PAPER 14

Design of Pump Intakes for Desalination Plant and Sewage Stations

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Design of Pump Intakes for Desalination [illegible]

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This paper summarises good design practice for pump intakes and cites useful references. For new and/or difficult applications, hydraulic scale-model investigations are the accepted design aid and the basic techniques are described.

Against this background typical requirements for desalination and sewage projects are detailed and the development of satisfactory designs in practical situations is illustrated by case histories from desalination plant and sewage stations in operation or building in the Middle East.

INTRODUCTION

Reliability rates high in the design of rotating machinery. Nevertheless many intrinsically reliable pumps suffer operational damage or failure because insufficient consideration has been given to interactions between the pump and the system in which it is installed. Some examples noted from recent general experience in the pump field serve to illustrate this point.

High-speed, high-pressure, boiler-feed and oil-well injection pumps can suffer cavitation erosion at the impeller eye, if insufficient suction head is available relative to impeller-eye design and system operating conditions; again, these pumps can contribute to severe pipework oscillation by a clash of pump forcing and pipe natural frequencies.

Extraction pumps may risk cavitation damage as a result of poor hydraulic design of pipework between suction vessel and pump; also, in the canister type, they may be exposed to dry running seizures by inadequate venting, or water service arrangements, under start-up or standby conditions.

Low-speed, low-pressure, circulating or transport pumps can experience surging under abnormal operation where long pipe runs are involved; they can also suffer vibration, noise, or even mechanical failure, where approach flow is unsatisfactory.

The last item, while not a new problem, is still manifest, despite considerable attention over the last thirty years. Approach flow problems are most common with high specific speed pumps, ie, large flowrates and low system resistance and, in particular, with the suspended-bowl or axial types drawing direct from a free surface sump.

Controlling the flow of large quantities of water with a free surface is not easy when site conditions and economic pressures dictate the basic civil geometry and split responsibility between pump manufacturer and civil engineer adds a communication problem which, it is hoped, this paper may help to overcome.

INTAKE DESIGN

Examples of the repercussions of bad intake design abound. Messina (1), Bird (2), Paterson on Chang and Prosser (3), Paterson and Campbell (4), and Elder, Hamil and Tullis (5), record site problems covering the trouble spectrum from excessive noise and vibration, through component failure, to complete pump breakdown.

Intakes can be classified as wet well or dry well. In the former the pump is suspended directly in the free-surface sump and is sensitive to flow bias, swirl and air entraining vortices therein, while in the latter intervening pipework can be used to reduce flow bias and swirl emanating from the sump, but air injestion remains a problem.

Design codes exist and Figure 1 from Paterson and Noble (6), summarises the recommendations of the two most used codes, namely the American Hydraulic Institute (7), and the British CIRIA/BHRA publication (8), together with corresponding data from the authors' own experience. In applying these codes it is important to note that the minimum submergencies quoted are based on different criteria.

The Hydraulic Institute states no criteria simply covering all recommended dimensions as composite averages from many pump types and specific speeds.

CIRIA/BHRA define minimum submergence as that at which air entraining vortices form. A margin is therefore necessary for safe operation and no guidance is given on this. Only the minimum submergence given by the authors is the safe site operational limit. Both codes instance good and bad features of intake design and both recommend that, if the code guide-lines cannot be satisfied, hydraulic model tests should be undertaken. Further useful guidance can be obtained, however, by gathering available information in the form of experience graphs. The authors' experience is shown in Figure 2, where submergence is plotted against pump suction velocity, both non-dimensionalised using bellmouth diameter or equivalent. The graph can be separated into four areas:-

Area 1 - Ample submergence ensures satisfactory operation of all but fundamentally bad designs. Excavation costs may dictate lower submergence.

Area 2 - With these submergence levels good intake design principles, experience from similar designs in operation, or Hydraulic-Model-Aided Design (HMAD) is necessary.

Area 3 - Careful detail design local to the pump itself is required in addition to the above in order to operate satisfactorily at these submergences.

Area 4 - Low submergence; unlikely to permit satisfactory operation.

Good intake design must be built in to the civil works from the start for economy and effectiveness.

Approach works must promote equally distributed, uniform, swirl-free flow to the pump sump(s).

Basic geometry, screens, auxiliary walls, and guide vanes should be located and shaped with this in mind. Guide vanes, for example, should intercept flow where it is uniform or be offset at inlet proportional to flow bias. Abrupt changes in area or direction and obstructions in the flow path should be avoided. Sumps should accept flow without introducing bias or separation and channel it to the pump suction, preferably with a smoothly accelerating flow. Flushing curtain walls should be used to avoid vortices being shed from sump dividing wall ends. Pumps should be located in centre-line of individual sumps if possible and close to the end wall. Vortices in the pump wake can be precluded by the use of front curtain walls, submerged roof, or haunching behind the pump.

The pump suction is normally fitted with a convergent bend or bellmouth but, for low submergences, may require additional control vanes or surfaces, which will however incur a slight loss in efficiency.

For unconventional or difficult designs or cases where the design principles noted above cannot be applied, the intake design should be investigated using an hydraulic scale model.

HYDRAULIC-MODEL TECHNIQUES

Pump intake models are normally fixed bed and sufficiently limited in extent to permit adoption of undistorted linear scales. Basic scaling procedures for dynamical similarity are then well established (4,8) but there is an incompatability problem.

The main forces involved with predominantly free surface models are gravitational and viscous, with surface-tension effects of importance only when shallow flow sections and pronounced surface curvatures are involved.

The Froude, Reynolds and Weber force ratios applicable cannot each be satisfied by the same operating velocity with cold water, the pumped fluid in the model and at site.

Gravity governs basic flow patterns and entails a model velocity proportioned as the square root of the scale ratio relative to site.

Viscosity affects flow regime, boundary layer flow, separation, and losses, for which the model velocity should be inversely proportional as the scale ratio.

Surface tension is rarely significant but if so would require a model velocity inversely proportional to the square root of the scale ratio for correct representation.

The basic Froude/Reynolds conflict must be carefully considered in relation to model size, intake geometry, and design criteria applicable in each investigation. The recommended procedure is to run the model at Froude-scale velocities, ensuring that the model is sufficiently large to reproduce the site flow regime and assessing scale effects on separation, vortices, and losses. Increased operating velocities can be used in this assessment provided the basic flow patterns are preserved.

For desalination plant the sea-water pump intakes are normally the only candidates for model investigation though occasionally other services such as product water are sufficiently unique to merit HMAD. The standard procedures outlined cover these investigations.

Sewage applications introduce several factors which need further consideration. Variable inflow to the sump with level controlled pump-out, siltation and sewage settlement in stagnation and low-velocity zones, aeration and release of hydrogen-sulphide gas, all complicate operation of the model and interpretation of the results.

Recommended methods of dealing with these additional factors are discussed and illustrated in the case histories quoted on sewage plant.

DESALINATION PLANT CASE HISTORIES

Dubal Power Station and Desalination Plant: SW Intakes

Three inlet pipes channelled seawater by way of a common forebay to nine SW service pumps and one standby, each in its own sump and individually protected by coarse and travelling-band-type screens. The intake was well designed; inlet pipe and screen ports were submerged; velocities were moderate; transitions were gradual; and design principles, in general, were observed and not compromised to save space.

With multiple pump arrangements such as this, operational requirements usually dictate various combinations of pumps and possibly inlet pipes. Despite thoughtful design the common forebay under these circumstances becomes an area of complex flow patterns. In order to investigate the nature of these flow patterns and establish their effect on flow distribution at entry to each screen chamber a one to thirteen scale model of the approach works was constructed. The S.W. intake arrangement is shown in Figure 3 and the model test facility has been added to illustrate the type of test rig and instrumentation applicable.

Operation with all inlet pipes and all, or the majority, of the pumps gave satisfactory conditions. Flow crossed the forebay in three main streams, diffusing as they approached the screen chamber inlets. Low-velocity reverse circulating flows occurred between streams and along the forebay side walls but flow bias at screen chamber inlets was negligible. Reduction in number of pumps in operation and/or outage of a supply pipe resulted in deflection of the main streams and variation in extent of the associated subsidiary flows, causing a more pronounced bias at entry to certain screen chambers. Quantitative data, in the form of isovelocity plots, were determined using miniature current meters and, in conjunction with flow-pattern plots, used as reference inlet conditions for a larger scale model of one screen and pump chamber. Figure 4 shows the extent modelled. The model test rug has again been added, in this case to illustrate the screen and gate method of varying entry conditions to simulate the extremes applicable to any of the ten pumps, and the techniques used to monitor flow conditions which resulted in the pump chamber. A plot typical of the biased entry flow conditions to be tested is inset.

In many cases it was found that the bias was sufficient to alter the normal sump-flow regime, causing flow down one side to dominate, with the result that mass circulation built-up in one direction at pump inlet and asymmetry in the wake of the pump casing allowed vortices to develop.

Swirl at pump inlet alters impeller-blade and shaft loading and causes mismatch of blade and flow angles while vortices imply intermittent air injestion to pump and system. The combination of increased loading with intermittent air entrainment can cause severe fatigue effects which have in one case resulted in impeller-blade fracture.

Vortex formation behind the pump was prevented by introducing a curtain wall across the sump, upstream of the pump, while any residual swirl around the pump was curbed by fitting vertical control vanes to the wall behind the pump. Flow pattern and modifications are shown inset in Figure 4.

Dubal Power Station and Desalination Plant: Blended Water Intake

The blended-water transfer pumps intake was unusual in that the pumps were located in-line, in a long narrow channel constructed at one end of a reservoir. The reservoir was divided by a central wall such that the pumps could be fed from either or both ends of the channel. Flow to operating pumps inevitably had to pass other operating and stationary pumps.

Since this is not a recommended arrangement an hydraulic scale model was built to examine the approach flow from reservoir to channel and the various flow regimes in the pump suction channel.

The intake is shown in Figure 5. Tests covering the various operational groupings indicated three sources of trouble.

Vortex formation occured at the change of flow direction at channel entry from the reservoir. Coupled with the downward flow through the channel entry port this led to air entrainment by the outermost pumps.

Swirl of varying severity was experienced at the pump inlets, caused by the concentration of flow along the outer wall of the channel.

Surface swirls tended to form in the wake of the pump suspension column under critical flow-past velocities. After considerable experimenting the most effective and least costly modifications were two in number, viz:

Extension of the inner channel wall in the form of a local curtain wall with a vented shelf or ceiling between the wall soffit and stop gate location to control approach flow; and

Splitter vanes upstream and downstream of the pump bellmouths to control swirl at pump inlet.

These modifications are shown inset to Figure 5.

