THE HYDRAULIC DESIGN TOOLBOX: THEORY AND MODELING FOR THE LAKE TOWNSEND SPILLWAY REPLACEMENT PROJECT

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Abstract

Lake Townsend is the primary water supply for the City of Greensboro, North Carolina. The dam's concrete gated spillway is suffering from severe deterioration due to alkali silica reactivity and has inadequate capacity to meet spillway design flood (SDF) requirements. To address these dam safety issues, the existing spillway will be replaced with a labyrinth weir located just downstream of the existing structure.

A combination of theory, computational fluid dynamics (CFD), and physical modeling were applied to the design of the replacement spillway for Lake Townsend Dam. In addition, updates related to ongoing research related to hydraulic performance of labyrinth weirs are presented.

Labyrinth discharge can be estimated using reliable empirical methods and through physical or numerical modeling. The hydraulic design for this spillway included several challenges, including weir submergence at high flows and providing energy dissipation over a wide range of flows and tailwater conditions.

Spillway hydraulic modeling included 2-D and 3-D CFD and a physical model study of a 1.5 cycle labyrinth weir with several stepped chute configurations. These models were used to:

- estimate discharge capacity and compare with empirical methods,
- develop a shortlist of stepped chute and stilling basin geometries (using 2-D CFD),
- evaluate four selected configurations (step heights and geometries) using the physical model,
- assess the hydraulic effects of the existing partially demolished spillway on weir performance,
- evaluate splash and wave run up to size training walls, and
- estimate pressures on the downstream face of weir for use in structural design,

The two stage, seven cycle labyrinth spillway is designed to pass 82,000 cfs without embankment overtopping. This flow (about 60 percent of the SDF) is roughly equal to the theoretical capacity of the existing gated spillway. To safely pass the SDF (143,000 cfs), the embankment will be armored with articulating concrete blocks (ACB). The ACB design also presented challenges due to the high tailwater and occurrence of a hydraulic jump on the downstream slope of the embankment.

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Introduction

Lake Townsend Dam is located on Reedy Fork Creek in Guilford County, North Carolina. Lake Townsend is a 1,635-acre water supply reservoir that serves as the City of Greensboro's primary water supply. Reservoir storage is 6,330 million gallons at normal pool and the drainage area at the dam is 105 square miles.

The dam consists of a concrete spillway flanked by earth embankments (Figure 1). The spillway is an ogee-shaped weir divided into nine, 25-ft wide bays and one 15-ft wide bay (Figure 2). Ten foot high vertical lift gates are located in each of the 25-ft wide bays and a skimmer gate is located in the 15-ft wide bay. A 200-ft wide excavated earthen emergency spillway is located through the north abutment (Figure 1). Table 1 summarizes pertinent elevations of the dam and spillways.

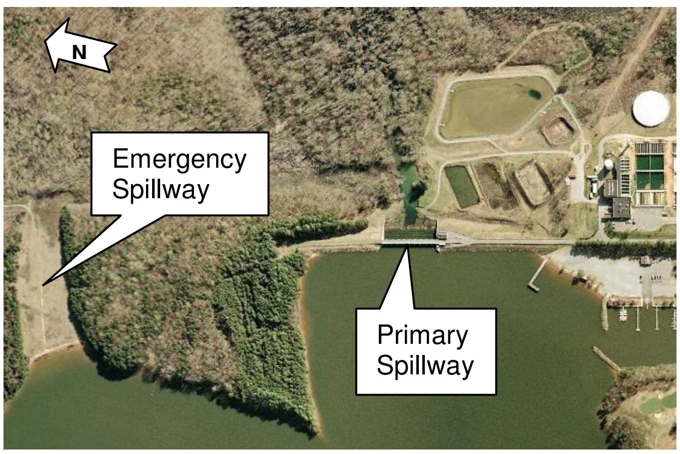


Figure 1 - Existing Project Features

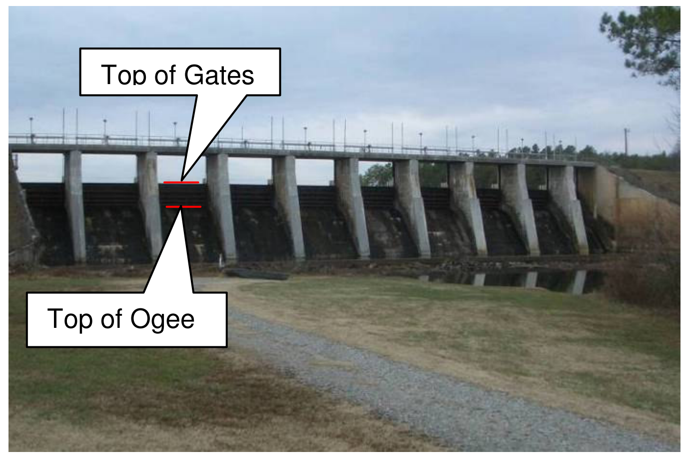


Figure 2 - Existing Spillway

Table 1
Lake Townsend Dam - Critical Elevations

Feature	Elevation (NAVD 88)
Crest of Ogee	705.5
Top of Gates (normal pool)	715.5
Emergency Spillway	718.5
Top of Dam (embankment crest)	725.5

Prior to 1980, major concrete elements of the dam, particularly the concrete spillway piers, began exhibiting cracking. An investigation several years later revealed that the cracking distress was caused by alkali-silica reactivity (ASR) in the hardened concrete of the spillway. A relatively high degree of cracking and spalling distress has progressively continued in the concrete piers, training walls and spillway mass concrete. Recent investigations have found that this concrete distress and deterioration will continue, rendering the structure unsafe. Figures 3 and 4 illustrate the cracking and damage from ASR.

Figure 3 – Pattern Cracking of Piers

Figure 4 – Cracking of Mass Concrete and Piers

In addition to the ASR damage, the dam does not meet current dam safety criteria in accordance with North Carolina Department of Environment and Natural Resources (NC DENR) dam safety requirements. According to NC DENR regulations, Lake Townsend Dam is classified as a Class C (high hazard), “Large” dam. Based on this classification, the spillway design flood (SDF) for is the ¾PMP.

As part of the hydrologic and hydraulic analysis for the dam, the possibility of designing the upgrading for Lake Townsend Dam to withstand failure of upstream dams was evaluated. Three smaller lakes (Lake Jeannette, Lake Brandt, and Lake Higgins) are located upstream of Lake Townsend (see Figure 5). Lake Brandt and Lake Higgins are both City-owned water supply reservoirs. Since no habitable structures are located between these dams and Lake Townsend, incorporating the failure of the upstream dams into the SDF for Lake Townsend Dam would eliminate the need to upgrade these dams to address inadequate spillway capacity. The City of Greensboro elected to consider the failure of the three upstream dams in development of the SDF for Lake Townsend. This will reduce the SDF for the two City-owned dams to their current capacity. This approach was reviewed with and approved by NC DENR.

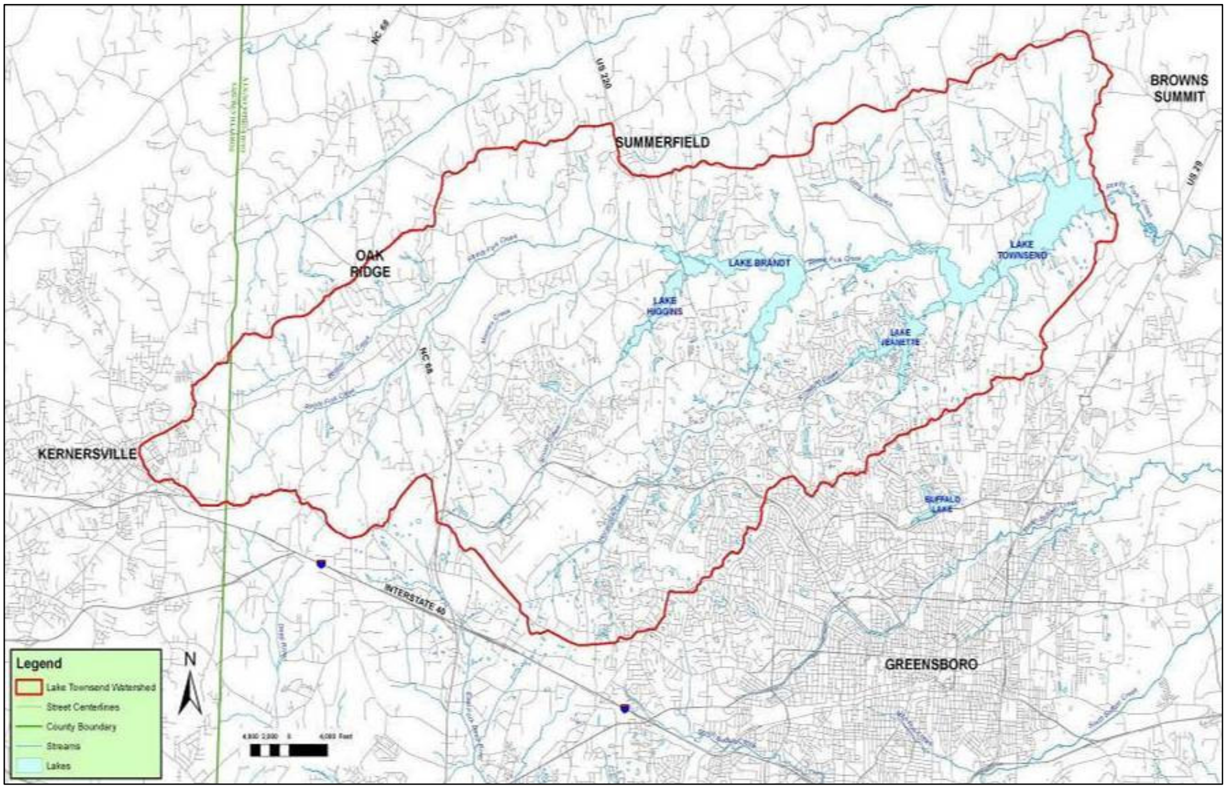


Figure 5 – Lake Townsend Watershed and Upstream Lakes

A labyrinth weir was selected for the replacement of the deteriorating spillway at Lake Townsend Dam based on its hydraulic efficiency and to avoid the need for frequent gate operation by City personnel. The labyrinth replacement spillway was designed to have similar hydraulic capacity as the existing gated structure for a reservoir level at top of dam. This flow is about 82,000 cfs, which corresponds to 60 percent of the PMP (assuming no failure of upstream dams).

