HYDRAULIC MODEL STUDY
East Dearborn CSO Control Program
CSO Outfall 017
Report CEE 05-07

Final Project Report to
NTH Consultants, Ltd.
Detroit, Michigan

By
Steven J. Wright
Megan Holcomb
Michael Konieczki

Department of Civil and Environmental Engineering
The University of Michigan
Ann Arbor, Michigan

June 2005

TABLE OF CONTENTS

List of Figures

EXECUTIVE SUMMARY

A physical hydraulic model study of the CSO Regulator No. 017 control facility, located in East Dearborn was constructed at a 1:19 scale of the prototype. Model dimensions were scaled from provided Basis of Design drawings. The purpose of the model was to:

- Measure head losses through various parts of the model for different baffle wall elevations,
- Investigate the retention time of flow entering the treatment facility,
- Determine the effect of the baffle wall height on contact time through the treatment facility,
- Identify potential forces of falling water on the floor of the shaft

The following recommendations are made regarding the treatment shaft configuration:

The baffle wall height (bottom elevation) should be set at an elevation that does not have a significant effect on the upstream hydraulic grade line elevation and keeps the chlorine contact time above the allowable limit of 10 minutes. From the headloss experiments, this elevation is between 457 and 502 feet. The dye breakthrough experiment results indicated any elevation between 458 and 513 feet had no significant effect on contact time. In the design derived in part from the model results, the baffle wall elevation has been set at 459.5 feet. This falls within the acceptable range of elevations for the baffle wall.

The divider wall at the inlet does not have any effect on the hydraulic function of the structure. It can be included in the constructed project for structural reasons, but has no affect on the hydraulics.

Attempts were made to determine the feasibility of measuring local impact pressures on the bottom of the shaft from falling inflow during initial phases of the filling process. Investigation of possible scale effects at the low initial discharges that would be associated with the early phases of the filling process showed equivocal results and it was concluded that it would not be possible to accurately determine prototype pressures in the 1:19 scale model. However, at all but the most extreme inflow conditions, it appears that the influent will not fall as a free jet at the initial inflows that will be expected at the commencement of the inflow hydrograph. By the time the inflow increases to a rate that would result in a free jet, a water cushion will be present at the bottom of the shaft, significantly reducing the magnitude of the impact pressures.

INTRODUCTION

A treatment shaft facility is proposed to provide for combined sewer overflow (CSO) control in the City of Dearborn, Michigan. Since As with most CSO detention facilities, the structure is intended to reduce the number and magnitude of combined sewage releases into the environment and to provide treatment (disinfection, skimming and settling) for any releases that do occur. A key aspect of the treatment shaft design is to make use of limited available space by constructing a vertical large diameter shaft, approximately 160 feet deep and 95 feet in diameter, to provide the required storage volume. The storage volume is selected to provide the required disinfection contact time prior to overflow to the Rouge River. Under overflow circumstances, the treatment shaft is designed to pass flow without exceeding a limiting hydraulic grade line elevation upstream of the structure. Released effluent must also be disinfected by maintaining a minimum chlorine contact time prior to discharge. Although the structure is designed to provide a mean residence time (defined by the storage volume divided by the flow-through rate) to satisfy the minimum contact time requirement, short-circuiting of flow through the structure would result in a reduced contact time. A vertical baffle wall will be installed at the center of the shaft to direct flow downward on the inlet side and underneath the baffle wall in order to avoid short-circuiting of the flow. The concept of the treatment shaft is relatively new and the physical hydraulic model study will be used to back up analyses related to the head loss and other aspects of the system behavior.

A key aspect of the design is selection of the design elevation of the bottom of the baffle wall. Too low an elevation will increase the head loss through the structure and increase the upstream hydraulic grade line above acceptable levels. On the other hand, if the elevation of the baffle wall is too high, effluent may pass through the structure more quickly than desired, resulting in shorter chlorine contact time for individual parcels of water.

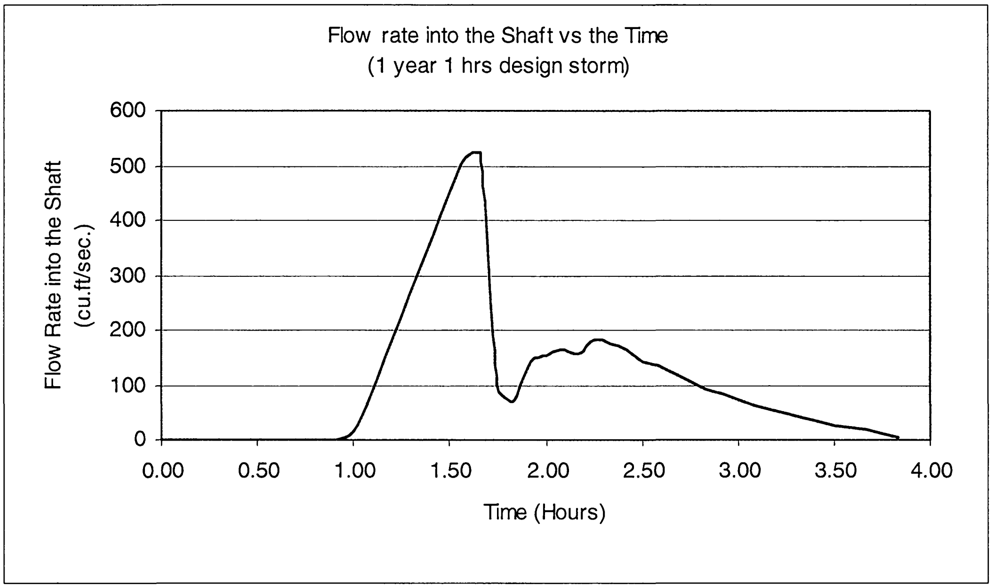
The primary purpose of the hydraulic model was to examine the effect of the baffle wall bottom elevation both on head loss through the structure and the breakthrough of a tracer injected into the influent. Experiments in which the baffle wall elevation was systematically varied were performed to address each of these issues. A few additional issues were addressed during the testing. One of these was the necessity of installing a splitter wall on the inlet side of the shaft. A second issue was the magnitude of large impact pressure fluctuations that might be expected on the floor of the shaft during the initial phase of shaft filling when the influent would be free falling from the inlet to the shaft bottom. Additional experiments were performed to address these issues. This report describes the physical hydraulic model that was constructed to investigate these various issues, the testing procedures that were employed, and the testing results. Implications with respect to structure design are addressed.

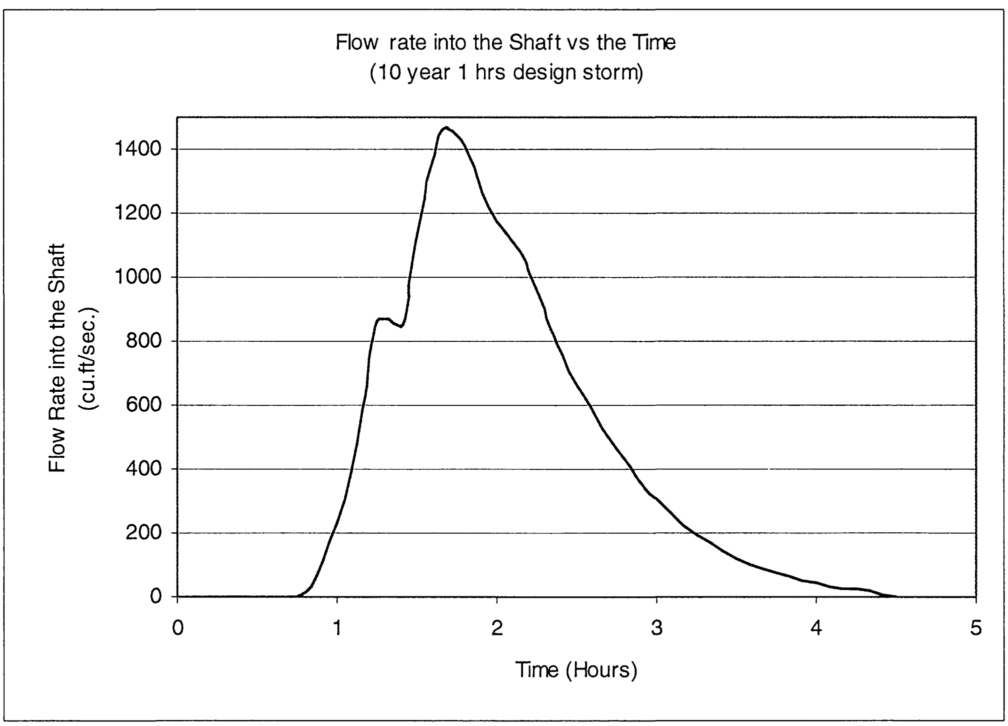
GENERAL SYSTEM DETAIL

More than one treatment shaft has been designed; the one in question is referred to as CSO Shaft No. 017. Original designs intended to place the shaft in line by removing a section of the existing sewer (approximately 14-foot by 18-foot arch) and installing the shaft with upstream and downstream transitions to the existing sewer. Subsequent modifications to the original design resulted in a change of shaft location and required the construction of upstream and downstream sewer segments to tie the shaft to the existing system. The general layout of the structure is indicated in Appendix A. The proposed diameter of the treatment shaft is 95 feet and the maximum depth of the structure is approximately 167 feet. Although the structure is covered, it is intended to flow with a free surface under normal operating conditions. During dry weather flow conditions, water does not enter the shaft but is carried through existing interceptors to the wastewater treatment plant. In the event of a significant rainfall, the lack of capacity in the existing sewer system will cause overflow into the shaft. Smaller rainfall events will be contained entirely within the shaft but for greater inflows, the storage capacity of the shaft will be exceeded and overflow will pass through the structure to a discharge point in the Rouge River. A number of hydrological conditions have been analyzed that relate to limiting hydraulic conditions investigated in the hydraulic model. These conditions were analyzed by others and were provided as inputs into this study. Key hydrological parameters include:

- Various inflow hydrographs have been developed for inflow into the shaft. See Figure 1.
- The maximum discharge through the structure for the 5-year, 24-hour storm at average Rouge River levels is 1,867 cubic feet per second (cfs);
- The peak five-minute flow associated with the 10-year, 1-hour storm is 1,457 cfs;
- The maximum allowable upstream hydraulic grade line elevation at the inlet to the structure is 583.4 feet. The system is intended to maintain this condition at the flow rate of 1,867 cfs;

- The elevation at the outlet side of the structure is selected to remain above the average Rouge River elevation of 575.5 feet.





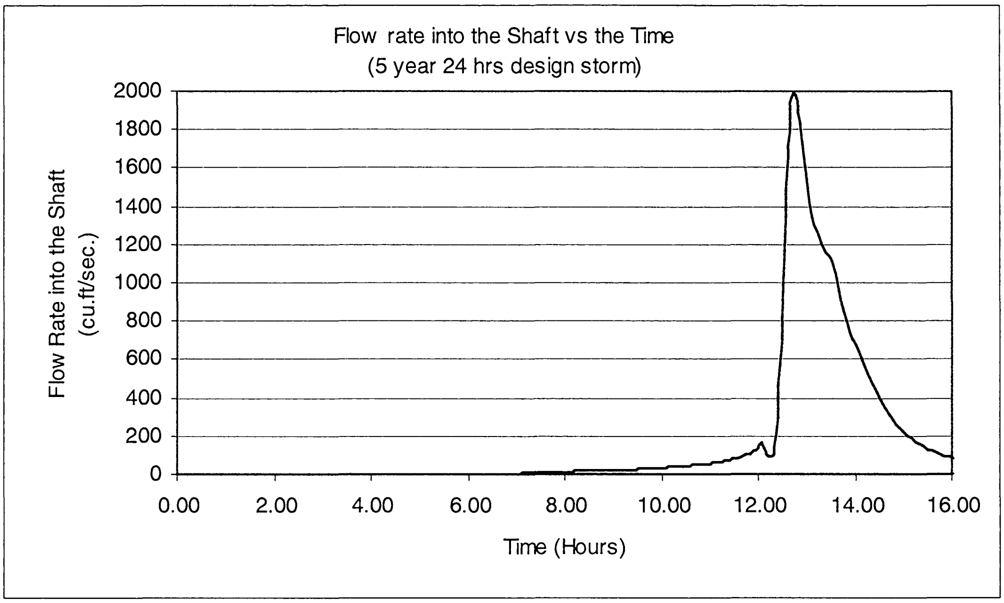


Figure 1: Inflow Hydrographs

There are various design features associated with the proposed shaft. To maintain clearance for flushing at the bottom of the shaft, the minimum elevation for the bottom of the baffle wall cannot be less than approximately 452 feet. The bottom of the shaft will be constructed with a sloping floor (see Appendix A) to streamline the flow and to deflect deposited solids to the center of the shaft. Solids will be flushed after an inflow event using high pressure nozzles. Horizontal screens are to be installed on the outlet side of the shaft to prevent excessive debris from discharging to the river. The screens were not included in the physical model, but the amount of head loss associated with them is known and can be accounted for by adding the screen losses to the loss in the remainder of the system. Finally, the flow at the exit side of the shaft is influenced by backwater effects from the Rouge River and this effect must be accounted for in the physical model.

