the approach velocity.

3.2 Actions Required for Better Efficiency

Actions required for remedy of the defects mentioned in 3.1 and for obtaining better efficiency of sedimentation are as follows:

For flocculation basins,

i. Strengthening of flocculation intensity	to make flocs dense and heavy,
ii. Adoption of tapered flocculation	to enlarge flocs,
iii. Change of hole size of the upper perforated wall	to decrease development of density flow, and

For settling basins,

i. Installation of perforated wall 7.15 m down from the inlet	to minimize density flow,
ii. Same work as i 7.15 m before the outlet	to cut off density flow; reduce carryover of flocs,
iii. Installation of troughs in the outlet zone	to reduce approach velocity by extended weir,
iv. Placing flushing pipes on upper 1/3 basin bottom	to loosen compacted sludge by water jet.

4. Theoretical Considerations on Design of the Sedimentation Basin

Among the water treatment processes, sedimentation including flocculation is the most difficult one to gain good results, that is, maximum in quantity and best in quality, because it can be obtained only in case that all the following factors are made adequately:

(1) Design of basins
(2) Chemical operation, and
(3) Mechanical Operation.

As the sedimentation basins of the subject have less mechanical parts and the study on the chemical opration is prepared in Appendix A-2, merely theoretical part will be considered here for designing the sedimentation basin.

4.1 Flocculation Basin

Indexes currently in wide use to examine flocculation efficiency and G, GT and GCT values, where G, T and C are velocity gradient, retention time and concentration of suspended solids in raw water respectively. G* and GT** values were brought forward by T.R. Camp and GCT*** value by N. Tanbo.

As the concentration of suspended solids in the Nile water does not vary much after the completion of the High Dam, the flocculation efficiency will be examined by using G and GT values. Appropriate range of G value is 70 to 10 sec^{-1} and for tapered flocculation which enlarges the floc size the higher value is applied to the upper part of the basin and the lower value to the downward part. The range of GT value is 30,000 to 150,000, and a higher value of the range may produce better efficiency of flocculation though more costly in construction.

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* T.R. Camp: Velocity gradients and internal work in fluid motion. Jour. Boston Society of Civil Engineering Vol. 30, 1943.
** T.R. Camp: Flocculation and flocculation basins. Jour. of ASCE Vol. 179, 1953.
*** N. Tanbo: Floc Formation in pipelines. Jour. of Japan Water Works Association. Vol. 426, 1970.

In the case of the channel flocculation as of the subject, G and GT values are expressed as

$$G = (gh/kT)^{1/2} \quad \text{.............................} \quad (1)$$

$$GT = (ghT/k)^{1/2} \quad \text{.............................} \quad (2)$$

where g: Acceleration of gravity,

h: head loss in basins,

h: kinematic viscosity,

T: retension time of basin.

(1) GT Value

The magnitude of head loss in the basin to give GT value 50,000, a lower value of the appropriate range, will be calculated for the average temperature 20 degrees C. From equation (2) the head loss is

$$h = k.GT^2/gT \quad \text{.................................} \quad (3)$$

where k: 0.0131 cm^2/sec

g: 980 cm/sec^2

T: 1,072 sec.

Substituting these values to equation (3), the head loss is obtained as

$$h = \frac{0.0101 \times 50,000^2}{980 \times 1,072} = 24.0 \text{ cm}$$

This magnitude of head loss will be used for planning.

(2) G Value

In order to have tapered flocculation, the size of G value is lessened to downwards, and the following values are applied for the planning:

$G = 65\ sec^{-1}$ for upper part, retention time 319 sec

$G = 45\ sec^{-1}$ for middle " 362 sec

$G = 25\ sec^{-1}$ for lower " 391 sec

Employing these values for G, the head loss in each part is calculated by equation (4), transformed from equation (1):

$$h = kT.G^2/g \quad \text{...............................} \quad (4)$$

Head loss for each part is

$$h_1 = 0.0101 \times 319 \times 65^2/980 = 13.89 \text{ cm},$$
$$h_2 = 0.0101 \times 362 \times 45^2/980 = 7.55 \text{ cm},$$
$$h_3 = 0.0101 \times 391 \times 25^2/980 = 2.52 \text{ cm},$$

where h_1, h_2 and h_3 are the head losses in upper, middle and lower part of the basin respectively. The total loss in the basin is $h = h_1 + h_2 + h_3 = 13.89 + 7.55 + 2.52 = 24.0$ cm, and this magnitude of head loss is equivalent to that calculated in 4.1. (1).

4.2 Settling Basin

There are many factors for the design of settling basins such as overflow rate, detention time, mean flow velocity, etc. Out of these factors, the overflow rate is the most important one. On the other hand, means which ensure the well designed basin to function at the desired efficiency is to apply perforated walls and outlet troughs. The overflow rate as well as this means will be discussed in the following paragraphs.

(1) Overflow Rate

A surface overflow rate, 15 to 25 cu.m/day/sq.m, is widely adopted for the horizontal flow settling basin in Japan. That used in the United States of America is 15 to 23 cu.m/day/sq.m. Considering the above two cases, the overflow rate of 36 cu.m/day/sq.m is somewhat excessive to gain a good settling efficiency for such light flocs as currently formed. For the heavy floc experienced in the years before the High Dam construction, this rate might have been suitable. Creation of heavy flocs is very imperative at the moment, and so this matter is dealt with in Appendix A-2.

(2) Perforated Wall

Perforated walls are extensively used for the settling basin in order to make flow uniform and reduce the adverse effect of density flow. The substantially improve settling efficiency. An essential factor for the wall is the proper ratio of openings to the cross area of the basin. The smaller the ratio, the more uniformity of flow is obtained; too small ratio however breaks up flocs into tiny pieces because of the accelerated flow through the holes of the wall. So an appropriate ratio of openings must be sought.

Velocity which breaks flocs may be different according to the nature of flocs, but a number of experiments and observations in Japan show that the proper value of the velocity is 15 to 20 cm/sec. This value may be applied to this basin for the reason, in addition to the above-stated result, that the present value which does not destroy flocs at all is 15 cm/sec as mentioned in 2.2, and the ratio of openings to produce this velocity shall be calculated and employed in the design of the basin.

(3) Outlet Trough

At the outlet of settling basins troughs are often installed to minimize carryover of flocs. The basin of this plant is not equipped with such troughs but a full width weir at the outlet, so the weir loading is as high as 1,770 cu.m/day/m.

The loading widely used in the United States of America is only 150 to 180 cu.m/day/m for low turbidity water with light flocs and it is one-tenth of the above value. The value used in Japan is 400 to 500 cu.m/day/m when a perforated wall with proper openings is installed just before the outlet troughs.

Considering the characteristics of the Nile water in addition to the above, the present value shall be lssened to some extent. As a perforated wall is to be installed near the outlet of the basin, such a small value as 150 cu.m/day/m may not be necessary, but it is preferable to lessen the value than 400 cu.m/day/m taking into account the nature of raw water and the light floc. Hence, the value of 180 to 200 cu.m/day/m is selected and applied for the weir loading of the outlet trough for the basin.

5. Design of Improvement Works

On the basis of the considerations made in 3.2 and 4, improvement works of the sedimentation basin are designed hereunder, and locations and outlines of the improvement works are illustrated in Fig. A-5-1.

5.1 Flocculation Basin

(1) Installation of Baffles to Channels

In order to gain a tapered and sufficient-intensity flocculation, baffles will be installed to the existing channels of the basin. The number and the size of the baffles are determined by the magnitude of head losses for each part, and they are shown in the drawing of Fig. A-5-2.

By this modification, the water level at the inlet of the basin will rise by 0.26 m from the designed one including a rise of 0.10 m caused by outlet troughs. Designed water levels in the basin and that modified are compared in the table below.

Table A-5-1. Comparison of Water Levels

Location	Water Level: Designed	Water Level: Modified
Before inlet weir	26.48 m	26.74 m
Beginning of basin	26.41	26.72
Enf of basin	26.40	26.50

(2) Raising the Height of Wall Top

In case of channel flocculation as this, a big head loss in the basin is produced, and the top of wall of the flocculation basin is often 10 to 30 cm higher than that of settling basin. By the above modification the water level will rise to 6 cm below the top. The top of wall has therefore to be raised by 20 cm so as to have a sufficient free board. This is shown in Fig. A-5-1.

(3) Modification of the Existing Perforated Wall

There are two existing perforated walls at the outlet of the basin. The upper one has rectangular openings and the lower circular holes.

Fig. A-5-1 MODIFICATION OF SEDIMENTATION BASIN.

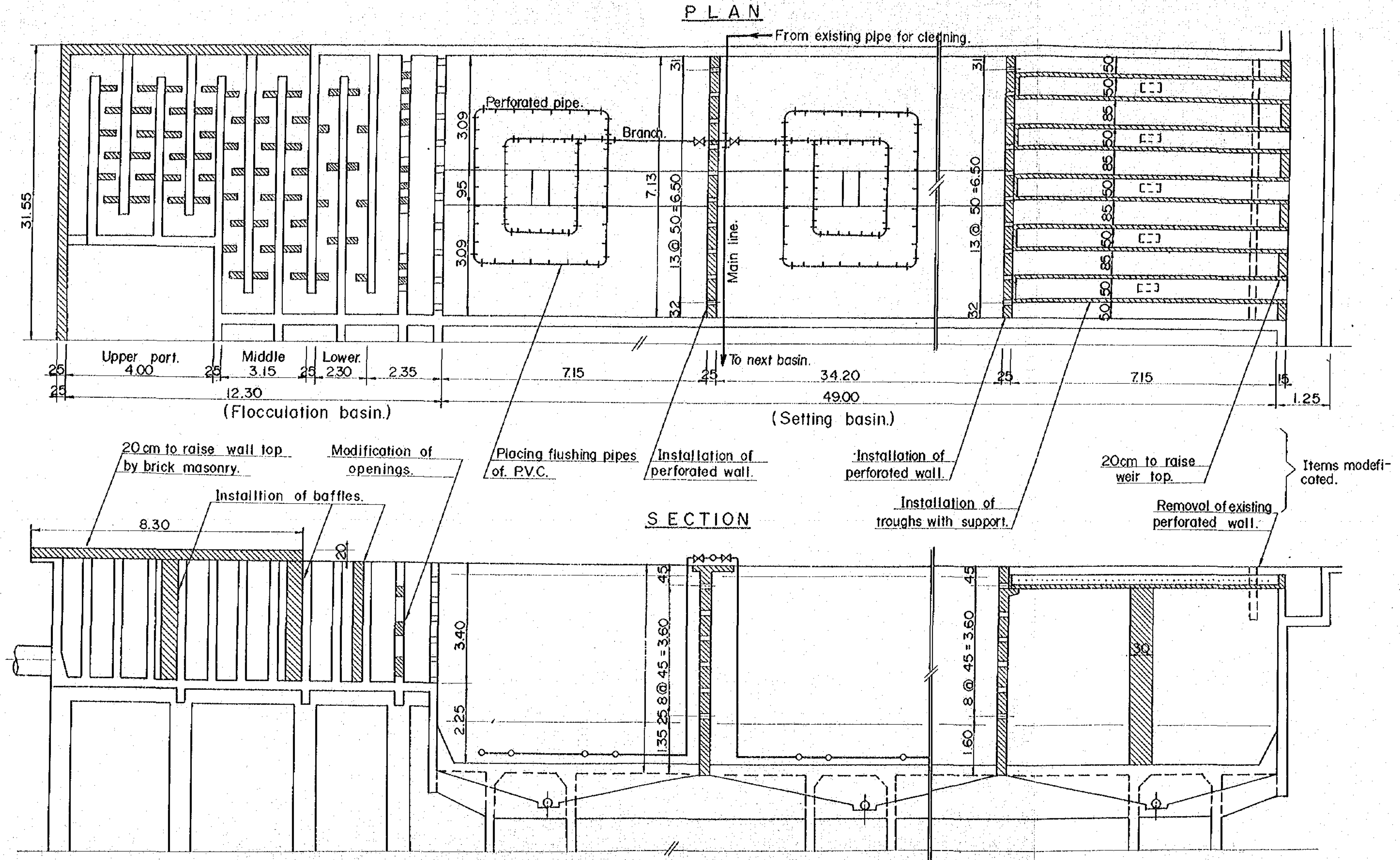


Fig. A-5-2 INSTALLATION OF BAFFLES

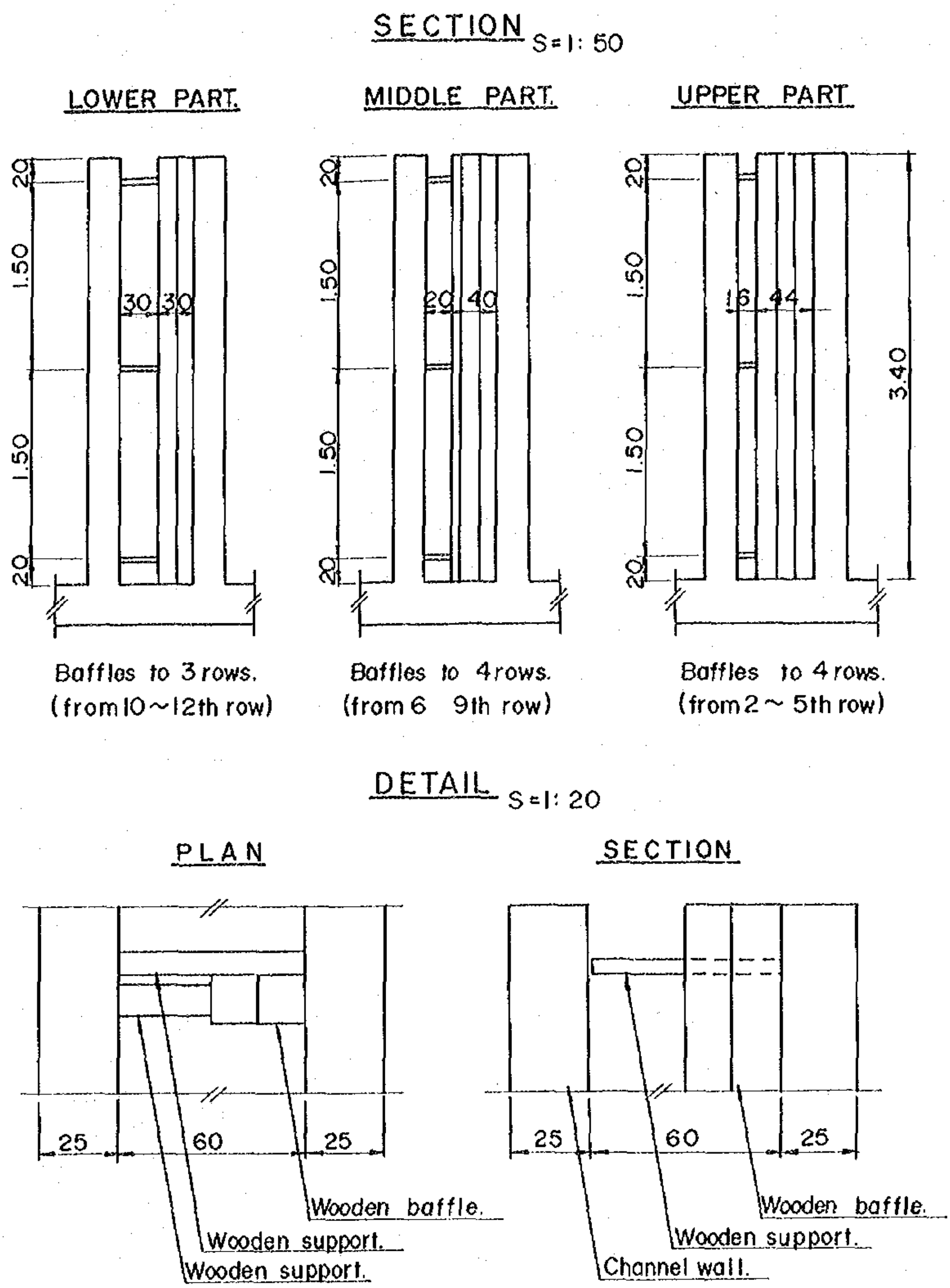
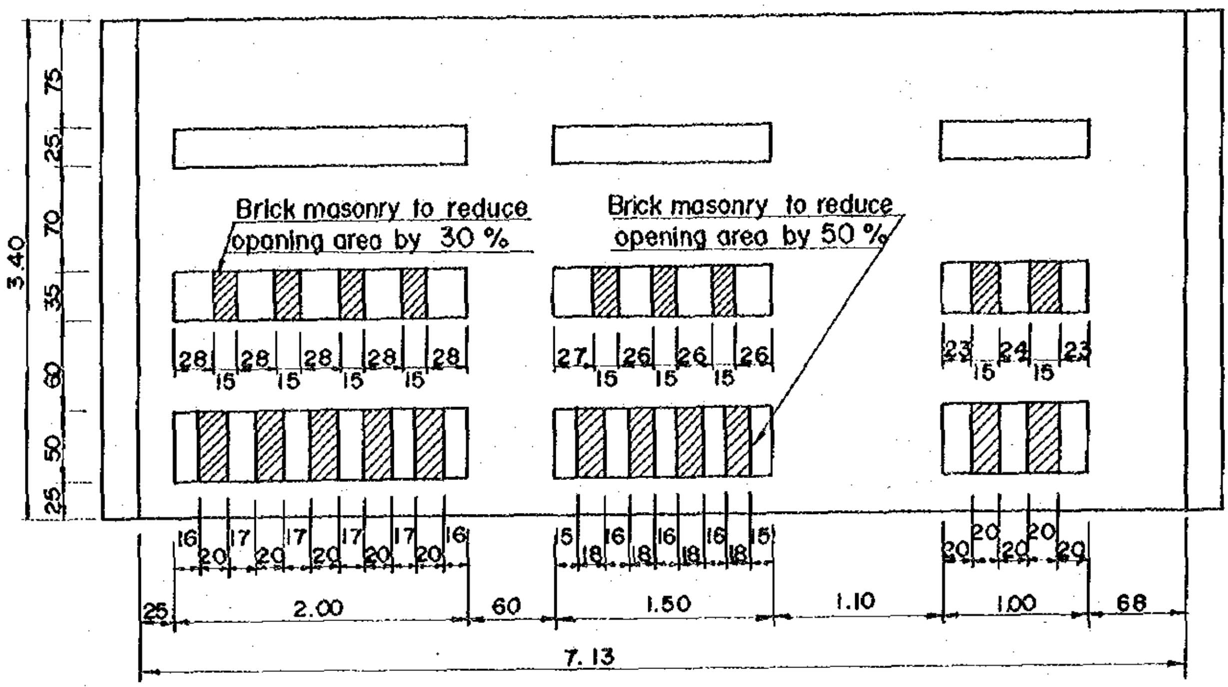
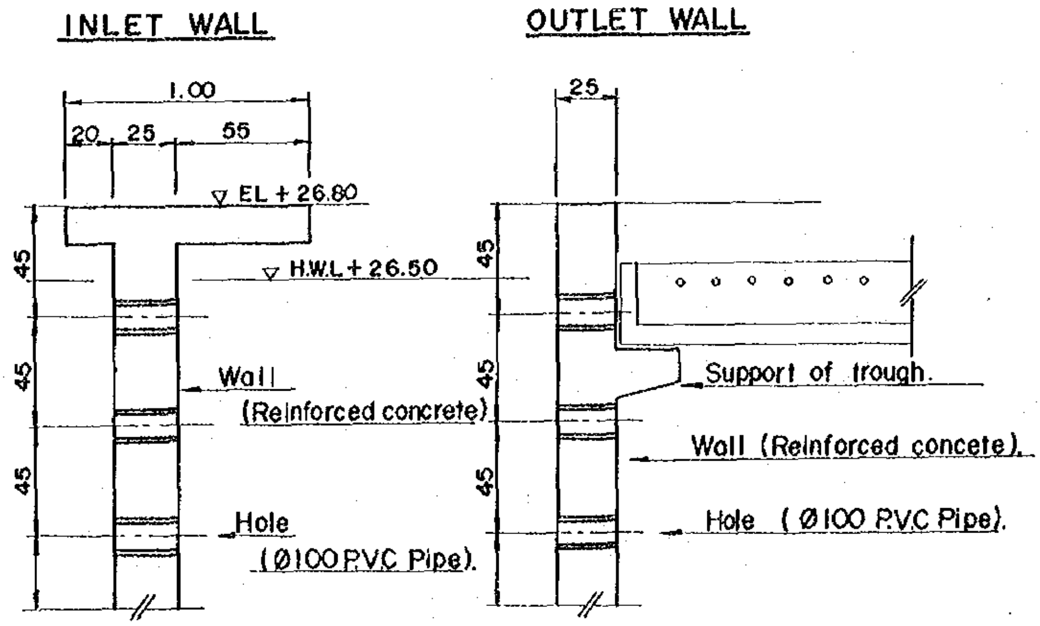


Fig.A-5-3 IMPROVEMENT OF PERFORATED WALL

MODIFICATION OF EXISTING WALL S = 1:50



INSTALLATION OF PERFORATED WALL S = 1:30



The size of the rectangular openings is smaller at the higher part and becomes larger to the bottom. For this reason, majority of flow passes the lower part of the wall leading to producing the density flow into the basin. The openings near the bottom will therefore be reduced in size as shown in Fig. A-5-3, which will allow a flow velocity of 4.3 cm/sec. for the designed flow.

5.2 Settling Basin

(1) Installation of Perforated Walls

Two perforated walls will newly be installed in the settling basin: one near the inlet and the other near the outlet of the basin. For determining the openings area of the wall, the mean velocity 0.15 m/sec for the wall mentioned in 4.2.(2) is employed and the area (A) is calculated from this velocity (v) and the discharge (Q) to the basin:

$$A = Q/v = 0.1467/0.150 = 0.978 \text{ sq.m}$$

Supposing the diameter of each hole d = 0.10 m, the number and an appropriate arrangement of the holes are given as

$$a = 3.14 \times 0.1002/4 = 0.00785 \text{ sq.m, and}$$

$$N = A/a = 0.978/0.00785 = 125,$$

where a: area of each hole,

N: number of holes.

The holes are arranged at a space of 45 cm vertically and 50 cm horizontally, and the total number of the holes is 14 x 9 = 126 for the wall near the inlet and 126 - 8 = 118 for that near the outlet. 8 out of 126 holes will be closed by outlet troughs to be installed. These are shown in Fig. A-5-3. The opening ratio of the walls is:

$$R = A/Ao = Na/Ao = 126 \times 0.00785/(7.125 \times 4.00) = 3.4\%$$

for the inlet wall, and R = 3.3% for the outlet wall,

where R: openings ratio of the wall,

Ao: cross area of the basin.

(2) Installation of Outlet Troughs

Between the perforated wall near the outlet and the existing outlet weir, outlet troughs will be placed. The length between them which is equal to that of the trough is 7.15 m. Employing 180 cu.m/day/m for the weir loading, the number of troughs for each basin is calculated as

$N = Q/(2LR)$

where N: number of troughs,
Q: discharge,
L: length of troughs, and
R: weir loading

Therefore $N = 12{,}670/(2 \times 7.15 \times 180) = 5$.

Supposing the width of each trough 0.50 m and using the above N and other values concerned, water levels for the troughs and in the basin are calculated as shown in the table below, being compared with those for the designed flow. The drawing for the installation of the troughs is shown in Fig. A-5-4.

Table A-5-2. Water Levels in Settling Basin

	Water Level	
Location	Designed	Modified
In basin	26.40 m	26.50 m
Beginning of trough	...	26.42
End of trough	...	26.37
Outlet channel	26.30	26.30

(3) Flushing Pipe in the Sludge Hopper

Flushing pipe will be fitted to the upper two rows of sludge hoppers in order to flush down the settled and compacted sludge along the sloped side of the hopper. The reason for not fitting the flushing pipe to the remaining two-third hoppers is not to disturb and let settled light flocs rise up in the water. The flushing pipe is made of PVC pipe with a row of small holes and fixed to the hopper as shown in Fig. A-5-4. Pressurized water for flushing will be supplied by branching from the existing pipe for cleaning the basins.

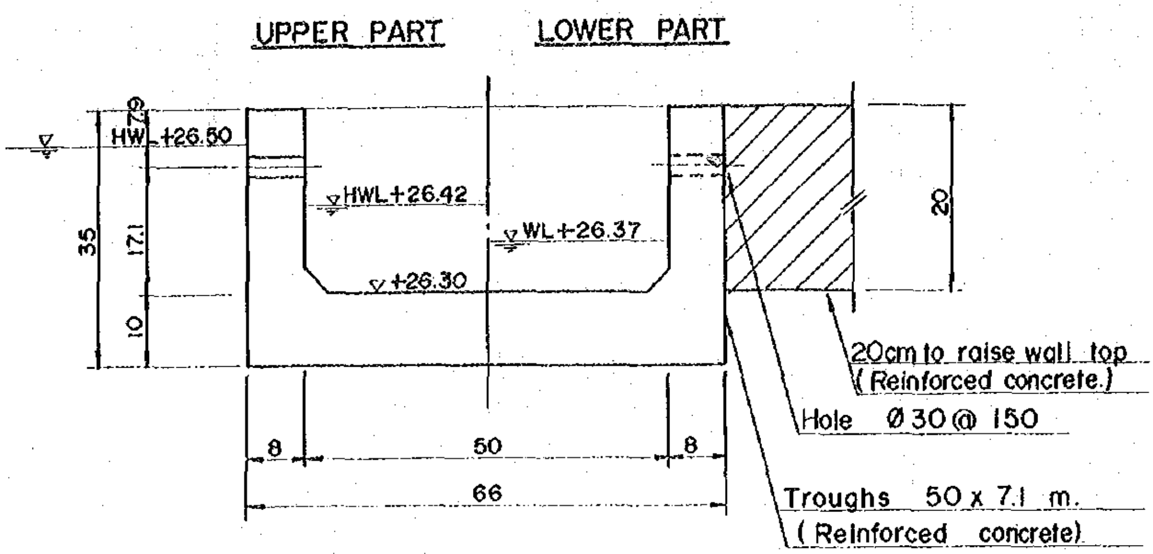
(4) Others

The existing perforated wall at the outlet will be removed, and the existing weir at the outlet will be raised by about 20 cm so that settled water is taken all from the troughs.

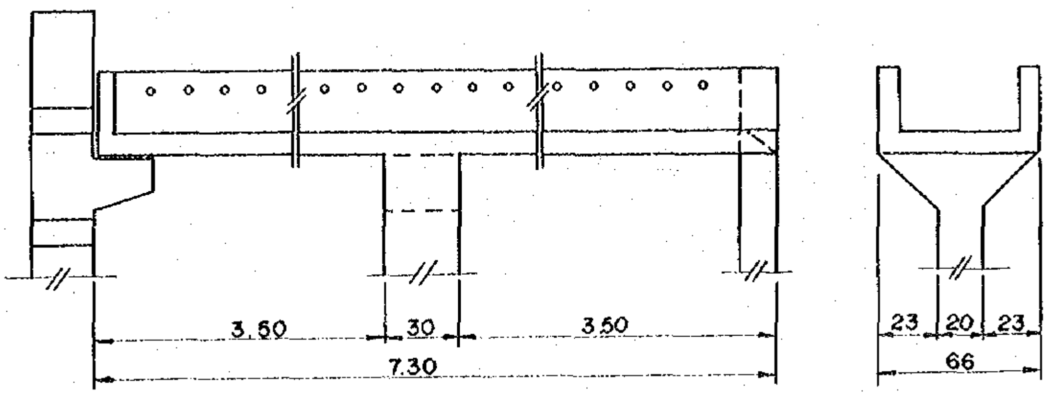
Fig. A-5-4 INSTALLATION OF TROUGH & PIPE.

EQUIPMENT OF OUTLET TROUGHS

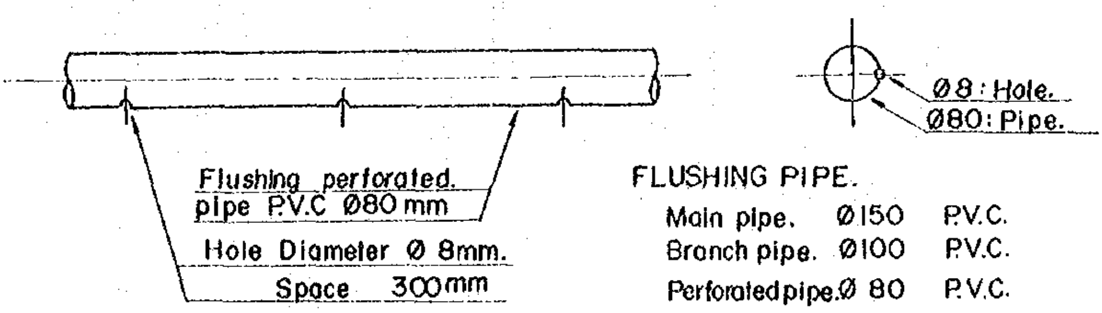
DETAILED SECTION S = 1 : 10



SECTION S = 1 : 30



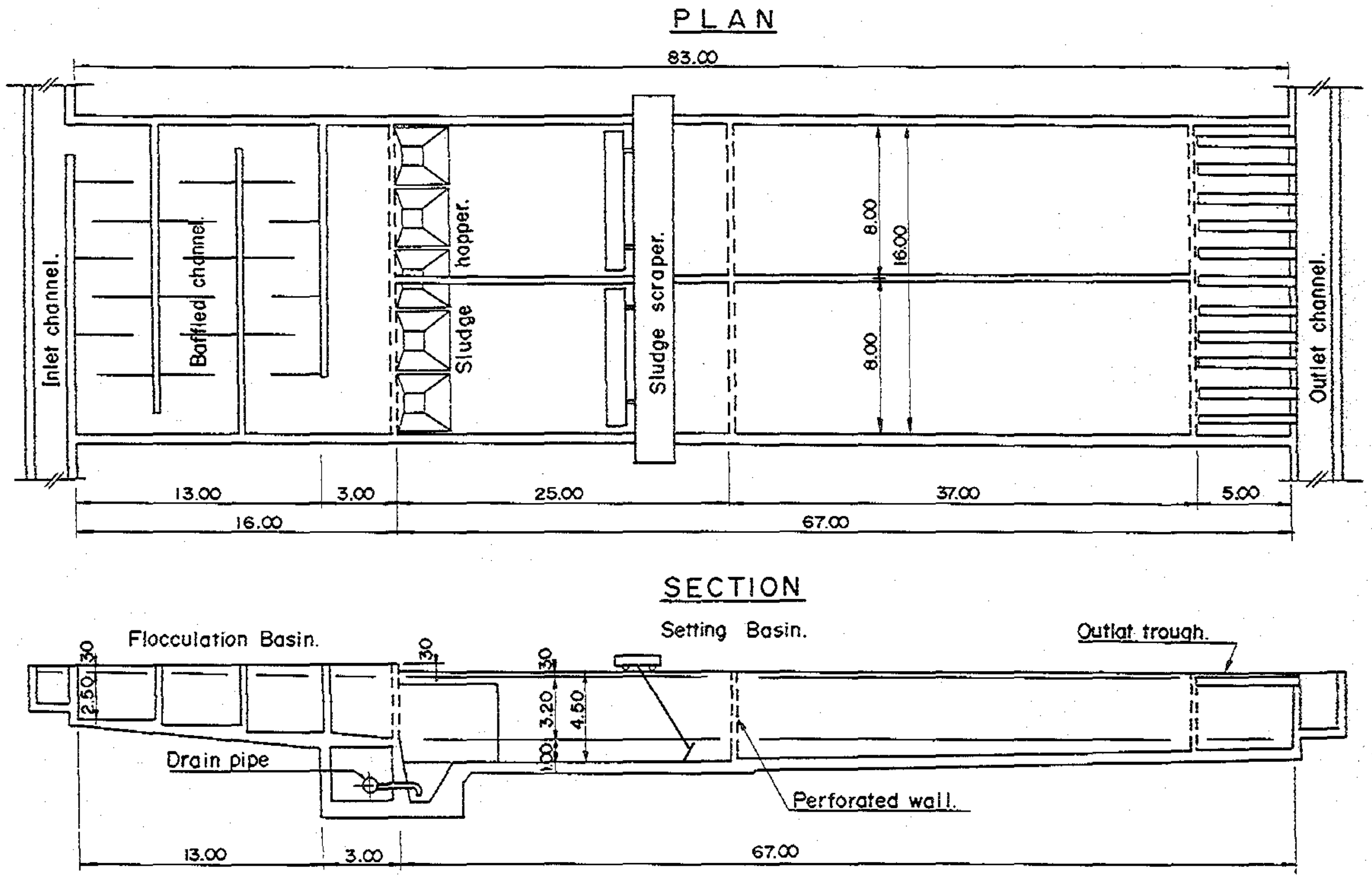
PLACEMENT OF FLUSHING PIPE



FLUSHING PIPE.

Main pipe.	Ø150	P.V.C.
Branch pipe.	Ø100	P.V.C.
Perforated pipe.	Ø 80	P.V.C.

Fig.A-5-5 GENERAL DRAWING OF SEDIMENTATION BASIN S=1:300



6. Cost Estimate

Cost required for the improvement works so far described is roughly estimated as shown in the following table.

Table A-5-3. Estimated Cost of Improvement Works

Work Items	Materials	Quantity	Unit Price	Cost for a Group
Flocculation Basin				
Installation of baffles	Timber	53.0 cu.m	160	8,480
Raising of wall top	Brick masonry	8.6 cu.m	15	129
Modification of wall	Brick masonry	1.6 cu.m	15	24
Settling Basin				
Installation of inlet perforated wall	Reinforced concrete	40.9 cu.m	60	2,454
	P.V.C. pipes ϕ100	126.0 m	8	1,008
Placement of flushing pipes	P.V.C. pipes ϕ150	35.0 m	13	455
	ϕ100	20.0 m	8	160
	ϕ80	192.0 m	5	975
	Sluice value ϕ100	8	(200)	(1,600)
Installation of outlet perforated wall	Reinforced concrete	37.0 cu.m	60	2,220
	P.V.C. pipes ϕ100	126.0 m	8	1,008
Equipment of outlet troughs	Reinforced concrete	21.6 cu.m	100	2,160
Removal of wall	Reinforced concrete	8.0 cu.m	12	96
Raising of wall top	Concrete	0.7 cu.m	18	13
Miscellaneous				818

Estimated cost for a group of basins,

Local Currency Portion	£E 20,000
Foreign Exchange Portion	US$ 1,600

Total cost for all the groups of basins for final stage,

Local Currency Portion	£$ 60,000
Foreign Exchange Portion	US$ 4,800

Note: This treatment plant will consist of 3 groups of sedimentation basins in the final stage and a group is composed of 4 flocculation and settling basins.

Values in ( ) is of foreign exchange portion of which unit is US$.

7. A Typical Design of a Basin for Future Treatment Plant

A typical design of sedimentation basin for a future treatment plant, of which the production capacity is tentatively assumed at 600,000 cu.m/day, considered most efficient from the characteristics of the Nile water is made in Fig. A-5-5 applying the following criteria for the design:

1) Capacity Criteria

Intake capacity: 660,000 cu.m/day = 600,000 x 1.10
Production capacity: 600,000 cu.m/day
Number of sedimentation basin: 24
Capacity of each basin: 27,500 cu.m/day = 660,000/24
For this case, appropriate number of filters may be 48.

2) Design Criteria

For flocculation basin

Type	Around-the-end baffled channel type
Retention time	30 min
G value	40 sec^{-1}(70 sec^{-1} upper to 10 sec^{-1} lower)
GT value	70,000
Head loss	30 cm

For settling basing basin

Type	Rectangular horizontal-flow type
Overflow rate	25 cu.m/day/sq.m
Retention time	3 hr
Perforated wall	Openings ratio to allow velocity 0.15 m/sec
Weir loading	300 cu.m/day/m

APPENDIX A-6. PLANNING AND DESIGNING OF PIPELINES

This paper is prepared for engineers of the General Organization for Greater Cairo Water Supply. They are preoccupied with so busy routine work that they have seemingly been deciding necessary pipe diameters by rule of thumb. The result of such design is not always satisfactory, especially in the Greater Cairo where water demand is sharply increasing due to growth of population and expansion of builtup areas. In view of this situation, this paper presents simplified methods of planning and designing of pipelines. It will not require much time nor complicated work, but produce good results, if the methods are followed in their daily work.

1. Design Criteria

All planning and designs must be based on some fundamental conditions; in the case of pipelines, designs lacking such conditions may turn out to be too small or excessively large; both will result in abuse of time and money. The conditions are so-called design criteria. Provisional design criteria, which must be revised in the future so as to suit the real situation by checking each factor obtained from the operation of the water supply facilities, are shown below:

(1) Per Capita Consumption

250 l/c/d for per capita consumption will be employed, as presently used by the Organization. Per capita consumption usually varies with locality in the served area. For example, a residential area has a different value from that of commercial or industrial areas. Therefore, per capita consumption to be used for final design criteria must be determined at a later stage from actual figures. Water demand of a street block will be obtained as product of per capita consumption and population in that block. If the population of that block is not available, it may be assumed from population density which may be obtained from population statistics.

To simplify this operation, unit consumption expressed as l/ha/sec may be useful for design of pipelines. This unit can be obtained by studying a few typical blocks in the served area.

(2) Pressure at Pipe End; Hydraulic Gradient

Pressure at the end of pipelines should preferably be 2.0 kg/sq.cm for secondary mains or 2.5 kg/sq.cm for primary mains, which is a pressure enough to supply to the third floor of ordinary buildings. This figures will have to be revised in the future according to the actual need in various districts. Necessary diameter of a pipeline will be determined from this pressure.

Hydraulic gradient at peak hour will be decided at 2/1,000 to 5/1,000. By this standard, pressure drop in a distance of 10 km is about 20 to 50 m. Most parts of the served area in the Greater Cairo are located within this range, and so supply without booster pumps may be possible except for some distant or high-elevated places.

(3) Coefficient

120 may be an adequate value for coefficient of Hazen-Williams formula, for various losses of head due to bends, valves, etc. are usually omitted and increasing loss of head with years are also omitted in the calculation. Though, new ductile cast iron pipe with mortar lining, use now in the Greater Cairo, may have a coefficient of 140 to 150. Regarding old pipe, coefficient should separately be determined.

(4) Service Area of Each Plant

Although all pipelines are interconnected and there are no definite boundaries for the treatment plants, service area of each plant should be fixed for convenience of design of pipelines. The service area must be so decided that the total demand of that area is equivalent to the production of the related treatment plant. It must also be noted that in case a pipeline from a treatment plant is to be connected with another pipeline from another plant, the calculated diameter will be modified to a diameter necessary for emergency cases.

2. Procedure for Planning and Designing

Procedure and order of the work to be taken in planning and designing are as follows:

(1) Determine blocks. A block is a unit supply area which is to be treated as a supply point. A street block is usually equivalent to this block.
(2) Determine water demand by each block, using 1. (1) Per Capita Consumption.
(3) Determine the route of pipeline, and the length of the line will be measured on the map.
(4) Calculate the necessary diameter of the line from the above three factors, using tables, diagrams or sliderule. Such tables or diagrams are available in the water supply textbooks.
(5) Take into consideration prospective future developments of adjoining areas at the time of the above work.