The proposed labyrinth consists of a staged weir with two cycles at normal pool (EL 715.5) and five cycles at EL 716.5. This configuration will contain the normal flows to the portion of the spillway in line with the downstream channel. Estimates from stream gage data indicate that the high stage weir will likely flow one to five times per year. The proposed labyrinth geometry is presented in Table 2 and Figure 6.

Table 2
Labyrinth Geometry

Weir Height (P)	20 ft*
Cycle Width (W)	42 ft
Total Cycle Depth (B)	83.5 ft
Sidewall Angle (α)	11.4º
Weir thickness at crest	18 in

* Normal Pool Stage. High Stage Weir Height = 21 ft

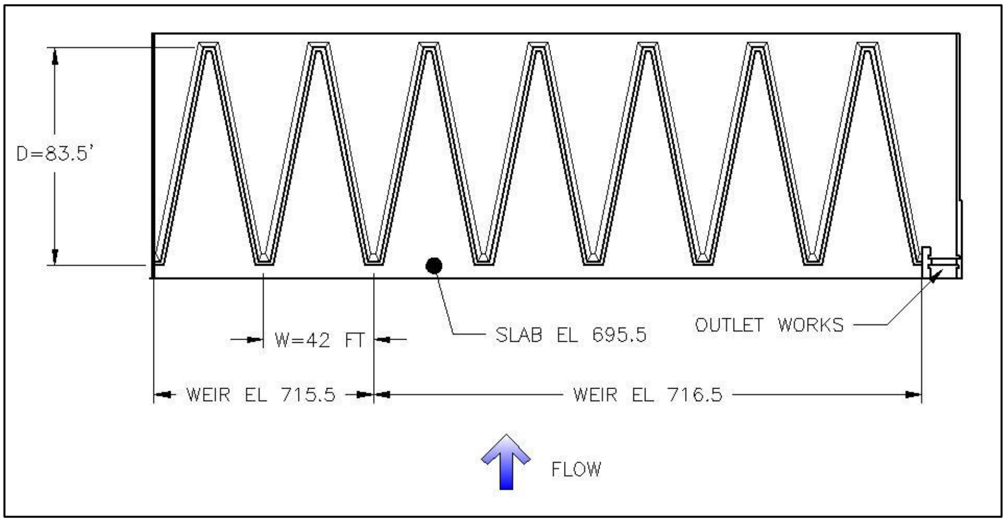


Figure 6 - Labyrinth Spillway Plan

Hydraulic Methods and Modeling

To evaluate the hydraulic behavior of the proposed spillway, the design team used a combination of empirical methods in conjunction with numerical and physical model studies.

Empirical Methods

In the United States, two of the most commonly applied theories for hydraulic modeling of labyrinth weirs are those by Lux and Hinchliff (1985) and Tullis et al. (1995), referred to herein as the Lux and Tullis methods. Both of these methods are summarized in *Hydraulic Design of Labyrinth Weirs* (Falvey, 2003).

To compare the two methods, the discharge coefficient for the Lux method for a given geometry was obtained and converted into the same terms of the Tullis discharge coefficient (C_d). The comparison of the methods for a labyrinth with a sidewall angle (α) of 12 degrees are presented in Figure 7, where W is the plan width of a single cycle, P is the weir height, H is the total upstream energy head, and L is the length along the labyrinth weir crest.
The results indicate that application of these two methods yield similar results; however, for this configuration, a higher discharge coefficient is obtained for low H/P ratios (<0.3) when using the Lux method.

Initial modeling of the labyrinth for Lake Townsend was performed by applying the Tullis method. For the sidewall angle of 11.4 degrees, the discharge coefficient was obtained by interpolating between the computed discharge coefficient for eight and twelve degree sidewall angles. For this modeling, the Tullis et al (1995) data for an eight degree wall angle was modified based on unpublished corrections provided by Utah State University. While the results of the application of this data are presented herein, the details of the research and results will be provided in a future publication. This modeling will be referred to herein as the *modified* Tullis method. The proposed labyrinth geometry has a sidewall angle of nearly twelve degrees; therefore, the results obtained using this method are not expected to be significantly different than if the Tullis method were applied.

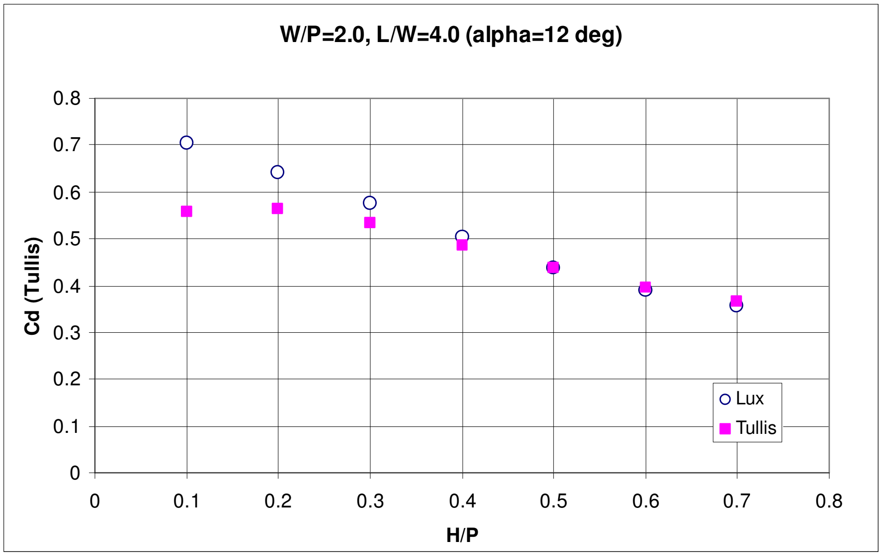


Figure 7 - Comparison of Lux and Tullis Methods

Physical Model Study

A 1.5-cycle labyrinth weir model was constructed at 1:21 scale in a 3-ft wide laboratory flume. The model includes the labyrinth weir and the downstream transition to the channel. The elevation difference between the maximum water surface elevation and the transition slab was approximately 55-ft prototype (2.6-ft model). A photo of the model is presented as Figure 8.

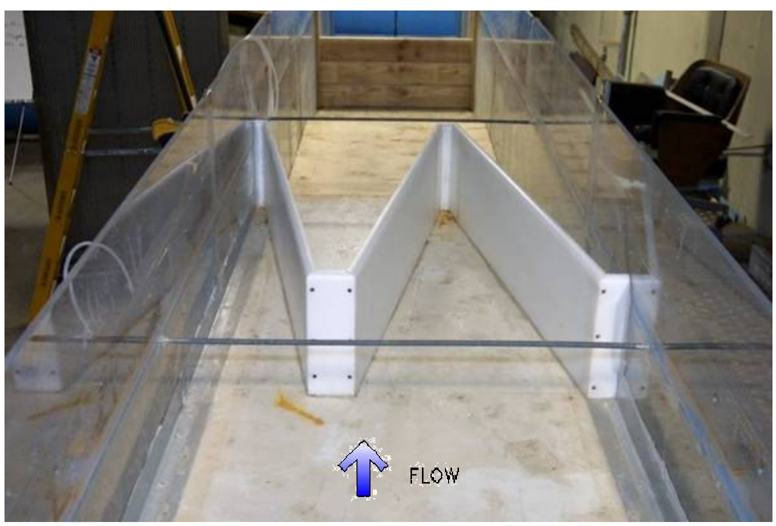


Figure 8 - Physical Model

The labyrinth weir was constructed based on the weir height of 20-ft, a wall thickness of 18-in, and a quarter-round crest shape with a radius of approximately two-thirds of the wall thickness. The labyrinth weir was fabricated from high density polyethylene (HDPE). The weir walls were approximately 0.86-in thick, 0.95-ft in height, and the total weir length was approximately 12.67-ft. The quarter round crest shape was machined using a standard 5/8-in rounded-over router bit. The sectional model was placed upon a platform to model various step/drop configurations downstream of the weir. The steps/drops were fabricated using plywood to facilitate quick structure modifications. Due to the height of the weir platform, the spillway approach was modified by placing a ramp upstream of the platform, from the flume floor to the platform, to minimize flow turbulence. A stop log structure downstream of the sectional model was used for tailwater control.

Baffles were placed at the upstream end of the flume to condition the flow approaching the model. A vertical grating was installed across the width of the flume and a floating baffle (sheet of plywood) was used to suppress surface wave activity. The approach flow in the flume was relatively uniform with the predominant flow direction normal to the labyrinth weir centerline.

