HYDRAULIC MODEL STUDY

INFLUENT SEWER JUNCTION CHAMBER TO MARKET AVENUE RETENTION BASIN

GRAND RAPIDS, MICHIGAN

Report UMCEE 91-12

By

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FINAL PROJECT REPORT

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For

McNamee, Porter, and Seeley
Ann Arbor, Michigan

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INTRODUCTION

The City of Grand Rapids is constructing a retention basin on their combined sewer system to minimize overflows into the Grand River. A part of this facility involves the construction of a pump station to deliver flow to the Market Avenue Retention Basin (MARB). This physical model study examined the flow conditions within a junction chamber in the East Side Interceptor sewer upstream from the pumping station at and near the point where the flow may be diverted to the retention basin during high flow conditions or continuing to the existing Market Avenue Pumping Station (MAPS) and ultimately to the wastewater treatment plant during dry weather conditions.

The purpose of the model was to study the flow at the junction chamber under a variety of different flow conditions to examine the sewer capacity at that location and the potential for excess turbulence, etc. in the junction chamber. Water levels, head losses and velocity distributions were measured in the model along with visual observations of the general flow conditions. The physical model was also used to guide modifications to be incorporated into the final junction chamber design.

The testing sequence included the following components:

- Measure head losses across the junction chamber
- Measure velocity distributions in the conduit to the retention basin downstream from the junction chamber
- Record visual information on the nature of the flow within the junction chamber

If potential problems were indicated, the physical model was used to investigate alterations to be incorporated into the final junction chamber design. This report documents the testing procedures and the modifications that were investigated in the attainment of the project objectives.

GENERAL SYSTEM DETAIL

The 25 year flow entering the junction chamber has been computed to be 1100 MGD, 700 MGD of which enters the the main branch of the East Side Interceptor while 400 MGD enters through a side branch. Fig. 1 indicates the general layout of the junction chamber and the limits of the physical model. As part of the retention basin work the connection of the branch sewer has been redesigned so that is is more nearly parallel to the main branch. Hereafter, the 17 ft wide sewer is referred to as the main channel while the 10 ft wide branch will be referred to as the branch line. In order for the combined flow to reach the Market Avenue Retention Basin, it must be deflected to the left by 60° within the junction chamber. Alternately, flow would pass straight through the junction chamber in order to reach the existing Market Avenue Pumping Station or to reach the overflow gates (or the Buffalo pumps for high river level conditions) to the Grand River. Two of the six sluice gates which are placed in the right wall of the East Side Interceptor are located just downstream from the junction chamber as indicated in Fig. 1 while the other four are beyond the limits of the model. Similarly, the Buffalo pumps are located further downstream than the end of the physical model. All dry weather flows are to proceed to the MAPS and must not overflow the 2-ft (original design) high broad-crested weir at the entrance of the retention basin conduit. This is intended to prevent dry weather flows from entering the retention basin. Wet weather flows in excess of approximately 60 MGD are to be diverted to the MARB through the pumping station which has an installed pumping capacity of 1050 MGD, and a firm pumping capcity of 945 MGD (10 pumps at 105 MGD each). Thus the design flow (1100 MGD) may require a diversion of up to 155 MGD to the Grand River by means of the sluice gates or the Buffalo pumps depending upon the river stage. The initial design for the junction chamber included a concrete fillet on the right wall of the sewer and a splitter wall/vane in the center of the chamber that were both intended to deflect the flow to the left and into the channel leading to the retention basin. These are herafter referred to as flow diverters. The main purpose of the model study was to investigate the flow within the junction chamber during high flow conditions especially as related to the function of the flow diverters.

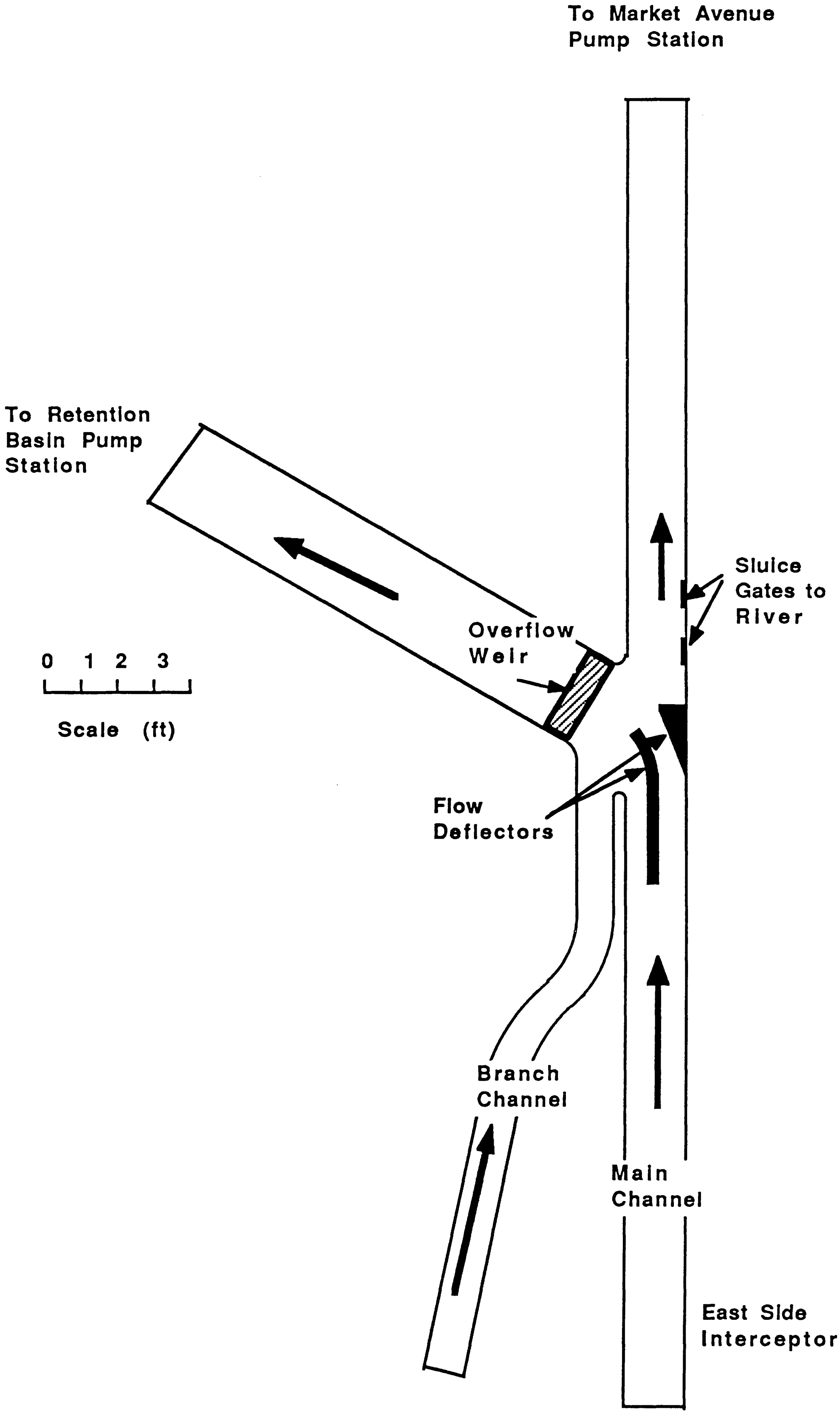

Figure 1. General Layout of Junction Chamber Reproduced in Physical Model.

CONCLUSIONS AND RECOMMENDATIONS

• The operation of the Market Avenue Pumping Station is intended to produce a significant backwater effect in the East Side Interceptor. In part because of this backwater effect the flow diverters tend to serve only a marginally useful function. The investigation indicates that both head losses and the flow distribution in the approach channel to the retention basin pump station are basically the same whether or not the diverters are in place. Because of the added cost of constructing the flow diverters and to maintain flexibility in utilizing the sewer in other flow conditions, it is recommended that the design be changed by omitting the wall fillet and guide vane.

• The flow at the broad-crested weir as originally design resulted in considerable flow separation and turbulence. These result in excess head losses that can be partially eliminated with a better design of the weir cross section. An ogee section with a weir height of 3 ft (as opposed to the original design of 2 ft) was tested and found to have somewhat lower head losses than the original design. Since the extra foot of weir height allows for the passage of more dry weather flow at a given downstream elevation, this design allows for more economical operation during dry weather conditions by reducing pumping head in the Market Avenue Pumping Station. It is recommended that this design change be implemented.

• Approximately 60 MGD can be passed down the sewer to the Market Avenue Pumping Station through the two 3 ft sluice gates without overflowing the 3-ft high ogee weir.

• With the modifications suggested above, no severe flow conditions were observed in any of the flow cases investigated. The distribution of flow in the approach channel to the retention basin pumping station is fairly uniform with an excess of flow on the right side of the channel. This was observed whether or not the diverters were in place. By removing the flow diverters, vortex shedding in the junction chamber was largely eliminated. Only minor flow disturbances associated with flow separation at corners was noted in the model and these probably cannot be eliminated for all flow conditions.

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 prototype and the corresponding one in the model ($L_r = Length_{prototype}/Length_{model}$). For a Froude scaled model, assuming the same fluid in model and prototype, the following relations must hold:

PARAMETER		RATIO
Length	L_r	$L_r = 10$
Velocity	V_r	$L_r^{1/2} = 3.16$
Discharge	Q_r	$L_r^{5/2} = 316$
Time	T_r	$L_r^{1/2} = 3.16$

The critical factors with respect to model testing facilities are the model size and discharge. The scale ratio may be determined by either the space available in the laboratory facility or the installed pumping capacity. If the scale ratio is too large, viscous effects may become too great in the model. This consideration generally fixes the minimum model size required to avoid distortion of the model flow due to the effects of viscosity. Roberson, et al (1988) suggest that a minimum Reynolds number of about 100,000 be maintained in most physical models to correctly reproduce the effect of viscosity on the flow behavior. In the context of the present study, this Reynolds number should be defined in terms of the flow in the approach sewers with $Re = Vd/\nu$ and V the average flow velocity, d the flow depth and ν the kinematic viscosity of the water. This constraint becomes instrumental in the selection of the physical model scale and requires a length scale ratio of no greater than about 10, depending upon the specific flow condition to be studied.

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 the model test basin which is 15 meters long by 11 meters wide.

Model Construction

The physical model was constructed at a scale ratio of 10. This general model size was selected to keep the Reynolds number defined above to be greater than about 100,000 at the lowest modeled flow rates. At a flow of 400 MGD, the Reynolds number in the 10 ft wide influent sewer line will be about 200,000. Similarly, the Reynolds number in the main channel was also around 200,000 for the maximum flow of 700 MGD. Smaller Reynolds numbers would result for smaller modeled flow rates.

The extent of the physical model is indicated in Fig. 1. The intention of the particular choice of model extent was to model on the order of ten water depths or more in each conduit entering or leaving the junction chamber. This criterion was taken as an indicator that the specific entrance or exit used in the model would not influence the flow within the junction chamber.