3. Simplified Practical Method

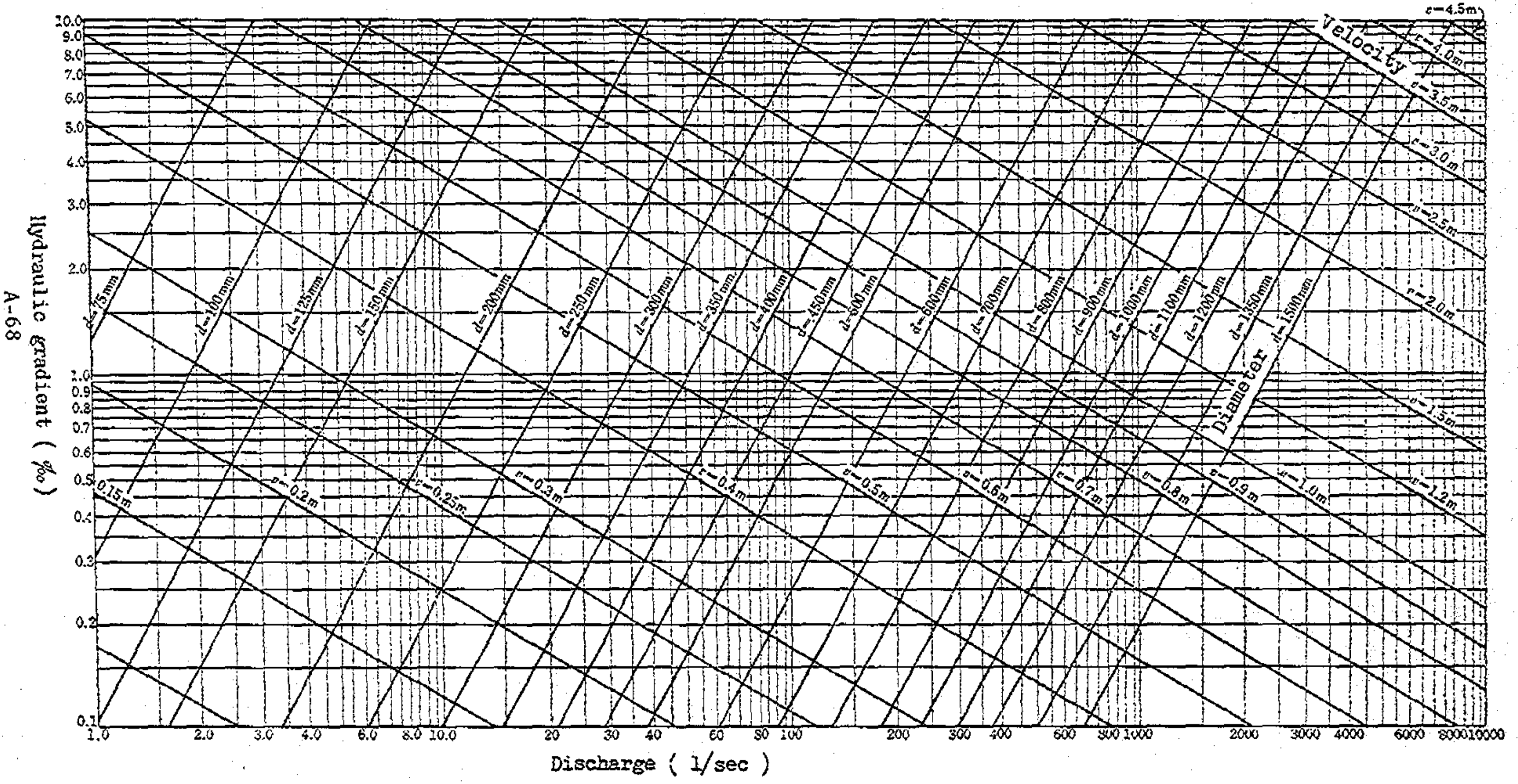
Water supply pipelines composed almost always a network. Computation of a network is usually very complicated. Even without using such computation however a satisfactory result will be obtained by the following method.

--- Dissolve a network into a number of single pipelines, and calculate each pipeline by the above method ---

An area to which water supply is being considered consists generally of many street blocks, and pipelines form a network. But this network can be decomposed into a few lines with branches. Then each pipeline will be treated in the same manner described so far. In this case the results of calculation must be checked to see if water pressure at some connections of two lines is nearly equal.

FIGA-6-1 **FLOW DIAGRAM OF HAZEN-WILLIAMS FORMULA**

C = 120



4. Example of Design

An example of design is presonted here, which is made following the procedure described so far.

(1) Determination of Blocks

Fig. A-6-2 shows the area to be designed of this example. The area includes six block, and population of each block is written in the figure.

(2) Water Demand

The water demand of these blocks can be calculated using per capita consumption. In this case, per capita consumption 250 l/c/d is adopted as employed by G.C.W.O., and the ratio of peakhour flow to the average day demand is assumed as 150%. It is presumed that pipelines sized for peakhour will be nearly enough to meet the fire fighting requirement inclusively, because the amount for fire fighting 12 l/sec, used in the Greater Cairo, is small compared with the flows of this case and can be neglected.

Pipe sizes will be based on peakhour demand as shown below:

Block	Peakhour demand	
1	Q_1 = 39,700 x 25 x 1.5 =	172 l/sec
2	Q_2 = 22,500	98
3	Q_3 = 24,200	105
4	Q_4 = 27,600	120
5	Q_5 = 34,600	150
6	Q_6 = 20,700	90 l/sec

(3) Route of Pipeline

The streets where pipes to be laid must be selected depending on the traffic conditions and existing public utilities undergound. The routes of pipeline of this example are determined as shown in Fig. 1. All the lines compose a network.

(4) Decomposition of the Network

The above network can be decomposed into three lines, Line I to III, FIG A-6-2

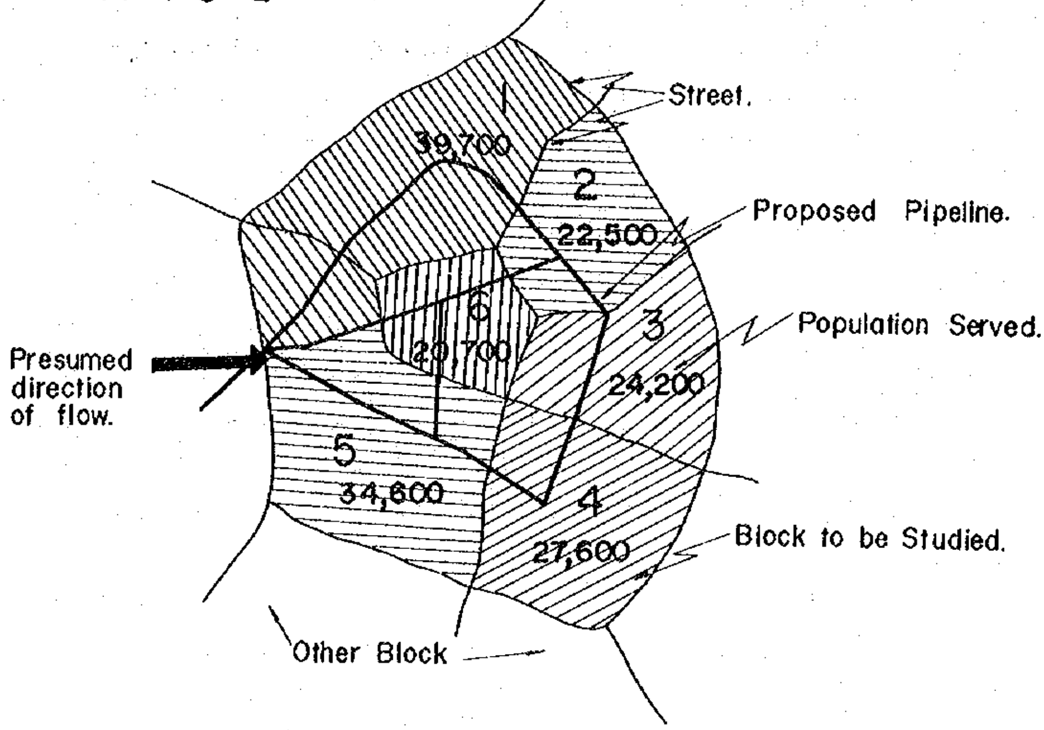
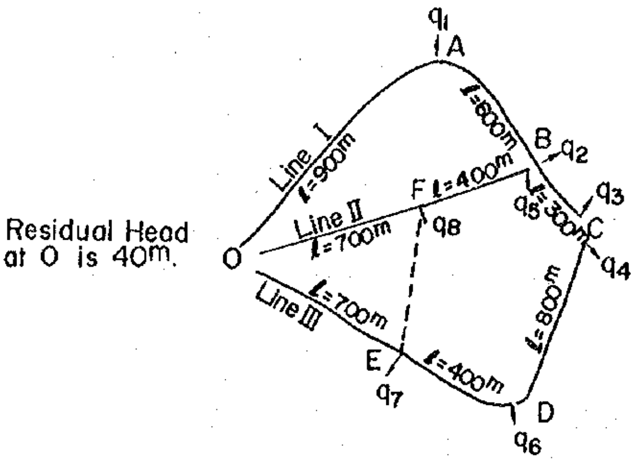


FIG A-6-3



GROUND LEVEL.

A	+ 24.00 m
B	+ 27.00
C	+ 27.00
D	+ 29.00
E	+ 28.00
F	+ 26.00
O	+ 20.00

as shown in Fig. A-6-3, so as to make computation easy. In the same lengths of the lines and the ground levels are all shown.

(5) Discharge at Points

Discharges into the blocks from points A, B, C, D, E, F and O on the pipelines are assumed as below:

Line I:	$q_1 = Q_1$	=	172 l/sec
	$q_2 = Q_2/^2$	=	49
	$q_3 = Q_3/^2$	=	53
Line II:	$q_5 = Q_2/^2$	=	49
	$q_8 = Q_6$	=	90
Line III:	$q_4 = Q_3/^2$	=	52
	$q_6 = Q_4$	=	120
	$q_7 = Q_5$	=	150

(6) Calculation

Firstly, obtain the difference of residual head at O and desirable head at D, from which rough gradient can be calculated. In the flow diagram, a necessary pipe diameter will be obtained from the intersection of the calculated gradient and q_i stated in (5). In this case, the diameter which is closest to the intersection should be selected.

Secondly, real gradient, loss of head as well, will be obtained by the intersection of the selected diameter and q_i. Summing up losses of head on one line, the residual head at the pipe and D will be known.

Finally, check whether the residual head is adequate or not, and whether the two residual heads obtained on the two lines are nearly equal or not. If there is a big difference between the two residual heads, diameter of some span/s must be modified until a satisfactory result is gained.

Results of the above calculation are shown in the following table.

Table A-6-1. Result of Calculation

Span	Pipe Size	Hydraulic Gradient	Loss of Head (m)	Water Table Level (m)	Residual Head (m)
Line I					
O-A	ø500	4.0 0/00	H = 0.9 x 4.0 = 3.6	56.4	32.4
A-B	ø400	1.9	H = 0.6 x 1.9 = 1.1	55.3	28.3
B-C	ø250	5.6	H = 0.3 x 5.6 = 1.7	53.6	26.6
Line II					
O-F	ø400	3.4	H = 0.7 x 3.4 = 2.4	57.6	31.6
F-B	ø300	2.0	H = 0.4 x 2.0 = 0.8	56.8	29.8
(F-B	ø250	4.8	H = 0.4 x 4.8 = 1.9	55.7	28.7)
Line III					
O-E	ø600	2.2	H = 0.7 x 2.2 = 1.5	58.5	30.5
E-D	ø450	2.9	H = 0.4 x 2.9 = 1.2	57.3	28.3
D-C	ø250	5.6	H = 0.8 x 5.6 = 4.5	52.8	25.8

Note: The second correction is needed for (F-B).

APPENDIX A-7, PREVENTION OF WATER WASTAGE

I GENERAL

1.1 Introduction

One of the Most important problems in the water supply business is to increase the amount of water chargeable to the consumers, in other words, to decrease the amount of what cannot be charged to any consumer, and in the Greater Cairo in particular it is important to save the loss of water by leakage, because the production capacity is still insufficient. No matter whether the water supply is operated on the self-supporting basis or not, the idea of operation on the least cost is all the same.

The construction cost as well as operating cost of a water supply is approximately in proportion to the amount of production, and the income from the water sales, which is supposed to support directly or indirectly the cost, is roughly proportional to the amount of water consumed. From this every water supply organization strives to minimize wasteful production and maximize chargeable water. The effort for preventing wastage of water is due to this reason.

1.2 Purpose and Scope of This Paper

This paper is compiled as a guideline for wastage prevention work. It deals in general with causes of wastage, detection work of leaks and general means for prevention of wastage. An emphasis is placed on how to conduct leak detection work. It is very usual that distribution systems have not been designed and executed for convenience of leak detection. Therefore some additional works to remedy this insufficiency are always needed. Without it the work can hardly be performed. This paper handles these matters fairly in detail.

II WASTAGE

2.1 Definition of Terms

Wastage:	amount of water lost by leakage and wasteful use.
Leakage:	amount of water lost by leaks.
Production:	amount of water delivered into the distribution pipes from distribution pumps or a reservoir
Accounted for water:	water used and to be charged.
Unaccounted for water:	water used but not to be charged.

2.2 Visible and Invisible Leaks

Some leaks show up on the surface of ground, but there are numerous leaks which occur under ground and find their way into gutters, sewers, groundwater body, rivers or canals and never appear above ground. The former are visible leaks and the latter invisible leaks.

i) Visible Leaks. These leaks are easy to handle for they are found by people of the water supply business or by the public, and they are informed to the water works. The exact point of leak can be easily found out is immediately repaired. This paper will not treat the leaks of this nature, because they should be and are dealt with as routine work.

ii) Invisible Leaks. They are seldom found by chance. In order to detect them, a particular work of time- and energy-consuming nature must be performed. The leakage resulting from invisible leaks has a tendency of continuous increase. New leaks occur one after another, and leakage from a leaking point ever increase with time. Unless repair is carried out, total leakage will reach quite a high percentage of production.

Invisible leaks cannot be found without using a systematic means. Even if some leaks were found and repaired without such a means, few fruitful results could be expected, and gain from such work would not pay the cost.

The usual practice for leak detection and repair will be described in detail in the following chapters, which cover a wide range of work from analysis of production and consumption to repairing works. And in order to obtain tangible and beneficial results, strenuous efforts must be continued over a long period and besides the work must be repeated over and over again.

2.3 Particulars of Wastage

i) Leakage. Leaks occur at various points of pipelines and plumbing systems. Frequently observed leaking points are as follows:

> Joints of pipes, breaks of pipe, valves, fire hydrants, corporation cocks, stopvalves, joints of pipe and meter, faucets.

ii) Wasteful use. This is caused by the consumer's careless use of water. When the connection is not metered it is likely to cause neglectful use of water facilities, for the consumer does not need to pay for his excessive use because he is charged on a flat rate regardless the actual use. Poor service of the water utility toward the customer invites his neglectful use of water. There are cases where worn packings or faulty faucets are left long unrepaired. The consumers tend not to mind water running 24 hours from such faucets.

2.4 Causes of Leakage

Causes of leakage are diversified, but field investigations show that there are a few number of causes which dominate the leakage. Once they are found out, certain precautions must and can be taken for both materials and works of pipe laying and plumbing.

Causes of leakage which often take place are as follows:

- Improper design. Leaks due to this are sometimes found at bends of pipelines.
- Poor work. There are many cases of this kind of leak.
- Poor quality of materials. Pipes not strong enough are easily broken. Packings are the most usual case of leaks.
- Traffic load. As traffic load increases, more leaks occur.
- Increase of water pressure. When there is an increase of water pressure as a result of improvement work, more leaks are apt to happen.
- Aging and deterioration. Pipes and fittings deteriorate with years and their strength declines and cannot stand any more pressure.
- Injuries by other civil works. Along with the overall development of the area, various sorts of civil work take place on and along the street. They give quite frequently damages to the existing pipe system. There are such cases where the water pipe is joined with pipes for liquid or gas transportation.
- Water of low pH value causes corrosion of pipe and pitting of the pipe shell leading to leak. There are cases where acidic water in the ground causes pitting of pipe.

III DETECTION OF LEAKS

Before field work of leak detection and repair, preliminary and basic work must be completed. Detection of invisible leaks is fairly troublesome and time-consuming. If efforts for detection are carried out without a sound planning and systematic method, they will end up in vain. In this chapter, all the processes of detection of leaks are described in the order of performance of each process. They are summarized in Table A-7-1.

3.1 Production and Consumption; Selection of an Area for Work

It is a very important concern for water works to know what level the ratio of consumption to production stands on, for this figure shows the usefulness of the production, in other words, of all the efforts of water works and of all the investment in water supply.

The ratio varies from water supply to water supply and from place to place within the served area. In a certain place it is as high as 90%, and in another place it is barely over 50%. The low figure indicates that the place has larger leakage than other places.

The work starts with making a table of production and consumption broken down into districts, and checking the ratio. It often occurs that data for making such a table are hardly obtained. In that case, some additional meters must be installed on the lines enabling to produce such data. Such meters are necessary for future's sake. When steps to reduce leakage are urgently required, some provisional means instead of the above must be taken. It will be described at a later stage in this paper.

The area which requires prompt actions will be decided from the above checking, and all this process is done as indoor work.

Table A-7-1. Particulars of Leak Detection Work

Preparation of complete maps of existing pipes and others	Plot on the map all valves, hydrants and house connections. Make clear the location of valves which isolate the work block.
Isolation of work block	Shut valves. Read the main meter (A meter that is specially or temporarily installed for the work.)
Investigation and repair of plumbing	Check all the plumbing systems, house by house Repair completely
Main meter reading	Check the reduction in leakage. If possible, close all house connections, and decide if to proceed to the next step.
Detection of leaks by leak detector	Search for leaks line by line. Night work required. Use detector and stethoscope detector
Repair of leaks of the main	Leaks found must be repaird.
Main meter reading	Check again the reduction in leakage. Shut all the stopvalves when the reading is done. Judge if the work has been done all right.

3.2 Dividing into Work Block

For convenience of field work, that is, detection and repair, the selected area must be divided into work block, the unit size of which is dependent on complexity of the work block and arrangement of pipelines. When it is a congested part of the area, a few street blocks may be included in a unit work block. When it is a spares inhabited part, a unit work block can cover a rather wide area.

A map showing all the pipelines, valves, fire hydrants and, if possible, house connections must be prepared for each work block. It is to be used for decision of operating program, plotting and indicating leaks and their conditions.

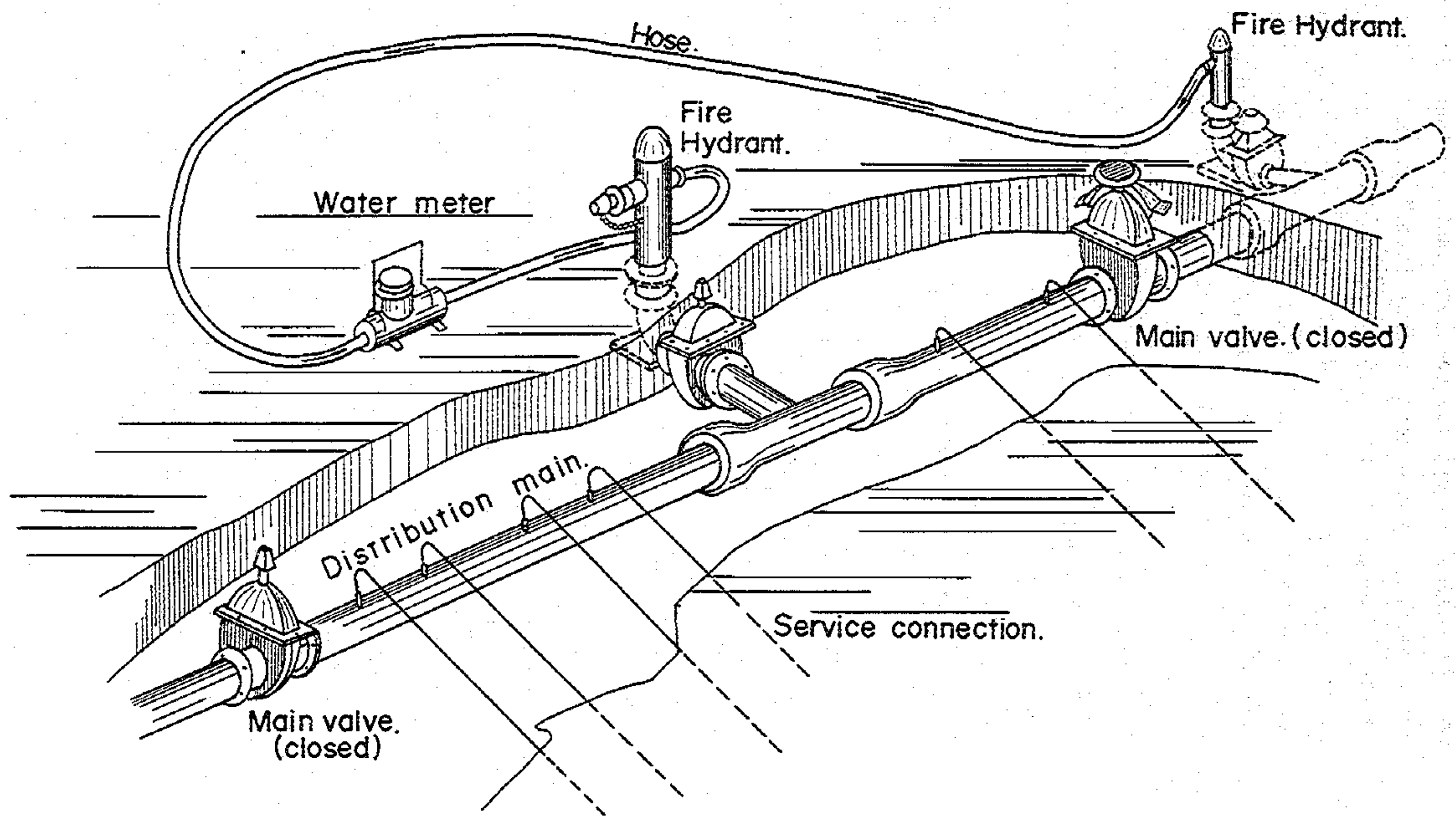
3.3 Field Work in Work Block

3.3.1 Isolation of Work Block

Close all the valves on the lines which connect areas outside of the work block except one line. This is for isolating this work block from other adjoining areas. If a meter can be installed on the line which is left open for this work, the flow of 24 hours can be measured. The least flow in the midnight shows possibly leakage in the work block. From this, total leakage for a day can be roughly calculated. An alternative method for a case where the meter cannot be installed is to connect this isolated block to a neighboring area with a hose. This hose is connected to a fire hydrant in the work block from a fire hydrant in the neighboring area. On the hose line a meter must be put in order to read the midnight least flow. (See attached Fig. A-7-1, Sketch of Hose Connection).

Leakage per km of pipeline is desirable to be as small as possible, but in reality leakage is unavoidable and it is a common practice to set a permissible level to leakage per km of pipeline. This value may be designated as Allowable Leakage, and it can be tentatively set at 50 cu.m/km/day regardless water pressure and diameter of pipe. Some day when prevention of wastage materializes up to a desired point the level should be lowered gradually, from 50 to 30, or to 20 cu.m/km/day.

Fig. A-7-1 SKETCH OF HOSE CONNECTION WITH A FIRE HYDRANT.



There is another standard by which the necessity of leak detection work is judged, except for the case tanks of consumers are taking water all through day and night. The minimum flow in the night is ascribed mostly to leakage. The ratio of the minimum flow to the peak flow in the daytime is taken as an index of the necessity of the work. The permissible ratio must be decided after considering the present condition of water supply. Tentatively it can be fixed at 30%, and after a few cases of actual work this value must be modified to a lower figure.

3.3.2 Repair of Leaks of Plumbing Systems; Reading of Main Meter

The worn-out packings of faucets must be first replaced. And then the meter indicates leakage, if any, with all the faucets closed. The leaking point can be detected by attaching the bar of stethoscope to the pipe. The closer the bar approaches to the leaking point, the louder sounds the earphone.

After completion of the repairing work of plumbing systems, a second reading of the main meter takes place. It is frequently observed that a considerable amount of leakage is reduced by the above work. If the minimum flow is below the allowable leakage or the ratio is below the permissible ratio, which was explained above, no further field work is required for this work block.

3.3.3 Detection of Leaks on the Main

First, the location of the pipe must be confirmed. When it is not known, it can be located by the pipe-locator. The location must be plotted on the map and marked on the surface of the road. Second, along the marked line, testing by the stethoscope is performed. Two or three operators carrying a stethoscope each stand on the line and listen to the leaking sounds. If there are sounds, they try to find out the exact point of leaking. The sounds become louder with the stethoscope approaching nearer to the leaking point. Then the point must be marked on the road and on the map as well.

When the sounds are very suspectable but not clear, the leak detector is utilized. The suspectable sounds can be amplified as loud as to be caught by the ear with ease.

This process of leak detection is to be carried out along the whole mains in the work block. Succeeding this each leak must be repaired as soon as possible.

In parallel to this processes, an observation of gutters or ditches along the road in question is always done in an attempt to find out any leaks from the water pipe. Leaking water, without showing up on the road surface, finds its way directly into a gutter or ditch nearby. If any water flowing out of the earth is found, the nearby pipe must be checked by the stethoscope or detector. Observation of the road surface is sometimes very useful. When there is a leak under ground, it makes in some cases a cavity in the earth and a depression is seen on the surface of the pavement.

Simultaneously with repair work of leaks, packings of valves and fire hydrants are desirable to be replaced. When a valve is operated after a long time of standing, a leak always develops due to aging of packing.

3.3.4 Recording of Work

If the remaining leakage after completion of all repairing work is below the allowable leakage, the leak detection and repairing for this work block is regarded to have been completed.

All the details of the work must be recorded and kept for future reference. An example of recording from is shown below.

FORM

DAILY REPORT ON REPAIR WORKS

Date:

Place:

Plumbing Items	No.	Mains Items	No.
Faucet		Broken pipe	
Replacement		Loosened joint	
Packing		Valve	
Piping		Fire hydrant	
Stopvalve		Disconnection of not used service branch	
Meter			
Others		Others	

Labor	Materials
List of Crew who were engaged in the work	List of materials which were used for the work

Remarks:

3.3.5 Cycle of Work

New leaks may occur soon after repairing work and increase in quantity with time. The cycle of work therefore should not be too long and it is usually decided at two to three years. The shorter the cycle, the better, but the work requires labor, time and materials to a large extent and gives inconvenience to the consumers. The cycle must be fixed based upon the comparison of gain and loss.

The gain is calculated as a product of saved water by unit price of water. The loss is a total of salary, labor cost, materials and depreciation for the equipment. Generally speaking, the gain is far greater than the loss.

At the inception of the work, the cost appears high, because the remedial and additional work for the present facilities, such as installation of valves, main meter, stopvalves, etc., is always needed. But from the second time such work is no more required.

3.4 Organization of Wastage Prevention Section

Prevention of wastage is a complicated work and requires much labor and skill. Unless all the work is well organized, fruitful results cannot be expected. Hence, details of wastage prevention acitivities are first itemized as follows, and then a tentative organization will be suggested.

1) Office Work
   a. Checking of records of production and consumption for the area in question
   b. Preparation of maps of piping
   c. Preparation of materials and equipment
   d. Preparation of field work schedule
   e. Recording

2) Field Work
   a. Field observation
   b. Checking of function of valves involved in the wastage prevention activities
   c. Operation of valves
   d. Installation of additional valves
   e. Main meter reading
   f. Detection of leaks
   g. Repairs.

To carry out the above work, a suggested organization for wastage prevention section is as follows:

Chief Engineer	Engineer	1 Assistant 1 Clerk	Office work
	Engineer	2 Assistants 1 Valve operator 3 Detector operators Repairmen Laborers	Field work

Necessary tools and equipment for the work are as follows:

Small trucks, valve keys, pressure gauges, main meters, stethoscopes, leak detectors, pipe locators, pipe cutters, and other tools for repairing work.

The magnitude of the section for wastage prevention varies by necessity and complexity of the work. The above suggested one is nothing but provisional. The crew for repairing work cannot be fixed before actual needs arise, but preparations for extra repairmen and labor must be made whenever needed.

IV OVERALL PREVENTION OF WASTAGE

As is often said prevention is the best cure; the same applies for wastage. Together with the leak detection and repair, preventive work must be made in order to reduce wastage. Preventive work cannot be expected to produce good results if it is dependent solely on the water works. There is a need for understanding cooperation toward the water works on the consumer side. The water system is widely spread all over the served area and pipes are extended even in the private land hidden from the sight of the public. It makes public cooperation indispensable in the wastage prevention work.

There are many factors which the water works itself can handle. They must first be tackled. In the following paragraphs all these matters will be dealt with.

4.1 Public Relations

4.1.1 Information from the Consumer and Quick Responding Action

Even when the water supply is being carried out successfully, consumers always send in some complaints, and if water is not being supplied under a favorable condition, a great many complaints will pour in. About these complaints some sort of quick action must be taken. For some of them, mere kind and convincing explanations may be enough. But for some others, remedial work on the side of the water works will be needed, or repairing work must be done. For all the complaints, some measures, whatever they are, must instantly be taken. It will promote good relations between the consumer and the water works. Especially as for leaks, they must be repaired as quickly as possible; otherwise, they mean throwing away money.

4.1.2 Packings of the Faucet

Illustrating in figures, 3 drops of water per second from a defective faucet is equal to 48 l/day. Leaks of this much, or more, occur quite often with the faucet. A small leak such as mentioned above develops gradually to a bigger leak. And besides this small flow can pass the water meter without moving the mechanism of the meter. Although the consumer can use and may be using this water, the water works is losing this water, because the meter does not register. It comes to a great amount of water or of lost income in a year to the water works.

From this consideration, replacement of packings is usually done free of charge in most water works. Replacement of the packing is a very easy job if a pincher is available. Some water works furnish packings to the consumer upon his request. This arrangement is helpful for promotion of the customer's interest in the water supply.

4.1.3 Other Activities for Good Public Relations

Activities which lead to good public relations and bring about co-operation to the water works must be looked for and planned. Most water works put news relating water supply in the newspaper, publish pamphlets of water for distribution among consumers, or invite citizens to an observation trip to the treatment plant. All these activities let the consumer know and understand what the water supply is, or what the problems are, and in later days most consumers may willingly extend their helping hand to the water works. These measures will help reduce the delinquencies of bills.

4.2 Metering

Metering is most effective for prevention of wasteful use of water, and at the same time is most fair for charging the consumer. There are cases where after metering consumption per month has reduced to a certain degree. This is an evidence that the consumers who were metered have become meter-conscious and stopped wasteful use. Metering is not only wastage-saving but revenue-producing as well.

Meter is a precision instrument and its accuracy declines with years of use. The wheel-type meter in particular has this tendency. The error due to wear sometimes reaches as much as 50% of the true value. As for meters in long use it must be checked on the spot. A portable check meter is designed for this purpose. It can be connected readily with the faucet, and closing all other faucets of the plumbing system and letting water run only through this check meter connected with faucet, the test of the installed meter can be made. When the error is introlerable, the meter must be replaced. The accurate meter is not only waste-saving but remarkably revenue-producing.

4.3 Map of Distribution System

The map of distribution lines is essential for operation of distribution, and indispensable also for maintenance of the system. Correct locations of valves and fire hydrants must be plotted on the map. Valves which are not frequently operated are likely to be covered with soil and lost from sight. If correct locations are indicated on the map, they will easily be detected even when the valve box is burried under ground. Sometimes a lost valve becomes vitally important for interupting water flow, for such as repairing a burst pipe or isolating a work block. If the map is correct, exact materials for repair can be prepared before digging out the defective pipe and others.

4.4 Improvement of Materials and Workmanship

Leakage depends largely on materials and workmanship. Nowadays new materials and pipe-joints are brought one after another in the market, and used by water works. Quality of these materials are gradually improved, and joints are becoming simpler and easier in handling. Even though, skill and experience in the use of such materials and fittings are important factors for perfect work.

CONTROL SYSTEM FOR WATER WORKS FACILITIES

Control systems vary in general with the time of construction, the role of facilities and other factors. From this standpoint, results of observation of the existing system, and concepts of future control system are described here for reference.

1. Existing Control Systems

The control systems are not much varied despite being constructed at different times, mainly because all the plants are treating the same Nile water and consequently types of the facilities are much similar. Two cases out of many plants, one an old plant and the other a rather new one, are briefly described below.

(1) Rod El Farag Plant

Indicating instruments are provided at the strategic points and well maintained. For example, flow meter and pressure gauge are equipped on delivery pipes of pumps, apparatuses for pH, conductivity, temperature and others in the laboratory.

Control is made almost all manually according to the information obtained by the above instruments. Only postchlorine is dose automatically according to residual chlorine. Safeguard for large machines, such as pumps, diesel engines, etc., is provided.

Regarding exchange of information within the plant, there is no special device, and necessary information is exchange orally or in writing. Water level of the elevated tank connected to the plant is informed by telephone.

(2) South Cairo Plant

It is provided with almost the same control system as the Rod El

Farag plant. The only difference is the information exchange system, namely the operating condition of pumps and filters are shown on the panel in the control room.

2. Concept of Future Control System

It is indicated in Chapter II. General Planning of Future Urban Water Supply that (1) the production will meet estimated demand, (2) operation at the normal rate will be restored, (3) networks will be strengthened until most of the elevated tanks are fed by not only nearby plant/s but also a key plant, the largest plant in the district, and (4) all reservoirs will have enough capacity to make up ordinary peak demand with the sored water.

From the above (1) and (4), it is safe to say that any artificial control of water quantity will not need to be made under the condition of ordinary demand. However it is prudent to arrange a measure for possible occurrence of excessive peak demand beyond anticipation.

By this consideration, it is advantageous to apply a central control system in the following manner: water levels of all elevated tanks and clear water wells in the treatment plants are informed to a key plant, and the key plant adjusts its delivery so as to maintain the desired water levels, in other words, the desired storage in the district, taking advantage of the above (3).

When this control system is applied, plants other than the key plant can almost always be operated at the normal rate, and it will ensure producing treated water with safe quality by avoiding excessive overloading.

This system however may impose some operational burdens as to quantity control on the key plant, though this plant is designed to be efficient in treating the present Nile water. The plant should be equipped with such simple control systems as an automatic filter washing system backed up with manual operating system and a central watching system of the major facilities, similar to that of the Roda treatment plant, in order to facilitate easy and quick operation.

APPENDIXES ON RAW WATER

APPENDIX R-1 METEOROLOGICAL DATA

APPENDIX R-1

HELWAN

Table R-1-1 Meteorological Data (1)

		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Period
Mean Air temperature	(C°)	13.2	14.8	17.8	21.2	24.2	27.8	27.6	27.9	26.2	23.8	19.6	14.8	1961-70
Mean Max. Air temp.	(C°)	18.8	20.5	23.4	28.4	32.4	34.8	35.2	34.9	32.4	30.0	25.3	20.4	1931-70
Mean Min. Air temp.	(C°)	8.4	9.3	11.3	14.1	17.8	19.7	21.4	21.6	20.2	18.4	14.8	10.5	1931-70
Mean Relative Humidity	(%)	54	48	41	36	36	39	50	51	52	51	57	54	1961-70
Mean Total Amount of rainfall	(mm)	3.4	3.7	2.1	0.9	3.0	Trace	0.0	0.0	Trace	1.1	3.1	6.0	1931-70
Max. Amount of rain in one day	(mm)	5.9	16.2	11.9	10.4	27.5	Trace	0.0	0.0	Trace	21.7	23.8	32.7	1931-70
Mean Evaporation Piche	(mm)	5.7	7.2	9.7	12.5	15.2	15.9	14.1	13.2	11.7	10.6	7.2	6.0	1931-70
Mean Surface Wind Speed	(Knots)	5.6	7.1	8.6	9.5	9.3	9.8	8.5	8.0	8.5	8.5	7.9	6.2	1961-70
Mean of day Actual Bright Sunshine	(Hours)	7.7	8.6	9.2	10.3	10.9	12.6	12.4	11.9	10.9	9.8	8.6	7.4	1931-60
Frequency of Wind Blowing by Directions(%)														
Calm		8.1	5.1	3.5	3.6	3.1	0.8	2.3	1.6	2.8	4.5	5.4	10.8	
Variable		0.7	1.5	0.2	1.0	0.7	0.8	0.1	0.2	0.4	0.8	1.0	1.7	
from 345° to 014°		5.8	12.4	13.2	13.4	21.5	25.9	34.0	38.4	41.1	22.1	22.5	7.1	
" 015 " 044		7.5	17.8	15.4	21.3	26.6	22.0	8.5	13.5	21.5	27.0	29.2	15.5	
" 045 " 074		6.0	10.2	7.7	11.3	11.3	7.5	1.4	1.6	5.0	8.1	7.7	8.5	
" 075 " 104		6.4	5.3	5.7	5.4	2.7	2.0	0.4	0.7	0.9	4.0	2.7	6.3	
" 105 " 134		12.5	5.8	4.6	2.7	0.6	0.5	0.1	0.3	0.1	1.5	3.1	9.2	
" 135 " 164		9.8	5.2	4.6	2.1	0.5	0.4	0.0	0.1	0.1	1.0	2.6	8.1	
" 165 " 194		9.1	7.2	6.7	3.3	1.0	0.5	0.1	0.1	0.2	1.4	2.7	9.0	
" 195 " 224		12.1	6.9	6.9	3.6	1.7	1.4	0.5	0.3	0.4	2.4	4.6	6.6	
" 225 " 254		5.0	3.3	4.5	2.7	1.1	0.4	0.4	0.1	0.3	1.0	1.0	3.6	
" 255 " 284		6.5	4.1	6.2	4.5	2.8	1.7	2.8	3.0	1.3	3.8	3.5	3.8	
" 285 " 314		4.5	7.0	8.8	9.5	10.8	8.4	9.7	10.6	6.5	6.8	4.7	4.5	
" 315° " 344°		6.0	8.2	11.8	15.6	15.6	27.7	39.7	29.5	19.4	15.6	9.3	5.3	
Frequency of Wind Blowing by Speed														
from 1 to 3 Knots		23.3	15.1	13.8	12.4	9.5	9.7	9.2	13.8	12.4	13.0	18.7	29.7	
" 4 " 6 "		33.1	28.4	23.4	19.0	21.1	20.5	26.7	25.5	22.1	23.6	24.9	28.0	
" 7 " 10 "		20.8	27.6	26.9	30.0	28.5	31.7	35.5	33.3	32.6	30.4	27.8	20.8	
" 11 " 16 "		13.9	22.2	27.4	28.9	31.3	29.2	25.1	25.3q	27.7	24.4	20.0	9.0	
" 17 " 21 "		0.8	1.5	4.2	5.3	6.0	6.5	1.2	0.5	2.0	3.6	2.9	1.6	
" 22 " 27 "		0.0	0.1	0.7	0.8	0.5	1.6	0.0	0.0	0.4	0.5	0.3	0.1	
" 28 " 33 "		0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
more than 34 "		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

Meteorological Data (2)

ALMAZA

		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Period
Mean Air temperature	(C°)	13.8	14.9	17.4	21.0	24.8	27.4	28.0	28.0	26.0	23.3	19.5	15.4	1945-70
Mean Max. Air temp.	(C°)	19.4	20.8	23.8	28.3	32.7	34.7	35.2	34.7	32.4	30.1	25.6	20.9	1941-70
Mean Min. Air temp.	(C°)	8.8	9.5	11.4	14.2	17.6	20.4	21.8	22.0	20.3	17.8	14.2	10.4	1941-70
Mean Relative Humidity	(%)	57	53	49	43	41	45	54	56	58	58	61	58	1945-70
Mean Total Amount of rainfall	(mm)	4.6	4.0	3.1	0.7	1.6	0.0	0.0	Trace	Trace	0.6	2.4	6.8	1941-70
Max. Amount of rain in one day	(mm)	8.7	10.7	10.9	2.8	9.7	Trace	Trace	0.1	0.2	4.0	15.4	44.0	1941-70
Mean Evaporation Piche	(mm)	5.7	7.0	8.9	10.6	12.8	12.9	11.1	9.9	9.1	8.5	6.5	5.6	1941-70
Mean Surface Wind Speed	(Knots)	6.9	7.2	7.6	7.8	7.4	6.6	5.6	5.6	6.2	6.5	6.0	6.6	1952-70
Mean of day Actual Bright Sunshine	(Hours)	7.4	8.0	8.7	9.7	10.7	11.5	11.5	11.1	10.4	9.5	8.5	7.4	1955-70
Frequency of Wind Blowing by Direction (%)														
Calm		0.7	1.3	0.5	0.4	0.2	0.1	0.1	0.2	0.2	0.4	0.9	1.0	
Variable		1.5	1.7	0.7	0.8	0.2	0.3	0.3	0.5	0.4	1.0	0.9	2.1	
from 345° to 014°		4.6	7.6	9.1	13.3	12.9	20.1	29.5	28.9	21.5	11.5	7.6	4.8	
" 015 " 044		7.6	12.3	15.3	20.7	28.6	27.8	23.5	25.9	36.0	28.3	18.9	9.9	
" 045 " 074		8.5	12.2	13.3	16.8	21.8	18.6	7.9	12.2	21.4	25.9	24.7	10.9	
" 075 " 104		4.8	5.2	4.8	4.5	6.6	3.3	1.2	1.8	3.4	5.5	7.1	5.1	
" 105 " 134		9.0	5.7	5.4	5.3	4.2	1.8	0.9	1.0	2.0	4.6	5.8	6.7	
" 135 " 164		7.6	4.9	3.4	2.8	1.8	1.0	0.4	0.5	1.0	2.8	3.8	5.1	
" 165 " 194		10.6	6.4	4.8	1.7	0.8	0.3	0.1	0.1	0.2	1.4	4.0	10.1	
" 195 " 224		17.5	11.8	8.0	3.6	1.7	1.7	0.2	0.2	0.4	2.6	7.1	17.9	
" 225 " 254		11.3	9.2	7.3	3.8	2.2	1.2	0.9	0.5	0.5	2.1	5.5	10.2	
" 255 " 284		9.5	9.7	11.0	7.5	4.4	2.4	4.3	2.4	0.9	2.9	4.5	7.8	
" 285 " 314		4.6	6.3	8.7	9.6	6.8	8.1	10.3	8.5	3.3	4.2	4.5	4.7	
" 315° " 344°		4.4	5.7	7.7	9.2	7.8	13.3	20.4	17.5	8.8	6.8	4.7	3.7	
Frequency of Wind Blowing by Speed	(%)													
from 1 to 3 Knots		26.1	23.5	18.7	18.7	18.3	23.4	29.6	35.4	26.8	26.1	31.3	28.5	
" 4 " 6 "		25.4	26.4	25.2	24.7	26.8	30.3	34.5	31.9	28.5	28.2	30.1	26.6	
" 7 " 10 "		26.7	27.6	29.0	30.3	31.4	28.7	27.4	25.7	31.1	30.6	25.7	26.0	
" 11 " 16 "		17.8	17.0	21.3	21.9	20.0	15.3	8.0	6.4	12.8	13.8	10.8	14.9	
" 17 " 21 "		2.3	3.0	3.9	3.2	2.7	1.9	0.4	0.4	0.6	0.8	1.0	2.4	
" 22 " 27 "		0.9	1.1	1.1	0.7	0.5	0.3	0.0	0.0	0.0	0.1	0.2	0.5	
" 28 " 33 "		0.1	0.1	0.2	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	
more than 34 "		0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

Meteorological Data (3)

CAIRO A.P.

