Evaluation of
U.S. Army Engineers Waterways Experiment Station
Report HL-82-4

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For

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INTRODUCTION

The U.S. Army Corps of Engineers performed a physical model study of the Red Run Drain Pollution Control Structure in 1982 (*Pollution Control Facilities Project Southeastern Oakland County Sewage Disposal System*, Oakland County Michigan, Report HL-82-4 by Ronald R. Copeland, Waterways Experiment Station, February. 1982). As part of a recent design effort to upgrade the facility to improve its capability to handle stormwater flows, the results of this earlier study were reviewed as part of establishing the basis for design for the facility modifications. Previously, Debbie Bauer and Sam Glovick of Wade Trim and Associates had performed an analysis of the physical model tests results for the outlet weir structure in an attempt to replicate the hydraulic grad line changes through this structure. The results of their analysis were transmitted in two faxes dated July 6, 2000 and consisted of estimates of the head changes through the various structural components of the outlet weir structure. The results of their analysis concluded that the head changes could be estimated reasonably well for flow conditions in which the outlet weir operated in an unsubmerged mode. However, their analysis resulted in lower head changes through the structure compared to the physical model study results when the weir was in a submerged mode. This analysis provides an additional look at the flow through the outlet weir as well as an analysis of the flow through the existing inlet weir structure.

OUTLET WEIR HYDRAULICS

The analysis by Debbie Bauer and Sam Glovick indicated that the head change across the weir itself is the major contributor to the head change through the entire outlet structure under unsubmerged flow conditions. Additional losses were assigned to the baffle walls immediately upstream of the outlet weir by treating the baffle wall as an orifice/expansion type of flow. The head losses estimated for the baffle wall were relatively minor (on the order of 0.05 ft) for a flow of 6000 cfs. While the specific assumptions utilized in the analysis might be subject to revision, it appears to be a reasonable conclusion that these head losses provide a relatively minor contribution to the total head change through the structure. The present analysis therefore does not consider that aspect of the system flow but rather concentrates on the weir flow in order to determine the validity of the physical model test results.

The weir wall has a thickness of 1.5 feet, which is greater than would be required to satisfy the condition for a sharp-crested weir according to a variety of references (e.g. *Discharge Measurement Structures*, 3rd edition, ILRI Publication 20, M.G. Bos, ed.) However, Hager and Schwalt (*Broad-Crested Weir,* Journal of Irrigation and Drainage Engineering, Vol. 120, No. 1, pp. 13-26) and others indicate that so long as the weir head to crest length ratio is greater than 1.5 (and possibly as small as 0.4), the weir would behave as a sharp crested weir. The ratio of 1.5 is not exceeded for most of the flows in the physical model study but the lower value of 0.4 is apparently satisfied for the entire flow range covered in the tests. In subsequent analyses, sharp-crested weir relations will be utilized since there is a broader range of experimental conditions available from which to estimate weir coefficients and it is more clear how to apply them to the specific configuration. However, it is also likely that the analyses are only accurate to within a few percent.

For sharp-crested weirs, there is information available from which the effect of downstream submergence can be estimated readily and Figure 5.5 in Handbook of Hydraulics, 7th ed. Brater, King, Lindell, and Wei provides a compilation of previous experimental results. The study by Hager and Schwalt referenced above also provides data on the effect of submergence on broad-crested weir discharges but only for relatively greater weir crest lengths and their results are apparently not applicable to the present analysis.

In the Corps of Engineers report, the head changes through the downstream section of the control structure are provided in Figure 10. At flow rates between 4000 and 6000 cfs, the curves drop with a minus 1 slope on the left hand side of the figure before they level off on the right hand side. This minus 1 slope is indicative of a free discharge at the weir. The curves in Figure 10 were inspected too determine the tailwater elevations at which they begin to deviate from the minus 1 slope and estimates for these tailwater elevations are tabulated below:

TABLE 1.

Flow Rate (cfs)	Total Head Loss (ft)	Tailwater Eelevation (ft)
4000	1.5	611
5000	2.5	610.3
6000	3.4	609.6

The tailwater elevations are subject to a fair degree of uncertainty since they are estimated only from an apparent and small deviation from the minus 1 slope on Figure 10. Nevertheless these tailwater elevations are quite small when compared to the heads required downstream from a sharp-crested weir in order to provide submergence effects (Figure 5.5 from Brater, et al). From Figure 5.5, the downstream head needs to be at least ten percent of the upstream head in order to create a submergence effect. All of the tailwater elevations listed above are below the weir crest elevation of 611.25 ft, implying that there may be an additional head loss downstream from the weir prior to the location where the tailwater elevation was measured in the model study. This loss is most likely due to the hydraulic jump-like flow in the plunging jet on the downstream side of the weir and was not accounted for in the analysis by Bauer and Glovick. The following table represents an analysis for the estimated heads at the three discharges of, 4000, 5000 and 6000 cfs at the tailwater limit for downstream submergence effect:

TABLE 2.

