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16. Abstract
Safety end treatments are installed on new and existing culverts to protect vehicular traffic if the vehicle should leave the roadway. Backwater effects associated with end treatment designs used by the Texas Department of Transportation have not been measured previously. Such measurements are provided through the hydraulic modeling program reported upon. Backwater effects are represented through minor loss coefficients (K_m), which are found to be small with a representative value $K_m = 0.021$ for all culvert end configurations investigated. Performance curves and coefficients are also presented for these different end configurations, which supplement results presented in 0-2109-1.

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Evaluation of Hydraulic Effects of Culvert Safety End Treatments

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1. INTRODUCTION

1.1 BACKGROUND AND SIGNIFICANCE OF WORK

This work is part of Texas Department of Transportation (TxDOT) research project number 0-2109, "Evaluation of Effects of Channel Improvements, Especially Channel Transitions, on Culverts and Bridges," conducted by a team of researchers at the Center for Research in Water Resources (CRWR), which is part of the University of Texas at Austin (UT Austin). The results from earlier work are reported in "Hydraulics of Channel Expansions Leading to Low-Head Culverts" by Charbeneau et al. (2002). The purpose of this study is to evaluate the impact of safety end treatments (SETs) on the hydraulic performance of culverts.

SETs have been proposed for extensive use on new culvert projects as well as retrofitting existing culvert projects by TxDOT. Previous studies conducted by other investigators evaluate various aspects of the effects of SETs on the hydraulic performance of culverts, but there is no specific guidance for design engineers to quantify the effects of SETs in the hydraulic design process for retrofitting culverts with SETs or applying SETs to new culverts. This study attempts to address this issue. This chapter provides an overview of the study, introduces relevant terminology and concepts, and discusses specific goals for this research.

A culvert is a structure that conveys surface water through a roadway embankment or away from the highway right-of-way. Culvert design involves both hydraulic and structural aspects. A culvert must carry construction, highway traffic, and earthen loads and allow natural stream flows to pass beneath the road to ensure adequate drainage and preserve the structural integrity of the road. Culverts have numerous cross sectional shapes including circular, box (rectangular), elliptical, pipe-arch, and arch. This research is concerned with box culverts having rectangular cross sections.

Performance equations express the relationship between culvert headwater and discharge under conditions of inlet control, and may be used to predict the cross sectional area needed to pass flows expected to result from storm events of specified recurrence intervals (i.e., the 10-, 25-, or 50-year storm). Headwater is the upstream specific energy as measured relative to the elevation of the culvert invert. Inlet control for a culvert is when the flow capacity is controlled at the entrance by the depth of headwater, entrance geometry, and barrel shape. Outlet control for a culvert is when the hydraulic performance is determined by inlet conditions, barrel length and roughness, and tailwater depth. Culverts are usually designed to operate with the inlet submerged if conditions permit, allowing for increased discharge capacity.

SETs are designed and installed at inlets and outlets of culverts to reduce potential impacts due to vehicular collision with these structures. The term SET, according to TxDOT, consists of a number of particular features including sloping ends, clear zone slopes, concrete slope paving, metal appurtenances, and safety pipe runners. For the purposes of this report, SET refers to the safety pipe runners. SETs must be designed with minimal size to limit interference with water flow while maintaining sufficient strength to support a vehicle. Commonly, there are two types of safety end treatments: pipe safety grates and bar safety grates. This study focuses on pipe safety grates (Figure 1.1). It is necessary to understand the impact of SETs on culvert hydraulics to ensure they do not affect the functionality of the culvert.