Blueprints provided by McNamee, Porter, and Seeley gave the detailed dimensions to which the model was constructed. The model was constructed mainly from plywood and was painted to provide a smooth finish. Small radius corners and internal columns were formed from PVC pipe as close to the correct diameter as possible. Larger radius bends, flow diverters, etc were constructed from sheet metal tacked to a plywood form. With this combination of construction techniques, all essential design detail was reproduced in the model at the correct scale. No lid was present over most of the model since the sewer is not generally intended to function in a surcharged condition. The absence of a lid allowed for easier inspection of the flow conditions within the junction chamber. The one exception is the channel leading to the retention basin pump station which is intended to have approximately nine feet of surcharge at the design flow. For this section of channel, the model was constructed with a plywood cover except for the sloping roof just above the weir which was constructed from plexiglass to allow for visual inspection of the flow.

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 which allow the distribution of the flow to both of the influent channels. The model flows were regulated by means of a gate valve on each of the influent lines to obtain the desired total flow and to control the flow distribution. The flows were metered in each individual line by means of an installed pipe orifice meter. Water levels within the model were controlled by means of adjustable height weir plates that were installed at the ends of both discharge channels. Water flowed over the weir plates and back into the sump. For the low flow rate (dry weather flow) conditions, a separate line with a 1.5-in. venturi meter and a 150 gpm pump was used to deliver the flow.

Photographs of the general system detail are provided in Fig. 2. Figure 2(a) is an overall view of the entire model including all piping. The pump is beyond the top of the photograph and the orifice meters are in the dark flanges near the top of the photo. Fig. 2(b) shows a view of the channel proceeding to the retention basin pump station with the aluminum weir plate clamped on the downstream end of the channel. The plexiglass plate above the broad-crested weir is visible near the left side of the photo. Fig. 2(c) indicates the original broad-crested weir configuration viewed from upstream, while Fig. 2(d) shows the ogee section that later replaced it. Fig. 2(e) shows the flow diverters installed according to the initial design. Fig. 2(f) shows the model of the two 3 ft x 3 ft sluice gates in the channel proceeding to the MAPS. Not all of these components of the model were installed simultaneously; in particular the gates in Fig. 2(f) were only present during the final stages of the testing when the dry weather flow conditions were studied.

Instrumentation

Flow rates were measured using sharp-edged pipe orifice meters constructed to the specifications described in Brater and King (1976) for vena contracta meters. Flow coefficients given in Brater and King were used in lieu of a calibration of the individual meters. There were at least 10 diameters of straight pipe upstream and downstream from the orifice in order to minimize flow disturbances. Pressure differences were measured with water-air differential manometers. The orifice diameter was selected to provide a minimum manometer deflection of approximately one foot. For the dry weather flows of less than 60 MGD, the manometer deflection was insufficient to allow accurate flow determination. Consequently, a 3-in PVC line was constructed to deliver the flow. A 1.5-in throat

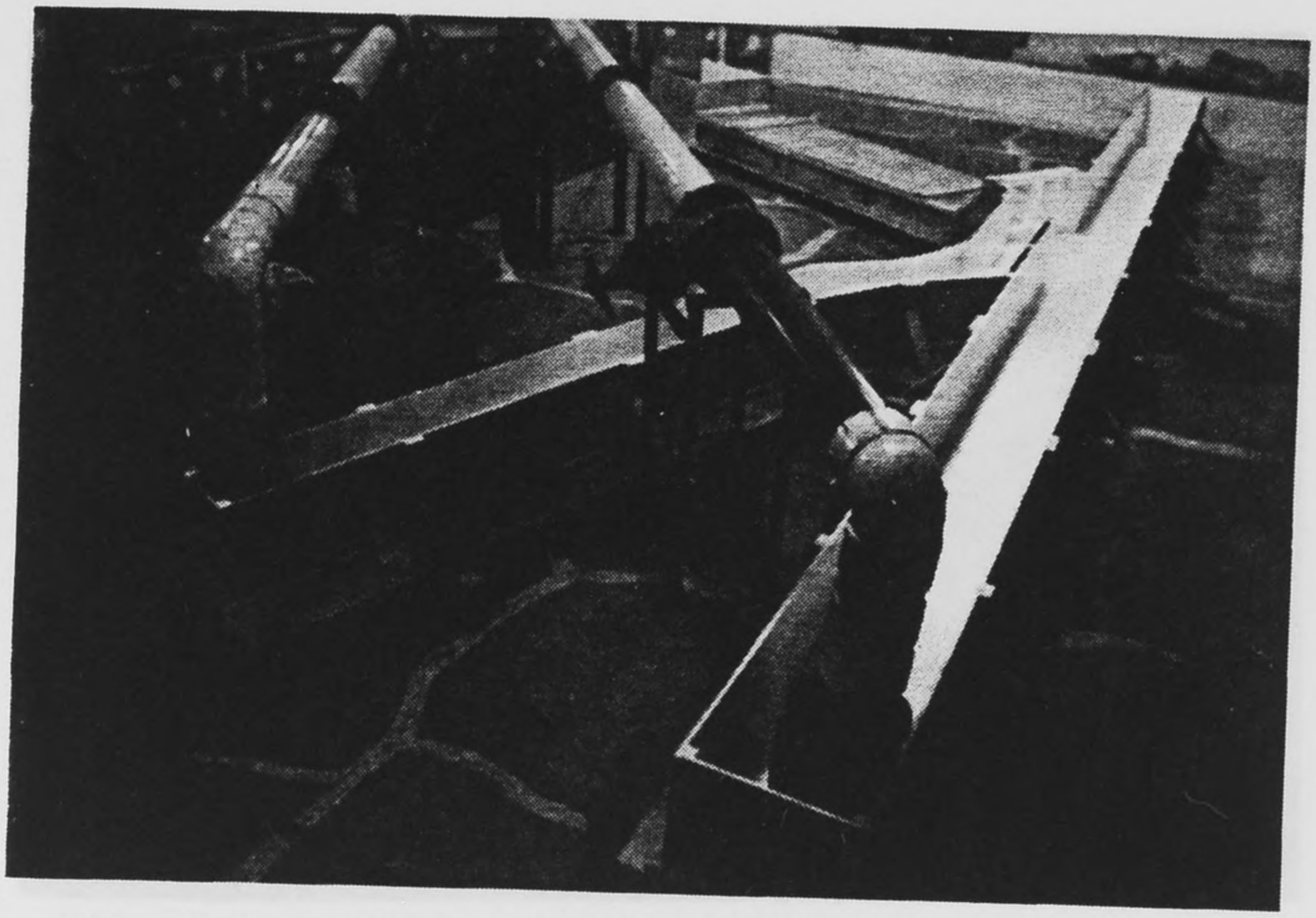

Figure 2a. Overall View of Physical Model from Upstream End

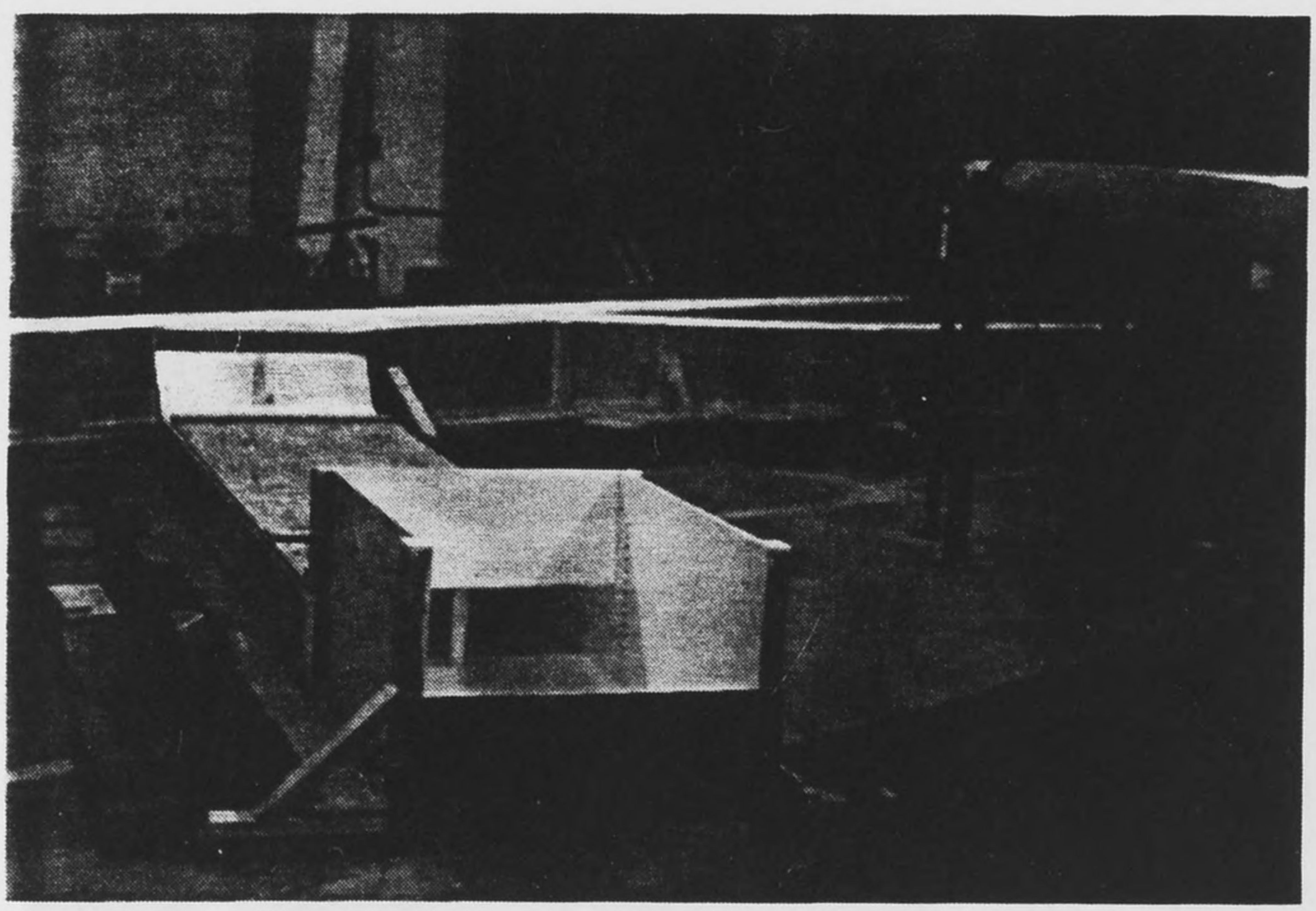

Figure 2b. View From Downstream End of Model Channel Leading to Market Avenue Retention Basin.

