FINAL PROJECT REPORT
HYDRAULIC MODEL STUDY
Twelve Towns Retention Basin Model Study

Report UMCEE 01-05

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INTRODUCTION

Extensive modifications are planned to 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. A physical model was used initially to check apparently anomalous results from previous model testing of the RTF. The model was also modified to reflect proposed design changes to the structure and tested to determine the effect these changes had on flow through the weir structure.

The major issue in the RTF was the change in the hydraulic grade line (HGL) through a series of existing weirs and baffle walls. The weirs are to retain stormwater 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 downstream control structures as well as the total discharge and distribution among the various inlet conduits. Results from the existing facility were compared with data obtained by the U. S. Army Corps of Engineers (COE) during a previous model study. The proposed modifications were then made to the model and the change in HGL was measured for the same ranges of discharges and tailwater elevations.

The proposed testing sequence therefore consisted of the following components:

- HGL results using the existing weir configuration, including rating curves for the existing system.
- HGL results for the proposed design.
- HGL results for different alternative configurations with the purpose of reducing the total head change through the structure

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. A number of inlet culverts including a triple box culvert (the South section), a horseshoe culvert (the North section), and another box culvert (Red Run enclosure) deliver flow to the upstream side of the inlet weir. This weir is V-shaped as indicated schematically in Figure 1. The weir is constructed with a baffle wall in front of the weir crest to restrict the downstream movement of floating trash. In addition, flap gates are installed on the downstream side of the weir crest for purposes of odor control. The existing operation also has two 10 ft high by 18 ft wide Broome gates just downstream of the weir, which are normally closed during dry weather conditions. Dry weather flows are diverted into a 60-inch sanitary sewer just upstream of the 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, corresponding to a flow over the weir of approximately 2000 cfs, 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. These modifications were intended to retain dry weather flows without using the Broome gates and to reduce the amount of head loss within the system during high flows without the use of the Broome gates, thereby lessening flooding issues upstream while still capturing material that could impact downstream water quality. A conceptual sketch of these modifications is indicated in Figure 2.

Inflow into the inlet weir structure can vary depending on the rainfall patterns over the collection system area. In the earlier study (*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), essentially all testing was performed under a specific flow distribution in which 50 percent of the inflow was delivered through the triple box inlet, 30 percent through the Red Run enclosure inlet and 20 percent through the horseshoe culvert inlet. The current tests were also performed with this same flow distribution in order to provide a consistent basis for comparison of results. However, a limited number of tests were performed varying the inflow distribution to determine the effect on structure head losses.

Tailwater elevations are controlled by downstream structures. 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 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. The intermediate weir structure design was also subject to a physical model study (Report UMCEE 01-06) to determine system losses through the weir structure. Tailwater elevations at the inlet weir structure may therefore range from a low of just over 614.5 ft to a high on the order of 620 ft. The latter elevation is sufficient to surcharge most of the inlet weir structure.

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 structures within the system. The entire length of the RTF was not modeled 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 existing inlet weir was therefore built at 1:20 scale. At the high discharges on the order of 6000 cfs, typical model Reynolds Numbers are on the order of 100,000. For the lower range of flows studied, viscous effects may be present but are presumed to be negligible. Any effect will be to slightly increase model head losses relative to the corresponding prototype conditions under low discharge conditions.

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 1:20. The physical model was constructed of plywood, Plexiglas and PVC pipe. Plexiglas was used where visual access to the flow was required as well as the weir crests where dimensional control was critical. Exterior grade plywood was used in the remainder of the construction for reasons of model construction economy. The model consisted of the entire inlet weir structure as well as short sections of the inlet culverts and the outlet tunnel. Flows were introduced to the model through three eight-inch PVC pipes suspended above the model. Each pipe delivered flow to a separate inlet culvert. These inflows were regulated using valves located at the upstream end of the model. Tailwater elevations were regulated by means of an adjustable gate at the downstream end of the model. A key issue with regards to interpretation of all visual images of the model is that a mirror image of the actual structure was constructed in order to facilitate placement of the model in the laboratory. That is, the right hand side (looking downstream) of the actual structure is located on the left side of the model and vice versa. Since the weir structure itself is symmetrical, this issue is irrelevant with respect to the weir, but the positions of the inlet culverts are inverted as well as the location of the current staff gage in the structure. In reporting results, all data will be given in the context of actual prototype orientation.