MODEL DESCRIPTION

The purpose of the model was to:

- Measure head loss through the treatment shaft structure including the entrance and exit transitions and determine the variation in head loss with changes in baffle wall elevation.
- Verify chlorine contact time through the basin and determine the effect of the baffle wall elevation on fluid movement through the structure (breakthrough).

- Determine the best configuration of the baffle wall bottom elevation to simultaneously maintain small head losses and prevent premature breakthrough of the inflow.
- Determine whether or not the divider wall on the inlet side of the baffle wall has any influence on head loss or breakthrough.
- Determine maximum pressures on the shaft floor during the initial phases of filling when the influent is free falling to the bottom of the shaft.

Modeling Criteria

Physical models used to examine flow behavior in free surface flow 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 is acceleration due to gravity, and L is any system length. The relations between prototype and model parameters are related to the scale ratio $\mathbf{L_r}$ which is the geometric ratio between any length in the model and the corresponding one in the prototype ($\mathbf{L_r}$ = $\text{Length}_{\text{model}} / \text{Length}_{\text{prototype}}$). For a Froude scaled model, assuming the same fluid in model and prototype, the following relations must hold in which the ratio $\mathbf{Q_r}$, for example, represents the ratio of the discharge in the model to the corresponding prototype flow rate:

PARAMETER		RATIO
$\mathbf{L_r}$	Length	$\mathbf{L_r}$
$\mathbf{V_r}$	Velocity	$\mathbf{L_r}^{1/2}$
$\mathbf{Q_r}$	Discharge	$\mathbf{L_r}^{5/2}$
$\mathbf{T_r}$	Time	$\mathbf{L_r}^{1/2}$

The critical factors with respect to model testing facilities are the model size and discharge. If the scale ratio is too small, both viscous effects and surface tension may become too great in the model; in an extreme condition, flow that is turbulent in the prototype situation could become laminar in the model. A good rule of thumb is to maintain relevant model Reynolds numbers defined by VL/ν to be greater than about 100,000 in which ν is the kinematic viscosity of the test fluid. This consideration generally fixes the minimum model size required to avoid distortion of the model flow. An additional limitation is imposed by limitations on flow and/or physical space in the laboratory. It was determined that the maximum model size in order to fit the model into the vertical space available in the fluids laboratory at the University of Michigan was approximately 1:20. This size of model would be sufficient to maintain the Reynolds numbers in the inlet and outlet channels in excess of 100,000. The final model scale was selected at 1:19 to allow the use of an available tank for construction of the model's storage shaft.

Model Test Facilities

The model was constructed and tested in the Civil and Environmental Engineering Hydraulics Laboratory located in the G.G. Brown building, North Campus of The University of Michigan. The laboratory has a constant head re-circulating water supply capable of producing constant discharges at the rate required for a Froude scaled model at the selected scale size. The flow is metered by a Venturi meter installed in the supply line.

Model Construction

The physical model was constructed at a scale of 1:19 This scale was chosen due to height constraints within the hydraulics laboratory and to accommodate the use of a 5-foot diameter polyethylene chemical storage tank to reproduce the 95 foot diameter of the storage shaft. The storage tank rested on the laboratory floor with the remainder of the model located near the ceiling of the laboratory. The prototype shaft has a bottom elevation of 427 feet and a center floor diameter of 16 feet, where it slopes up at 1V:2H to the outer edges of the shaft. To simulate this, the floor of the model shaft was constructed using eight trapezoidal segments; see the imagein Appendix B.

The model included all geometric detail of the shaft as well as the inlet and outlet sections up to and including short segments of the connection sewers. Flow was introduced at the upstream end through an eight-inch diameter supply pipe that entered through the bottom of the modeled upstream end of the inlet conduit. A piece of expanded metal was placed in the inlet ahead of any measurement location to straighten the inflow. The outlet to the model was through an eight-inch pipe draining through the bottom of the exit channel. In order to control hydraulic grade line elevations in the model, an adjustable gate was installed just prior to the exit pipe. By raising or lowering the gate position, the hydraulic grade line elevation in the model could be adjusted to the desired target level.

Photographs of various components of the hydraulic model are included in Appendix B.

Instrumentation

Model discharges were measured with the master Venturi meter. Although the estimated precision of a venturi meter is generally considered to be on the order of 1-2%, the range of flows measured with the meter produced fairly small manometer deflections and the discharges are considered to be accurate to within about 5 percent.

Point gage measurements were made at various locations to determine head losses. Pressure taps were installed in the floor of the inlet and outlet sections (three at inlet locations and one at the outlet) in order to measure piezometric heads at those locations. Each pressure tap was connected to a stilling well that the point gage measured the elevation of the water surface within. One additional pressure gage was installed to measure the water surface elevation directly within the shaft on the downstream side of the baffle wall. The point gages registered elevation to the nearest 0.001-foot; the estimated accuracy of head loss measurements is less than this as discussed in the following section.

Retention times were measured in the model by initiating an injection of Rhodamine B fluorescent dye at the inflow and withdrawing samples at the outlet section and measuring the dye concentration with a fluorometer. The dye concentrations are measured relative to those in the injection. The fluorometer measures on a scale of 1 to 100 and the instrument was adjusted so that the inlet concentration generally registered a reading of greater than 40, preferably close to the maximum reading of 100.

TESTING PROCEDURES

Two major testing sequences were performed: measurements of head loss and measurements of dye breakthrough as a function of baffle wall elevation. The baffle wall was constructed in the model with a fixed section at the top and an adjustable lower portion that could slide to any desired bottom elevation. The procedures associated with each experiment are discussed in more detail below.

System head loss

There were five point gages measuring piezometric head within the model. The five locations that were monitored were:

- upstream inlet sewer, just prior to the inlet expansion,

- just into the inlet expansion,
- at the brink where the inlet expansion just entered the main shaft,
- within the shaft, downstream of the baffle wall, at a location that was relatively close to the baffle wall, and
- at the outlet brink along the left side (facing downstream) of the channel.

In order to measure head losses, all point gages were referenced to the outlet brink elevation of 575.5 feet. This was accomplished by installing a temporary dam at the model outlet and filling the model with an arbitrary volume of water. A temporary point gage measured the depth of water at the outlet brink and the point gage reading for each of the permanent gages was recorded. By subtracting the measured water depth at the outlet from each reading, a reference elevation for all point gages was determined. The difference between the actual head recorded at any flowing condition and this reference elevation defined the piezometric head relative to the outlet brink (575.5 feet elevation). The total energy head was computed by calculating a mean velocity head at that location determined with the measured flow rate and the total flow area. Head losses were then computed as the difference between the total energy head at the furthest upstream point gage and that at the measurement location.

After the completion of the experiments, the reference elevations were re-checked and they were found to be slightly different from those initially determined. It appears that the explanation for this is some slight settlement in the model during the course of the measurements. When the data were analyzed, it was found that the experimental results were more consistent when the final reference elevations were used; therefore all results are analyzed with these reference levels. An estimate of the precision of the piezometric head difference measurement is about 0.005 feet in the model; this translates to a prototype precision of 0.09 feet or slightly more than one inch. Another source of measurement error is in the setting of the flow rate. If it is assumed that the head loss is proportional to the discharge squared and that the flow rate is metered to within five percent, a typical head loss would be subject to another 0.5 inch (prototype) uncertainty.

Dye Breakthrough

The chlorine contact time in the shaft was investigated by monitoring dye concentrations following initiation of a maintained dye injection at the inlet (specifically, in the eight inch pipe just upstream of the entrance into the model). A concentrated Rhodamine B solution was injected at a constant rate through the sidewall of the pipe. Seven-milliliter fluid samples were collected at the outlet brink at 15 second intervals; it took approximately a second to collect each sample. Experiments were run for six minutes after initiation of dye injection; this was sufficiently long that the dye concentration at the outlet reached a constant value. Figure 2 indicates typical time histories of relative dye concentration during several repetitions of one flow condition. The fluid samples are small enough that turbulent fluctuations are not averaged out (the turbulent eddies are much larger in size than the sample spatial resolution) and the results should be considered by smoothing the results to eliminate the effects of the turbulence. Some of the preliminary experiments exhibited larger dye concentration variations than later experiments and these are felt to reflect the improvement in the experimental technique during the course of the experiments.

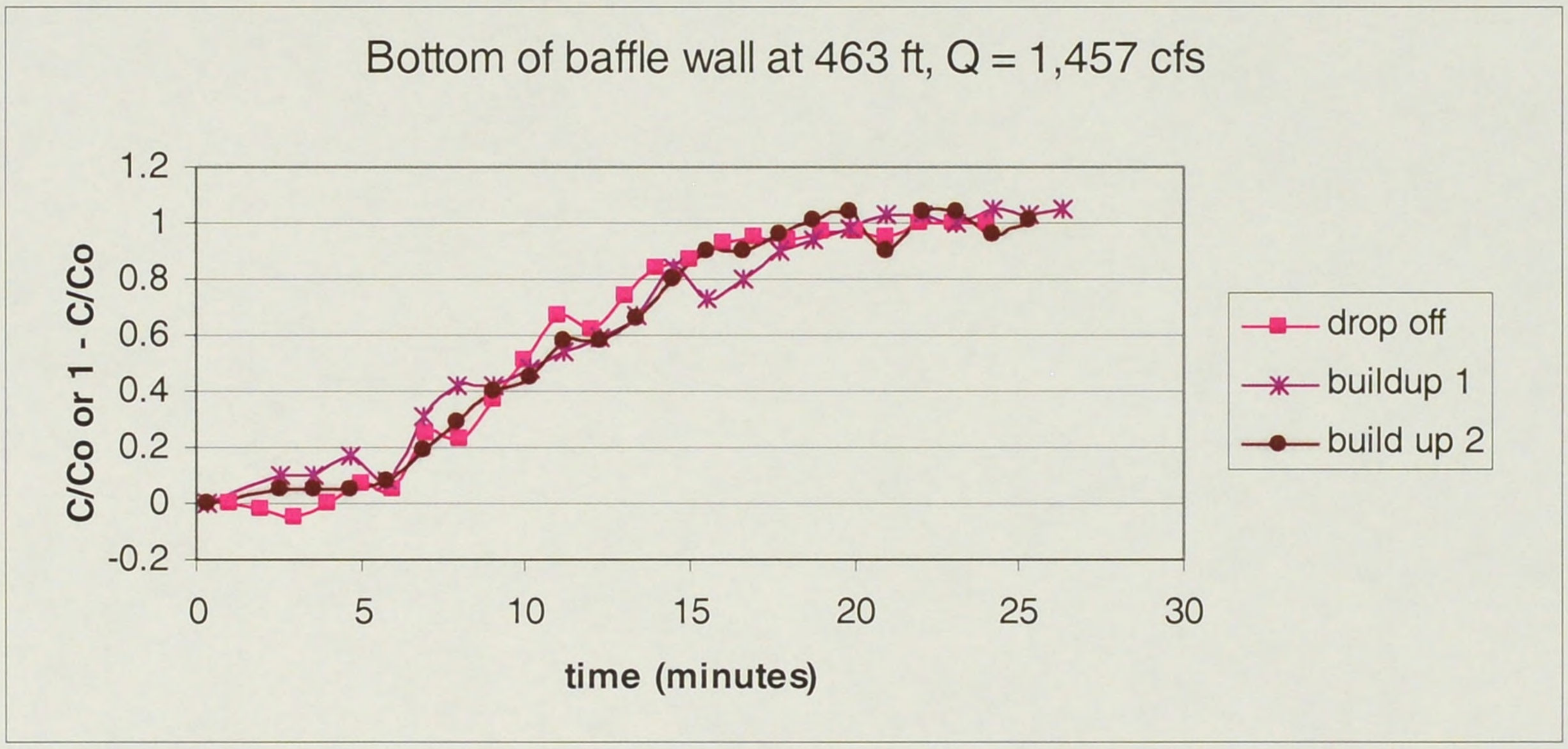


Figure 2: Typical Dye Concentration Time History

TEST RESULTS

System Head Loss

A series of tests were performed to measure the head loss through the structure as a function of the baffle wall elevation with the prototype design flow of 1,867 cfs. In each of these experiments, the downstream control gate was adjusted to provide a hydraulic grade line elevation close to the maximum allowable of 583.4 feet on the upstream side of the structure.