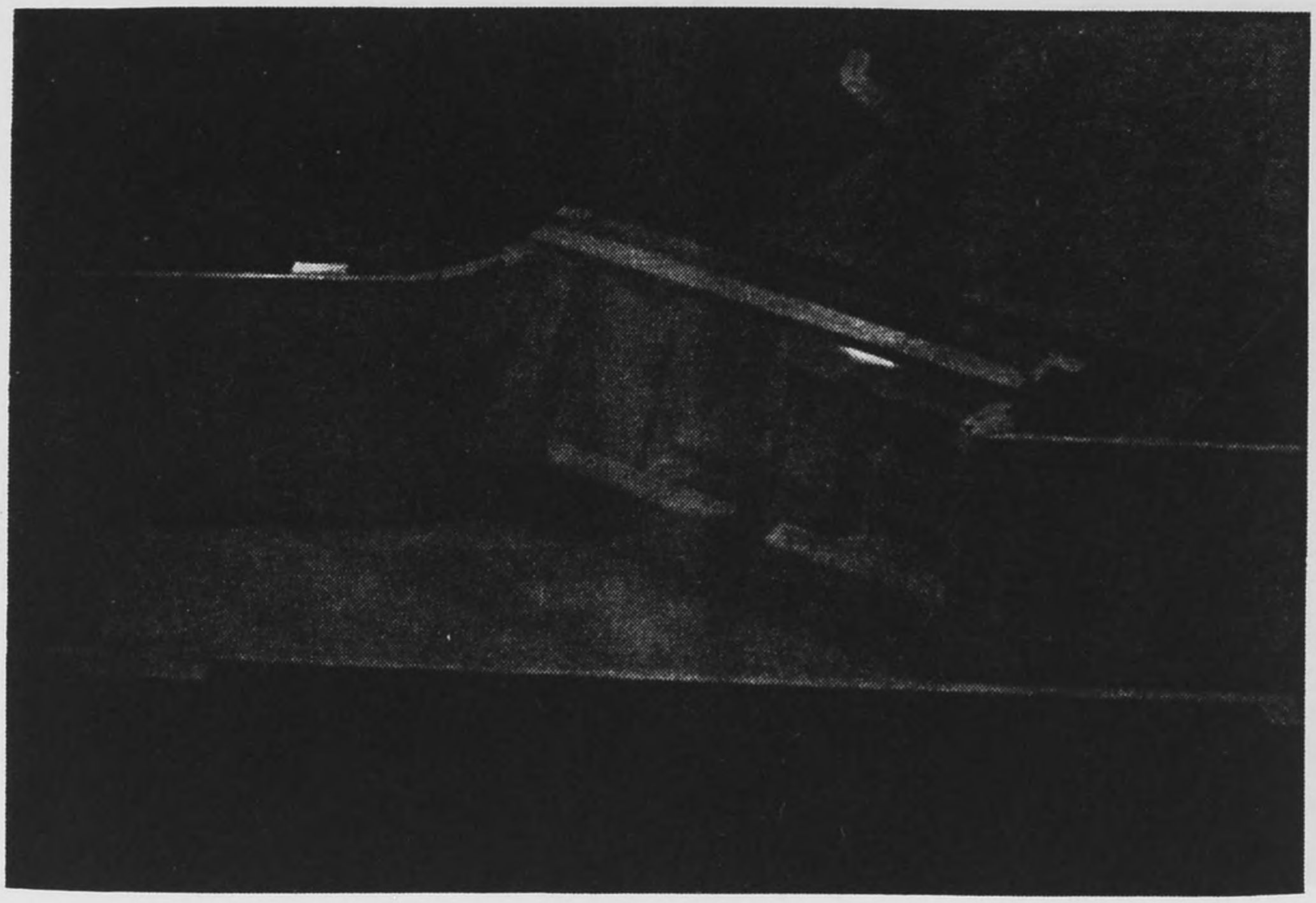

Figure 2c. View of Original Weir Section from Upstream Looking Down Channel to Market Avenue Retention Basin Pump Station

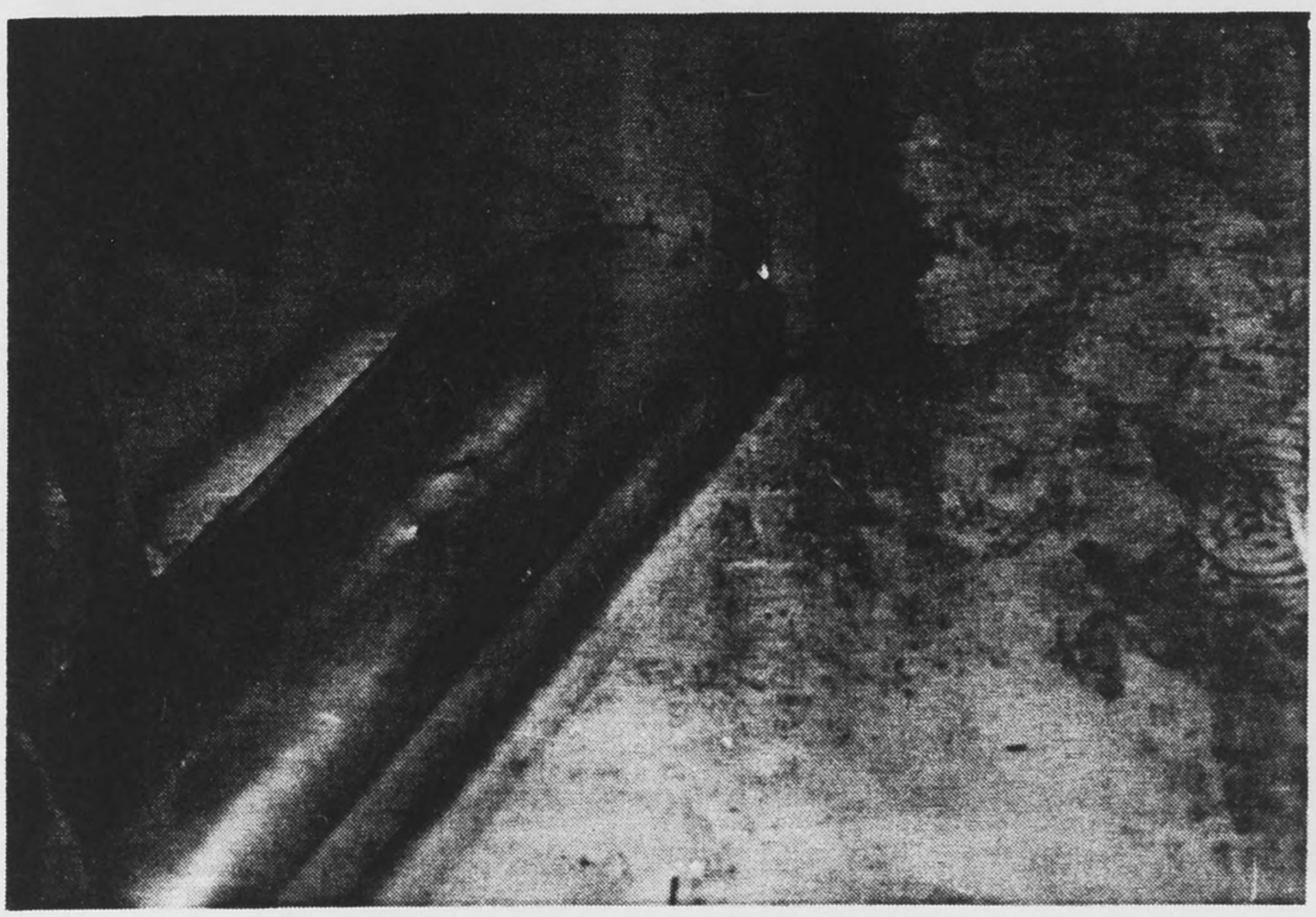

Figure 2d. View of Ogee Weir Section from Upstream Looking Down Channel to Market Avenue Retention Basin Pump Station

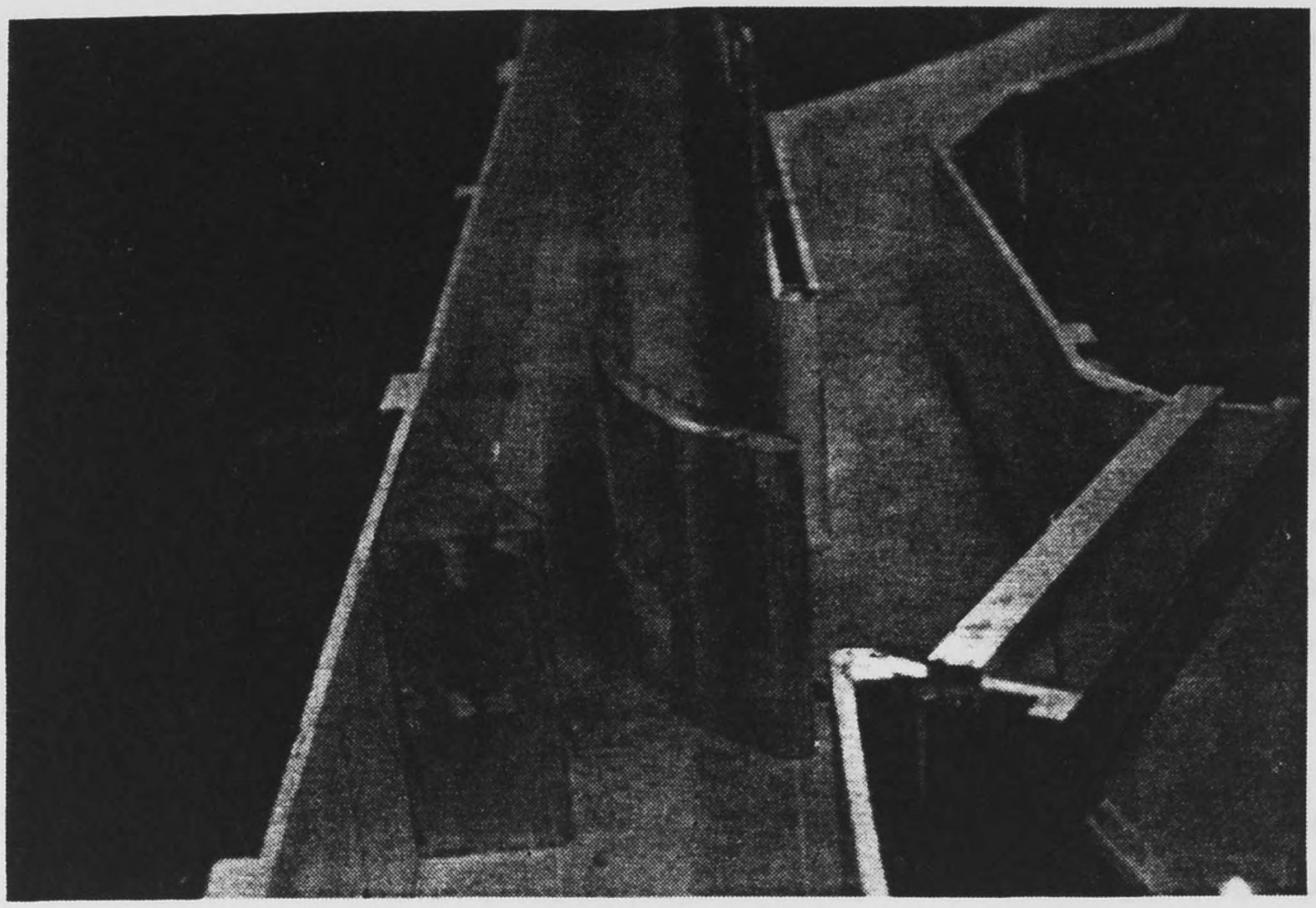

Figure 2e. Flow Diverters in Initial Design as Installed in Model.