Numerical Model

A three-dimensional solids model was developed in AutoCAD for use in numerical modeling using Flow-3D®, a commercially available Computational Fluid Dynamics (CFD) program. The model is shown in Figure 9.

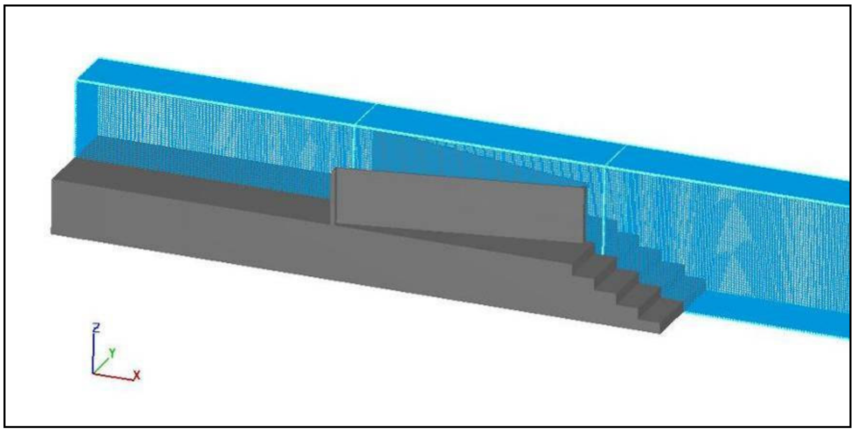


Figure 9 - CFD Model

Flow-3D® uses a numerical technique called the finite-volume method to solve the governing equations of fluid flow; more commonly referred to as the Reynolds Averaged Navier-Stokes (RANS) equations. To track the free water surface (air/water interface) as it moves in time and space, the program implements a sophisticated algorithm called the Volume-of-Fluid (VOF) method. The VOF method tracks the amount of fluid in each computational cell. Cells range from completely empty to completely full and the volume changes as flow moves in or out from neighboring cells. A similar algorithm called the Fractional-Area-to-Volume-of-Fluid (FAVOR) method is used to define the labyrinth weir and the spillway, defining cells where the flow is partially or completely restricted in the computational space. The two-equation Renormalized Group theory (RNG) model was implemented to model turbulence in the simulation.

The numerical simulations were completed by defining a computational domain around the flow area of interest. For the 3-D models, the flow domain included one cycle of the labyrinth weir structure and the downstream spillway. For the 2-D analysis, the flow domain primarily included only the downstream spillway and tailwater. The computational domain appears as a box around the area of interest that is discretized into a Cartesian grid of variable-sized hexahedral cells. The numerical solution to the RANS equations provides pressures at the center of each cell and velocity values at the cell faces, at each time step.

To model the flow, boundary conditions were established for each side of the computational box. The 3-D models were computed by using a fixed upstream headwater (HW) depth along with a fixed downstream tailwater depth depth. The headwater depth was defined as a stagnation boundary condition. For the 2-D models, the tailwater was fixed like the 3-D models. However, the upstream boundary was defined by a given flow rate. The flow rate boundary was established by defining a Froude number and using this Froude number to compute the upstream velocity and depth.

Discharge Rating and Results

Data obtained from empirical methods and the models was used to develop a discharge rating for the proposed spillway. This data was adjusted to account for tailwater and a rating curve was developed to perform reservoir routing for various hypothetical storm events, including the SDF.

Labyrinth Discharge Estimates

Figure 10 is a plot of the discharge coefficient data obtained from the physical and CFD models along with estimates using the *modified* Tullis method and theoretical data for a straight weir with a quarter round crest shape. This data was used to develop the equation relating the C_d to H/P.

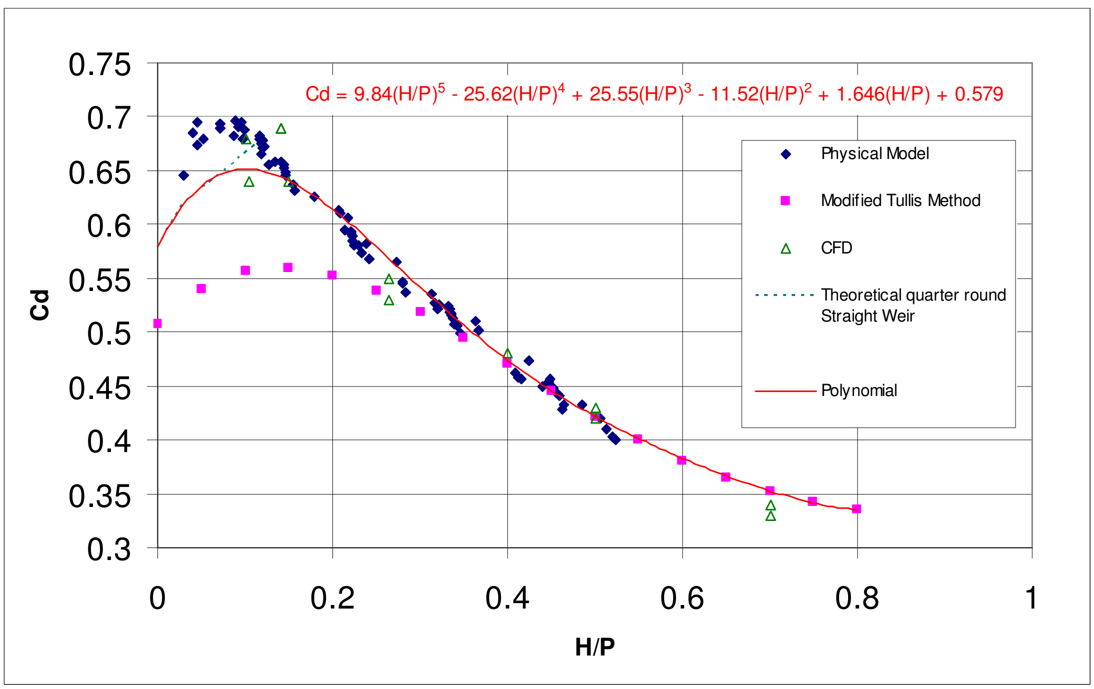


Figure 10 - Labyrinth Discharge Coefficient

The application of the modified Tullis method for this weir geometry appears to underestimate the discharge coefficient for low H/P values (<0.3). This finding supports the comparison of the Lux and Tullis methods as presented in Figure 7.

The equation shown in Figure 10 was used to develop a free discharge (no submergence) rating curve for the two stage labyrinth.

Tailwater and Submergence

The flow depth in Reedy Fork Creek downstream of the dam affects the discharge capacity, energy dissipation, and structural design of the proposed spillway. A hydraulic model of Reedy Fork downstream of Lake Townsend was developed using the HEC-RAS computer program. Stream and floodplain geometry and roughness parameters were developed from HEC-2 data files developed for FEMA flood mapping. The computed tailwater rating curve is presented as Figure 11.

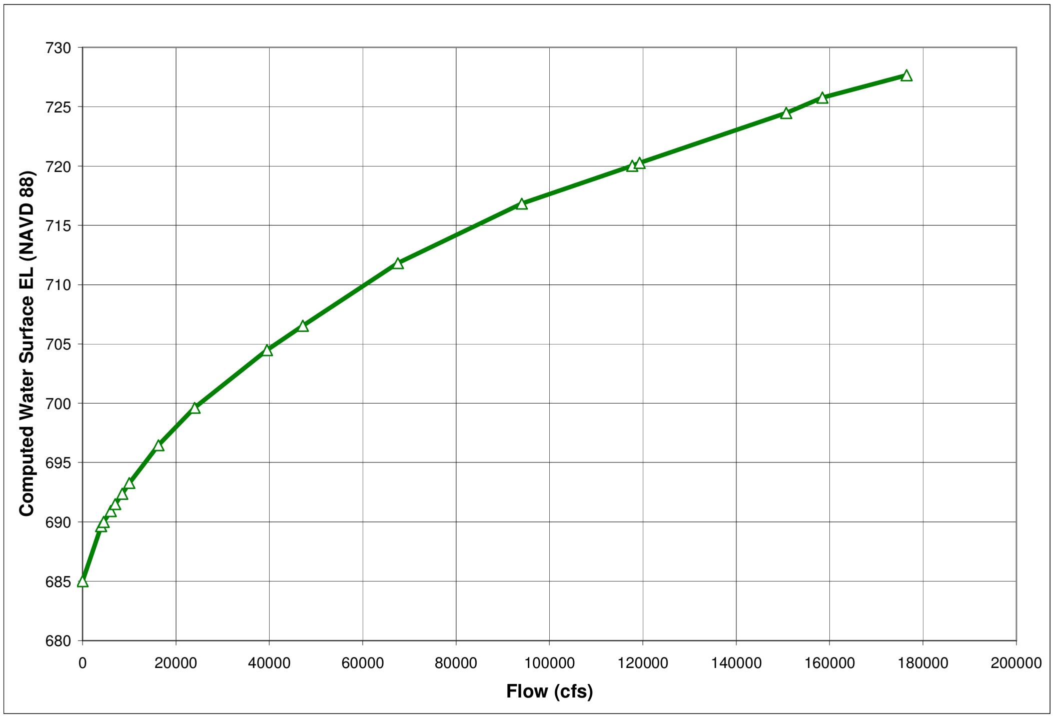


Figure 11 - Tailwater Rating Curve

Discharge capacity of the labyrinth is reduced when the tailwater exceeds the crest of the weir (EL 715.5). Based on the tailwater data and discharge rating curve, this occurs when the flow through the dam exceeds about 85,000. For the proposed spillway, this corresponds to a headwater elevation is just above the top of dam (EL 725.5). Tullis et al (2007) studied submergence of labyrinth weirs and published data for labyrinths as it compares to Villemonte's (1947) relationship for submerged linear weirs. The data, presented in Figure 12, relates the ratio of tailwater surcharge head (h_d) and reservoir head (h) to the ratio of tailwater influenced discharge (Q_s) and free weir discharge (Q_1).