This was accomplished by marking the side wall in the inlet channel at the 583.4 feet elevation and adjusting the gate position until the upstream water surface elevation approximately matched that level. The minimum bottom elevation tested for the baffle wall was 452 feet; this elevation is the minimum allowed to provide access clearance for flushing the shaft bottom after an inflow event. Baffle wall elevations up to a maximum of about 502 feet were investigated. Figure 3 presents the measured results over the range of baffle wall elevations tested. This figure presents results for the head loss measured from the influent channel (prior to the inlet expansion) to the overflow brink; little additional head loss would be expected in the gradually contracted flow to the outflow channel. The trends in the results are as expected; at low baffle wall elevations, the restricted flow beneath the baffle wall results in increased head losses while at higher wall elevations, there is no significant effect on the head loss through the structure. As Figure 3 indicates, the transition elevation above which the variation in head loss is negligible is on the order of 457 feet. Variations in head loss for wall elevations above that level are within the measurement precision and there is no apparent trend to these variations. The average head loss for the data above that level is 0.415 feet. Once the baffle wall is lowered below the 457 feet level, increases in head loss are apparent. Although there is some scatter in the data, these are within the estimated measurement precision discussed previously.

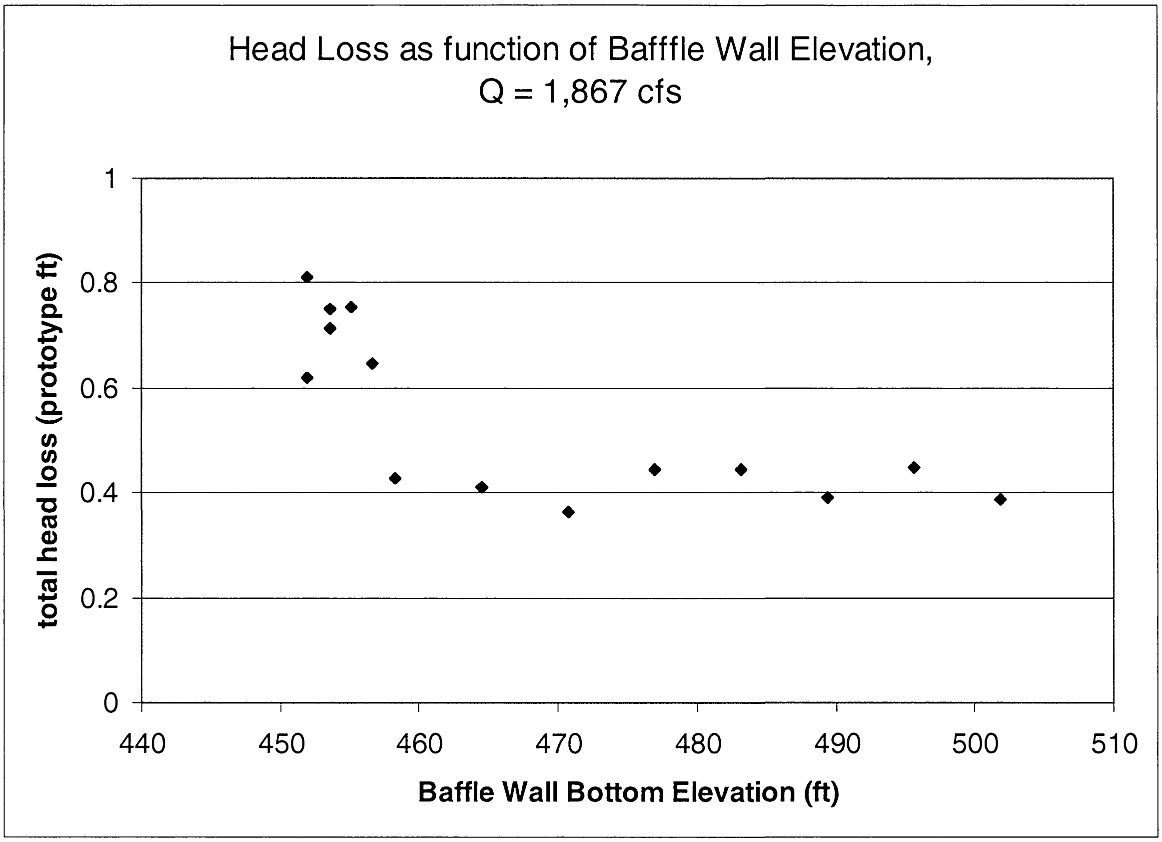


Figure 3: Measured Head Loss Results for Various Baffle Wall Elevations (1,867 cfs)

For most of the experiments presented in Figure 3, point gage measurements were made at all five locations so that the distribution of head loss within the structure could be estimated; these results are presented in Figure 4. With the very small head losses measured and given the level of measurement precision, it is difficult to make definitive statements other than that most of the head loss was experienced at the inlet to the structure. There are two mechanisms that could contribute to this head loss. The first is the flow separation at each of the three divider walls within the inlet expansion while the second is the sudden expansion loss due to the increase in the ceiling elevation passing from the inlet channel to the inlet expansion. An estimate of the sudden expansion loss associated with the increase in ceiling elevation yields a value of about 0.08 feet; it is less straightforward to estimate the losses associated with the leading edge of the divider walls. Nevertheless, the results in Figure 4 appear to be reasonable and indicate little loss within the shaft itself at large baffle wall elevations but increasing as the baffle wall flow opening is reduced.

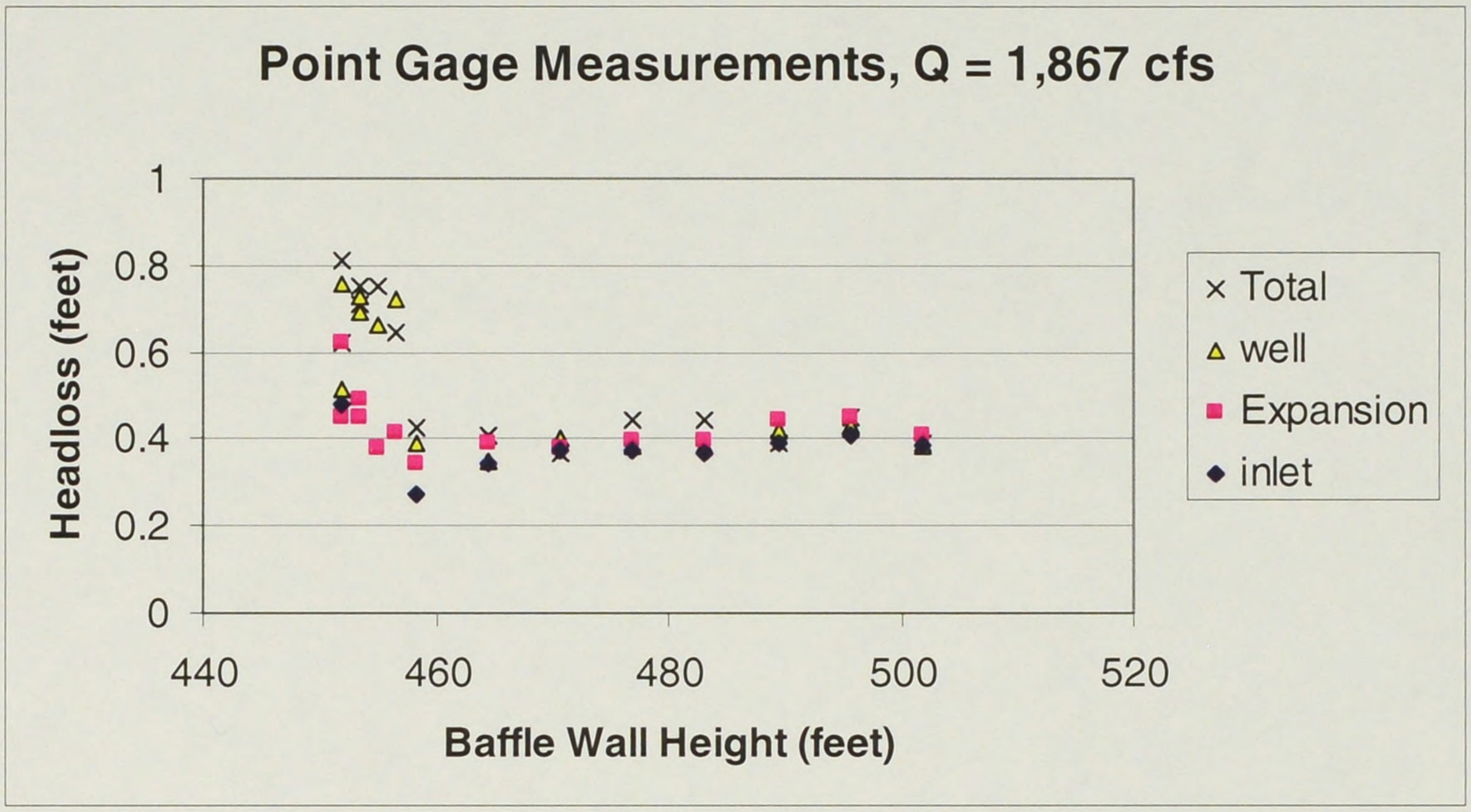


Figure 4: Point Gage Measurements for various Baffle Wall Elevations (1,867 cfs)

Once the experimental investigation was nearly completed, a preliminary design decision to place the baffle wall elevation at approximately 462.5 feet was made. Two additional sets of measurements were made at this point. For the design flow, 1,876 cfs, the head loss was measured both with the divider wall in place and with it removed. A set of three repetitions was made for each case. The average difference in head loss between the two cases was 0.043 feet prototype with the divider wall producing a slightly greater head loss. The difference is less than the estimated experimental measurement precision and it is concluded that the two conditions are equivalent to each other. A similar conclusion presented below with respect to the dye breakthrough characteristics indicates that the divider wall serves no useful purpose with respect to the flow through the shaft.

An additional request was made to measure the head loss through the structure at a prototype flow rate of 486 cfs. For this 10-year, 1-hour average flow rate, the treatment shaft volume provides the required detention time in meeting the Presumptive Criteria. Assuming that head loss scales with the square of the system discharge, the results from the higher flow rate of 1,867 cfs indicate that a prototype head loss on the order of 0.03 feet would be expected. Since this is less than the estimated measurement precision, any measurement result will be subject to considerable uncertainty. Three repetitions of the experiment resulted in measured head losses of -0.10, -0.08, and -0.08 feet. A negative head loss is physically impossible but the three measurements were fairly consistent with each other. It is concluded that either the method of computing the total energy head by an average velocity head creates slight discrepancies in the results or that the reference level of the upstream point gage was not absolutely correct. Given the difficulties in making accurate measurements at this low flow condition, no further efforts were made to resolve this minor issue.

Dye Breakthrough

These experiments were all performed at a prototype flow rate of 1,457 cfs and an upstream hydraulic grade line elevation of approximately 581 feet that was estimated to correspond to the limiting hydraulic grade line of 583.4 feet at the higher flow rate of 1,867 cfs. A total of 15 individual runs were made. With the exception of the first three sets of measurements that exhibited considerably more data scatter than later experiments but the same trends, it is difficult to ascertain a difference between the various experiments. These experiments included varying the baffle wall bottom elevation between 458 and 513 feet and the case with a baffle wall elevation of 463 feet and no divider wall on the inlet side of the baffle wall. Conclusions are somewhat complicated by the turbulent fluctuations in dye concentration but the experimental results are consistent.