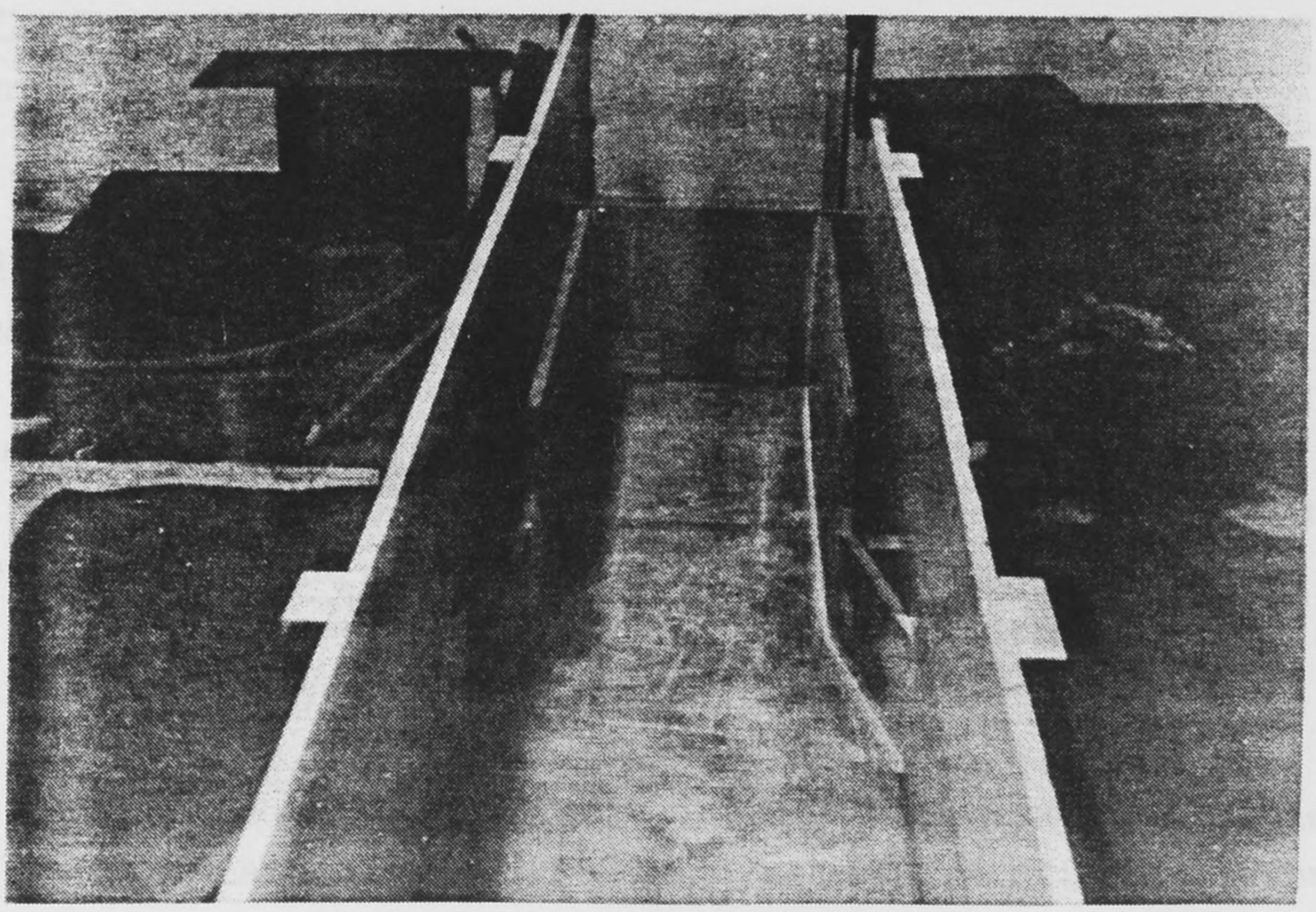

Figure 2f. Model of 3 ft by 3 ft sluice gates in channel to Market Avenue Pumping Station.

venturi meter was used to meter the flow with flow coefficients taken from Olson and Wright (1990).

Velocity measurements were obtained with a mini-propeller meter. This meter has a 1 cm diameter propeller and was calibrated in a towing tank to determine the velocity vs. revolution frequency relation. All measurements were averaged over at least 50 sec to filter out turbulent fluctuations.

Water levels were obtained at several locations within the model by means of installed point gages. All point gages were referenced to a common elevation by filling the model with an arbitrary amount of water and measuring the static water levels. Locations where point gages were installed for at least some measurements are indicated in Fig. 3. Head losses (more precisely hydraulic grade line changes) were determined from the differences in water surface elevations between the measurement points.

Many of the observations of flow conditions within the junction chamber were visual in nature and were recorded on videotape and still photographs. A videotape recording the nature of the flow for all tests performed was obtained and an edited version of the tape will be provided along with this project report.

TEST CONDITIONS

There are four different general cases that were studied. These are:

- 1. All influent flow to the retention basin. In general, these tests involved a total flow rate of 945 MGD with various distributions from the two influent channels. In some cases, a total flow of 1100 MGD was tested.
- 2. Dry weather flow conditions with no flow to the retention basin and all flow continuing down towards the MAPS.
- 3. An influent flow of 1100 MGD all discharged to the Grand River through the overflow gates (basin down condition).
- 4. An influent flow of 1100 MGD with 945 MGD to the retention basin and the remaining 155 MGD bypassed down the channel towards the MAPS.

The water depths in the prototype are generally intended to be maintained at as high of levels as feasible in the downstream wet wells to minimize the pumping lift at the two pump stations. During wet weather flows, the retention basin pump station provides the depth control and is intended to prevent significant surcharge

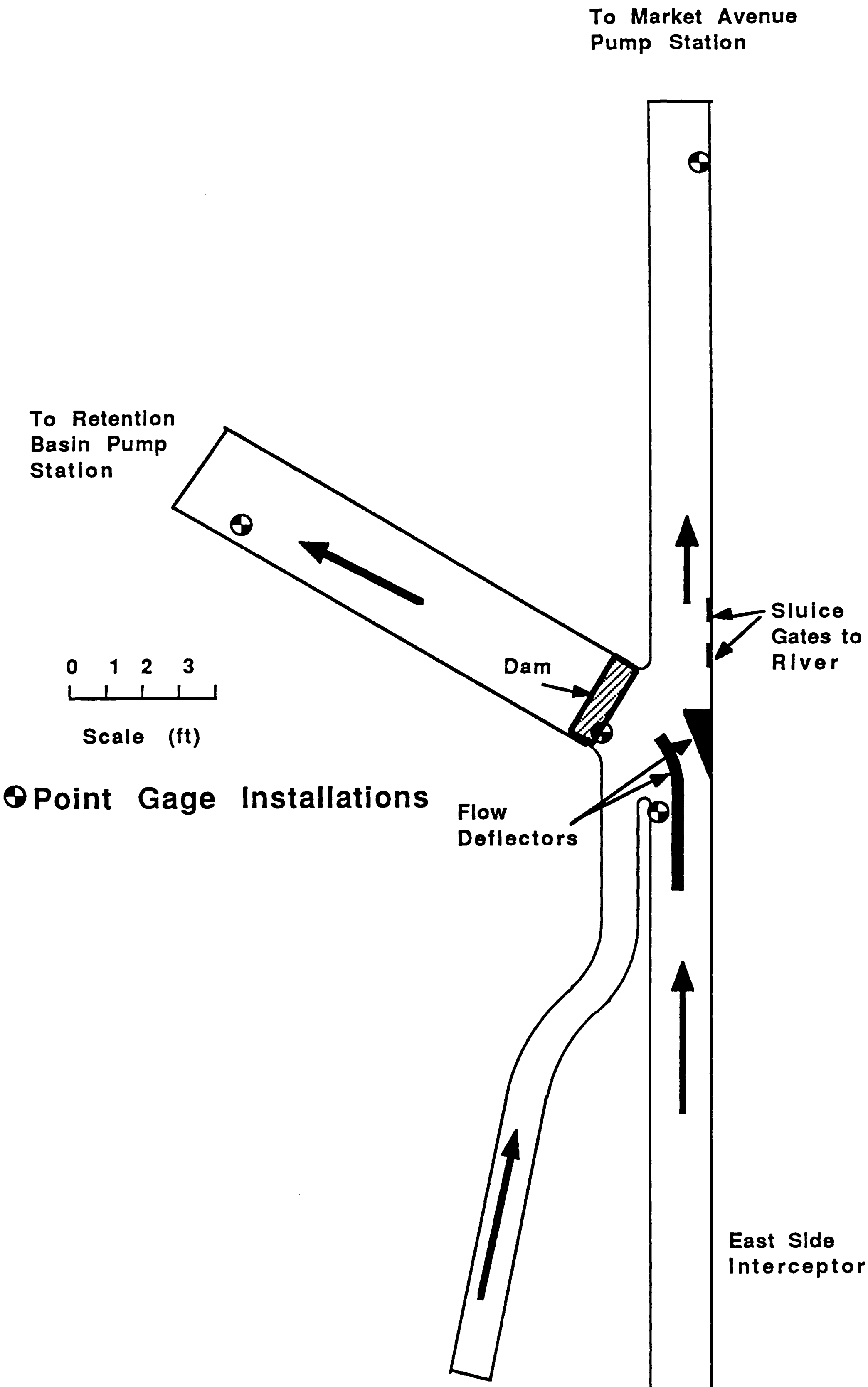

Figure 3. Locations for Point Gage Installation for Water Level Measurements.

in the upstream sewer. The wet well water surface elevation has been initially specified at 594 ft. Since the wet well was not part of the model, the water level within the model was determined by estimating the head losses through the bar screens and in the upstream conduit up to the downstream end of the model. The bar screen losses were estimated at 0.5 ft by McNamee, Porter, and Seeley engineers, and friction losses in the sewer between the end of the model and the retention basin were computed to be approximately 0.4 ft at a flow of 945 MGD. Allowing for miscellaneous additional losses, the downstream level in the model was maintained at approximately 595 ft. This was accomplished with the adjustable weir plates in a trial and error fashion in which the weir plate was set, the flow rate established and the water level checked. Subsequent adjsutment of the weir plate was performed until the downstream level of approximately 595 ft was obtained.

For dry weather flow conditions, water levels in the sewer must be maintained sufficiently low that overflow of the weir in the junction chamber does not occur. The length of the East Side Interceptor downstream from the junction chamber is sufficiently long that the flow approaches its uniform flow depth. Therefore, it was not necessary to regulate the flow depth with a weir at the downstream end and a free overfall condition was utilized in many of the tests. For those tests in which the effect of the two 3 ft by 3 ft sluice gates was examined, the water level on the downstream side of the model gates was maintained at a prototype depth of 2 ft in accordance with discussions regarding the intended operation of the system (water levels to be maintained in the MAPS).

