

FINAL PROJECT REPORT

LABORATORY INVESTIGATION

**RIVER ROUGE COMBINED SEWER
OVERFLOW CONTROL FACILITY**

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INTRODUCTION

The city of River Rouge is planning to construct a combined sewer overflow (CSO) retention basin as part of an overall strategy for controlling combined sewer overflows during periods of high stormwater runoff. The lower chamber of the retention basin will be filled by gravity from two influent sewers; once the capacity of the lower basin is approached, up to five stormwater pumps will be used to lift the stormwater to an upper chamber. Although the retention basin is designed with six pumps, only five of them are intended to be operating during the normal maximum design condition. If the upper chamber is subsequently filled, the stormwater will be discharged to the Rouge River by flowing over an effluent weir and through two discharge pipes to the river.

A physical hydraulic model was constructed to study the nature of the flow within the upper chamber. The purpose of this model was to study the discharges from the stormwater pumps, the flow over the effluent weir into the discharge pipe, and the flow through the discharge pipe to the Rouge River outfall and into the river. A specific objective was to examine the possibility of air entrainment in the flow over the weir for a variety of discharges and river elevations. The testing sequence included the following components:

- Establish a flow rate through the system that corresponds to a particular combination of stormwater pumps in operation;

- Record visual information on the nature of flow in the model as it may relate to potential problems with system operation.

If potential problems were indicated, the physical model was used to investigate modifications to be incorporated in the final design. This report documents the testing procedures and modifications that were investigated.

CONCLUSIONS AND RECOMMENDATIONS

- The flow velocity through the stormwater pump discharge columns was sufficiently high that with the discharge elevation in the design, the flow would impact upon the roof of the upper chamber. Since the pump motors are to be located on the floor above the chamber, this could be a problem unless the seal around the pump shafts is water tight. This potential problem can be avoided by increasing the diameter of the pump discharge columns.
- During normal operation at high discharge capacity (five stormwater pumps in operation) stormwater will intermittently spill over the emergency overflow weirs. This occurrence is attributable in part to the relative elevations of the proposed effluent weir and the overflow weirs and also to the proximity of and flow through the closest stormwater pumps. Design changes to both the stormwater pumps (increase in discharge diameter) and to the effluent weir configuration should alleviate this situation.
- The proximity of the effluent weir to the entrance to the discharge pipes led to significant air entrainment into the discharge pipes. The origin of this air is entrainment into the plunging flow over the weir. The exit section between the effluent weir and the entrance to the discharge pipes is too small to allow an opportunity for this air to escape before it flows into the discharge pipes. Although the air entrainment is worse at low Rouge River levels, the presence of a free surface within the discharge pipes allows this air to escape during passage through the discharge pipes. At highest river elevations, the backwater effect through the discharge pipes increases water levels within the exit section and reduces the plunging of the flow over the weir and consequently, the air entrainment is much reduced. The worst conditions are therefore at intermediate river elevations (and different for each pipe) at which the river level is just above the crown of the pipe at the river discharge. Air entrainment problems were more pronounced in the eight foot diameter discharge pipe but were similar in nature in the six foot pipe. Under these flow conditions, air was observed to pass through the pipes in large bubbles, leading to surges in the flow. This surging behavior and air entrainment in general was worse at higher discharges but was still observed in the eight foot diameter pipe when only one stormwater pump was in operation.
- A modification to the model was made to examine the possibility for air release by installing manholes in the discharge pipes. The location of these manholes is constrained by the fact that the hydraulic grade line of the flow through the pipes is above ground elevation over much of the pipe length at high discharges. Placing these manholes at the farthest feasible downstream location resulted in significant removal of the air from the pipes but not all of it could be removed. The surging problem noted above was, however, transmitted to the manhole and prototype fluctuations of water level up to several feet can be expected in the manholes. This will

result in intermittent spilling of stormwater onto the ground surface. Another potential problem noted by the model function was the possibility of significant foaming in the manholes, probably resulting in foam spilling onto the ground surface through the manhole.

- Additional investigations in the model included changing the effluent weir configuration. Weirs located just downstream of the bar screens were effective in decreasing air entrainment. Although some air still passes through the discharge pipes, it is more in the form of discrete, small bubbles and the surging behavior noted with the original design was not observed. The volume of air passing down the discharge pipes is also significantly reduced, primarily due to the fact that the exit section was increased in volume sufficient to allow air to escape prior to its flow into the discharge pipes. When the downstream end of the eight foot diameter discharge pipe is just submerged, a substantial portion of the air flowing through the discharge pipe is entrained by intermittent air core vortices forming at the entrance to the discharge pipe. An investigation of the security bar grates proposed for the downstream end of the discharge pipes indicates that they will not have any discernible impact on the discharge to the Rouge River. The amount of water level increase in the exit section of the upper chamber is so small even at the highest flow rates that it cannot be reliably detected.

- The Rouge River water surface elevation necessary to create a backwater condition within the retention basin upstream of the re-designed effluent weirs is estimated to be on the order of 578 ft. This is above the 100 year flood elevation estimated for the Rouge River; thus it appears that backwater effects from Rouge River flooding should only occur under extreme high water conditions.

GENERAL SYSTEM DETAIL

The stormwater retention basin upper chamber is to be constructed at an inside diameter of 140 feet. There are six low lift pumps to be spaced at equal 60° intervals around a 48 foot radius to lift the stormwater from the lower to the upper chamber. The pump motors reside on the floor above the upper chamber with drive shafts extending down through the pump discharge column to the impeller located near the bottom of the column. The discharge column was designed to be 42 inch diameter with a flare at the exit to about 68 inches. Each pump has a fixed capacity of approximately 114 cfs and the pump operation is to be sequenced during normal operation so that pumps on opposite sides of the chamber are operating insofar as possible. The system has been designed so that a maximum of five pumps are in operation simultaneously for a total system capacity of about 570 cfs; sufficient hydraulic capacity has been provided to handle a flow associated with the inadvertent operation of all six pumps.

Stormwater filling the upper basin passes through bar screens and out over an effluent weir which was designed with a crest elevation of 580.3 feet. Water spilling over the weir drops into a small exit section before entering one of two discharge pipes mounted in the caisson wall of the retention basin. The larger discharge pipe has a diameter of eight feet and an invert elevation of 566.25 ft while the smaller six foot diameter pipe has an invert elevation of about 572 ft at the exit to the retention basin. The six foot diameter pipe is an existing one and there is some uncertainty regarding the exact elevations and pipe slopes. These pipes run approximately 800 feet to the Rouge River where they pass through a headwall with a security grating and the discharge flows into the river. The eight foot pipe has zero slope resulting in a crown elevation at the river of about 574.25 feet while the existing six foot diameter pipe has a crown elevation at the river of about 576 feet. The Rouge River water surface is subject to fluctuation with the following estimates of water levels:

Low Water	571.9 feet
10 year Water Level	576.5 feet
25 year Water Level	576.9 feet
50 year Water Level	577.4 feet
100 year Water Level	577.7 feet

Thus it is clear that conditions on the outlet of both pipes may be either free flowing or submerged, depending on the Rouge River level. However, the eight foot diameter pipe should have a submerged outlet more often while the six foot pipe will be submerged only during higher water levels.

MODEL DESCRIPTION

Modeling Criteria

Physical models to examine free surface flows are performed using Froude number similarity, which fixes the relations between model and prototype conditions once the physical model scale has been selected. Dynamic similarity requires keeping all Froude numbers defined by $V/(gL)^{1/2}$ equal in the model and prototype, where V refers to any representative fluid velocity, g the acceleration due to gravity, and L is any system length. The relations between prototype and model parameters are related to the scale ratio L_r which is the geometric ratio between any length in the model and the corresponding one in the prototype ($L_r = \text{Length}_{\text{model}}/\text{Length}_{\text{prototype}}$). For a Froude scaled model, assuming the same gravity in model and prototype, the following relations must hold:

PARAMETER		RATIO
Length	L_r	$L_r = 1:8$
Velocity	V_r	$L_r^{1/2} = 1:2.83$
Discharge	Q_r	$L_r^{5/2} = 1:181$
Time	T_r	$L_r^{1/2} = 1:2.83$

The critical factors with respect to model testing facilities are the model size and discharge which are related by the Froude scaling laws presented in the table above. If the scale ratio is too small, both surface tension and viscous effects become too great in the model. This consideration generally fixes the minimum model size required to avoid distortion of the model flow due to viscous effects. Padmanabhan and Hecker (*Scale Effects in Pump Sump Models*, Journal of Hydraulic Engineering, Vol. 110, 11, November, 1984, pp. 1540-1556) suggest that a minimum Reynolds number (based on the definition $Re = Q/dv$ with d the depth of flow and v the kinematic viscosity) of about 30,000 be maintained in pump sump models to correctly reproduce the effects of viscosity on the flow behavior.