The model consists of the basic control structure, including the weirs, upstream baffle walls, flap gates, and the Broome gates. All internal columns and guide walls were reproduced as well. The flap gates were constructed from sheet aluminum with a thickness determined to produce a properly scaled weight of the prototype gates. Dimensions for the existing structure were obtained from a series of design and as-built drawings provided by TetraTech MPS. Additional design drawings were provided for the structure modifications. In the case of the existing structure, there are some discrepancies among the various drawings, in particular with regards to the dimensions of the inlet culverts. The dimensions taken from the as-built drawings were used to construct the inlet sections. The previous COE model included all the ceiling beams in the model construction; these would influence head losses if the structure were in a surcharged condition. The COE study also investigated the effect of providing a smooth surface at the bottom of the ceiling beams and concluded that this had only a minor effect on structure head loss. Consequently, the current model was constructed with a flat roof with a ceiling elevation at the bottom of the beams. An additional issue is that the COE report gives the weir crest elevation of 614.85 ft as opposed to 615.5. This difference appears to be due to the use of a different elevation datum; all comparisons in the results from the two studies reflect an adjustment of all tailwater elevations in the COE report by adding 0.65 ft.

The dry weather flow bypass was not constructed in the model, so all flow exiting the model passed through the structure and out the downstream tunnel section.

Images of the model are provided in Figures 3-6, which indicate various aspects of the structure and the associated flow delivery system.

INSTRUMENTATION

The total discharge to the model was measured with a venturi meter that is part of the laboratory water supply system. Flow rates to the individual inlets were measured using pipe orifice meters constructed to ASME specifications. Approach flow influences were minimized by making the amount of straight pipe upstream and downstream from the orifices as long as possible. Pressure differences were measured through the use of water-air differential manometers. These meters were used to set the desired flow distribution among the various inlets.

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 at each side of the entrance to the model (station 0+00), one at the midpoint of the model at station 1+65 (the right side corresponding to the current location of a staff gauge mounted in the prototype) and one at the exit of the model at station 2+84 (just into the downstream storage tunnel). The 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 615.5 feet. All subsequent elevations are referenced relative to these readings. This procedure was repeated several times throughout the study, usually associated with a situation where one point gauge that was exposed to laboratory traffic was bumped, altering its orientation. Generally speaking, the results of the re-zeroing procedure provided measurements that were repeatable within about 0.002 ft in the undisturbed point gauges.

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

- Meaurement 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 venturi 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. Repeated measurements with a single point gage indicate a repeatability of on the order of 0.05 ft prototype. Since HGL difference is based on the difference between point gage readings, a basic uncertainty therefore ought to be on the order of 0.1 ft. Examining data for a fixed flow rate and surcharged conditions so that the head change should be independent of tailwater elevation indicates that this is in the correct range. Earlier tests may exhibit slightly more scatter as testing protocols were being resolved. 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

The testing procedure was fairly straightforward. In general, the desired discharge was set in the model and the control valves in each inlet line were adjusted to provide the desired flow split. 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 generally several different but arbitrary elevations in the range of interest were 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. Typically, all four point gauge readings were taken in a particular measurement. The two upstream point gauge (on opposite sides of the model just into the model from the inlet culverts) readings were averaged to give a single upstream elevation. HGL differences were determined by subtracting the downstream tailwater elevation from appropriate upstream elevations. This study was done in four distinct phases. In the first phase, measurements at several discharges were made through the unmodified inlet control structure for a range of tailwater elevations. These were done for flows in which the Broome gates were either open or closed. This data was then compared to the results from the earlier COE study. Rating curves were then created from the data for the unmodified structure. The distribution of flows among the various inlet culverts was also varied at a single flow rate of 7000 cfs to see what influence this would have on the structure HGL differences.