		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Period
Mean Air temperature	(C°)	13.7	14.9	17.5	20.9	24.6	27.2	27.8	27.7	25.8	23.4	19.3	15.3	1947-70
Mean Max. Air temp.	(C°)	19.1	20.6	23.9	28.3	32.2	34.8	35.0	34.6	32.4	29.8	25.2	20.7	1947-70
Mean Min. Air temp.	(C°)	8.8	9.4	11.5	14.1	17.4	20.2	21.5	21.8	20.0	17.8	13.9	10.4	1947-70
Mean Relative Humity	(%)	58	54	49	45	43	46	54	57	58	57	61	60	1947-70
Mean Total Amount of rainfall	(mm)	5.2	3.9	2.4	0.9	0.7	0.2	0.0	Trace	Trace	1.2	3.2	6.7	1947-70
Max. Amount of rain in one day	(mm)	9.6	10.4	10.0	3.8	6.0	3.6	0.0	Trace	0.1	13.8	18.5	50.0	1947-70
Mean Evaporation Piche	(mm)	7.6	9.0	11.5	14.3	16.4	17.2	14.2	13.0	12.2	11.1	8.2	7.5	1947-70
Mean Surface Wind Speed	(Knots)	8.0	8.1	8.7	8.7	8.8	8.1	6.7	6.4	6.6	7.1	6.5	7.6	1947-70
Mean of day Actual Bright Sunshine	(Hours)	-	-	-	-	-	-	-	-	-	-	-	-	-
Frequency of Wind Blowing by Direction (%)														
Calm		6.2	5.8	4.6	4.3	3.5	5.1	6.0	6.8	8.8	6.7	10.2	6.7	
Variable		1.6	1.3	1.1	1.3	1.2	1.0	0.8	1.2	1.3	2.1	1.1	3.1	
from 345° to 014°		2.5	4.8	5.8	8.8	11.6	17.0	20.5	20.6	17.7	8.9	7.2	2.6	
" 015 " 044		5.4	8.7	9.7	15.7	20.8	22.3	15.7	18.8	26.8	21.9	15.8	6.8	
" 045 " 074		7.8	10.0	12.6	15.4	21.1	13.0	5.6	6.7	14.0	21.4	16.1	9.3	
" 075 " 104		5.1	7.2	7.0	8.1	8.7	5.1	1.6	2.1	5.1	10.3	9.0	7.6	
" 105 " 134		5.2	6.2	5.1	4.9	4.2	2.4	0.7	0.9	2.0	3.8	5.3	6.1	
" 135 " 164		5.5	4.8	3.3	2.9	1.6	0.9	0.2	0.2	0.5	1.0	2.1	4.2	
" 165 " 194		11.5	6.6	4.4	2.2	0.9	0.2	0.1	0.1	0.2	1.3	3.7	9.6	
" 195 " 224		19.2	11.4	7.8	5.3	1.2	0.9	0.2	0.3	0.3	2.5	6.9	18.4	
" 225 " 254		11.5	10.4	7.9	4.6	2.2	1.2	0.8	0.8	0.6	3.0	6.0	9.5	
" 255 " 284		7.8	7.3	9.1	5.6	3.7	2.9	4.2	2.8	1.7	2.5	4.2	6.4	
" 285 " 314		5.7	8.4	11.4	9.1	6.6	8.9	13.6	12.3	5.4	5.2	5.2	5.2	
" 315° " 344°		5.0	7.1	10.2	13.6	12.7	19.1	30.0	26.4	15.6	9.4	7.2	4.5	
Frequency of Wind Blowinb by Speed (%)														
from 1 to 3 Knots		16.7	19.3	16.2	14.3	14.0	17.2	24.3	24.7	22.0	20.0	19.1	21.4	
" 4 " 6 "		21.9	21.0	20.7	19.9	20.4	20.5	24.9	24.6	22.5	22.8	24.3	22.4	
" 7 " 10 "		25.7	25.0	25.9	28.1	30.0	28.7	27.8	29.4	29.3	29.5	27.4	24.6	
" 11 " 16 "		21.9	20.7	23.6	26.6	26.7	24.5	16.3	14.0	16.4	18.9	16.9	18.4	
" 17 " 21 "		5.6	5.3	6.3	5.3	4.4	3.7	0.7	0.4	0.9	1.9	1.6	4.6	
" 22 " 27 "		1.6	2.1	2.1	1.4	0.9	0.3	0.0	0.0	0.1	0.2	0.5	1.6	
" 28 " 33 "		0.3	0.7	0.3	0.1	0.1	0.0	0.0	0.1	0.0	0.0	0.0	0.3	
more than 34 "		0.1	0.1	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

Direction in Degrees East of North on the Scale (000° to 360°)

One knot = 185 kilometers per hour

Tab. R-1-2. CALCULATION OF EVAPO-TRANSPIRATION

			JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
(1)	Mean Temperature Ta	(°C)	13.2	14.8	17.8	21.2	24.2	27.8	27.6	27.9	26.2	23.8	19.6	14.8
(2)	Relative Humidity	(%)	54	48	41	36	36	39	50	51	52	51	57	54
(3)	Actual Sunshine n	(hr)	7.7	8.6	9.2	10.3	10.9	12.6	12.4	11.9	10.9	9.8	8.6	7.4
(4)	Possible Sunshine N	(hr)	10.8	10.4	12.5	13.0	14.2	14.0	14.4	13.7	1.24	11.8	10.7	10.6
(5)	Sunshine Rate (3)÷(4)	(%)	71.3	82.7	73.6	79.2	76.8	90.0	86.1	86.9	87.9	83.1	80.4	69.8
(6)	Wind Speed	(knots)	5.6	7.2	8.6	9.5	9.3	9.8	8.5	8.0	8.5	8.5	7.9	6.2
(7)	Wihd Speed (6)x27.62 U_2	(Mile/day)	154.7	198.9	237.5	262.4	256.9	270.7	234.8	221.0	234.8	234.8	218.2	171.2
(8)	Radiation Rate Ra	(mm)	8.5	10.5	12.7	14.8	16.0	16.5	16.2	15.3	13.5	11.3	9.1	7.9
(9)	Reflection Coefficient r	(%)	25	25	25	25	25	25	25	25	25	25	25	25
(10)	1-r		0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
(11)	0.18+0.55x(5)		0.572	0.635	0.585	0.616	0.602	0.675	0.654	0.658	0.663	0.637	0.622	0.564
(12)	(8)x(10)x(11)		3.65	5.00	5.57	6.84	7.22	8.35	7.95	7.55	6.71	5.40	4.25	3.34
(13)	Saturated Vapour Pressure ℓa		12.0	13.0	16.0	20.0	24.0	28.0	28.0	28.0	26.0	24.0	18.0	13.0
(14)	Actual Vapour Pressure ℓd ℓax(2)		6.5	6.2	6.6	7.2	8.6	10.9	14.0	14.3	13.5	12.2	10.3	7.0
(15)	$\sqrt{\ell d}$		2.55	2.49	2.57	2.68	2.93	3.30	3.74	3.78	3.67	3.49	3.21	2.65
(16)	σTa^4		13.45	13.79	14.43	15.13	15.76	16.52	16.47	16.45	16.17	15.67	14.80	13.79
(17)	$0.56-0.092\sqrt{\ell d}$		0.325	0.331	0.324	0.313	0.290	0.256	0.216	0.212	0.222	0.239	0.265	0.316
(18)	0.10+0.90x(5)n/N		0.742	0.844	0.762	0.813	0.791	0.910	0.875	0.882	0.891	0.848	0.824	0.728
(19)	(16)x(17)x(18)		3.24	3.85	3.56	3.85	3.62	3.85	3.11	3.08	3.20	3.18	3.23	3.17
(20)	H (12)-(19)		0.41	1.15	2.01	2.99	3.60	4.50	4.84	4.47	3.51	2.22	1.02	0.17
(21)	0.35(ℓa-ℓd)		1.93	2.38	3.29	4.48	5.39	5.99	4.90	4.80	4.38	4.13	2.70	2.10
(22)	$1+0.0098U_2$		2.52	2.95	3.33	3.57	3.52	3.65	3.30	3.17	3.30	3.30	3.14	2.68
(23)	(21)x(22) Ea		4.86	7.02	10.96	15.99	18.97	21.86	16.17	15.22	14.45	13.63	8.48	5.63
(24)	Δ		0.42	0.45	0.54	0.62	0.76	0.90	0.90	0.90	0.82	0.74	0.60	0.45
(25)	ΔH		0.17	0.52	1.09	1.85	2.74	4.05	4.36	4.02	2.88	1.64	0.61	0.08
(26)	0.27Ea		1.31	1.90	2.96	4.32	5.12	5.90	4.37	4.11	3.90	3.68	2.29	1.52
(27)	Δ+0.27		0.69	0.72	0.81	0.89	1.03	1.17	1.17	1.17	1.09	1.01	0.87	0.72
(28)	Et=(ΔH+0.27Ea)/(Δ+0.27)		2.14	3.36	5.00	6.93	7.63	8.50	7.46	6.95	6.22	5.27	3.33	2.22

APPENDIX R-2

REFERENCE DATA FOR WATER HAMMERING

Calculation in case Head is 100 m

(1) North-East Cairo Intake Pumping Station

a) Basis of Calculation

i) Specifications of Pumps

Item/Pump	Pump A (Proposed)	Pump B (Existing)
Type:	Vertical double suction volute pump	Vertical double suction volute pump
Bore: (suction x discharge)mm	500 x 300	300 x 250
Total pumping head: (H) m	100	100
Pumping capacity: (each pump) cu.m/min	31.0	12.0
Rotation speed: r.p.m	1,450	1,450
Pumping efficiency: %	80	77
Parallel operation: unit	3	3

ii) Specifications of Motor

		Motor for Pump A	Motor for Pump B
Output	kw	730	285
$\overline{GD}^2$ (per unit)	Kg-sq.m	153.6	44.3

iii) Specification of Pipeline

Type of pipe:	Ductile cast iron pipe
Diameter:	1,350 mm
Gauge:	16.5 mm
Young's modulus	$E = 1.6 \times 10^{10}$ kg/sq.m
Actual head:	89 m
Total length	L = 8,900 m

b) Numerical Calculations

i) Flow velocity in pipeline (V)

V = Q/A

$$Q = \frac{31 \times 3 + 12 \times 3}{60} = 2.15 \text{ cu.m/sec}$$

$$A = \frac{\pi D^2}{4} = \frac{3.14 \times 1.35^2}{4} = 1,431 \text{ sq.m}$$

$$V = 2.15/1,431 = 1.502 \text{ m/sec}$$

ii) Inertia Moment of Rotation (I)

$$I = \frac{\overline{GD^2}}{4g} \times S$$

where $\overline{GD^2_A}$ = 153.6 kg-sq.m

$\overline{GD^2_B}$ = 44.3 kg-sq.m

g = 9.8 m/sec

S_A = 3 units

S_B = 3 units

$$I_A = \frac{153.6}{4 \times 9.8} \times 3 = 11.755 \text{ kg-sq.m/m/sec}^2$$

$$I_B = \frac{44.3}{4 \times 9.8} \times 3 = 3.387 \text{ kg-sq.m/m/sec}^2$$

iii) Propagation Velocity of Pressure Wave (a)

$$a = \frac{1,425}{\sqrt{1 + \frac{k}{E} \times \frac{D}{t}}}$$

where k = Bulk modulus of water 2.07×10^8 kg/sq.m

$E = 1.6 \times 10^{10}$ kg/sq.m

$D = 1.35$ m

$t = 0.0165$ m

$$a = \frac{1,425}{\sqrt{1 + \frac{2.07 \times 10^8}{1.6 \times 10^{10}} \times \frac{1.35}{0.0165}}} = 993 \text{ m/sec}$$

d) Period μ

$$\mu = \frac{2\,L}{a}$$

$L = 8,900$ m

$$\mu = \frac{2 \times 8,900}{993} = 17.92 \text{ sec}$$

e) Pipe Constant

$$\rho = \frac{a\,v}{2gH}$$

$$= \frac{993 \times 1.502}{2 \times 9.8 \times 100} = 0.761$$

f) Damping Coefficient (K)

$$K = \frac{91,200\ H\ Q}{I \cdot N^2 \cdot \zeta}$$

N_A = 1,450 r.p.m.

N_B = 1,450 r.p.m.

ζ_A = 80%

ζ_B = 77%

$Q_A = 31 \times 3 \div 60 = 1.55$ cu.m/sec

$Q_B = 12 \times 3 \div 60 = 0.60$ cu.m/sec

$$K_A = \frac{91,200 \times 100 \times 1.55}{11,755 \times 1,450^2 \times 0.8} = 0.715$$

$$K_B = \frac{91,200 \times 100 \times 0.6}{3,387 \times 1,450^2 \times 0.77} = 0.988$$

3) Examination by Computer

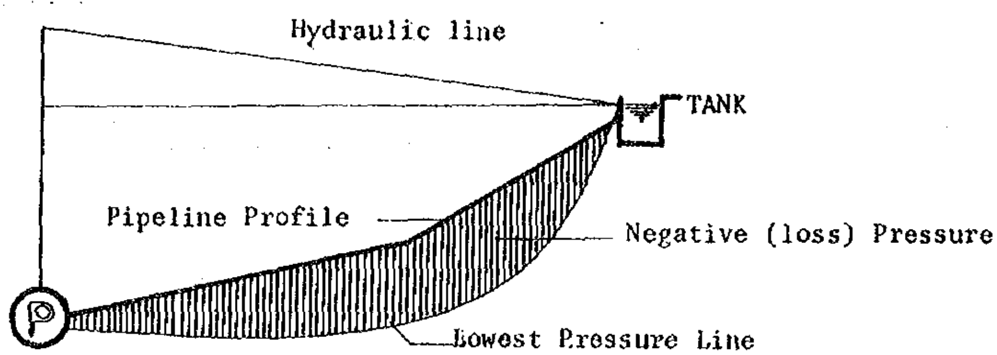
Transient phenomena of water hammering have been examined by using of electronic computer which employed graphical solution method based upon aforementioned equations. The results of examination are summarized below.

a) In case no anti-hammering is provided

In the case that 3 of pumps A and 3 of pumps B are being operated parallelly and supply water is joining into 1,350ϕmm distribution main. When running pumps are suddenly stopped by electricity failure, the pressure in pipeline would be rised or droped in the maximum range. The examination of water hammering is made for the case that the 6 pumps are stopped simultaneously.

When all running pumps are suddenly stopped, the pressure in pipeline drops suddenly as the lowest pressure curve shown in Figure R-2-1. When the lowest pressure line became lower than the pipeline profile, the inner pipeline would have negative (loss) pressure, as is illustrated below.

Fig. R-2-1



The biggest problem in case no anti-water hammering device is provided, the negative pressure in pipeline and pumps will be increased substantially when water pressure is dropped by accident.

When the negative pressure exceeds 10 m, vacuum space occurs in the pipeline, otherwise known as "water column separation." When such phenomenon (water column separation) occurs, the water in pipeline reflexes and rejoins, and will cause abnormal high pressure in the pipeline. Pipeline is sometimes broken by the occurrence of water column separation; hence, it is necessary to eliminate such dangerous phenomenon by providing an adequate anti-water hammering.

b) In case Oneway Surge Tank is provided

As the proposed plan for the pumping facilities requires a large capacity of water storage with high pumping head, providing a conventional surge tank or airvessel might create various engineering problems. Therefore, it is proposed that oneway surge tanks of 20 m above the ground will be installed at three places to prevent possible occurrence of water column separation. The calculations by computer are shown in Fig. R-2-2. In case surge tanks are provided, it may be possible to restrain negative pressure in the pipeline from 5 to 6 m as well as pressure kept lower than 13 kg/cm^2. These proposed surge tanks are numbered from the side of pumping station, as follows:

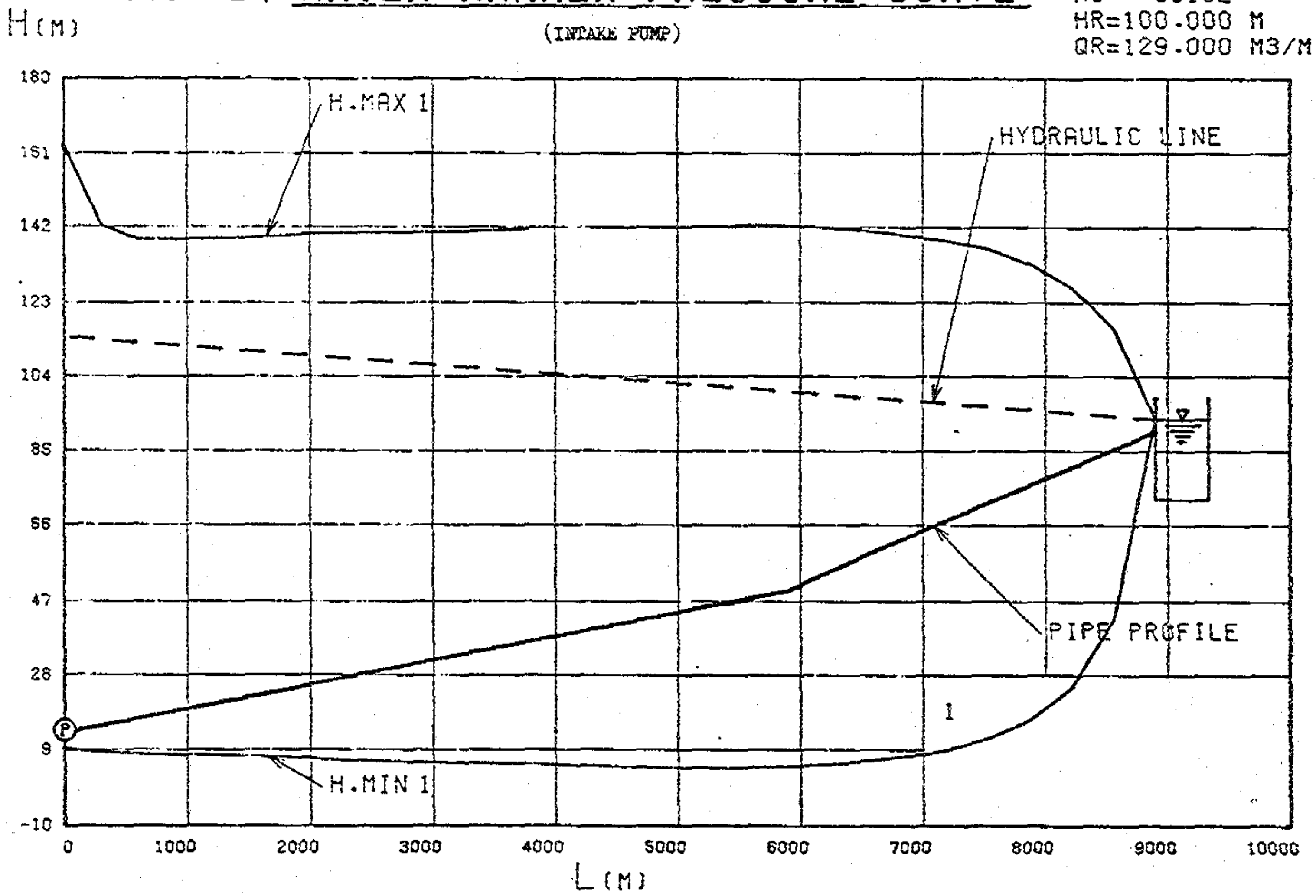
Specification of Surge Tanks

Surge Tank	No. 1	No. 2	No. 3
Distance from pump: (m)	2,300	5,000	7,300
Height from G.L.: (m)	20 (±)	20 (±)	20 (±)
Diameter: (m)	3.5	2.8	5.0
Storage capacity: (cu.m)	35	20	100
Connected pipe: (mm)	700	600	700

FIG R-2-1 WATER HAMMER PRESSURE CURVE

(INTAKE PUMP)

NO. 80102
HR=100.000 M
QR=129.000 M3/M

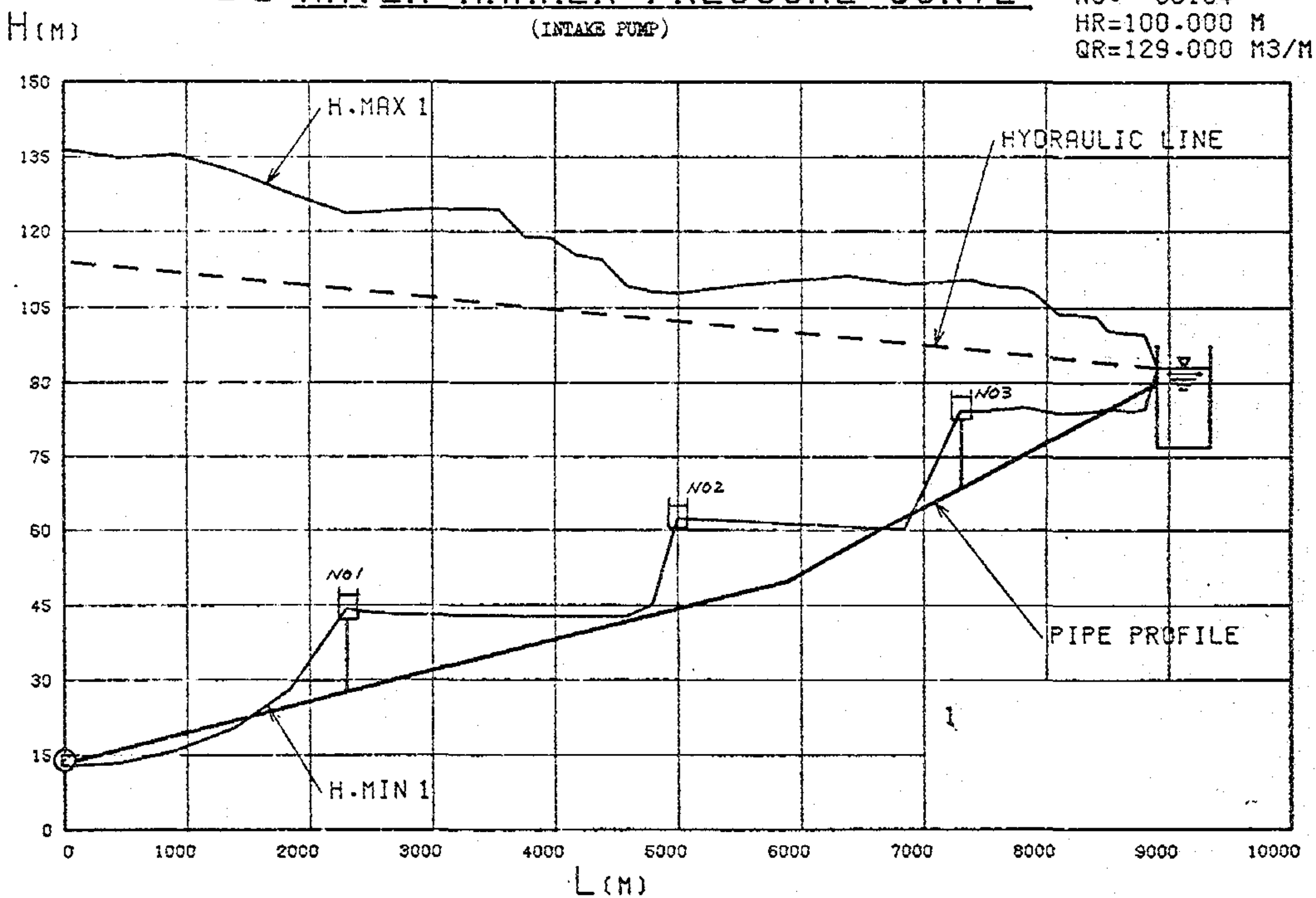


SCALE H 13.6M/10MM (7.4MM/ 10M) 5M
L 434.8M/10MM (2.3MM/100M)

FIG R-2-2 WATER HAMMER PRESSURE CURVE

(INTAKE PUMP)

NO. 80104
HR=100.000 M
QR=129.000 M3/M



SCALE H 10.7M/10MM (9.3MM/ 10M) 5M
L 434.8M/10MM (2.3MM/100M)

c) Results of Examination

Based on the results of examination by computer, it may be possible that the water column separation in pipeline can be prevented thus the maximum pressure will be restrained to lower than 13 kg/cm^2 by providing oneway surge tanks. Although the examination by computer has employed the standard type check valve to be provided at the outlet of pump, it will be better to provide slow closing type check valve to prevent the rising of water pressure in pipeline and maintain it as low as possible.

(2) Heliopolis Booster Pumping Station

1) Basis of Calculation

a) Specification of Pumps

Type:	Horizontal double suction volute pump
Bore:	500mm (suction) x 300mm (discharge)
Total pumping head:	67 m
Pumping capacity:	31.0 cu.m/min
Rotation speed:	1,450 r.p.m.
Pumping efficiency:	82%
Parallel operation units:	3

b) Specification of Motor

Output:	480 kw
$\overline{GD^2}$	87.2 kg-sq.m

c) Specification of Pipeline

Type of pipe:	Dactile cast iron pipe
Diameter:	1,200 mm
Gage:	15 mm
Young's modulus	$E = 1.6 \times 10^{10}$ kg/sq.m
Actual head:	54 m
Total length:	4,630 m

2) Numerical Calculations

a) Flow Velocity in Pipeline (V)

$$V = Q/A$$

$$Q = \frac{31 \times 3}{60} = 1.55 \text{ cu.m/sec}$$

$$A = \frac{\pi D^2}{4} = \frac{3.14 \times 1.2^2}{4} = 1.13 \text{ sq.m}$$

$$V = 1.55/1.13 = 1.371 \text{ m/sec}$$

b) Inertia Moment of Rotation (I)

$$I = \frac{\overline{GD^2}}{4g} \times S$$

$\overline{GD^2}$ = 87.2 kg.sq.m

g = 9.8 m/sec

S = 3 unit

$$I = \frac{87.2}{4 \times 9.8} \times 3 = 6.67 \text{ kg.sq.m/m/sec}^2$$

c) Propagation Velocity of Pressure Wave (a)

$$a = \frac{1,425}{\sqrt{1 + \frac{R}{E} \times \frac{D}{t}}}$$

where R = Bulk modulus of water 2.07×10^8 kg/sq.m

E = 1.6×10^{10} kg/sq.m

D = 1.2 m

t = 0.015 m

$$a = \frac{1,425}{\sqrt{1 + \frac{2.07 \times 10^8}{1.6 \times 10^{10}} \times \frac{1.2}{0.015}}} \doteq 999 \text{ m/sec}$$

d) Period (μ)

$$\mu = \frac{2L}{a}$$

L = 4,630 m

$$\mu = \frac{2 \times 4,630}{999} = 9.27 \text{ sec}$$

e) Pipe Constant

$$\rho = \frac{a\nu}{2gH}$$

$$= \frac{999 \times 1,371}{2 \times 9.8 \times 67} = 1,043$$

f) Damping Coefficient

$$K = \frac{91,200 \ H \ Q}{I \cdot N^2 \cdot \zeta}$$

$N = 1,450$ r.p.m.

$\zeta = 82\%$,

$$K = \frac{91,200 \times 67 \times 1.55}{6.67 \times 1,450^2 \times 0.82} = 0.823$$

3) Examination by Computer

a) In case no anti-water hammering device is provided

Employing the above-mentioned equations and figures, examinations by computer are made in case 3 parallel running pumps are stopped suddenly due to electricity failure. As is seen in Figure R-2-3, negative pressure in pipeline and pumps rises substantially and possible occurrence of water column separation is also foreseeable. Therefore, it is deemed necessary to provide an adequate device in the pipeline system to eliminate possible occurrence of such water column separation.

b) In case Oneway Surge Tank is provided

In order to prevent pipeline from water column separation, oneway surge tanks should be installed at 3 places for the water supply main. The results of examination made by computer are illustrated in Figure R-2-4. This method will prevent pipeline from water column separation and gradual rise of water pressure to a certain extent.

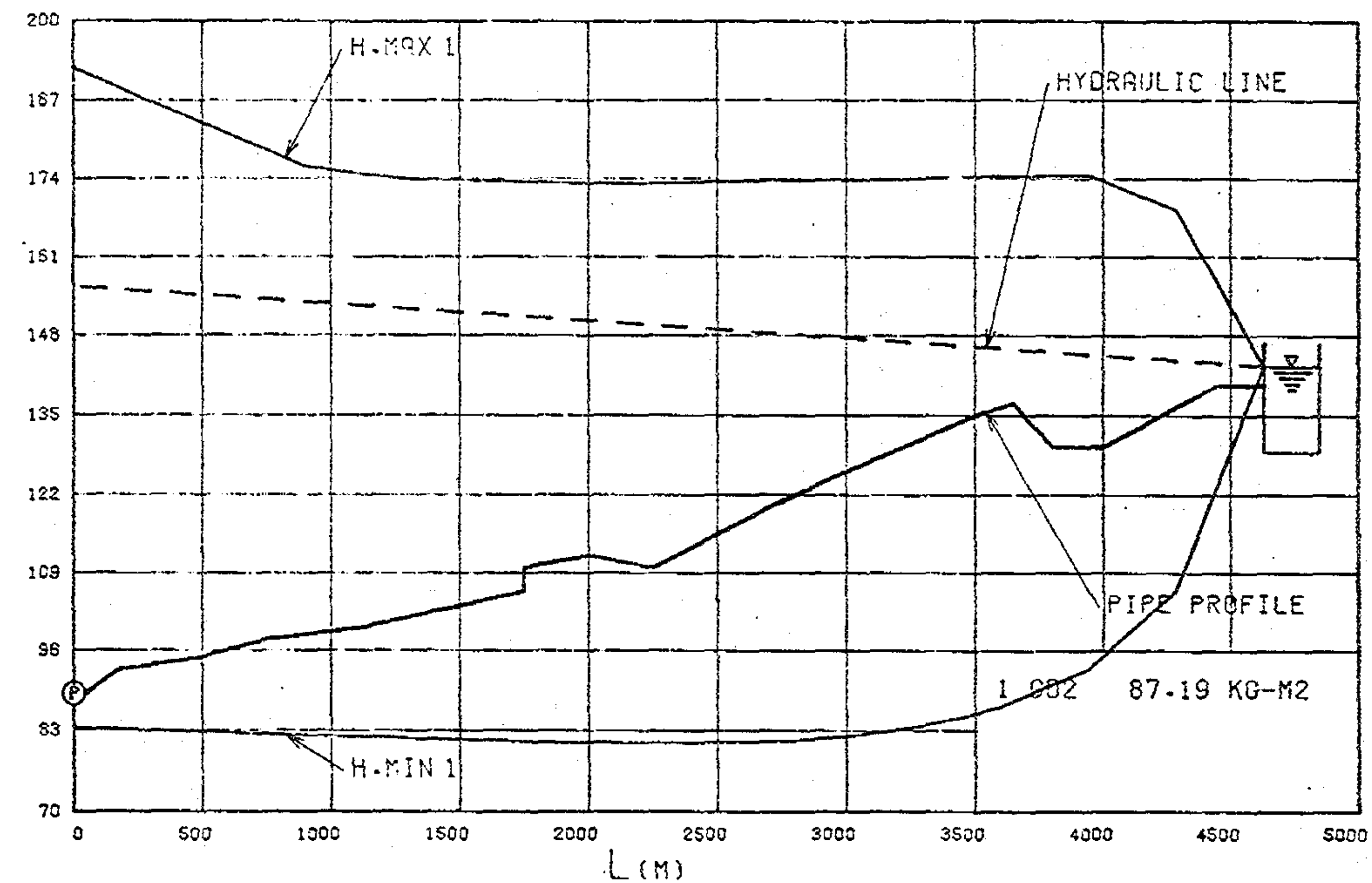
The following is the specification of proposed surge tanks:

Surge Tanks	No. 1	No. 2	No. 3
Distance from pump: (m)	1,000	2,500	3,600
Height from G.L.: (m)	10 (±)	20 (±)	3(±)
Diameter: (m)	2.4	4.0	2.8
Capacity: (cu.m,)	12	50	20
Connected pipeline: (mm)	600	600	600

FIG R-2-3 WATER HAMMER PRESSURE CURVE

(BOOSTER PUMP)

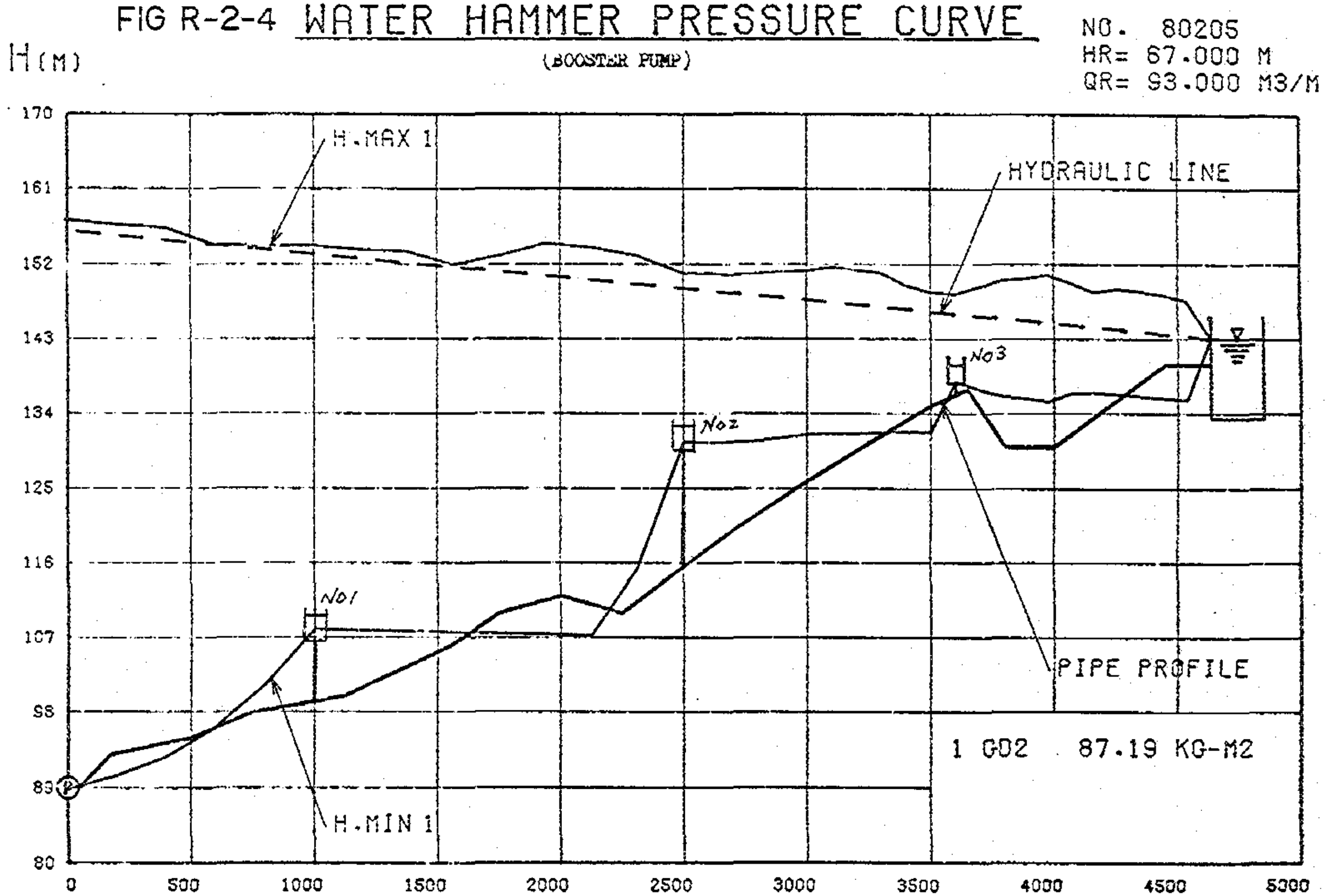
NO. 80202
HR= 67.000 M
QR= 93.000 M3/M



SCALE H 9.3M/10MM (10.8MM/ 10M) 5M
L 217.4M/10MM (4.6MM/100M)

FIG R-2-4 WATER HAMMER PRESSURE CURVE

(BOOSTER PUMP)



SCALE H 6.4M/10MM (15.6MM/ 10M) SM
L 217.4M/10MM (4.6MM/100M)

c) Result of examination

By providing surge tanks as mentioned in para. b) above, the negative pressure in pipeline and the maximum water pressure are restrained within the range of 5 to 6 m and 7 kg/cm^2 , respectively. If the maximum water pressure is kept at 7 kg/cm^2 as much as that of pumping head, the rise of water pressure in pipeline by water hammering will be hardly expected.

For the final planning of the pipeline, it is recommendable that the maximum design water pressure be determined at about 9 kg/cm^2 including some allowance to cope with the possible water hammering impacts. It is also desirable that slow-closing check valve will be provided at the outlet of pump to avoid possible accident.