During wet weather flow conditions and the retention basin not in operation, water levels would be controlled by the elevation of the Grand River, the rating curves of the Buffalo pumps, and/or the six sluice gates since these high flow rates would have to be discharged to the Grand River. A rating curve giving flow versus elevation was provided in a memo by Jim Smalligan of Fishbeck, Thompson, Carr & Huber prepared subsequent to a January 9, 1991 meeting in Grand Rapids in which projected flows were discussed. This memo contains a figure of the overflow capacity through four sluice gates and the Buffalo pumps and it appears that a maximum flow of close to 1200 MGD could be passed through the six sluice gates provided that the river elevation is less than about 582 ft. It is also assumed that this computation is associated with a water level of 594.79 in the sewer and the test for the overflow condition was performed with this elevation at the downstream end of the model. Bypass flows were modeled with this same downstream level.

TEST RESULTS

In the following presentation, the results from the testing program are presented.

Initial Design

The model was initially constructed and tested without the flow diverters in place. Several different flow conditions were tested with the flow to the retention basin, the flow diverters were installed and several of the tests repeated. This approach was taken as it was suspected that the backwater effects from the pumping station wet wells would sufficiently reduce the inertia in the approach flow that the flow would be able to negotiate the 60° bend without significant difficulty. This was essentially the case as the results of Table 1 indicate. Visually, the flow appeared very little different in the two sets of experiments with more differences indicated between individual tests in each group (depending upon the flow split between the two influent channels) than between otherwise equivalent flows with and without the flow diverters. It was expected that the inertia of the flow would concentrate the flow on the right hand side of the channel to the retention basin and this was found to be the case. Velocity measurements made at the quarter points across the channel always indicated the lowest velocities to be in the center of the channel, apparently due to the wake behind the support columns. The highest velocities were always on the right side of the channel and were on the order of 25 percent higher at the right quarter point compared to the left quarter point. The support column at the center of the weir caused turbulence and presumably considerable head loss in its wake. Dye injection indicated flow separation over the broad crested weir and dye would move upstream along the sloping face of the downstream end of the weir. This is also presumed to lead to significant head loss. Air also tended to be trapped along the roof of the conduit just downstream from the sloping lid, due to flow separation at the junction between the sloping lid and the flat roof. Finally, minor disturbances in the flow were observed at the corners to the channel leading to the retention basin, in particular on the left side where an intermittent air core vortex formed under some flow conditions. Figs. 4(a) and (b) indicate the nature of the flow at this corner for the two different weir configurations. It appears that little

TABLE 1

Initial Tests with Trapezoidal Weir

Test Condition	Hydraulic Grade Line Elevation Change (ft)	Ratio of Center Velocity to Left Velocity	Ratio of Right Velocity to Left Velocity
945 MGD, no flow diverters 344 side channel 601 main channel	1.82	0.78	1.5
1100 MGD, no flow diverters 400 side channel 700 main channel	2.12	0.63	1.27
945 MGD, no flow diverters 472.5 side channel 472.5 main channel	1.92	0.72	1.12
945 MGD, with flow diverters 472.5 side channel 472.5 main channel	2.02	0.95	1.19
945 MGD, with flow diverters 344 side channel 601 main channel	2.01	0.66	1.18
945 MGD 245 side channel 700 main channel	2.02	0.89	1.33
1100 MGD, with flow diverters 400 side channel 700 main channel	2.27	0.74	1.33

Note: Velocity Measurements were Taken at Mid-depth in Model Near at End of Retention Basin Channel. Right and Left Velocity Measurements were Taken at Quarter-points Across the Channel Width. See Fig. 3 for Point Gage Locations in Main Channel and at end of Retention Basin Channel.

Table 1. Summary of Experimental Test Results for Retention Basin Flows

TABLE 1 (Continued)

Final Tests with Ogee Weir

Test Condition	Hydraulic Grade Line Elevation Change (ft)	Ratio of Center Velocity to Left Velocity	Ratio of Right Velocity to Left Velocity
945 MGD, no flow diverter 344 side channel 601 main channel	1.62	0.77	1.41
945 MGD, flow diverter downstream 344 side channel 601 main channel	1.62	0.78	1.37
945 MGD, flow diverter upstream 344 side channel 601 main channel	1.64	0.81	1.18
1100 MGD, no flow diverter 472.5 side channel 472.5 main channel	1.93	0.83	1.28

Note: Velocity Measurements were Taken at Mid-depth in Model Near at End of Retention Basin Channel. Right and Left Velocity Measurements were Taken at Quarter-points Across the Channel Width. See Fig. 3 for Point Gage Locations in Main Channel and at end of Retention Basin Channel.

Table 1. Summary of Experimental Test Results for Retention Basin Flows.

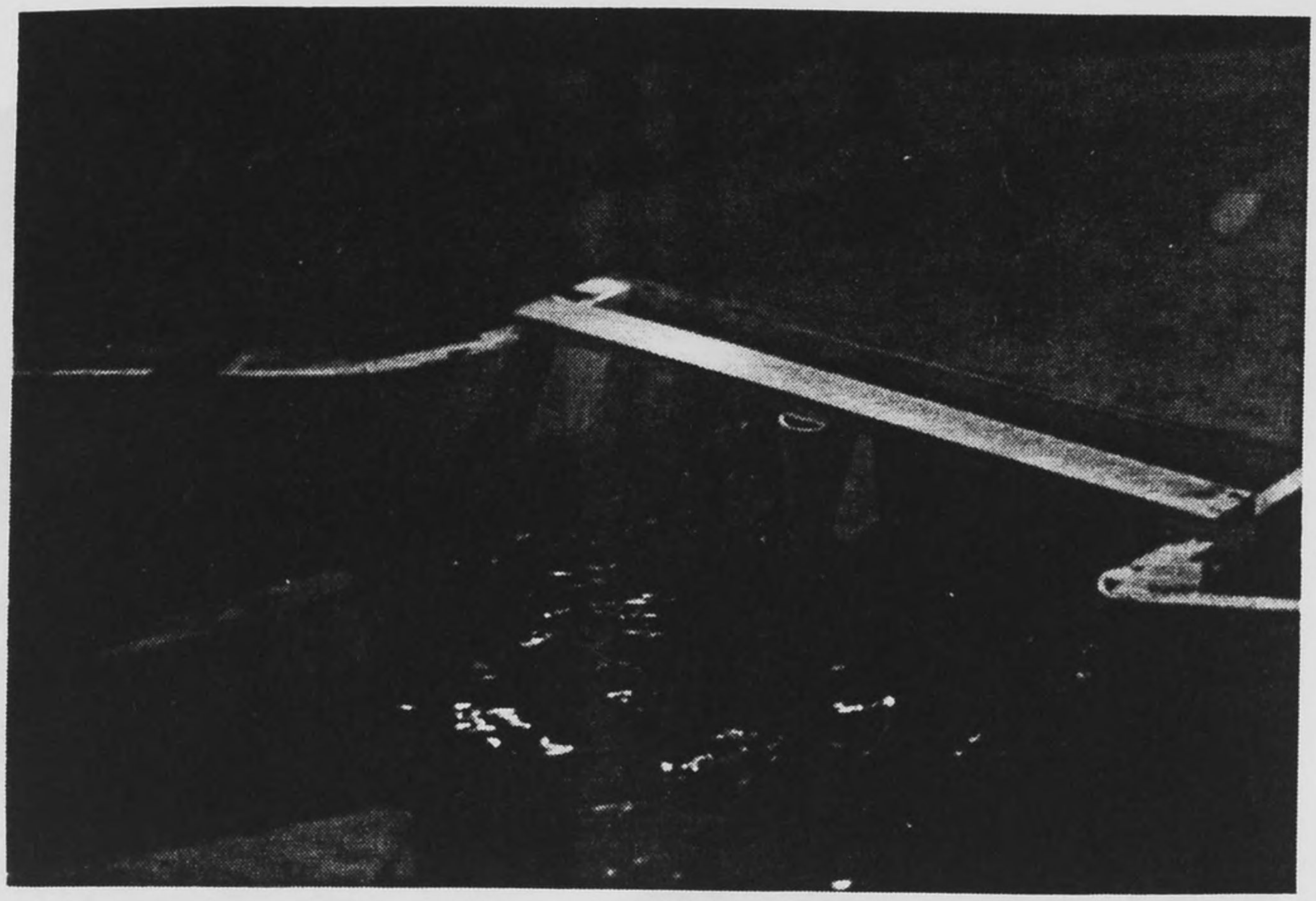

Figure 4a. Surface Disturbances Due to Deflection of Flow in Junction Chamber to Retention Basin. Flow Diverters in Place, Original Weir Section.

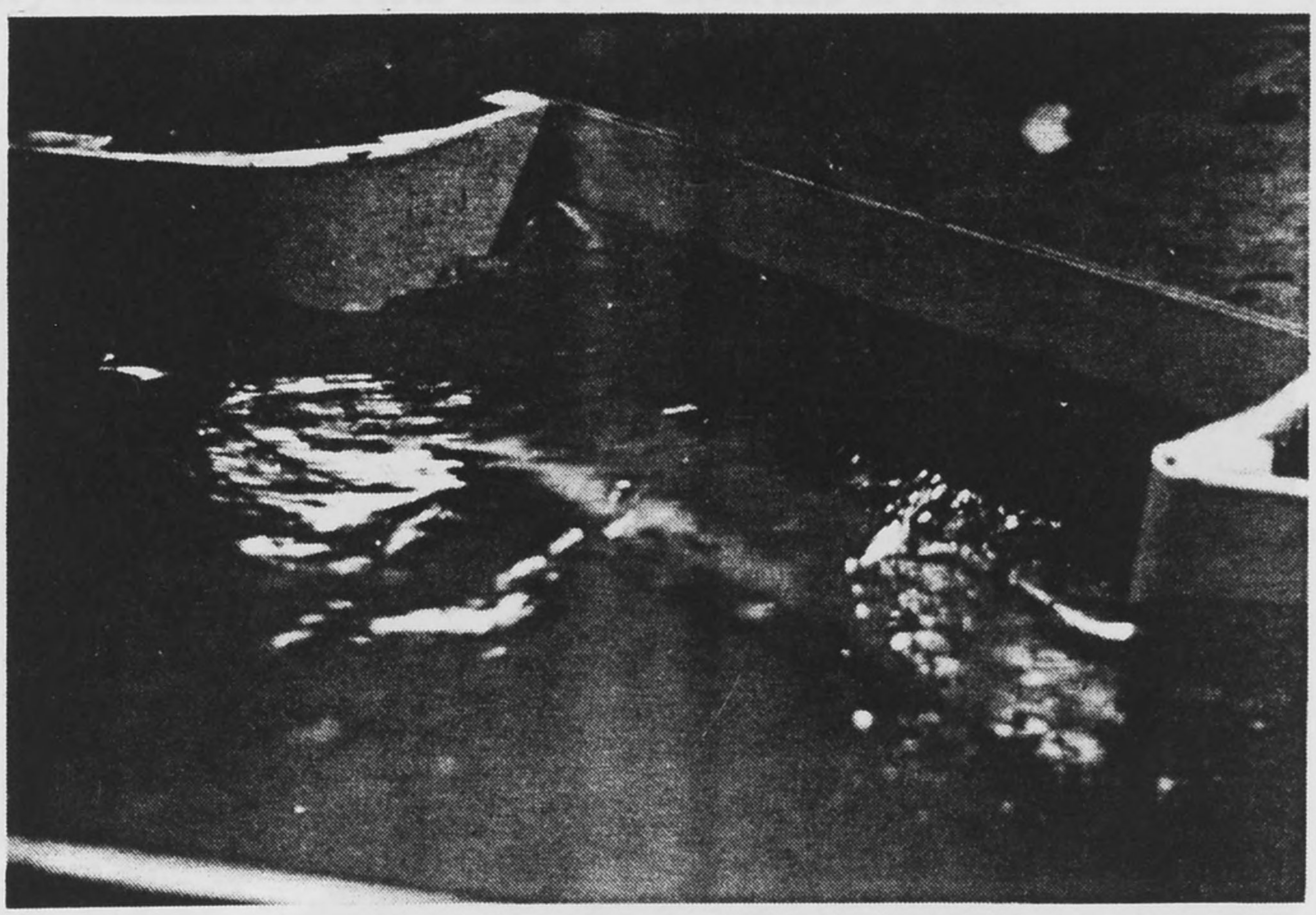

Figure 4b. Surface Disturbances Due to Deflection of Flow in Junction Chamber to Retention Basin. Flow Diverters Removed, Ogee Weir Section.