In the second testing phase, the model was modified to reflect proposed design modifications. The upstream weir was lowered from an elevation of 615.5 ft to 613.5 feet and a weir extension was constructed according to the plans presented in Fig. 7. The elevation of the weir extension was also 613.5 ft and baffle walls and flap gates were added at the weir extensions. Measurements at 4000, 6000 and 7200 cfs were taken for a range of tailwater elevations. The hydraulic grade lines calculated were then compared with data from the unmodified model. The HGL changes in this phase of the testing were found to be higher than anticipated, leading to subsequent testing.

Further model modifications were completed in the third testing phase. This phase consisted of measurements of flow at a single flow rate of 7200 cfs and usually three different tailwater elevations that corresponded to surcharged conditions within the model. The purpose of this phase was to explore possible design modifications that might potentially reduce the HGL changes across the model.

In phase four, 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.

TEST RESULTS

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

Phase 1 Results

This first phase consisted of three parts, the comparison of hydraulic grade line results to the COE results, the production of rating curves for weir flow for the current configuration, and the study of different ratios of inlet flow through the three inlets and the resulting effect on head loss. During the first phase of the testing, flow was apportioned among the three inlets using the following distribution provided by Tetra Tech: 50% of flow from the southern triple box culvert, 30% of flow from the middle box culvert, and 20% of flow from the northern horseshoe shaped culvert to match the same flow distribution used in the COE model study. This flow relationship was used throughout the model testing unless otherwise noted.

Hydraulic grade line differences were first measured for 4000 and 6000 cfs and the Broome gates open for a variety of tailwater elevations. These measurements can be seen in Figure 8. Superimposed on this figure are results from the previous COE study with the adjustment in tailwater elevation of 0.65 ft as discussed previously. These data are for the HGL change across the entire structure and compare to data presented in Figure 11 of the COE report. The current model results were found to correspond fairly well with the COE results. 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. Visual observations indicate that this transition is not sudden in the model; rather the upstream end of the weir section submerges or surcharges at a lower tailwater elevation than the downstream end of the weir. Thus, at intermediate and high tailwaters, the downstream head at the upstream end of the weir is greater than that at the downstream end and therefore less flow should pass through the upstream portion of the weir section compared to the downstream portion.

A data set for 2000 cfs and second set of 4000 cfs data was next taken for a variety of tailwater elevations, again with the Broome gates open. The head difference between the measurements at station 1+65 and 2+84 was compared to data presented in the COE report as Figure 14. This comparison is presented in Figure 9 and shows significant discrepancies between the two data sets especially at low tailwater elevations. Data provided by the USACE was found to underestimate the head loss between the staff gauge location (station 1+65) and downstream sections of the model. This is especially curious given the relatively good correspondence in Figure 8 for the same flow of 4000 cfs over the entire structure. The COE results imply a significant head loss between stations 0+00 and 1+65; this was never indicated in any phase of the current model study. An additional problematic condition is in Figure 16 of the COE report that presents HGL at station 1+65 as a function of discharge and tailwater elevation. In the COE report, for low tailwater conditions, the HGL at station 1+65 is predicted to be less than the weir crest elevation up to the maximum discharge of 8000 cfs. A test was performed to determine the maximum discharge that could be passed through the Broome gates without spilling over the weir; this was found to be approximately 3000 cfs. Figure 10 presents HGL changes between stations 1+65 and 2+84 for the Broome gates closed and a discharge of 2000 cfs. In this case, both studies yield fairly consistent results. The discrepancies noted in Fig. 9 are not apparent.

Given the above comparisons, it is concluded that a data analysis error probably resulted in an incorrect HGL in the COE study for station 1+65 and the Broome gates opened. There seems to be no other basis to explain the results in Figs. 8-10. Without access to the original data, no definite conclusions are possible, but this appears to be the most likely explanation.