Ras Abu Fontas Desalination Plant: S.W. Intakes

In this arrangement seawater flowed by way of an approach culvert into a stilling chamber with sloping sidewalls. The stilling chamber had five screen chambers along the rear wall leading to a common pump chamber, wherein the pumps were situated along the rear wall separated by narrow piers with gaps between them and the wall, as shown in Figure 6.

Stilling chamber flow was characterised by a wide main stream diffusing from culvert to screen chambers, with lazy reverse circulations at the sides. Even under reduced unit operation, flow into the screen chambers was only slightly biased with negligible effect on flow conditions downstream of the screens.

Conditions in the pump chamber were far from satisfactory however. The offset of screen exits with respect to pumps resulted in asymmetric flow in all cases, aggravated by separation from the pier heads and flow through the gaps at the rear. A typical flow plot is shown inset to Figure 6, the problems of mass circulation round the pumps and vortex formation behind them being obvious.

Fitting a curtain wall flush along the front of the piers, thickening up the piers and closing the gap at the rear, in effect creating individual pump sumps with a large length-to-width ratio, yielded a considerable improvement, but some bias in flow at sump inlet remained in most cases owing to the restricted distance from band screen exit ports to pump sumps. This bias tended to induce surface swirls downstream of the curtain walls and mass circulation at the pumps somewhat higher than normally accepted. Extension of the curtain-wall soffit along the sump and fitting twin control vanes behind the pump as shown inset to Figure 6 removed these residual faults.

SEWAGE-PLANT CASE HISTORIES

Al Ain Pumping Station No. 1

The site layout is shown in Figure 7 and the finally developed layout in Figure 8. Modifications were extensive to solve the hydraulic problems encountered.

It has been the authors' experience that sewage sump design derives little from the recommended codes of practice with regard to pump-suction submergence and associated sump hydraulic design. In order to minimise fluid-retention time sewage sumps are small in comparison to most other pump intakes and hydraulic scale models are frequently made necessary because pump-suction submergence is low, sump inlets are high and there is little directional flow control within the sump.

On the left-hand side of Figure 7 a plan and section of the original site arrangement for this station is shown. Flow entered each half of the sump *via* twin outfall sewer pipes and a screen chamber. Normal high water level in the sump was below sump inlet invert level. The hydraulic problems encountered were two-fold comprising, first, severe turbulence and the generation of masses of air bubbles in way of pump suctions Nos. 6 and 3 and, second, problems of swirl and vorticity caused by the off-set entry to each half of the sump.

The hydraulic scale model built to study these and other phenomena was of the type indicated on the right-hand side of the diagram. Water was pumped from an underfloor reservoir into a constant-head tank upstream of the model at various flowrates up to the maximum future handling capability of the station. Abstraction of flow from the model was by a drain/syphon system in which flowrate was measured using calibrated orifice plates and controlled using gate valves downstream of the orifice-plate location.

Referring to the final layout on Figure 8, the first step in the development was to isolate the inlet turbulence and associated air bubbles from the inner pump suctions and to produce a more controlled approach to all pump suctions. This was done by installing a long wall along the centre of the sump incorporating a streamlined 180-deg turn at each outer end.

Problems of vorticity persisted, however, because of the extremely low pump suction submergencies required. At low water levels the crest of each suction bend became uncovered and as flow negotiated this obstacle eddies formed in the lee of each bend and matured into air entraining vortices. These vortices were countered by the installation of short baffle walls which prevented the passage of surface flow across the bend crest and thereby prevented eddy shedding and consequent vortex formation.

It should be noted that when installing such baffle walls cognisance should be taken of the floating debris in the sump. The walls should, therefore, be positioned and sized in keeping with the philosophy adopted for handling such materials.

<u>Al Ain Pumping Stations W2 and W4</u>

The layouts of these stations were more or less identical and a plan and elevation are shown on Figure 9. Incoming water passed from the sewer through a bar screen and down into the sump <u>via</u> a section of curved benching. Inlet turbulence was largely confined to the area upstream of the inlet chamber dividing wall and distribution to the pump suctions, only one of which was a working unit, was through the slot beneath the wall.

While the hydraulic problems of vorticity and turbulent air bubble entrainment were evident, other causes for concern in this investigation were more specific to sewage projects, namely turbulence and its effect upon the release of hydrogen sulphide gas, and silt/solids deposition.

The very nature of the fluid being handled dictates that some hydrogen sulphide (H_2S) gas will be released regardless of the part of the world in which the station is built. In the Middle East, however, this problem is considered to be more severe than most owing to the concentrated acidity of the sewage.

In turbulent flow areas the gas is released and is readily oxidised by bacteria in the presence of air to sulphuric acid, which is both destructive and a danger to health. Where high level inlets are associated with low sump-water levels, a common feature of sewage-station design, turbulence is unavoidable. Nevertheless, steps should be taken to minimise it where possible. One source of such turbulence on this station was identified as the bar screen. As flow entered the screen chamber area from the sewer it impacted upon and spread across the sloping section of the curved benching, creating a thin film flow through the bar screen. This film separated from the screen bars and generated concentrated "fingers" of flow which did not follow the radius of curvature of the benching but fell directly on to the water surface below, aggravating the turbulence in that area.

In order to eliminate this source of turbulence the shape of the bar screen was altered to curve under the incoming sewer in such a way as to screen bulk flow rather than thin film flow. While turbulence persisted at the foot of the curved benching at low water levels, forthe obvious reason of water-level difference, the severe concentrated turbulence generated by the "fingers" separating from the screen members was eliminated and the residual turbulence equalised over the whole of the enclosed area.

The other problem encountered on this model was one of silt/solids deposition on the apron approaching the pumps' suction.

On the plan view of Figure 9, two sparge pipes can be seen entering the screen chamber area. These pipes were led from the pump discharge pipework and the valves were manually operated to jet the pumped fluid across the apron, to drive the settled materials towards the pump suction and through the pumps. A third sparge pipe can be seen between the pump suctions for clearing the small suction trough. As the pumps were of low capacity and, because they were handling pumped fluid, the sparge pipes had to pass a specified solid/sphere diameter, the jetting velocity was relatively low and tests showed that operating both sparge pipes together at half capacity had little effect upon clearing the deposited materials.

Adjustment of the jetting angle and operation of one pipe at full capacity followed by the second at full capacity, before operating both together, successfully cleared the apron of all settled sand and silt.

It is worth noting that these tests were run on the basis of varying inflows rather than at steady state condition, that is equal inflow and outflow. The model was arranged in a similar fashion to that shown on Figure No. 7, except that butterfly valves were used on the drain/syphon outlets to give a faster reaction time to simulate pumps starting and stopping, much as they would do on site. This permitted conditions to be examined under the influence of less turbulent inflows and less sump water movement, thereby providing a more reliable basis for assessment of likely settlement patterns.

Qatar; West Bay Pumping Station

A further example of the aspects of sewage-pumping-station design which should be taken into consideration is the generation of air bubbles from the high-level inlet waterfall.

The layout shown on Figure 10 is that of the West Bay Pumping Station in Qatar. On the left of the figure, the planview of the station shows the inlet baffled by a metal box upstream of a bar screen. While this small deflector box served to prevent direct impingement of incoming sewage on top of the first pump suction, turbulence and the generation of air bubbles occurred at the outlet from the box and the bubbles were carried towards and into the two upstream suctions.

The difficulty here is in the assessment of how dangerous this is and in making such an assessment it must be borne in mind that the air bubbles generated on the model will have venting characteristics similar to those on site. This being the case, in the higher-velocity environment of the site the bubbles will carry further and more will be transported. Accordingly the problem was tackled by improving control at inlet in the manner shown on the right of the figure.

The screen was moved upstream and raised and an 'L'-shaped baffle was installed beneath it ensuring that the access opening to the sump was submerged at cut-out water level. As a result of this modification turbulence was contained within the baffled area and those bubbles which were entrained through the bottom slot vented along the underside of the baffle to the surface before reaching the pumps.

CONCLUSIONS

Many pump operational problems have their source in unsatisfactory approach flow, particularly with low-head, high-flow-rate pumps. Hydraulic-model-aided design has proved a useful method of evolving satisfactory designs, as illustrated by the various case histories discussed.

In desalination plant multiple pump sumps are normal and the complex flows upstream of the pump chambers, occasioned by flexibility of operation, raise the alternatives of adopting space hungry good design principles, or accepting a more economic arrangement involving disturbed flow in the forebay and requiring that the pump sump be made insensitive to this. The latter is practicable with little increase in pumping head and is the more favoured arrangement.

With sewage plants the additional features to be considered are turbulence in relation to hydrogen sulphide release and air bubble injestion, and sudden changes in flow section in relation to silt/ solids deposition. Minimising and isolating turbulence and air from the pump suctions and avoiding low velocity regions by thoughtful introduction of benching are methods of overcoming the problems.

In all cases early involvement of the pump manufacturer in the civil works design is important and his experience and advice regarding intake design and the need for model testwork should be sought at this stage.

ACKNOWLEDGEMENTS

The authors wish to thank the directors of Weir Pumps Limited for permission to publish this paper and also the undernoted customers for giving approval to use the material cited in the case histories.

Dubal Aluminium Co Limited, U.A.E.	)	Dubal Power Station and
Weir Westgarth Ltd, UK	)	Desalination Plant
Ministry of Electricity & Water Qatar	)	Ras Abu Fontas
	)	
Ewbank & Partners, UK	)	Desalination Plant
Al Ain Sewage Projects Committee U.A.E.	)	Al Ain Sewage Stations
	)	
D. Balfour & Sons, UK & UAE	)	
Ministry of Public Works, Qatar	)	West Bay Pumping Station
Pencol Limited, UK	)	