As discussed previously, the experiments were initiated by opening a valve controlling the dye inflow into the eight-inch pipe just upstream from the modeled inlet chamber. The intention was to gain mixing from the re-direction of the inflow from the pipe to the actual model section. Since the injection was at a very low rate of a concentrated dye solution, the injection did not contribute significantly to the actual system flow. Observations made of dye inflow into the model showed that there was not a sharp dye front entering the model structure and complete initial mixing in the inflow was not be achieved. Therefore, the observed width of the breakthrough curve is partially due to the method of dye injection. In any case, dye samples were collected at the outlet at 15-second intervals for total injection duration of six minutes. This length of dye injection resulted in a fairly constant dye concentration for the last minute or two of the sampling interval. All results presented are in terms of a ratio C/C_0 in which C is the instantaneous dye concentration and C_0 is the final steady state dye concentration defined from the average concentration of the last six samples collected in any one experiment. Thus, in this format, relative dye concentrations should vary from 0 at the beginning of an experiment to 1 at the completion of the experiment. In one case, the experiment was performed by injecting dye at the beginning of the experiment for six minutes and then commencing sampling at the six-minute point when the dye injection was shut off. In that particular experiment, the relative concentration $1 - C/C_0$ should be consistent with all other experiments. This case is indicated in the results presented in Figure 2. Essentially three repetitions are presented, one performed with sampling after the dye injection was shut off (labeled "drop off"), and two performed in the conventional manner (labeled "build up"). These experiments were performed with the divider wall on the inlet side of the baffle wall removed and the bottom of the baffle wall set at 463 feet. The results from the three experiments are basically consistent with each other, discounting for small differences in individual runs due to turbulent fluctuations. If the mean breakthrough time is taken as the time at which $C/C_0 = 0.5$, the contact time implied by these experiments is estimated at about 11 minutes. Attempting to estimate the volume of water in the structure at this flow condition and dividing by the flow rate yields a retention time of approximately 11.5 minutes. The differences between these two results are at about the limit of the ability to measure the flow rate and the interpretation of the breakthrough curve. Therefore, there is no indication of any significant dead zones associated with the flow through the shaft.

The sampling location was just into the outlet transition and therefore, additional retention time is provided downstream of that location. For example, if the water volume within the outlet transition up to the downstream 20 ft wide condition is considered at the flow rate of 1457 cfs, an additional 5-10 seconds of detention time is provided prior to entry into the discharge conduit. Actual retention time prior to discharge to the Rouge River is available in the discharge conduit but dimensions of that conduit were not provided.

Results for the breakthrough curves for four different baffle wall elevations are presented in Figure 5. Although results can be presented for additional experiments, these four are representative of the range of baffle wall elevations tested and adding additional sets of data to the figure only make it more difficult to read. At the very highest baffle wall elevations (above 500 feet), there may be a slight tendency for the breakthrough curves to be altered although it is difficult to arrive at that conclusion due to the turbulent fluctuations in the concentration curves. This result is also consistent with the lack of effect on the head loss in this same range of baffle wall elevations indicating the flow is basically unaffected by the baffle wall location within the range tested.

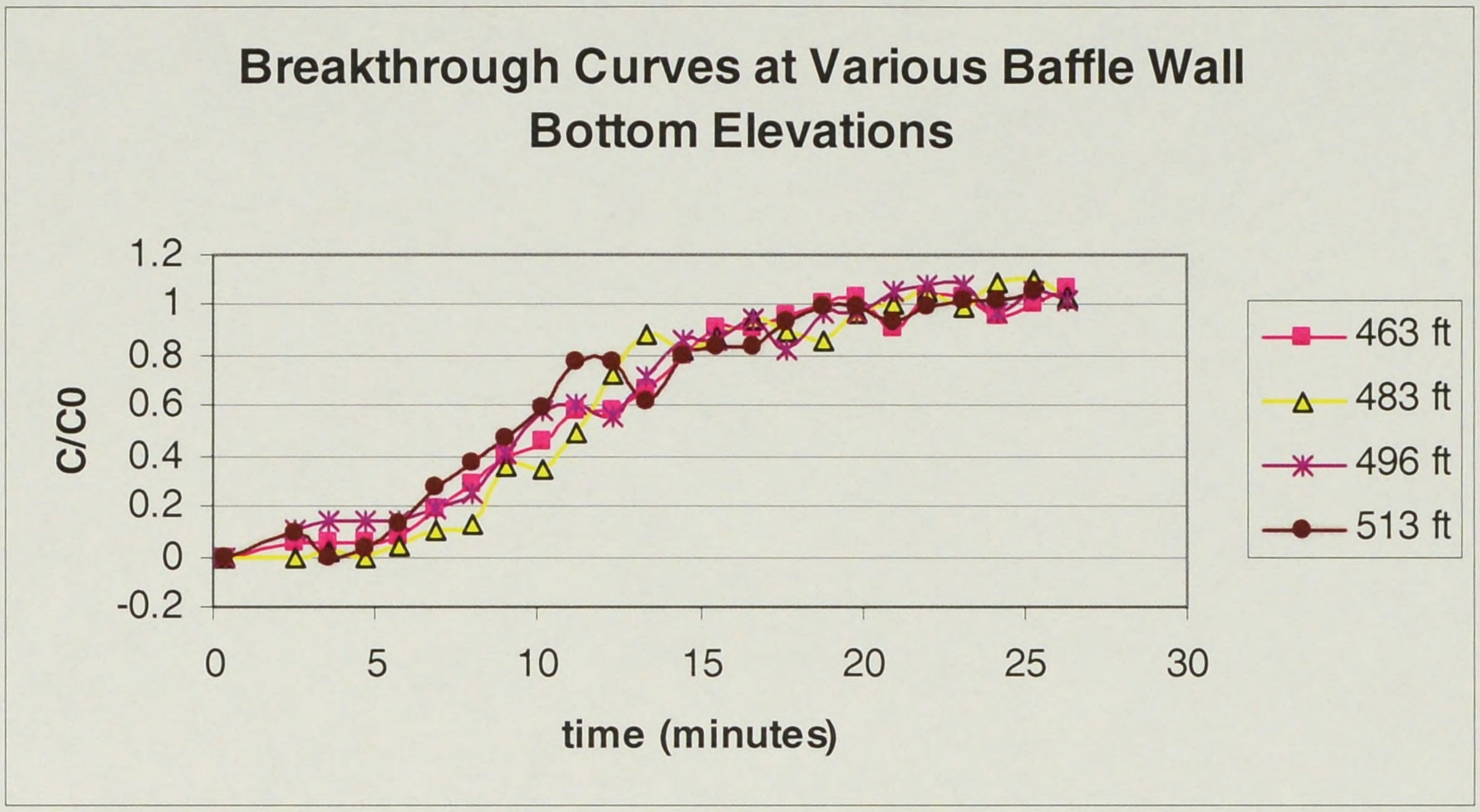


Figure 5: Dye Breakthrough for Various Baffle Wall Elevations (1,457 cfs)

Finally, the breakthrough curves measured for a baffle wall elevation of 463 feet with and without the divider wall on the inlet side of the baffle wall in place are presented in Figure 6. The two sets of measurements labeled "No divider" in Figure 6 are repetitions of the same configuration. Again, the lack of difference between these two cases indicates that the divider wall has no significant effect on the flow hydraulics through the structure. It is concluded that the divider wall is not an essential part of the structure design from the standpoint of the system hydraulics and it could be deleted unless required for structural function.

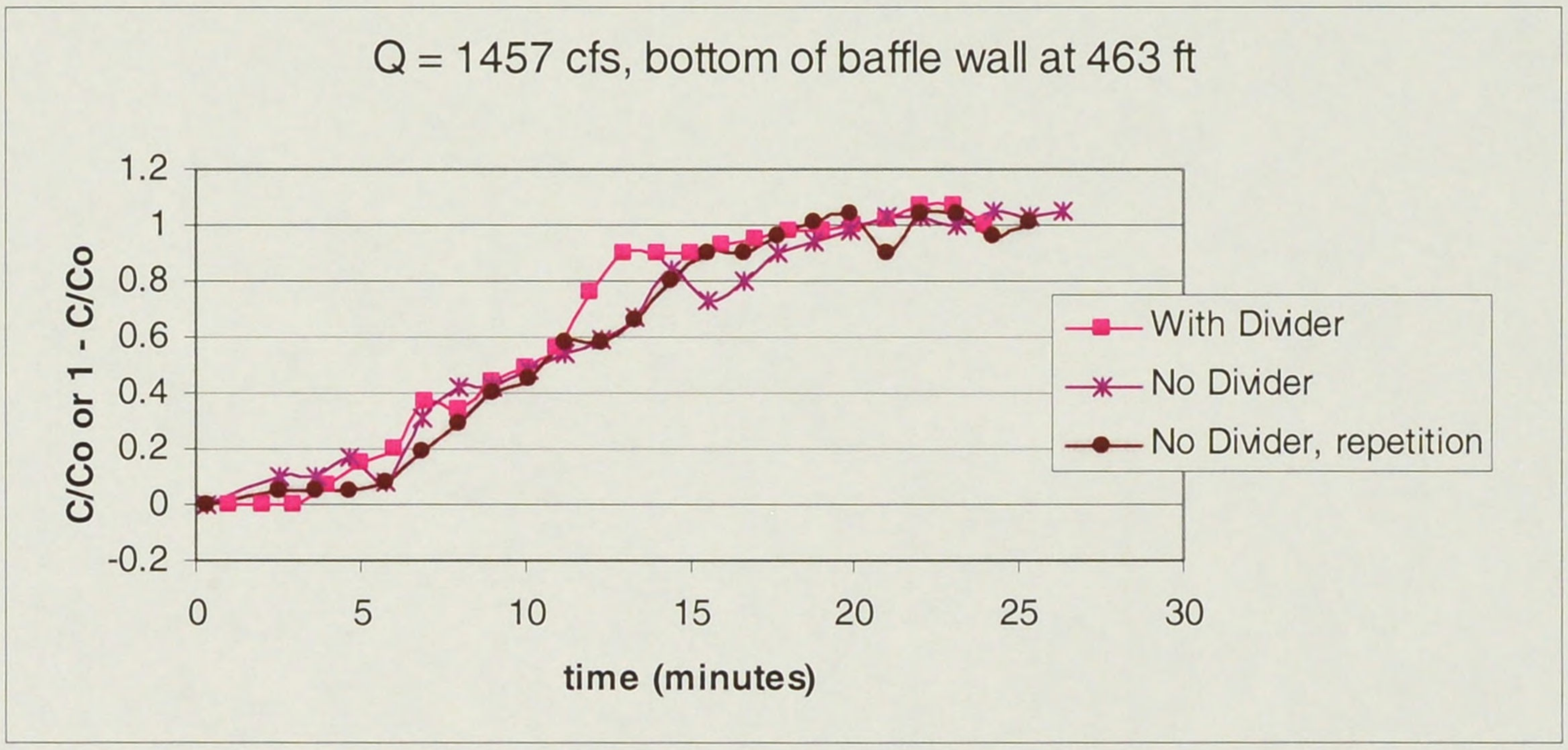


Figure 6: Effect of Divider Wall on Breakthrough Curve

Pressures on the shaft floor at initial inflow

An original objective of the hydraulic model study was to determine the maximum pressures on the floor of the shaft due to the inflow. These high pressures would be expected when the shaft is nearly empty and the inflow would basically fall freely the nearly 145 vertical feet from the inlet brink and impact on the sloping floor with the water level in the shaft below the impact point. It had been anticipated that these measurements would be difficult to make for a number of reasons. First of all, the width of the falling jet would be very narrow and it would be difficult to install a pressure transducer directly in the location of the plunging jet, especially since the jet would likely fluctuate with time. Secondly, it was expected that air will be entrained into the falling jet and the air entrainment will not scale properly in a Froude scaled model since surface tension will have an influence on the air entrainment. Finally, the issue associated with these high pressures relates to the ability of the concrete floor to resist erosion and it is not clear how to relate measured pressures to concrete erosion that will be partially controlled by the presence of solids present in the flow.

Previous studies on pressures under impinging jets have been primarily performed for plunge pools at the base of dams where a plunging jet falls into a stilling basin with a horizontal floor (e.g. Ervine, et. al., 1997 and Puertas and Dolz, 2005). Most of these studies have been performed with a significant cushion of water between the floor and the plunging jet. A key finding is that an increasing thickness of the water cushion serves to reduce the magnitude of pressures on the floor as would be reasonably anticipated. In the case of very limited water depth, the water flowing away from the impact point serves to cushion the flow to some extent. Another result is that if the water falls freely as a solid column, the impact velocity can be approximated by a free fall velocity of $(2gH)^{1/2}$ in which H is the drop height of the water. For small jets, air will be entrained into the falling jet and this will decrease the impact velocity and therefore the mean pressures.