A rating curve giving the head discharge relationship for unsubmerged flows with the Broome gates closed was next compiled. Data was taken from the staff gauge at station 1+65 and the upstream station 0+00. This curve can be seen in Figure 11. Data from both measurement locations indicate about the same head changes showing little loss in the portion of the model upstream from the weirs. From this data, a relationship between water height above the weir and discharge was established for unsubmerged flows between 1000 and 3500 cfs. This equation is as follows:

$$h = 0.0008575Q^{0.9744} \quad (1)$$

where h is the water head above the unmodified weir crest (in ft) and Q is the discharge in cfs through the system.

The weir coefficient for the unsubmerged weir was calculated using data from the staff gauge at station 1+65 and the upstream gage at 1+00. This data is presented in Figure 12 and in Table 1 below.

	Weir Coefficient		
Flow (cfs)	Upstream	Staff Gauge	Average
879	4.2	4.3	4.25
1073	4.0	4.2	4.10
1101	4.3	4.4	4.35
1502	3.5	3.9	3.70
1729	3.0	2.8	2.90
2039	3.5	3.7	3.60
2269	2.7	2.6	2.65
2617	2.9	2.7	2.80
2673	2.9	2.7	2.80
2873	3.0	2.8	2.90
2925	2.9	2.8	2.85
3184	2.7	2.6	2.65
3553	2.4	2.3	2.35

Table 1: Weir Coefficients versus Flow

As expected, the weir coefficient becomes less as discharge increases and additional losses due to the baffle walls, etc. become more significant. Weir coefficients vary from a high of 4.35 at about 1000 cfs to a low of 2.3 at 3500 cfs. The high values at low flow rates are subject to the uncertainty in the weir crest elevation as discussed previously. The weir head is estimated to be too small by the experimental procedure. At the lowest flow rates (and thus weir heads), an uncertainty of 0.2 ft in the crest elevation could result in a 40 percent error in the weir coefficient, reducing it from a value of 4.2 to 3.0. A table providing the relationship between the head in inches above the weir crest and the flow rate is provided in the appendix.

The relationship between head loss and inlet flow distribution was also studied for a number of different entrance flow combinations under surcharge conditions. All combinations had a total flow rate of 7000 cfs. A high flow rate was chosen because the potential flow difference between various inflow points would be the largest at high flows, giving one the greatest difference in observations between ratios. Table 2 below relates the different proportions of flow and the resulting head loss through the model.

% Flow Distribution (7000 cfs)

Triple Box/Center/Horseshoe	Head loss T-box side (ft)	Head loss Hshoe side (ft)	Average
50/30/20	1.66	1.44	1.55
50/25/25	1.78	1.52	1.65
50/15/35	1.72	1.28	1.50
45/20/35	1.62	1.32	1.47

Table 2: Flow Distribution and Resulting Head loss

The variation in average head loss among the various runs is slightly greater than the uncertainty estimate provided above. It was found that there was not a large difference in head loss as the inflow proportions were varied. The losses through the triple box side appear to be independent of the flow split; this is not surprising since that proportion of the total flow varied the least. It was noted that the average head loss decreased as the amount of flow coming out of the center box culvert lessened. This may be due to the fact that much of the flow passes through the Broome gates at high flows. Flow entering through the horseshoe inlet would pass more directly through the Broome gates than through the Red Run enclosure inlet, apparently resulting in less system head loss on that side of the structure.

Phase 2 Results

In the second phase of testing several modifications were made to the retention treatment facility model. As mentioned earlier, the upstream weir elevation was lowered two feet from 615.5 feet to 613.5 feet. The existing flap gates also had an extra two ft wide panel added at the bottom to cover the larger weir opening. An additional weir was added downstream of the Broome gates on either side of the model. These weirs were set at elevation 613.5 feet as well and were each 90 feet in length. A baffle system similar to the one in front of the upstream weir was also added 36 inches in front of the downstream weirs. An elevated floor sloping back towards the Broome gates was placed in front of the downstream weirs, reducing the size of the opening made between the downstream baffle walls and floor. This floor was intended to flush everything back towards the dry weather flow channel.