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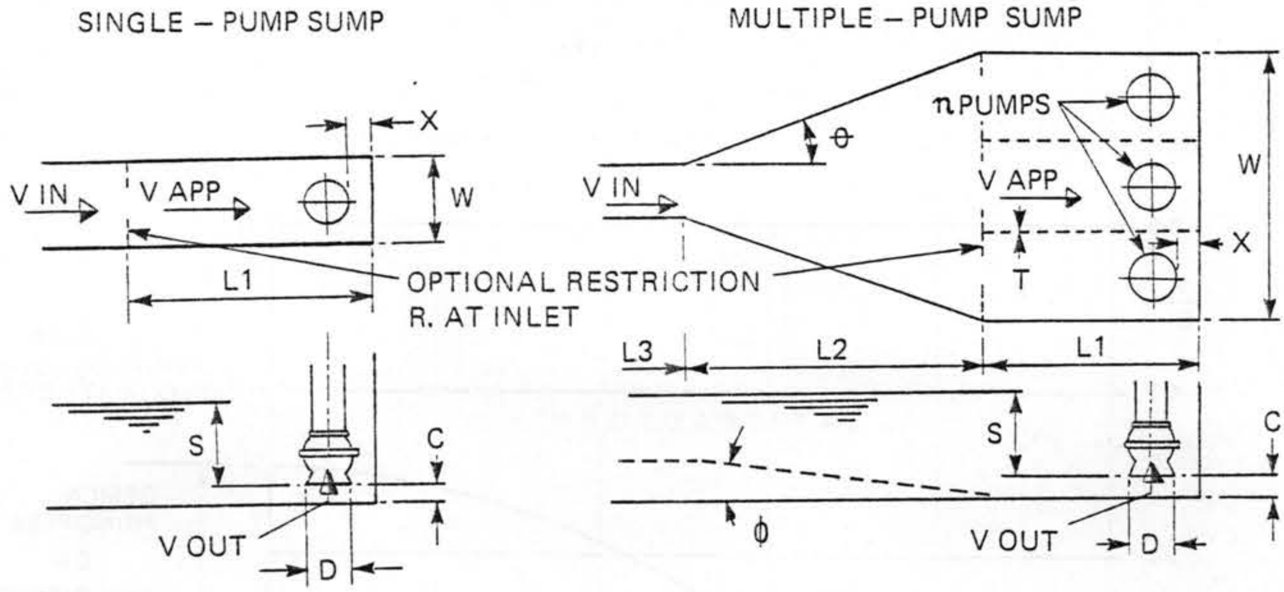
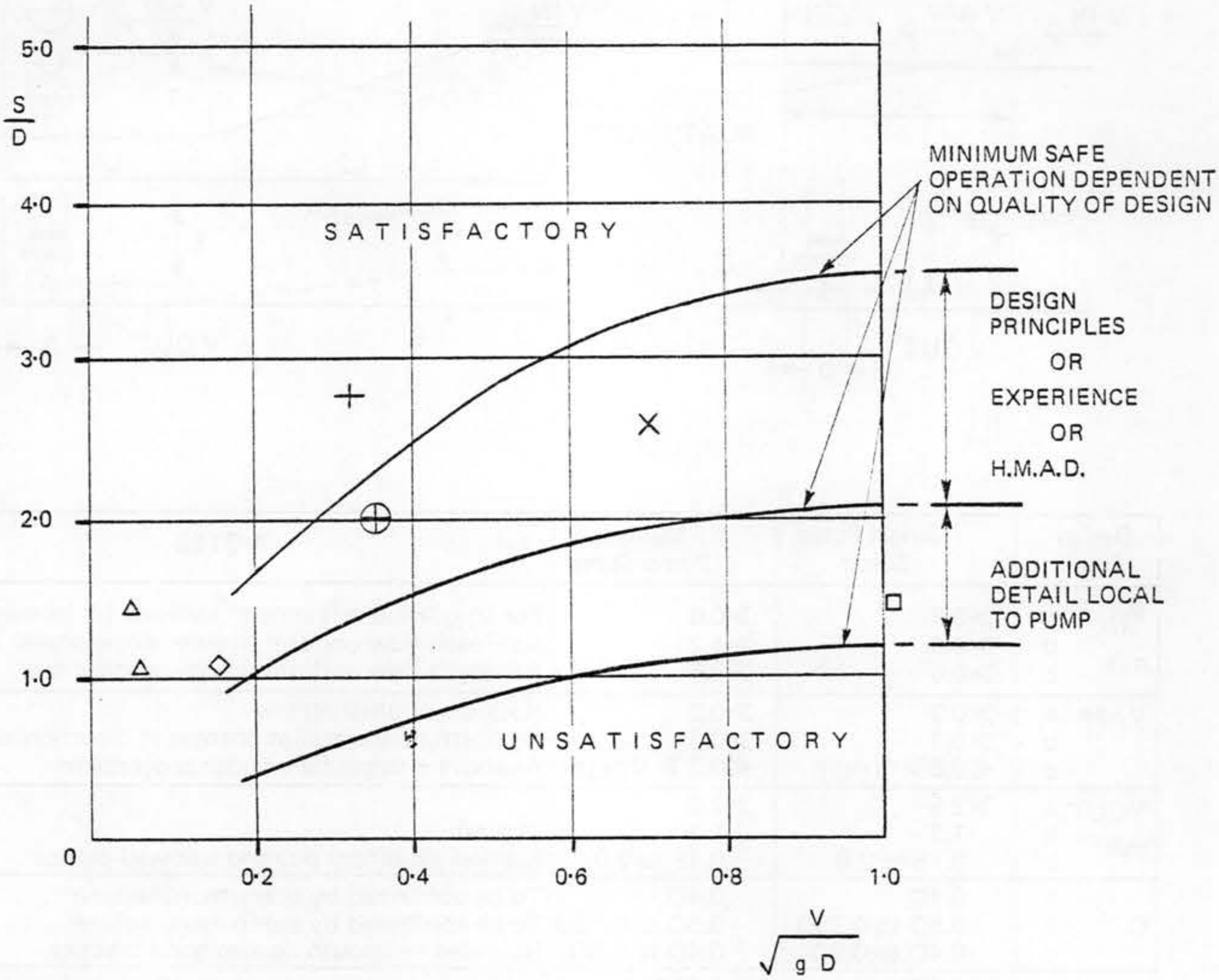


Design Item		Single Pump Sump	Multiple Pump Sump	NOTES
V_{IN} m/s	a	$\ngtr 0.6$	$\ngtr 0.6$	For $V_{IN} > 0.6$ m/s control vanes or L1 increased.
	b	$\ngtr 0.6$	$\ngtr 1.2$	Approach flow uniform, steady, single phase.
	c	$\ngtr 0.6$	$\ngtr 0.6$	Approach flow uniform, steady, single phase.
V_{APP}	a	$\ngtr 0.3$	$\ngtr 0.3$	No turns or obstructions.
	b	$\ngtr 0.3$	$\ngtr 0.3$	No obstructions/abrupt changes in direction/area
	c	$\nless 0.3 \ngtr V_{OUT}$	$\nless 0.3 \ngtr V_{OUT}$	As above + target for smooth progression
V_{OUT} m/s	a	$\ngtr 2.6$	$\ngtr 2.6$	
	b	1.3	1.3	Typical
	c	0.75 to 2.0	0.75 to 2.0	Suction pipes (dry pit) and wet well pumps
C	a	0.4D	0.4D	To be confirmed by pump manufacturer
	b	0.5D to 0.75D	0.5D to 0.75D	To be confirmed by pump manufacturer
	c	0.4D to 0.6D	0.4D to 0.6D	Rounded bellmouth lip also good practice
X	a	$\ngtr 0.35D$	$\ngtr 0.35D$	Avoid axial in-line unless $L > 4D$, $W > 5D$
	b	0.25D to 0.5D	0.25D to 0.5D	Avoid axial in-line unless $L > 8D$, $W > 3D$
	c	$\nless 0.25D$	$\nless 0.25D$	Or min. practicable. In-line as above + baffles
W	a	$\nless 2D$	$\nless 2nD + (n-1)T$	Inter walls if all run. If not gap at rear/omit
	b	2D to 3D	$\nless 2nD + (n-1)T$	Inter walls if inflow skew/$<n$ run. Vanes in L1 and L2
	c	$\nless 2D$	$\nless 2nD + (n-1)T$	Inter walls if inflow skew/$<n$ run. Vanes in L1 and L2
L	a	$\nless 3D$	$\nless 5.5D$	$\theta \ngtr 45^{\circ}$ (15° preferred), L large
	b	$\nless 4D$	$\nless 0.7W$ or 4D	$\theta \ngtr 20^{\circ}$ $\phi \ngtr 10^{\circ}$. Keep slope turbulence from pump
	c	$\nless 4D$	$\nless 0.7W$ or 4D	$\theta \ngtr 20^{\circ}$ $\phi \ngtr 10^{\circ}$. Keep slope turbulence from pump
L (R at inlet)	a	(3W/R) D	(3W/R) D	Pipe/channel. (Use up to $W/R = 4$)
	b	(5W/R) D	(5W/R) D	Channel/Channel. $W/R > 2$, L = 10D + Vanes
	c	(5W/R) D	(5W/R) D	Channel/Channel. $W/R > 2$, L = 10D + Vanes
S_{MIN}	*a	3D to 2D	3D to 2D	Reduction with size 0.2 m^3/s to 15 m^3/s
	b	1.5D	1.5D	Cones or splitters reduce swirl. May affect perf.
	c	3.5D to 0.5D	3.5D to 0.5D	Dependent on V_{OUT}/size/HMAD/experience

a – Reference (7) b – Reference (8) c – Authors

* Min. submergence from flow aspects; NPSH requirements also apply; use whichever greater

FIG. 1 SUMP DESIGN RECOMMENDATIONS



LEGEND –

S – SUBMERGENCE
D – BELLMOUTH OR PUMP SUCTION 'DIAMETER'
V – VELOCITY AT BELLMOUTH OR SUCTION
g – GRAVITATIONAL CONSTANT

MIN OPERATING CONDITIONS:

⊕ – DUBAL S.W. PUMPS
\+ – DUBAL BLENDED WATER PUMPS
× – RAS ABU·FONTAS SW PUMPS
□ – AL AIN STATION 1
△ – AL AIN STATIONS W2 / W4
◇ – QATAR WEST BAY