Given the above considerations, it is reasonable to expect that the maximum pressures at the bottom of the shaft will be expected while the water level is below the impact point. Since this will occur only very early on in the filling process, the inflow rate will necessarily be relatively low. One question is whether the falling jet of water will even leave the side wall of the shaft in which case the plunging flow will not actually occur as a free jet. Figure 1 presents the computed inflow hydrographs for several difference design events. Figure 7 shows the results of converting the hydrographs to a comparison of flow versus water elevation within the shaft. Considering the worst case situation where the plunging jet strikes close to the shaft wall (19 feet above the shaft bottom), a maximum flow rate of less than 150 cfs can occur prior to the formation of a water cushion at the bottom of the shaft. The inflow brink is designed with a radius as indicated in Figure 8. When a brink radius representative of what could be expected in the prototype was constructed and a low flow of 100 cfs was passed through the model, the entire flow remained attached to the wall and flowed down the shaft wall. When the flow was increased to a prototype rate of 500 cfs, the flow remained attached to the wall over most of the brink length but did leave the wall near the center of the brink. It was unclear that the results from the reduced scale model could be readily extrapolated to the prototype situation.

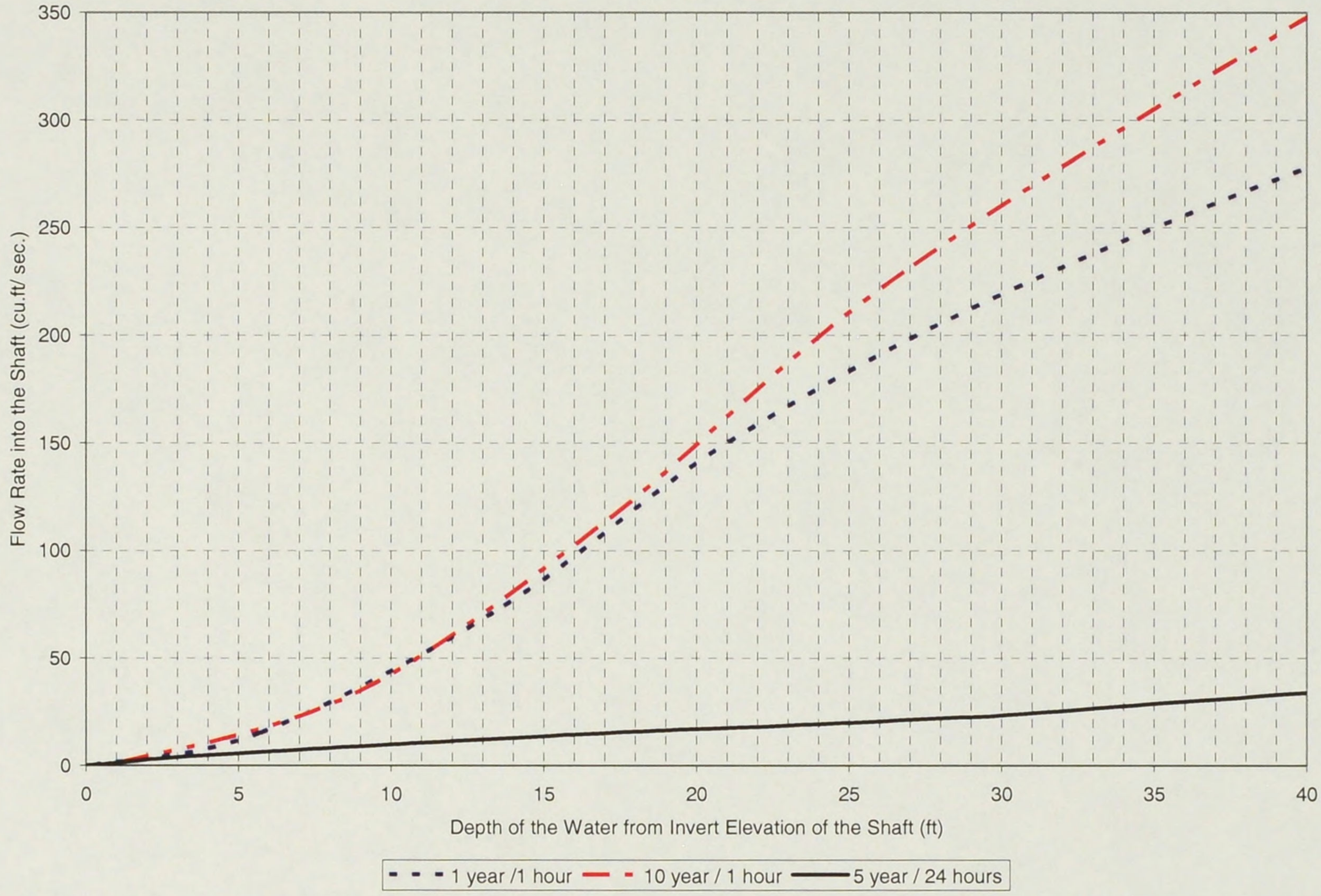
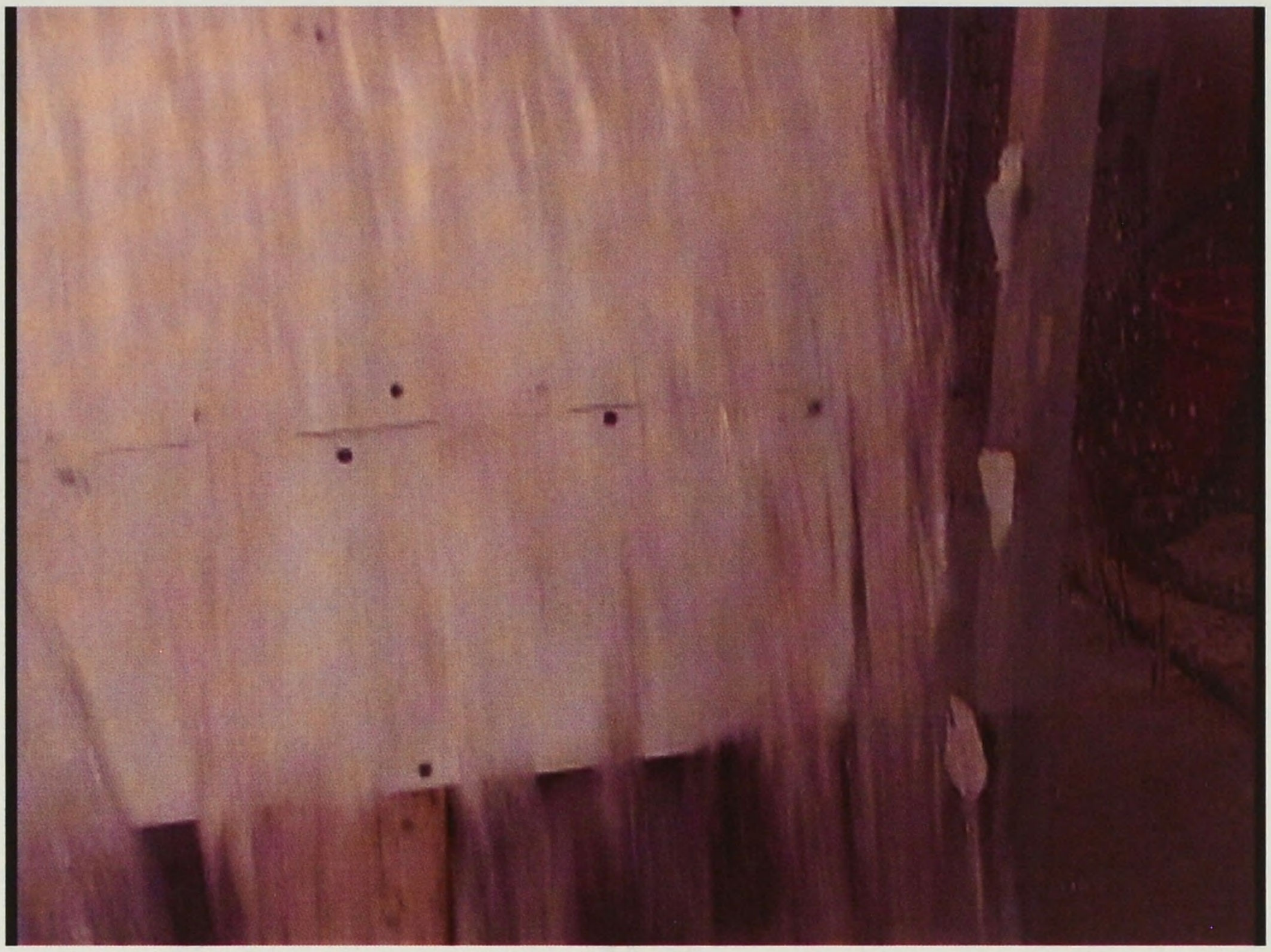


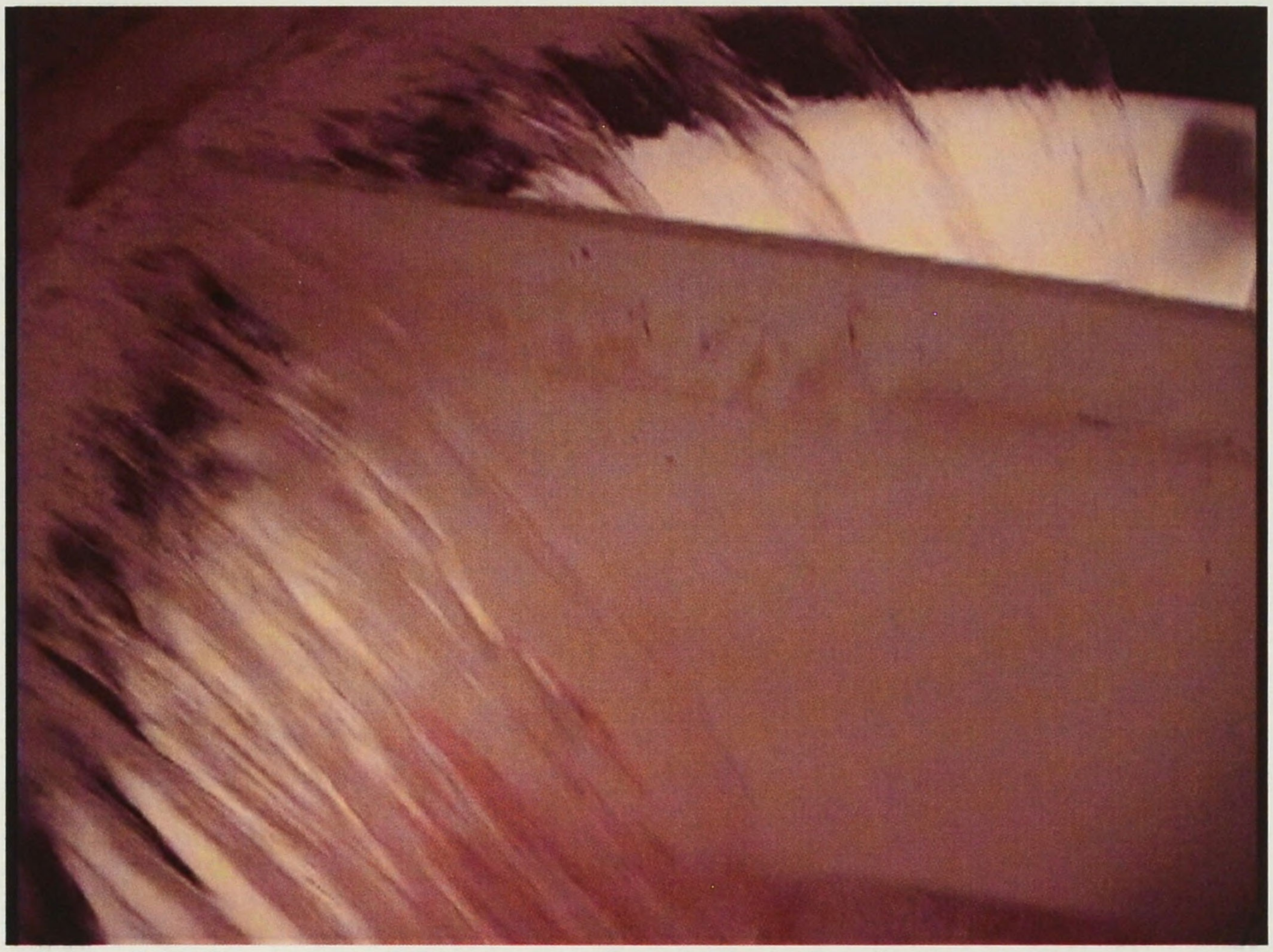
Figure 7: Flow Discharge versus Water Depth in Shaft for Various Hydrologic Events

In order to address this issue further, a 1:2 scale sectional model of the inlet brink was constructed in a laboratory flume 2 feet wide; Figure 8 indicates this model. The radius of the brink was taken at 3 feet (prototype) as designed. It was felt that at a 1:2 scale, any scale effects due to surface tension, viscosity or other factors would be minimized. The model was run over a range of prototype flow rates to observe the brink behavior. What was observed is that the flow tended to become non-uniform across the width and tended to remain attached to the wall in the low flow areas and detach from the wall at the higher flow regions. The effects of the flume sidewalls probably had a significant effect on this flow behavior. Small instabilities would propagate from the sidewalls and cause the flow over the brink to deviate from a two-dimensional state. It is unclear to what extent this same effect would be experienced in the prototype but it is likely that some basic instability phenomenon contributes to this breakup of the flow as the behavior was noted in both the sectional model and the smaller scale model as indicated in the images in Figure 9. However the effect was more pronounced in the sectional model. What was observed at a flow rate corresponding to 250 cfs prototype is that some of the flow separated from the wall while the remainder remained attached. At a lower flow rate of 100 cfs, there was more of the width that remained attached to the wall but some regions still separated from the wall. It is concluded that the flow over the inlet brink will not separate cleanly from the shaft wall at the low flow rates that would correspond to early fill times (prior to the formation of a water cushion even at the extreme 10-year, 1-hour design event). In the 1:19scale model, this behavior was not observed at a flow of 100 cfs. It is concluded that the small scale model cannot adequately represent the flow conditions in the plunging jet and it is not possible to accurately estimate the impact pressures in that model.