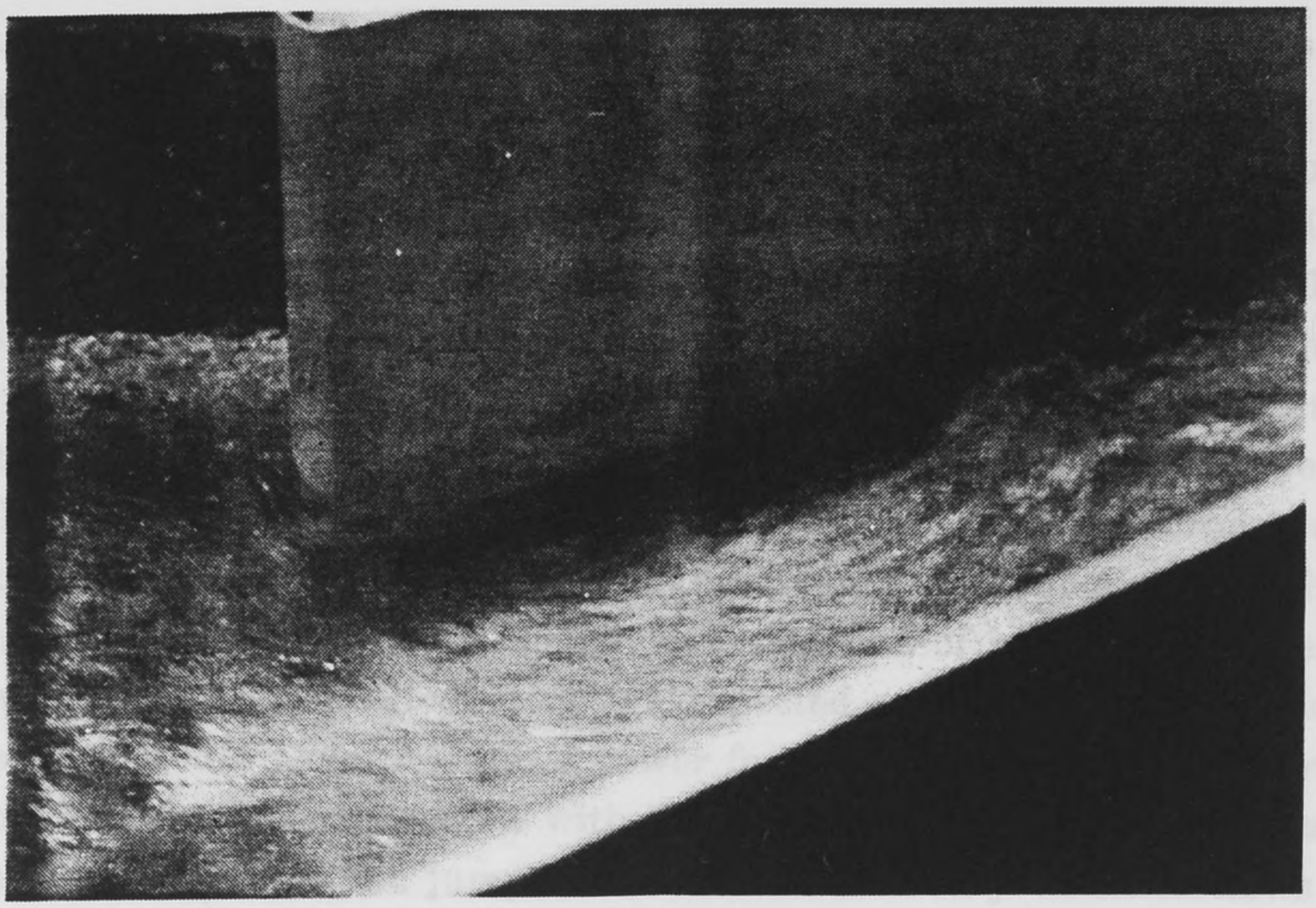

Figure 4c. Hydraulic Jump Formed at Downstream End of Junction Chamber During Dry Weather Flow Tests with Flow Diverters in Place.

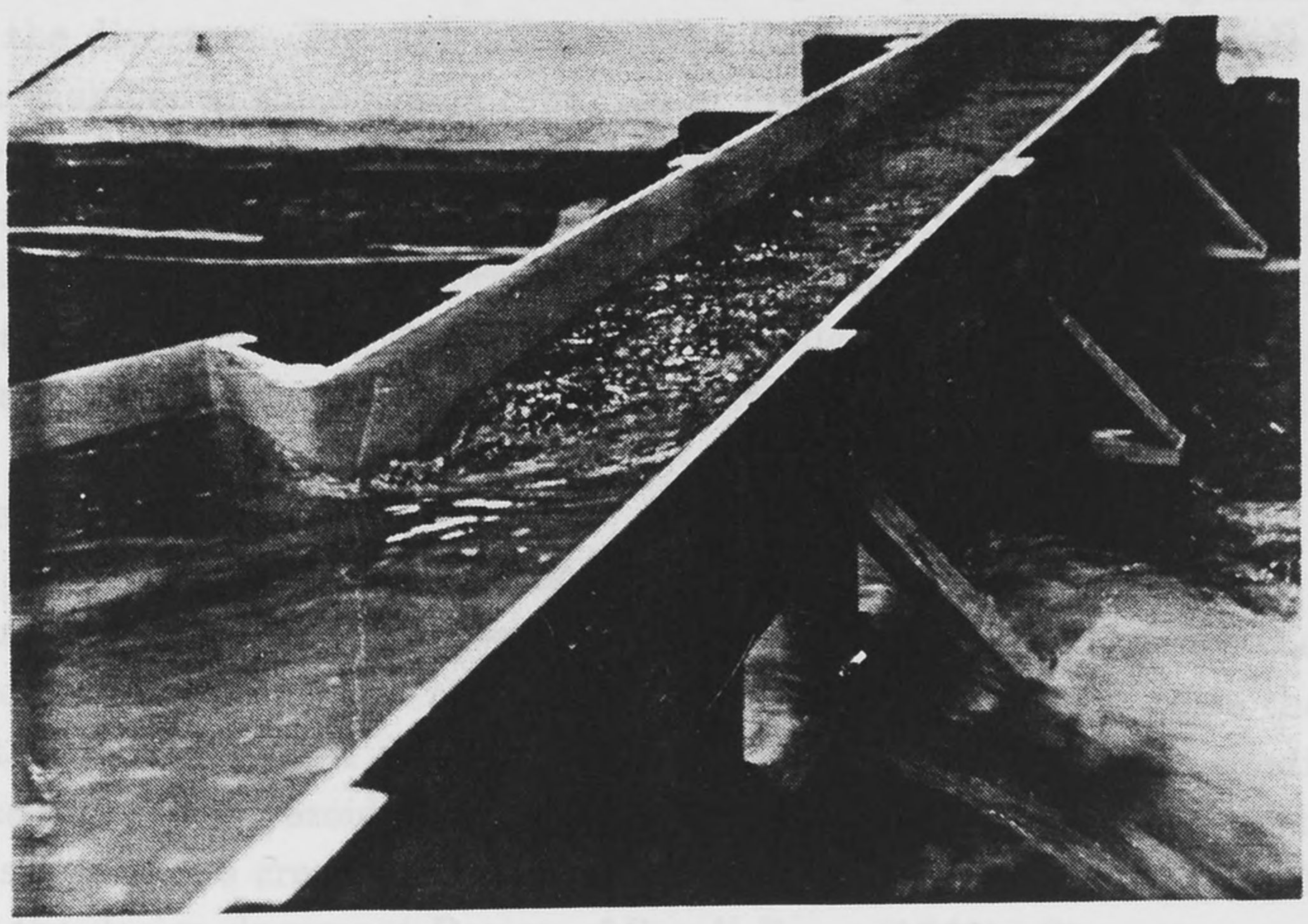

Figure 4d. Surface Disturbances with 1100 MGD to Grand River Overflow.

else can be accomplished to improve the flow at these corners and these flow disturbances are relatively minor in any case.

The major differences in the flow with and without the diverters in place occurred in those flows that continued on to the MAPS. This could be anticipated since the projected area of the diverters essentially blocked the main channel, deflecting the flow first to the left from which it must go back towards the right before it can proceed down the channel. This resulted in a significant flow separation off the right corner of the upstream end of the channel to the retention basin (left side of the main channel). The flow from the influent branch conduit also tended to create a separation point there, but the problem was much worse with the diverters installed. Although the condition of high wet weather flows overflowing to the Grand River is not an expected condition (the retention basin pump station would have to suffer a power failure or some similar problem), the flow would be clearly unacceptable with the diverters in place for that situation as a large standing hydraulic jump formed. Even the dry weather flows would result in a hydraulic jump off that corner as indicated in Fig. 4(c). Because of this situation and the general ineffectiveness of the flow diverters to improve the flow to the retention basin, a decision was made fairly early in the testing schedule to remove the diverters. Fig. 4(d) indicates the flow at the downstream side of the junction chamber at the extreme condition of 1100 MGD passing through the two sluice gates contained within the model (visible in the lower right side of the photograph) and on towards the other overflow devices further downstream. Again, the separation at the corner is visible in the photo, but the condition is much less severe than with the flow diverters in place and as mentioned, this flow condition is not expected to be encountered during any normal operating state.