FIG. 2 DESIGN CHART

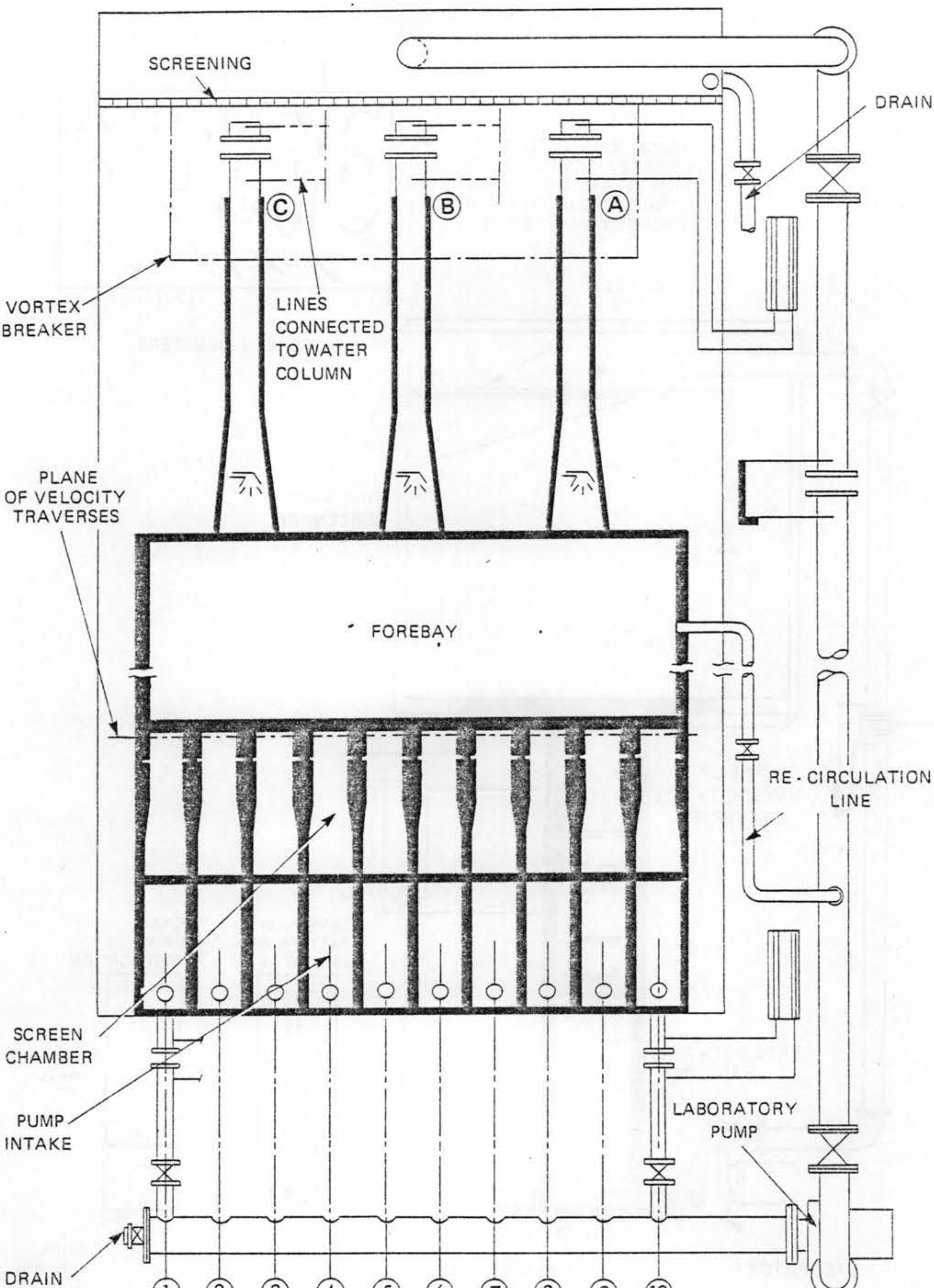


FIG. 3 PLAN OF S.W. INTAKE WITH MODEL FACILITY ADDED

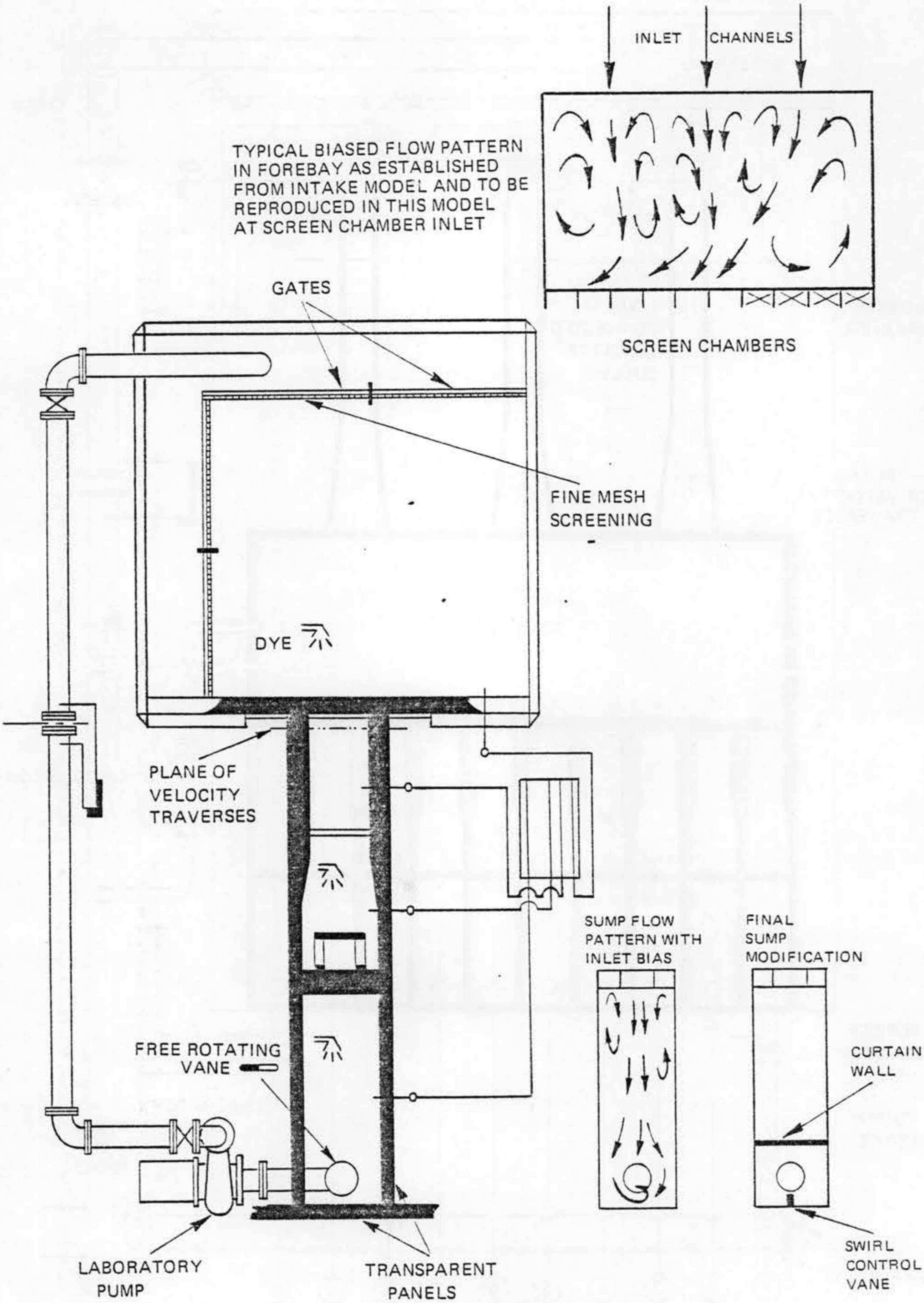


FIG. 4 PLAN OF INDIVIDUAL PUMP/SCREEN CHAMBER WITH MODEL FACILITY ADDED

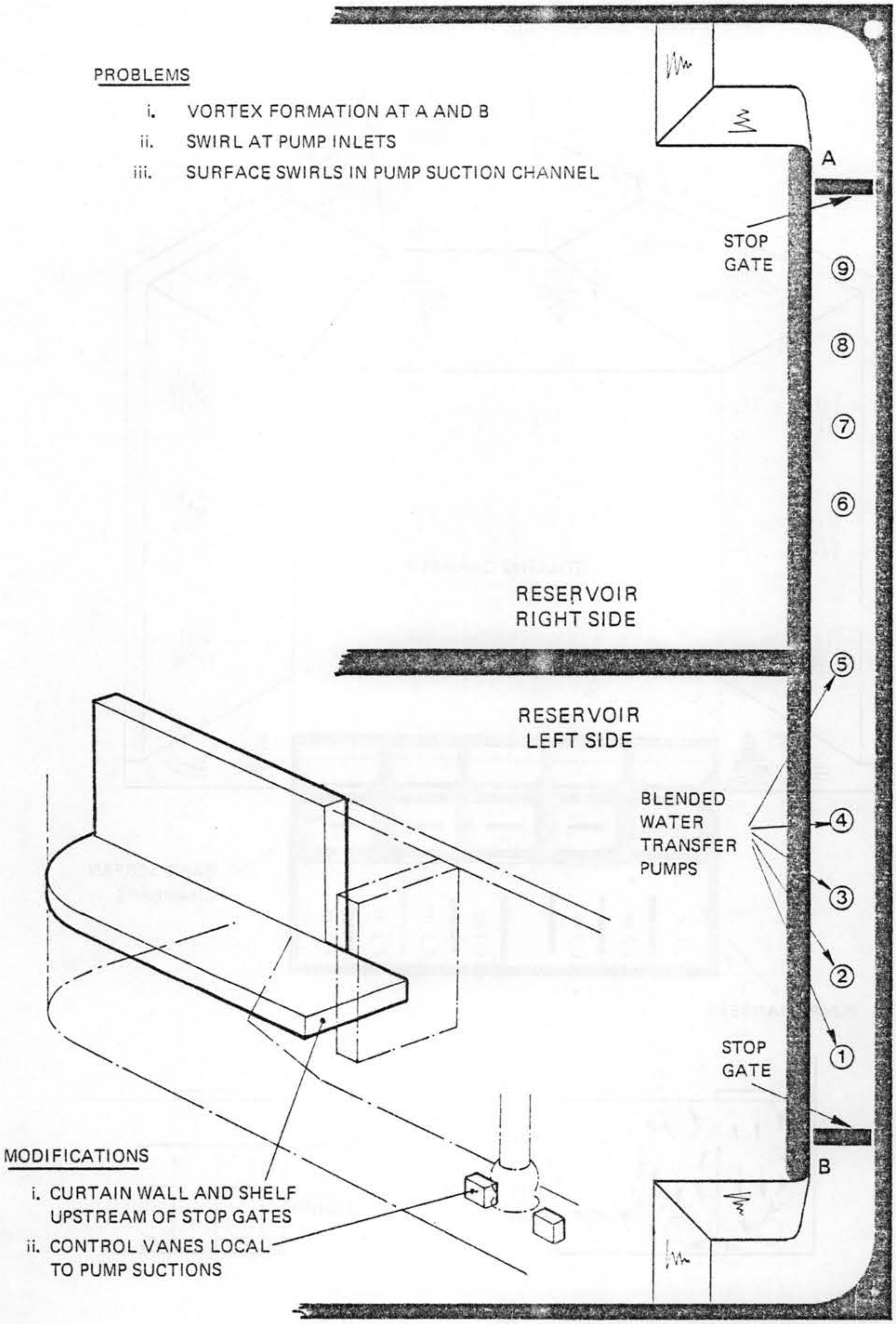


FIG. 5 BLENDED WATER INTAKE

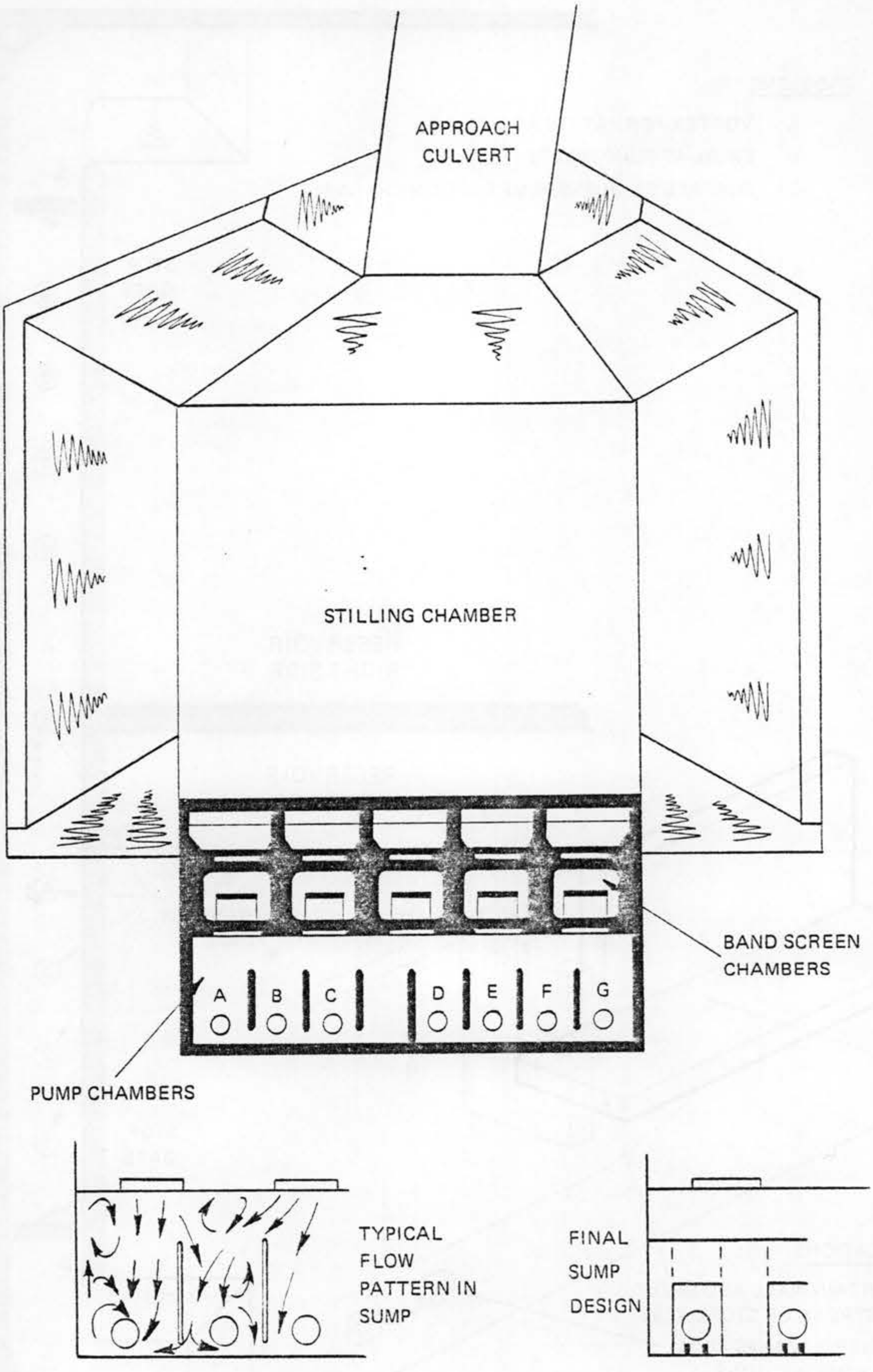


FIG. 6 S.W. INTAKE MODEL

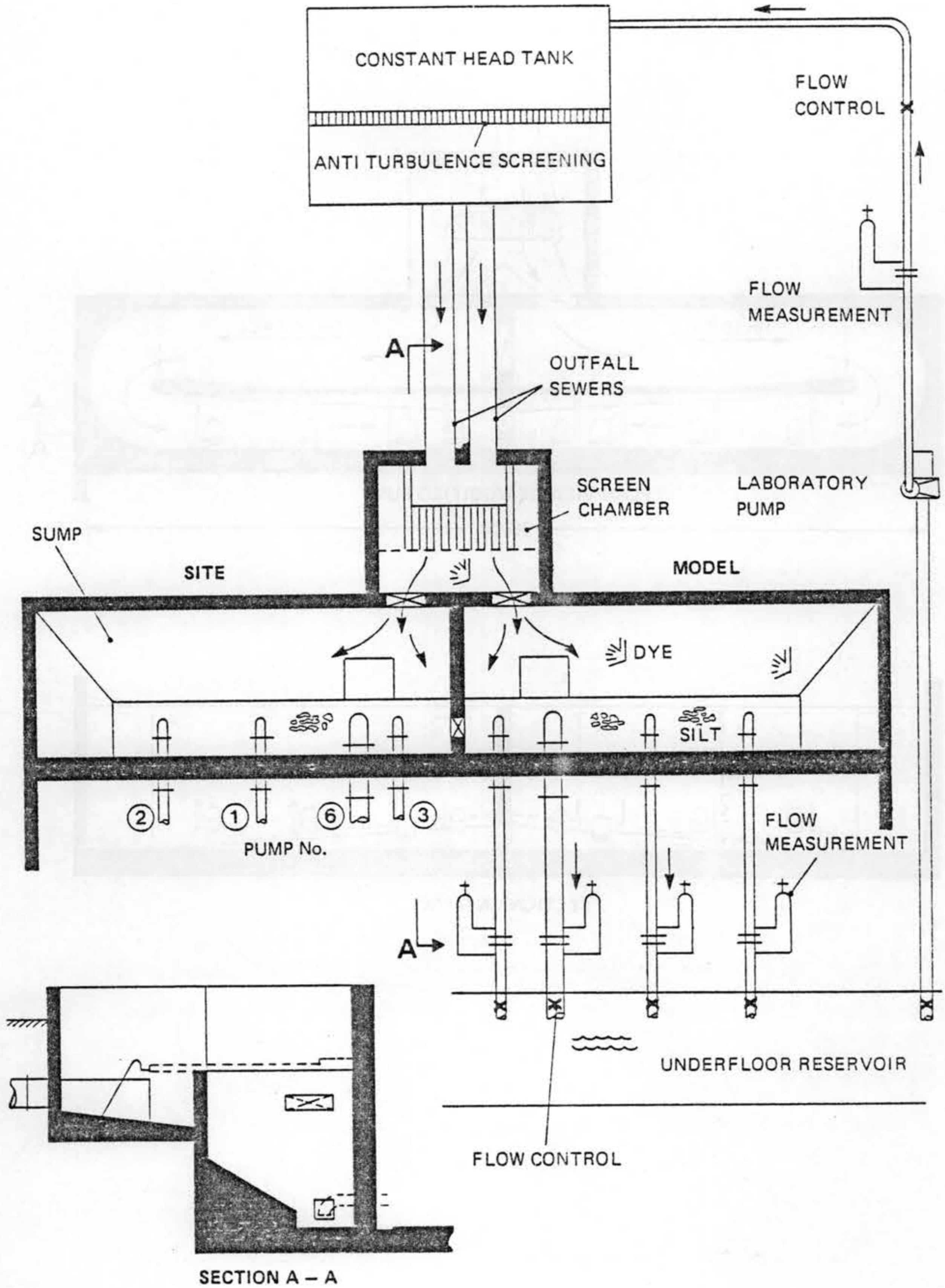


FIG. 7 ORIGINAL LAYOUT OF SEWAGE SUMP

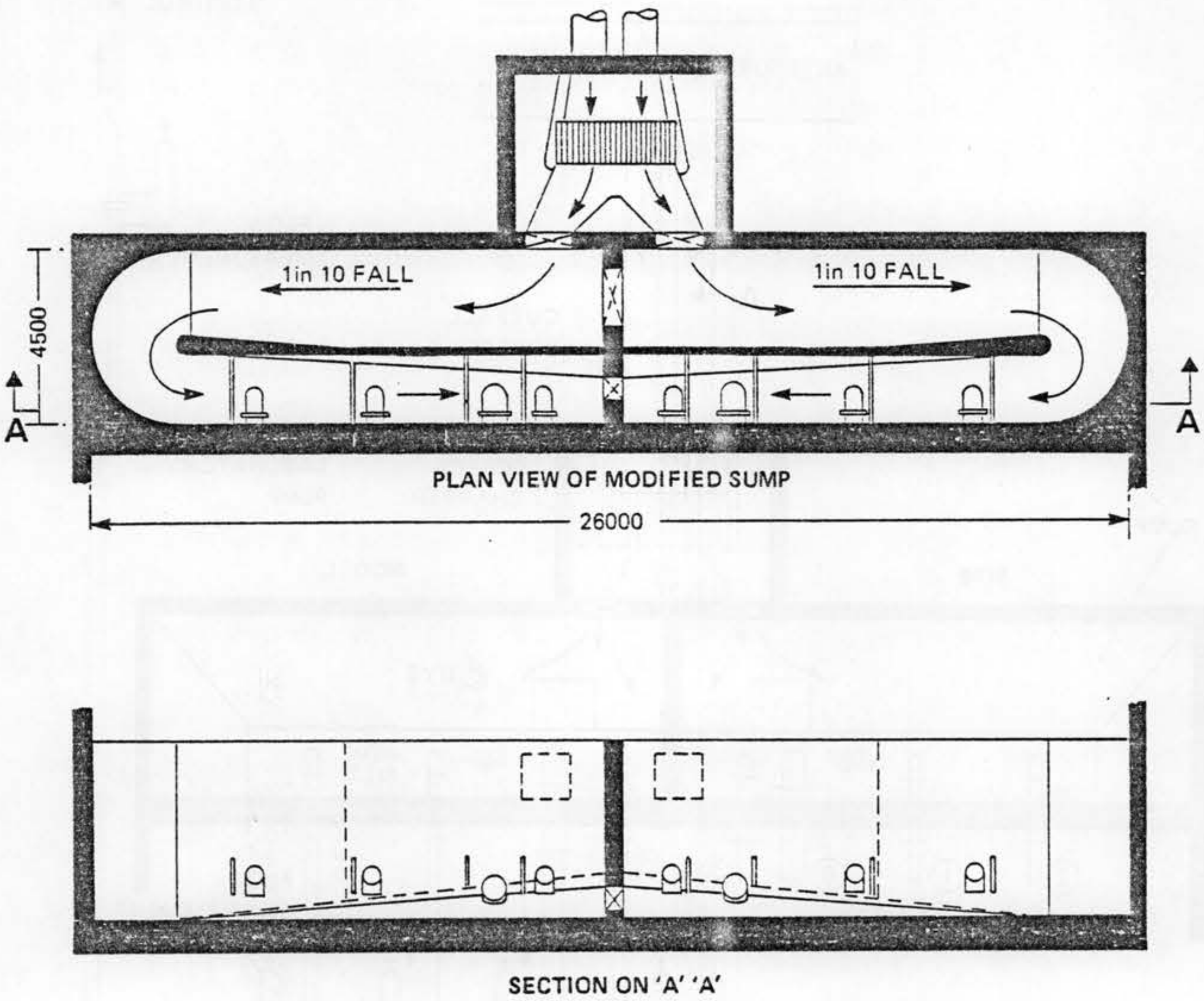


FIG. 8 FINAL LAYOUT OF SEWAGE SUMP

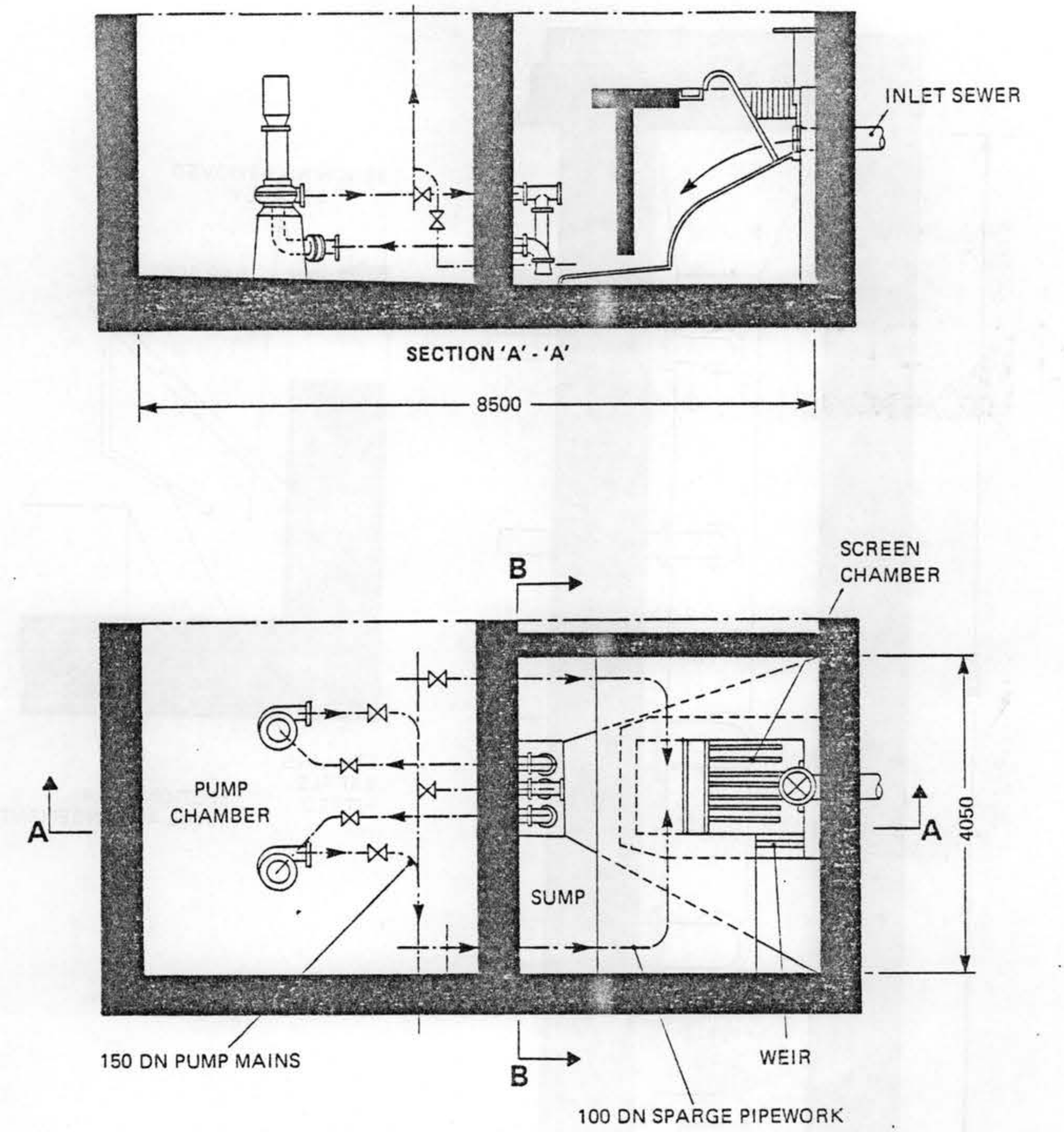


FIG. 9 LAYOUT OF PUMP CHAMBER

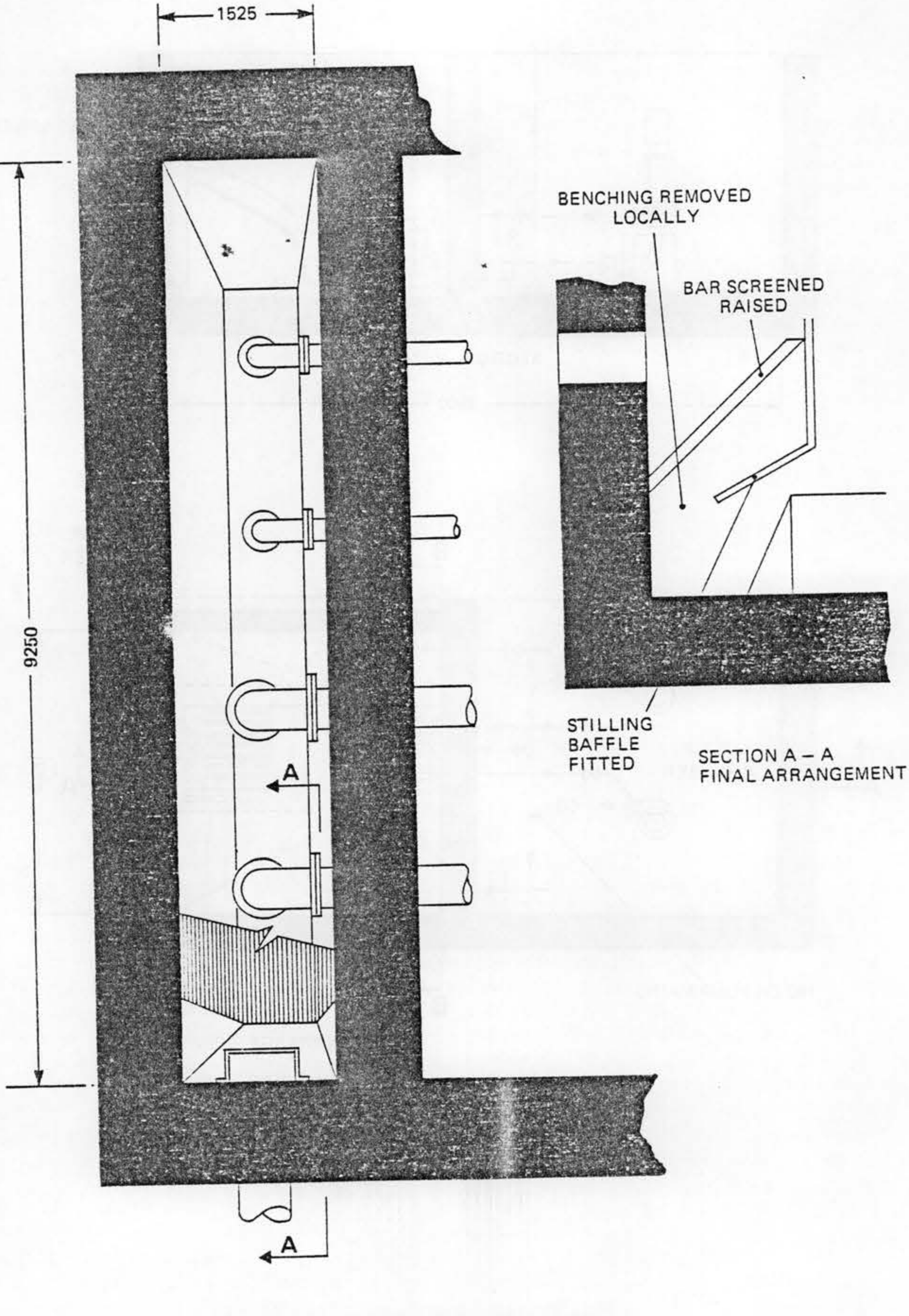


FIG. 10 LAYOUT OF SUMP

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Irrigation and Drainage Networks of 17 July Project - A Case Study

by

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INTRODUCTION

The "17 July Project", which is one of the biggest agricultural projects in Iraq is located in the alluvial plain of the Tigris river, south-west of Kut, approximately halfway between Baghdad and Basrah. The project covers a total area of about 25,000 hectares. Two distinct hydraulic systems, namely the 17 July canal system and the Ishtiraki canal system, exist within the area. The hydraulic systems are supplied by the Gharraf river which is an outgoing tributary of the Tigris river.