The plotted trend line and equation for labyrinth weirs shown in Figure 12 was developed to generally fit the plotted data by Tullis et al (2007). This relationship was used to reduce the estimated discharge for the proposed spillway for given headwater and tailwater levels. Developing the submerged discharge rating curves is an iterative process since the tailwater, level of submergence, and outflow cannot be independently computed.

The size of the flume used for the physical model study limited the total upstream water surface elevation to about El 727.3 (prototype); therefore significant modeling of submerged conditions was not performed. For the tests performed, h_d/h values ranging from .07 to .27 produced estimated Q_s/Q_1 values ranging from 0.99 to 0.97 which corresponds reasonably with the equation and data contained in Figure 12.

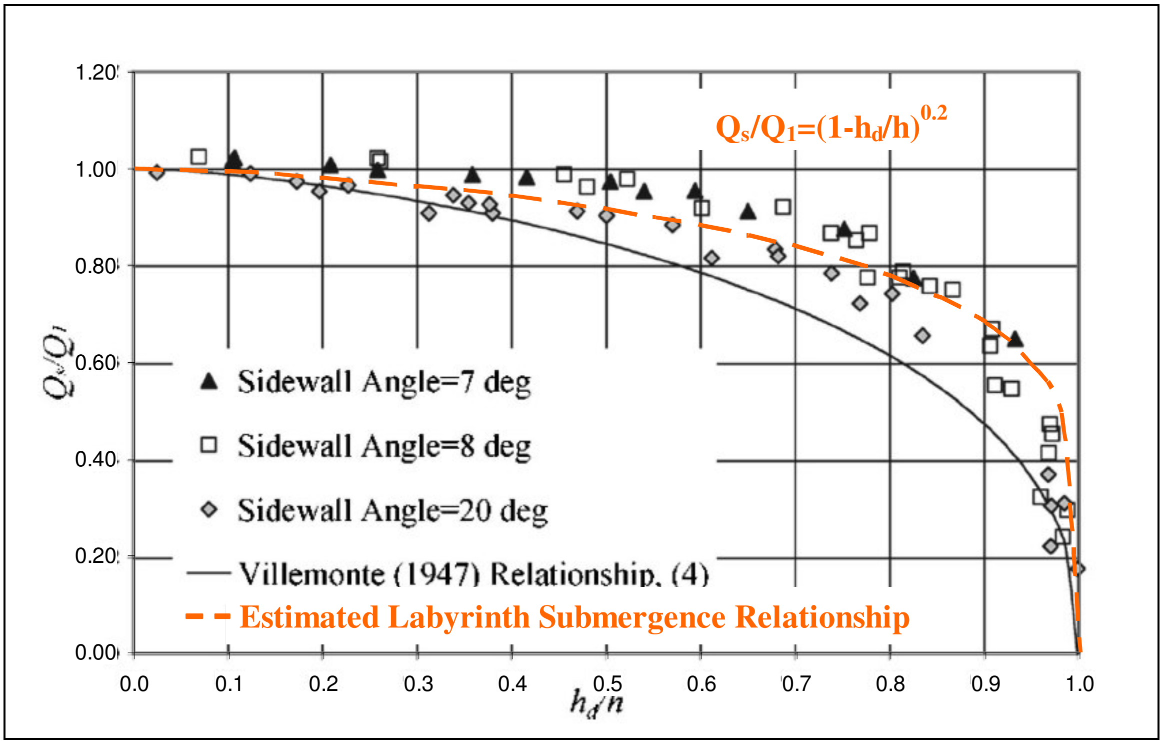


Figure 12 - Submergence Estimates based on Tullis (2007)

Rating Curve and Reservoir Routing

The computed rating curve (with and without submergence effects) for the proposed labyrinth spillway is presented in Figure 13.

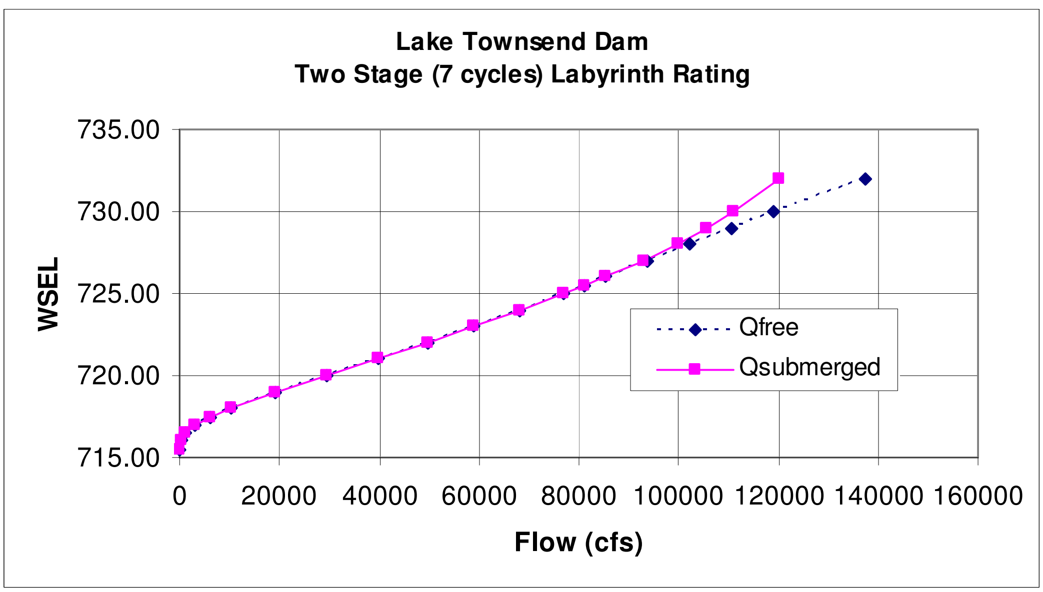


Figure 13 - Discharge Ratings

An earthen spillway stability evaluation was performed using the SITES computer program and indicated that erosional failure of the existing emergency spillway would be likely for the flows anticipacted during the SDF; therefore, a dike will be constructed across the emergency spillway. The top of the embankment at Lake Townsend Dam is EL 725.5;

therefore, the dam will overtop for flows exceeding about 82,000 cfs. The embankment will be protected against erosional failure due to overtopping by armoring the crest and downstream slope with articulating concrete blocks (ACB). The reservoir routing considered the reduction in capacity by closing off the emergency spillway and included flow over the armored embankment. An overtopping length of 900 ft and a discharge coefficient of 2.7 were used in the analysis. The computed headwater and tailwater rating curves are presented in Figure 14. Flow over the embankment is included in the total rating curve.

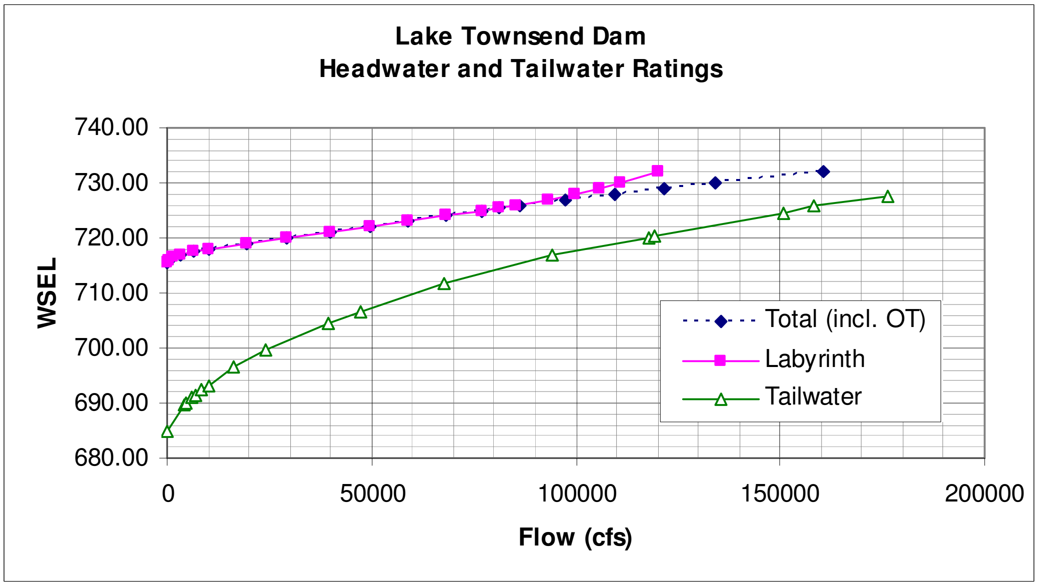


Figure 14 - Discharge Ratings

The inflow hydrograph for the ¾ PMP storm, including flow resulting from the modeled failure of the upstream dams, was routed through the proposed replacement dam and spillway using the the HEC-1 computer model and the spillway rating curve shown in Figure 14. Results are presented in Table 3.

