FINAL PROJECT REPORT
HYDRAULIC MODEL STUDY
George W. Kuhn Drain
Intermediate Weir Structure Model Study

Report UMCEE 01-06

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INTRODUCTION

Extensive modifications are planned for the existing Retention Treatment Facility (RTF) for the Twelve Towns and George W. Kuhn Drainage Districts of the Southeastern Oakland County Sewage Disposal System. These changes were proposed to reduce the number and volume of combined sewer overflows to the Red Run Drain that ultimately discharges to the Clinton River in Oakland County, Michigan. An intermediate weir structure is being designed to provide additional in-system storage. A physical model of this structure was tested to measure the head changes through the intermediate weir structure under a variety of flow conditions.

A major issue in the RTF design is the change in the hydraulic grade line (HGL) through a series of existing weirs and baffle walls. The weirs are to retain storm water flows within the RTF and provide in-system storage. The complex configuration of these weirs did not allow for the determination of HGL changes through standard hydraulic formulas. The HGL change is a function of the tailwater elevation induced by a downstream control structure as well as the total discharge.

The testing sequence consisted of the following components:

- HGL results for the proposed design.
- Examination of the flow distribution within the weir structure.
- HGL results for different alternative configurations with the purpose of reducing the total head change through the structure.

Qualitative experiments were performed to examine sedimentation patterns within the model.

GENERAL SYSTEM DETAIL

A 293 ft long inlet weir was constructed in about 1965 to provide approximately 32 million gallons of in-system storage. This structure is located just north of 12 Mile Road and west of Interstate 75. The weir is constructed with a baffle wall in front of the weir crest to restrict the downstream movement of floating trash. Dry weather flows are diverted into a 60-inch sanitary sewer just upstream of a pair of Broome gates. During storm conditions, flow begins to pass downstream once the water backs up to the weir crest, which is at an elevation of 615.5 ft (Detroit datum). Once the upstream water level increases to 617.5 ft, the Broome gates are opened, allowing the passage of higher flow rates with smaller HGL changes through the structure. Proposed modifications to the existing inlet weir system include removal of the Broome gates, lowering of the upstream weir crest by two feet to 613.5 and the addition of a weir, flap gate, and baffle system downstream of the existing Broome gates. Under this proposed design, flow will begin to pass through the inlet weir structure whenever the upstream water level backs up to the 613.5 ft elevation of the weir crest.

Flow passing through the inlet weir structure currently enters a 12,000 ft long tunnel with an approximate cross-section of 65 ft wide by 22 ft high. This tunnel serves to provide additional in-system storage and water levels are controlled by a downstream outlet weir structure with a crest elevation of 611.9 ft and a crest length of 950 ft. Flow over this weir discharges to the Red Run Drain. Additional system storage is being designed at an intermediate weir facility downstream of the inlet weir that involves a considerable expansion to the existing tunnel width and the installation of a 2000 ft long weir system with a crest elevation of 614.5 ft. Figure 1 provides a schematic of the relationship between the inlet and intermediate weir facilities, while Figure 2 is a floor plan of the intermediate weir facility. The structure comprises a 1200 ft long expansion of the existing tunnel to a total width of a little greater than 300 ft by adding additional volume on either side of the tunnel. A screen facility is located near the middle of the structure and a pair of 1000 ft long U-shaped weirs are provided in the downstream portion of the facility in order to develop the additional storage. Baffle walls will be placed upstream of the weir crest. This structure adds approximately 31 million gallons of additional in-system storage.

A key constraint on the system design was the allowable change in hydraulic grade line through the entire system. This head change would include all losses at the weirs/baffle walls/screens (all three facilities) as well as the internal friction within the RTF. The physical hydraulic model was used to evaluate the hydraulic grade line changes associated with the weir/baffle wall units in the intermediate weir structure. Head changes through the inlet weir structure were also investigated by means of a physical model study and results are reported elsewhere (*Twelve Towns Retention Basin Model Study*, S.J. Wright and D.J. Lautenbach, Report UMCEE 10-05, May 2001). The entire length of the RTF was not modeled in the laboratory due to the inability to reproduce it at a reasonable model scale. Instead, friction losses downstream were estimated using the modeling software HEC-RAS. Others estimated these losses.