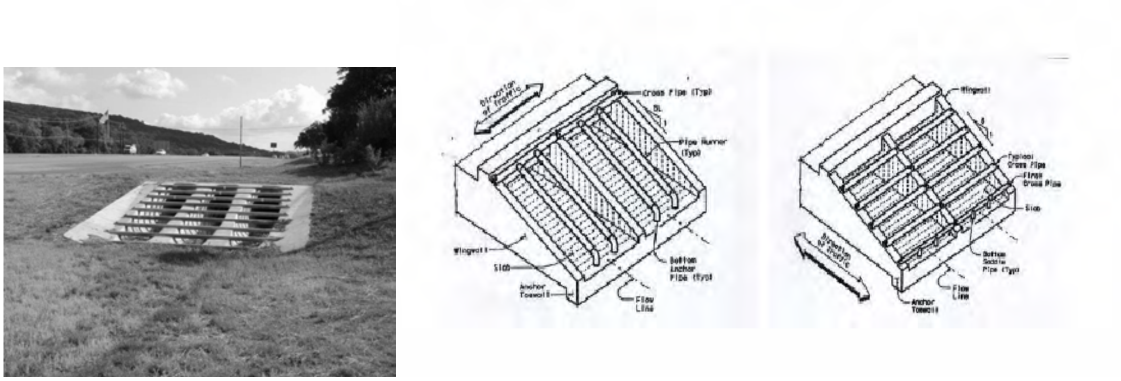


Figure 1.1 Pipe grate safety end treatment for a culvert

SETs function as flow barriers and can affect the hydraulic performance of the culvert in two main ways. First, the “backwater” effect from the installation of SETs may cause an increase in the upstream headwater depth and entrance head losses. Second, SET installation may cause clogging. Both of these effects may lead to flooding of upstream properties because the influence of SETs on headwater depth is not usually accounted for in the design procedures for culverts.

1.2 STUDY OBJECTIVES

The objective of this study is to evaluate the hydraulic effects of SETs on culverts through physical modeling and to provide TxDOT with guidance on the influence of SETs in the hydraulic design of culverts. Tabular values for minor loss coefficients will be provided to TxDOT to fulfill this objective. For the scope of this research, the investigations include the following single-barrel culvert models:

1. Box culvert with vertical headwall and parallel wingwalls at 0-degree skew,
2. Box culvert with mitered headwall (3:1 slope) and parallel wingwalls at 0-degree skew,
3. Box culvert with mitered headwall (3:1 slope) and parallel wingwalls at 30-degree skew,
4. Box culvert with parallel, mitered wingwalls (6:1 slope) and vertical curb headwall at 0-degree skew, and
5. Box culvert with 15-degree, mitered wingwalls (6:1 slope) and vertical curb headwall at 0-degree skew.

These model setups are described more fully in Chapter Three.

The specific objectives of this research are:

1. To study the nature of water level difference upstream of the culvert due to SET presence.
2. To evaluate and compare the headwater-discharge relationships (performance curves) for different end configurations with culverts operating under inlet control, and to compare them to performance curves developed from earlier research reported in “Hydraulics of Channel Expansions Leading to Low-Head Culverts” (Charbeneau et al. 2002).

3. To provide minor loss coefficients due to the presence of SETs for different end configurations, that may be used in design procedures.

Physical models of a single-barrel box culvert with different end configurations were constructed to collect data for conditions with and without SETs installed at the inlet end of the culvert model. Both unsubmerged and submerged culvert inlet conditions were considered. The collected data was utilized to evaluate parameter difference (water level, specific energy, discharge, etc.) and to calculate minor loss coefficients.

1.3 OVERVIEW

Following this introduction, Chapter Two reviews literature relevant to this study. Chapter Three discusses the methodology used to obtain the results including the design and construction of the physical models and data acquisition and processing procedures. Results and analysis are presented in Chapter Four, while Chapter Five presents a summary of the study and conclusions. Experimental data tables are presented in the appendix.

2. LITERATURE REVIEW

Literature on culvert hydraulics has been reviewed in Charbeneau et al (2002), and is not repeated here. Literature specific to this project includes definition of minor loss coefficients, performance curves for box culverts, safety end treatment (SET) design standards, and previous studies of SETs.

2.1 MINOR LOSS COEFFICIENTS

In closed conduit and open channel flow, energy losses are usually separated as those caused by wall friction and turbulence along uniform flow sections, and those caused by expansions, contractions, and other obstructions. These latter energy losses are referred to as minor losses; though this name is somewhat of a misnomer because minor losses can exceed friction losses (White, 1986; Streeter and Wylie, 1985). Generally, the theory for calculation of minor losses is weak, and energy losses are usually expressed through the use of a minor loss coefficient, K_m. The minor loss coefficient is the ratio between the minor head loss and the uniform flow velocity head:

$$K_m = \frac{h_m}{v^2/2g} \qquad \text{(Eq 2.1)}$$

In Equation 2.1, h_m is the minor head loss and v is the velocity of uniform flow in the channel or conduit. For this research effort with application to culvert hydraulics, the reference velocity head for calculation of culvert entrance losses is based on the culvert barrel velocity within the culvert entrance, rather than the approach velocity in the upstream channel (Normann et al. 1985, pg 35).