Calculations in case Head is 150 m

(1) Basis of Calculation

a) Specification of Pumps

Type:	500 x 300 vertical volute pump
Total pumping head:	150 m
Pumping capacity:	31 cu.m/min each
Rotation speed:	980 r.p.m.
Pumping efficiency:	80%
Number of pumps:	3 units with parallel operation plus 1 unit as spare

b) Specification of Motor

Type:	
Output:	1,100 KW
$\overline{GD}^2$:	600 kg.sq.m/unit

c) Specification of Pipeline

Type of pipe:	Ductile cast iron pipe
Diameter:	1,200 mm
Gage:	17 mm
Young's modulus:	$E = 1.6 \times 10^{10}$ kg/sq.m
Actual head:	125.5 m

(2) Numerical Calculations

a) Flow velocity in pipeline (V)

$V = Q/A$

$$Q = \frac{31 \times 3}{60} = 1.55 \text{ cu.m/sec}$$

$$A = \frac{\pi D^2}{4} = \frac{3.14 \times 1.2^2}{4} = 1.13 \text{ sq.m}$$

$$V = 1.55/1.13 = 1.371 \text{ m/sec}$$

b) Inertia Momont of Rotation (I)

$$I = \frac{\overline{GD}^2}{4g} \times S$$

$\overline{GD}^2$ = 600 kg.sq.m

g = 9.8 m/sec^2

S = 3 units

$$I = \frac{600}{4 \times 9.8} \times 3 = 45.92 \text{ kg.sq.m/m/sec}^2$$

c) Propagation Velocity of Pressure Water (a)

$$a = \frac{1{,}425}{\sqrt{1 + \frac{R}{E} \times \frac{D}{t}}}$$

where R = Bulk modulus of water
2.07 x 10^8 kg/sq.m

E = 1.6 x 10^{10} kg/sq.m

D = 1.2 m

t = 0.017 m

$$a = \frac{1{,}425}{\sqrt{1 + \frac{2.07 \times 10^8}{1.6 \times 10^{10}} \times \frac{1.2}{0.017}}}$$

= 1,030 m/sec

d) Period (μ)

$$\mu = \frac{2L}{a} \qquad L = 12{,}600 \text{ m}$$

$$= \frac{2 \times 12{,}600}{1{,}030} = 24{,}461 \text{ sec}$$

e) Pipe Constant (ρ)

$$\rho = \frac{av}{2gH}$$

$$= \frac{1{,}030 \times 1{,}371}{2 \times 9.8 \times 150} = 0.48$$

f) Damping Coefficient (K)

$$K = \frac{91{,}200\ H \cdot Q}{I \cdot N^2 \cdot \zeta}$$

ζ = 0.8

N = 980

$$K = \frac{91{,}200 \times 150 \times 1.55}{45.92 \times 980^2 \times 0.8} = 0.601$$

(3) Examination by Computer

Examination by using computer has been made to transient phenomena of water hammering by employing graphical solution method based on the aforementioned equations and figures. The results of verification are summarized below.

a) In case no anti-water hammering device is provided

When 3 running pumps are simultaneously stopped due to electricity failure, a very high negative pressure occurs in pipeline and pumps and breaks the pipeline by the occurrence of water column separation. So, it is required to provide anti-hammering device to prevent pipeline from such water hammering.

b) In case oneway surge tank is provided

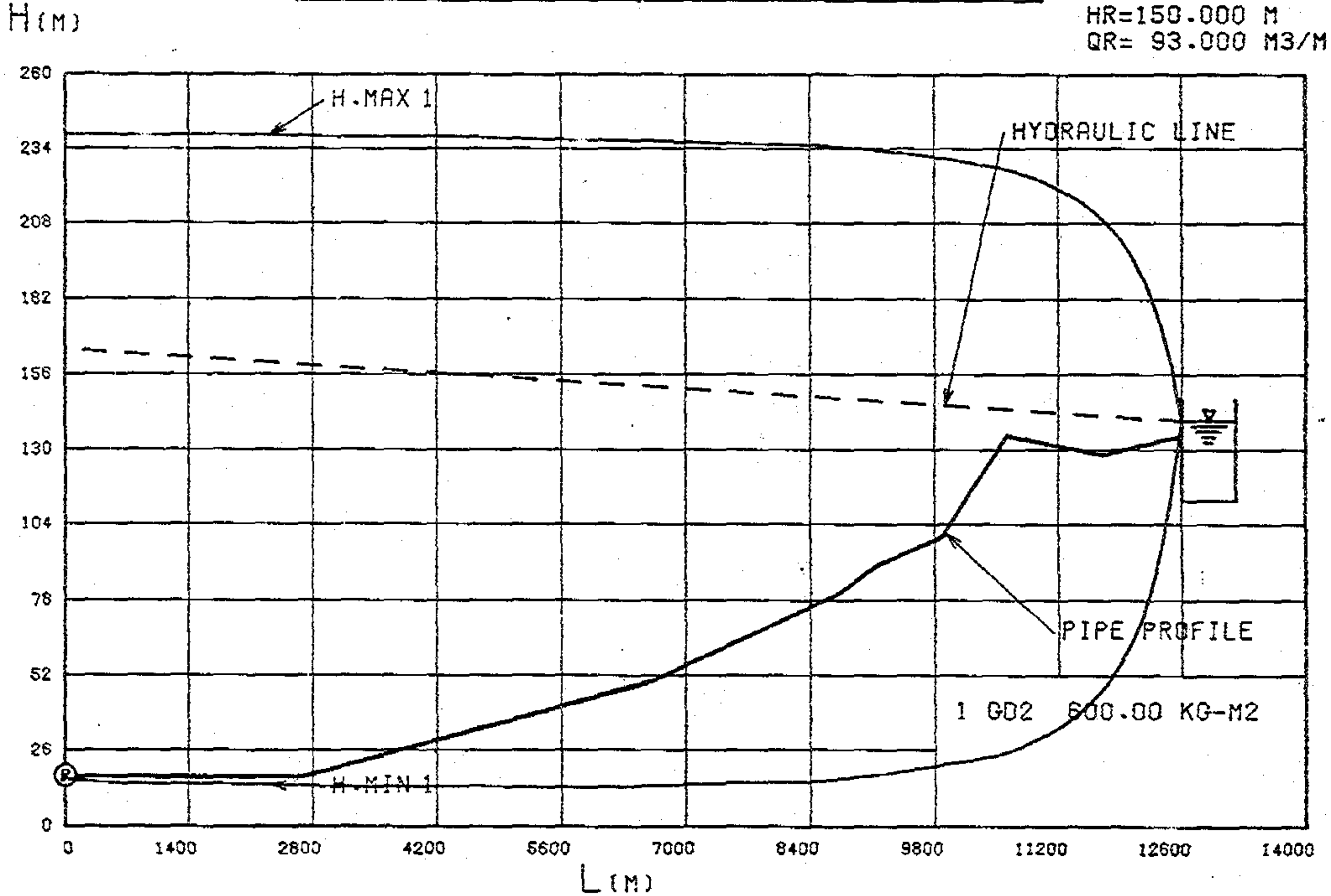
Various examinations have been made in case oneway surge tank is provided for the water distribution system. As a result, 4 oneway surge tanks are proposed to be provided to prevent occurrence of water column separation in pipeline. Two tanks will be installed at about 20 m above the ground, and the remaining 2 will be installed at the ground level. The examination is illustrated in Figure R-2-6. From this, a negative pressure can be restrained within the range of 5 to 6 m.

Specification of Surge Tanks

Surge Tank	No. 1	No. 2	No. 3	No. 4
Distance from pump: (m)	6,300	8,400	10,100	10,500
Height above G.L.: (m)	20	20	3	0
Diameter: (m)	2.5	2.0	1.0	13.0
Capacity: (cu.m)	15	10	1	350
Connected pipeline: (mm)	600	600	300	600

FIG R-2-5 WATER HAMMER PRESSURE CURVE

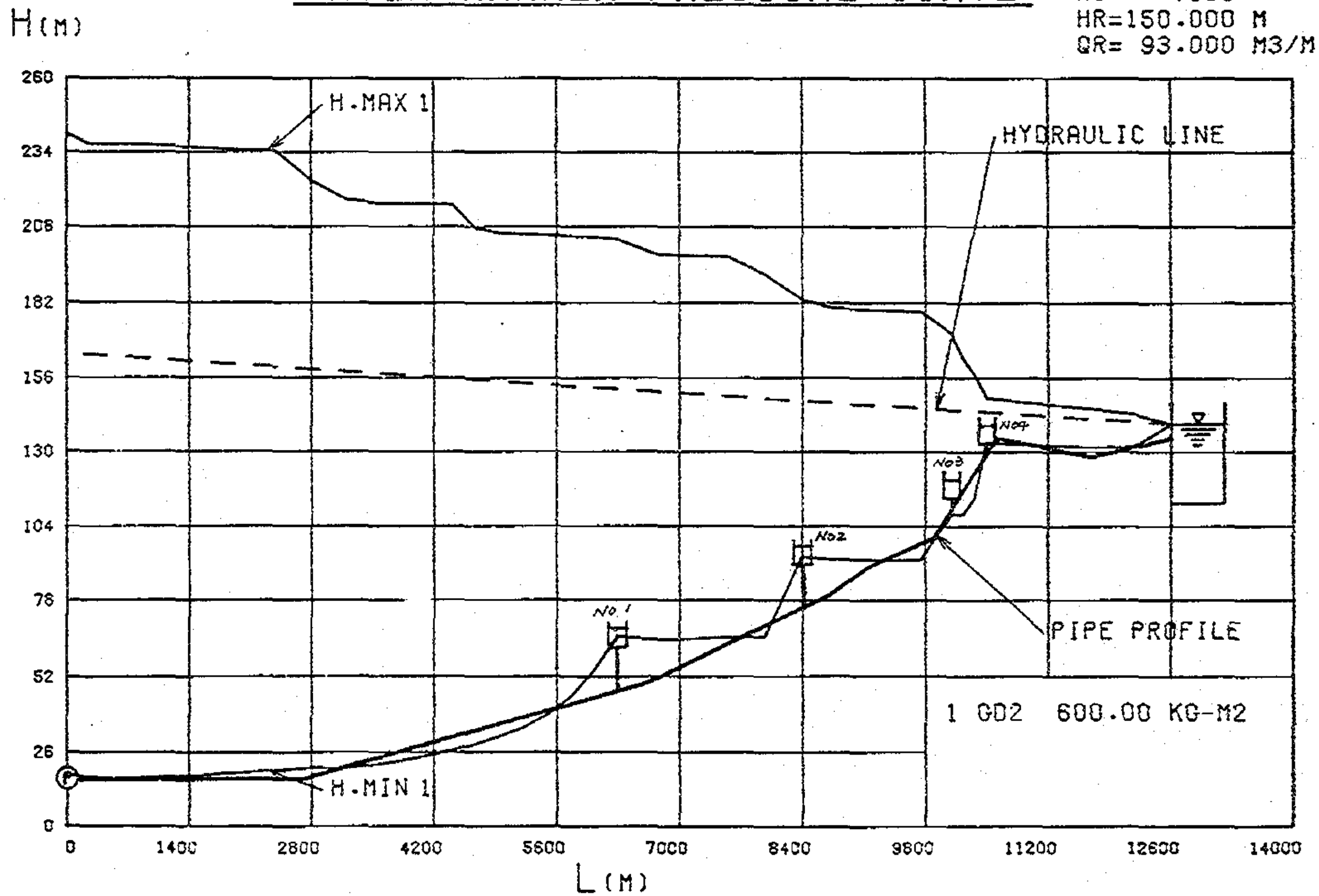
NO. 3201
HR=150.000 M
QR= 93.000 M3/M



SCALE H 18.6M/10MM (5.4MM/ 10M) 5M
L 608.7M/10MM (1.6MM/100M)

FIG R-2-6 WATER HAMMER PRESSURE CURVE

NO. 7101
HR=150.000 M
QR= 93.000 M3/M



SCALE H 18.6M/10MM (5.4MM/ 10M) 5M
L 608.7M/10MM (1.6MM/100M)

The maximum pressure exceeds 20 kg/cm^2 at the outlet of pump which indicate the case with check valve. In this respect, an examination has been made in case of slow-closing check valve in order to restrain the rise of pressure in the pipeline.

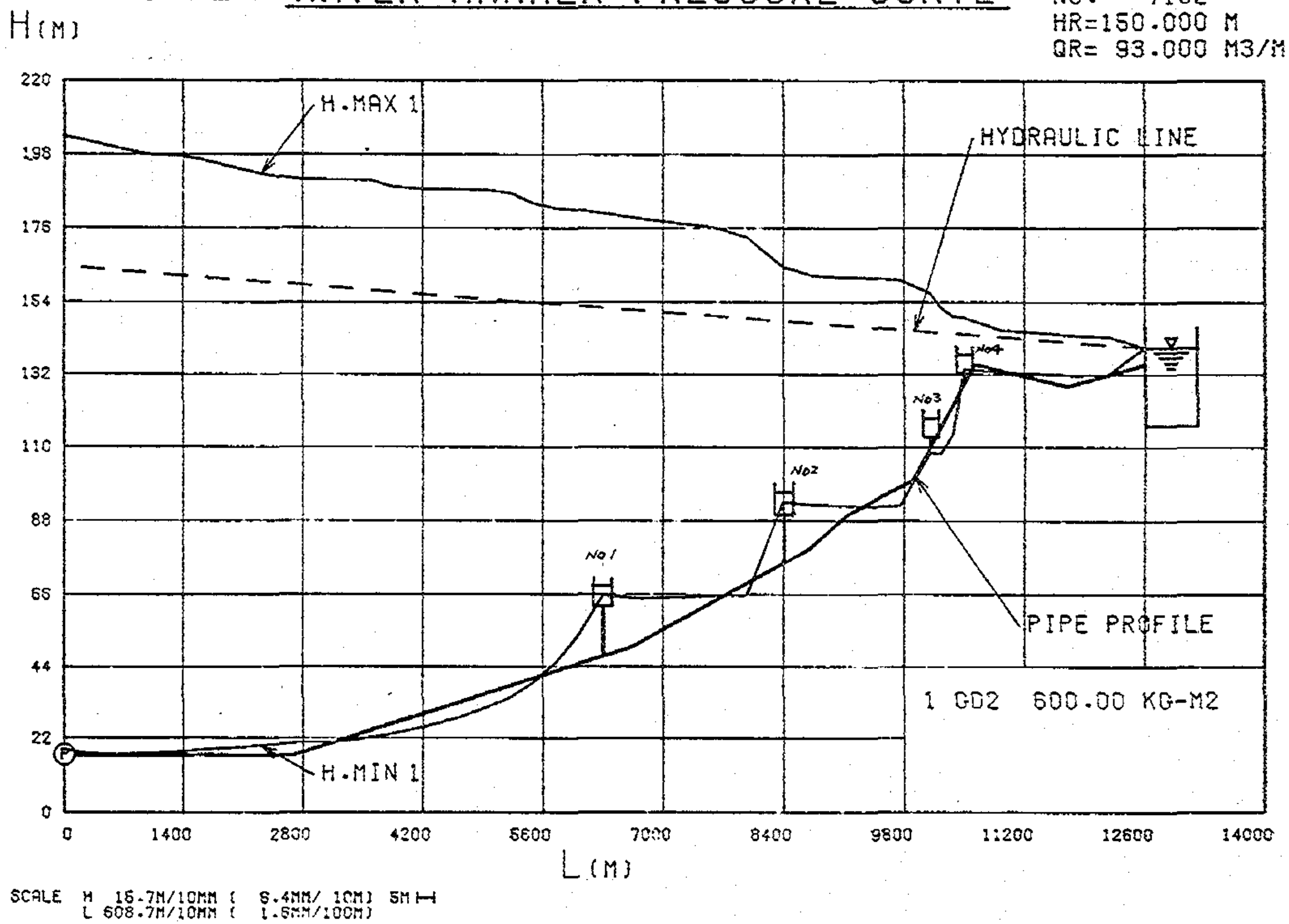
c) In case slow-closing check valve is provided

Examination has also been made by employing slow-closing check valve in which closing time may be controlled by oil pressure dash-pot. In this case, pressure rise at pump outlet will be restrained from exceeding 20 kg/cm^2, as shown in Figure R-2-7.

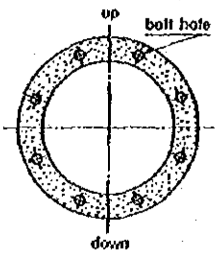
d) Conclusion

The results of the examinations revealed that water column separation will be prevented and the maximum pressure can be restrained from exceeding 20 kg/cm^2 by providing 4 surge tanks at 4 places and using slow-closing check valves.

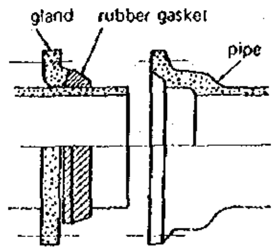
FIG R-2-7 WATER HAMMER PRESSURE CURVE

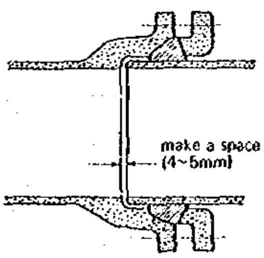


■The main points of Jointing

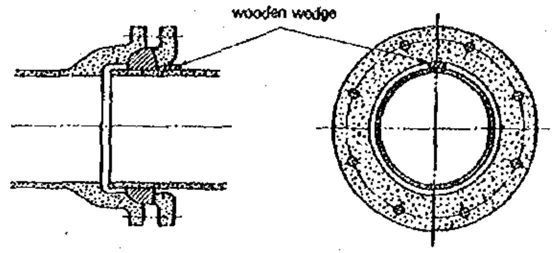


(1) Lower the Pipe to the fixed spot by Landing. Absolutely do not give shocks. If the positions of bolt holes are parted evenly from the center, screwing becomes easier.
(2) Remove cross shape wood carefully from spigot.
(3) Clean completely, oil, sand, and other foreign stickings attached to spigot outside of socket for the width of approx. 40mm from the end and bolt holes.
(4) Put gland on spigot.
Clean completely, sand, other foreign matters attached to gland outside and inside as well as bolt holes. Face the side to be in contact with gasket to socket. Gentle revolving will make the work easier.
(5) Coat spigot outside and socket inside with soap water. (30cc power soap is dissolved in 1 litre water.) Coating will protect rubber gasket from being scratched, will make surfaces smooth and processing will become easier.
(6) Coat rubber gasket with soap water and slip it on spigot. Place rubber gasket 120–150mm from spigot end.
(7) Insert spigot into socket. Insert slowly avoiding shock. Leave clearance between two pipes, allowing for future flexibility of pipe.

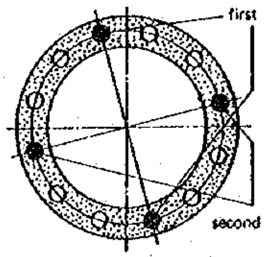




(8) Maintain clearance evenly between spigot outside and socket inside and then push the rubber gasket into the fixed position of the socket inside. Rubber gasket should lean into position of the socket. Unless both centers of spigot and socket of pipes are in correct alignment, the rubber gasket may get twisted or it may not adhere to the pipes; this will cause breaking of gland during bolting, a consequent water leak and other accidents. Even in the case when insertion is difficult the rubber gasket should not be forced by the gland but after being coated with soap water should be tapped softly with a stick.
(9) Slide the gland and line up the centers of the pipe bolt holes with those of the gland.
Make a space equally with the aid of a wedge between the gland and the outside surface of spigot. If both bolt holes do not line up, inclined force may lead to failure, therefore the centers of those holes should be lined up exactly.



(10) Clean up bolts and nuts.
(11) At first, four bolts should be inserted into four bolt holes above, below, to the right and to the left and gently bolted.
(12) Then remaining bolts should be put into all other holes. Check whether all bolt holes are filled by bolts and nuts, since there is a tendency to forget to fill all bolt holes when there are a large number of bolt holes.
(13) Tighten symmetrically and by degrees -- that is, above and below at first, then to the right and to the left and so on -- using a ratchet wrench or a spanner as shown in figure 6. Do not tighten at once. Tighten all bolts one by one degree. Finally tighten all bolts to necessary torque. Do not tighten down a particular bolt excessively, as this may cause gland to break.
(14) Check the nuts and bolts one by one once more to make sure that all nuts have the necessary torque.



Even if a bolt was tightened under the necessary torque, it can become loose again after the next one has been bolted. Therefore, repeat tightening of all bolts and nuts uniformly several times.

(15) Proper tightening torques are shown as per the Table below.

Size of bolt	torque (kg·m)	diameter of Pipe (mmφ)	length of arm of wrench which is suitable for bolting in needed torque for an ordinary worker (cm)
M20	10	200~600	25
M24	14	700~800	35
M30	17	900~1,200	40
M30	20	1,350~2,400	45

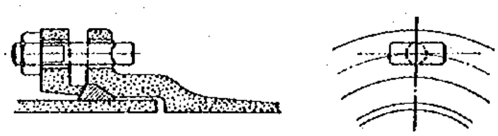
Check your own physical strength with the aid of torque – wrench in advance.

Check it occasionally for the purpose of proper bolting. Bolting by a ratchet wrench is much efficient and easier than by a spanner.

(16) Close attention should be paid to the following points for bolting:

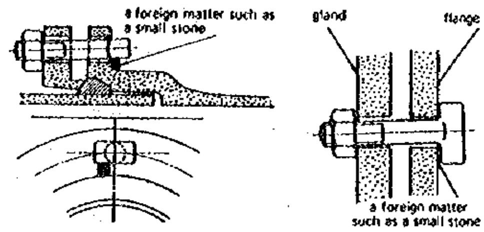
1) Example of good bolting

Bolts should be absolutely parallel with the pipe axis. The gland should be at a perfect right angle to the pipe axis. Nuts should be absolutely level with the surface of the gland.

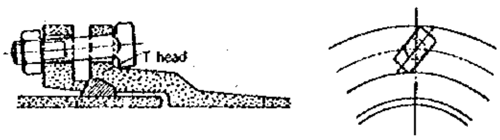


2) Examples of bad bolting

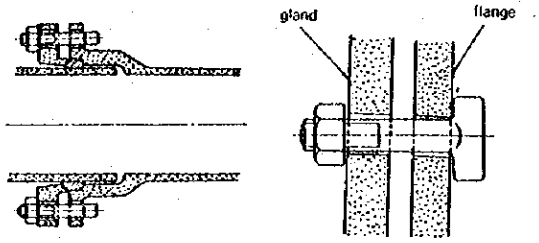
① If bolts or pipes are not sufficiently clean, or foreign matters such as a small stone comes between a bolt and the flange of the pipe, the bolt will be inclined or bent and bolting will be faulty.



② When the T head of a bolt is riding up against the shoulder part of the flange at the socket of the pipe, bolting will be faulty.



③ When the gland axis does not coincide with the pipe axis, the lip extension of the gland touches against the flange of another pipe, the gland will not be straight, or the adherence of the rubber gasket will be unequal.



④ If axis of bolt holes of the gland and the pipe do not line up, bolts will incline or bend and bolting will be faulty.

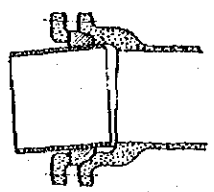
(17) Permissible degree of bend in the case of bending piping at a joint is shown as per a Table below.

Pipe dia. and effective length (mm/M)	Deflective length permissible to a pipe (~cm)	Permissible deflection
75x4	35	5°00'
100x4	35	6°00'
150x5	44	6°00'
200x5	44	5°00'
250x5	35	4°00'
300x6	35	3°20'
350x6	50	4°50'
400x6	43	4°10'
450x6	40	3°50'
500x6	35	3°20'
600x6	29	2°50'
700x6	26	2°30'
800x6	22	2°10'
900x6	21	2°00'
1000x6	19	1°50'
1100x6	17	1°40'
1200x6	15	1°30'
1350x6	14	1°20'
1500x6	12	1°10'
1600x4 ~ 2400x4	10	1°30'

Arrange the pipes in a straight line at the beginning of the jointing work. Set the rubber gasket at the correct position and screw nuts loosely. Then incline the pipe. Finally tighten up the bolts to necessary torque.

Consequently, in this case a trench should be dug out of considerable width at the place where pipe-laying is to be curved. When Pipe is bent future ground fluctuations should be taken into consideration, making the curve as slight as possible and keeping enough elasticity at the jointing part.

(18) If the rubber gasket does not fit smoothly, it should not be forced by bolting but should be removed and cleaned once more.

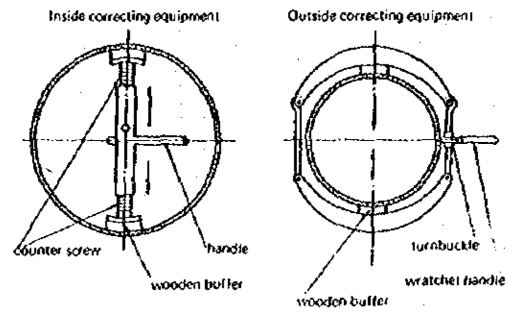


Main points about jointing of cut pipes and elliptical pipes

Due to high temperature in the manufacturing process, the Ductile Iron Pipe is somewhat elliptical. There is no trouble in assembling pipes because, the spigot of the pipe is an almost perfect circle and is corrected. The center part of the pipe, however, is rather elliptical in some cases. (within a permissible deviation in every case, and with the circumference of the pipe also in the tolerance.)

Therefore close attention should be paid to the following.

(1) Use the specially measured pipe as a cut pipe. Cut pipe on which a white line is wound at about one meter from the spigot end can be cut at any place. Therefore, these pipes to be cut should be kept separated from others.



(2) If by chance the cut place of the pipe is an ellipse, ductile iron pipe can be corrected on the spot due to its high ductility.

There are 2 types of correcting equipment, one correcting equipment, one correcting from the inside, the other from the outside.

(3) Joint a pipe under correction to required size and it required torque with the aid of correcting equipment.

(4) Remove the correcting equipment after bolting completely.

Check that there are no tools left inside after removing the correcting equipment.

(5) Check whether the nuts have become loose after removing the correcting equipment.

Tighten it again to needed torque if necessary.

APPENDIX R-4

MEASURES FOR PREVENTION OF SCHISTOSOMIASIS

(1) General

The medical circles have developed the preventive measures of schistosomiasis as a result of the studies on the outbreak process and the behavior of three kinds of schistosomes, that is, the Schistosome Japonicum, Schistosome Mansoni and Schistosome Haematobium. Such measures should be taken in planning some preventive measures of schistosomiasis in raw waters to be supplied in the project.

These schistosomes find their way into the human body through his skin. . The Schistosome Japonicum and Schistosome Mansoni are parasitic on the portal veins, while the Schistosome Haematobium on the venous veins in the pelvis region and especially on the venous vein sacs for the pelvis and anal. The former two move to capillary veins of the intestinal tract in order to lay eggs. Therefore, the eggs are excreted with faecal matter, while the eggs of the latter with urine through the pelvis, urethra and urete. The symptom and condition of the schistosomiasis caused by the former is different from these caused by the latter due to the fact that they lay their eggs at a different part of the human body.

The birthplace of the Schistosome Japonicum is Japan, and that of the Schistosome Mansoni and Schistosome Haematobium is originally the Continent of Africa. However, the latter two have come to stay in the South America and the Middle and Near East. Especially in the basin of the Nile river, a number of these schistosomes reaches to a considerable extent. So it is afraid that many Schistosome Mansoni and Schistosome Haematobium live in the project area.

(2) Pathogen and its Growth

A schistosome looks like an oelworm in appearance, but having a sucking mouth, it is a trematoda. The body of a male pathogen is 8-22 mm long and 0.5 mm wide, and has a sucking mouth at the upper part of its body and a sucking disk on its abdomen. The size of sucking organs nearly the same. Its upper body is cylindrical, but the lower body from the sucking disk is flat, and forms sheath-wise shape, which is convenient for the male and female to embrace.

A female pathogen is dark-brown, and also stringy. The body of a female pathogen is generally 22 mm in length, and its maximum diameter is 0.3 mm. Its upper body is thin, and has a sucking mouth on its uppermost. The suck disk is larger than its sucking mouth.

A male imago and female imago, embracing each other, are parasitic on the portal vein. In the egg laying stage, a couple of the imagos moves to the capillary veins of the intestinal tract keeping embracing each other, and then the female imago, stretching her body, lays eggs one by one, moving backward direction. As a result, the eggs clog a capillary vein, but finally are flown to the intestinal tract, and excreted with the faecal matter.

However, most of the eggs is carried by blood into the liver, lag and other internal organs. No miracidium is formed in an egg immediately after it is laid. However, when the eggs are extracted with faecal matter, the miracidium, being hatched, floats out. The miracidium, being covered by ciliums, actively moves in water, and intrude into a snail, the secondary host for the schistosomes. The miracidium intruded into the snail, takes off the ciliums, and start multiple division as a sporocyst. The daughter sporocyst is grown up by the sporocyst, and it moves to the liver of the snail forming a slender bag-wise shape, in which many cercarias are formed. All of such cercarias are of one sex. These cercarias move into water and find their way into the mammals.

The cercarias intruded into the body of mammals through the skin, the last host, go into the blood and stay in the veins for the liver for about two weeks. However, after three weeks from the intrusion, some couples of imagos are found embracing in the venous vein of the portal veins. They start laying eggs about four weeks after the intrusion. The medium host is an amphibious snail of dioecism and its body is 4 to 7 mm long. They live in water during their early life period, but move to the earth and water during their rest life.

(3) Infection Course and its Prevention

Cercarias leave the snail, the middle host, move into the water and attach themselves to the skin of human body, thus, human is infected with Schistosomiasis. These cercarias cannot be alive for more than 24-48 hours even at an optimum water temperature of 23°C. A cercaria consists of a trunk and a cilia, and it intrude into human body driving itself by moving cilia. However, the cilia is easily taken off. For example, all the cercarias loose their cilia, and cannot intrude into human body anymore, if the water, in which they live, is dropped from about 30 cm elevation to make enough sprays.

Cercarias itself is quite weak against chemical fluid. Chroline can easily kill cercarias. Therefore, it is considered that 100% of cercarias in the treated water is dead. In raw water supply, most cercarias are killed through the process of pumping up and conveyance in pipelines with a water velocity of more than 1 m/sec.

However, it has been impossible to exterminate the schistosomiasis because the snail, the secondary host, provide the schistosomas with a place to live in, and furthermore, ten thousands cercarias are born in one snail of the host.

The snail is extremely small. Furthermore, young snails are much more smaller than the adult snail. Therefore, young snails are easily transported by water flows. In supplying raw water, it is afraid that young snails start propagation in water holes, ponds, and rivers, if they are transported and scattered with irrigation water. The snails living in the basin of the Nile river might be Biomphalaria or Bulinus tropicus, which is the secondary host for Schistosome Haematobium.

These snails have durability against chemicals. It takes about 8 hours to kill the snail by chemicals. However, they cannot live without sunshine, except MONOWARAGAI which can live without sunshine if it is provided with water temperature of 23°C and enough algae. Under the circumstances, detailed study should be made on the secondary host in the stage of final design. Copper sulfate might be available for this purpose but as it is poisonous, it cannot be used to a full extent to exterminate the snails completely.

(4) The Present Condition of Schistosomiasis in Cairo City

Cairo city was not infected seriously by schistosomiasis in the past because secondary hosts were swept away by the floods of the Nile river, but the prevention of floods of the Nile river will bring water-hyacinths into existence and the number of secondary hosts will increase for lack of oxygen. Especially, the number of the secondary hosts will increase remarkably in tributary streams, canals and swamps of the Nile river because they have not plentiful flowing water, but the secondary hosts will not increase at its main stream because it has always stable flowing water. Making an inference from the growth and infection course of schistosomiasis mentioned above, study on present conditions of schistosomiasis was made as follows.

(a) There are no cases of infection from drinking water at all because drinking water is conveyed by pressure under the control of perfect facilities for sterilization.

(b) The way of raw water supply in old Cairo is to sprinkle pressured water by pumps, rubber hoses and so forth. In this case, young snails and water are sprinkled at the same time but they can not grow because of the dry area. Even if they grow, eggs discharged from human body will not intrude into hosts because excrementitious matters are not scattered in urban area.

(c) There are possibilities of growth for young snails in the areas which are irrigated under the condition of flat raw water without sprinkling water and in the areas which have pools for a certain period, but there will be little probabilities to be original birth places because water in the said areas is often exhausted there. Counterplan regarding operation and maintenance of the said areas should be studied so that they may not be original birth places.

(d) Maddi area is supposed to be the nest of schistosomiasis because this area adopts gravity type of irrigation after pumping up raw water. That is, the optimum temperature, sunshine and nutrition let young snails grow during their stay in open channels and they live in channels, pools, the concrete walls of turn-outs and so forth. In case that eggs intrude into young snails, cercarias will be born. As the result of this, this whole area will fall into the most dangerous situation.

(e) It is feared that swamps along the Nile river, areas which possess pools continuously at the northern part of Cairo city and the other areas, may be the nest of schistosomiasis.

(5) Preventive Measures for Schitosomiasis

Concerning the raw water supply facilities, the next two items were decided so that the diseases caused by schistosomiasis might be prevented.

(a) To perish cercarias.

(b) To prevent the second birth of secondary hosts.

i) Regarding (a), chlorination is performed at water lifting stations. Pipelines are used for water conveyance and the velocity of water conveyance is over 1 m/sec. Water sprinkling method, using rubber hoses, sprinklers and the other facilities, is to be taken for irrigation purpose.

ii) Regarding (b), efforts are made to prevent the formation of pools, and especially water sprinkling should be performed by rotation so that the land becomes dry from time to time to prevent the growth of young snails. In Cairo city, it is also necessary to remove pools and to keep City clean customarily.

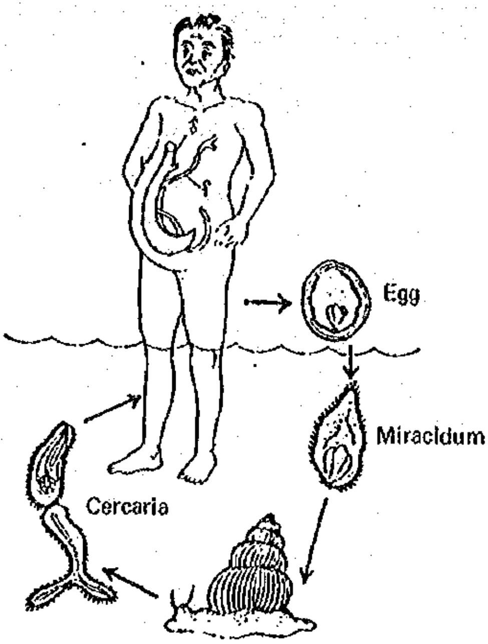
iii) The sutiable places for taking raw water are to be central and deep parts of rivers to prevent young snails from intruding into water.

iv) Calcium cyanamide should be often sprinkled into pools, swamps and on banks of rivers to prevent the multiplication of snails. As it is a chemical fertilizer, it would incidentally help the growth of vegetation. And also sprinkling of B.H.C is an effective counter-measure for numerous birth of snails.

v) In order to prevent cercarias from intruding into human body, dealing with water by hands directly should be avoided and pimethl phihalate, benzyl benzoate and other chemicals should be applied to the skin points which are feared to touch water directly. And also machines should be used to remove earth and sand from settling reservoirs and tanks so that the chances to touch water directly may be reduced.

After taking into account of the preventive measures mentioned above, the raw water supply plan should be made.

Description on measures for prevention of schistosomiasis has been explained above. But, strictly speaking, this cannot necessarily apply to all cases because this description is a general consideration. So, proper steps should be taken positively to cope with the various situations.



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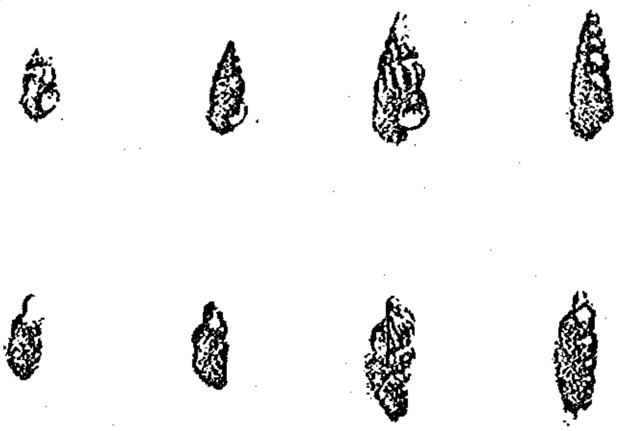
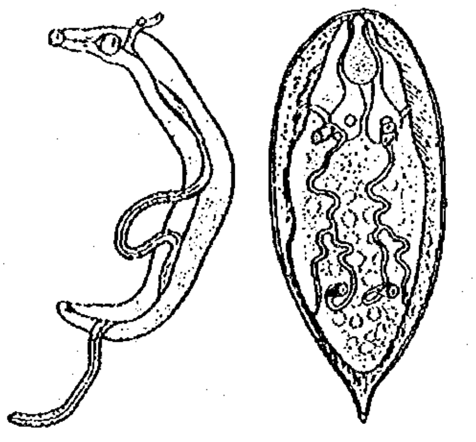
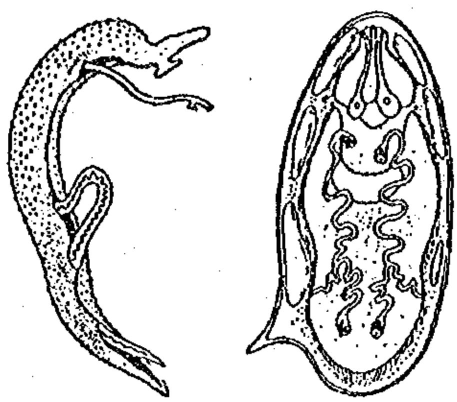


Secondary Hosts

1: *Oncomelania nosophora* (Japan)
2: *O. huponsis* (China)
3: *O. formosana* (Formosa)
4: *O. quadrasi* (the Philippines)



Schistoma Haematobium and its Egg
(Quoted from the Faust, Russell & Jung)



Schistosoma Mansoni and its Egg
(Quoted from Faust, Russell & Jung)

APPENDIXES RELATING TO THE PROJECT

APPENDIX C-1. SPECIFICATIONS OF PUMPING FACILITIES

North-east Cairo Intake Pump Station

1) Raw Water Supply Pump (Intake Pump)

a) Specifications of Pump

Quantity	4 sets (including 1 set for stand-by)
Type of pump	Vertical type Double suction volute pump
Suction dia	500 mm
Discharge dia	300 mm
Rated capacity	31 cu.m/min
Rated total head	100 m
Speed	1,450 r.p.m.
Prime mover	Vertical induction motor
Required horse power	730 kw
Location	Indoor

b) Accessories (per 1 set of pump)

Bed	1 set
Foundation bolts and nuts	1 set
Jointed vertical shaft and coupling	1 set
Compoung gauge	1 set
Pressure gauge	1 set
Air vent valve	1 set
Priming detector	1 set
Magnet valve (gland sealing)	1 set
Tool and box	2/4 sets (2 sets per 4 sets of pump)

c) Material of pump main parts

Casing	Cast iron
Impeller	Stainless cast steel
Shaft	Carbon steel

Shaft sleeve	Stainless steel
Liner ring	Stainless steel

2) Valves

a) Suction valve

Quantity	4 sets
Type of valve	Manual operate sluice valve
Dia	500 mm

b) Discharge valve (1)

Quantity	4 sets
Type of valve	Motor operated sluice valve
Dia	450 mm

c) Discharge valve (2)

Quantity	4 sets
Type of valve	Manual operated sluice valve
Dia	450 mm

d) Check valve

Quantity	4 sets
Type of valve	Slow closing type check valve
Dia	450 mm

e) Valve for discharge main pipe

Quantity	1 set
Type of valve	Manual operated sluice valve
Dia	1,200 mm

3) Drainage pump for pump room

Quantity	Duty pump	1 set
	Stand-by pump	1 set
type of pump	Submersible motor pump	

Dia	80 mm
Rated capacity	0.5 cu.m/min
Rated total head	20 m
Submersible motor	3.7 kw
Accessories (per 1 set of pump)	
Submersible motor	1 set
3.7 kw 380 V 50 rlz	
Sluice valve	1 set
Check valve	1 set

4) Steel pipe for around pump room

Suction & discharge steel pipe for around pump room 1 set

Dia	300 mm - 1,200 mm

5) Travelling crane

Quantity	1 set
Type of crane	Indoor motor operated overhead crane
Capacity (Main)	7.5 tons
Span	13.7 m
Lift	14 m
Travel length	38 m
Control	Pendant push button switch at floor level

6) Sluice gate for pump suction pit

a) Specifications

Quantity	1 set
Type of gate	Sluice gate rising stem type
Gate width	2,000 mm
Gate height	2,000 mm
Valve operator	Manual
Location	Outdoor

b) Materials

Frame	Cast iron

Gate body	Cast iron
Frame seat	Brass
Gate body seat	Brass
Stem	Stainless steel

7) 3,000 KVA Power Transformer

No. required	2 sets
Type	Oil immersed, self-cooled sealed tank type
Location of service	Outdoor
Ratings	
Kind of rating	Continuous
Rated output	
Primary	3,000 KVA
Secondary	3,000 KVA
No. of phases	3 phase
Frequency	50 Hz
Voltage	
Primary	P11.0-F10.75-R10.5-F10.25-F0.0 KV
Secondary	6,300 V

R denotes the rated voltage and F the full capacity tap voltage

Insulation level (B.I.L.)	
Primary	90 KV
Secondary	60 KV
Connection	
Primary	Delta
Secondary	Wye, with neutral brought out
Polarity	Subtractive
Angular displacement and vector group	Dy11 degree: 330
Neutral grounding method	
Primary:	

Secondary: Solidly earthed

Tertiary:

Temperature rise

Temperature rise shall be not more than 55°C for winding as measured by resistance method and shall be not more than 55°C for oil as measured by thermometer method at ambient temperature 40°C.