MODEL DESCRIPTION – MODELING CRITERIA

Physical models to quantify 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 the acceleration due to gravity, and L is any system length. The relations between prototype and model parameters are related to the scale ratio $\mathbf{L_r}$ that 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**
Length	$\mathbf{L_r}$	$\mathbf{L_r}$
Velocity	$\mathbf{V_r}$	$\mathbf{L_r}^{1/2}$
Discharge	$\mathbf{Q_r}$	$\mathbf{L_r}^{5/2}$
Time	$\mathbf{T_r}$	$\mathbf{L_r}^{1/2}$

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. This consideration generally fixes the minimum model size required to avoid distortion of the model flow. It is generally accepted that a minimum model Reynolds Number is required in order to avoid these problems. The value depends on the specifics of the flow being studied and the exact definition of the Reynolds Number, but generally if the value exceeds a minimum in the range of 10,000 to 100,000 then the model results will be Reynolds Number independent. The exact size of the models was consistent with the availability of laboratory space and the desire to construct the model at as large a scale as possible. The intermediate weir structure was built at 1:35 scale in order to fit within the space available in the laboratory. At the high discharges on the order of 6000 cfs, model Reynolds Numbers are within the range given above. For the lower end of the flow range studied, viscous effects may be present; any effect will be to slightly increase model head losses relative to the corresponding prototype conditions under low discharge conditions. However, critical conditions for system head loss are at the highest flow rates and the model will provide accurate results for those situations.

MODEL DESCRIPTION – MODEL CONSTRUCTION

The model study was conducted in the Civil and Environmental Engineering Hydraulics Laboratory located in the G. G. Brown Building at the North Campus of The University of Michigan. The physical model was constructed at a scale ratio of 1:35. The physical model was constructed of plywood, Plexiglas and PVC pipe. Plexiglas was used where visual access to the flow was required, primarily on the ceiling of the structure. The model consisted of the entire intermediate weir structure as well as short sections of the inlet and outlet tunnels, particularly the bend in the tunnel right at the structure inlet. Flows were introduced to the model from the laboratory constant head water supply through a section of PVC pipe suspended above the model. An orifice meter installed in this pipe was used to meter the flow. The model consists of the basic control structure, including the weirs and upstream baffle walls. All internal columns, guide walls, truck access ramps, etc. were reproduced as well. Low flow channels in the basic flow section were reproduced in the model at the correct elevations and approximately the correct alignment; smooth bends were reproduced with corners for convenience in model construction. Dimensions for the existing structure were obtained from a series of design drawings provided by TetraTech MPS. Screens were not placed in the model in the screen facility at the center of the structure so additional losses attributed to the flow through the screens will need to be added to the model results in order to obtain total system head losses.

The structure is to be constructed with a grid of concrete beams on the ceiling. These were not included in the model construction; rather the model was constructed so that the ceiling elevation corresponded to the bottom of the beams. A previous model study of the inlet and outlet weir facilities by the Corps of Engineers Waterways Experiment Station, (*Pollution Control Facilities Project, Southeast Oakland County Sewage Disposal System, Oakland County Michigan*, Technical Report HL-82-4, U.S. Army Engineer Waterways Experiment Station, February, 1982) concluded that the additional losses due to the rough ceiling were minor. These extra losses would only occur when the structure is surcharged in any case, and this condition only occurs for the very highest tailwater elevations.

The current design allows for a bypass of the fine screens/overflow weir portion of the structure by allowing gates to be opened in front of four coarse screen bays located at the center of the screen facility. In the results presented, this condition is referred to as the "gates open" condition while the normal operating state is the "gates closed" condition. Gate closure or opening was simulated in the model by sliding down or raising a single Plexiglas sheet in front of the coarse screen bays. This approach simulates a larger gate opening than actually exists in the proposed design. Measurements indicated that the total losses through the screen facility are a small fraction of the total system head loss so the effect on the test results is considered to be negligible.