2.2 CULVERT DESIGN AND BOX CULVERT PERFORMANCE CURVES

A culvert is any structure under the roadway, usually for drainage, with a clear opening of 20 feet or less measured along the center of the roadway between inside of end walls (TxDOT Hydraulic Design Manual 2002). Culverts are usually covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed or concrete riprap channel serving as the bottom of the culvert. Culvert design involves not only structural design component but also hydraulic design components. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit (TxDOT Hydraulic Design Manual 2002).

A box culvert more readily lends itself to low allowable headwater situations. The height may be lowered and the span increased to satisfy hydraulic capacity with a low headwater. A culvert should provide the flow it is conveying with a direct entrance and a direct exit. Any abrupt change in flow direction at either end will retard the flow and require a larger structure that is not economical. One approach to avoid this additional economic expense when the centerline of the road is not perpendicular to the flow direction is to skew the culvert to make its centerline parallel to the flow direction. The barrel skew angle can be defined as the angle measured between the centerline of the road and the culvert centerline. The inlet skew angle is the angle measured between the line perpendicular to the centerline of the culvert and the culvert face. The Texas Department of Transportation (TxDOT) normally considers 0- to 60-degree skews in 15-

degree increments. Figure 2.1 illustrates the skew angle definitions. Culverts that have a barrel skew angle often have an inlet skew angle as well because headwalls are generally constructed parallel to a roadway centerline to avoid warping of the embankment fill (Norman et al. 2001). The inlet skew angle varies from 0 degrees to a practical maximum of about 45 degrees, dictated by the difficulty in transitioning the flow from the stream into the culvert fill (Normann et al. 1985). Skewed inlets slightly reduce the hydraulic performance of the culvert under inlet control conditions (Normann et al. 1985). For the purposes of this research, skew angle was studied at 0 degrees and 30 degrees.

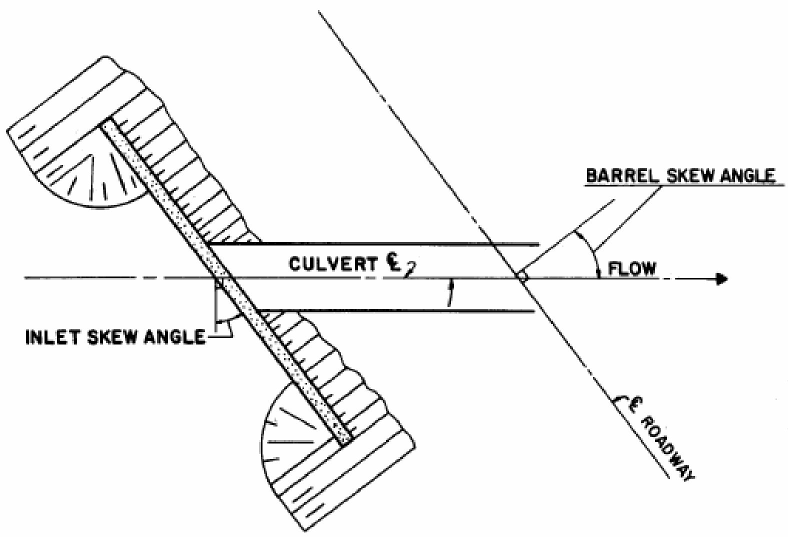


Figure 2.1: Skewed culverts (taken from Normann et al. 1985)

Performance curves relate the headwater and discharge for a culvert operating under inlet control. The performance curves developed by the Federal Highway Administration (FHWA) (Herr and Bossy 1965; Normann et al. 1985) for unsubmerged and submerged inlets are the most widely used in practice. However, as discussed in Charbeneau et al. 2002, there are conceptual issues with use of these curves for box culverts in that the curves for unsubmerged and submerged conditions do not join, and there is no apparent transition from one to the other. Furthermore, the measured data result in performance curves that differ substantially from the FHWA curves.