Model Testing Facilities

The model study was conducted in the Civil Engineering Hydraulics Laboratory located in the G.G. Brown Building at the North Campus of the University of Michigan. The model was constructed in a model test basin which was 20 feet wide by 40 feet long. A pump with a maximum discharge capacity of approximately 2400 gpm is available in the basin to provide the necessary model discharge. Water is circulated from a sump, through the pump and into the model, through the discharge pipes into the tailwater box, and back into the sump.

Model Construction

The physical model was constructed at a scale ratio of 1:8. This general size is dictated by the capacity of the model pump as well as the desire to keep the model Reynolds numbers as large as possible. The exact model size was controlled by the necessity to model the circular discharge pipes with available PVC pipe.

The extent of the physical model is indicated in Fig. 1. The intention of the particular choice of model extent was to model the pumps and other details of the discharge chamber and effluent weir which have an effect on the hydraulic aspects of the flow entering the discharge pipes. The four storm water pumps in closest proximity to the effluent weir were modeled as well as bar screens and other local geometry. The back side of the chamber was not modeled because it was not felt to have a direct influence on the flow at the effluent weir but provisions were made in the model to provide for the flow that could come from the two pumps in the portion of the chamber that was not modeled.

Blueprints provided by Sigma Associates gave the detailed dimensions to which the model was constructed. The model was constructed primarily from plywood and was painted to provide a smooth finish. The outer wall of the model (inside of the retention basin caisson) was constructed by bending 1/4 inch plywood and attaching to a formwork consisting of segments of straight 3/4 inch plywood. Other detail inside the upper chamber such as stairwells, the section at the center where basin influent drops into the lower chamber and the effluent weirs were also constructed from plywood. Internal columns were formed from PVC pipe as close to the correct diameter as possible. A surface baffle proposed to eliminate the passage of floating scum was installed upstream from the effluent weir; the location is indicated in Fig. 1. Similar baffles located upstream of the emergency overflow weirs were not included on the blueprints and were omitted in the original model construction. However, subsequent discussion with Sigma personnel revealed the intention for their presence in the retention basin design and these were included for later model tests. The portion of the videotape showing flow at the emergency overflow weirs was recorded early in the testing program before these baffles were added. In any case, there was never any significant flow over the emergency overflow weirs during the model tests so the absence of the baffles should not have any impact on the model performance.

160 feet of the discharge pipes were modeled due to space constraints in the hydraulics laboratory; the effect of the actual pipe length was accounted for by estimating the head loss in the prototype pipe and adjusting the hydraulic grade line elevations in the model to be consistent. These pipes were discharged into a plywood box in which water levels could be controlled by means of an adjustable weir. By regulation of the water level in this discharge box, the effect of an appropriate Rouge River elevation could be simulated.

The flow was established by means of a mixed flow pump with a capacity of approximately 2400 gpm. Water was pumped from a sump into a piping system constructed from eight inch PVC pipe with five, four inch branches which allow the distribution of flow to the four modeled storm water pumps and the remaining flow which came through the influent drop shaft at the center of the basin (see Figure 1). This flow entered the model through a series of holes drilled through the plywood sheeting forming the panels on either side of the influent drop shaft at the back walls of the model. The location of these holes can be seen in the portion of the videotape that shows the model without water flowing. This inflow was located sufficiently far from the effluent weir that the exact discharge details would not influence the weir flow. In fact, there was no indication that the pump discharges at locations other than the two closest ones to the effluent weir would have any impact on the discharge from the retention basin and these only influenced flow at the emergency overflow weirs as discussed below.

The total discharge was regulated by means of a gate valve connected to the main eight inch line while the distribution of flow was controlled by individual valves on each of the branch lines. The total discharge was metered upstream of the branches in the eight inch pipe by means of an installed pipe orifice meter and separate orifice meters were installed in each branch line.

The four stormwater pumps modeled involved a discharge from a four inch supply line through the floor of the model. This supply line was connected to a six inch diameter PVC pipe simulating the 42 inch diameter pump discharge column. This provides a slightly larger than required diameter for geometric similarity. Fiberglass expansions were fabricated in the shop and were affixed to the top of each riser to reproduce the correct pump discharge diameters and the flare in the discharge line as indicated in the cross sectional drawing in Figure 2 (Figure 3-c is a photograph of one of these columns). Relative vertical elevations of important system components of the model as constructed are presented in Figure 2. The elevation of the top of these expansions was at a prototype elevation of 581.3 relative to a floor elevation of 565.2. The relationships between the top of the pump columns, the effluent weir and the emergency overflow weirs are critical. The effluent weir must be at as high an elevation as possible to maximize basin storage, while the pump columns must be high enough to minimize discharge back down them if the pump is not operating. Finally, the emergency overflow weir must be high enough not to overflow during normal operation, but with sufficient capacity to pass the maximum discharge should the bar screens become completely blocked. Subsequent discussions with Sigma personnel indicated that the elevation of the top of the pump discharge columns was intended to be greater than 581.3 and of the order of the elevation of the emergency overflow weirs. Since this discrepancy was established after the completion of the testing and because the design of the pump discharge columns is to be altered, the correct

configuration was not tested. The impact of this elevation difference on the internal hydraulics of the retention basin is discussed below in the presentation of model results.

It should also be noted that the elevations depicted in Figure 2 represent the conditions of the original design. Once a decision was made to alter the configuration of the effluent weir, the elevation of the emergency overflow weir was changed as well.

The bar screens were specified to be of 3/4 inch thickness with 1.5 inch openings. Calculations over the range of possible basin flow rates indicated that the head loss through the bar screens would be minimal under any system discharges and that these would have only a minimal influence on the flow. Consequently, the only attempt to model the bar screens was to estimate the head loss and to use a screen that provided roughly the same head loss for a given flow rate.

The effluent weir was modeled under the assumption that the prototype thickness was 16 inches (as scaled off the blueprints) and that the upper edges had square corners. A sectional model (not accounting for possible end effects) was tested in a separate channel to determine the weir coefficient C as defined in the equation:

$$Q = C L H^{3/2}$$

with Q in cfs, L the length along the crest, and H the weir head in feet. The calibrated weir coefficient was 3.3 which is essentially the value for a sharp crested weir. This coincides with observations that the flow over the weir separated from the upstream corner and did not reattach to the upper face.

Emergency overflow weirs are provided in the basin design to divert flow to the discharge pipes in the event that the bar screens become plugged or the flow capacity over the effluent weir is decreased in any manner. The original design on these weir crests indicated an elevation of 583.9 and these were included in the model although it was anticipated that there should be no flow over them under normal operating conditions.

A photograph that indicates the majority of the model is presented in Fig 3a. More detailed features are presented in Figs. 3b-d. Fig. 3-b shows the detail at the effluent weir including the entrance to the discharge pipes. Fig. 3-c shows one of the stormwater pump discharge columns while Fig. 3-d indicates the tailwater discharge box (with the tailwater level control weir removed) with the discharge pipes entering.

Instrumentation

All system discharge rates were measured using sharp-edged pipe orifice meters with pressure differences measured by means of water-air differential manometers. Velocity measurements were obtained with a

mini-propeller meter. Point gauges were used for measurements of relative water surface elevations. Many of the observations of flow conditions within the basin and in the discharge pipes were visual in nature and were recorded on video tape and still photographs. A videotape recording the nature of the flow for various tests performed was obtained and an edited version of the tape will be provided along with this project report.