Modifications To Model

As a part of the changes in the model that were made as a result of the initial investigation, the broad-crested weir was changed to attempt to reduce the head losses due to flow separation on the downstream side and also to raise the weir height to accommodate a higher water level during dry weather conditions. Consequently, the trapezoidal shape of the weir was altered to an ogee crest. The specific shape of the crest was obtained from guidance for free overfall spillways given in the design document *Design of Small Dams* (1960). Since the ogee crest will not be of the free overfall type in this configuration, judgement was applied to estimate a design suitable to this configuration. The upstream face of the ogee was moved upstream relative to the original trapezoidal section and the downstream end extended further downstream. A comparison of the two sections is given in Fig. 5. Tests with the new weir in place were conducted in a manner similar to those for the original design and the results are presented in Table 1. The initial testing with this new weir configuration considered the possibility of retaining the diverter on the right wall of the junction chamber. Preliminary experiments indicated no change in upstream head if the diverter was removed and the flow was somewhat less disturbed within the junction chamber, primarily due to the absence of vortices shed off the downstream end of the flow diverter. The tests at 945 MGD indicated that a slight improvement in the flow distribution within the retention basin channel could be obtained if the flow diverter was in place.

Initial dry weather flow tests are not reported herein because they were conducted with the initial design and did not consider the presence of the two 3 ft by 3 ft sluice gates installed in the channel to the MAPS downstream from the junction chamber. Final testing for dry weather flow conditions was conducted with the 3 ft high ogee weir crest in place and with all flow diverters removed. The main purpose of these tests was to determine the backwater effect at the junction chamber due to the sluice gates and to determine the maximum dry weather flow that could be passed to the MAPS without overflowing the ogee weir. Since the sluice gates are located further downstream from the junction chamber than the extent of the physical model, the tests were conducted by placing the model gates near the downstream end of the physical model, Fig. 2(f). These gates are also located in a section of the channel that is only 12 ft wide as compared to the 17 ft width in the section of conduit modeled in the physical model. Consequently an insert was placed within the channel that correctly reproduced the gate dimensions (opening plus wall thickness) as well as the total channel width. The location of this insert was approximately 75 ft (prototype) downstream from the end of the junction chamber whereas the actual sluice gates were approximately 218 ft downstream. The differences in backwater effect were accounted for with a numerical solution (presented in Appendix A) for the gradually varied flow from the gates upstream to the junction chamber for the flow rate of 61.8 MGD prototype. The lengths of the channel sections that were 12 ft wide (30 ft in the model and 100 ft in the prototype) were included in the numerical integration, but no losses at the transition sections were included. The downstream water depth (just upstream of the gates) used in the numerical

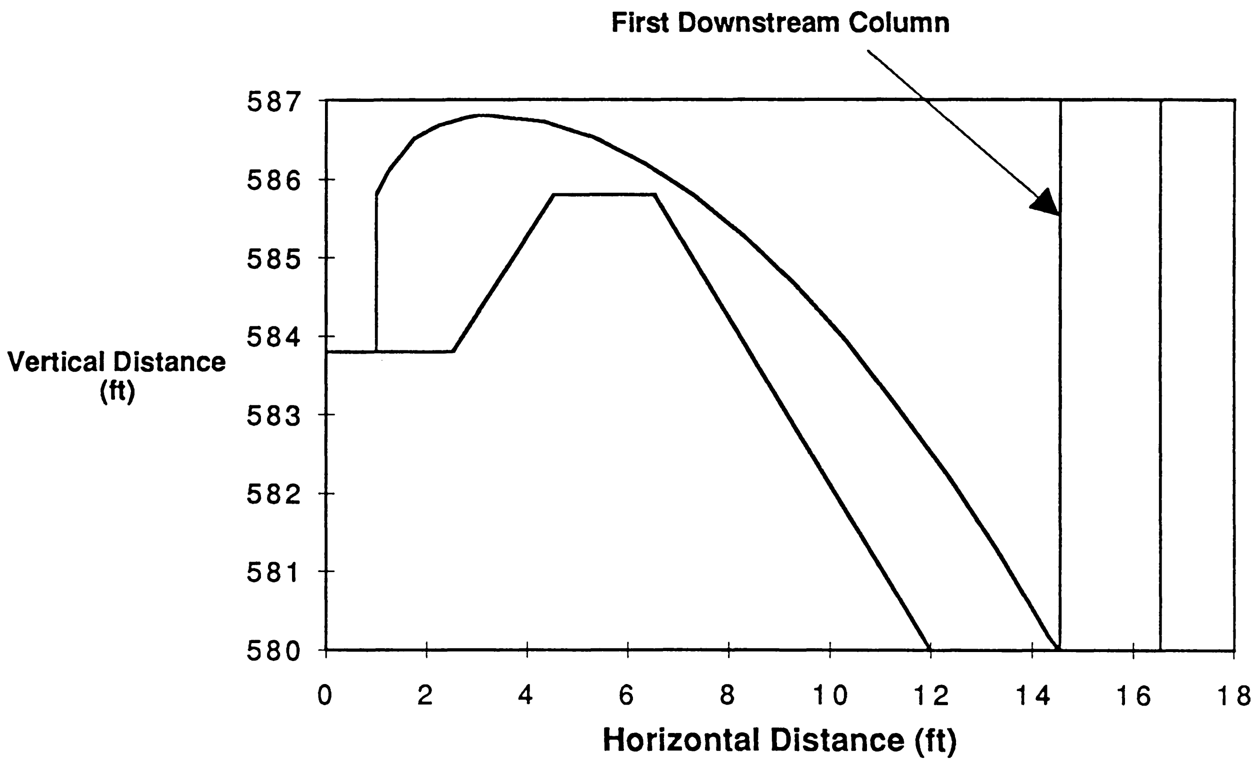

Figure 5. Comparison of Two Overflow Weir Cross-Sections. Indicasted Position of Ogee Weir Crest is Relative to Position of Trapezoidal Weir in Initial Design.

analysis was 3.0 ft since the water in the model was observed to be in contact with the top of the gates with an occasional free surface condition through the gates. According to the analyses presented in Appendix A, at a flow rate of 61.8 MGD, the water depth would be 2.93 feet at the downstream end of the junction chamber in the model. At this flow rate, water was observed to periodically spill over the 3 ft high ogee weir in the junction chamber, so this computational result appears to be quite reasonable. In the prototype, the water depth just downstream from the junction chamber was computed to be 2.81 ft or only 0.12 ft less than observed in the model. The difference between the two cases is probably within the uncertainty in the estimate of the resistance coefficient and so the maximum dry weather flow rate through the channel to the MAPS is estimated to be approximately 60 MGD without overflow of the ogee weir.

The results of water level measurements for the dry weather flow tests are included in Table 2. Water levels downstream from the sluice gates were maintained to provide a depth of approximately 2 ft; the measured values are recorded in the table. Also recorded are the water depths on the upstream side of the gate and in the main channel just upstream from the junction chamber (See Fig. 3 for the location of the point gage within that channel). During these flow tests, all water was passing down this main channel so it is reasonable that the water depth is slightly higher than in the junction chamber.

The remaining tests performed were relatively incidental to the overall investigation. Tests were performed for two different flow conditions with the junction chamber in the final design configuration, i.e. with the ogee weir crest in place and with no flow diverters.

The first case considered was a flow of 1100 MGD with 945 MGD diverted to the retention basin and the remaining 155 MGD passing downstream towards the MAPS. The assumption was that this flow would be discharged to the Grand River either through the Buffalo pumps or through the sluice gates downstream from the end of the model. This flow was established by setting an initial flow of 945 MGD through the retention basin channel and noting the water elevation at the end of the retention basin channel. A flow of 1100 MGD was then established and the weir plate on the channel passing to the MAPS was adjusted until the same water surface elevation was re-established. Only visual observations of this flow condition were made and this was recorded on the videotape. The only notable result that was observed was that the flow along the surface in the main channel tended to continue on downstream while the flow along the bottom was

TABLE 2

Flow Rate (MGD)	Depth Downstream from Gates (ft)	Depth Upstream from Gates (ft)	Depth Upstream of Junction Chamber
41.6	2.03	2.69	2.68
51.1	1.89	2.87	2.90
61.8	2.03	3.12	3.17

Note : For last test, water level was intermittently touching top of gates; all other test had free surface flow through gates.

Table 2. Water Levels Near Junction Chamber for Flow Through 3 ft by 3 ft Sluice Gates.

more likely to be diverted towards the retention basin. This must be mainly due to the inertia in the higher velocity flows at the surface. Dye injections along the bottom indicated that flow in the leftmost two-thirds of the main channel passed over the ogee weir while only approximately one-half of the flow along the surface did the same.

A second test that was generally similar in nature considered the flow in the channel under the unlikely circumstances of the retention basin pump station not operating at the same time that a flow of 1100 MGD is passing through the system. The main purpose here was to examine the nature of the flow in the junction chamber under the situation where the two sluice gates in the sewer wall (see Fig. 1 for approximate locations) just downstream from the junction chamber were fully open. No attempt was made to create a backwater effect on the downstream side of these gates as that would create the maximum flow through the sluice gates and thus the most disturbance in the flow passing through the junction chamber. As mentioned previously, a photograph of this flow condition is presented in Fig. 4(d). In that photograph, one can see the tendency for a drop in the water surface and a partial hydraulic jump to form off the left corner (looking downstream) of the downstream end of the junction chamber. However, the flow disturbance is relatively minor and since it would be extremely unlikely that this flow condition would ever occur, it does not appear to be a matter for concern. The flow passing through the gate was estimated by initially setting the downstream level at 595 ft with the weir plate and passing 1100 MGD through the system. The two sluice gates were blocked off and flow was re-initiated until the same downstream water level was achieved at a total flow rate of approximately 634 MGD. This implies that a total flow rate of 466 MGD was passing through the two sluice gates. Not accounting for the side contractions in the flow through the gates, a single sluice gate in an in-line configuration should be capable of passing around 320 MGD using the flow coefficients from Olson and Wright (1990). The average flow of 233 MGD per gate would appear to be reasonable given the combined effects of the side contraction and the fact that the main flow has to be deflected by 90° to pass through the gates. Assuming that all six sluice gates in the channel side have the same dimensions, they should have a combined capacity of at least 1400 MGD which should be more than sufficient to pass the design flow to the Grand River under a low river condition. Under higher river elevations, the flows passing through the sluice gates would be necessarily less but this was not considered in the testing, again because this flow condition is considered to be unlikely.

A videotape was made of segments of all the different experimental conditions examined. Copies are being provided along with the final report. A shorter edited version of the tape that gives more highlights of the testing results is also included. Documentation of the different tests being presented on the tape is also provided.

DISCUSSION

The test results indicated that the flow distribution within the retention basin channel is relatively insensitive to the approach conditions in and upstream of the junction chamber. All variations in flow velocity measured in the downstream end of the retention basin channel are qualitatively as expected. The flow tended to be higher on the right hand side of the channel, a result that would occur due to the inertia of the flow as it is forced through the 60° bend. Also, the higher the flow rate in the main channel, the larger the relative flow on the right hand side of the channel. The flow diverters tended to make the flow distribution more uniform with improvements of less than ten percent in most cases indicated by the velocity measurements. The same can generally be said for the presence of the flow diverter in the later tests with the ogee weir section in place. These improvements would appear to be relatively minor and difficult to quantify precisely since the model did not extend all the way down to the bar screens at the retention basin pumping station. The improvement in flow distribution does not appear to warrant the expense of installing the diverters and allows for better flow conditions over a wider range of flows when the water is not being diverted to the retention basin.