8) Three Phase Induction Motor

No. of set:	4 sets
Type:	Vartical Guarded drip-proof wound rotor type
Application:	For pump
Starting method:	Starting resistance method
Class of insulation material:	B Class

Ratings:

Output	730 kw
Volage	6,300 volts
Frequency	50 Hz
No. of poles	4 poles
Synchronous speed	1500 rpm
Time rating	Continuous

Accessories (for each):

1 set - Starting resistance

1 set - Starting controller

1 set - Motor operating brush lifting equipment

9) 10.5kV Switchboard

a) Incoming feeder (1) 2 panel

W 1000 H 2300 D 2200 mm

b) Incoming feeder (2) 2 panel

W 1000 H 2300 D 2200 mm

c) Metering

W 1000 H 2300 D 2200 mm

d) Outgoing feeder to transformer 2 panel
W 1000 H 2300 D 2200 mm

10) 6.3kV Switchboard

a) Incomeing feeder from transformer 2 panel
W 700 H 2300 D 2000 mm

b) Potential transformer 1 panel
W 700 H 2300 D 2000 mm

c) Outgoing feeder for pumps 4 panel
W 700 H 2300 D 2000 mm

d) Bus bar sectionalizer 1 panel
W 700 H 2300 D 2000 mm

e) Outgoing feeder for 100 kVA transformer: 2 panel
W 700 H 2300 D 2000 mm

f) 100 kVA transformer: 2 panel
W 1300 H 2300 D 2000 mm

11) 380V Switchboard, etc.

a) 380 outgoing feeder 5 panel
W 700 H 2300 D 800 mm

b) Auxiliary relay 3 panel
W 700 H 2300 D 800 mm

c) Instrumentation 1 panel
W 700 H 2300 D 800 mm

d) Battery charger 1 panel
W 1000 H 2300 D 1600 mm

e) Battery 1 panel
W 1500 H 2300 D 1600 mm

Heliopolis Booster Pump Station

1) Raw Water Supply Pump (Booster Pump)

a) Specification of Pump

Quantity	4 sets (include 1 set stand-by)
Type of pump	Horizontal Type Double Suction Volute Pump
Suction dia	500 mm.
Discharge dia	300 mm
Rated capacity	31 cu.m/min
Rated total head	67 m
Speed	1,450 r.p.m.
Prime morver	Horizontal Induction Motor
Required horse power	480 kw
Location	Indoor

b) Accessories (per one set of pump)

Common bed	1 set
Foundation bolts and nuts	1 set
Coupling	1 set
Compound gauge	1 set
Pressure gauge	1 set
Air vent valve	1 set
Priming detector	1 set
Magnet valve (gland sealing)	1 set
Tools and Box	2/4 sets (2 sets per 4 sets of pump)

c) Material of pump main parts

Casing	Cast iron
Impeller	Stainless cast steel
Shaft	Carbon steel
Shaft sleeve	Stainless steel
Liner ring	Stainless steel

2) Valves

a) Suspension valve

Quantity	4 sets
Type of valve	Manual operate sluice valve
Dia	500 mm

b) Discharge valve (1)

Quantity	4 sets
Type of valve	Motor operated sluice valve
Dia	450 mm

c) Discharge valve (2)

Quantity	4 sets
Type of valve	Manual operated sluice valve
Dia	450 mm

d) Check valve

Quantity	4 sets
Type of valve	Slow closing type check valve
Dia	450 mm

e) Valve for discharge main pipe

Quantity	1 set
Type of valve	Manual operated sluice valve
Dia	1200 mm

f) Entrace sluice valve for Heliopolis regulating tank

Quantity	2 sets
Type of valve	Manual operated sluice valve
Dia	1,350 mm

3) Drainage pump for pump room

Quantity	Duty pump 1 set Stand-by pump 1 set
Type of pump	Submersible motor pump
Dia	80 mm
Rated capacity	0.5 m^3/min.
Rated total head	20 m
Submersible motor	3.7 kw

Accessories (per one set of pump)

Submersible motor 3.7 kw 380 V 50 rlz	1 set
Sluice valve	1 set
Check valve	1 set

4) Steel pipe for around pump room

Suction & discharge steel pipe for around pump room: 1 set

Dia : 300 mm-1,200 mm

5) Travelling crane

Quantity	1 set
Type of crane	Indoor motor operated overhead crane
Capacity (Main)	5 tons
Span	11.5 m
Lift	11 m
Travel length	33 m
Control	Pendant push button switch at floor level

6) Sluice gate for Heliopolis regulating tank

a) Specification

Quantity	1 set
Type of gate	Sluice gate rising stem type
Gate with	2,000 mm
Gate height	2,000 mm
Valve operator	Manual
Location	Outdoor

b) Material

Frame	Cast iron
Gate body	Cast iron
Frame seat	Brass
Gate body seat	Brass
Stem	Stainless steel

7) 2000 kVA Power Transformer

No. required:	2 sets
Type:	Oil immersed, self-cooled sealed tank type
Location of service:	Outdoor
Ratings	
Kind of rating:	Continuous
Rated output	
Primary:	2,000 kVA
Secondary:	2,000 kVA
No. of phases:	Three phase
Frequency	50 Hz
Voltage	
Primary:	P11.0-F10.75-R10.5-F10.25-F0.0 KV
Secondary:	6300 V

R denotes the rated volatage and F the full capacity tap voltage.

Insulation level (B.I.L.)

Primary: 90 kV

Secondary: 60 kV

Connection

Primary: Delta

Secondary: Wye, with neutral brought out

Polarity: Subtractive

Angular displacement and vector group: Dy11

degree: 330

Neutral grounding method

Primary:

Secondary: Solidly earthed

Tertiary:

Temperature rise

Temperature rise shall be not more than 55°C for winding as measured by resistance method and shall be not more than 55°C for oil as measured by thermometer method at ambient temperature 40°C.

8) Three Phase Induction Motor

No. of sot: 4 sets

Type: Horizontal Guarded drip-proof wound rotor type

Application: For pump

Starting method: Starting resistance method

Class of insulation material: B Class

Ratings:

Output	480 kW
Volage	6,300 volts
Frequency	50 Hz
No. of poles	4 poles
Synchronous speed	1500 rpm
Time rating	Continuous

Accessories (for each):

1 set - Starting resistance

1 set - Starting controller

1 set - Motor operating brush lifting equipment

9) 10.5kV Switchboard

a) Incoming feeder (1) 2 panel
W 1000 H 2300 D 2200 mm

b) Incoming feeder (2) 2 panel
W 1000 H2300 D2200 mm

c) Metering
W 1000 H 2300 D2200 mm

d) Outgoing feeder to transformer 2 panel
W 1000 H 2300 D 2200 mm

10) 6.3kV Switchboard

a) Incoming feeder from transformer 2 panel
W 700 H 2300 D 2000 mm

b) Potential transformer 1 panel
W 700 H 2300 D 2000 mm

c) Outgoing feeder for pumps 4 panel
W 700 H 2300 D 2000 mm

d) Bus bar sectionalizer 1 panel
W 700 H 2300 D 2000 mm

e) Outgoing feeder for 100 kVA transformer: 2 panel
W 700 H 2300 D 2000 mm

f) 100 kVA transformer : 2 panel
W 1300 H 2300 D 2000 mm

11) 380V Switchboard, etc.

a) 380V outgoing feeder 5 panel
W 700 H 2300 D 800 mm

b) Auxiliary relay 3 panel
W 700 H 2300 D 800 mm

c) Instrumentation 1 panel
W 700 H 2300 D 800 mm

d) Battery charger 1 panel
W 1000 H 2300 D 1600 mm

e) Battery 1 panel
W 1500 H 2300 D 1600 mm

APPENDIX C-2. BREAKDOWN OF PROJECT COST

(1) CONSTRUCTION COST (Unit: US$1,000)

Items	Quantities	Unit Price	Price	Remakrs
Nasr City Area				
(Civil Works)				
Intake facilities	L.S.		400	
Foundation of Pumps	L.S.		600	
Building of Pump House	570 sq.m.	200/1,000m^2	114	
Excavation & Embankment for Water Supply Pipeline	5.1 km	17	87	
Small Structure for Pipeline	L.S.		9	
Regulating Tank	4,580 sq.m.	60/1,000m^2	275	
Sub-total			1,485	
(Equipment)				
Equipment and Others for Pumping Station	L.S.	470	4,182	
Pipe 1,200 mm	5.1 km	470	2,397	See next sheet
Valves & Others	L.S.		240	
Sub-total			6,819	
(Setting Works)				
Pumping Station			245	
Pipe 1,200 mm.	5.1 km	51	260	
Valves & Others			26	
Sub-total			531	
Overhead for Construction			806	
Total			9,641	F.C. 6,819 L.C. 2,822

Items	Quantities	Unit Price	Price	Remarks
Heliopolis Area				
(Civil Works)				
Excavation & Embankment for Supply Pipeline	9.8 km	25	245	
Regulating Tank	3,130+4,170	60/1,000m^2	438	
Small Structure for Pipeline	L.S.		25	
Sub-total			708	
(Material)				
Pipe φ1,350	9.8 km	586	5,743	
Pipe φ1,200	9.8 km	470	4,514	
Valves & Others	L.S.		1,026	
Sub-total			11,283	
(Setting Works)				
Pipe φ1,350	9.8 km	60	588	
Pipe φ1,200	9.8 km	51	500	
Valves & Others	L.S.		109	
Sub-total			1,197	
Overhead			762	
Total			13,950	F.C. 11,283 L.C. 2,667
Helwan Area				
(Civil Works)				
Excavation & Embankment for Supply Pipeline	4.8 km	8	38	
Small Structure for Pipeline	L.S.		4	
Regulating Tank	830 sq.m.	60/1,000m^2	50	
Sub-total			92	

Items	Quantities	Unit Price	Price	Remarks
(Materials)				
Pipe ϕ500	4.8 km	111	533	
Valves & Others	L.S.		53	
Sub-total			586	
(Setting Works)				
Pipe ϕ500	4.8 km	10	48	
Valves & Others	L.S.		7	
Sub-total			55	
Overhead			59	
Total			792	F.C. 586 L.C. 206

(2) COST OF CONSTRUCTION EQUIPMENT

(Unit: US$1,000)

Name of Equipment & Specifications	Number	Unit Price	Price	Remarks
Back hoe B.C. 0.6 cu.m.	5	50	250	B.C.= Bucket capacity
Truck clain 25 tons	3	75	225	
Dump truck 11 tons	3	25	75	
Truck 11 tons	2	20	40	
Feel tanker 5,000 l.	1	10	10	
Tire roller	2	20	40	
Tiredozer	2	40	80	
Inspection Car (Jeep)	4	5	20	Wagon Type
Trailor 30 tons	2	25	50	
Concrete Mixer with Engine 0.5 cu.m.	3	7	21	
Pumps with engine	4	0.5	2	
Sub-Total			813	
Spare Parts			81	10%
Transportation fee			304	
Total			1,198	

APPENDIX C-3. SOCIAL EVALUATION

The inherent implication of social indicators is, in nature, to comprehensively grasp a society as the field of people's living or human life system which has many attributes, to describe its state as it is and to evaluate it from the criterion of social values assumed.

Inferring from many current problems like pollution and urban problems, which most of developed countries now face, such the multi-atributed and comprehensive evaluating system as intended by social indicators have been urgently needed for selecting development policies in lieu of economic efficiency-oriented evaluating system.

If the application of social indicators to evaluate development projects is, however, confined only to description of the state of people's living, it is insufficient for that purpose. Without the technique that predicts, what changes in inputs to human life system would be caused by the development project components, and how the very changes would diffuse to many aspects of human life system through various feedback mechanisms built in the social system, and consequently what changes in outputs from human life system would be brought about, one can not have relevancy in evaluation of the development project. So it is this predicting technique that founds the application of social indicators to evaluate development projects.

Needless to say, Nasr City Raw Water Supply Project is one of the sub-projects in a broader development project which intends to improve the existing water supply system. The improvement of water supply system would, apparently, lead to various changes in inputs to human life system.

In production process, much more water as raw materials of higher quality and lower price could be available. Similarly in consumption process, much more water as consumption goods of the said characteristics could be consumed.

And furthermore these inputs' changes would turn out various outputs changes or changes of states in human life system through complicated various feedback mechanisms or causal sequences, that is, through diffusion processes. In production process, the said newly available water as raw materials would contribute to produce much more final products of higher quality and lower prices, which can be considered as outputs changes in production process. In turn, these final products get into consumption process and would cause to change the previous consuming behaviours of consumers. And also the said newly available water as consumption goods would contribute to change the previous life style of laundry and taking bath and to decrease the previous disease occurence rate.

Taking into account the characteristics of Nasr City Raw Water Supply Project;

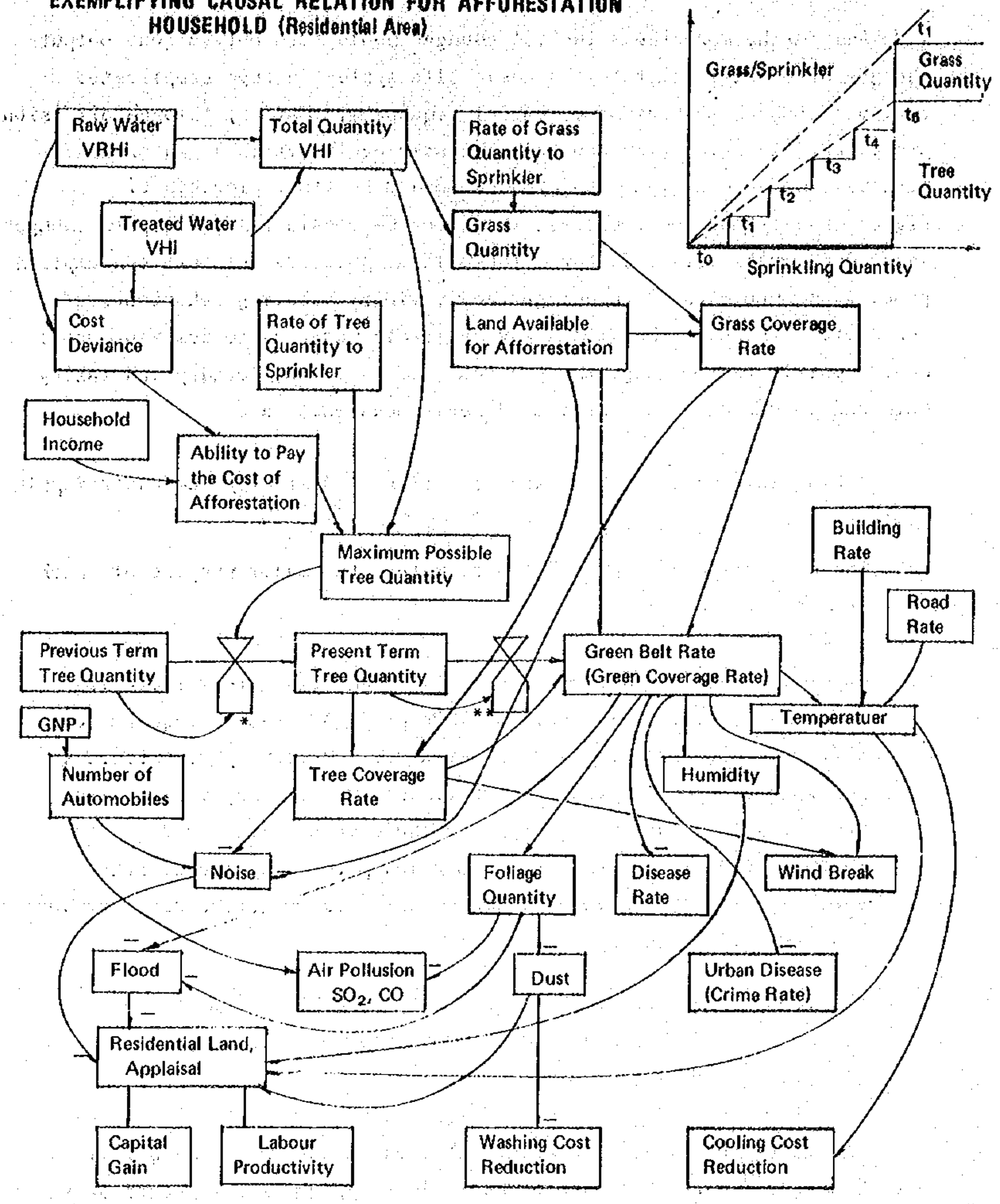
(1) The kind of water supplied is raw water, the major purpose of which is city irrigation.

(2) Major land uses in Nasr City served by raw water are residential and governmental office areas. This project's causal sequence or diffusion processes to be considered are necessarily restricted to relevant ones by the above characteristics. Its major diffusion processes to be accounted might be those ones which would appear in the water use pattern in consumption process by the household sector and in the urban-environment which is connected with various processes in social system.

The causal sequences to be considered is shown in Fig. C-3-1 paying attention on the main purpose of raw water utilization, that is, city irrigation.

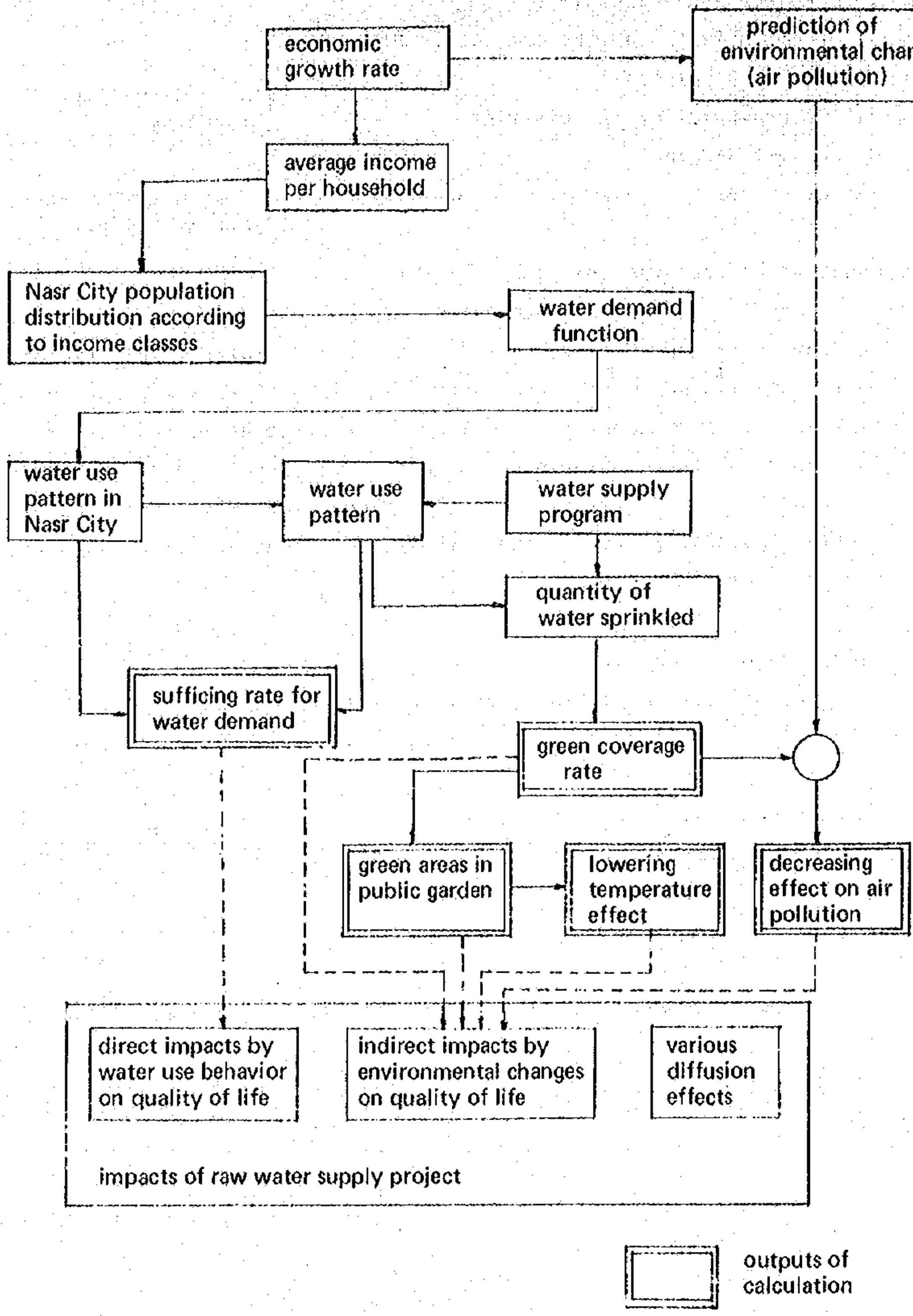
The concrete calculation method that estimates what consequences are brought about on the state of human life system is set out as shown in Fig.C-3-2.. on the basis of the above mentioned causal framework considering the restraint of data gathered during the researches carried outin Cairo.

FIG. C-3-1.
EXEMPLIFYING CAUSAL RELATION FOR AFFORESTATION HOUSEHOLD (Residential Area)



Note: * Increase Rate
** Decrease Rate

FIG. C-3-2.
CALCULATION METHOD ESTIMATING IMPACTS OF RAW WATER SUPPLY PROJECT ON THE QUALITY OF URBAN LIFE



The fundamental assumptions for the calculation are listed up as follows;

(1) The project area consists of,

a) residential area (the 1st, 6, 7, 8th zone)		Nasr City city planning
b) El Azhar University		
c) Governmental office district		
d) Public Garden		
e) Green belt	------------------------------	Nasr City Raw Water Supply Project

(2) Cases to be compared (cf. Table C-5-1)

Case 1 is Nasr City Raw Water Supply Project, which incorporates the raw water supply system besides the treated water supply system. On the other hand, Case 2 and Case 3 are without raw water supply systems. The net quantities of treated water to be served per day per capita in the household sector are 204 l/d/c for case 1, 15l/d/c for case 2 and 170 l/d/c for case 3, calculating from quantities of treated water delivered to the parcels of housing in the residential area for domestic purposes.

(3) Growth Rate of Population

The population growth rates in overall Egypt are supposed to gradually decrease from 2.24% a year in 1975 to 2.00% a year in 1985.

(4) Rate of Economic Growth

Gross domestic production is supposed to constantly increase by 7% of the annual growth rate from 1975 to 1985.

(5) Income distribution

Assuming income distribution in 1985, Table C-3-2 is followed.

(6) Nasr City population distribution according to income classes Cf. Table C-3-3.

Table C-3-1 CASES TO BE COMPARED

1985

	Total areas	green	Case 1 raw water sprinkling[a]	Case 1 treated water domestic	Case 2 treated water sprinkling	Case 2 treated water domestic	Case 3 treated water sprinkling	Case 3 treated water domestic
	ha	ha	cu.m	cu.m	cu.m	cu.m	cu.m	cu.m
1st zone								
housing	80.5	28.2	2,538	5,660	338[b]	4,167	169[e]	4,702
others	111.0	34.4	3,096	1,867	1,376[c]	1,374	688[j]	1,551
6th zone								
housing	182.2	46.3	4,167	8,857	556[b]	6,519	278[e]	7,358
others	187.4	60.7	5,463	3,869	2,428[c]	2,848	1,214[j]	3,214
7th zone								
housing	95.1	33.3	2,997	10,300	400[b]	7,582	200[e]	8,557
others	116.2	37.5	3,375	1,677	1,500[c]	1,234	750[j]	1,393
8th zone								
housing	88.7	31.1	2,799	7,972	373[b]	5,868	187[e]	6,623
others	72.9	22.6	2,034	1,385	904[c]	1,019	452[j]	1,150
total								
housing	396.5	138.9	12,501	32,789	1,667	24,136	834	27,240
others	487.5	155.2	13,968	8,798	6,208	6,475	3,104	7,308
TOTAL	884.0	294.1	26,469	41,587	7,875	30,611	3,938	34,548
					38,486		38,486	
El Azhar University	226.0	101.7	9,153	7,965	3,051[d]	4,306	2,034[j]	5,323
					7,357		7,357	
Governmental Office District	258.0	103.2	9,288	9,120	3,096[d]	5,355	2,064[j]	6,387
					8,451		8,451	
Public Garden	199.3	179.4	16,146	2,128	5,382[d]	1,124	5,382[d]	1,124
					6,506		6,506	
Green belt	60.0	60.0	5,400	0	0	0	0	0
					0		0	
GRAND TOTAL	1,627.3	738.4	66,456	60,800	19,404	41,396	13,418	47,382
					60,800		60,800	

total amount of treated water to be served $160,000^{person} \times 0.380^{cu.m/day}$

quantity of water to be sprinkled:
- a 90 cu.m a day/ha
- b 40 cu.m a day only for 30% of housing parcel
- c 40 cu.m a day
- e 20 cu.m a day only for 30% of housing parcel
- d 30 cu.m a day
- f 20 cu.m a day

Tab. C-3-2.

Average for Each Income Class of Gross Domestic Production per Capita in Egypt (GDP/c) — 1985 —

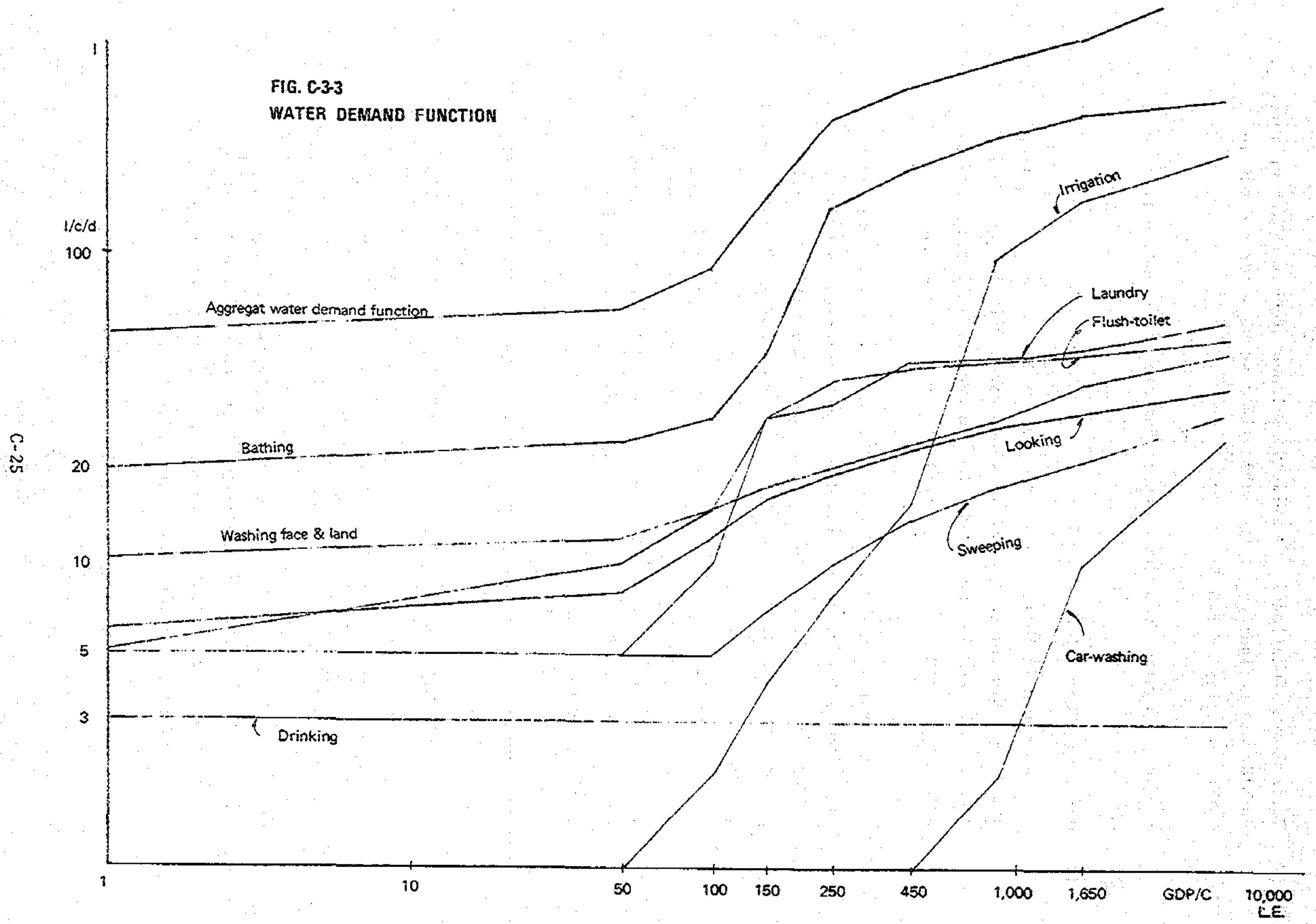
1	2	3	4	5	6	7	8	9	10	11
58,284	71,236	89,045	105,235	124,663	150,567	181,328	215,327	259,040	364,275	161,900

Tab. C-3-3.

Nasr City Population Distribution According to Income Classis — 1985 —

class	1	2	3	4	5	6	7	8	9	10	11
Distribution	1	2	5	10	14	16	15	13	13	11	100
Population (thousand)	1.6	3.2	8.0	16.0	22.4	25.6	24.0	20.8	20.8	17.6	160

FIG. C-3-3
WATER DEMAND FUNCTION



(7) Water demand function is formulated as linear functions taking into consideration findings of the small field surveys carried out in Cairo. Cf. Fig. C-3-3.

The major results from the calculations are shown and analyzed as follows:

(1) Sufficing Rate for Water Demand

As shown in Table C-3-4 and Fig. C-3-4, the ratios of case 2 and case 3 are followed from the calculated sufficing rate for each purpose and income class, and the average sufficing rate of each purpose, and the average sufficing rate of each income class.

The results for drinking purpose are omitted from the above table and figure because water demand for drinking is constantly 3 l/d/c in every income level so the sufficing rate for drinking purpose comes to always zero from the reason that its elasticity for income is always zero.

Water use for car-washing is also omitted from the above table and figure because the estimated number of cars owned by household sector in Nasr City in 1985 is about 4,700, but the derived water demand from those cars is negligible.

Findings drawn from these calculated results are as follows;

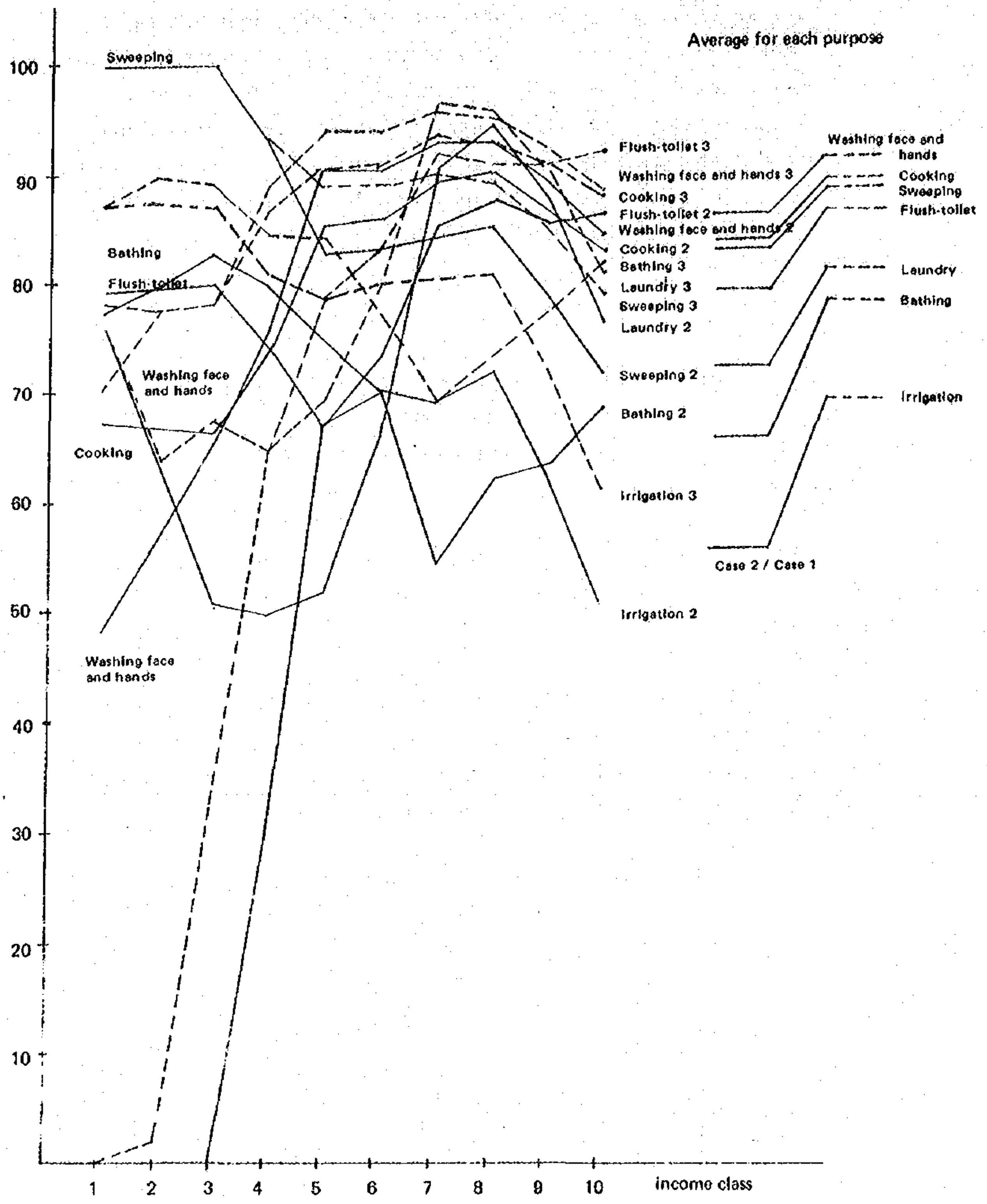
(a) Only case 1 is enough to satisfy water demand 31215 cu.m per day in 1985. The overall sufficing rate is 77% for case 2, and 87% for case 3. So the defficient quantity of water supply in the household sector in Nasr City will be 7 tons a day for case 2, and 4 tons a day for case 3.

(b) The average sufficing rates by purposes of consumption are in the same order in both Case 2 and Case 3; the lowest is one for the irrigation purpose and next follows in the order of bathing, laundry and flush-toilet.

Table C-3-4 Sufficing Rate for Water Demand in Household Section of Nasr City

		Income Class										Average in purpose
		1	2	3	4	5	6	7	8	9	10	
Cooking	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	67.3	66.9	66.3	74.0	85.4	85.9	89.4	90.1	86.6	83.1	83.7
	Case 3/1	78.3	77.6	78.2	86.9	90.7	91.0	93.7	92.9	91.0	88.1	89.8
Washing face and hands	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	48.2	56.5	65.2	75.7	90.7	90.6	92.8	93.0	89.4	84.7	86.5
	Case 3/1	70.3	77.6	78.2	88.9	94.0	94.1	95.8	95.1	92.5	88.7	91.7
Bathing	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	77.5	79.8	82.7	79.7	74.9	70.2	54.5	62.0	63.3	68.7	65.9
	Case 3/1	87.1	89.7	89.1	84.2	84.0	77.6	69.2	73.2	77.3	81.8	78.5
Laundry	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	75.9	63.0	50.9	49.8	51.9	65.8	90.4	94.3	87.7	76.8	72.5
	Case 3/1	75.9	63.6	67.3	64.8	69.3	79.7	96.5	95.9	90.2	81.0	81.5
Flush-toilet	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	79.1	79.4	79.7	73.9	67.0	73.2	85.2	87.4	85.6	86.5	79.6
	Case 3/1	87.0	87.4	87.0	80.7	78.9	83.3	91.9	91.0	91.1	92.2	86.9
Sweeping	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	100.0	100.0	100.0	93.3	83.0	83.2	83.9	84.6	78.8	71.9	83.9
	Case 3/1	100.0	100.0	100.0	93.3	89.1	88.9	90.0	89.1	85.1	79.0	88.8
Irrigation	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	Case 2/1	0	0	0	29.9	67.0	70.1	69.3	72.9	62.5	50.8	55.9
	Case 3/1	0	1.7	33.3	64.6	78.9	79.8	80.3	80.8	72.1	61.1	69.3
Average in class	Case 1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	
	Case 2/1	75.7	75.8	76.4	75.4	75.2	76.7	76.3	80.6	75.3	74.3	
	Case 3/1	83.6	84.4	83.8	82.5	83.5	83.1	84.0	83.4	83.4	83.3	

FIG. C-3-4
SUFFICING RATE FOR WATER DEMAND



This is because water demand for city irrigation has the higher income elasticity than for other purposes in all the income classes and its tendency becomes distinguished in the upper-income classes and also because both bathing and laundry water demands remarkably increase around the upper-middle income classes that constitutes the major part of the household sector in Nasr City. That the sufficing rates of bathing and laundry turn out to be low might be the negative factor that decreases the quality of urban life.

(2) Effect on Green Coverage Rate

Green coverage rate, green areas per capita and green areas in public gardens in each case are calculated as shown in Table C-3-5 on the basis of inter-relationship between the amount of water consumed for irrigation and green coverage rate.

These results clearly show that the green coverage rates of the housing parcel in the residential area are very small both in Case 2 and in Case 3.

This is because in the residential area the water consumption for irrigation by private housing sector (70%) does not come up to that of public housing sector (30%) on account of insufficient supply of water as well as low income level.

Green areas in public garden per capita show considerably high levels in all the cases in comparison with other countries.

Table C-3-5

Green Coverage Rate

	Case 1 %	Case 2 (a) %	Case 3 (b) %	Case 2/ Case 1 %	Case 3/ Case 1 %
Residential Area	33.3	24.8	18.7	74.6	56.2
Housing	35.0	22.1	18.2	63.0	52.0
Others	31.8	27.0	19.1	85.0	60.0
El Azhar University	45.0	35.5	27.0	79.0	60.0
Governmental	40.0	31.6	24.0	79.0	60.0
Public Garden	90.0	71.1	71.1	79.0	79.0
Greenbelt	100.0	0	0	0	0
Project Area	45.4	34.2*	28.1*	77.1*	63.4*

Note: * Excluding the greenbelt
(a) 16.2 m^3/ha quantity of water for irrigation.
(b) 11.0 m^3/ha quantity of water for irrigation.

	Case 1 m^2	Case 2 m^2	Case 3 m^2	Case 2/ Case 1 %	Case 3/ Case 1 %
Green Areas per Capita	46.2	32.7	26.9	70.8	58.2
Green Areas in Public Garden per Capita	17.5	11.0	10.4	62.9	59.4

(3) Effect on Air Pollution

One of the diffusion effects caused by green is its protection or decreasing effects on air pollutions. Considering Nasr City's characteristics of residential and governmental office areas, the main gaseous pollutant is carbon monoxide (CO) exhausted from automobiles.

As shown in Table C-3-6, CO density in the midst of the main roads in Nasr City and its decreasing rate to be measured at the edge of the main roads are estimated. This estimation relies on the function relating CO density to traffic (24 hours vehicles) that is formulated on the basis of data about CO density measured at several points in Cairo by National Research Center of Building and Construction in 1969 and data about daily automobile traffic for 24 hours at Cairo measured in 1965, and also on the estimation of traffic generation in Nasr City in 1985.

In respect to dust-form pollutants, the seizing effect caused by greens on dustfall is calculated and shown in Table C-3-7. As known from the table, the difference of the rates of seizing dustfall between Case 1 and Case 2 is only 2%, which, however, would lead to a considerably big difference in the improvement of urban life environment, taking into account the great amount of dustfall on Cairo affected by the sandstorm.

(4) Effect on Urban Climate

The effect on urban climate is another leading diffusion effect derived from greens. Particularly, the phenomena called "permeability phenomena" that the local climate of the higher humidity and the lower temperature presented at the large-scaled park (weeded green space) invades into the surrounding built-up urban area are known as important.

The "permeability phenomena" as to temperature caused by the public garden adjoining to 7th and 8th residential zones are estimated and shown in Table C-3-8.

Tab. C-3-6. Air Pollution CO Dencity

	Trend 1985	Case 1	Case 2	Case 3	Case 2/ Case 1 %	Case 3/ Case 1 %
CO Density at Main Road ppm	7.4 (100)	5.7 (76.5)	6.2 (84.0)	6.6 (88.5)	109.8	115.7
Under Density at the Main Road in 1975 = 1	4.3	3.3	3.6	3.8	109.8	115.7
Decreasing Rate %	-	23.5	16.0	11.5	68.1	48.9

Tab. C-3-7. Dustfall

	Average 1960	Case 1	Case 2	Case 3	Case 2/ Case 1	Case 3/ Case 1
Dustfall (t/sq.mile)	81.3 (100)	74.0 (91.0)	75.6 (93.0)	76.4 (94.0)	102.2	103.3
Seizing Rate Dustfall		7.3 (9.0)	5.7 (7.0)	4.9 (6.0)	78.1	67.1

Tab. C-3-8. Temperature Decreasing Effect (permeability phenomena)*

	Case 1	Case 2/ Case 3		Case 1	Case 2/ Case 3
Distance of Decreasing Effect till 0.2°C	180 (100)	169 (93.9)	Area of Decreasing Effect till 0.2°C	43.7 (100)	40.9 (93.6)
-ditto- 0.5°C	100 (100)	96 (96.0)	-ditto- 0.5°C	23.5 (100)	22.5 (95.7)
-ditto- 1°C	65 (100)	63 (96.9)	-ditto- 1°C	15.1 (100)	14.6 (96.7)

Note: * Using the relationship in Japanese cases on the condition that, in summer at 2 PM, bulkrate of built up area is below 0.3 and wind velocity is 3-5 m/sec. and blows from south.

Other promising diffusion effects derived from greens which can not calculated this time by the limitation of available data might be the following ones; decreasing effect on noise, decreasing effect on occurrence rate of respiratory disease and ophthalmological disease, in Japanese case the negative correlation between the green coverage rate and the occurrence rate of misdemeanour is also indicated.

The above calculated results are summarized into Table C-3-9 from the viewpoint to judge how much desirable effects Case 1 (Nasr City Raw Water Supply Project) brings about in comparison with Case 2 and Case 3.