Table 3
Reservoir Routing - ¾ PMP

Inflow to Lake Townsend	213,000 cfs
Outflow from Lake Townsend	143,200 cfs
Peak Stage (NAVD88)	EL 730.7
Overtopping Depth	5.2 ft

The computed capacity of the proposed labyrinth weir for the reservoir at top of dam (EL 725.5) is about 82,000, which corresponds to approximately equal to 60 percent of the PMP. This is similar to the capacity of the existing gated spillway, assuming the gates can be completely opened.

Energy Dissipation

A stepped chute or series of drops is proposed downstream of the labyrinth weir. This configuration is believed to be more cost effective to construct than a sloping chute and traditional energy dissipation structure. The steps provide energy dissipation, particularly for lower discharges, limiting the required size and complexity of the stilling basin. For higher flows, the tailwater submerges the flow downstream of the weir, reducing the required energy dissipation. An initial screening of numerous configurations was performed using the 2-D CFD model. Based on the initial CFD screening and project constraints, four configurations were evaluated in both the CFD and physical model, as presented in Figure 15. CFD results for two configurations at various flow rates are presented in Figures 16 and 17.

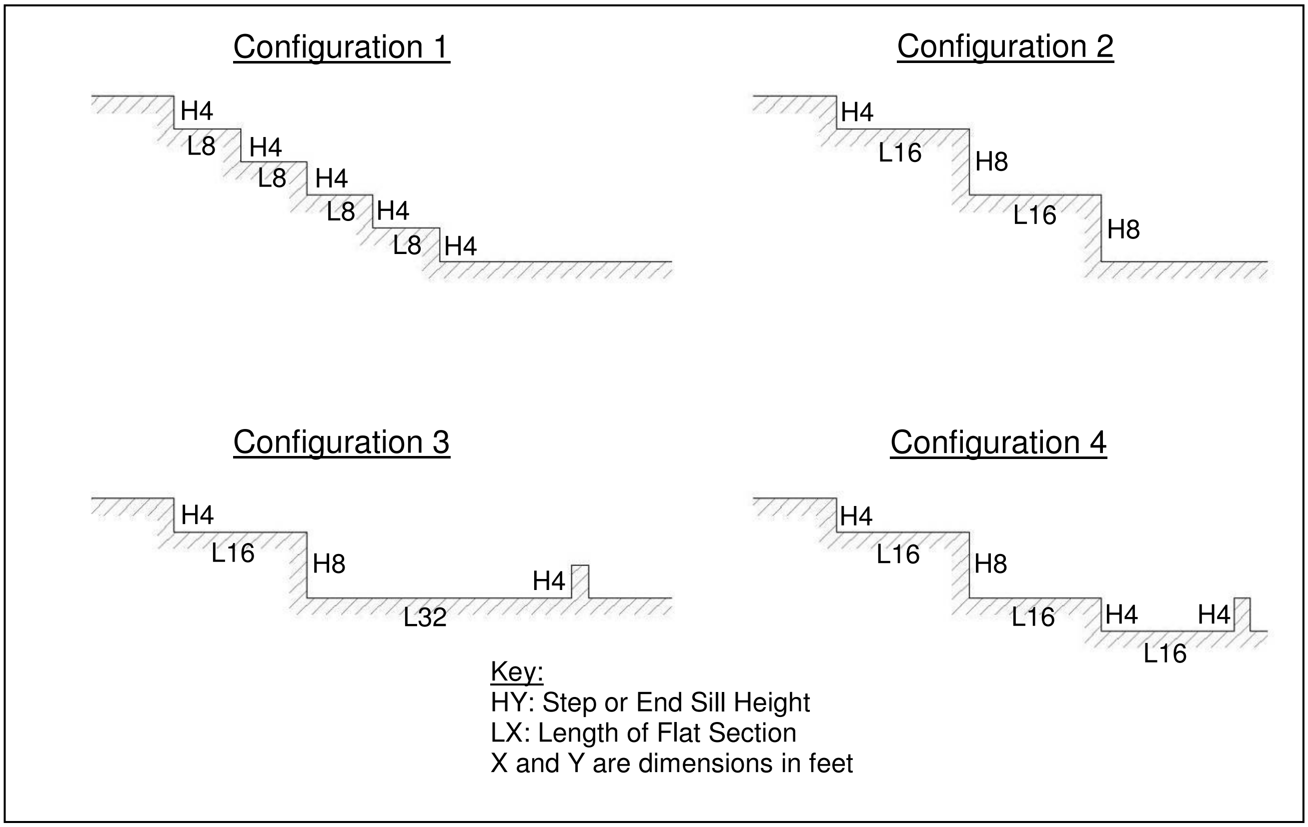
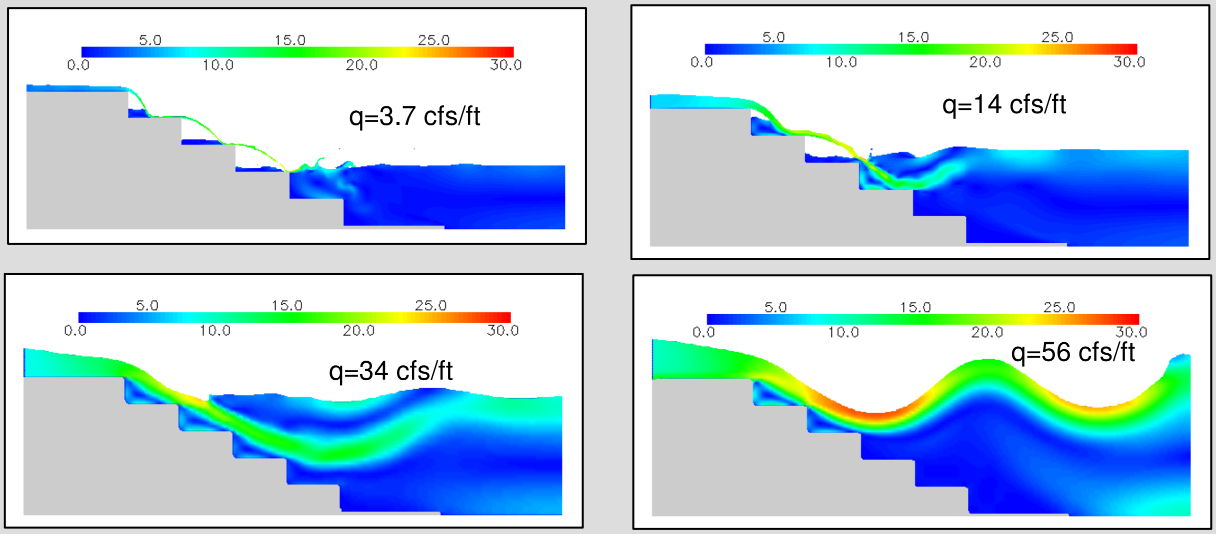
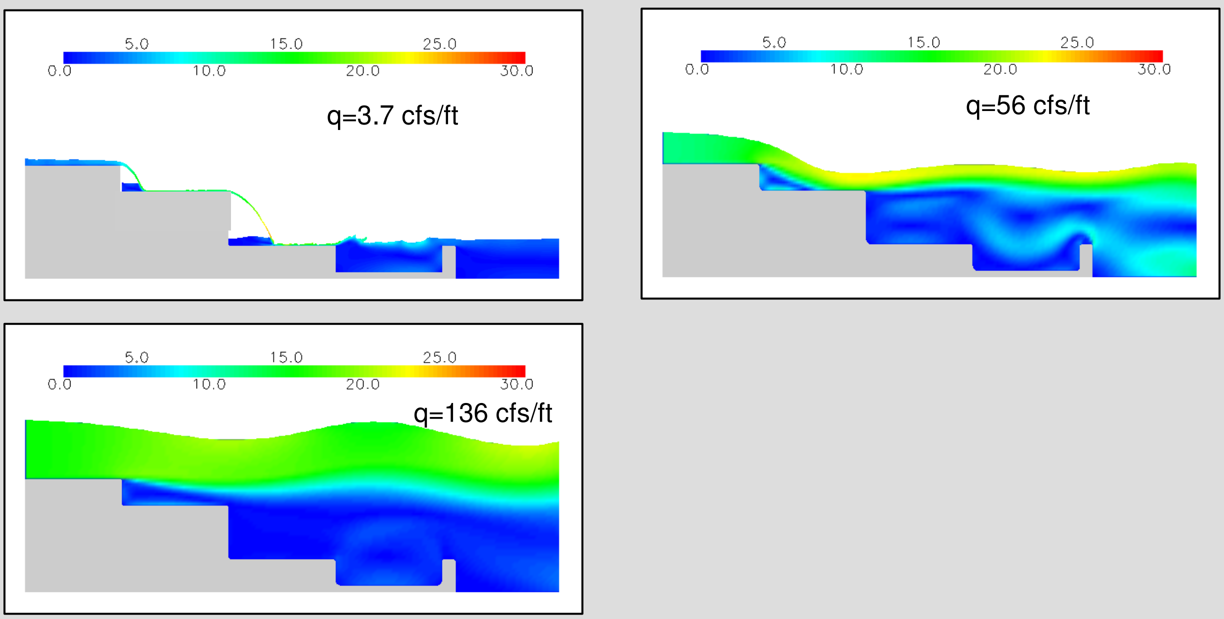


Figure 15 - Modeled Step Configurations



Scales represent velocities in feet per second

Figure 16 - 2-D CFD for Configuration #1



Scales represent velocities in feet per second

Figure 17 - 2-D CFD for Configuration #4

For Configuration 1 and a unit discharge of 56 cfs/ft, a standing wave appears to form downstream of the spillway. This condition is not as significant for Configuration 4. The results for these higher discharges may not represent true conditions since the 2-D model does not consider the converging, turbulent flow downstream of the labyrinth.