TEST CONDITIONS

The retention basin discharge can only be at discrete flow rates associated with between one and five pumps in operation. The specific combination of pumps that could be in operation at any one flow rate is also a variable since there is no specified sequencing of the pump operation other than a general requirement that an attempt will be made to distribute the flow spatially so that, for example, if three pumps were operating, these would not be adjacent to each other. This possibility leads to a large number of different flow configurations that could be studied with various combinations of pump operation and variable Rouge River tailwater elevation. Testing generally proceeded by attempting to identify worst case flow conditions with respect to the phenomenon being studied; this was then studied in detail with subsequent examination of less severe flow conditions to verify system performance. Flow downstream from the effluent weir was independent of the origin of the flow into the upper chamber, so the modeling of flow through different pump combinations was only important when considering internal chamber hydraulics. Similarly, there did not appear to be any backwater effect from the Rouge River upstream of the effluent weir unless extremely high river levels were modeled so studies of internal chamber hydraulics and the hydraulics of the discharge to the river could be essentially uncoupled from each other.

The worst conditions for flow consisted of the highest discharges in all cases. Therefore most initial testing was conducted at a flow rate that corresponded to five pumps in operation (570 cfs prototype discharge). Once the flow behavior was studied at this condition, additional confirmation runs for combinations of 1, 2, 3, or 4 pumps in operation were generally conducted depending on the specific testing objectives.

During the testing, it was observed that worst case conditions for air entrainment in each discharge pipe corresponded to conditions where the downstream end of the respective pipe was barely submerged. The videotape shows that a small change in submergence (going from a tailwater level just below the crown of the pipe to less than a foot above) at the downstream end of the eight foot diameter pipe has a significant impact on the nature of the air flow through the pipe. Therefore, most of the testing was conducted by adjusting the tailwater level to a condition where one or the other of the discharge pipes was in a just submerged condition. Air entrainment did not create adverse flow conditions if the discharge pipe was unsubmerged (since the presence of a free surface in the discharge pipe would allow air escape) so very low tailwater levels were not generally studied. Confirmation runs at high tailwater levels corresponding to flood conditions in the Rouge River were made to verify that no additional problems were associated with flows with high downstream submergence. It should also be noted that the nature of the impacts observed in the videotape with a small change in downstream submergence did not persist when the new effluent weir configuration was tested and the nature air discharge was insensitive to small tailwater variations.

RESULTS

Internal Chamber Hydraulics

The physical model was tested to examine the general nature of the flow within the upper chamber. The highest prototype discharge (570 cfs from five pumps) was tested first with the anticipation that any problems would be most apparent at the highest flows. There were two phenomena that were readily apparent at this high discharge condition that may interfere with the function of the basin, both of which are related to the discharge from the stormwater pumps. There is a significant bulge in the free surface above the pump discharge outlets that is associated with the velocity through the 42 inch diameter conduits. Figure 4 indicates the nature of this bulge under the flow condition where five stormwater pumps are operating. If only one pump was in operation, the super-elevation of the water surface above that pump was several feet (prototype) higher than the water level above a non-functioning pump. Since the bulge in the water surface is intermittent, it is difficult to quantify precisely, but measurements of an "average" surface super-elevation indicated a value of about 2.2 ft and a maximum (defined as the maximum water surface level observed over about a 10-20 second period) super-elevation of about 2.8 feet. Computing the velocity head for a flow rate of 114 cfs in a 42 inch diameter conduit yields a value of 2.2 feet, indicating that this super-elevation is directly associated with the distance required to dissipate the vertical velocity in the pump column. The fact that the rise is associated with a velocity for a 42 inch diameter as opposed to the 68 inch diameter at the outlet expansion indicates that the flow expansion is not gradual enough to prevent flow separation, which is realistic from basic fluid mechanics principles (a maximum expansion angle of 5-10 degrees is necessary to avoid separation). During an experiment with five pumps in operation, the average super-elevation of the water surface was to an elevation of about 586.3 and a maximum elevation of about 586.9. The ceiling from the floor above the upper chamber was estimated to be at an elevation of 586.7 - 587 from details provided in the blueprints so this surge would be sufficient to contact the ceiling. This situation can be alleviated by increasing the diameter of the pump discharge pipe. For example, if a 50 inch diameter conduit were used instead of the proposed 42 inches, the velocity head would be reduced from 2.2 feet to 1.1 feet which should be sufficient to avoid contact with the ceiling under all discharge conditions.

The second observation was also related to the disturbed water surface from the stormwater pump discharges. Water was observed to flow over the emergency overflow weirs under flow conditions when all five pumps were in operation. This flow was somewhat intermittent and was related to disturbances in the water surface from the pump discharge. Flow over the emergency weirs was not continuous in space but would tend to occur along isolated sections of the weir and was most frequent at the corners near the ends of the weir. Flow was almost continuous in time at some point along the weir length. A computation of the head behind the

weir verifies that the water level should be very close to the weir crest. Using the effluent weir crest elevation of 580.3, a crest length of 25.6 feet, a weir coefficient of 3.3 as discussed above, and a flow rate of 570 cfs, the water surface elevation behind the weir should be 583.87 which is almost equal to the 583.9 ft elevation of the emergency overflow weir crest. This situation was alleviated, however, by changing the effluent weir configuration as discussed further below.

Since the model was constructed with the elevation of the top of the pump discharge columns somewhat lower than intended, there would be some differences in the above observations if the columns were terminated at a level consistent with the top of the emergency overflow weirs. First of all, there would not be so much disturbance of the water within the upper chamber since the inflow from the pump columns would not be pushing water out of the way but would instead be dropping towards the water surface. The effect of this would be to make for much less disturbance in the water surface within the chamber and therefore less tendency for water to slop over the emergency overflow weirs. The second effect would be that the inflow would rise to a somewhat higher elevation. However, since the pump discharge column diameters has been increased in the final design, the problem associated with the flow impinging on the roof of the level above has been alleviated with this design change. Therefore, an improvement in internal chamber hydraulics would be noted if the elevation of the pump discharge columns was increased.

A final aspect to flow within the upper chamber is that the flow plunging over the crest of the effluent weir entrains air as it falls into the exit section prior to passing into the basin discharge pipes. The amount of air entrainment is dependent on the downstream Rouge River water surface elevation, but at low river levels, there is so much air entrainment that it is difficult to visually inspect the characteristics of the flow at the entrance to the basin discharge. This situation was the primary reason for conducting the model study in the first place and is discussed in more detail in the following section.

Flow Through Discharge Pipes

As discussed above, the flow exiting the basin over the effluent weir entrained a sufficient amount of air that the water surface in the exit section was frothy and it was not possible to visually inspect the flow to observe the nature of discharge into the exit pipes. This situation was more pronounced at the higher discharges, so most of the initial tests were performed with a system flow rate of 570 cfs (five pumps). Observations were made at the downstream end of the model (equivalent to 160 feet along the pipes discharging to the Rouge River) where the behavior was found to strongly depend on the simulated Rouge River level. At very low river elevations, the flow within the discharge pipes will have a free surface and air entrainment will not be a significant problem as the air can escape to the surface within the pipe and will be vented in that fashion. Even after

the 160 feet of travel distance in the model, there is still air in the flow, but it does not appear to have any deleterious effect on the flow. Once the downstream river elevation is raised, however, so that a backwater effect approaches a full conduit flow condition within the discharge pipe, the air alters the nature of the flow. It was observed that air tends to be expelled from the downstream end of the pipe in "bursts" of large bubbles; this can best be seen by viewing the videotape of the model tests. The condition is very sensitive to the downstream tailwater level and is the worst when the tailwater elevation is just above the crown of the pipe. These bursts cause transients in the water flow that apparently result in pressure variations that may effect system performance. The volume of air in these bursts is difficult to estimate precisely, but at the model scale appear to be on the order of a liter or so. Subsequent investigations indicated that these air bursts are intermittently entrained in the plunging flow over the weir as opposed to a coalescence of discrete air bubbles within the discharge pipe. Therefore, it does not appear that the model behavior in the shortened discharge pipes will be substantially different than in the prototype.

As mentioned above, these bursts create pressure pulses in the water flow that result in surges in the flow through the discharge pipes. This effect was most pronounced at high system discharges and when the downstream water level was just above the crown of the discharge pipe. Of course, since the two discharge pipes have different crown elevations, each one behaves differently at a particular river level. Since the eight foot diameter pipe has the lowest crown elevation, the problem will be more pronounced in it as the air entrainment will be greater in the exit section due to the greater plunge distance in that condition. An additional consideration is that the submergence of the crown for the eight foot pipe should occur more frequently due to its relation to typical water levels in the Rouge River (crown elevation of about 574.25 ft compared to a low water elevation of 571.9 ft and a 10 year flood level of 576.5 ft). The six foot pipe with a crown elevation of about 576 ft will only be submerged under flood conditions. Although this higher water level produces a smaller plunge distance in the flow over the effluent weir and therefore less air entrainment, the same qualitative behavior was observed in the six foot pipe but the nature of the surging was less dramatic. At these higher river levels, the surging in the eight foot pipe was also reduced for this same reason but was still present to some extent.