Images of the model are provided in Figures 3-7, which indicate various aspects of the intermediate weir structure model.

INSTRUMENTATION

The discharge to the model was measured using a pipe orifice meter constructed to ASME specifications. Making the amount of straight pipe upstream and downstream from the orifice as long as possible minimized the approach flow influence. Pressure differences were measured through the use of a water-air differential manometer. The meter was verified at high flow rates by checking it against the master venturi meter that is installed in the laboratory water supply system.

Hydraulic grade line elevations were measured at selected locations within the model. These were measured by installing pressure taps at the sides of the model that were connected to stilling wells made of transparent Plexiglas. Point gauges were installed in the stilling wells and could be read to a basic accuracy of 0.001 ft. A pressure tap and stilling well was installed on the left side of the entrance to the model (in the approach tunnel approximately 0.5 ft model dimension upstream from the structure) and one at the exit to the model (just into the downstream storage tunnel approximately 0.5 ft model distance downstream of the structure). Later in the study, two additional point gauges were added to measure HGLs on either side of the screen facility; these were installed on the right wall (looking downstream) of the structure in the area where the truck access allows passage through the screen facility. All point gauges have to be referenced to a common elevation in order to measure HGL differences. This is accomplished by damming the downstream end of the model, filling the model to an elevation where the water stands at the weir crest elevation, and recording the point gauge readings that correspond to the elevation of 614.5 feet. All subsequent elevations are referenced relative to these readings. This procedure was performed twice throughout the study, the second time when the additional two point gauges were added. There was a change of approximately 0.25 ft prototype in the upstream datum between the two checks, apparently due to settling of the model as it was filled with water. All data reported are in terms of the second set of reference elevations.

Measurement uncertainty in the model is due to three primary effects:

- Measurement of discharge
- Measurement of stilling well elevation
- Measurement of weir crest elevation

The latter is only important for measurements of weir head as opposed to HGL difference. Uncertainties in discharge measurements increase with decreasing discharge due to the smaller head differences across the orifice meter. At the design flows on the order of 6000-7000 cfs, the discharge is estimated to be accurate to within about 2 percent. At discharges on the order of 1000 cfs, this uncertainty increases to about ten percent. Multiple measurements with a single point gauge indicate repeatability on the order of 0.1 ft prototype. Since HGL difference is based on the difference between point gauge readings, a basic uncertainty in the HGL difference ought to be on the order of 0.2 ft. Later measurements with prototype flows of 1000-2000 cfs involved total system head losses approximately 0.1-0.2 ft so particular care was taken in the measurements to reduce measurement uncertainties. For the determination of weir heads, a point gauge reading is referenced to the point gauge reading taken when the model was filled to the weir crest. The fundamental issue is in the determination of when the water level is at the weir crest. In addition to slight variations in the crest elevation, surface tension controls when filling water overtops the crest; therefore the actual water elevation is most likely somewhat higher than the physical crest elevation. It is probable that this uncertainty could be as much as 0.2 ft prototype and consistent with an under-estimation of the true weir head.

TESTING PROCEDURES

In most measurements, the desired discharge was set in the model. Once the flow was set, adjusting the downstream gate setting controlled the tailwater elevation. It was not feasible to easily reproduce a particular tailwater elevation, so several different but arbitrary elevations in the range of interest (typically 613-620.5 ft) were generally set. After a tailwater elevation was set, it was necessary to wait several minutes for the water levels within the model to stabilize, at which time the necessary point gauge readings were taken and a new tailwater elevation set. HGL differences were determined by subtracting the downstream tailwater elevation from appropriate upstream elevations.

This study was done in four phases. In the first phase, measurements at several discharges were made through the model constructed according to the proposed design for a range of tailwater elevations. These were done for flow conditions in which the bypass gates were either open or closed.

In the second testing phase, the model was modified in an attempt to distribute the flow more uniformly upstream of the screen facility. This was done with the expectation that a more uniform flow distribution would result in decreased head losses. Diverters to push flow from the existing tunnel were installed in the model and the head losses at a few flow conditions were measured. This testing phase also involved the installation of additional point gauges to pinpoint where the measured head losses occurred within the model. By measuring head changes between successive sections, it could be determined where the bulk of the system head loss occurred.