As known from the table, Nasr City Raw Water Supply Project can be judged to have a great deal of desirable effects on the listed aspects. With due regard to so-called synergy between these effects, like leaving room of the gap between water supply and water demand would make possible to utilize water for the scavengery purpose, and furthermore in cooperation with the seizing effect on dustfall it would attain to keep the road being much more "clean liness", Nasr City Raw Water Supply Project can be judged to make Nasr City as a residential area by greatly appreciative in respect to its "visual satisfaction on greens", "cleanliness" and "silence".

Finally, provided that the state of human life system which Raw Water Supply Project would achieve in Nasr City is the social goal founded on the social value assumed in Egypt, the Project should be evaluated as one of the alternative means to achieve the social goal because the Project is not the only possible means to achieve the social goal.

Considering Treated Water Irrigation case which has only treated water supply system as another alternative means to attain the social goal besides the Raw Water Supply Project which incorporate the raw water supply system with the treated water supply system, the notion of social cost will be applied to the appraisal for selecting the appropriate one among the above mentioned two alternative policies.

Social cost can be defined briefly as the difference between the

Tab. C-3-9. Summary of Calculated Results

	Case 1/Case 2	Case 1/Case 3	Case 2/Case 3
Sufficing Rate for Water Demand	1.36	1.20	0.89
Green Coverage Rate	1.33	1.62	1.22
Green Areas per Capita	1.41	1.72	1.22
Green Areas in Public Garden per Capita	1.59	1.68	1.06
Decreasing Rate of CO Density	1.47	2.04	1.39
Decreasing Rate of Dustfall	1.28	1.49	1.16
Areas Affected to Decrease Temperature 0.2°C	1.07	1.07	1.00
-ditto- 0.5°C	1.04	1.04	1.00
-ditto- 1.0°C	1.03	1.03	1.00

cost which the optimal means achieving a social goal takes and the cost which the other means achieving the same goal. The optimal means, however, is usually unknown, so the methodology that regards as social cost the cost deviance between the means to be considered as second-best and the other one could be thought. This is the indirect method to measure social cost or social benefit which shall be employed this time. According to this method, the social cost of the other means can be thought as the social benefit of the means to be considered as second-best in the sense that the added cost which would have generated if the other means had employed would be saved by employing the means to be considered as second-best. So the formulation of the problem shall be as follows;

Considering Raw Water Supply Project as the second-best means, to calculate the social cost of Treated Water Splinkling Case and then to compare it with the total cost of Raw W. ter Supply Project. The social cost of Treated Water Splinkling Case can be said to consist of the following two terms; (1) the added cost caused by sprinkling treated water of the higher production cost instead of raw water. (2) Without making the treated water price congruent with the raw water price the quantity of treated water to be splinkled would not come up to the quantity of raw water to be splinkled because of the price elasticity of water demand. This implies that the price of treated water must be set considerably lower than its production cost. Consequently it would lead to loss caused by dèviation from the appropriate water use pattern (e.g. waste of water) particularly in other purposes than sprinkling. --- the loss derived from waste of water.

(1) $(2,5-0.5) \times 66,456 \times 365 \times [\sum_{i=1}^{30} \frac{1}{(1+0.05)^i}] = 7,458,000$ LE (US$ 19,370,000)

2.5 PT/cu.m	:	Production cost of treated water
0.5 PT/cu.m	:	Production cost of raw water
0.05	:	Discount rate
66,456 cu.m/day	:	Quantity of water to be splinkled

(2) $0.2 \times 2.5 \times (66{,}456+60{,}800) \times 365 \left[\sum_{i=1}^{30} \frac{1}{(1+0.05)^i}\right] = 3{,}570{,}000$ L.E
(US$9,273,000)

60,800 cu.m/day : Quantity of water consumed for domestic purpose

0.2 : The rate of loss simplified

From the simplified calculations considering the total amount of water to be consumed as constant for coming thirty years, the social cost of Treated Water Irrigation Case amounts to US$28,643,000.

The total cost of raw water supply Project consists of (1) Construction cost at the initial time and (2) operation and maintenance cost.

(1) US$16,525,000

(2) US$580 x 365 x $\left[\sum_{i=1}^{30} \frac{1}{(1+0.05)^i}\right]$ = US$3,254,000

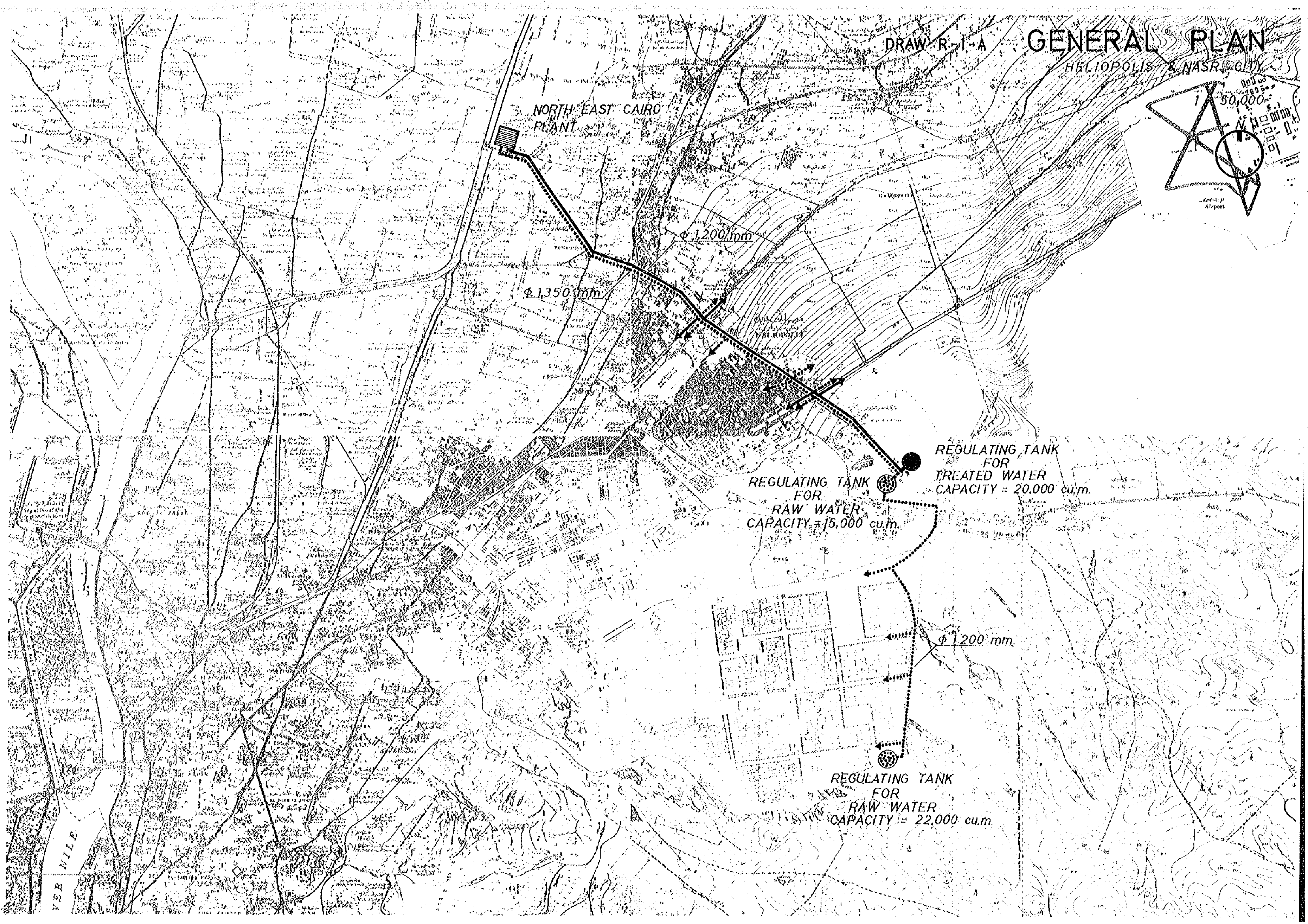
US$580 : Operation and Maintenance cost a day

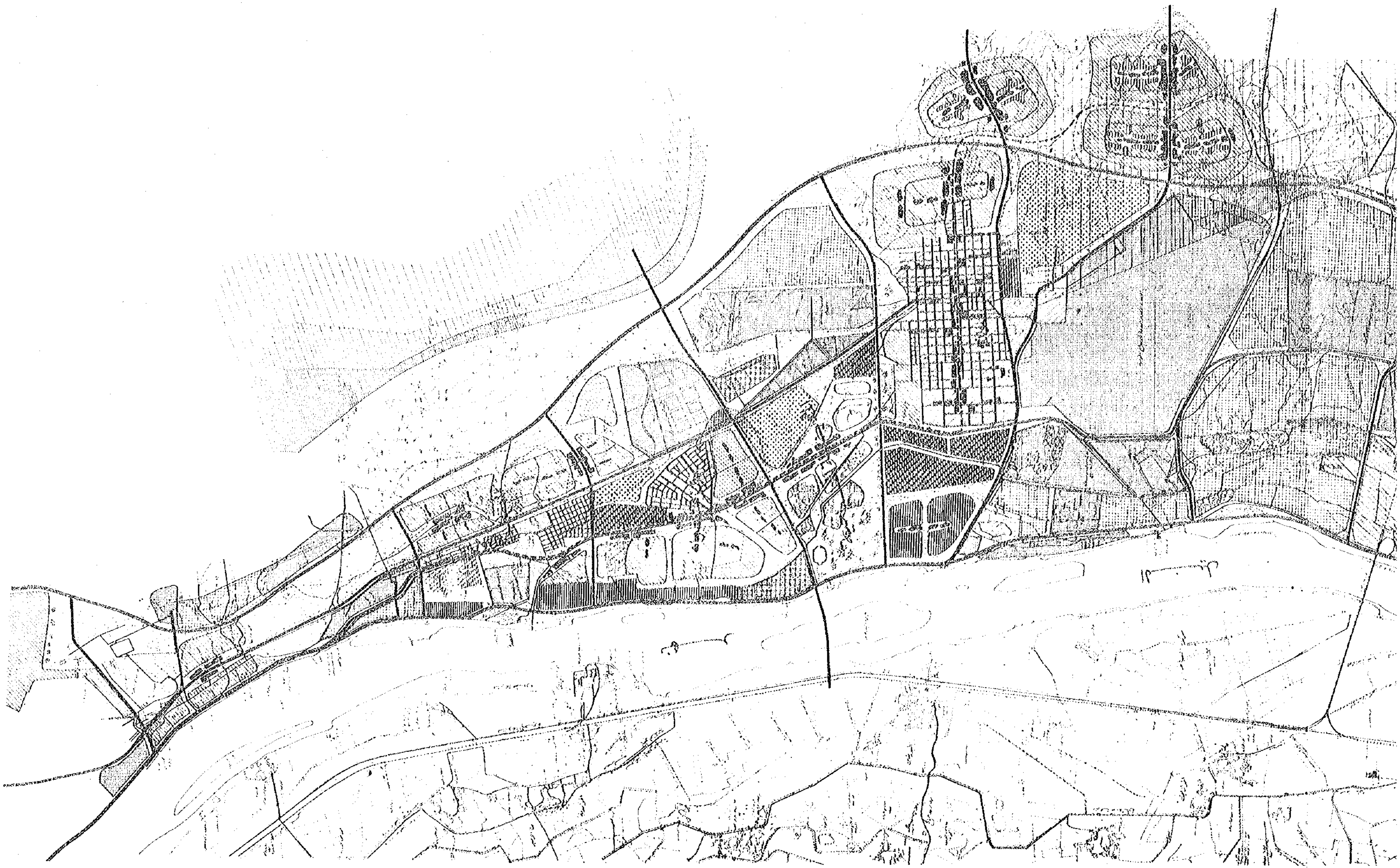
The total cost of Raw Water Supply Project amounts only to US$19,779,000.

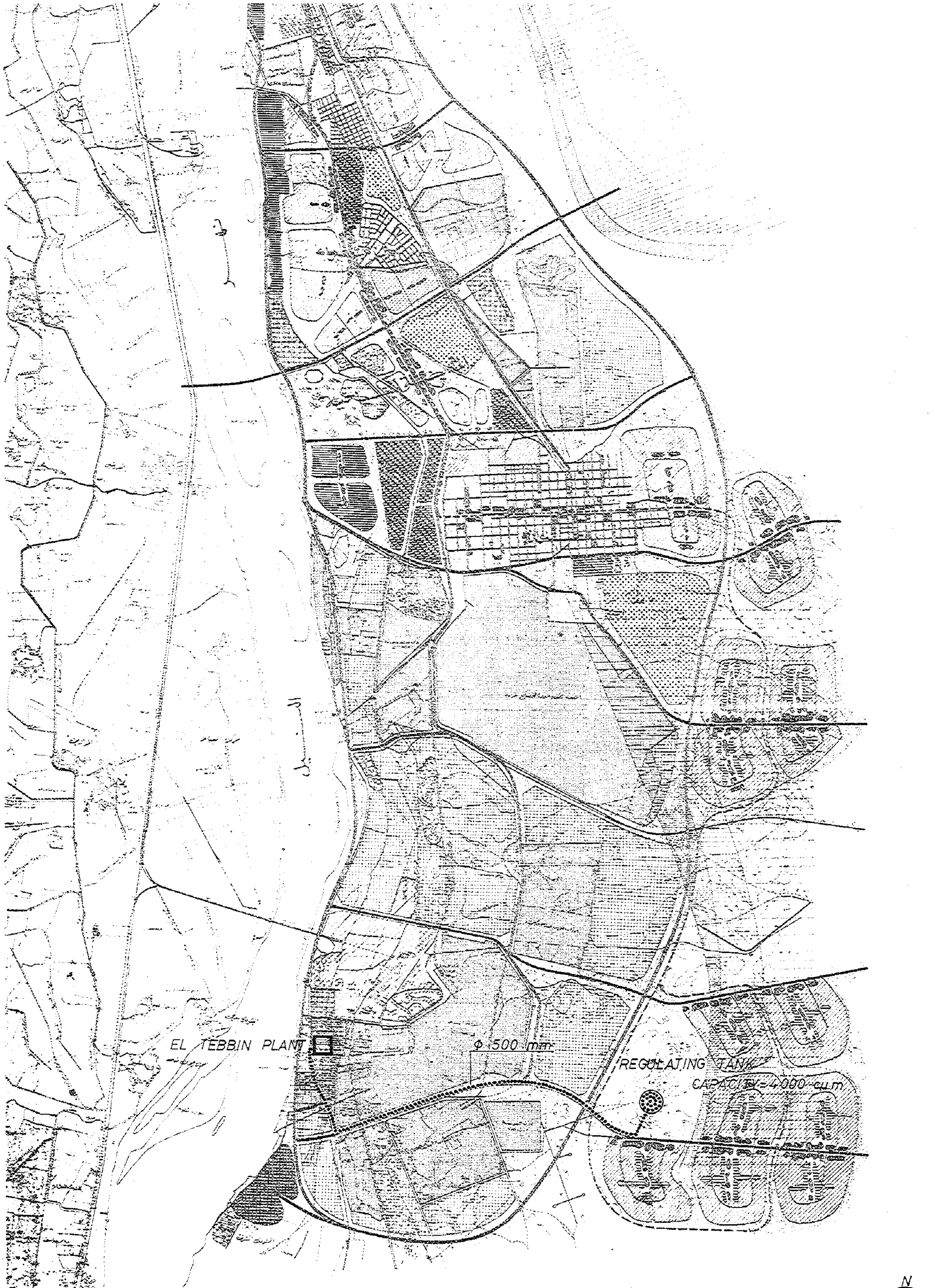
From these results, that show the social cost of Treated Water Irrigation Case exceed the total cost of Raw Water Supply Project, Raw Water Supply Project is judged to have enough relevancy to be selected instead of Treated Water Irrigation Case.

From the viewpoint of "social cost", the solution which incorporates the raw water supply system with the treated water supply system in order to achieve the social goal of afforestation is judged to have much more relevance than the solution which introduces the treated water supply system only.

APPENDIX C-4 DRAWINGS


DRAW. R-1-A
GENERAL PLAN
HELIOPOLIS & NASR CITY
1/50,000
NORTH EAST CAIRO PLANT
Φ 1,200 mm
Φ 1,350 mm
REGULATING TANK FOR TREATED WATER CAPACITY = 20,000 cu.m.
REGULATING TANK FOR RAW WATER CAPACITY = 15,000 cu.m.
Φ 1,200 mm.
REGULATING TANK FOR RAW WATER CAPACITY = 22,000 cu.m.
NILE