After the modifications were completed, the model was then run at a flow rate of 7200 cfs and measurements of head differences for different tailwater elevations were taken. This flow rate was not measured in the original COE study, so the results were scaled up to a flow rate of 8000 cfs to provide a basis for comparison with COE data. The comparison of the modified system data to the old system data can be seen in Figure 13. The new configuration was found to yield much higher head losses than indicated by both the unmodified model data and the earlier COE data. The head loss was found to be as much as 2.5 feet higher in comparison to the unmodified system. Measurements of head differences for different tailwater elevations were taken for additional flow rates of 4000 and 6000 cfs. Similar increases in head loss were found for both of these flow rates as well but with lower overall discrepancies at lower discharges as expected. The differences between the new 4000 and 6000 cfs head differences and the original measurements can be seen in Figures 14 and 15 respectively. This outcome was in conflict with the design objectives of equal or lesser head losses with the modified inlet weir.

Phase 3 Results

Modifications to the weir and baffle systems were made to lower the head loss through the retention treatment facility. An initial consideration was that while lowering the weir crest elevation two feet increased the flow area through the weir opening, nothing was done to change the opening at the baffle wall. In the high tailwater condition, both the openings at the weir and the baffle walls serve as an orifice and the smallest opening would be the most limiting by controlling the losses at the restriction. Several different modifications were made to the baffle and weir system. These tests were run at 7200 cfs as well and the head losses measured were compared to the original modified head loss measurements as well as to each other. Seven different geometric configurations were studied. Measurements for all seven modifications were taken with the lower section of the model surcharged, each case with three different tailwater elevations to reduce measurement uncertainty. Modifications and resulting head loss measurements are presented below in Table 3:

	Downstream	Head loss (ft)			
	Elevation	Right Side	Left Side	Average	Average of Total
1	622.6	3.2	3.7	3.5	3.5
	624.2	3.3	3.7	3.5	
	622.2	3.4	3.8	3.6	
2	624.8	2.5	2.9	2.7	2.7
	623.0	2.4	2.9	2.7	
	623.8	2.4	2.9	2.6	
3	623.5	3.4	3.7	3.6	3.5
	625.2	3.3	3.7	3.5	

	622.0	3.4	3.7	3.5	
4	624.6	3.0		3.0	3.0
	622.6	3.0		3.0	
	624.0	3.0		3.0	
5	622.5	2.5		2.5	3.2
	624.2	3.3		3.3	
	626.5	3.1		3.1	
6	623.6	2.9	3.0	3.0	3.0
	624.7	3.0	3.1	3.1	
	622.1	3.0	3.0	3.0	
7	622.8	1.6	1.9	1.7	1.7
	624.1	1.6	1.8	1.7	
	625.0	1.6	1.8	1.7	

Table 3: System Head loss for Different Upstream Geometric Configurations

The seven different geometric arrangements were as follows:

1. The original upstream configuration with the downstream modifications present. This configuration was used as the baseline for head loss.
2. Downstream baffle walls were removed and head loss was reduced by 0.8 feet
3. Downstream baffle walls were replaced but placed 48 inches away from the weir instead of the original 36 inches. There was no reduction in head loss.
4. The baffle wall bottom was raised to 614.5 feet from the original 612.5 feet and the baffles were placed back in their original configuration 36 inches away from the weir. Head loss was reduced by 0.5 feet
5. The downstream baffle walls were removed but the Broome gates were then lowered to an elevation of 612 feet to serve a baffle function. Head loss went up by 0.5 feet compared to Arrangement 2
6. Three-foot deep notches were cut in the upstream baffles, see Figures 16-17. These notches extended 45° from the columns until the correct depth was reached, making trapezoidal shaped holes at the bottoms of the baffles. The head loss was reduced by 0.5 feet
7. Both upstream and downstream baffle walls were removed. This reduced the head loss by 1.8 feet

We can see from the testing that a model setup was found that produced a head loss of only 1.7 feet. However, the upstream baffle wall cannot be completely removed due to structural concerns. Therefore, the upstream configuration for the remainder of the tests was the same as the sixth arrangement tested consisting of the three-foot deep trapezoidal notches in the baffles which involved removal of as much material from the baffle walls as possible without compromising their structural capability.