Several changes were made in the model to attempt to eliminate the surging resulting from the air bursts. The first attempt involved the installation of simulated manholes in the discharge pipes to provide a place for air to escape. The location of these manholes is somewhat constrained because there are substantial lengths of the discharge pipes where the hydraulic grade lines at high river levels and high discharges will be above the ground surface. The manholes were modeled as four feet in diameter mounted to the top of the discharge pipes and located about 45 feet downstream from the basin. This location was about as far downstream as the manholes could be located considering the hydraulic grade line

elevations relative to the ground surface but it was also just before bends in the discharge pipes that could possibly distort the flow and prevent successful air removal. In any case, this option was not determined to be a feasible option for air removal. The installation of the manholes did basically remove the bursts of air from the discharge to the river. Although there was still air flowing from the ends of the pipes, this air was now in the form of discrete bubbles and there was much more of a continuous stream than the highly intermittent flow observed previously. However, the intermittencies in the flow were transferred to the manhole shafts. In the model construction, the manholes were simulated by affixing six inch diameter PVC pipe to the tops of the discharge pipes at the desired location. These six inch diameter sections of pipe were approximately 18 inches high and the top did not represent the actual ground surface. The bursts of air did tend to be entirely removed by the manholes but the water levels within the vertical shafts fluctuated by over a foot in the model (eight feet prototype) in an intermittent fashion as the air entered the shaft and was expelled. It was difficult to quantify precisely the range of these fluctuations as the water surface in the shaft was covered with froth and it was not possible to visually observe the actual water surface. However, this magnitude of fluctuations in water levels would result in effluent intermittently discharging to the ground surface if the manholes were constructed as modeled. In addition, the amount of froth observed in the model manholes (using clean water from the domestic water supply) raises concerns about the possibility of significant foaming with a typical CSO effluent.

After consideration of other possible alternatives for alleviating the surging problem, it was decided to test the relocation of the effluent weir as this modification appeared to hold the most promise for success. The effluent weir was replaced by two separate weirs located just downstream from the bar screens as indicated in Fig. 5. This configuration accomplished several objectives simultaneously. The plunging flow was moved further away from the discharge pipes and allowed for more opportunity for air removal from the flow prior to entry into the discharge pipes. In addition, this option increased the total weir crest length resulting in a smaller head change within the basin over the range of possible discharges and avoided some of the difficulties with the flow over the emergency overflow weirs at high system discharges as discussed above. The elevation of the emergency overflow weirs was lowered as part of these design changes; this was done after the completion of the model testing so the model was not altered to reflect this change. Also, there would be a relocation of the surface baffles to location upstream of the new effluent weirs; this had not been established at the time of the model testing and they were not included in the testing since they had no impact on the air flow through the discharge pipes. Observations with the initial configuration had indicated only minimal influence of the surface baffle on the flow approaching the effluent weir so this was not considered to be a critical issue in the model testing.

The model testing with this configuration indicated that all of the large bursts of air were successfully vented before the flow entered the discharge pipes. Under the conditions of minimal submergence of the eight foot pipe, there is still air passing through that pipe, but it is at a greatly reduced volume and only in the form of discrete bubbles. The videotape of the model under this flow condition shows that the air flow is somewhat intermittent in nature. Most of this air appears to be entrained in intermittent air core vortices that form at the entrance to the discharge pipe due to the low submergence. This air is essentially eliminated at higher downstream water levels. Based on these observations in the laboratory model, it is not visualized that there is any remaining problem related to air flow through the discharge pipes. Since the modification testing the manholes was constructed prior to the testing of this option, it was possible to test the influence of the manholes on this option as well by either closing or opening a seal at the top of the manhole shafts. It was found that this action had no substantial impact on the flow and also that the violent surging in the manhole shafts was also gone as well. Therefore, the remaining air should have no impact on the flow within the pipes, nor will there be any impact in the river itself due to the small air discharge at the pipe crown.

Since the configuration tested appears to be close to the one that will be implemented in the final basin design, additional testing was performed to clarify other aspects of the hydraulic performance of the basin. The model was tested over the range of pumps (1 to 5) and over the range of downstream water levels with no indication of any problems associated with air entrainment. An additional concern that had been indicated associated with the relocation of the effluent weirs closer to the bar screens was that the velocity distributions at the bar screens would become more nonuniform which could potentially increase the possibility of problems with screen blockage. In order to assess this issue, a survey of the area immediately upstream from the simulated bar screen was made with a mini propeller meter to measure velocity variations. Velocities were made near the bottom, top and at mid-depth at several locations across the width of the flow. The variations from top to bottom at the center of the bar screens (approximately 50 percent higher velocity at the surface than near the bottom) are less than the variations across the width and thus it is concluded that the relocation of the effluent weir has no substantial effect on the approach velocity at the bar screens. Finally an attempt was made to measure the condition at which the Rouge River elevation was sufficient to submerge the flow at the effluent weirs and thus influence water levels upstream of the effluent weirs. This was done by slowly increasing the tailwater elevation in the model until an increase in the water surface elevation upstream from the effluent weirs in the model was noted. This is difficult to do with a high degree of precision due to fluctuations in the water level due to surface waves. However, an independent repetition of this measurement reproduced results within about 0.3 feet prototype elevation difference so it should be sufficiently accurate for purposes of predicting system performance. It was found that submergence of the

weirs (and thus upstream increase in water level) occurred at a tailwater elevation of about 579.5 ft in the model. Extrapolating this to a Rouge River elevation requires the estimate of friction head losses in the piping system including an estimate of the bend losses. Using reasonable estimates for these would produce a Rouge River elevation of about 578 feet for submergence to occur. Since this is above the 100 year water level, it is concluded that submergence should occur only during extreme flood stages in the Rouge River.

A security bar grate system has been proposed to cover the downstream end of the discharge pipes. The grates are proposed to be constructed from 4" x 1/2" vertical bars at 6" spacing. A geometric model of the bar grate for the eight foot diameter discharge pipe was constructed from sheet aluminum and affixed to the downstream end of the model pipe. Since disturbances in the flow are the worst under conditions where the downstream end of the pipe is just submerged, testing of the flow through the bar grates was conducted for this condition with the discharge from five stormwater pumps. Visual observations of the flow with the bar grates installed or removed indicated no detectable difference in the flow into the discharge box; this is not unexpected due to the minimal blockage of the flow. A videotape recording of these two conditions has been included on the tape produced of the model construction and testing. The videotape indicates a fairly irregular surface downstream of the security grate but this appears to be mainly related to the turbulent flow within the confined space of the tailwater box and not representative of the exact conditions associated with discharge into the river. An attempt was made to measure the increase in water surface level in the exit section of the upper chamber but the increase was so small that it could not be reliably measured with the turbulent fluctuations in water surface level that occur in that portion of the model. Formulas for estimating the head losses through bar grates indicate that this loss may be on the order of 0.1 to 0.2 ft for a discharge from five stormwater pumps so this is consistent with the observations.

If the Rouge River is near its low water level (571.9 feet), then the river level will be substantially below the crown of both pipes, in particular the six foot diameter pipe. If a large discharge through the retention basin occurred under this circumstance, the discharge pipes would not flow full and the flow within these pipes would be controlled by critical flow at the exit to the Rouge River. Estimating the split of discharge between the two pipes and calculating the critical flow depth in each yields a discharge velocity of about 10 ft/s from each pipe under these circumstances. These velocities will be reduced in the Rouge River due to mixing with the river water. Lowering the elevation of the discharge pipes would only lower these discharge velocities (down to about 7.9 ft/s for the eight foot diameter pipe, for example) and deep submergence of the discharge is apparently not possible.