For all flows investigated, the final junction chamber configuration produced a flow that was relatively free of turbulence and no severe flow problems are anticipated with this recommended design. The redesign of the overflow weir and the removal of the flow diverters have minimized the flow separation that may occur within the junction chamber and have removed the source of several large scale vortices that were observed during the testing. The smaller scale phenomena that remain in the testing with the final design are probably unavoidable due to the fact that several different flow conditions may occur with flow coming from or going to different locations, so no design can optimize the flow for all these different conditions.

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APPENDIX

RESULTS OF GRADUALLY VARIED FLOW ANALYSIS FOR FLOW UPSTREAM FROM 3 ft by 3 ft SLUICE GATES

Model Test, Gradually Varied Flow Profile

	A	B	C	D	E	F	G
1	Q	95.8			MODEL TEST	CONDITIONS	
2	b	17					
3	m	0					
4	g	32.2					
5	n	0.013					
6	Cm	1.49					
7	S0	0.001					
8			Ycritical		Ynormal		
9							
10		Yhi	0.99538808		Yhi	1.38419751	
11		Ylow	0.99538808		Ylo	1.38419751	
12		Yavg	0.99538808		Yavg	1.38419751	
13		F(y)	-2.442E-15		F(y)	1.1894E-08	
14		Yhinew	0.99538808		Yhinew	1.38419751	
15		Ylonew	0.99538808		Ylow new	1.38419751	
16							
17	Gradually	Varied Flow					
18				TRANSITION	FROM 12 ft	TO 17 ft	
19	Δx	-5		CHANNEL IS	AT 30 ft		
20	X1	0					
21	Y1	3					
22		x	y	S0	f(y)	y'	f(y')
23		0	3	0.001	0.00084836	2.99575819	0.0008477
24		-5	2.99575984	0.001	0.0008477	2.99152132	0.00084704
25		-10	2.99152298	0.001	0.00084704	2.98728779	0.00084637
26		-15	2.98728945	0.001	0.00084637	2.98305759	0.0008457
27		-20	2.98305927	0.001	0.0008457	2.97883077	0.00084503
28		-25	2.97883245	0.001	0.00084503	2.97460732	0.00084435
29		-30	2.97460902	0.001	0.00084435	2.97038727	0.00084367
30		-35	2.97038898	0.001	0.00093973	2.96569035	0.00093943
31		-40	2.96569109	0.001	0.00093943	2.96099394	0.00093913
32		-45	2.96099468	0.001	0.00093913	2.95629903	0.00093883
33		-50	2.95629978	0.001	0.00093883	2.95160562	0.00093853
34		-55	2.95160638	0.001	0.00093853	2.94691373	0.00093822
35		-60	2.94691449	0.001	0.00093822	2.94222337	0.00093792
36		-65	2.94222413	0.001	0.00093792	2.93753454	0.00093761
37		-70	2.93753531	0.001	0.00093761	2.93284726	0.0009373
38		-75	2.93284804	0.001	0.0009373	2.92816154	0.00093699

	A	B	C	D	E	F	G
1	Q	95.8			PROTOTYPE	CONDITIONS	
2	b	17					
3	m	0					
4	g	32.2					
5	n	0.013					
6	Cm	1.49					
7	S0	0.001					
8			Ycritical		Ynormal		
9							
10		Yhi	0.99538808		Yhi	1.38419751	
11		Ylow	0.99538808		Ylo	1.38419751	
12		Yavg	0.99538808		Yavg	1.38419751	
13		F(y)	3.7748E-15		F(y)	-1.98E-08	
14		Yhinew	0.99538808		Yhinew	1.38419751	
15		Ylonew	0.99538808		Ylow new	1.38419751	
16							
17	Gradually	Varied Flow					
18				TRANSITION	FROM 12 ft	TO 17 ft	
19	Δx	-5		CHANNEL IS	AT 100 ft		
20	X1	0					
21	Y1	3					
22		x	y	S0	f(y)	y'	f(y')
23		0	3	0.001	0.00084836	2.99575819	0.0008477
24		-5	2.99575984	0.001	0.0008477	2.99152132	0.00084704
25		-10	2.99152298	0.001	0.00084704	2.98728779	0.00084637
26		-15	2.98728945	0.001	0.00084637	2.98305759	0.0008457
27		-20	2.98305927	0.001	0.0008457	2.97883077	0.00084503
28		-25	2.97883245	0.001	0.00084503	2.97460732	0.00084435
29		-30	2.97460902	0.001	0.00084435	2.97038727	0.00084367
30		-35	2.97038898	0.001	0.00084367	2.96617065	0.00084298
31		-40	2.96617236	0.001	0.00084298	2.96195746	0.00084229
32		-45	2.96195919	0.001	0.00084229	2.95774774	0.0008416
33		-50	2.95774947	0.001	0.0008416	2.95354149	0.0008409
34		-55	2.95354323	0.001	0.0008409	2.94933873	0.0008402
35		-60	2.94934049	0.001	0.0008402	2.9451395	0.00083949
36		-65	2.94514126	0.001	0.00083949	2.9409438	0.00083878
37		-70	2.94094557	0.001	0.00083878	2.93675165	0.00083807
38		-75	2.93675344	0.001	0.00083807	2.93256309	0.00083735
39		-80	2.93256488	0.001	0.00083735	2.92837811	0.00083663
40		-85	2.92837992	0.001	0.00083663	2.92419676	0.00083591
41		-90	2.92419857	0.001	0.00083591	2.92001904	0.00083518
42		-95	2.92002086	0.001	0.00083518	2.91584498	0.00083444
43		-100	2.91584681	0.001	0.00083444	2.91167459	0.00083371
44		-105	2.91167643	0.001	0.00093587	2.90699708	0.00093555
45		-110	2.90699789	0.001	0.00093555	2.90232015	0.00093522
46		-115	2.90232096	0.001	0.00093522	2.89764484	0.0009349
47		-120	2.89764565	0.001	0.0009349	2.89297117	0.00093457
48		-125	2.89297199	0.001	0.00093457	2.88829914	0.00093424
49		-130	2.88829997	0.001	0.00093424	2.88362878	0.00093391
50		-135	2.88362961	0.001	0.00093391	2.87896009	0.00093357
51		-140	2.87896092	0.001	0.00093357	2.87429307	0.00093323
52		-145	2.87429392	0.001	0.00093323	2.86962776	0.00093289
53		-150	2.86962861	0.001	0.00093289	2.86496414	0.00093255
54		-155	2.864965	0.001	0.00093255	2.86030225	0.00093221
55		-160	2.86030311	0.001	0.00093221	2.85564208	0.00093186
56		-165	2.85564295	0.001	0.00093186	2.85098365	0.00093151
57		-170	2.85098453	0.001	0.00093151	2.84632698	0.00093116
58		-175	2.84632786	0.001	0.00093116	2.84167207	0.0009308
59		-180	2.84167295	0.001	0.0009308	2.83701893	0.00093045
60		-185	2.83701982	0.001	0.00093045	2.83236758	0.00093009
61		-190	2.83236848	0.001	0.00093009	2.82771804	0.00092973
62		-195	2.82771894	0.001	0.00092973	2.8230703	0.00092936

Prototype Conditions, Gradually Varied Flow Profile

	A	B	C	D	E	F	G
63		-200	2.82307121	0.001	0.00092936	2.81842439	0.000929
64		-205	2.81842531	0.001	0.000929	2.81378032	0.00092863
65		-210	2.81378125	0.001	0.00092863	2.8091381	0.00092826
66		-215	2.80913903	0.001	0.00092826	2.80449775	0.00092788
67		-220	2.80449868	0.001	0.00092788	2.79985927	0.00092751

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AIIM SCANNER TEST CHART # 2

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Greek and Math Symbols

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6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋗⋖⋗⋜≡

8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋗⋖⋗⋜≡

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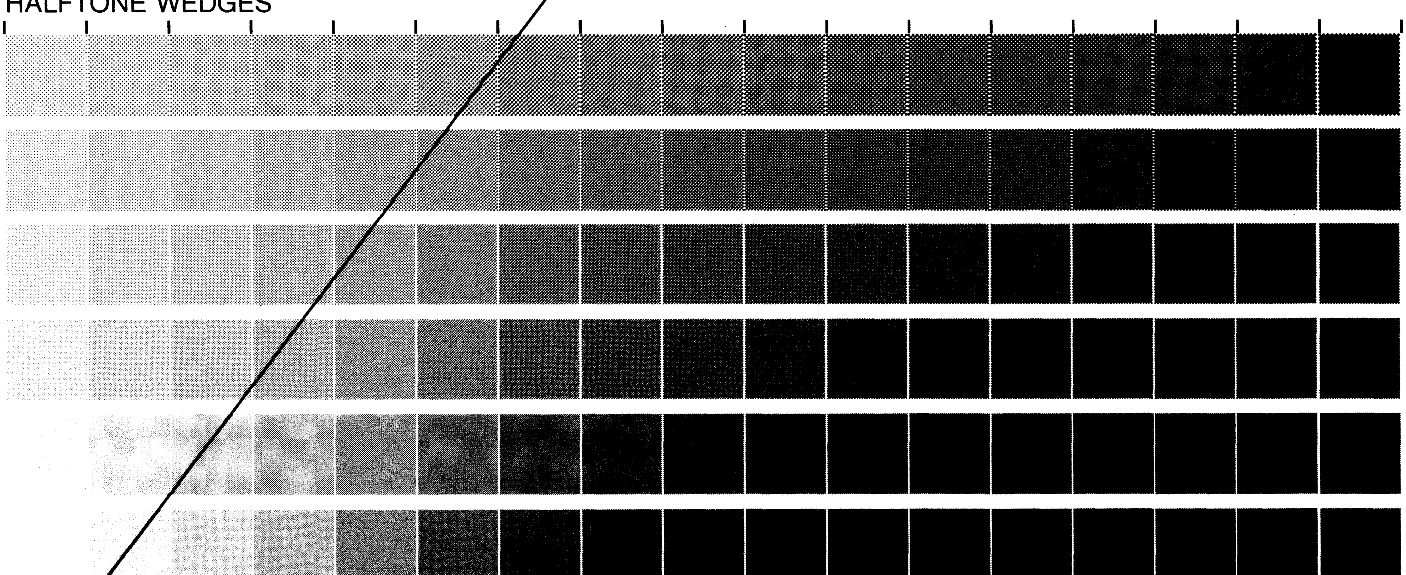

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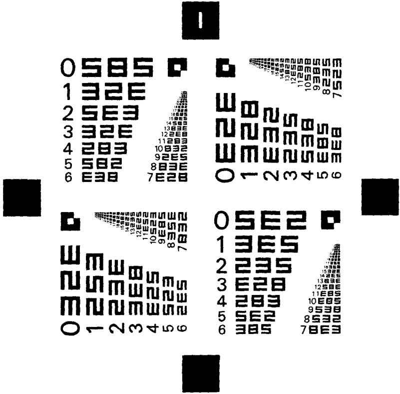
RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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