In phase three, a final configuration was selected and full rating curves were developed at 1000 cfs increments over a range of tailwater elevations for each flow rate.

The fourth phase involved making geometric changes to the model in order to try to reduce the head loss through the structure.

TEST RESULTS

In this section, results are discussed according to the different phases of testing described above

Phase 1 Results

This first phase consisted testing the model at three different flow rates (2000, 6000, and 7000 cfs) for a variety of tailwater elevations varying between about 613-620.5 ft. These were performed for conditions in which the gate in front of the coarse bypasses screens were both open and closed. Only the results for 6000 and 7000 cfs are reported here in Figures 8 and 9 since the general concern is with the maximum head loss to be expected through the system. Presented in this fashion, both sets of data indicate a weir phase at low tailwater elevations in which the weir crest is unsubmerged and the upstream HGL is independent of tailwater elevation. At higher tailwater elevations, the weir crest begins to submerge and the upstream HGL begins to rise; in both these ranges, however, the HGL difference declines with tailwater elevation. At a sufficiently high tailwater elevation, the tailwater produces a condition such that the tailwater is higher than the top of the weir opening and the weir essentially acts as an orifice with a fixed head loss regardless of tailwater elevation. At the higher flow rates, this condition is only approximately achieved at a tailwater elevation of 620.5 ft. Under this condition, which would give the largest upstream HGL, the loss through the structure is a little under 1.0 foot at a flow of 6000 cfs and about 1.25 ft at 7000 cfs when the coarse screen bypass gates are closed. This head change is only reduced by about 0.3 ft when the bypass gates were opened. This magnitude of head loss was higher than anticipated during the structure design.

An additional issue during the design phase was the entrance and outlet conditions to the structure. Concerns were expressed both with regards to flow distribution within the facility and with regards to minimizing head losses in the entrance and outlet sections. The issue of flow distribution was investigated in a preliminary fashion by injecting dye pulses into the inlet flow. Observations of the dye could be used to determine the relative speed of the flow in both portions of the original tunnel section as well as both side channels. Figures 10-12 are images of the dye different times after release at a system flow of 7000 cfs. It can be seen that the bulk of the flow, perhaps as much as 75 percent continues down the original tunnel section for flows with both the bypass gates opened and closed. Outflow into the side channels only occurs over the most downstream portion of the opening provided between the original and the side channel sections. Since the original tunnel approaches the screen facility at an angle, this alignment also causes preferential flow to the right side of the structure.

Additional testing was conducted in order to estimate the relative distribution of flow between the bypass section and over the weirs when the coarse screen bypass gates were opened. There was no direct way to measure the flow rates in each of the channels so the flow split was estimated by measuring velocities in the center tunnel (bypass channel) approximately 2.5 ft (model distance) downstream from the screen bays and using this to estimate the discharge in these channels. Measurements were made with a calibrated mini-propeller meter at prototype flow rates of 6000 and 7000 cfs at tailwater elevations ranging from about 613.5 to 619 ft. Initially, velocity measurements were made at a single location in the middle of the flow channel for a variety of tailwater conditions. Subsequent testing at 7000 cfs was performed at a tailwater condition in which the channel was just flowing full (about 618.5 ft) at the measurement section and at a tailwater of 613.6 ft by measurements at nine different equally spaced points within the flow cross-section. The flow depth was measured at this section in order to determine the flow area. Table 1 provides the results of these measurements. The single point measurements indicated that roughly the same amount of flow passed through the bypass channel at 6000 cfs compared to 7000 cfs. Differences are likely to be within the measurement uncertainty of estimating flow rate from a single velocity measurement. As expected, the measurements also indicate that more flow passes through the bypass channels at low tailwater conditions compared to higher values. The more detailed measurements at 7000 cfs indicate less flow down the bypass channels at the lower

Table 1. Relative Distribution of Flow Between Bypass Channel and Overflow Weirs.