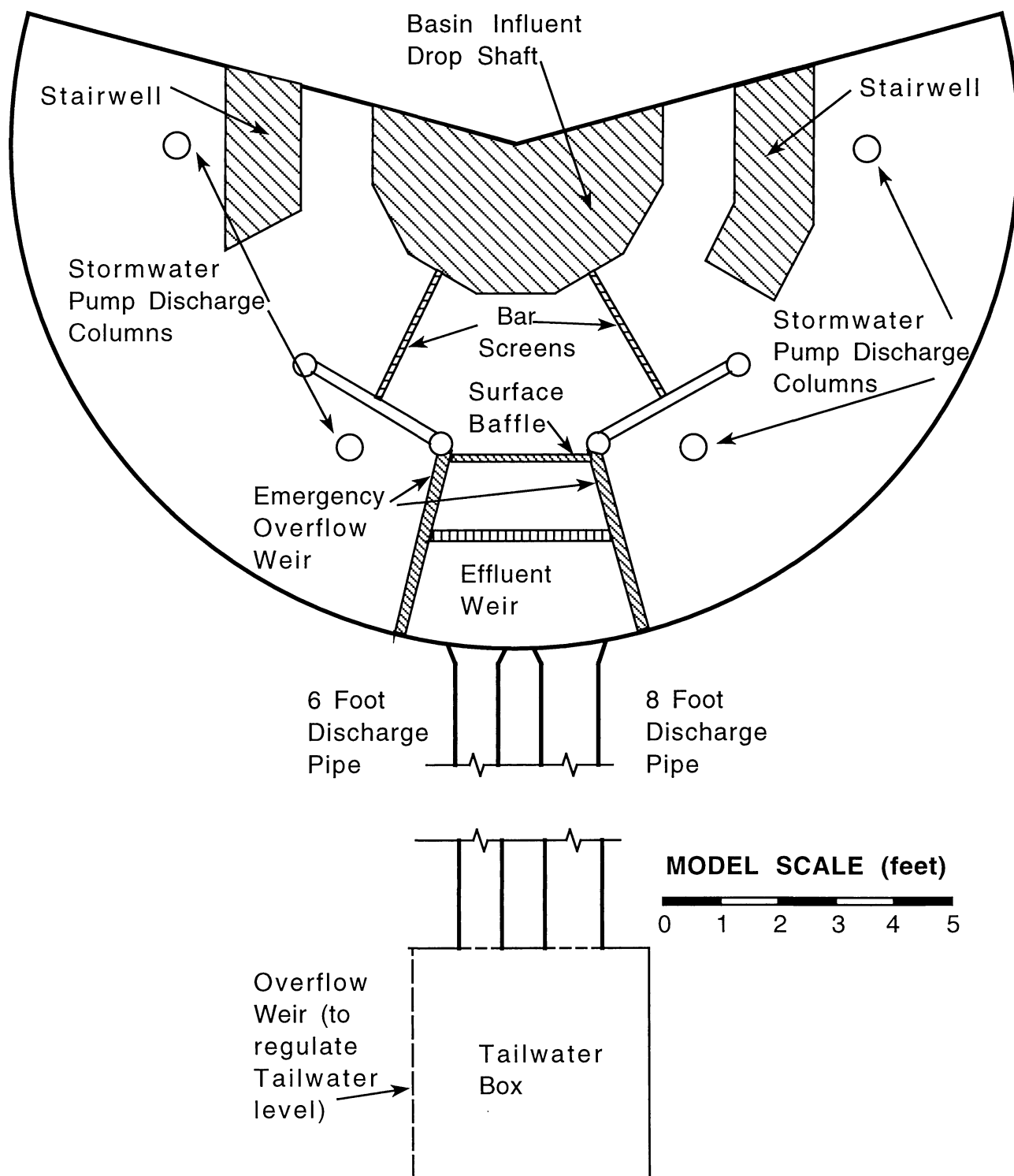


Figure 1. Plan View of Model Layout.

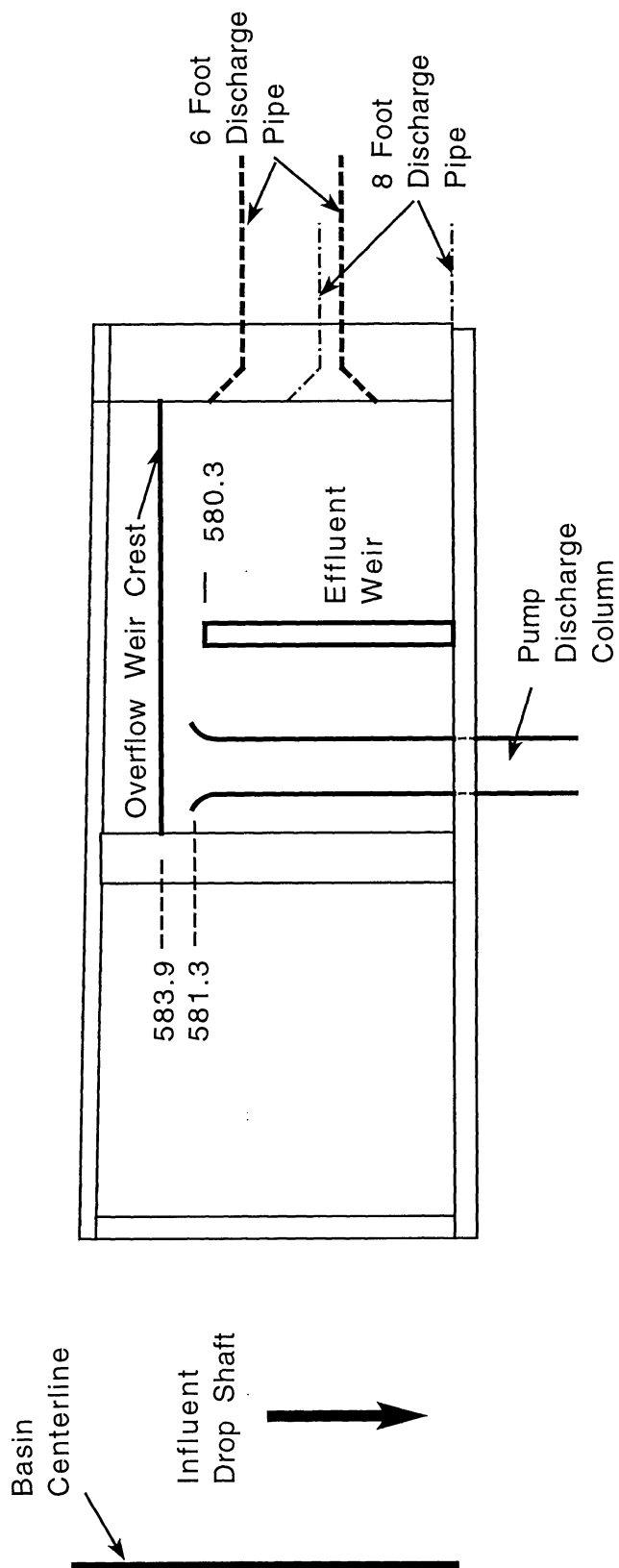


Figure 2. Horizontal View of Pertinent Model Components (Elevations as obtained from blueprints depicting original design).