Further investigation was performed to determine where the bulk of the remaining head loss occurred. This was shown to be in the section of the model downstream of the Broome gates as the flow through the 293-foot long upstream weir recombined with the flow through the weir extensions. In an attempt to reduce this head loss further, diverter walls were added to the channel on the immediate downstream side of the weir extensions as indicated in Figure 18 without any baffle walls on the upstream side of the weir extensions. This was determined to have a small but measurable reduction in system head loss, by approximately 0.25 ft at a flow of 6000 to 7000 cfs. Additional efforts to alter the diverter wall locations from that indicated in Fig. 18 had small but negative impacts on system head loss.

Phase 4 Results

In the fourth and final phase of testing, a full set of rating curves were developed for the configuration that incorporates the following modifications relative to the existing inlet weir structure:

- Lowering the existing weir crest to an elevation of 613.5 ft
- Adding a 2 ft extension to the existing flap gates
- Cutting trapezoidal notches in the existing baffle walls
- Removing the Broome gates
- Add 90 ft weir extensions on the downstream side of where the Broome gates were located.
- Flap gates on these weir extensions
- Diverter walls placed on the downstream side of the weir extensions.

A series of measurements were taken with this modified system for flow rates ranging from 1000 to 8000 cfs, each for a range of tailwater elevations. Plots of head loss versus tailwater, upstream head versus tailwater, and the head loss versus flow rate under surcharged conditions are shown in Figures 19, 20 and 21 respectively. One can see on Figure 19 that at surcharged conditions, head loss is approximately 1.6 feet through the entire structure at 6000 cfs and 3 feet at 8000 cfs.

CONCLUSIONS

The initial testing of the existing inlet weir showed some perplexing differences between the current results and the results reported in the previous COE report. Specifically, comparisons between the two different models were fairly good in terms of total system head loss through the structure at the flows tested and in the head loss from the staff gauge location to the downstream end of the structure if the Broome gates were closed. However, in comparing reported head losses from the staff gage location to the downstream end of the structure with the Broome gates open, the COE report indicates much lower head losses than observed in the current study. There are two reasons to distrust the COE results: 1.) Their findings indicate an unrealistically large head loss upstream from the weirs and 2.) The Broome gates are indicated to be capable of passing 8000 cfs without exceeding the weir crest elevation at low tailwaters. Neither of these results appears physically possible. It is concluded on the basis of the comparison of other data that there was some sort of data analysis error in the original study that led to an incorrect determination of the head losses between the staff gauge location and the downstream end of the structure with the Broome gates open.

The originally proposed modifications to the retention treatment facility are not acceptable from a head loss standpoint. The addition of the downstream weir and baffle system produces much additional head loss to the flow that previously passed through the Broome gates. This would also increase the flow through the existing weir. Lowering the existing weir from 615.5 to 613.5 feet does not mitigate this head loss.

The modifications tested in phases three and four eventually produced reduced head losses. These changes give a head loss of approximately 1.6 feet through the entire system under surcharged conditions at 6000 cfs. Modifications to the existing inlet weir structure in order to achieve this head loss include:

- Lowering the existing weir crest to an elevation of 613.5 ft
- Adding a 2 ft extension to the existing flap gates
- Cutting trapezoidal notches in the existing baffle walls
- Removing the Broome gates
- Add 90 ft weir extensions on the downstream side of where the Broome gates were located.
- Flap gates on these weir extensions
- Diverter walls placed on the downstream side of the weir extensions.