Flow Rate (cfs)	Tailwater (ft)	Proportion of Flow Through Bypass Channel
Single Point Velocity Measurements		
6000	614.3	64%
6000	614.9	62
6000	617.2	55
6000	618.6	50
7000	613.1	85
7000	614.5	71
7000	617	56
7000	618.6	44
Multiple Point Velocity Measurements		
7000	613.6	65
7000	618.5	48

tailwater but generally in the same range. Observations of dye releases at the upstream end of the channel as discussed above qualitatively confirm these results.

Phase 2 Results

Several modifications were made to the model in the second phase of testing with the objective of obtaining better flow distributions in the upstream section while attempting to minimize additional system head losses. Two general modifications were made in an attempt to achieve these objectives. The first involved a diverter on either side of the divider wall in the center of the inlet tunnel. Figure 13 is an image from overhead of the diverter installed in the model. The purpose was to force the flow away from the center walls and out into the side channels. A trial-and-error approach was used, varying the flare of the diverter until the flow distribution in the various channels was approximately equalized at a flow of 7000 cfs. Flow distribution was estimated by injection of dye upstream from the diverter and observing both dye transport speed and distribution. With the configuration indicated in Figure 13, an additional head loss of approximately 0.35 ft was introduced at the flow of 7000 cfs. This much additional head loss was deemed unacceptable. Additional efforts were made to modify this configuration with little effect on head loss. An alternate modification was made by installing a diverter that was initially parallel to the approach flow and flaring out to the sides as indicated in Figure 14. The concept behind this modification was to place this diverter so that flow intended to pass through the original tunnel sections passed on the inside of the diverters while pushing all flow on the outside into the side channels. This configuration had a lower head loss, on the order of 0.2 ft at a flow rate of 7000 cfs. Additional modifications to this configuration included removing two columns in close proximity to the diverter as well as the six-foot guide walls proposed in the original design. Neither of these changes had a detectable effect on the head loss.

Subsequent measurements in this phase of the investigation indicated that the contribution to the total structure head loss due to the flow through the upstream portion of the model was negligible. Therefore, any gains to be achieved by distributing the upstream flow were incidental and the placement of the diverters into the flow produced a drag force that contributed to overall head loss. Since the system head loss is a major concern in the context of the system behavior, the issue of equalizing flows in the upstream portions of the model was dropped from further consideration.

Additional experiments were undertaken in an effort to understand the distribution of the head changes within the structure. The four point gauges installed for this effort were at four locations along the structure: 1.) Upstream end, 2.) Just upstream of the screen bays, 3.) Just downstream of the screen bays and 4.) Downstream end. Results were obtained for several different but relatively high tailwater elevations for a flow of 7000 cfs both with the bypass gates open and closed. One issue with interpreting results is that the pressure taps are mounted at the side of the model in the stagnant area where the truck access is located on the right side of the model. It is unclear that the pressures measured here are exactly equal to those out in the main flow. Nevertheless, the results are fairly consistent as indicated in Figures 15 and 16. These figures indicate that the losses in the upstream end of the system (i.e. prior to the screens) are only on the order of 0.1 ft and very little loss occurs across the screens. Actually negative head losses are indicated across the screen bays; this is probably due to the location of the pressure taps as it obviously is not a real effect. The bulk of the loss, especially for the bypass gates closed is between the downstream side of the screen bays and the downstream end of the structure. Exactly where this loss occurs cannot be determined from the measurements and there was no convenient way to install pressure taps across the weir. However, given the findings from the inlet weir study (*Twelve Towns Retention Basin Model Study*, S.J. Wright and D.J. Lautenbach, Report UMCEE 10-05, May 2001) and a simple momentum calculation in the channels on the downstream side of the weirs, most of this HGL change is associated with the increase in momentum flux in the discharge passing down the exit channels. The momentum flux is zero at the upstream end of these channels and increases going downstream due to addition of the flow entering over the weir crest. This momentum increase must be directly offset by a pressure force decrease, which in a surcharged condition can only occur by to a drop in the HGL. A decrease consistent with the known flow area and a uniform distribution of flow over the weirs (which cannot actually occur in a surcharged condition due to the longitudinal change in pressure) predicts a HGL drop of close to two feet over the narrow portion of the exit channel. That this much doesn't actually occur is simply a reflection of the fact that the flow per unit length over the weir must be less at the upstream end in a surcharged condition due to the higher pressure on the downstream side of the weir in that area compared to further downstream.