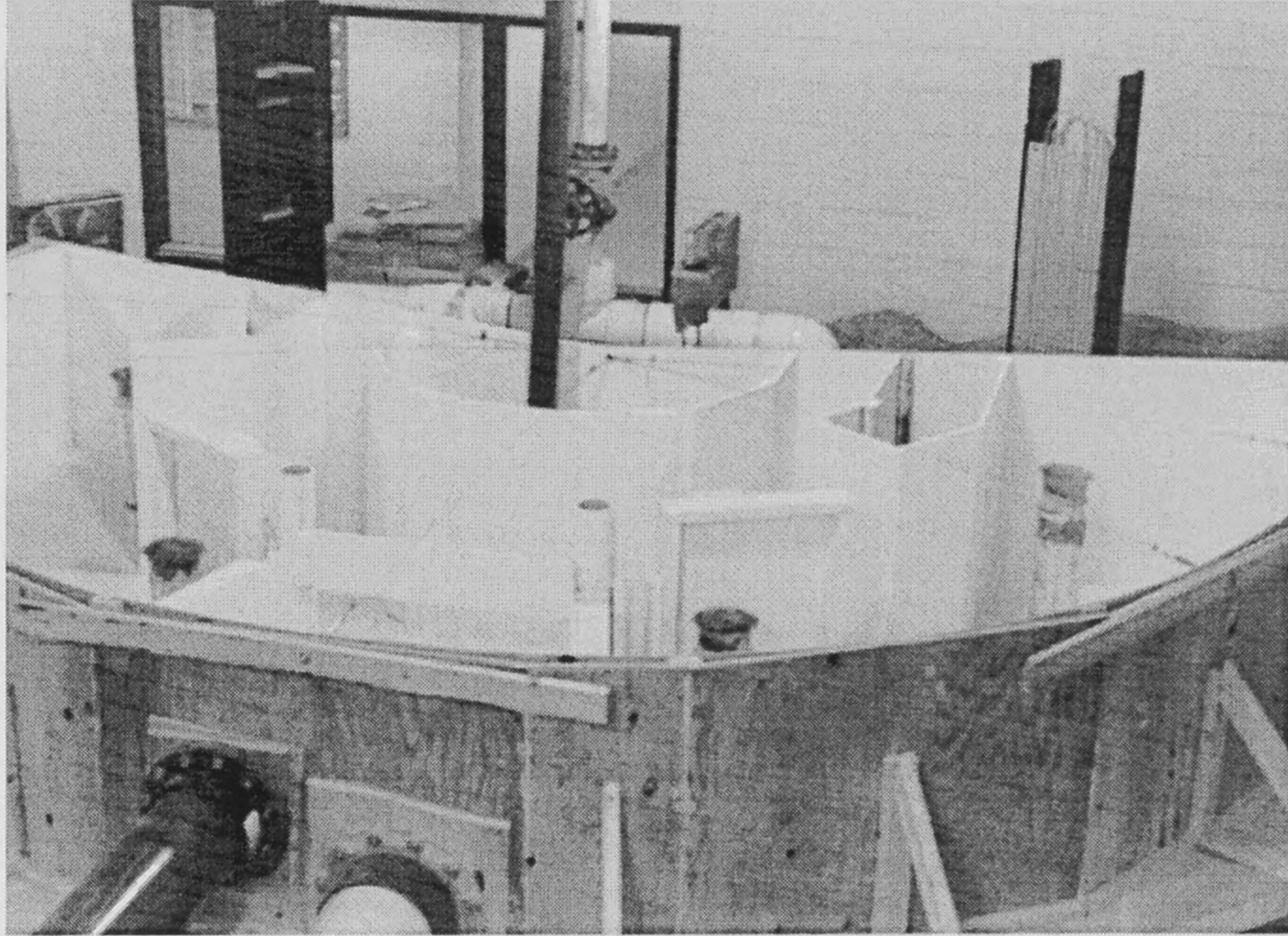


Figure 3a. Photograph of Physical Model

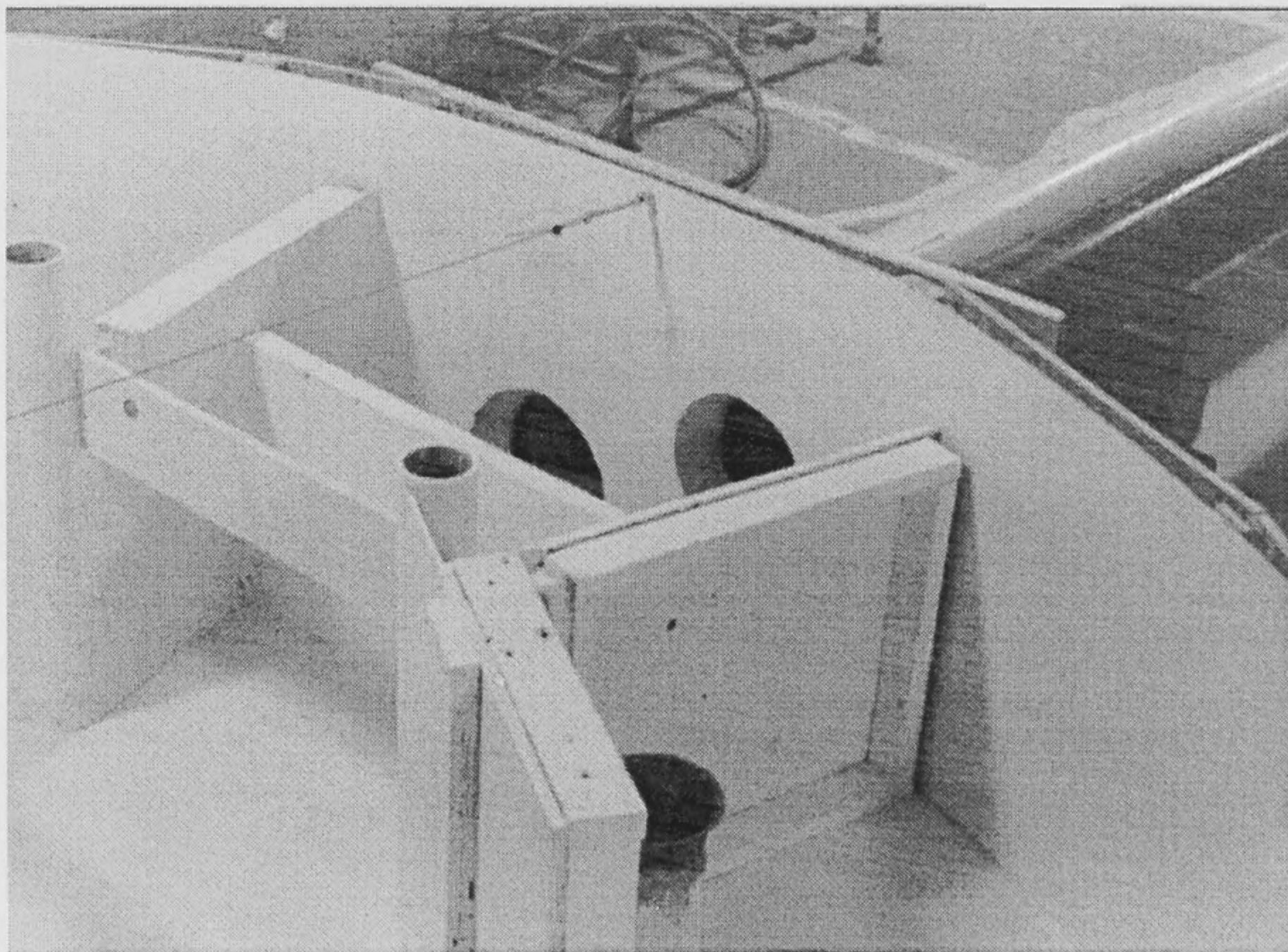


Figure 3b. Effluent Weir - Entrance to Discharge Pipes.