Figure 8: Two-Dimensional Flume Model to Study Overfall Condition.



a,) Sectional (flume) model (250 cfs)



b.) Small scale model (greater than 500 cfs flow)

Figure 9: Instabilities forming at inlet overfall

RECOMMENDATIONS AND CONCLUSIONS

The baffle wall height elevation should be set at an elevation that does not have an effect on the hydraulic grade line upstream and keep the breakthrough time above the allowable limit of 10 minutes. From the headloss experiments, this elevation is between 457 and 503 feet. The breakthrough experiment results indicated any elevation between 458 and 513 feet had no significant effect on breakthrough time. In the most recent drawings provided, the baffle wall elevation is set at 459.5 feet. This falls within the acceptable range of elevations for the baffle wall.

The divider wall at the inlet does not have any effect on the hydraulic function of the structure. It can be included for structural reasons, but has no affect on the hydraulics.

The maximum pressures on the floor of the storage shaft cannot be adequately determined by the 1:19 scale model and no attempt was made to measure the bottom impact pressures in the model. However, at all but the most extreme inflow conditions, it appears that the influent will not fall as a free jet at the initial inflows that will be expected at the commencement of the inflow hydrograph. By the time the inflow increases to a rate that would result in a free jet, a water cushion will be present at the bottom of the shaft, significantly reducing the magnitude of the impact pressures.

REFERENCES

Ervine, D.A., H.T. Falvey, and W. Withers. *Presure Fluctuations on Plunge Pool Floors.* Journal of Hydraulic Research, Volume 35, No. 2. 1997, pp. 257-284.

Puertas, J. and J. Dolz. *Plunge Pool Pressures Due to a Falling Rectangular Jet.* Journal of Hydraulic Engineering. Vol. 131 (5), May 2005, pp. 404-407.