Phase 3 Results

In the third phase of testing, a full set of rating curves were developed for the proposed design for the intermediate weir structure. These tests were performed at 1000 cfs increments between 1000 and 8000 cfs and each flow considered a range of tailwater elevations between approximately 613-620 ft. Furthermore, the tests were performed with the bypass gates in both the open and closed positions. The results of these tests are presented in Figures 17-21. Figures 17-19 are for the bypass gates closed. Figure 17 presents the head loss versus tailwater elevation measured for the range of flow rates while Figure 18 presents the same data as upstream HGL versus tailwater elevation. Figures 20 and 21 are the corresponding results for the bypass gates open. Figure 19 presents the results of the head loss through the model in a surcharged state versus discharge with the bypass gates closed. The head loss values presented are those measured for the highest tailwater elevation at each flow rate. The log-log plot of head loss vs. flow rate in Figure 19 indicates a straight-line relation over the entire range of flow rates.

At the completion of the tests to determine head losses through the structure, a series of tests were performed to examine sedimentation patterns in the model. A graded sand was sieved to provide different sizes of sand retained on sieves ranging from 80 mesh to 200 mesh. Initial tests were performed at a prototype flow rate of 2000 cfs. When the coarser sands were added at the inlet, they were almost immediately deposited on the model floor. This was taken to imply that the sand was not capable of being transported at this flow rate. The finest fractions of sand were transported through the model with very little deposition until they reached the downstream weirs where a fair amount of sand was deposited upstream from the weirs but some was passed over it and on out through the exit tunnel. Any prototype material with this settling characteristic would either be retained on the screens or, if small enough to pass the screens, would exhibit a similar settling pattern.

There was an intermediate range of sand size that could be transported some distance down the inlet tunnel before being deposited in the model. Most of this sand was deposited in the center tunnel; this is reasonable to expect given the fact that most of the entering flow continues down the center channel. The small amount of sediment that was transported into the side channels at the structure entrance tended to be rapidly deposited due to the low velocities in the side channels. The videotape shows the nature of the deposition patterns and subsequent scour of the central channel when the flow rate was increased to 7000 cfs. Figures 22 and 23 provide images of the deposition patterns. Overall, a reasonable conclusion is that the bulk of the material entering the intermediate weir structure doesn't encounter a significant reduction in velocity as it continues down the central channel and little deposition is expected. Some deposition can be expected just ahead of the screen facility and sufficiently large material will be retained on the screens. Some additional deposition of the material passing the screens will occur in the channels upstream of the weirs, but this should be a small amount.

Phase 4 Results

Under the assumption that the majority of the head change through the model occurs in the exit channel downstream of the weir crest, attempts were made to reduce velocities in the exit channels. Under the assumption that the majority of the head change in the model is associated with the increase in the momentum flux in the exit channel, a reduction in exit velocity would be required in order to limit the magnitude of the momentum flux leaving the side channels. Discussions with TetraTech personnel identified some potential modifications to the structure design. The first was to decrease the slope of the floor of the exit channel while a second was to increase the upstream widths of the exit channels. A potential issue with both these modifications is that the flow area at the end of the exit channels is unchanged since no modifications to either floor elevations or channel width are made there. However, computations of the flow velocity distribution along the exit channels under the assumption of uniform distribution of flow per unit length of weir crest indicated that the exit channel velocity would increase to a maximum in the narrower section of the exit channel, then reduce through the width transition before increasing again towards the downstream end of the exit channels. In reality, the flow distribution per unit length of channel is not uniform, increasing towards the downstream end; therefore the assumptions used in the analysis were not strictly valid. The two modifications to the model were made however, to determine any potential benefits in head reduction.