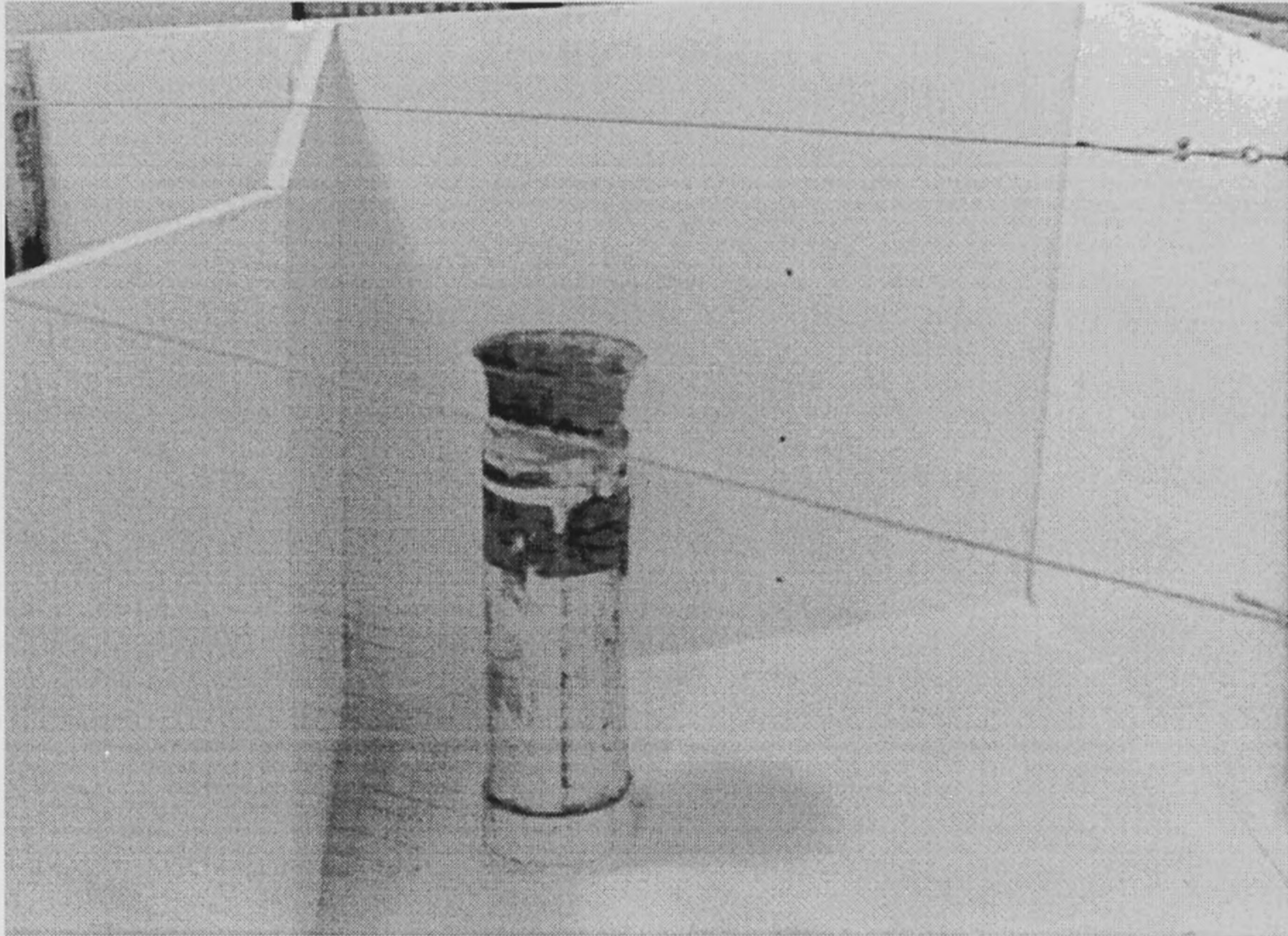


Figure 3c. Pump Discharge Columns.

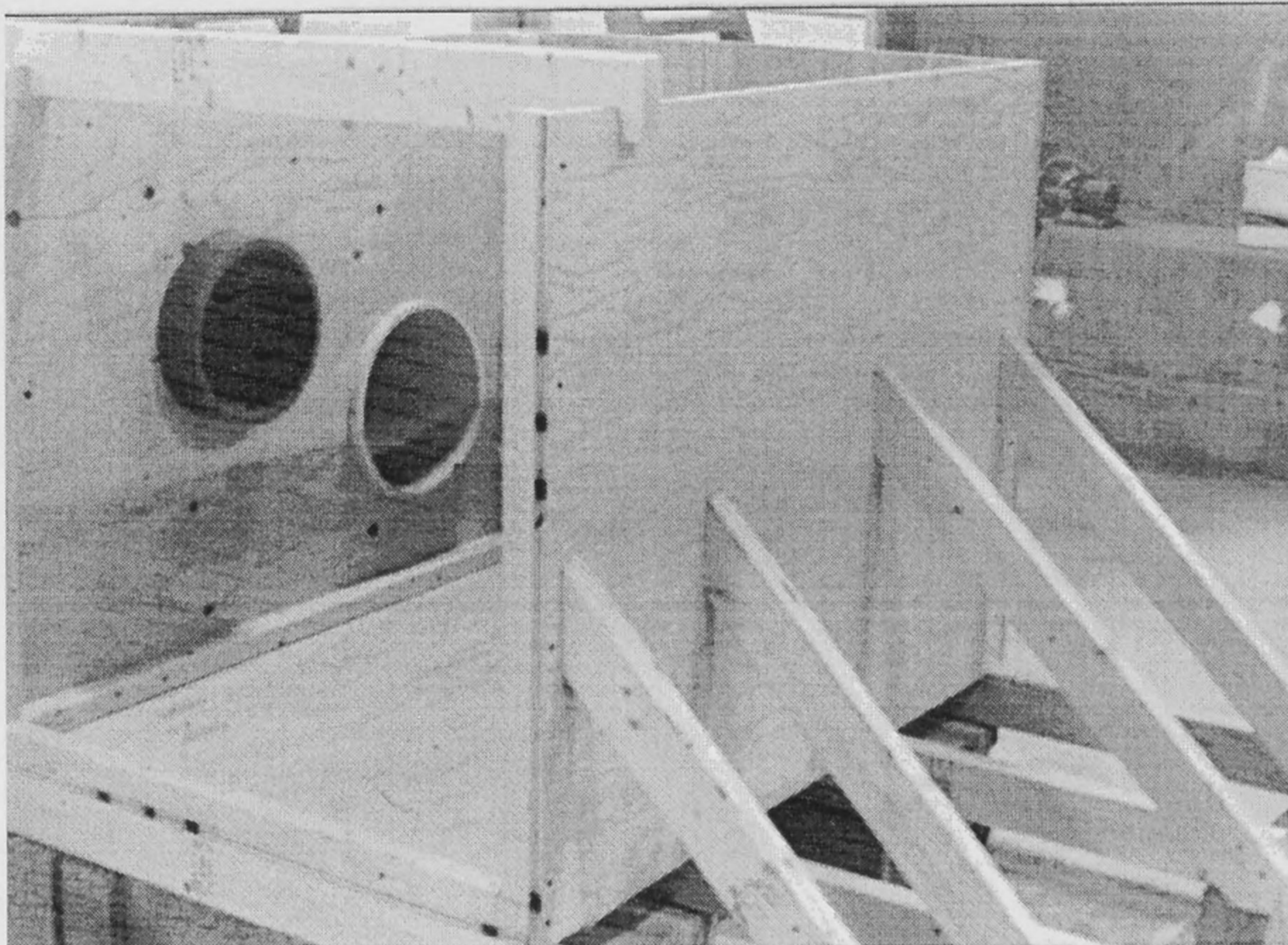


Figure 3d. Tailwater Box.