The project, which has been of significant benefit to the first stage of the 1969 development scheme, has made it possible to grow cereal crops over a few thousands of hectares. However, the project has been affected by increasing salinity of the soil due to the existing irrigation system being carried our without proper drainage facilities; hence crop yields have dropped.

In this paper the present irrigation scheme and the various networks involved in the project are evaluated and the salient features presented and discussed. Also, new proposals, including the design of essential components of a system based on the principle of optimal use of available water resources, which have wide seasonal fluctuations in quality as well as quantity, are presented the objective being to achieve harmonious overall development in the area concerned.

ENVIRONMENTAL CONDITIONS

Topography

The ground in the area has a small overall slope namely about 0.16% (0.0016) mean, a fugure taken from the available topographical survey (1).

Climatology

The climatic data are obtained from the meteorological station at Hai (2). The climate is of the sub-desert type: that is continental, arid and hot, with large daily and seasonal temperature variations and infrequent but violent and higly irregular winter showers (annual mean rainfall = 143mm); hot and dry winds blow from the north-west in late spring, often loaded with dust.

Temperature and humidity

The maximum temperature and minimum relative humidity occur in July and August and the minimum temperature occurs in January (coldest month) with maximum relative humidity. Temperatures below 0°C have been recorded in December, January and February.

Fortunately, the likelihood of frost with temperatures below -4°C is very limited during the critical months.

Winds

The dominant winds blow from the West to Northwest on average for 192 days a year. The sub-dominant winds blow from the East between May and October. The windy days per year for each direction are shown in Figure 1.

The maximum wind velocity occurs in June and July and peak velocity usually occurs at mid-day; while minimum wind velocity usually is experienced in November. The mean wind speeds at 6 a.m, 12 a.m and 6 p.m during June are 15.9 km/hr, 21.1 km/hr and 12.6 km/hr respectively, and during November 9.8 km/hr, 11.8 km/hr and 9.2 km/hr respectively.

Rainfall

The period within which rain falls is between october and May; there is no rain between June and september. However, the rainfall is non-uniform and has a wide variation from year to year and from month to month between October and May.