The initial modification reduced the floor elevation of the exit channel while leaving the width unchanged. Instead of a uniform slope from 611 ft down to 602 ft at the channel exit, the floor was lowered to 604 ft at the upstream end, again with a uniform but flatter slope. Testing this modification at flow rates of 6000 and 7000 cfs indicated imperceptible decreases in the system head loss as indicated in Figures 24 and 25.

The next modification involved leaving the lowered exit channel floor elevation while increasing the exit channel width over a portion of the length as indicated in Figure 26. Testing at flow rates of 6000 and 7000 cfs again indicated insignificant changes in the head change through the system as shown in Figures 27 and 28.

Since the two initial modifications discussed above produced only minor changes in system head loss, an additional investigation to determine the head variation along the exit channel was conducted. In order to perform this investigation the Plexiglas cover was removed from the downstream portion of the model. A small diameter flexible tube was connected to a point gauge on one end and the other end was simply inserted in the flow at various locations along the exit channel. Air was forced from the tubing to ensure that there was a continuous water column and the static pressure was estimated at several locations. Figure 29 provides the results of this measurement. Although the measurements probably are not highly precise, especially in regions with high velocity due to the flow disturbance associated with the tubing itself, a reasonably good picture of the head distribution in the exit channel is provided. Data are presented as head difference between the location sampled and the head recorded in the fixed downstream point gauge. The measurements were in the right (looking downstream) exit channel and the column station number refers to the rows of support columns required for structural support of the facility ceiling. Station 49 is at the upstream end of the exit channel, Station 58 is just into the section with the expanded width (with the width modifications as indicated in Figure 26), Station 65 is at the bend in the exit channel, and Station 88 is just upstream of the guide walls at the channel exit. From Figure 29, it is clear that the bulk of the head change occurs near the downstream end of the exit channel.

With the above information, considerations were given as to potential alterations to the downstream end of the exit channels. A relatively simple change to make in the model was to remove the six guide walls located where the two exit channels recombine and exit the structure through the original tunnel section. Simply removing these walls resulted in a reduction of system head change on the order of 25 percent as indicated in Figures 30 and 31. In the actual system, these guide walls were intended to provide structural support for the ceiling in addition to their hydraulic effect, so it will probably not be possible to remove these guide walls entirely in the prototype. However, the guide walls could be replaced by circular columns as necessary to provide the structural support for the ceiling and still result in a net reduction of system head loss.

CONCLUSIONS

Head losses reported in this report do not include screen losses that have been estimated by others. These should be added to HGL changes to obtain estimates of total system head losses.

The head losses through the intermediate weir structure are higher than initially expected; at high tailwater elevations, this is primarily due to the conversion between pressure and momentum in the exit channel after the flow passes over the weir crest. This head change represents the major drop in HGL through the entire structure. The only way to prevent that head loss is to increase the flow area in the exit channel, or as indicated in the testing of possible design modifications, to reduce the structural elements disturbing the flow leaving the exit channels. Since the cross-sectional area of the exit channels is similar in dimension to the exit tunnel, very little reduction in flow velocity appears to be actually feasible, but reducing the velocities of the combining flows should result in some reduction in head loss.

Head changes through the structure with the bypass gates open are reduced only a little at the high tailwater condition with more reduction at lower tailwaters. For example, at 7000 cfs, the reduction in head loss is about 0.3 ft at a tailwater elevation of around 620 ft. This appears to primarily be associated with the fact that less than half the flow passes through the bypass channel in this flow condition. If the change in the exit channel mentioned above were made, it would also contribute to a reduction in head loss with the bypass channel open, but with not as large of an effect.

The bulk of the flow entering the intermediate weir structure passes down the original tunnel sections until it is forced to spread laterally to pass through the screens. Although this situation is not optimal with respect to head loss, the head losses in this portion of the structure are not a significant portion of the total head change across the structure. In addition, attempts to divert the flow into the side channels in order to distribute the flow more uniformly added more head loss than was gained by uniformly distributing the flow. Therefore, it is not recommended to attempt to distribute the flow uniformly through the structure, as there is no inherent benefit in achieving this objective.