Charbeneau et al. (2002), present the following set of performance curves for box culverts operating under inlet control. For unsubmerged conditions

$$\frac{Q}{BD\sqrt{gD}} = C_b \left(\frac{2}{3}\right)^{3/2} \left(\frac{HW}{D}\right)^{3/2} \qquad \text{(Eq 2.2)}$$

In Equation 2.2, Q is the barrel discharge, B is the culvert span (width), D is the culvert rise (height), C_b is the width contraction coefficient, and HW is the headwater (upstream specific energy). For submerged conditions the performance curve is given by

$$\frac{Q}{BD\sqrt{gD}} = C_d \sqrt{2\left(\frac{HW}{D} - C_c\right)} \quad \text{(Eq 2.3)}$$

In Equation 2.3, C_d is the discharge coefficient and C_c is the soffit (or ceiling) contraction coefficient. The three coefficients are related through

$$C_d = C_b C_c \quad \text{(Eq 2.4)}$$

The transition between unsubmerged and submerged conditions occurs when

$$\frac{HW}{D} = \frac{3}{2} C_c \quad \text{(Eq 2.5)}$$

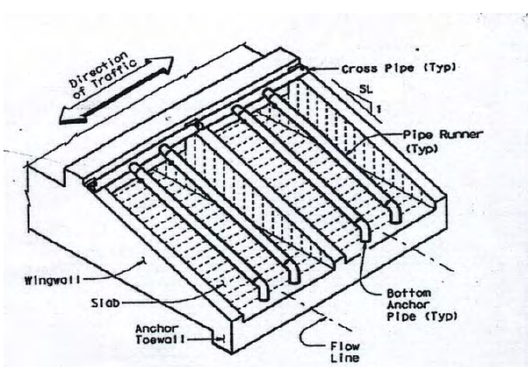
$$\frac{Q}{BD\sqrt{gD}} = C_b C_c^{\ 3/2} \quad \text{(Eq 2.6)}$$

Through analysis of their experimental data, Charbeneau et al. (2002) found $C_b = 1$, $C_c = C_d = 2/3$ for a box culvert with vertical headwall and no wingwalls.

2.3 TEXAS DEPARTMENT OF TRANSPORTATION SAFETY END TREATMENTS DESIGN STANDARDS

SET standards issued by the Bridge Division of TxDOT are reviewed in this section. This research focuses on pipe safety grates. These design standards were utilized when developing the SET model component.

There are two kinds of drainage—cross drainage and parallel drainage. Cross drainage means the traffic is across the flow and parallel drainage means the direction of traffic is parallel to the direction of the flow. Correspondingly, there are two kinds of SET installations, which are shown in Figure 2.2 (Safety End Treatment Standards, TxDOT 2000). Only the cross drainage SETs were studied in this research.



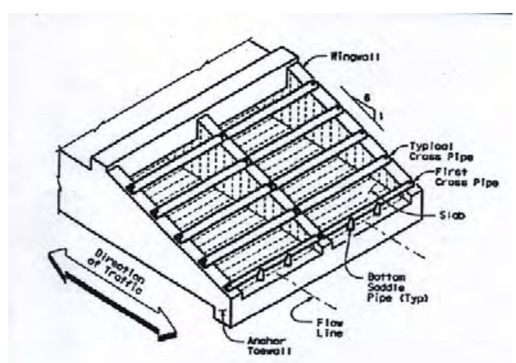


Figure 2.2: Typical installation of SETs for cross drainage and parallel drainage

There are three main parts of safety grates: cross pipe, pipe runner, and bottom anchor pipe. The cross pipe is flush with the top of the wingwall that runs across the culvert. There are two options for the construction of cross pipe. The first option is constructing it discontinuously, with one segment for each barrel and sleeve pipes serving as connections outside the wingwalls. The other alternative is making the cross pipe continuous across the inside wingwalls, so that the sleeve pipes are omitted. The total length of the cross pipe should be about the same as the culvert width, which can be seen in Figure 2.2. The cross pipe size should be the same diameter as that of the pipe runner (diameter determination explained below).