The same four configurations were modeled in the flume with the 1.5 cycle labyrinth model. Photos from the physical model are presented in Figures 18 and 19.

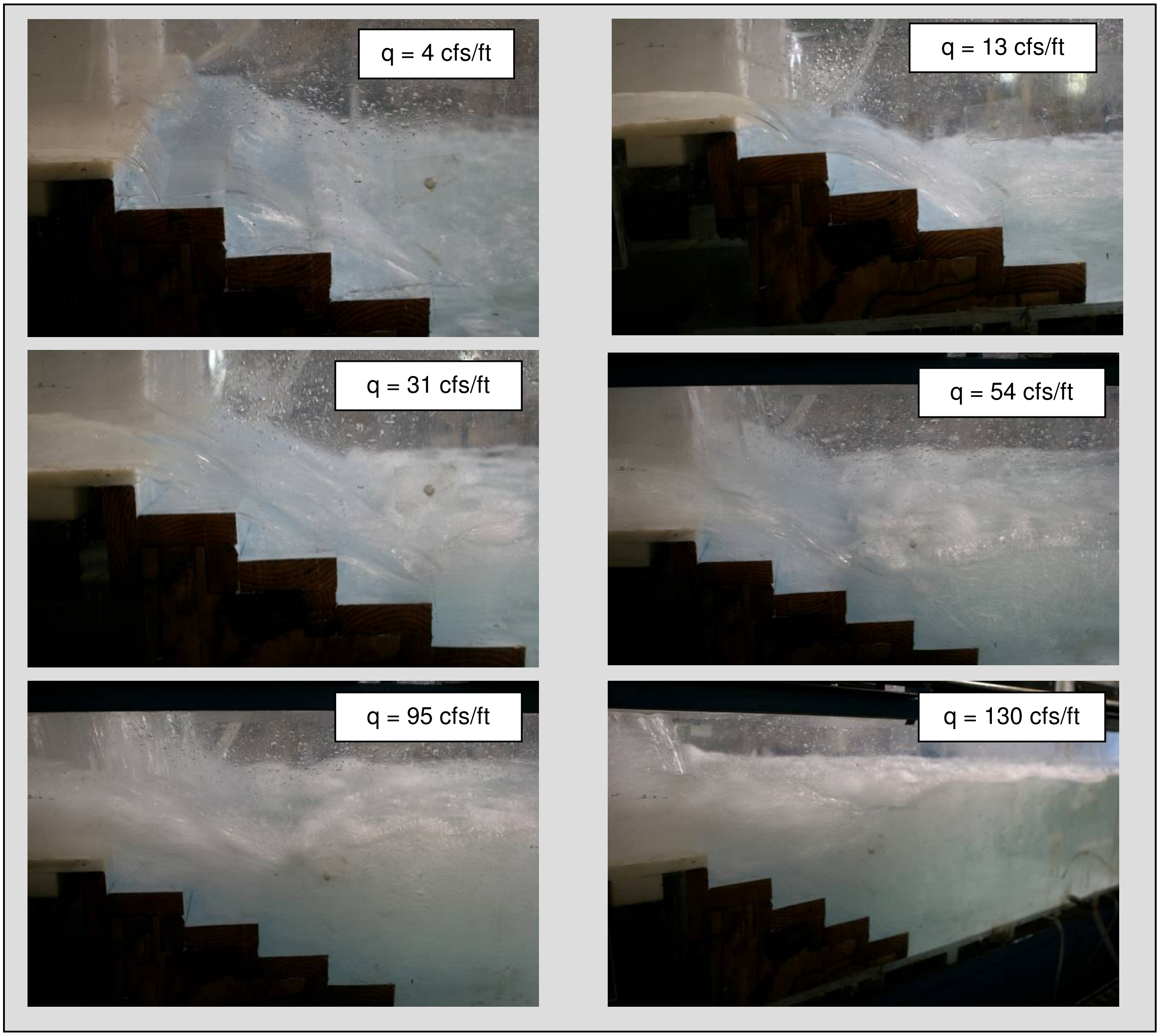


Figure 18 - Physical Model Study for Configuration #1

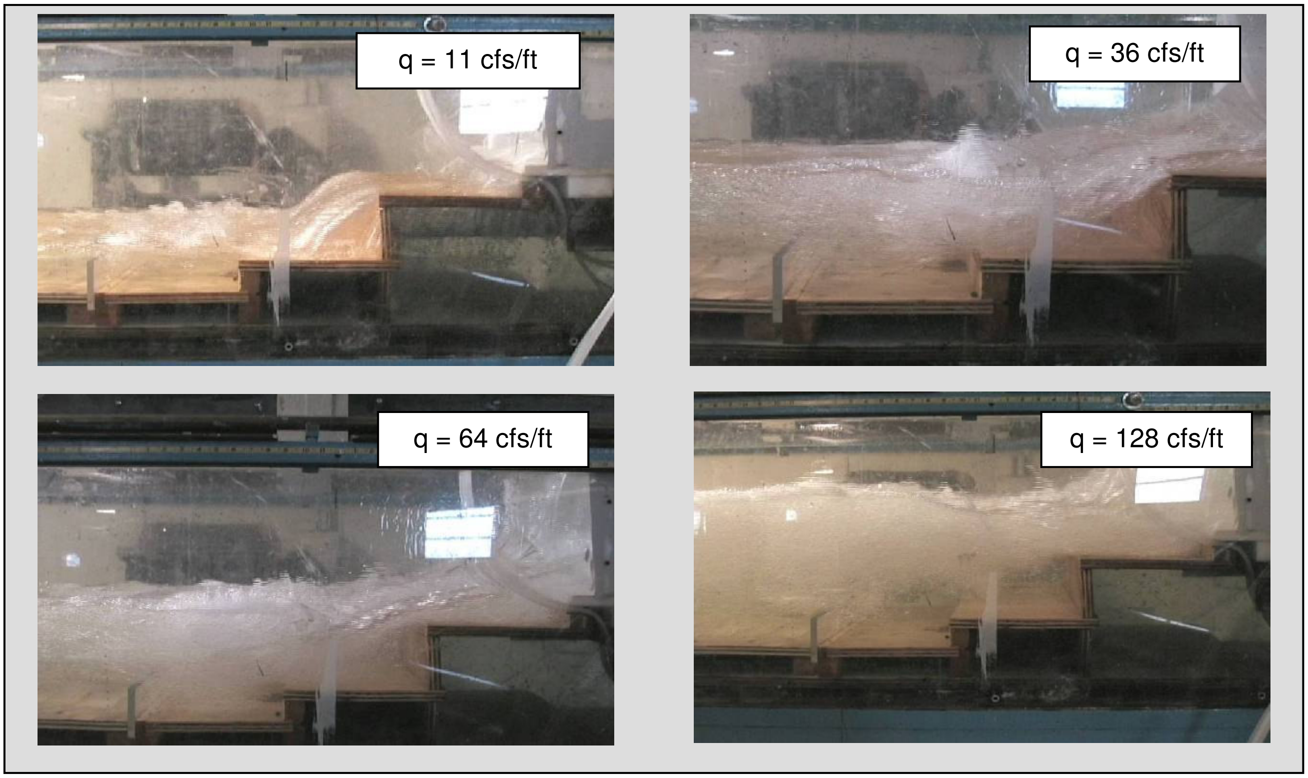


Figure 19 - Physical Model Study for Configuration #4

The flow behavior observed in the physical modeled matched closely with the 2-D CFD results, particularly for low flows. For unit discharges (q) less than about 60 cfs/ft, all step configurations evaluated appear to be relatively effective at dissipating energy; however, Configurations 2 through 4 appeared to be more effective with increasing flows. Both the CFD and physical models showed that for a unit discharge of about 30 cfs, high velocity flows plunged deeper into the tailwater for Configuration 1 than for the other configurations. For unit discharges above 60 cfs, Configurations 2 through 4 also provided more effective energy dissipation that Configuration 1. For very large discharges (q greater than about 80 cfs/ft) the tailwater levels control flow conditions downstream of the labyrinth. Neither highly turbulent nor high velocity flows were observed along the ground surface downstream of the structure, indicating that significant erosion is not likely.

Based on the results of the CFD and physical modeling, applicability to site conditions, and anticipated construction complexity and cost, Configurations 3 and 4 were selected for the new spillway at Lake Townsend Dam. Configuration 3 will be used downstream of the normal flow weir and in line with the creek and Configuration 4 will be used for the remaining portion of the spillway (downstream of the high flow portion of the labyrinth weir).

Other Modeling

In addition to the modeling to estimate labyrinth discharge and to evaluate the energy dissipation characteristics of various configurations downstream of the labyrinth, modeling was performed to:

- assess the impacts of the existing ogee spillway on spillway discharge,
- evaluate splash and wave run up to size training walls, and

- estimate the pressures on the downstream face of the labyrinth weir walls

Effect of Existing Spillway on Discharge

The proposed labyrinth spillway will be located about 110 ft downstream of the existing ogee spillway. Since the existing spillway will be used as the coffer dam for construction of the new structure and the reservoir cannot be lowered, demolition of the existing spillway will be performed underwater. Limiting the required amount of spillway demolition will significantly reduce the project cost. A 2-D CFD model was used to evaluate the effect of the removing only the gates and superstructure. The existing ogee crest is 10 ft below normal pool (EL 715.5). Results of the CFD modeling assuming the existing structure is left in place and completely removed are presented in Figure 20.