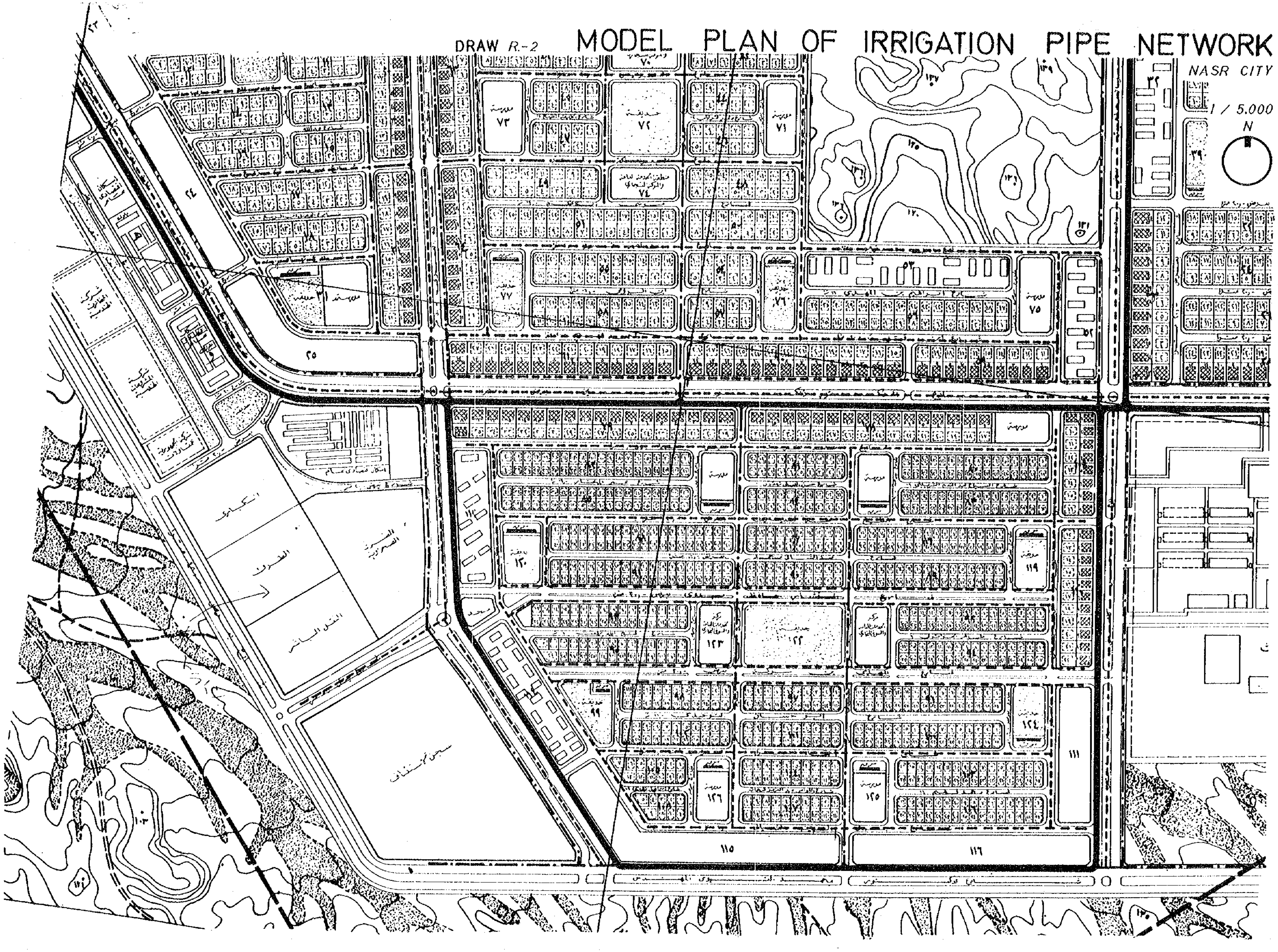


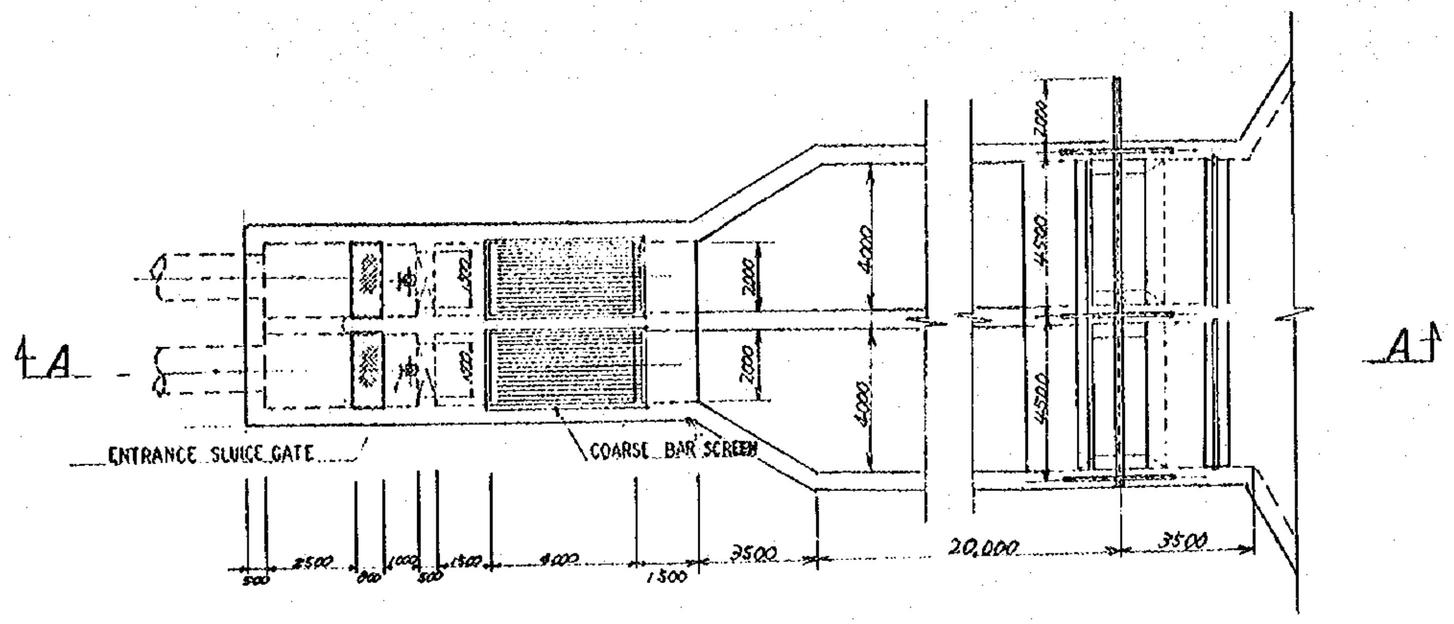
HELWAN

DRAW R-1-B

GENERAL PLAN

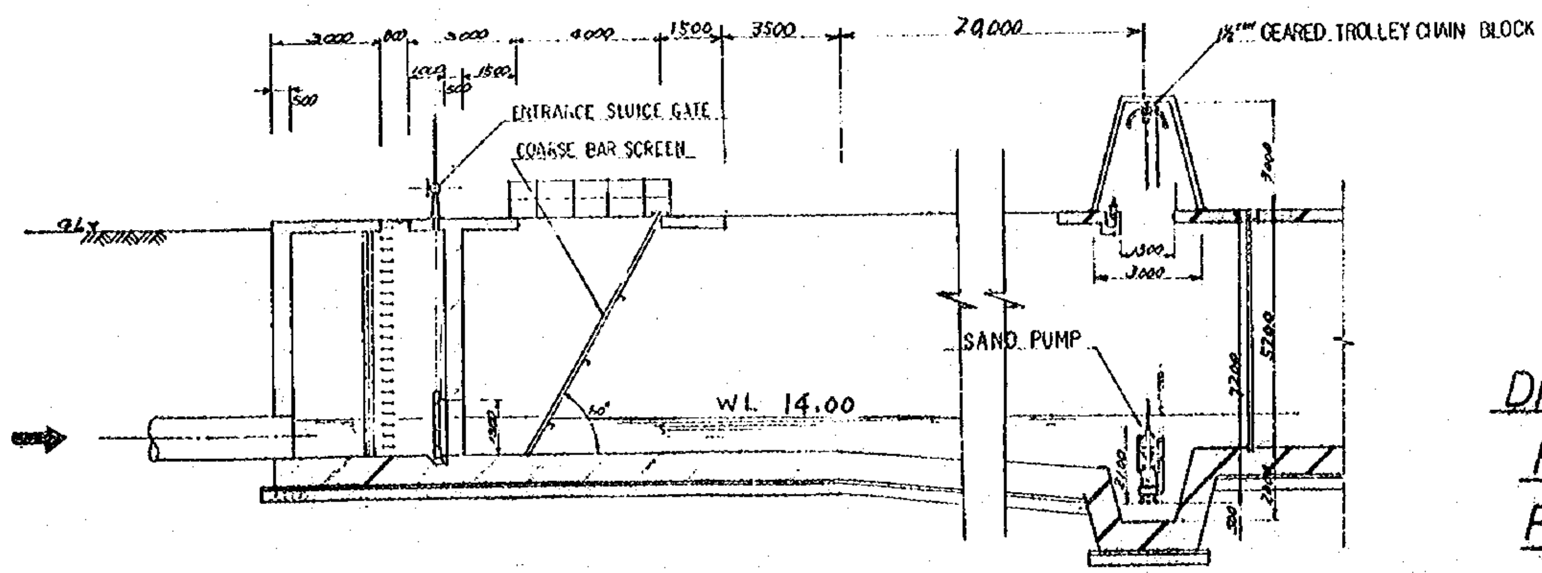
DRAW R-2 MODEL PLAN OF IRRIGATION PIPE NETWORK





PLAN S=1/200

A—A SECTION S=1/200

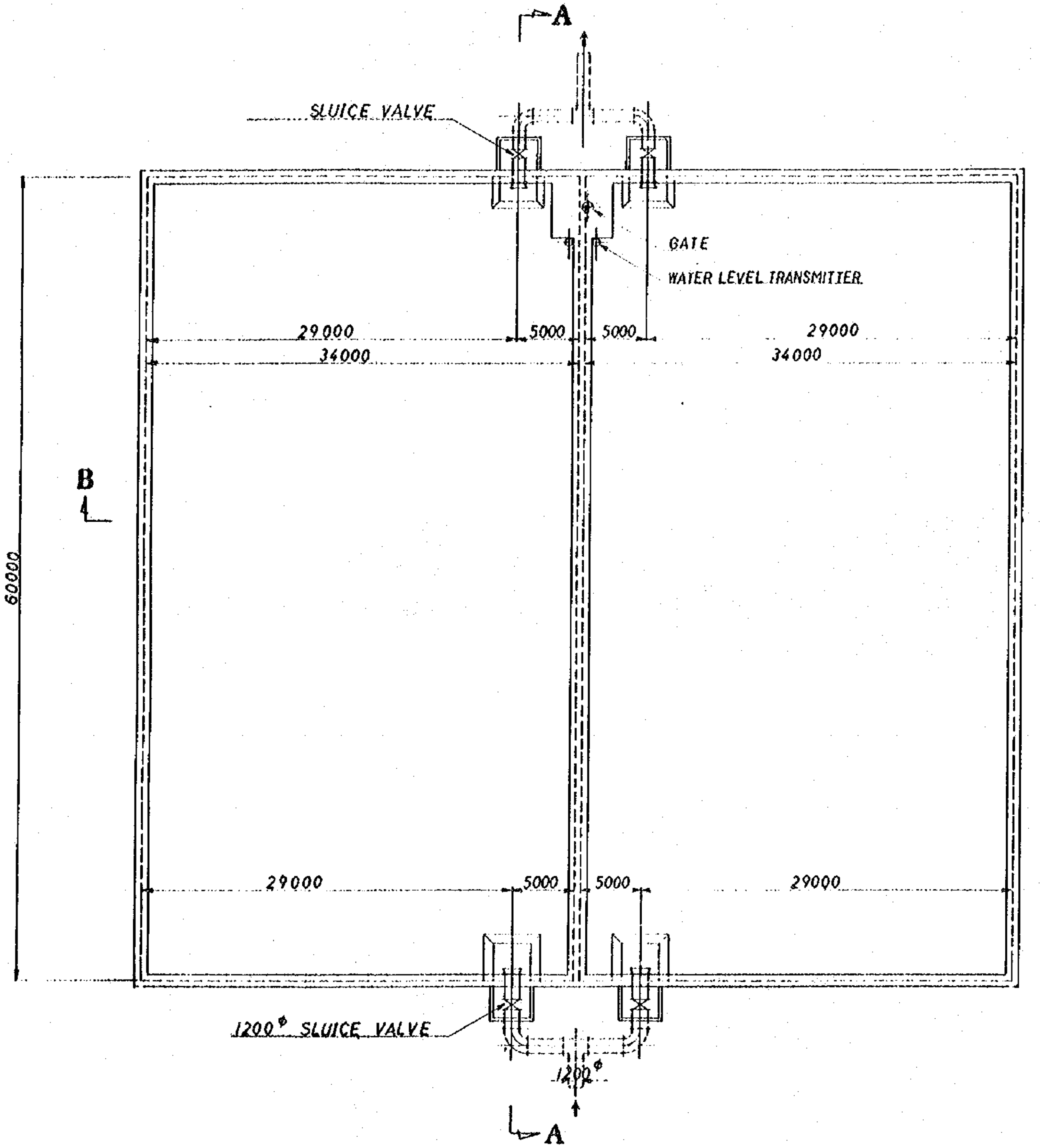


DRAW—R—3

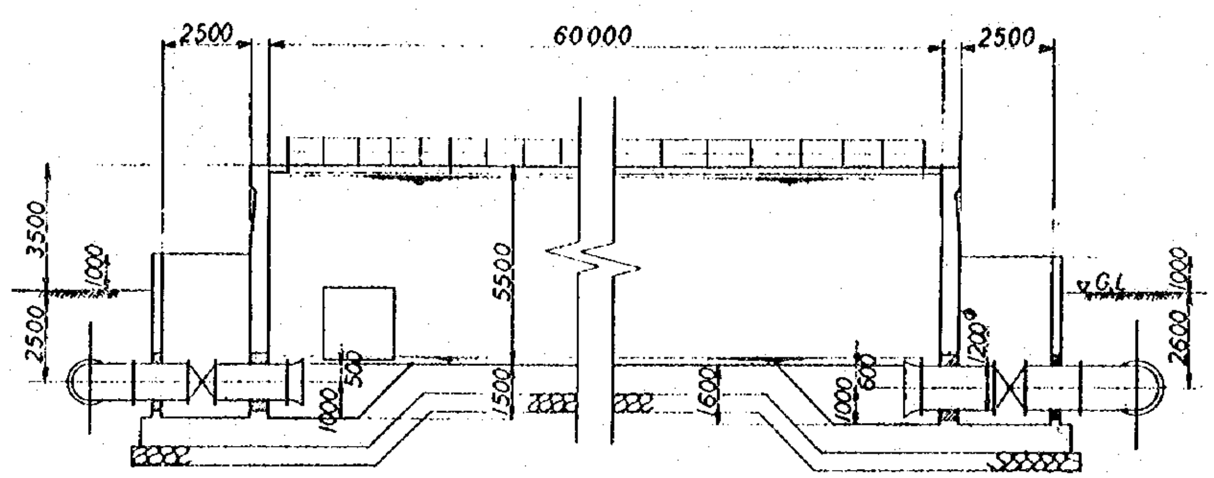
PRELIMINARY DESING OF INTAKE

FOR RAW WATER IN NORTH EAST CAIRO STATION

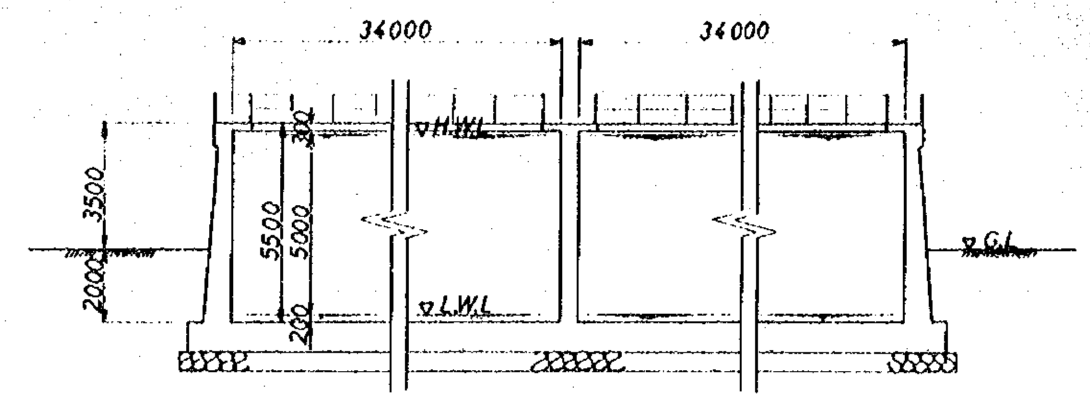
PLAN S:1/300



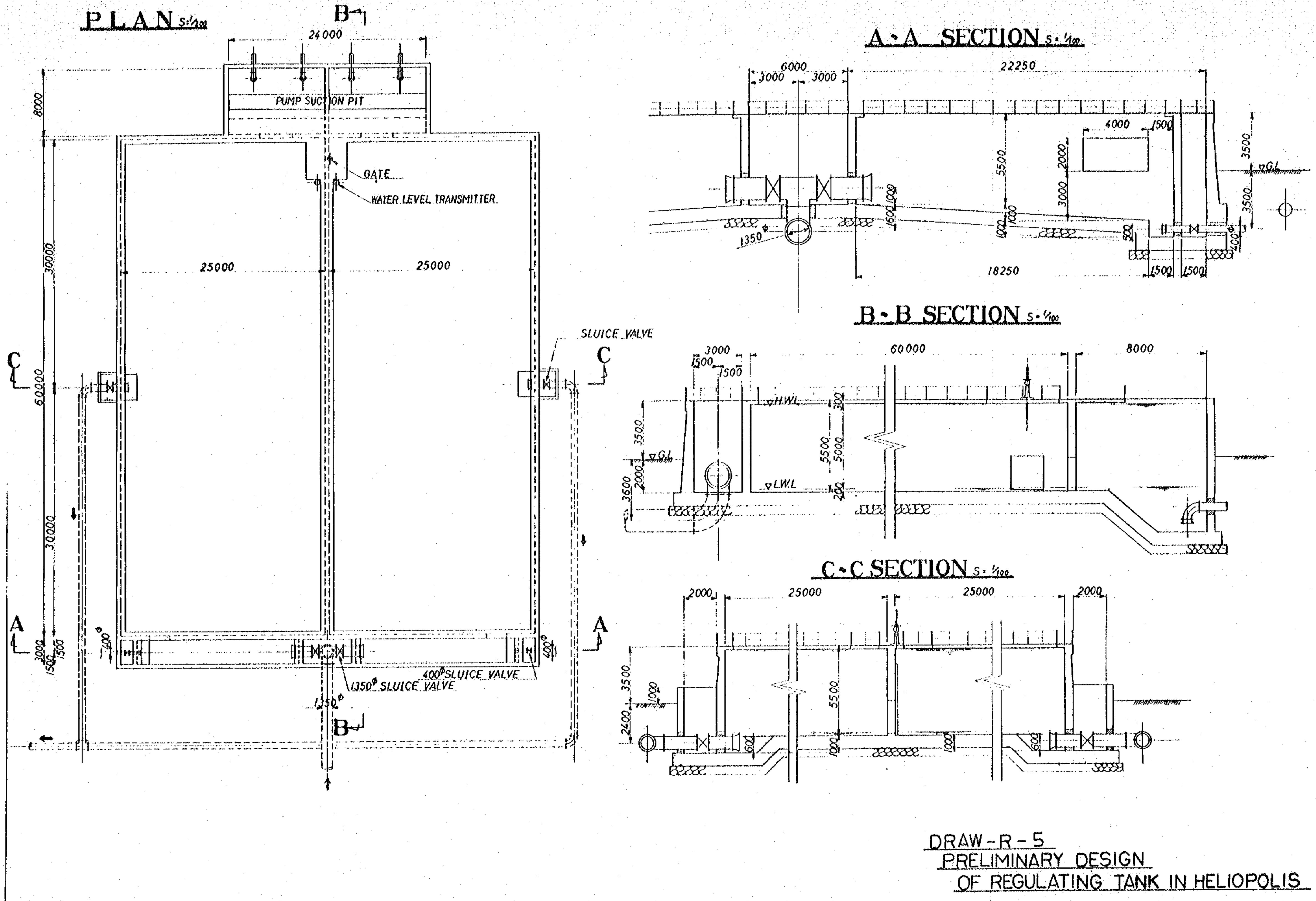
A~A SECTION S:1/100



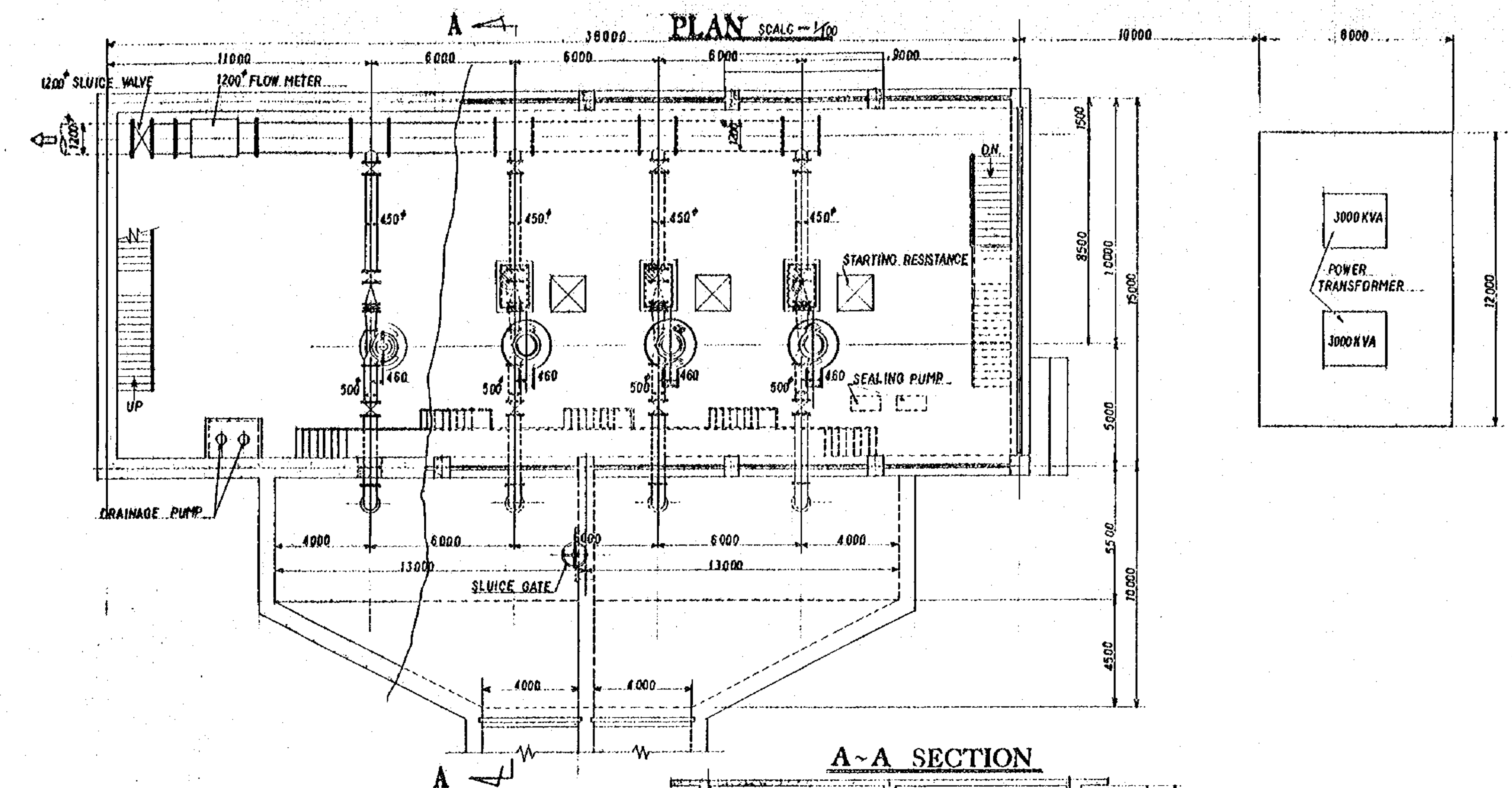
B~B SECTION S:1/100

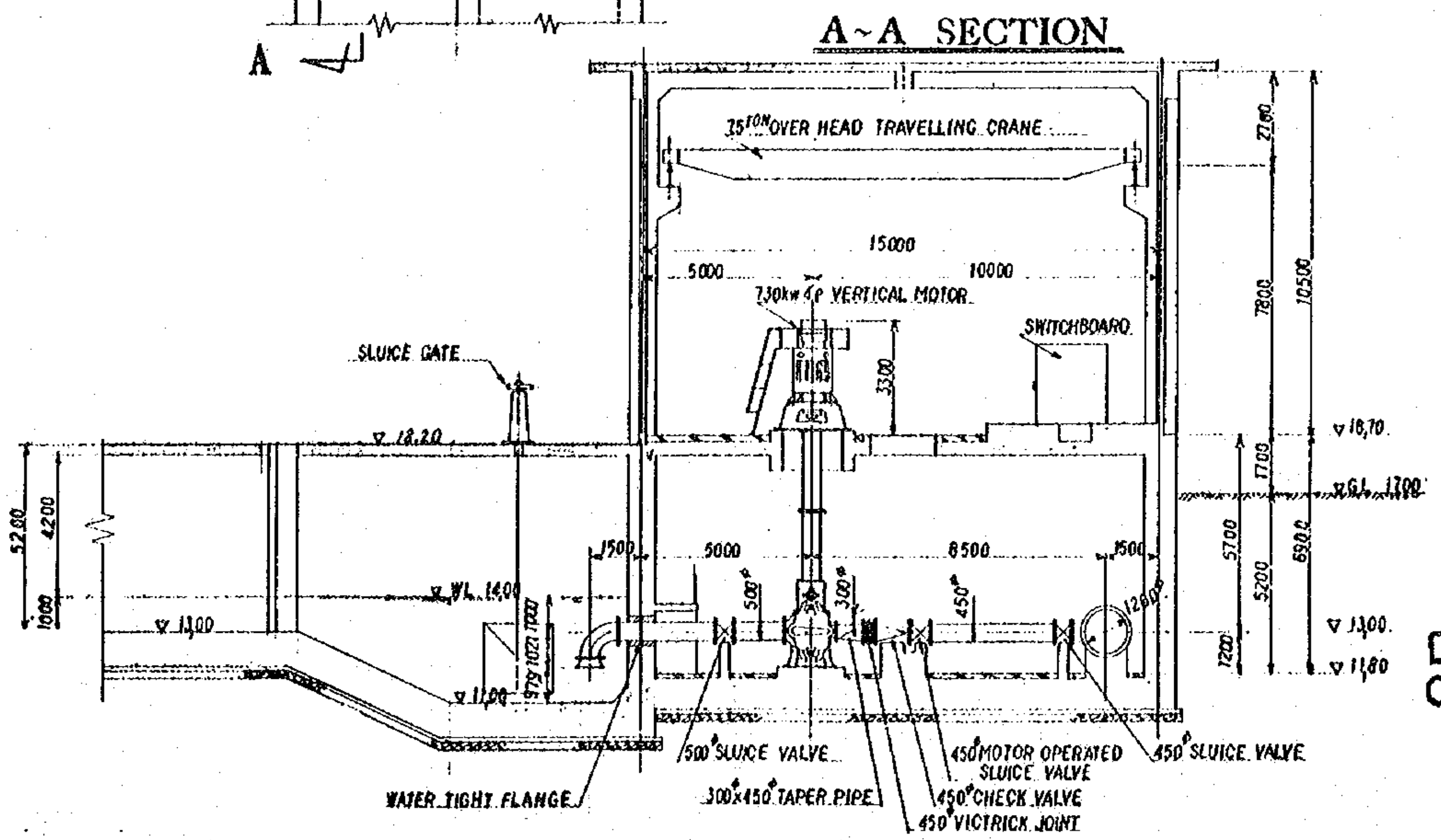


DRAW-R-4
PRELIMINARY DESIGN
OF REGULATING TANK IN NASR CITY



DRAW-R-5
PRELIMINARY DESIGN
OF REGULATING TANK IN HELIOPOLIS

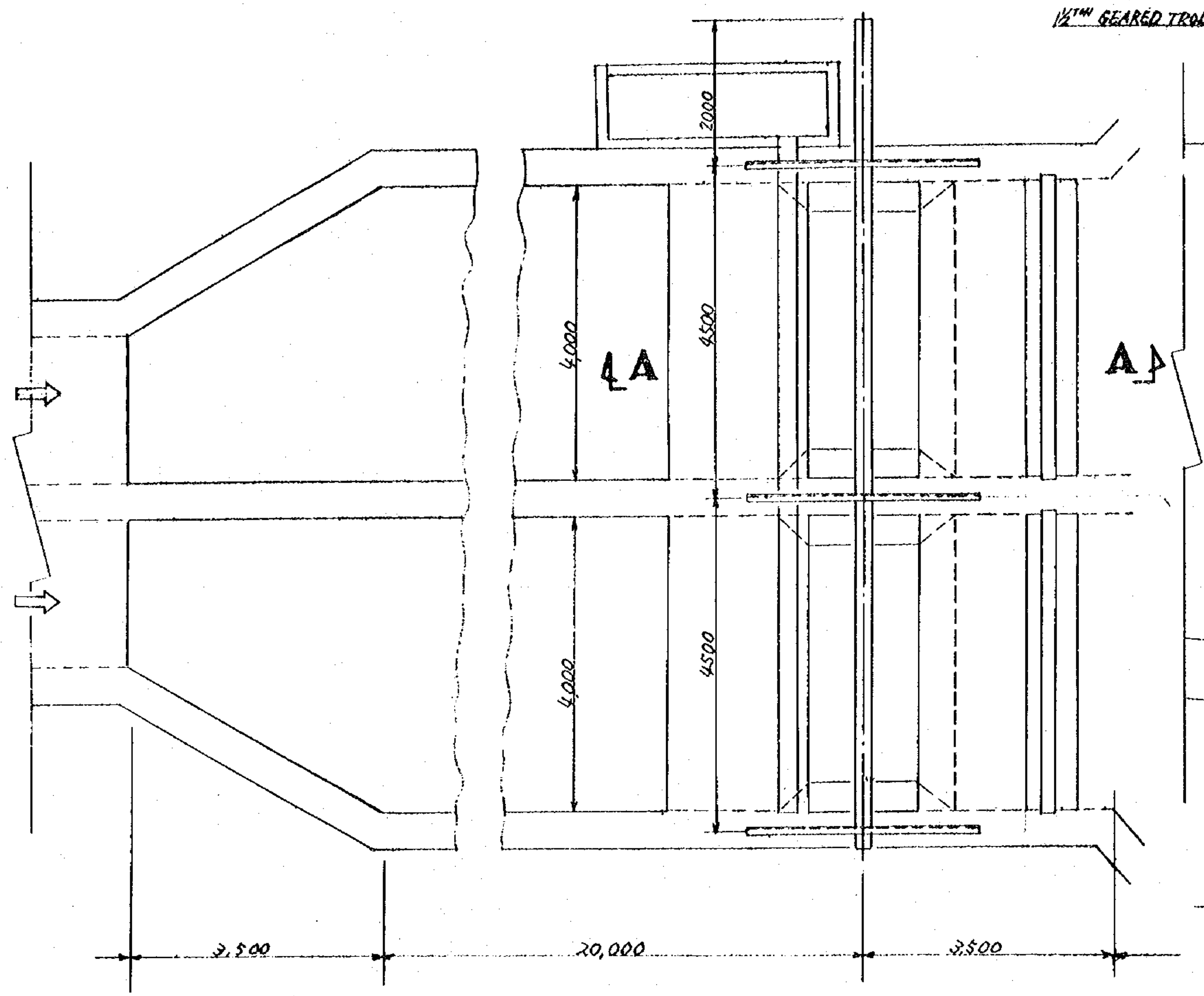




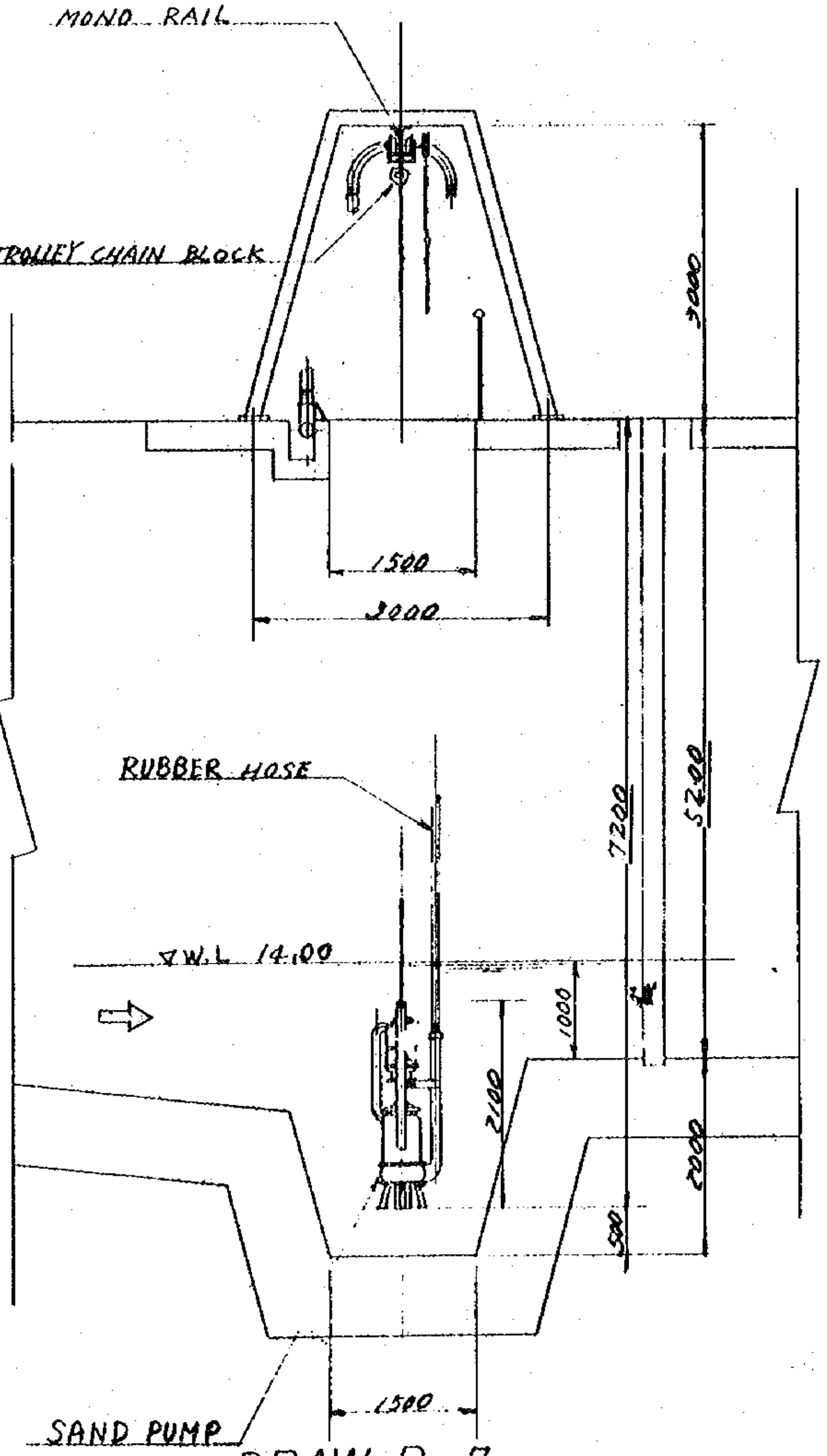
SPECIFICATION			
PUMP		MOTOR	
TYPE	VERTICAL VOLUTE PUMP	TYPE	GUARDED DRIP-PROOF WOUND-ROTOR
SUCTION DIA	500 MM		
DELIVERY DIA	300 MM	OUT PUT	730 KW
TOTAL HEAD	100 M	VOLTAGE	6300 V
CAPACITY	310 M³/MIN	CYCLES	50 HZ
NO. OF STAGE	1	NO OF POLES	4 P
SPEED	1450 RPM	SPEED	1500 R.P.M
QUANTITY	4 SET	QUANTITY	4 SET

DRAW-R-6 PRELIMINARY DESIGN
OF PUMP STATION FOR NORTH EAST CAIRO

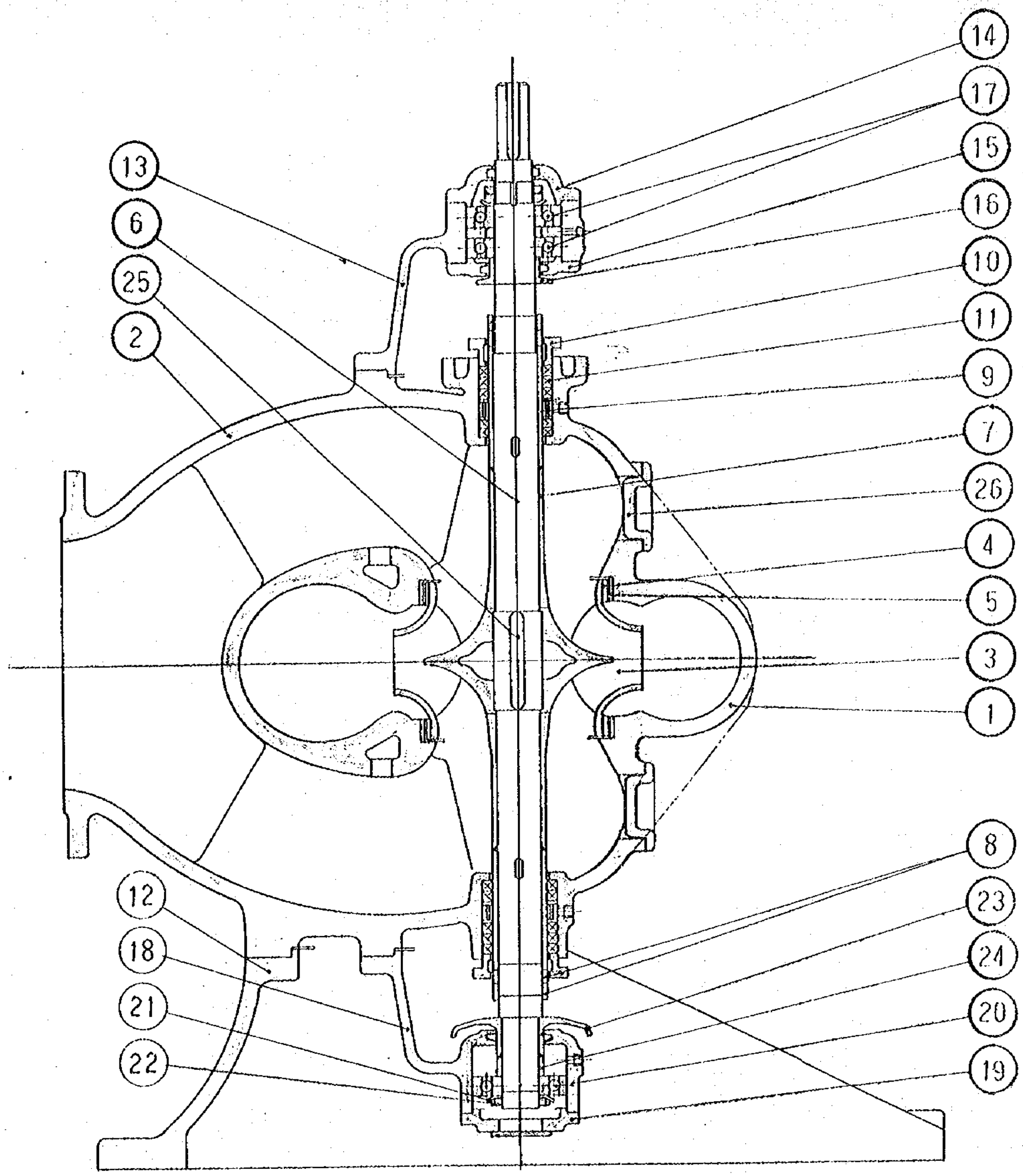
PLAN



A·A SECTION



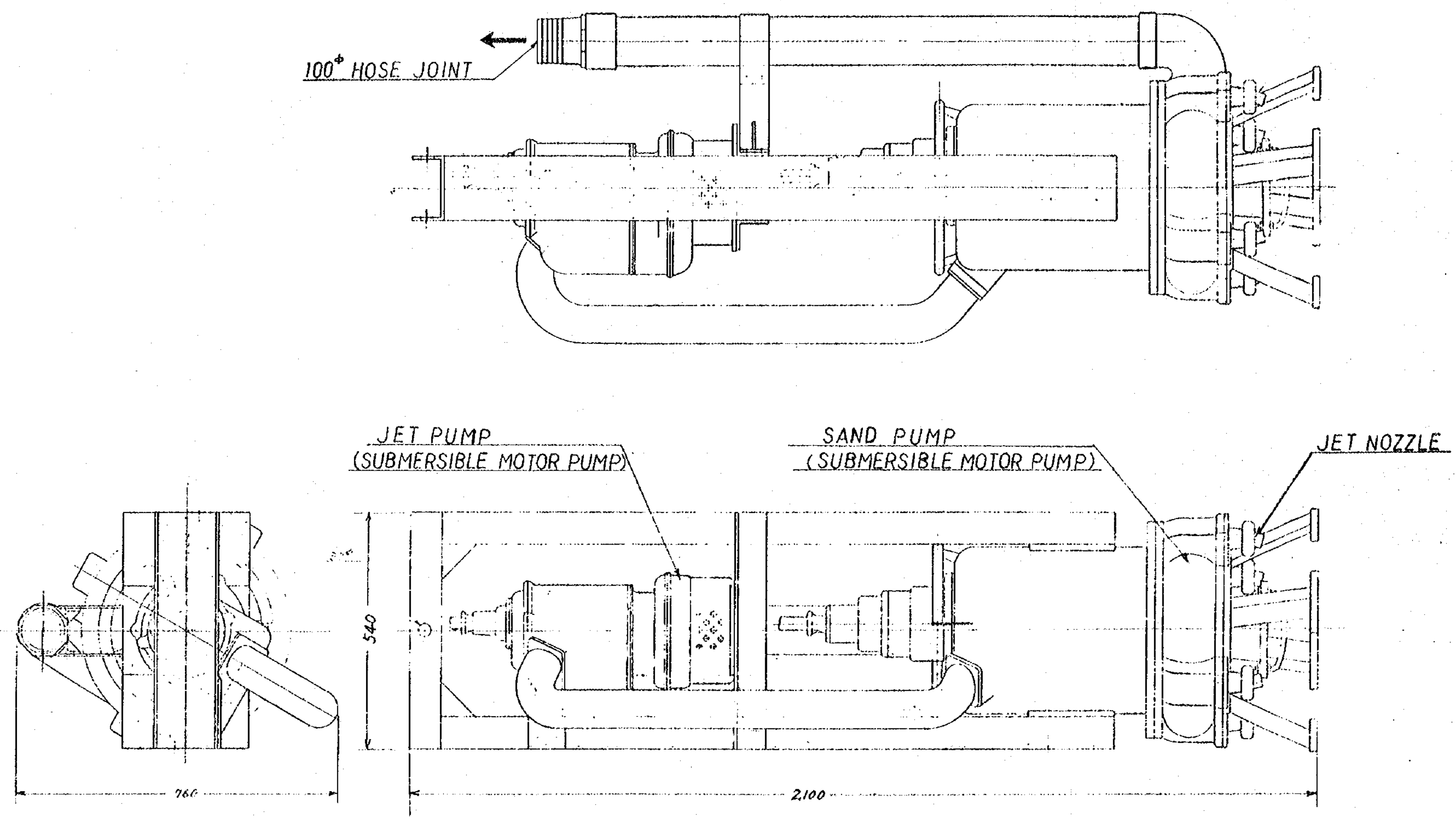
DRAW-R-7
MODEL OF SEDIMENTATION BASIN
FOR NORTH EAST CAIRO STATION



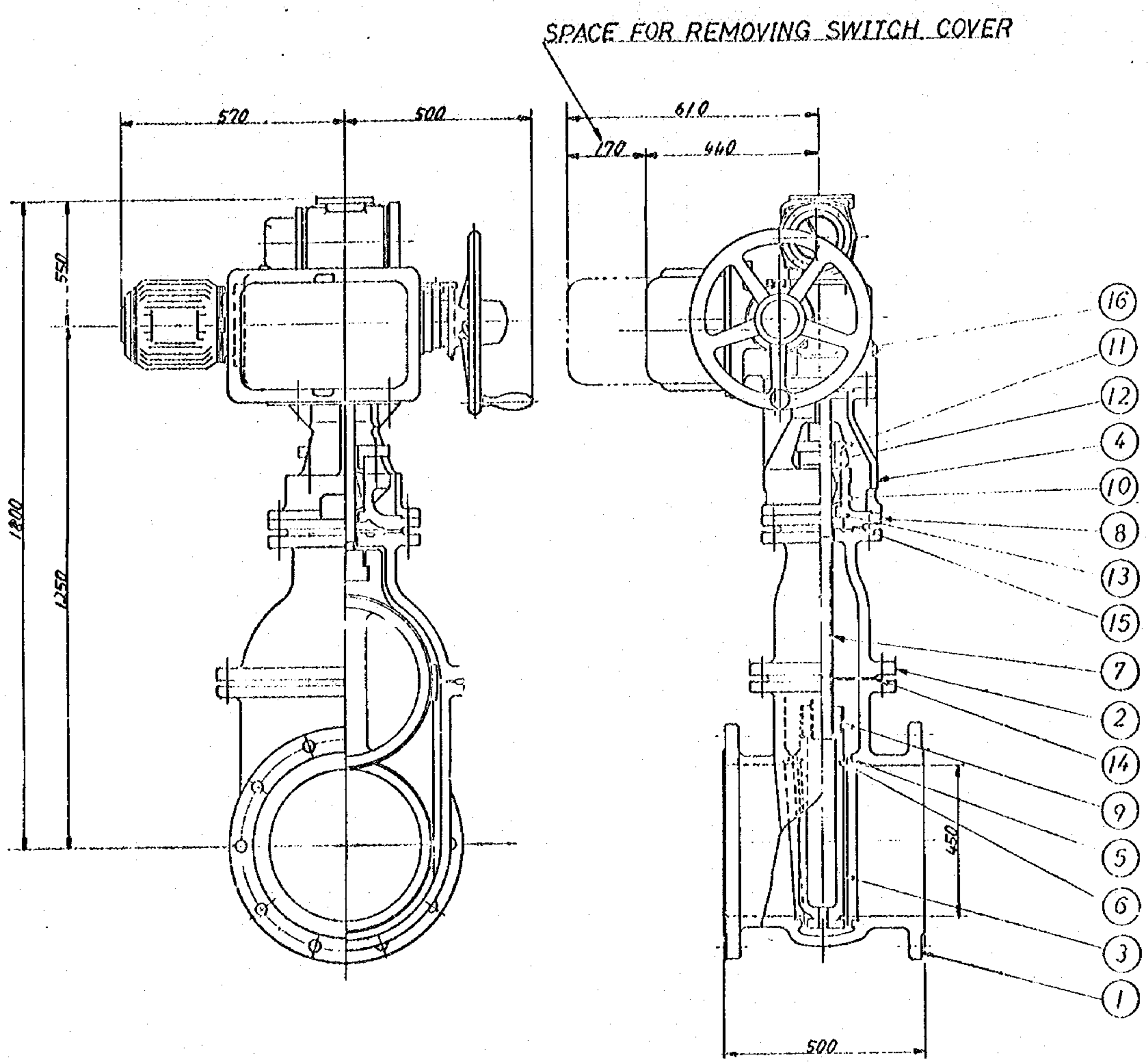
NO	NAME	MATERIAL
1	UPPER CASING	CAST IRON
2	LOWER CASING	CAST IRON
3	IMPELLER	STAINLEES CAST STEEL
4	IMPELLER LINER RING	STAINLEES CAST STEEL
5	LINER RING	STAINLEES CAST STEEL
6	MAIN SHAFT	CARBON STEEL
7	SLEEVE	STAINLEES STEEL
8	SLEEVE NUTS	STAINLEES STEEL
9	LANTERN RING	BRONZE
10	GLAND	CAST IRON
11	GLAND PACKING	GRAPHITE
12	PUMP BED	CAST IRON
13	BRACKET (1)	CAST IRON
14	BEARING COVER (1)	CAST IRON
15	BEARING COVER (2)	CAST IRON
16	WATER SLINGER (1)	BRONZE
17	RADIAL BALL BEARING	
18	BRACKET (2)	CAST IRON
19	BEARING COVER	CAST IRON
20	RADIAL BALL BEARING	
21	WASHER	STEEL
22	LOCK NUT	STEEL
23	WATER SLINGER (2)	BRONZE
24	DISTANCE RING	BRONZE
25	IMPELLER KEY	STEEL
26	INSPECTION COVER	CAST IRON

DRAW-R-8

STANDERD DESIGN OF INTAKE PUMP



DRAW - R - 9 SUND PUMP

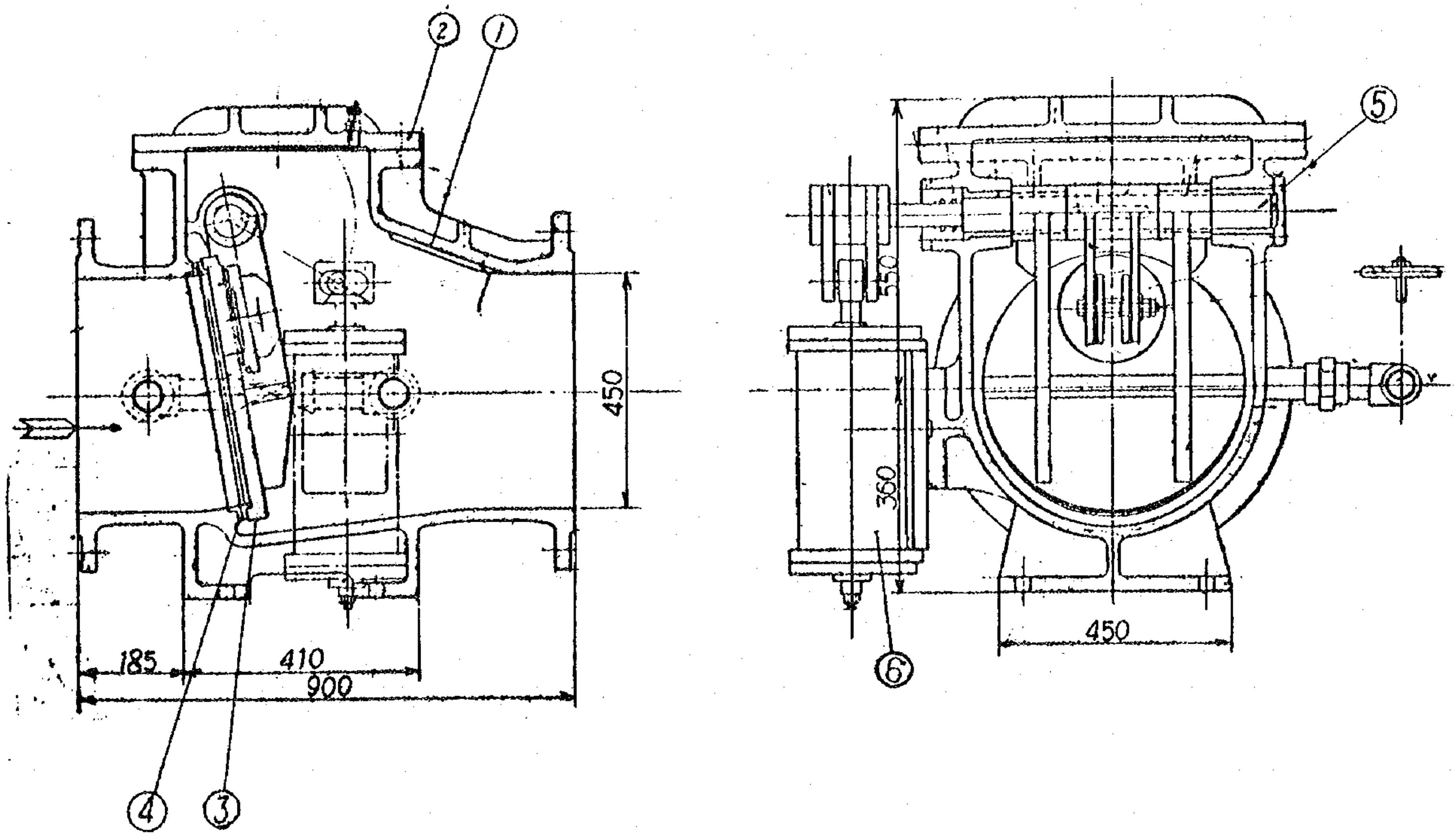


SYM	PARTS NAME
16	CONTROLLER
15	GASKET
14	GASKET
13	BUSH
12	BUSH
11	PACKING GLAND
10	PACKING
9	RISING SCREW BLOCK
8	PACKING BOX
7	VALVE SPINDLE
6	VALV SEAT ON VALVE DISC
5	VALV SEAT ON VALVE BODY
4	STAND
3	VALVE DISC
2	BONNET
1	VALVE BODY