Flow rate (cfs)	Weir Head (ft)	Upstream w.s. Elevation(ft)	Total Head Loss (ft)	Downstream Head loss (ft)
4000	1.18	612.43	1.43	0.4
5000	1.37	612.61	2.31	1.1
6000	1.54	612.8	3.20	1.8

In this table, the weir head is estimated using the weir coefficients for sharp-crested weirs given by Bos (referenced above). The upstream water surface elevation simply adds the weir head to the crest elevation of 611.25 ft. The total head loss is computed as the difference between the upstream water surface elevation and the limiting tailwater elevation given in Table 1, while the downstream head loss is estimated as the difference between the limiting head required for submergence ($H_{downstream}/H_{upstream} = 0.1$ as estimated from Brater, et al) and the tailwater elevation. The total head losses estimated by this procedure and listed in Table 2 are slightly less than those observed in the physical model and listed in Table 1. The differences between these could be due to the additional losses at the baffle wall, but there are also uncertainties associated with the estimates of the weir coefficients as well and these may be equally important. In addition the limiting downstream tailwater elevation is estimated from a gradual deviation in Figure 10 from the limiting slope, so overall, it appears that the two results are fairly consistent.

A reasonable conclusion from these analyses is that losses upstream from the weir crest are fairly small and the major contribution to head changes is the head across the weir. In the submerged condition, there are apparently additional head losses on the downstream side of the weir due to the hydraulic jump. As the flow becomes less submerged, these downstream losses become less important. Unfortunately, there does not seem to be adequate information from which these downstream losses can be estimated for a general tailwater condition.

EXISTING INLET WEIR

The following analysis relies on some of the conclusions derived from the analysis of the outlet weir structure. Flow through the inlet weir structure is complicated by the fact that the Broome gates are open in the physical model tests and a majority of the flow passes through the gates. If the gates are fully open, it is reasonable to assume that they behave as broad-crested weir structures. An analysis was made to determine whether this assumption was reasonable, at least under conditions in which the discharge is determined by a free discharge state. Again, in Figure 11, the head losses in the inlet structure follow a minus 1 slope with tailwater elevation on the left side of the Figure, indicating an unsubmerged flow condition. A specific tailwater elevation of 612.5 ft was selected for the analysis that indicates an unsubmerged flow condition for discharges of 8000 cfs or less. The total head change through the structure was estimated from Figure 11 for several flow rates, from which the upstream water surface elevation was computed. The head on both the inlet weir and the broad-crested weirs (Broome gates) was computed by neglecting all upstream losses and the flows across the two weirs were computed (weir crest elevation of 614.85 ft and gate floor elevation of 606 ft). Weir coefficients were estimated to be 3.3 for both weirs; these are reasonable consistent with the values given in Brater, et al and Hager and Schwalt. The results are tabulated below:

TABLE 3.

Discharge (cfs)	Head Loss (ft)	Upstream w.s.	Weir Head (ft)	Weir Q	Gate Q	Total
4000	3.1	615.6	0.75	675	3530	4200
5000	3.5	616.0	1.15	1280	3760	5040
6000	4.0	616.5	1.65	2200	4040	6240
8000	4.65	617.15	2.30	3625	4420	8040

In all cases, the flow balance is reasonably good and probably within the uncertainties in reading the figures and estimating the weir coefficients. Note that the flow conditions change from where the discharge across the inlet weir is only about fifteen percent of the total at 4000 cfs up to where it is nearly half of the total at 8000 cfs. This analysis indicates that the head changes can be fairly well accounted for by considering only the flows at the weir and through the Broome gates, at least in unsubmerged conditions. Not attempt was made to perform similar analyses for submerged flow conditions due to the lack of information on submerged flow conditions for broad crested weirs. However, given the results presented above for the outlet weir structure, it is reasonable to expect that in partially submerged flow conditions, there will be some additional head loss on the downstream side of the Broome gates and the inlet weirs that will affect the relative flow distribution.

CONCLUSIONS

Several simplifying assumptions were made in order to perform simplified analyses for the head changes at both the outlet and the inlet structures. Nevertheless, the flow behavior was computed to be fairly consistent with the results measured in the Corps of Engineers physical hydraulic model study. Therefore, it is concluded that the hydraulic model study results are reasonable. In addition, the following conclusions are obtained:

- Head losses due to upstream baffle walls and other structural elements are probably negligible for typical flow rates and can probably be neglected in any preliminary design analyses;

- Under partially submerged flow conditions, there are head losses downstream from the weir structures that contribute to a head difference on the immediate downstream side of the weir and further downstream where tailwater conditions may be better defined. These losses should be minor in the case of free discharge conditions and will become increasingly more important as the weirs become partially submerged or influenced by the tailwater condition.

- Weir coefficients for the structures can probably be fairly well estimated from information on standard sharp-crested weirs.

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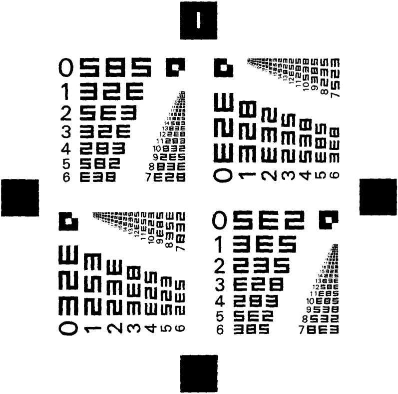

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