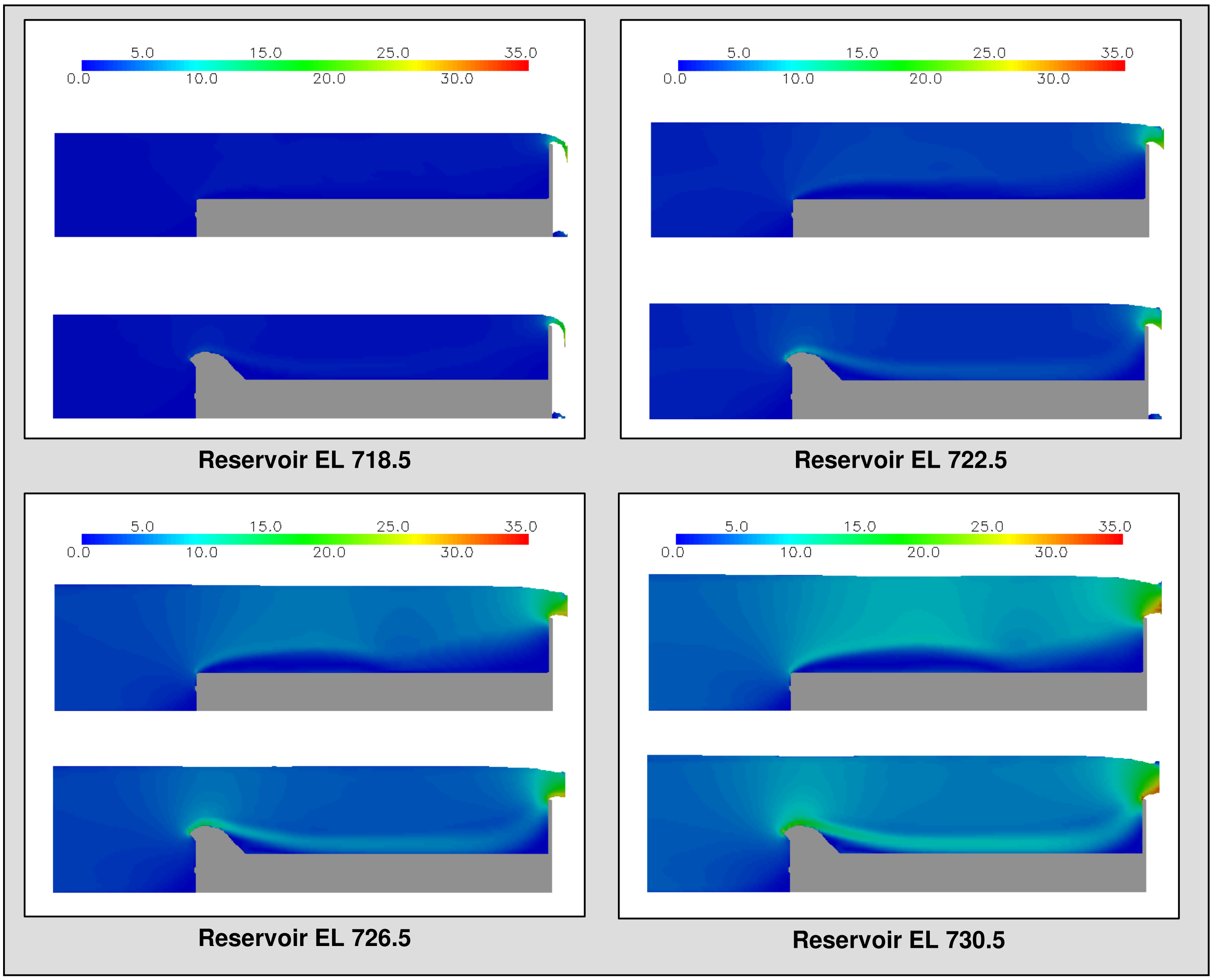


Figure 20 - 2-D CFD Modeling of Existing Spillway Effects on Flow

For reservoir elevations less than about EL 725.5 (top of dam), the 2-D CFD model indicates that the existing ogee crest has little to no effect on flow conditions upstream of the proposed spillway. For higher reservoir elevations, the existing spillway affected flow patterns;

however, discharge was not measurably reduced. Based on these results, the existing ogee will be left in place after construction of the labyrinth.

Water Surface Profiles

The water surface along the flume walls of the physical model was measured and plotted. The measured values were used in setting the top elevation of the spillway training walls to prevent flow over these walls during large floods.

Pressure Estimates

Pressures on the downstream face of the labyrinth for various discharges and corresponding tailwater elevations were obtained from the physical model. Pressure tap locations are shown in Figure 21 and the computed pressures along with the tailwater levels downstream of the structure are plotted in Figure 22.

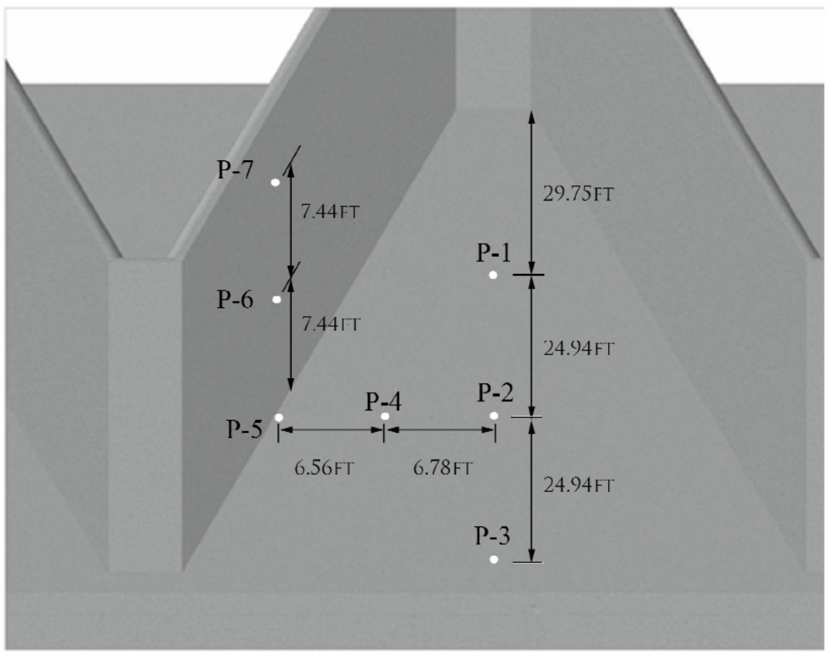


Figure 21 - Pressure Tap Locations

The results show that the pressures increase from downstream to upstream through the center of the labyrinth cycle. Near the downstream end of the labyrinth, the pressures are greater in the center of the labyrinth cycle than at the walls, likely due to the “mounding” of flow at this location. The measured pressures are generally consistent with the observations of the water surface in the model. The photos in Figure 23 show the water surface conditions downstream of the weir. The measures pressures were generally greater than the downstream tailwater, indicating that the use of a pressure equal to or slightly less than the tailwater elevation would be appropriate for structural design of the weir.