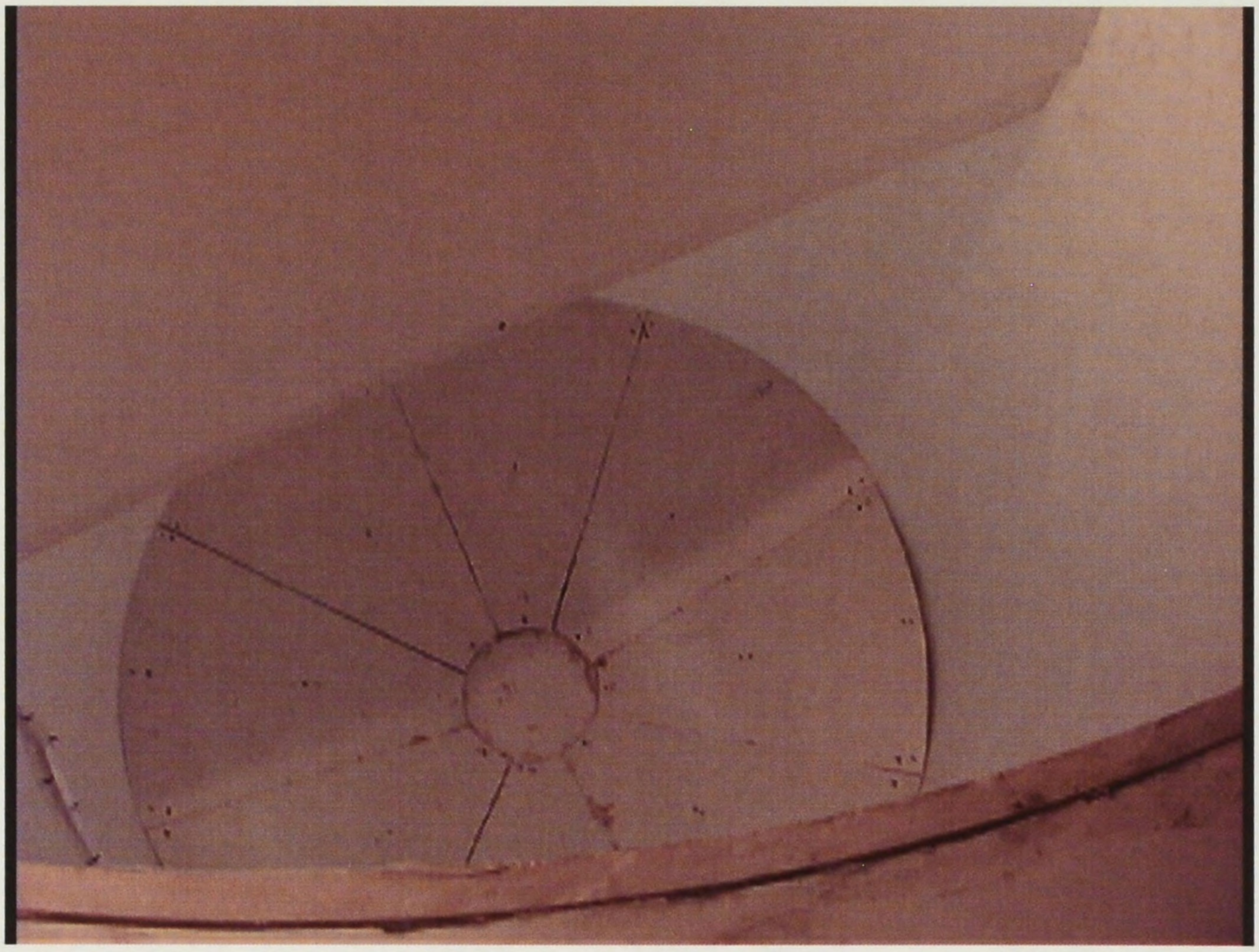
APPENDIX A: GENERAL SYSTEM DETAIL/LAYOUT

Page Missing in Original Volume

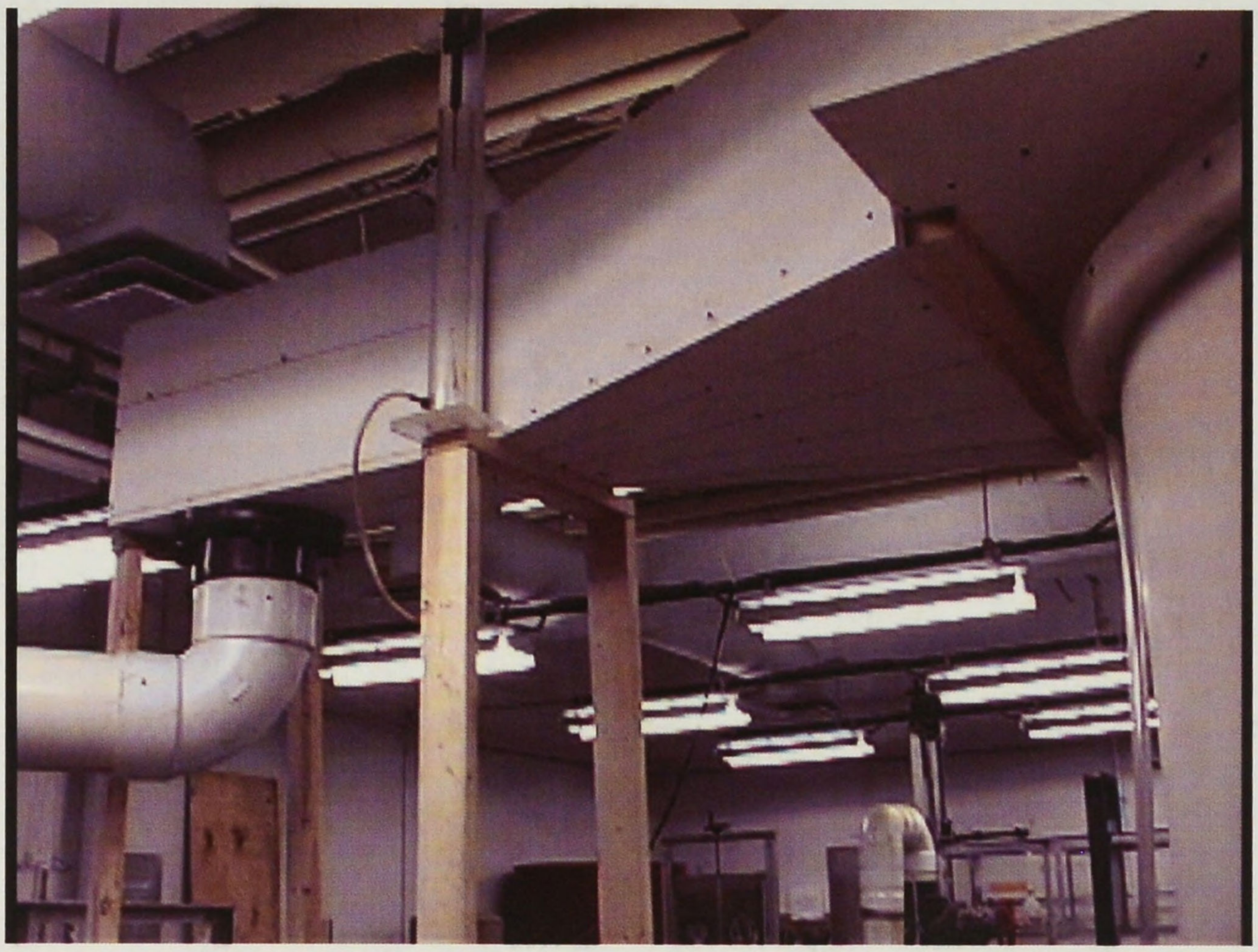
Page Missing in Original Volume

APPENDIX B: MODEL PHOTOS

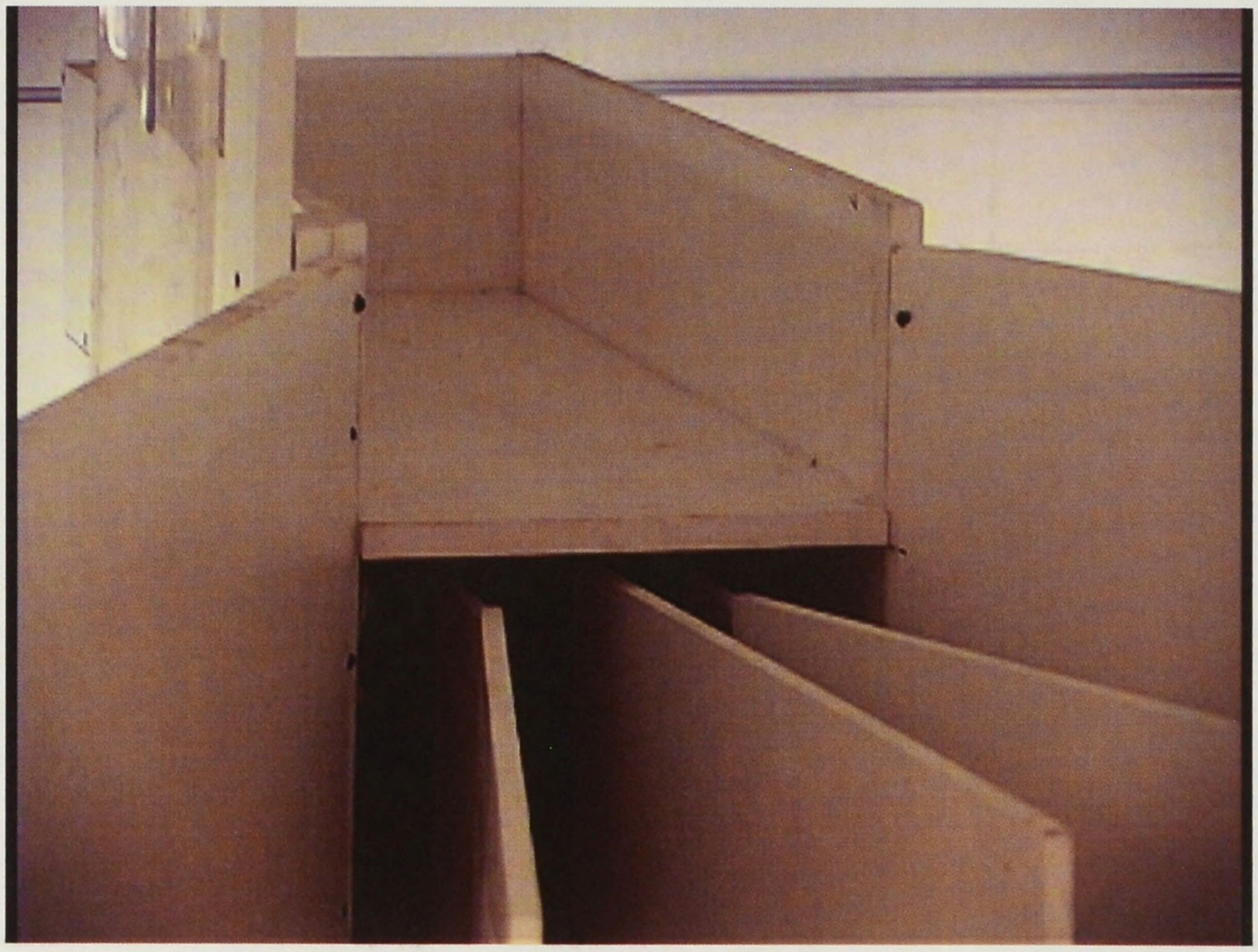
Overview of physical model



Top view of model floor and baffle wall



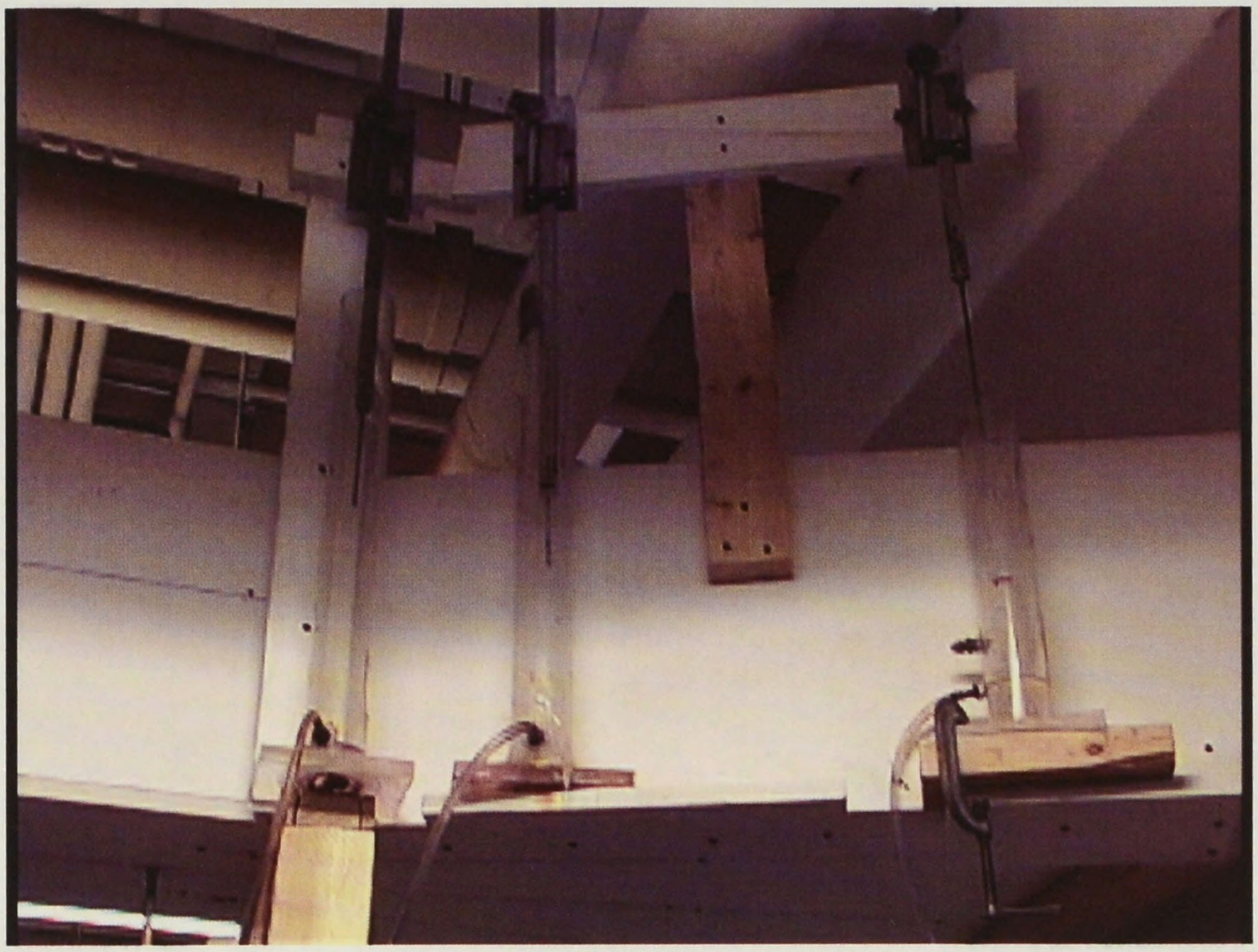
Inlet configuration



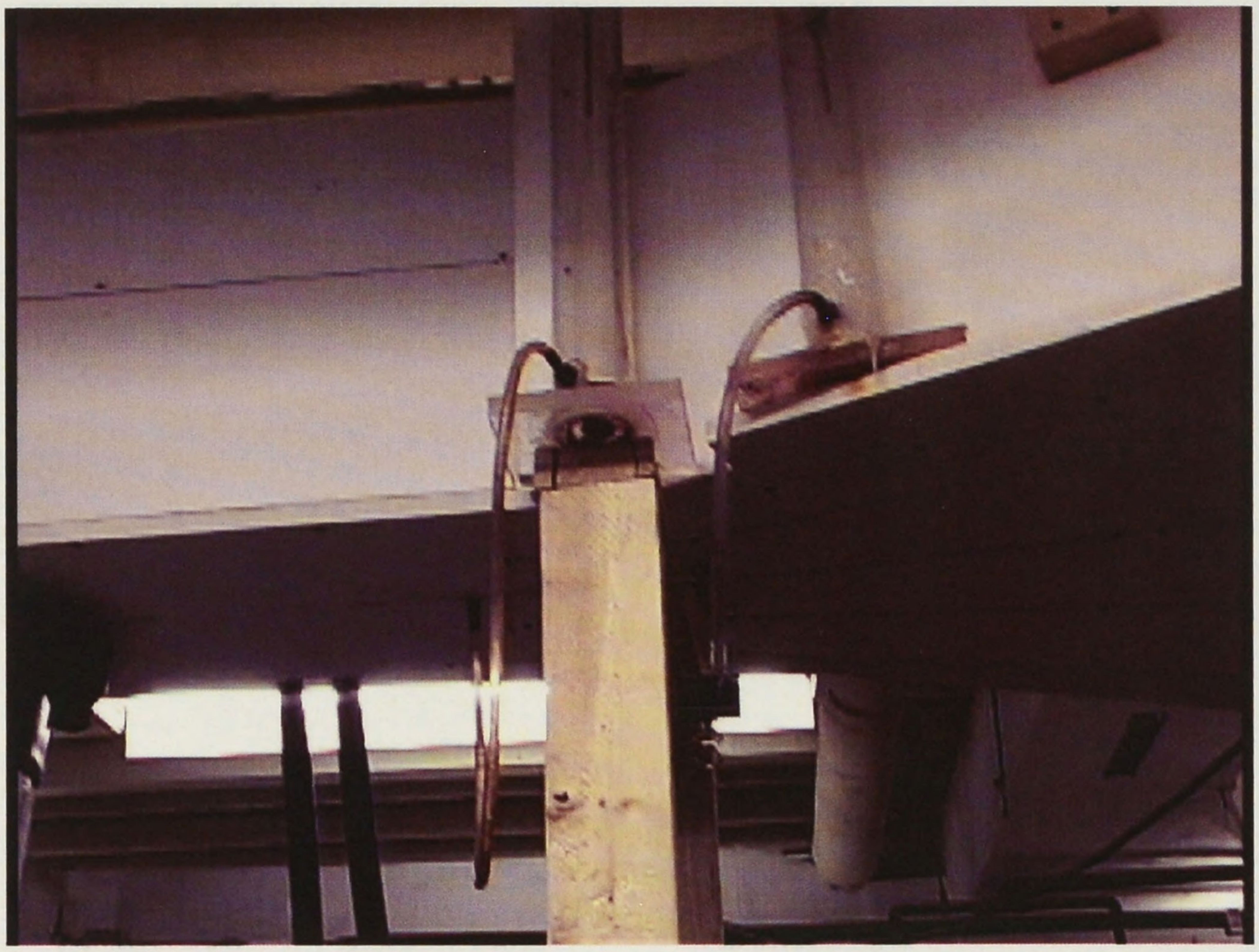
Divider walls at the inlet

Beveled inflow overfall

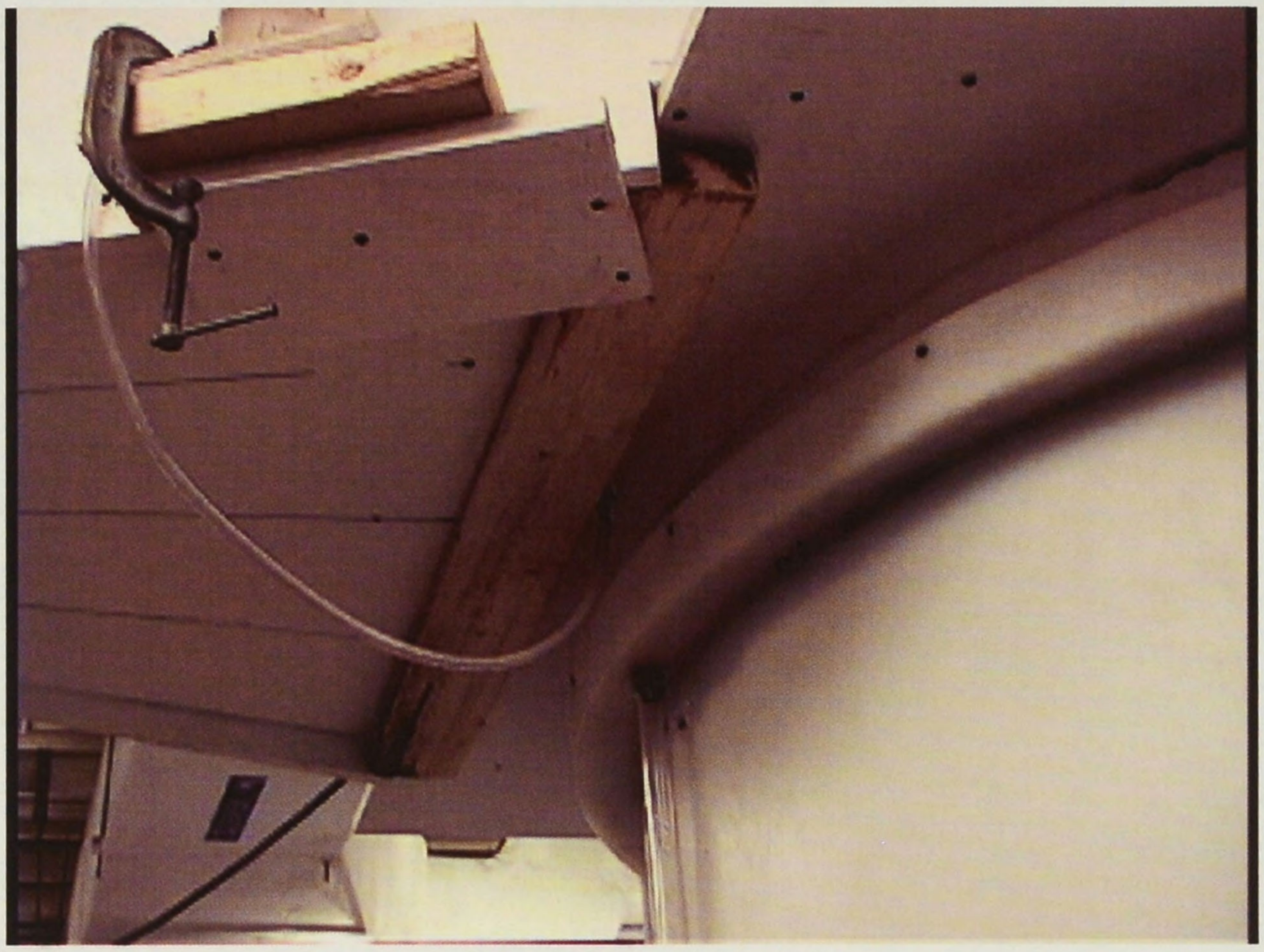
Downstream transition section



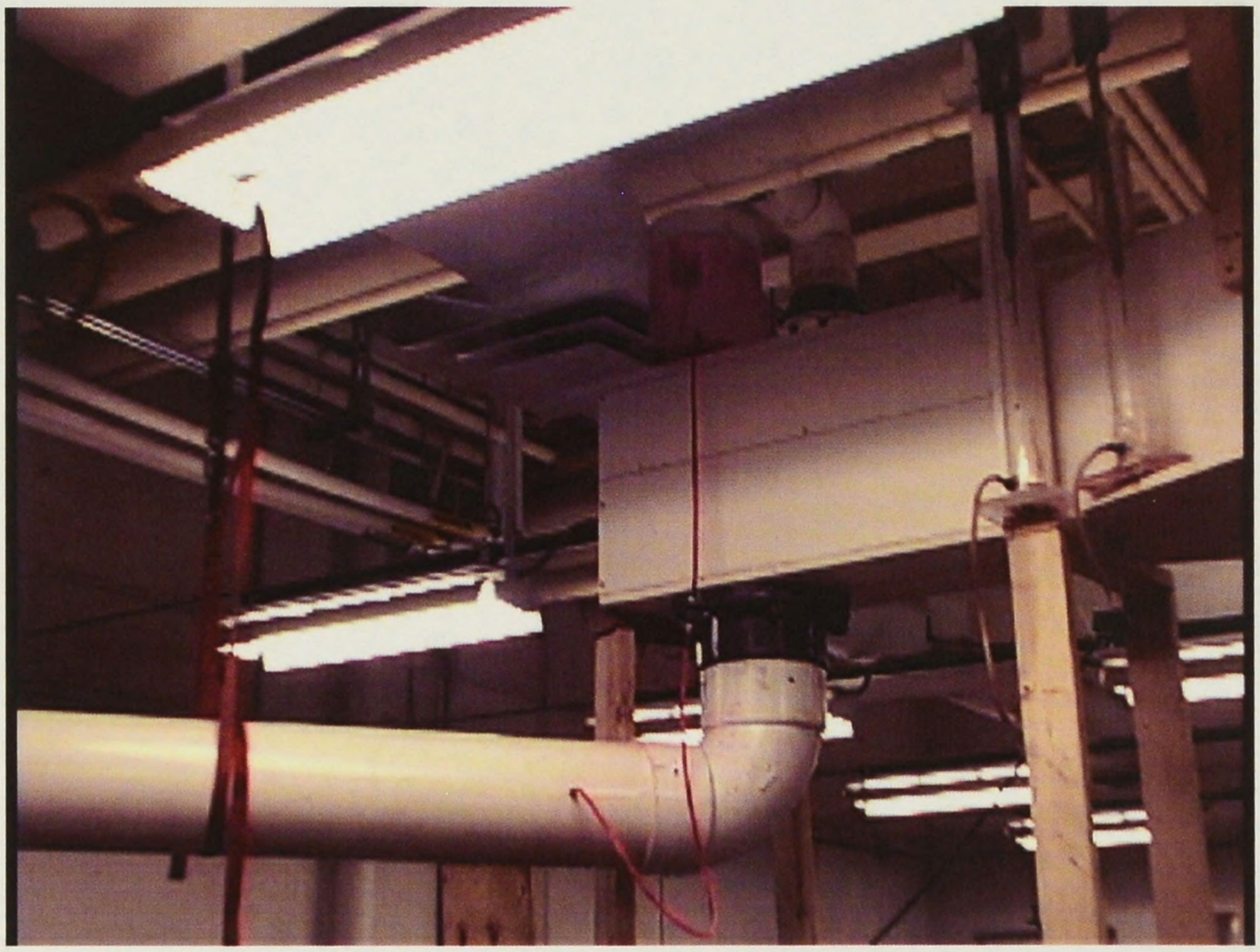
Three upstream stilling wells



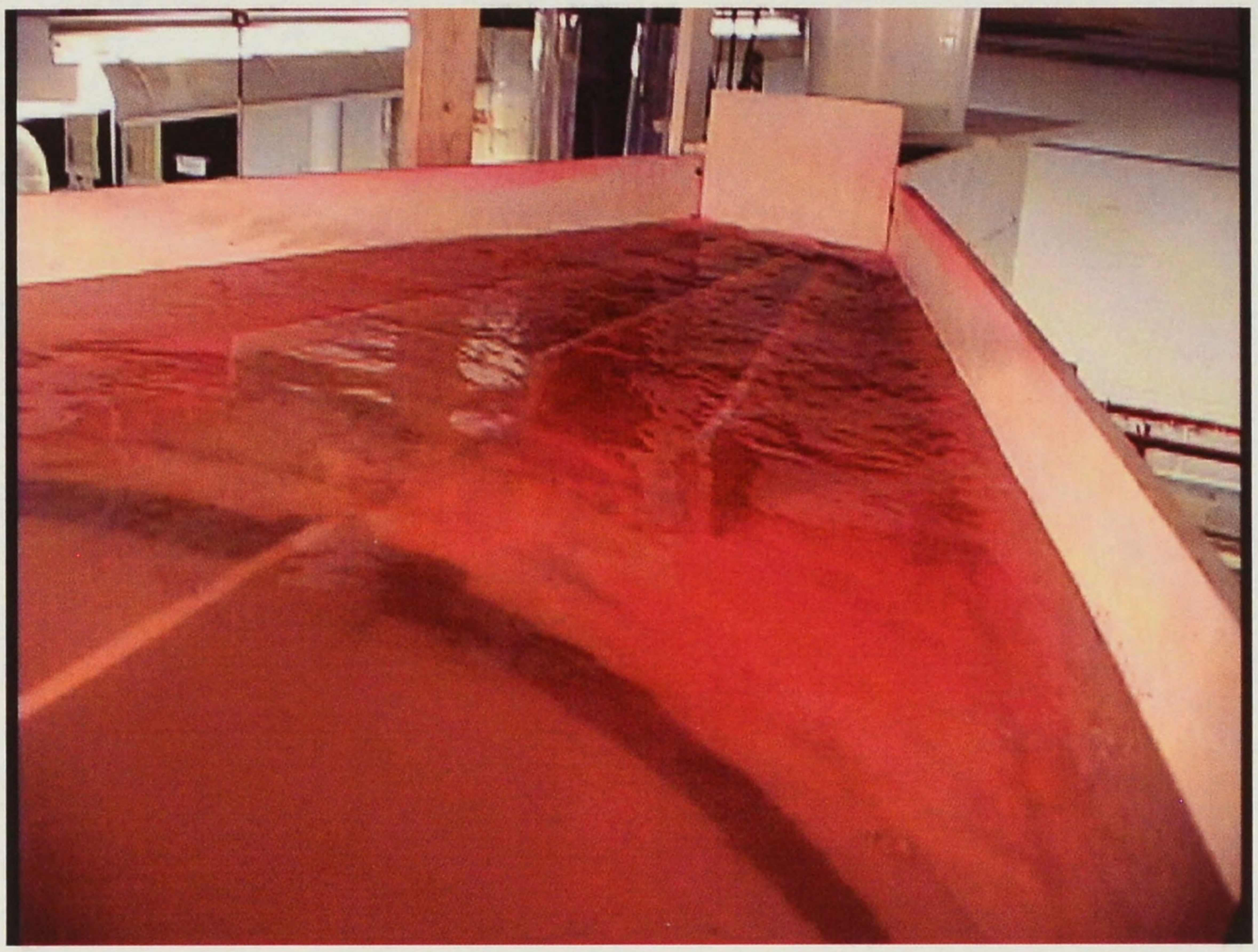
Stilling well location at the inlet expansion



Pressure tap location at the inlet brink



Dye injection configuration



Dye entering at the upstream end

Flow at the outlet

Inlet brink flow (100 cfs)

Inlet brink flow (500 cfs)

UNIVERSITY OF MICHIGAN
3 9015 10140 5341

AIIM SCANNER TEST CHART # 2