The monthly average rainfall (2) is presented in Table I, together with the range of the rainfall from October to May. Because of the wide range of rainfall in any month, the effective rainfall which is of use to the crops, has been defined as the rainfall which could occur with 80% probability.

Water resources

The project area is located on the west bank of the Gharraf river, which is controlled by a head regulator located immediately upstream of a barrage at Kut (160 km from Baghdad). The Gharraf river, 168 km long, serves several irrigated areas on both banks. it is equipped with four check structures which, to some extent control the water levels in the river. Two distinct hydraulic systems feed the Al-Shtiraki and the 17 July canals. The intake structures are located between two check structures.

Quality of water

The quality of water for the project is taken from the data obtained from Tigris water samples taken at the Kut barrage which is fairly close to the water supply to the project. The seasonal fluctuations of the water salinity are shown in Table II; the values are the average of the available data. The water is slightly saline and can be used as irriation water if drainage conditions are good. It is to be noted that the SAR (Sodium absorption ratio) values lie between 1 and 5 and so the water can be classified as being low-alkali hazard.

There is a large variation in water salinity during the year because of the influence of melting snow and rainfall and also due to the low run-off during the dry period. For that reason, monthly salinity values are used in the computation of the water-salt balance and the leaching requirement is also based on monthly figures instead of annual average figures.

Soil conditions

The project is located in the Mesopotamian lowland, which is characterized by alluvial soils. The alluvial material has been deposited in thick layers either in the form of medium-to-fine particles (loam-to-clay textures) corresponding to spreading phases generalized throughout the area in the course of several seasons, alternating with sudden flood phases depositing coarse to verycoarse particles (sandy loam to coarse sand textures). These variations result in a very intricate stratification of the soil and, often in very wide heterogeneity vertically as well as horizontally.

Although the water used for irrigation is not really saline, the summer season, especially, causes excessive salt concentration in the upper soil layers.

Because of the lack of drainage and the fact that there is absence of leaching, some important surface areas have become very saline. The salinity of the surface-layers of the soil varies widely, between 8 mmhos/cm and over 15 mmhos/cm (for the saturation extract).

The values of hydraulic conductivity vary widely, due to variations in soil texture and the effect of depth. The average hydraulic conductivity of the soil up to 250 cm below the ground surface is between 1.75 to 3.00 m/day, in the area which is classified as being good drainage. However, there are some areas which have a hydraulic conductivity of less than 1.75 m/day, a figure which is classified as representing poor drainage. However, areas of this type are small compared to the area as a whole.

IRRIGATION, DRAINAGE AND SALINITY

Crop consumptive use

The actual evapotranspiration of a given crop at a given stage of growth depends upon the climatic factors. The potential evapotranspiration is calculated on a monthly basis from the available climatic data by means of the Blaney-Criddle formula (3). The series of potential evapotranspiration values is shown in Table III and a sample of statistical distribution is represented in Figure 2. The statistical distributions reveal that the inter-annual variation in the monthly potential evapotranspiration is very high for the off-season months, namely November, March and April. However, the variation is lower in winter and summer.

For the project design reference to potential evapotranspiration had to be made and regard taken of the risks initiated by a temporary inadequacy between water demand and water availability. Several values of frequencies (probabilities) have been chosen to find the potential evapotranspiration, depending upon the statistical distribution.

The chosen frequencies are

- in January, February and July to September 0.6
- in October, December and March to June 0.7
- in November 0.8

As mentioned previously, some of the rainfall, ie: with 0.8 frequency, can be considered useful to the crop. Then the potential evapotranspiration is calculated by deducting the effective rainfall from the reference potential evapotranspiration. Table IV shows the monthly means of potential evapotranspiration that have been instrumental for calculating the actual evapotranspirations (crop consumptive use) for each crop with related corrective coefficients for the entire duration of growth cycle. Five crops can be suggested to be grown in the project, an area of 20% being allocated for each crop. The consumptive use figures for the five crops are shown in Table V.

Estimation of leaching requirements

The leaching requirement is the quantity of water brought in as a supplement to normal irrigation which allows leaching of the soil in such a manner as to maintain the salinity at a suitable level. The major objective of leaching and drainage consists of reducing the soil salinity to a level consistent with crop yields that yield optimum financial return on capital investment.

A salinity level is selected which is admissible for most of the crops planned for the project. Soil salinity of 3.7 mmhos cm for the saturation extract (6mmhos/cm for the field capacity) has been chosen as an objective.

The leaching requirement, LR, is given by

$$LR = \frac{D_{dw}}{D_{iw}} = \frac{EC_{iw}}{EC_{dw}} \qquad (1)$$

where,

D_{dw} = depth of drainage water,

D_{iw} = depth of irrigation water,

EC_{iw} = electrical conductivity of irrigation water, and

EC_{dw} = electrical conductivity of drainage water.

Electrical conductivity of the drainage water, EC_{dw}, represents a salinity level which is tolerant to the crops to be grown. The depth of irrigation water is equal to the sum of the consumptive use and the drainage water. The depth of irrigation water in terms of conductivity ratio can be given as

$$D_{iw} = \frac{EC_{dw}}{(EC_{dw} - EC_{iw})} D_{iw} \tag{2}$$

where

D_{cw} = depth of the consumptive use.

The leaching requirement is calculated using the monthly salinity of the available water as a base.

Irrigation - Frequency and depth

The irrigation frequency and depth depends upon the infiltration rate and the available water depth between the field capacity and the wilting point. An efficiency of 90% has been assumed for the various methods of irrigation which are to be used in the project. The total irrigation depth for various crops, including the consumptive use and the leaching requirements, are shown in Table V. The common irrigation depths will be used, namely 60mm, 90mm, 120mm, 150mm, 180mm and 240mm per month. The adjustable irrigation depths for the various crops are given in Table VI; however, the adjustable total depth for each crop is greater or equal to the required depth.

According to the suggested irrigation depth the salinity of the soil will be slightly different from that of the allowable value, but remains within the allowable value at the end of irrigation cycle. Depending on the field capacity of the soil and the consumptive use to the crops, the minimum irrigation frequency occurs in the summer for a period within 6 days. The maximum irrigation frequency occurs in winter for a period of about 30 days.

Irrigation canals: Discharge rates and design

To satisfy the required demand each 24 hours continuous discharge is required at the farm. The maximum weighted affective water depth desired is 4.2 mm/day, which is required particularly in the period January to May. Then the continuous discharge corresponding to that required water depth is:

$$\frac{4.2 \times 10,000}{24 \times 3600} = 0.5 \text{ l/sec per hectare.}$$

A field canal running between two farms, 50 hectares each, can be used for irrigation. The productive area of each farm can be considered as 85% of the total area. The canals can be lined with concrete, to minimize the loss of the rare water resource in the project.

Lining all irrigation canals is a costly operation; however, this practice will cover the cost of seepage water and the maintenance of the unlined canals. A conveyance loss of 10% is considered in the design of the canals.

The design discharge of the farm canal is 50 l/sec. The network design is based in the Manning formula. The Manning roughness coefficient is taken as 0.014, the side slope of the cross-section being 1:1. The canal bottom slope is the range of 0.01%. Table VII lists typical canal sections and the estimated discharges which are needed in the distribution of water from the intake structure on Gharraf river to the various farms.

Drainage

Field drains

The design of field drains should be based completely on experience and observations made on the existing networks under similar soil and water conditions and also on appropriate theory. In the arid conditions prevailing in southern Iraq, irrigated land requires intensive field drainage for permanent control of salinity.

The drainage water reaches a depth based on a distance of 100 mm per month (3.4mm per day), obtained from Tables V and VI. The drain level is fixed at a depth of 1.8m to prevent resalination of the root zone, which is in the range of 1.2m, due to capillary rise.

The calculation of the spacing of field drains can be made according to the steady-state formulae or, alternately, to the non-steady-state formulae. The Hoghoudt formula and the Glover-Dumm equation will be used for the steady-state and non-steady-state respectively. The Hoghoudt formula (3) is:

$$S^2 = \frac{8K_b dh}{q} + \frac{4K_a h^2}{q} \quad (3)$$

where;

s = the spacing between two field drains,

q = the discharge of the field drains,

h = the maximim height of the water table above drains level,

d = the equivalent depth of the impermeable layer below drains level,

k_a and k_b = the horizontal hydraulic conductivity above and below drain level respectively.

However, the Glover-Dumm (4) equation, for an initial water table that is not completely flat but has the shape of a fourth-degree parabola, can be given as:

$$L = \pi \left[\frac{KD}{\eta} t \right]^{1/2} \left[\ln 1.16 \frac{h_o}{h_t} \right]^{-1/2} \quad (4)$$

where,

L = the spacing between the drains,

K = hydraulic conductivity of the soil,

η = drainable porosity of the soil,

h_o= initial height of water table above the drains level, taken midway between the drains.

h_t= height of the water table above the drains level, taken midway between the drains at time t after the drainage cycle starts,

D = average depth of the flow = $d + \frac{h_o}{2}$,

d = the equivalent depth of the impermeable layer below drains level.

The drainage criteria which are used in the calculation of spacing are given in Table VIII. Accordingly the calculated spacing from the Hoghoudt formula and the Glover-Dumm equation are 150 m and 160 m respectively. Then, the field drain spacing can be 150 m.

CONCLUSION

A case study is made of the "17 July Project" in Iraq. The present irrigation practice and networks of the project are evaluated. The various environmental conditions of the project are presented and discussed. New proposals including the design of essential components of the system are made, based on the principle of the optimal use of the available water resources.

References

1 Sabti, N.A., et al., "Evaluation of 17 July Project". Report submitted to the Ministry of Irrigation. Iraq, 1978.

2 "Meteorological data at Hai station". Unpublished report.

3 Luthin, J.N., "Drainage Engineering", John Wiley & Sons, INC, 1966.

4 Dumm, L.D. "Transient Flow Concept of Sub-Surface Drainage", Transactions of the American Society of Agriculture Engineers, 1964, 7 (2).

Table I; Rainfall Data

Rainfall (mm)	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Year
Average Value	3	19	23	28	20	23	20	7	143
Lowest Value	0	0	0	0	0	0	0	0	-
Highest Value	20	134	70	98	64	117	102	30	-
Effective Value (P=80 %)	0	0	9	11	5	6	3	0	-

Table II, Seasonal fluctuations of water salinity

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	mean
Water Salinity (ppm)	460	406	423	447	464	380	400	280	306	388	413	436	400
Electrical* Conductivity (mmhos/cm)	.72	.64	.66	.70	.73	.59	.63	.44	.48	.61	.65	.68	.63

* EC (mmhos/cm = Salinity/ppm)/640

Table III, Series of potential evapotranspiration values.