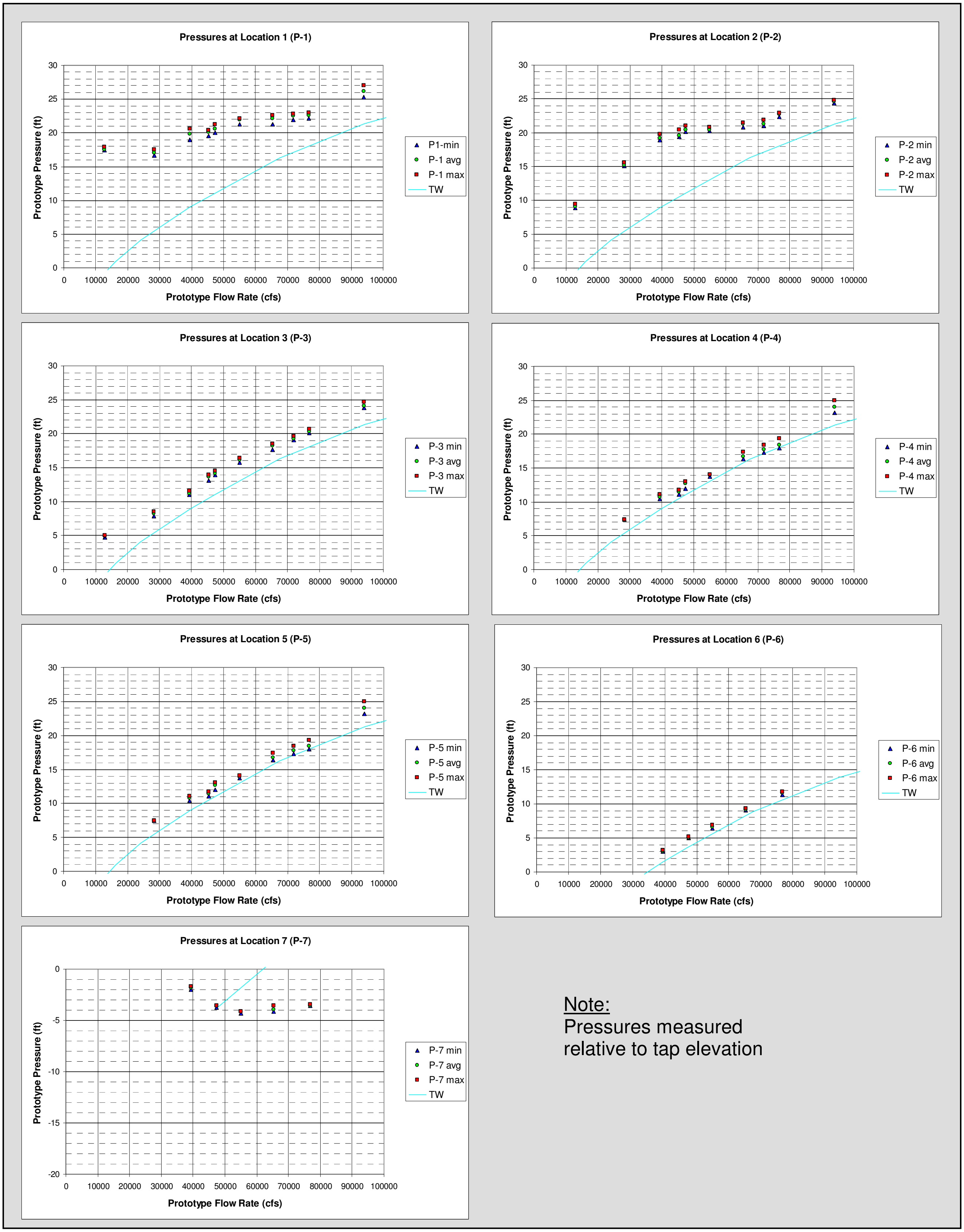


Figure 22 - Pressure Tap Data

16,000 cfs (Prototype – 7 cycles) | 61,000 cfs (Prototype – 7 cycles)

Figure 23 – Physical Model Study – Flow Downstream of Weir

Articulating Concrete Block (ACB) Design

As previously noted, the earth embankment will overtop for storms greater than about 60 percent of the PMP. The computed depth of overtopping for the SDF is 5.2 ft; therefore, the embankment needs to be protected from erosive failure due to overtopping flow.

Articulating concrete block (ACB) systems have been used on numerous dams to protect the embankment from failure due to overtopping. Model testing has been performed on these systems and methods for evaluating the stability of the blocks have been developed based on the test data.

For the design of Lake Townsend Dam, the stability of the ACB system was evaluated using methods presented in *Articulating Concrete Block Revetment Design – Factor of Safety Method* (NCMA TEK 11-12, 2002). Typically, the velocity and depth on the downstream slope is estimated assuming a steady flow condition (normal depth) is reached; however, the tailwater at Lake Townsend Dam would submerge flow on the downstream slope during the extreme floods that would cause overtopping. Since the steady flow condition was not considered appropriate for estimating depths and velocities, flow over the embankment was modeled using the HEC-RAS computer program. Table 4 presents the overtopping depth, headwater and tailwater elevations, and maximum estimate velocities using the normal depth calculation and HEC-RAS. The downstream slope of the embankment is 2.5H:1V, which corresponds to a normal depth energy grade of 0.4 ft/ft.

The results shown in Table 4 illustrate that steady flow conditions are not reached due to the high tailwater, particularly for larger overtopping depths. Using the traditional, normal depth calculation for stability analysis would be overly conservative, therefore, the flow conditions representing the maximum velocity and energy grade obtained from HEC-RAS were used in the ACB stability evaluation. Stability analyses were performed for several overtopping depths and block sizes and a safety factor of 2.0 was obtained for the selected ACB system, which consists of cable tied, open cell, six-inch-thick, tapered blocks.

Table 4 – Overtopping Flow Conditions

	Overtopping Depth		
	1 ft	**3 ft**	**5 ft**
Headwater	EL 726.5	EL 728.5	EL 730.5
Tailwater	EL 716.2	EL 719.7	EL 723.6
Max. Velocity (Normal Depth)	12 fps	23 fps	27 fps
Max. Velocity (HEC-RAS)	12 fps	20 fps	24 fps
Flow Depth (Normal Depth)	0.2 ft	0.6 ft	0.8 ft
Flow Depth (HEC-RAS)	0.2 ft	0.7 ft	1.3 ft
Energy Grade (HEC-RAS)	0.40 ft/ft	0.28 ft/ft	0.16 ft/ft

Summary and Conclusions

The existing concrete gated spillway at the City of Greensboro's Lake Townsend Dam requires replacement due to deterioration of the concrete from ASR and the because the spillway has inadequate capacity to pass the state-mandated spillway design flood (SDF). A labyrinth weir was selected as the replacement spillway based on its hydraulic efficiency and because gate operations are not necessary to pass storm events. The new spillway and embankment will be located just downstream of the existing structure (Figure 24). The replacement spillway will have capacity to pass about 60 percent of the PMP, which is comparable to the theoretical capacity of the existing gated structure. To safely pass the SDF, which is the ¾ PMP, the new embankments will be armored with articulating concrete block (ACB). The design of the new structure also incorporates the ability to accommodate the failure of three upstream dams during the SDF, allowing the City to avoid spillway capacity upgrades to these dams.

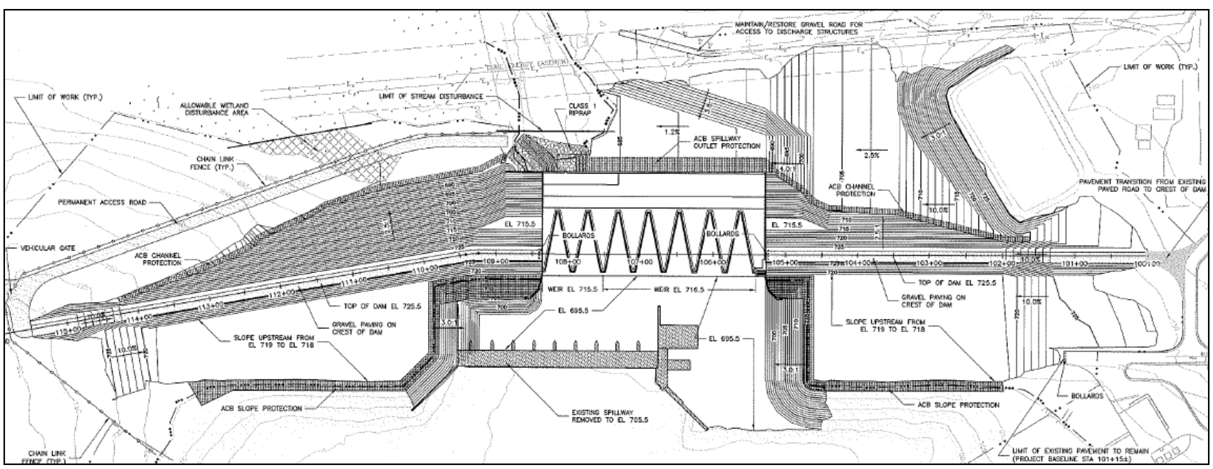


Figure 24 – Final Project Layout

To model the labyrinth weir hydraulics, a combination of empirical methods, a physical model study, and numerical modeling were performed. This modeling was performed to estimate the labyrinth discharge and qualitatively evaluate energy dissipation of the proposed stepped chute downstream of the weir for various flow conditions. This stepped chute provides energy dissipation and is believed to be more cost effective to construct than a traditional chute and energy dissipator. The application and comparison of empirical methods and the results of the modeling resulted in the following findings:

- The Lux and Tullis methods were compared for a labyrinth with the approximate geometry of the proposed spillway. The results indicate that these methods yield similar results; however, the Tullis method may underestimate discharge for low H/P ratios (less than about 0.3).
- The physical and numerical models and the *modified* Tullis method were used to develop a discharge relationship for the proposed spillway. This relationship also showed that the Tullis method may underestimate discharge for low H/P ratios.
- The tailwater downstream of the proposed spillway will submerge the labyrinth weir during high flow conditions. A relationship for computing submerged flow for labyrinths was developed based on Tullis et al (2007).
- 2-D CFD modeling was performed to evaluate and develop a shortlist of step geometries downstream of the labyrinth. Four configurations were selected for further evaluation using 2-D CFD and the physical model. The models yielded similar results, particularly for lower flows and a configuration was selected based on the physical model study results.

In addition to discharge estimates and evaluation of energy dissipation, the following modeling was performed.

- 2-D CFD was used to evaluate the impacts of the existing spillway on the proposed labyrinth weir. The findings indicate that if the structure (minus the gates and piers) is left in place, it will not adversely impact labyrinth performance. Therefore, costly underwater demolition of the existing mass concrete structure will not be required.
- The physical model was used to estimate splash and wave run up on the spillway sidewalls for setting the top elevation.
- Pressures on the slab and walls downstream of the labyrinth control section were obtained at several locations in the physical model. These pressures generally matched those expected based on the water surface profile observed in the model. Measured pressures were generally higher than the downstream tailwater and are considered appropriate for use the structural design of the spillway.

The earth embankment will be armored with ACB to prevent failure from overtopping. The estimated maximum overtopping depth is about 5 ft. Since tailwater is relatively high on the embankment for floods where overtopping would occur, the typical approach for estimating flow depth and velocity on the downstream slope was considered overly conservative. These flow conditions were estimated using the HEC-RAS computer program and used in the stability evaluation for sizing the ACB.

The application of hydraulic theory along with the physical and numerical modeling were essential tools to the Lake Townsend Dam design team. These tools provided substantial project benefit, including reduced estimated construction cost, reliable discharge estimates, and functional energy dissipation for a wide range of flows.

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