An investigation was performed to determine where the remaining head losses are concentrated in the flow through the model. Because of the transparent nature of the cover portion of the model, it was possible to observed water levels at various points of the model when the system was surcharged at tailwaters above about 620 ft. From these observations, it was clear that the head loss across the existing weir was fairly small and that the bulk of the loss occurred in the downstream portion of the model where the flows through the existing weir and the weir extensions were re-combining. Due to the nature of the flow processes as these separate streams mix, there is no practical way to reduce this loss significantly within the flow area constraints posed by the existing structure.

AIIM SCANNER TEST CHART # 2

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Greek and Math Symbols

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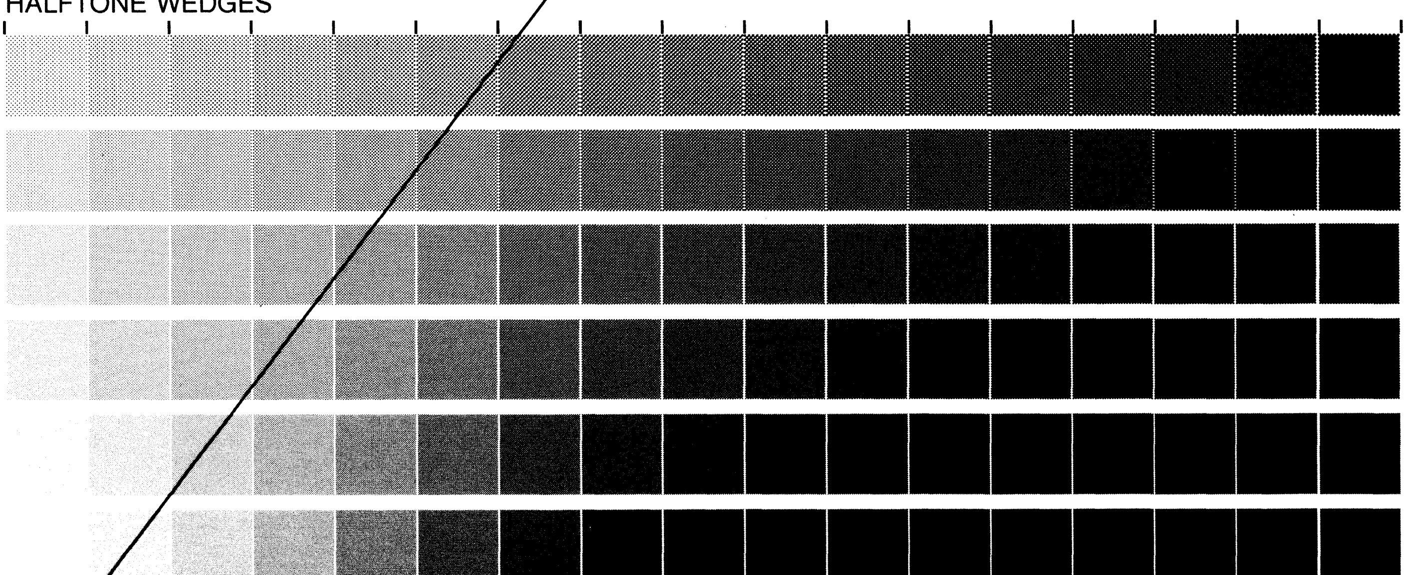
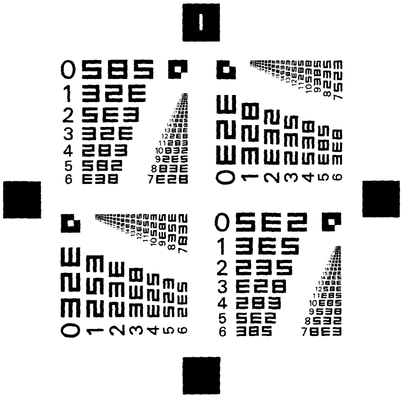


RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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