Sedimentation is not considered to be a major issue in this structure. The flow distribution in the structure upstream of the screen facility has most of the flow passing down the original tunnel section. Therefore the transport capability of the flow will not be materially diminished compared to the entrance tunnel flow. Most of the material transported into the structure will continue downstream to the screen facility or possibly be deposited directly upstream of it where de-watering channels are present. Most of the material small enough to pass through the fine screens will continue to be transported through the structure and down through the remainder of the system.

AIIM SCANNER TEST CHART # 2

Spectra
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Times Roman
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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

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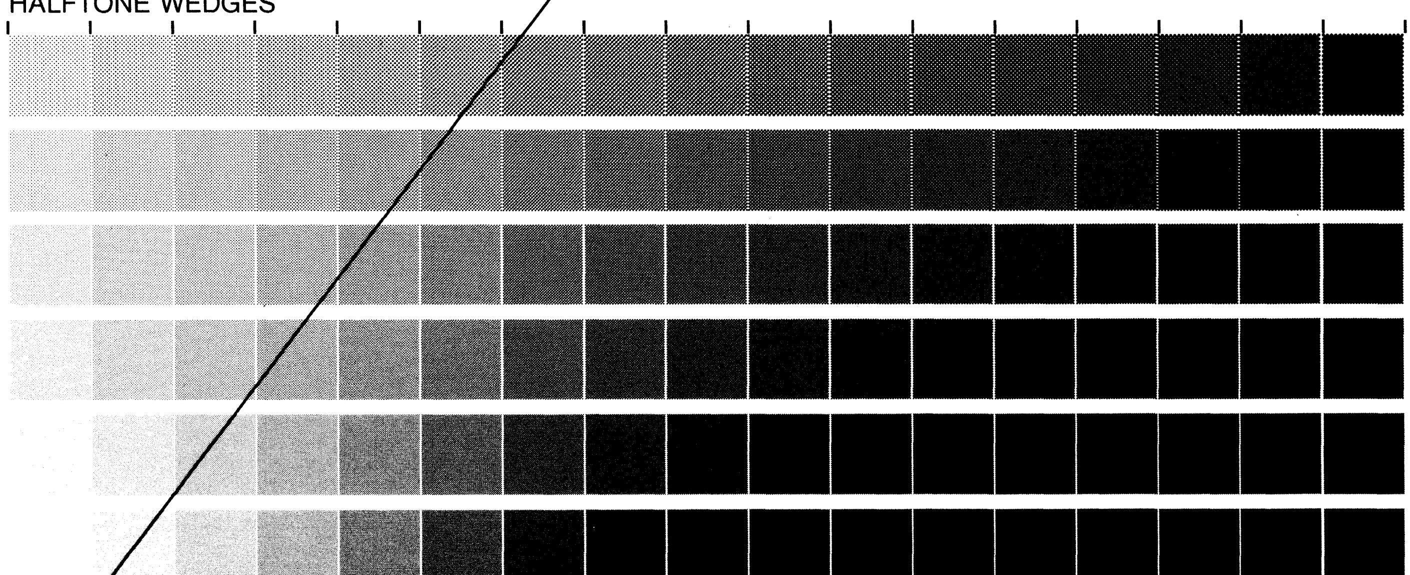
4	5	6	7	o	•
8	9	0	h	I	B

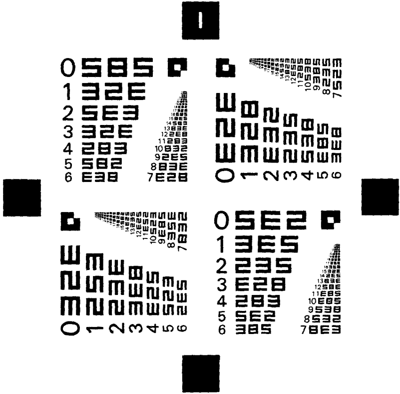
0123456 65432

A4 Page 6543210

MESH HALFTONE WEDGES



RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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