Spectra

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Times Roman

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Century Schoolbook Bold

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

News Gothic Bold Reversed

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Bodoni Italic

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Greek and Math Symbols

4 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡
6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡
8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡
10 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡

White

Black

Isolated Characters

e	m	1	2	3	a
4	5	6	7	o	•
8	9	0	h	I	B

0123456 65432

A4 Page 6543210

MESH HALFTONE WEDGES

65

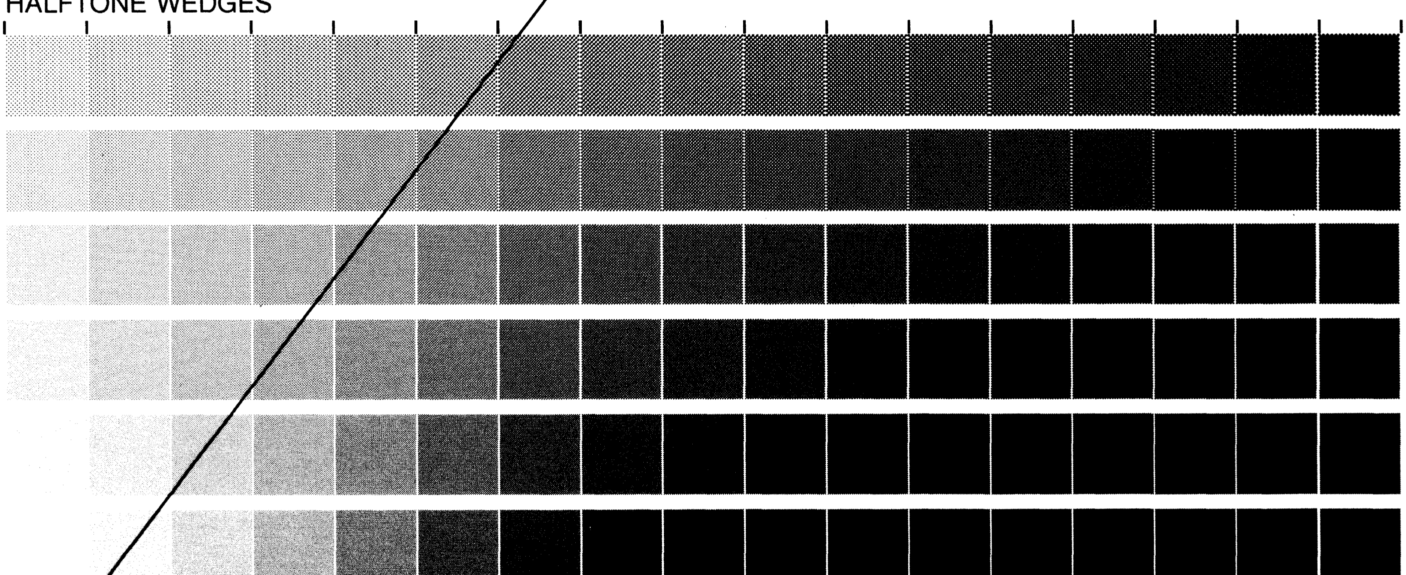
85

100

110

133

150



RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71

PRODUCED BY GRAPHIC ARTS RESEARCH CENTER

ROCHESTER INSTITUTE OF TECHNOLOGY, ONE LOMB

MEMORIAL DRIVE, ROCHESTER, NEW YORK 14623