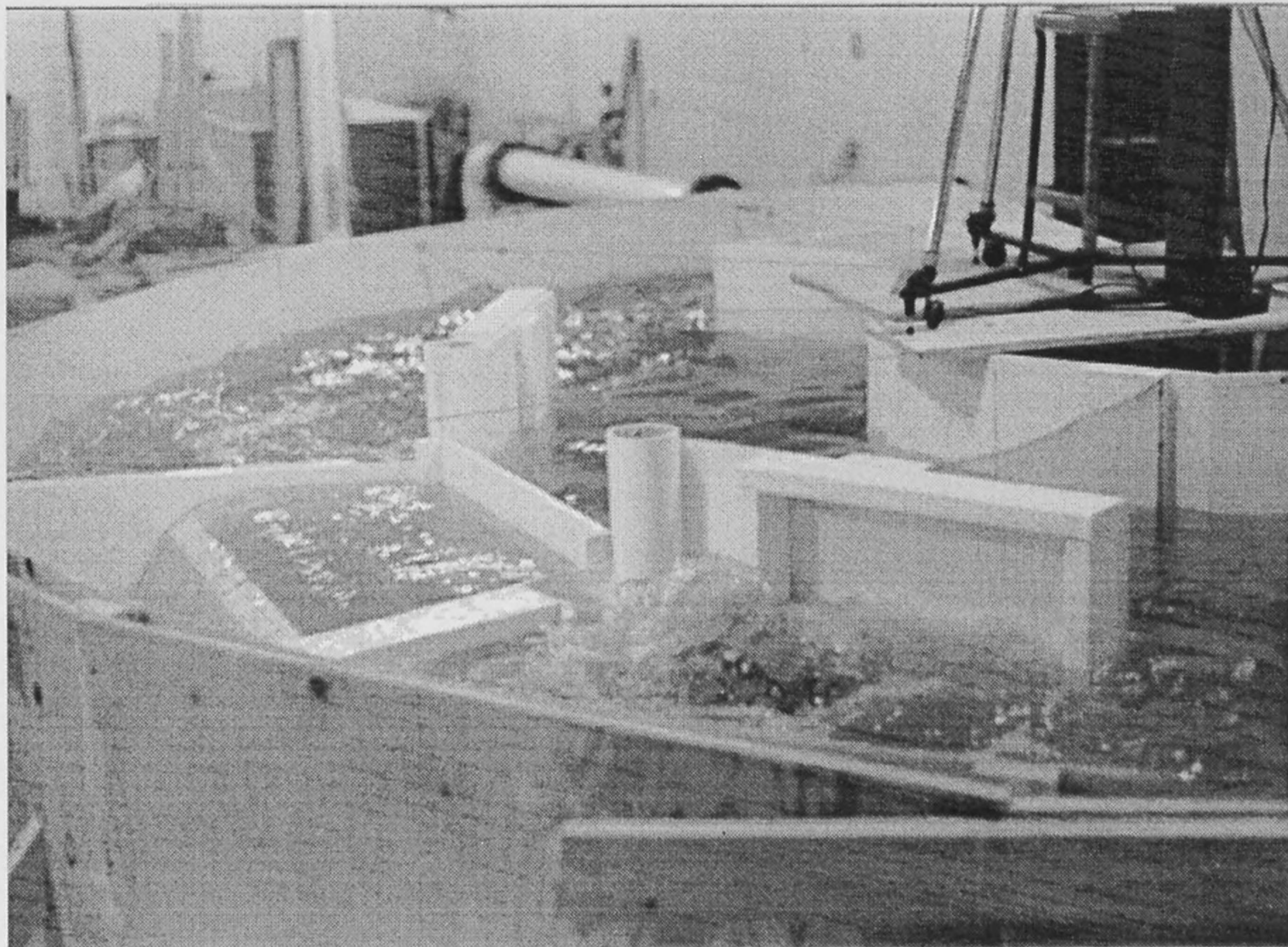
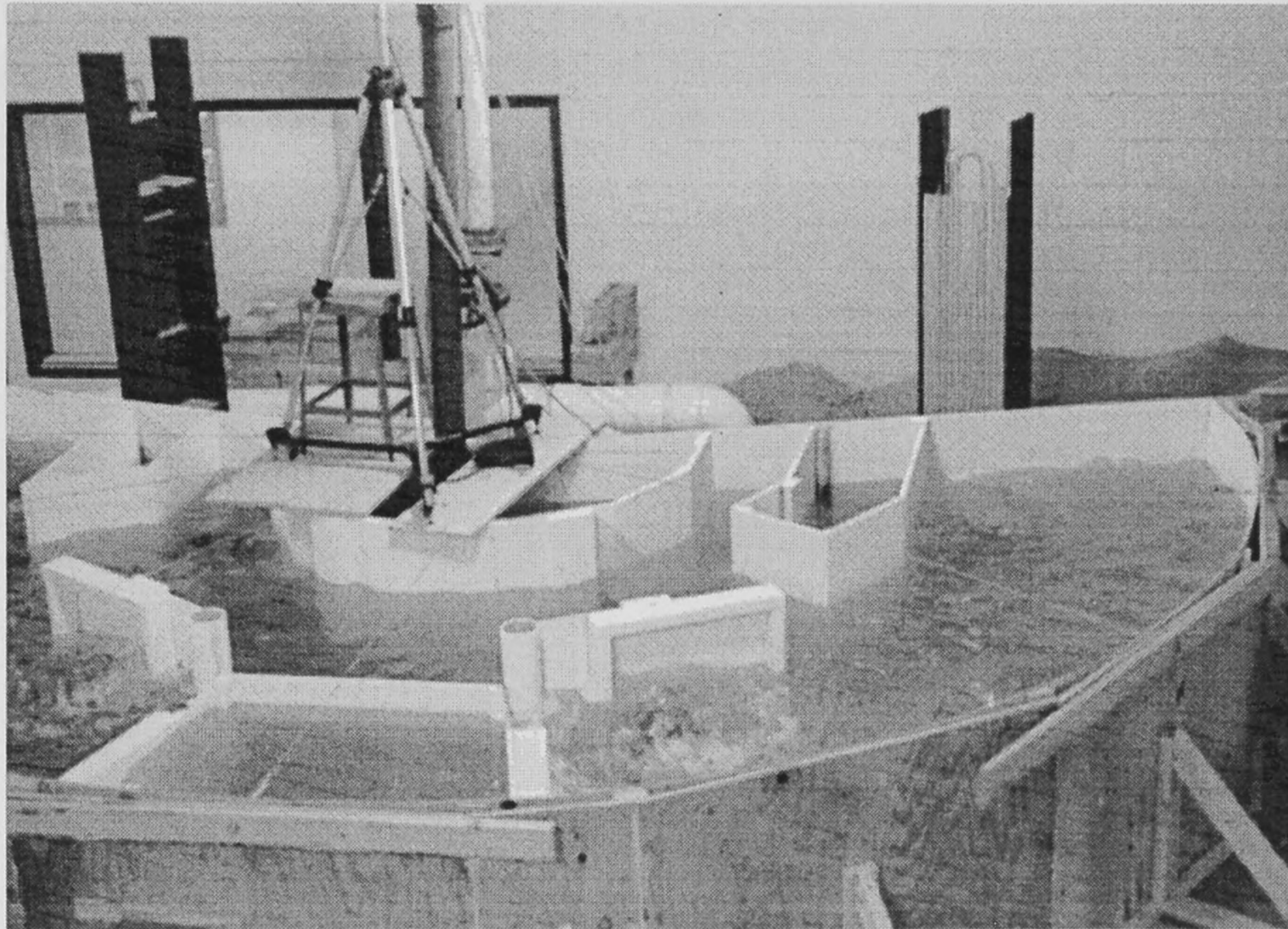


Figure 4. Water Surface Super-elevation above Pump Discharge Columns.

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AIIM SCANNER TEST CHART # 2

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Greek and Math Symbols

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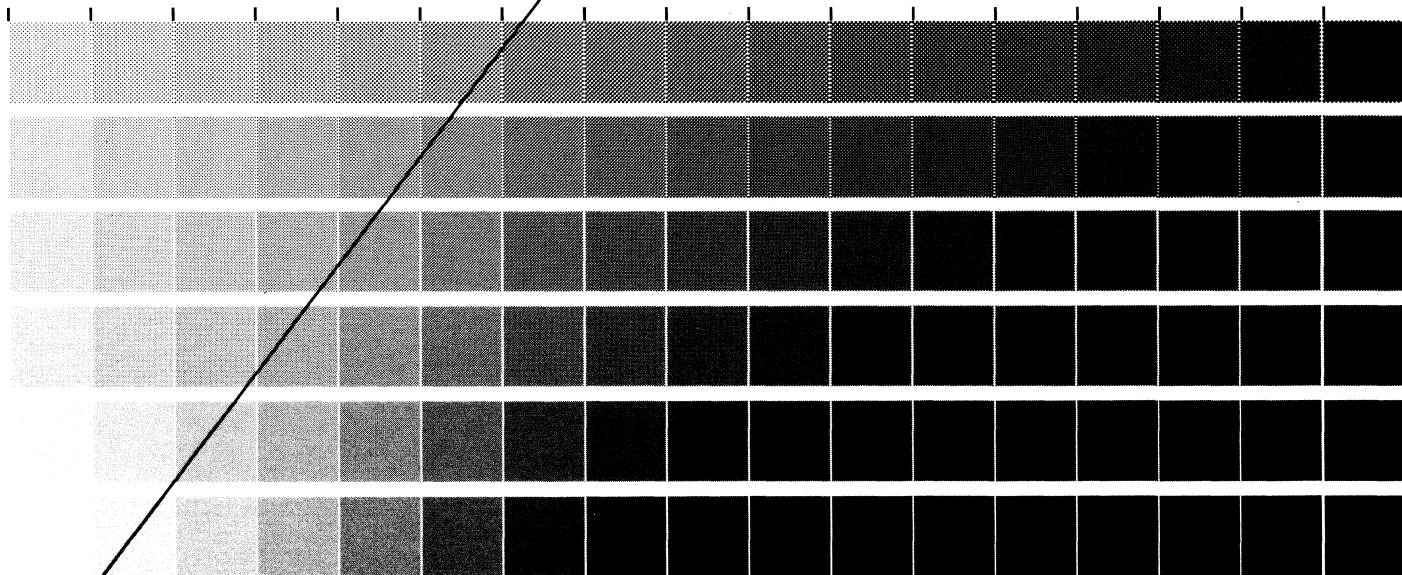
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RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-171

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