Oct.		Feb.		Jun.	
154	165	72	44	295	273
172	165	61	76	265	276
178	155	67	69	300	287
181	158	59	69	285	276
178	174	60	72	278	264
163	177	62	70	296	291
179	147	68	69	283	284
149	184	64	75	296	300
164	167	65	49	271	258
176	172	67	64	289	270
160	180	77	71	271	290
177	186	68	80	278	287
172	164	68	68	285	264
176	159	74	56	291	278
166	183	75	80	279	270
159	177	65	59	255	286
169	172	63	60	271	298

Table IV, Potential evapotranspiration data

	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Year
Reference P.E.T. (mm)	173	112	73	70	78	117	167	242	287	312	298	235	2164
Effective rainfall (mm)	0	0	9	11	5	6	3	0	0	0	0	0	34
P.E.T (mm)	173	112	64	59	73	111	164	242	287	312	298	235	2130

Table V, Consumptive use and total irrigation depth for various crops

Crop	Cropping Percentage	Consumptive Use (mm)											
		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Alfalfa	0.2	140	100	60	40	60	90	160	230	270	245	240	200
Wheat	0.2	0	40	45	65	120	125	45	0	0	0	0	0
Barley	0.2	60	80	70	75	60	20	0	0	0	0	0	0
Berseem	0.2	30	70	70	75	80	105	160	200	0	0	0	0
G. Barley G. Maize	0.2	0	45	60	75	20	0	0	95	260	275	0	0

Table V (continued)

Crop	Cropping Percentage	Total irrigation depth (mm)											
		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Alfalfa	0.2	193	134	81	55	83	119	215	289	345	327	323	273
Wheat	0.2	0	54	88	133	174	165	61	0	0	0	0	0
Barley	0.2	83	108	95	103	83	26	0	0	0	0	0	0
Berseem	0.2	41	94	95	103	111	139	215	251	0	0	0	0
G. Barley G. Maize	0.2	0	61	81	103	28	0	0	119	332	367	0	0

Table VI, Adjustable irrigation depth

Crop	Irrigation Depth (mm)											
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Alfalfa	240	120	150	150	150	180	240	240	240	290	240	240
Wheat	0	60	90	150	180	150	60	0	0	0	0	0
Barley	90	120	90	120	60	60	0	0	0	0	0	0
Berseem	60	90	90	90	120	150	240	240	0	0	0	0
G Barely & Maize	0	60	90	120	60	60	60	120	240	240	120	0

Table VII, Typical Canal Sections

Type	Max. Discharge (m^3/sec)
b=0.4 , h=0.7m , f=0.5m	0.22
b=0.7 , h=1.0m , f=1.0m	0.60
b=0.7 , h=1.2m , f=1.0m	0.92
b=1.0 , h=1.0m , f=1.5m	0.75
b=1.0 , h=1.2m , f=1.5m	1.12
b=1.0 , h=1.5m , f=1.5m	1.71
b=2.0 , h=1.2m , f=2.0m	1.86
b=2.0 , h=1.5m , f=2.0m	2.73
b=2.0 , h=1.8m , f=2.0m	3.78
b=2.0 , h=2.3m , f=2.0m	6.50
b=2.0 , h=2.5m , f=2.0m	7.50

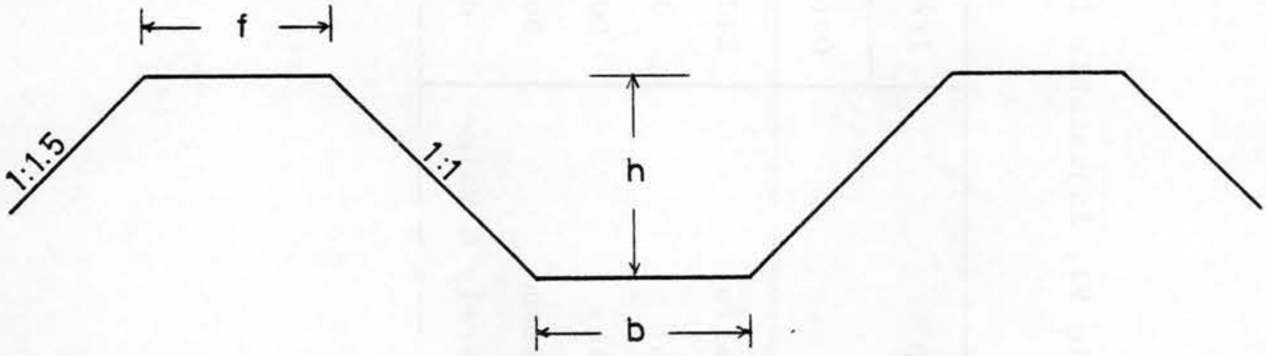

Table VIII, Drainage Criteria

Depth of impermeable layer	= 5m
Depth of drains	= 1.8m
Horizontal hydraulic conductivity, K_a	= 4.8m/day
K_b	= 4.3m/day
Depth of water level	= 1.2m

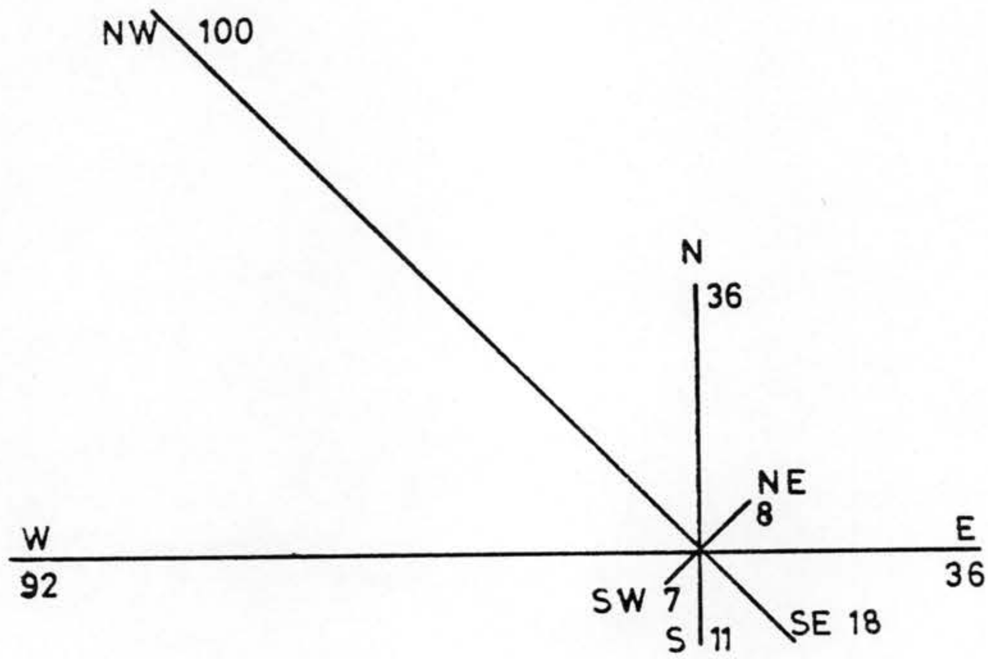

Figure 1 Number of windy days per year for each direction

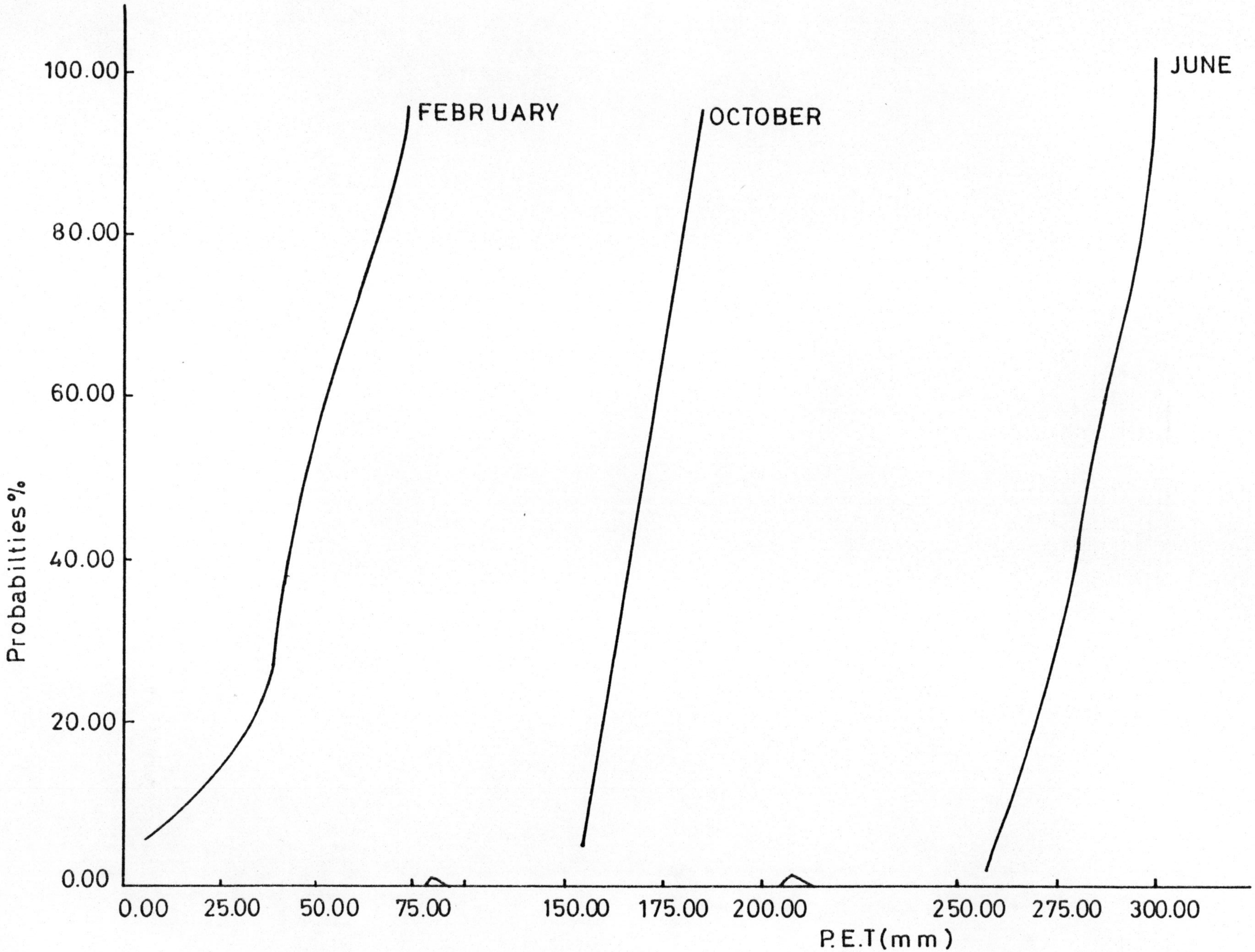

Figure 2 Cumulative probabilities curve for monthly P E T