DRAW-R-10

STANDERD DESIGN

OF ELECTRIC REGULATING VALVE

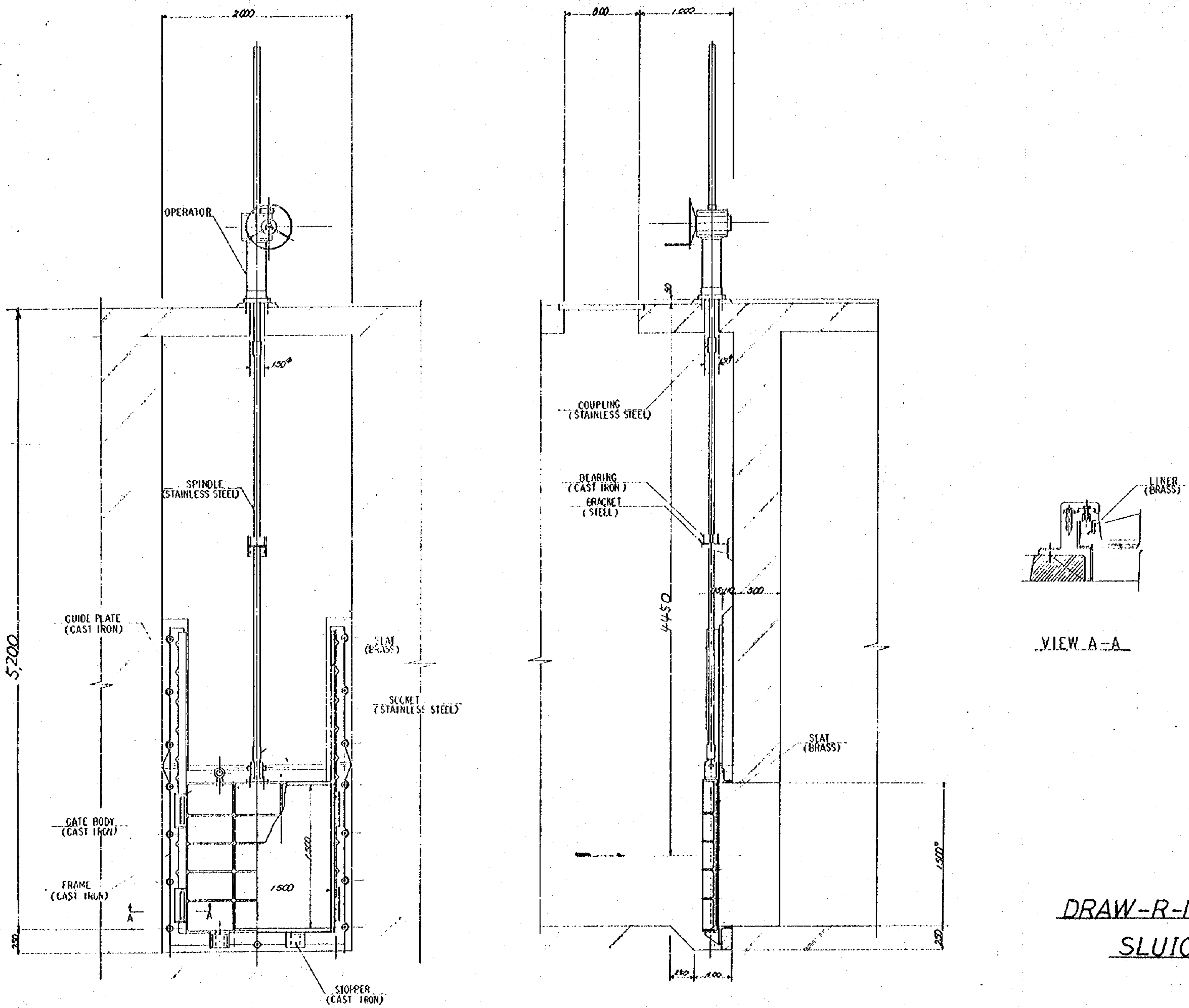


NO	NAME
6	OIL DASH POT
5	SPINDLE
4	SEAT
3	DISC
2	COVER
1	BODY

DRAW-R-11

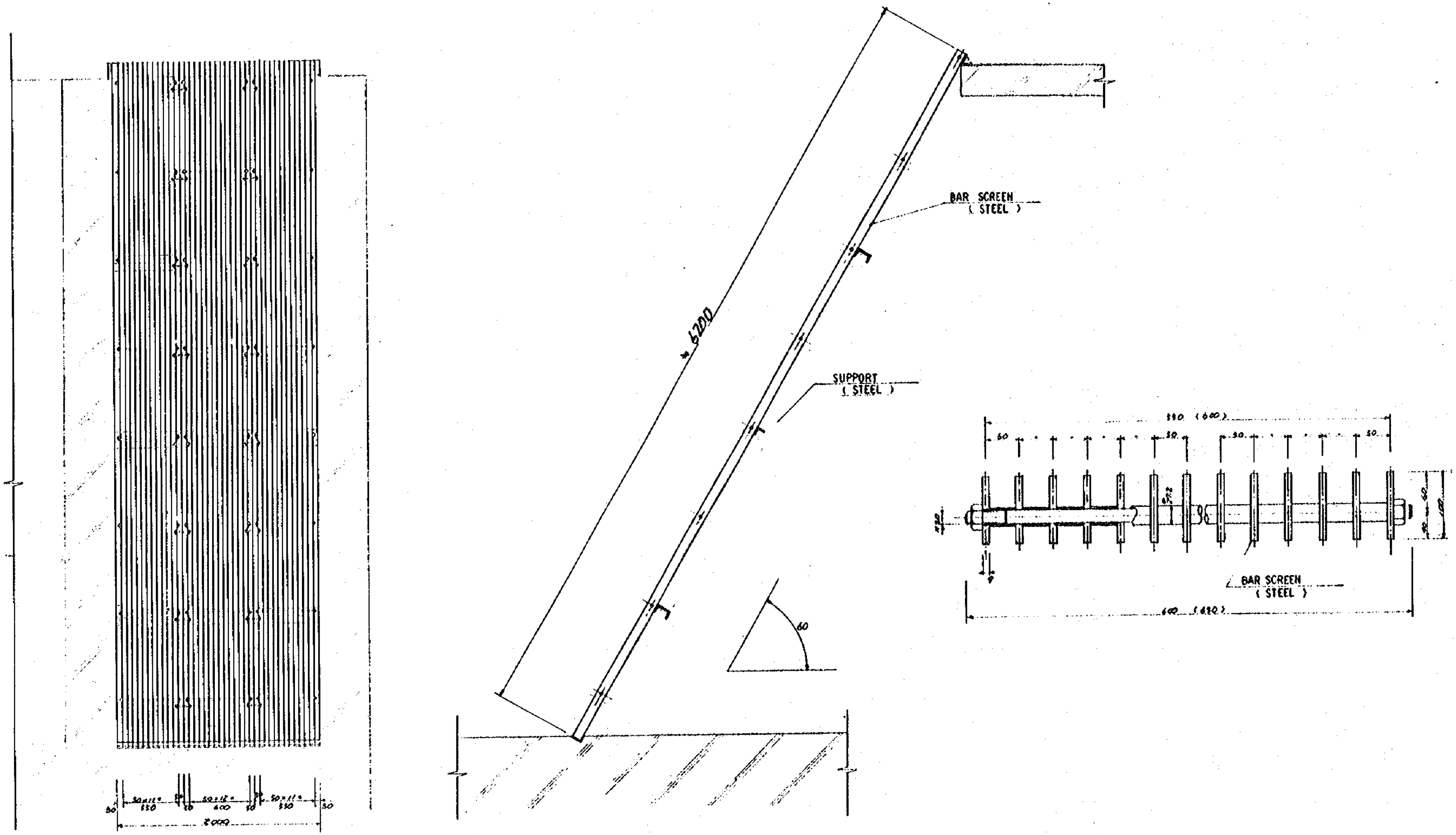
STANDERD DESIGN

OF SLOW CLOSING CHECK VALVE



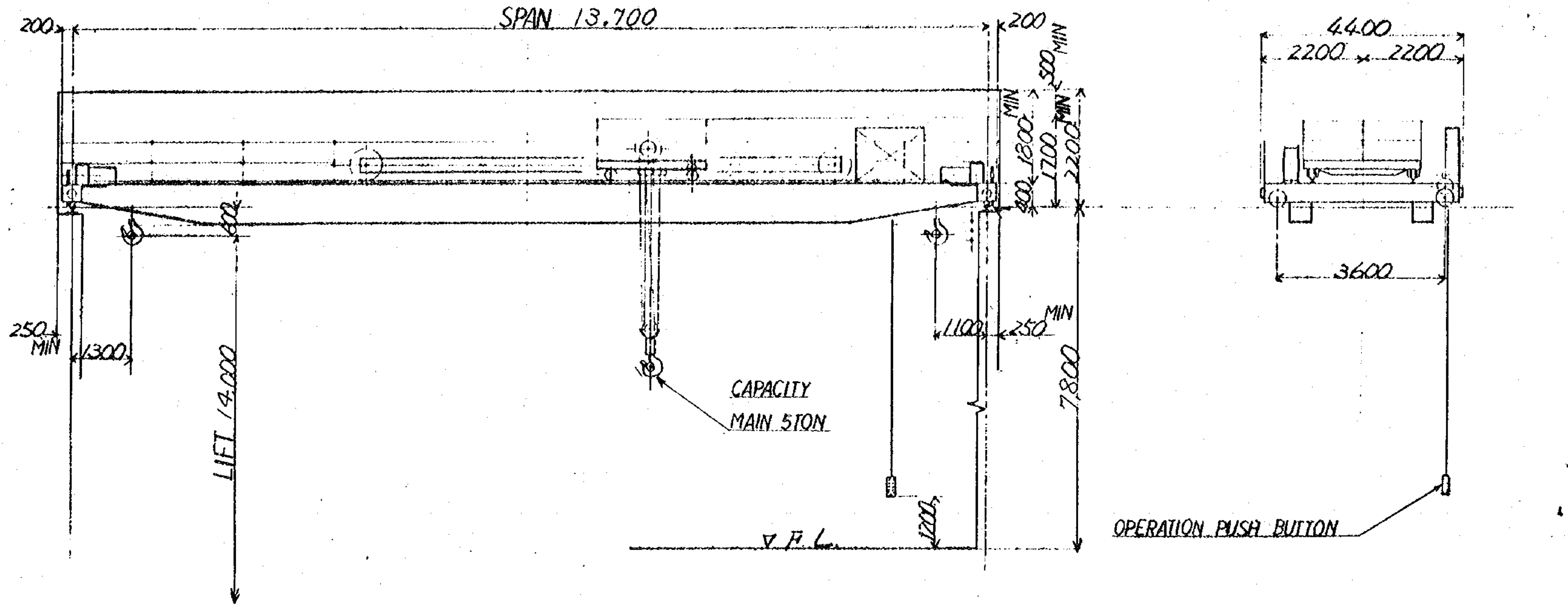
DRAW-R-12

SLUICE GATE



DRAW-R-13

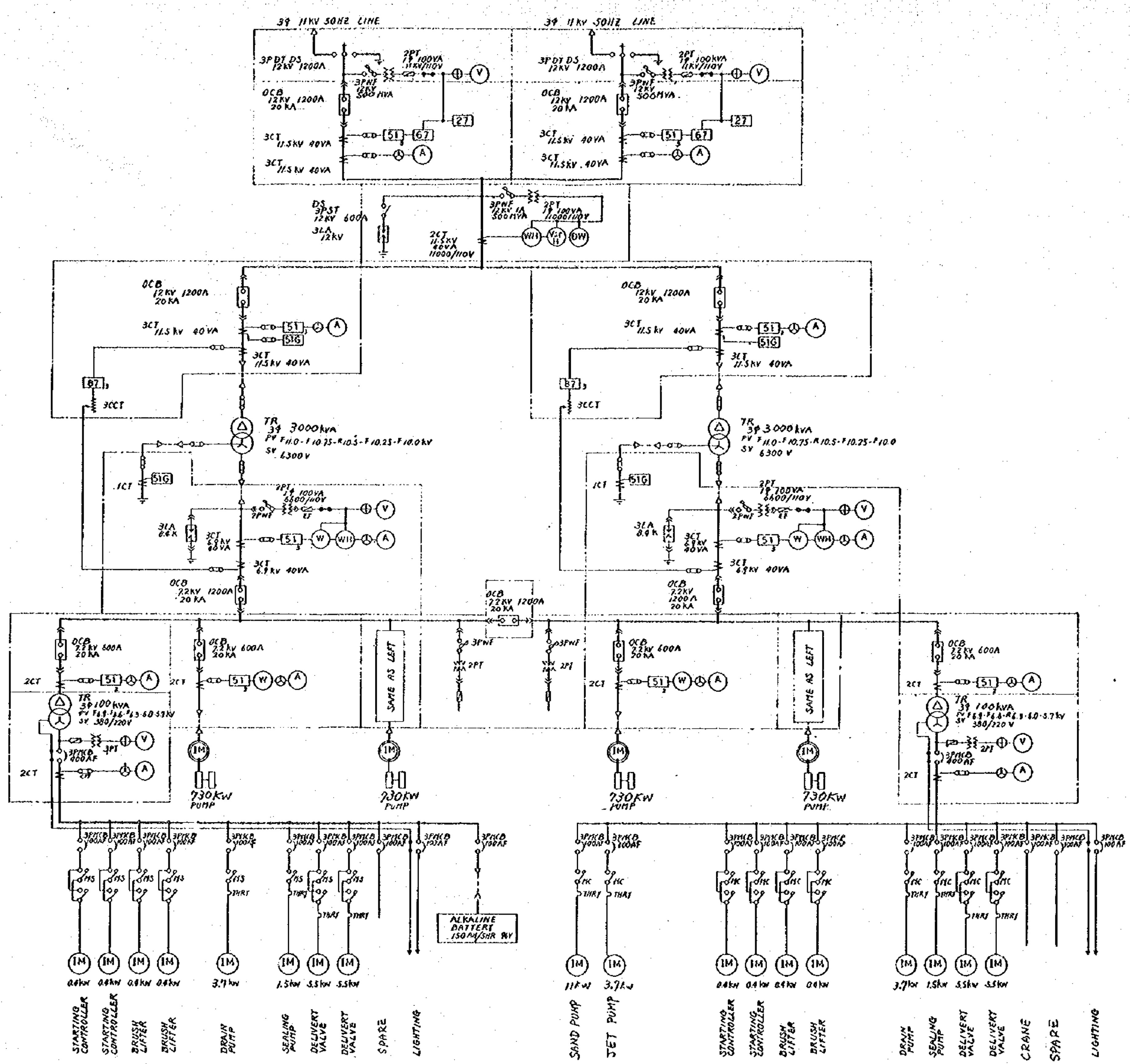
BAR SCREEN



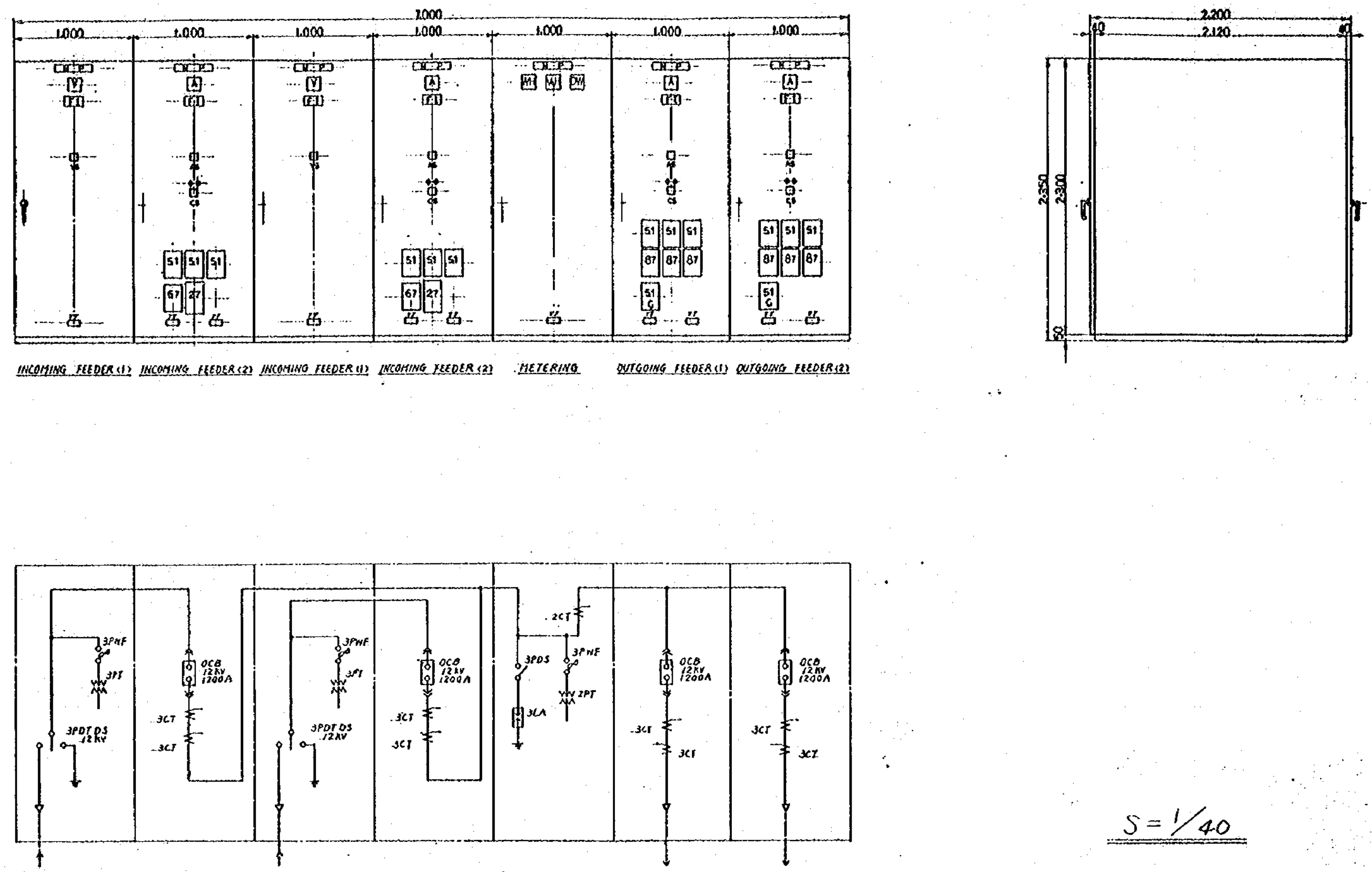
DRAW--R--14

TRAVELING CRANE

FOR NORTH EAST CAIRO PUMP STATION



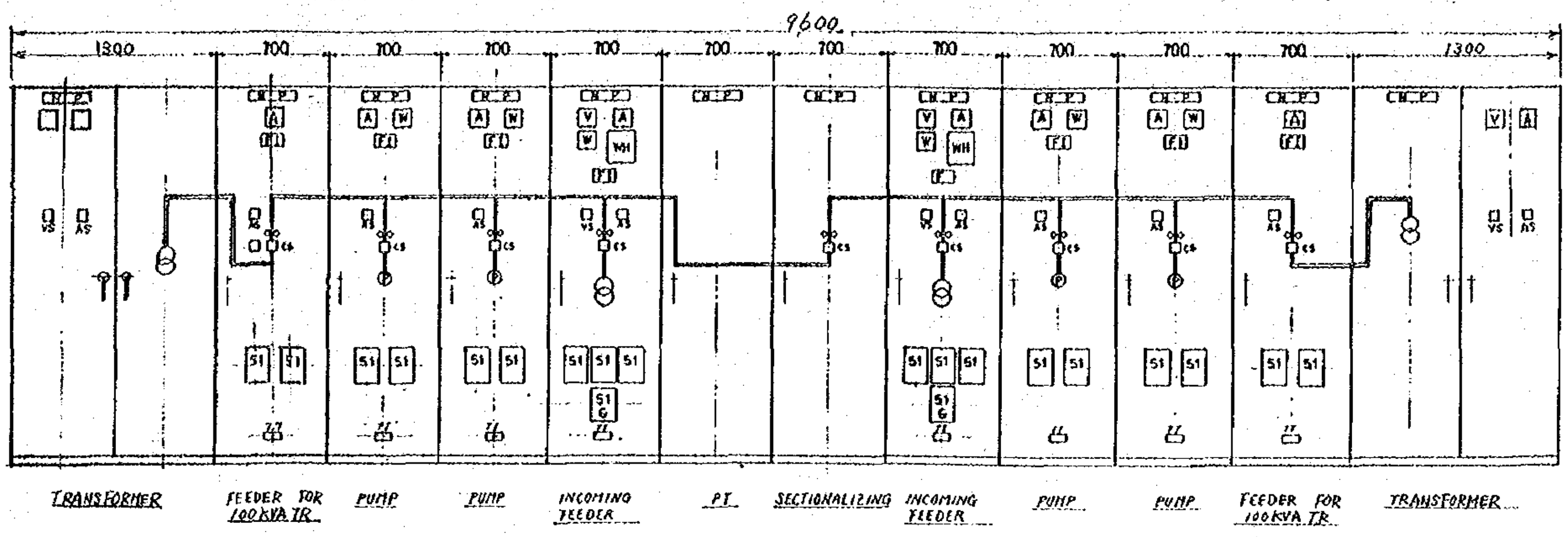
DRAW-R-15 FLOW SEET OF WIRING FOR NORTH EAST CAIRO STATION

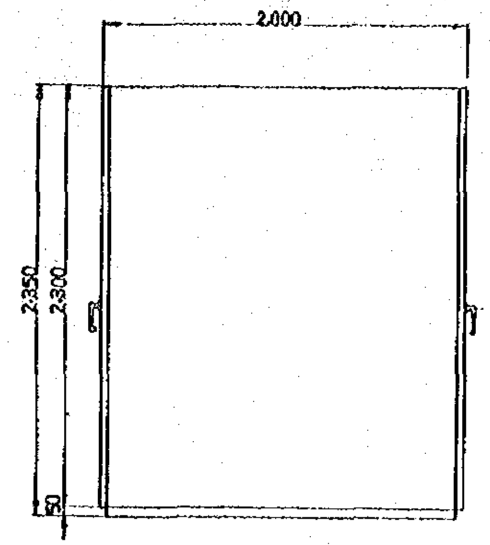


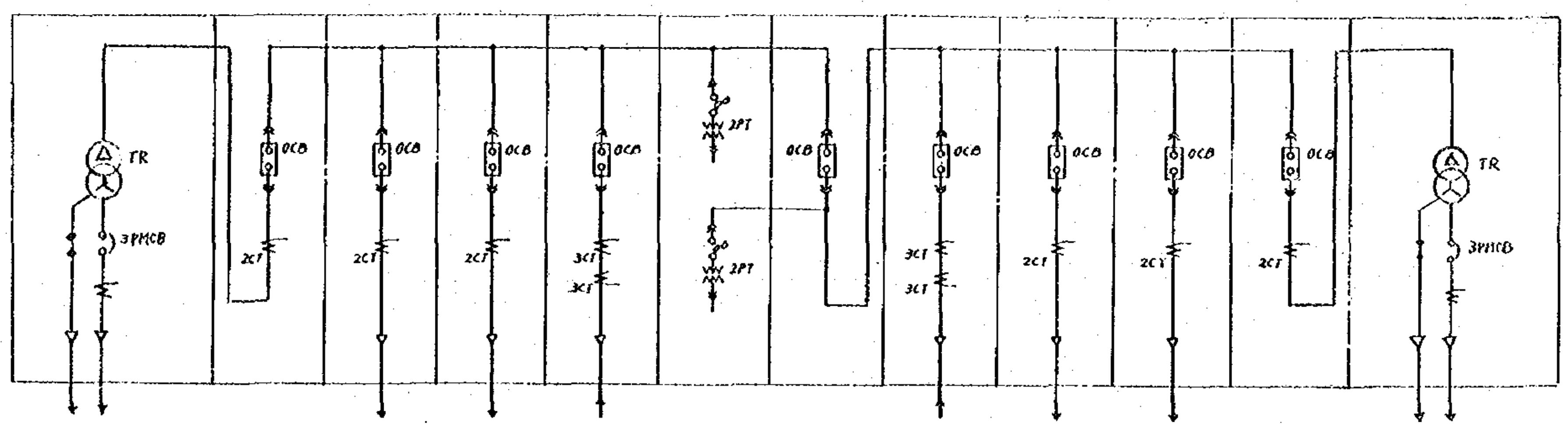
S = 1/40

DRAW-R-16

SWITCHBOARD WITH 10.5KV

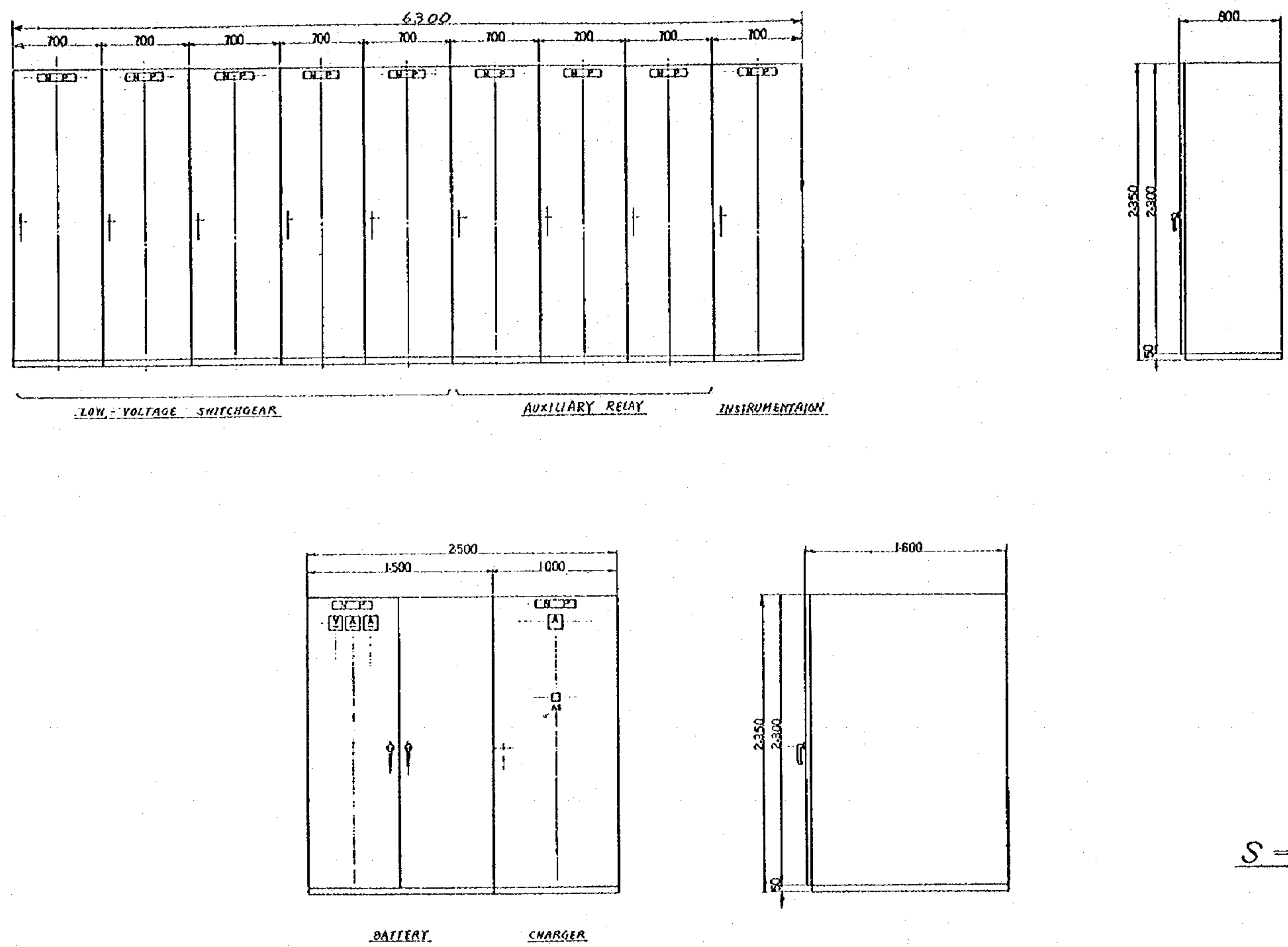






S = 1/40

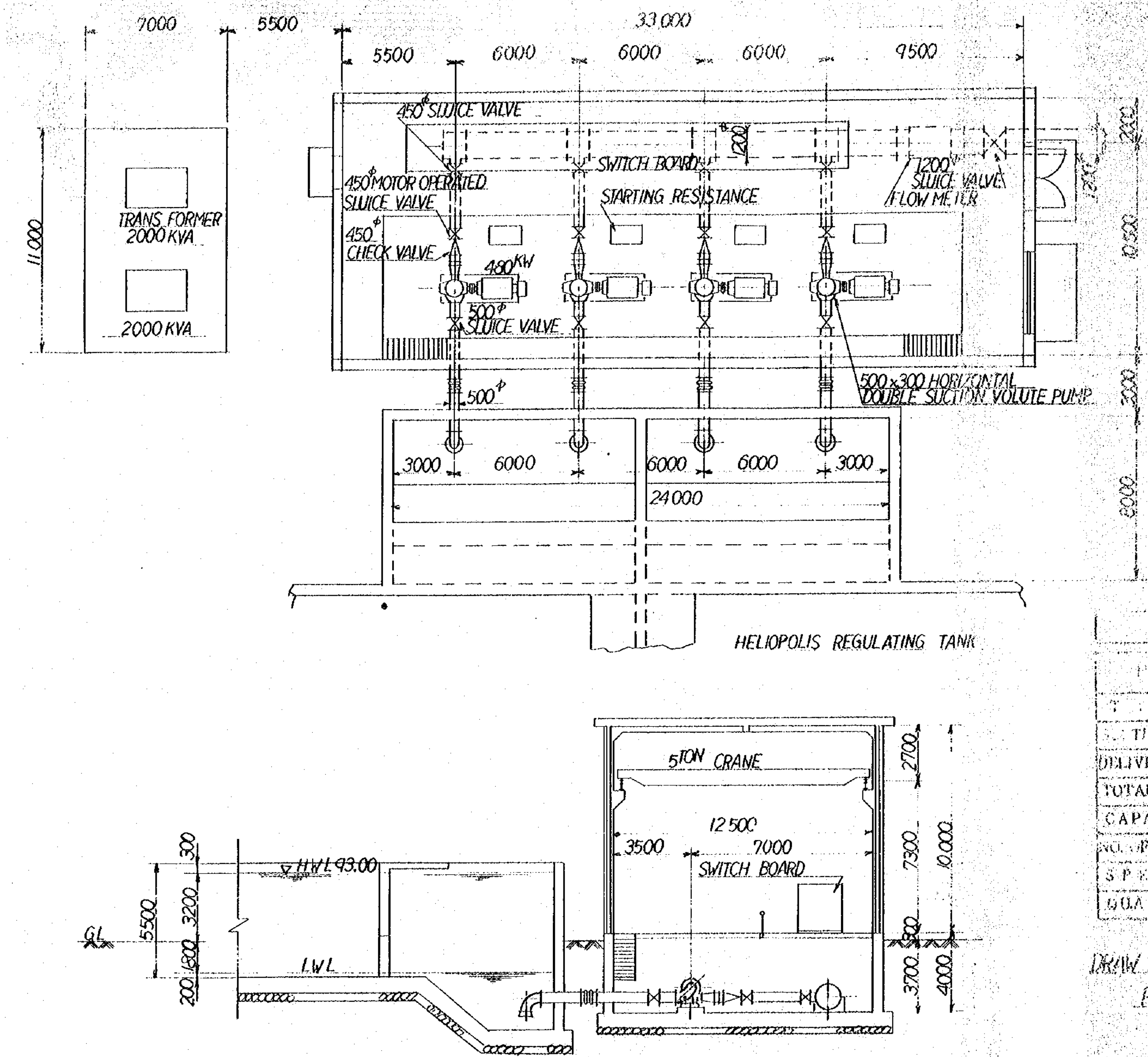
DRAW-R-17
SWITCHBOARD WITH 6.3KV



S = 1/40

DRAW-R-18

LOW VOLTAGE SWITCHBOARD

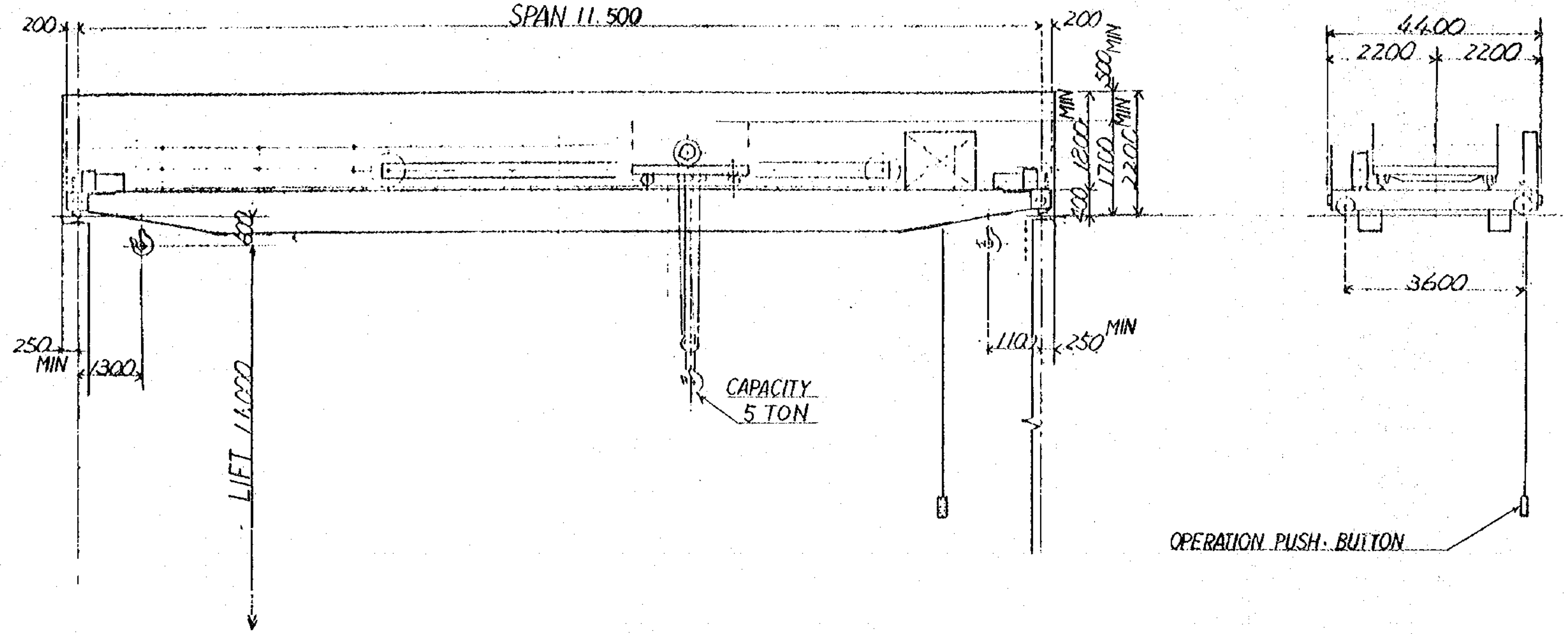


SPECIFICATION			
PUMP		**MOTOR**	
TYPE	DOUBLE SUCTION VOLUTE PUMP	TYPE	GUARDED DRIP-PROOF WOUND ROTOR TYPE
SUCTION DIA	500 MM		
DELIVERY DIA	300 MM	OUT PUT	480 KW
TOTAL HEAD	67 M	VOLTAGE	6300 V
CAPACITY	31 M3/MIN	CYCLES	50 HZ
NO. OF STAGE	1	NO OF POLES	4 P
SPEED	1450 RPM	SPEED	(S.S)1500 R.P.M
QUANTITY	4 SET	QUANTITY	4 SET

DRAW R-19

PRELIMINARY DESING

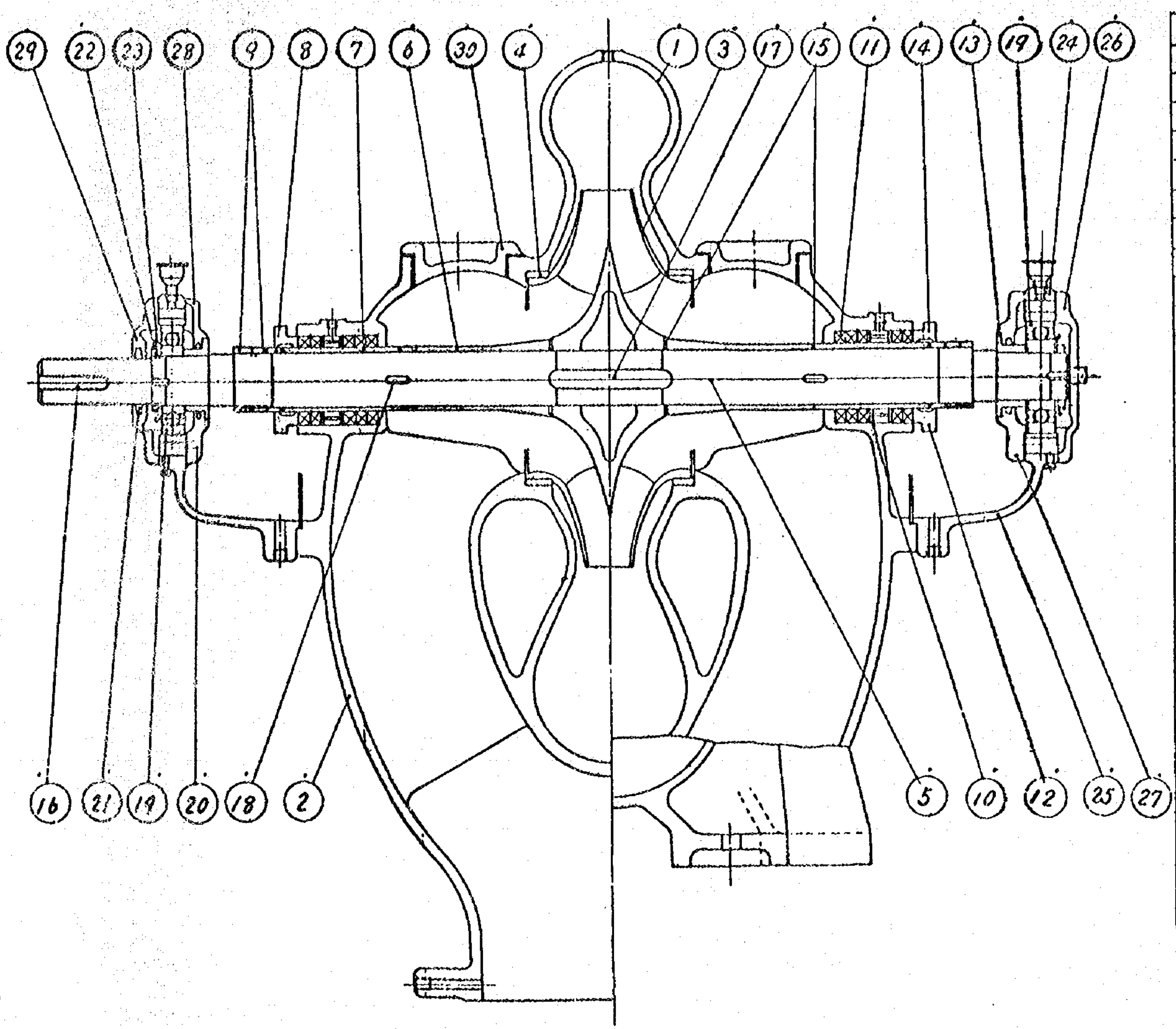
OF PUMP STATION FOR HELIOPOLIS



DRAW-R-20

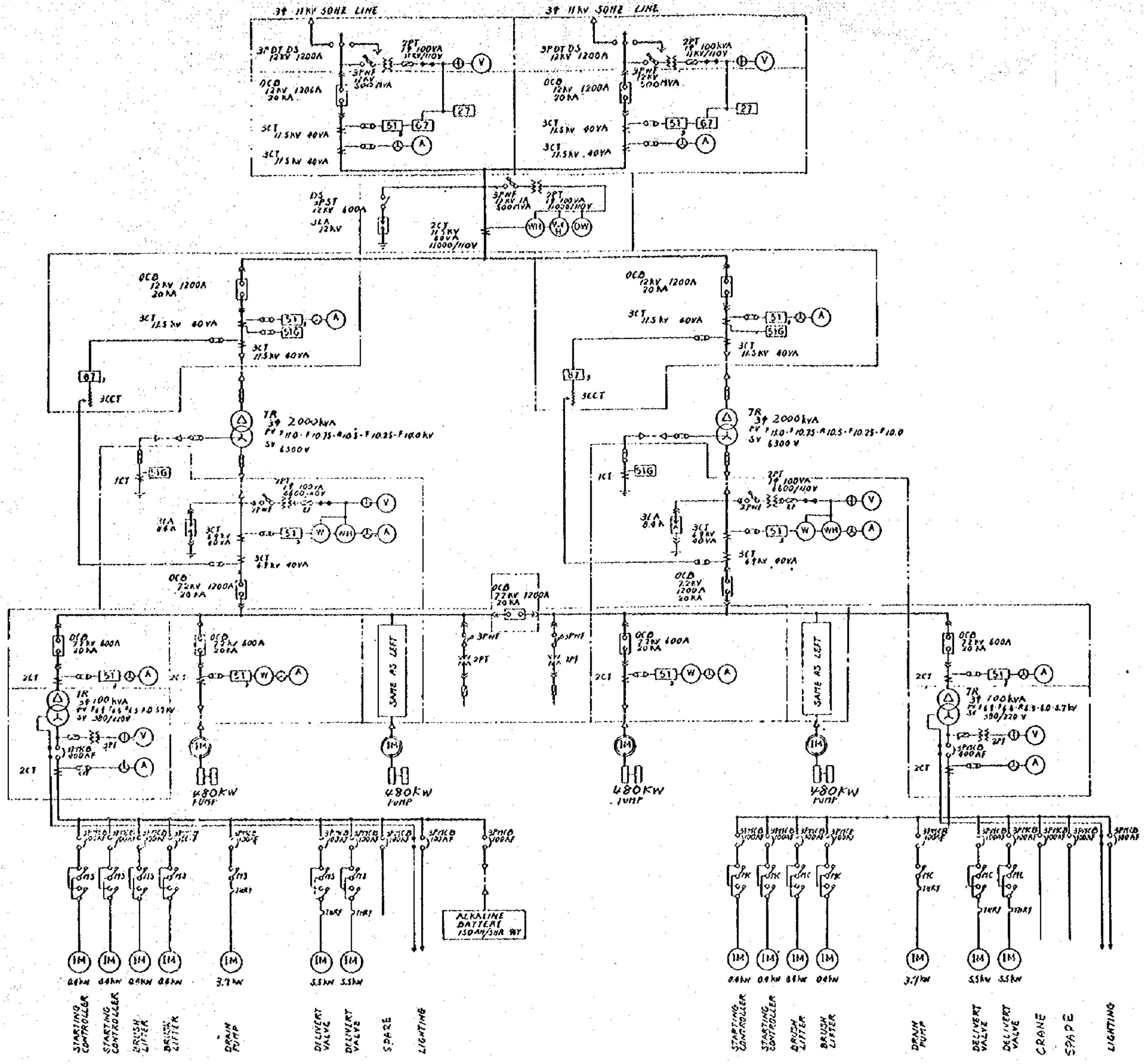
TRAVELING CRANE

FOR HELIOPOLIS PUMP STATION



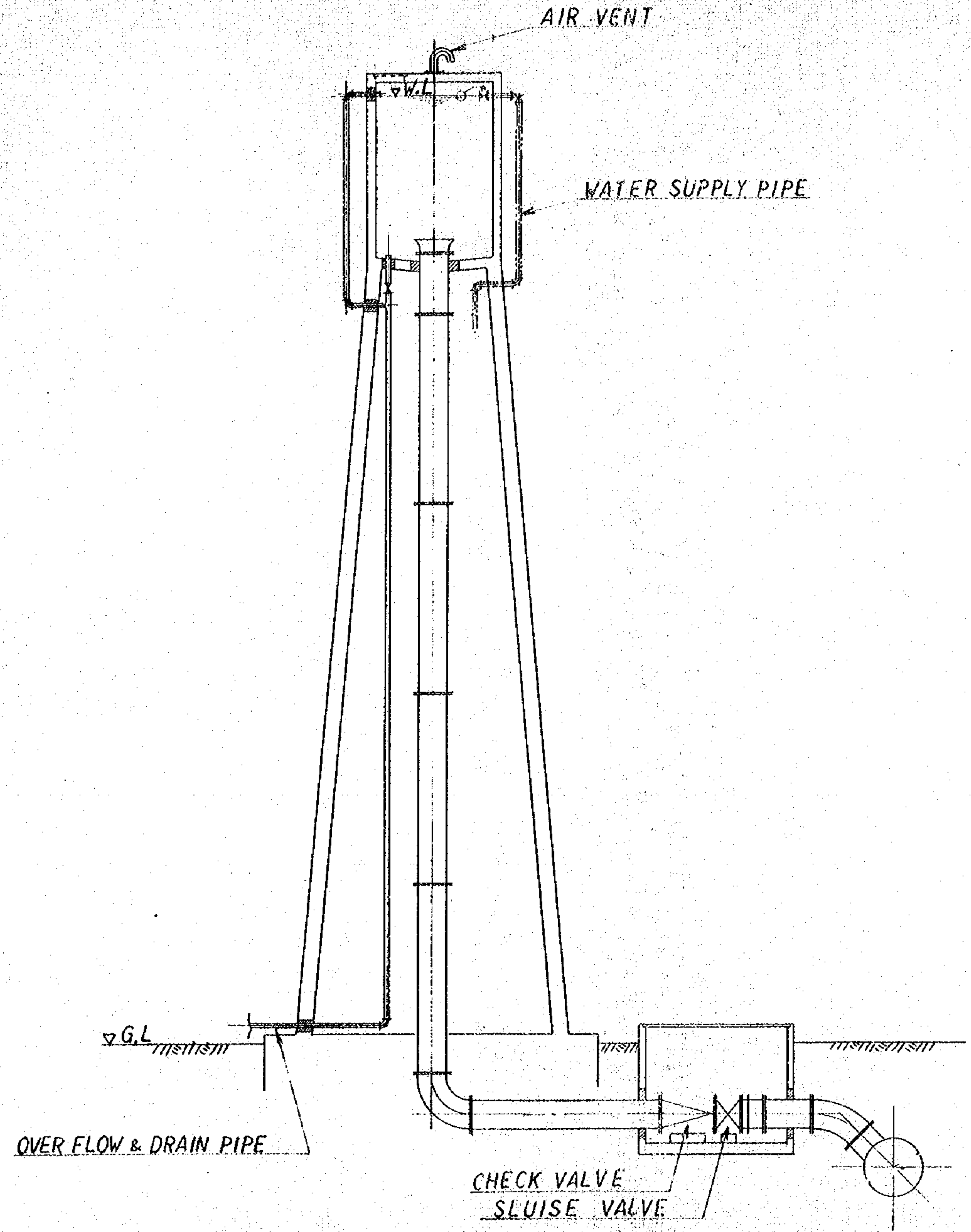
NO.	NAME	MATERIAL
1	UPPER CASING	CAST IRON
2	LOWER CASING	CAST IRON
3	IMPELLER	STAINLESS CAST STEEL
4	LINER RING	STAINLESS STEEL
5	MAIN SHAFT	CARBON STEEL
6	SLEEVE (1)	STAINLESS STEEL
7	SLEEVE (2)	STAINLESS STEEL
8	PACKING RING	STAINLESS STEEL
9	SLEEVE NUT	STAINLESS STEEL
10	LANTERN RING	BRONZE
11	GLAND PACKING	GRAPHITE COTTON
12	GLAND	CAST IRON
13	WATER SLINGER	BRONZE
14	RUBBER GASKET	RUBBER
15	METAL GASKET	CUPPER
16	KEY	CARBON STEEL
17	IMPELLER KYE	CARBON STEEL
18	SLEEVE KEY	CARBON STEEL
19	BALL BEARING	
20	FELT RING	FELT
21	FELT RING	FELT
22	BEARING NUT	STEEL
23	WASHER	
24	UPPER BEARING CASE	CAST IRON
25	LOWER BEARING CASE	CAST IRON
26	BEARING CASE COVER	CAST IRON
27	BEARING CASE COVER	CAST IRON
28	BEARING CASE COVER	CAST IRON
29	BEARING CASE COVER	CAST IRON
30	INSPECTION COVER	CAST IRON

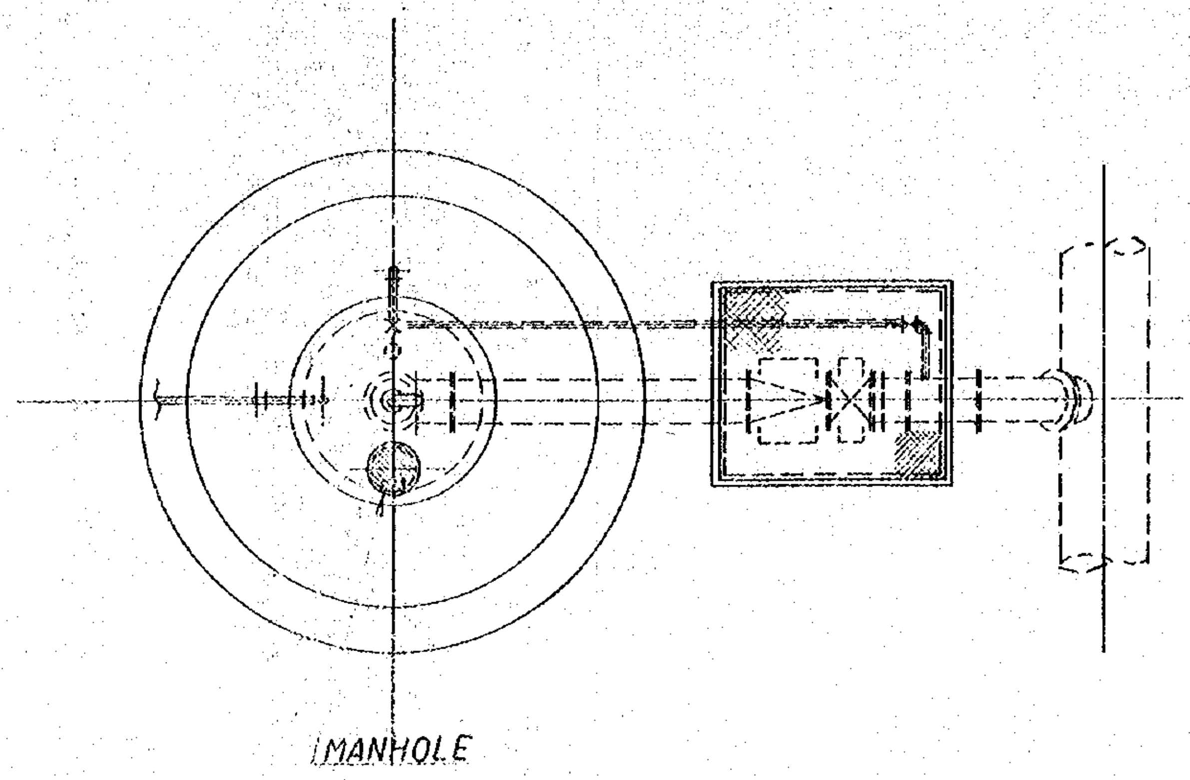
DRAW—R—21
STANDARD DESIGN OF BOOSTER PUMP



DRAW-R-22 FLOW SEET OF WIRING

FOR HELIOPLIS PUMP STATION





S = 1/100

DRAW-R-23

ONE WAY SURGE TANK