The slope of the pipe runner is the same as the slope of the wingwalls (embankment) and should be no steeper than 3:1 (horizontal: vertical). Recommended values of slope are 3:1, 4:1, and 6:1 for SET installation. The length of the pipe runners can be determined from the wingwall length (height) and the slope. All the pipe runners are equally spaced based on the centerline of each pipe runner. The allowable pipe runner spacing ranges from 2.5 feet maximum to 2.0 feet minimum measured from the centerline of the pipe runners. The number of pipe runners is determined as a function of maximum allowable pipe runner spacing. The size of the pipe runner should be as shown in Table 2.1 (Safety End Treatment Standards, TxDOT 2000).

Table 2.1: Standard pipe sizes and maximum pipe runner length

STANDARD PIPE SIZES & MAXIMUM PIPE RUNNER LENGTH			
Pipe Size	Pipe O.D.	Pipe I.D.	Max Pipe Runner Length
2" STD	2.375"	2.067"	N/A
3" STD	3.500"	3.068"	10'-0"
4" STD	4.500"	4.026"	19'-8"
5" STD	5.563"	5.047"	34'-2"

There are two options to construct the bottom anchor pipe as shown in Figure 2.8 (modified from Safety End Treatment Standards TxDOT, 2000). For the development of the SET model component used in this research, option 2 was selected.

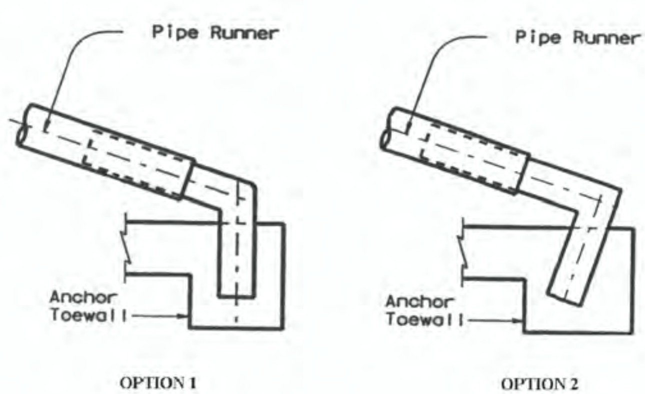


Figure 2.3: Bottom anchor pipe installation options

2.4 PREVIOUS STUDIES OF SAFETY END TREATMENTS

Most of the previous SET studies are related to pipe or circular culverts, not box culverts. Circular culverts can behave differently than box culverts.

Some observations on the hydraulic performance of safety grates were reported by Kranc et al. (1989, 2000). Their investigation was restricted to circular culverts with nine types of end sections. Both bar safety grates and pipe safety grates were tested. Headwater elevation-discharge correlations of open-ended and grated-ended culverts were compared. It was concluded that, in general, the additional losses incurred by adding the grate on the performance of inlet end section treatment were small. However in some cases, especially for culverts with box end sections, the culvert capacity under weir control could be reduced. In other cases, such as the mitered end sections with grates, the presence of the grate seems to accelerate the transition to outlet control. However, it was not recommended that designs rely on this effect. Overall the flared end section had the best performance. It was also observed that the bar safety grates had a greater negative effect on the culvert performance than the pipe safety grates but the style of the grate only matters slightly. Regarding clogging, it was found that the effect of inlet blockage was highly variable. Generally, modest accumulations could be tolerated, but a substantial buildup of debris could lead to added losses under outlet control and reduced discharge coefficients for inlet control. A 10 percent to 20 percent blockage may not justify cleaning, but 50 percent blockage may demand immediate attention. Outlet end section treatments were not found to be particularly critical, assuming that blockage with debris does not occur. Vortices did develop and a vortex suppressor was studied. It was determined that the reduction of the vortex did not have a noticeable effect on performance, at least over the range of data for the study.

Another investigation on the hydraulic performance of culverts with safety grates was conducted by The University of Texas at Austin for the Texas State Department of Highways and Public Transportation in 1983 (Mays et al. 1983). Box culvert and pipe culvert models were studied. The slope of the headwalls for both culverts was fixed as 4 to 1. The box culvert was tested with both bar grates and pipe grates, and the pipe culvert was tested only with pipe grates. It was found that for box culvert, the effect of the pipe safety grates was negligible while there was an increase in headwater depth for bar safety grates. When comparing the entrance head loss coefficients, the pipe grates showed little or no effect while the bar grates showed a more consistent increase in entrance head loss coefficient for all flow regimes tested for culvert slopes of 0.008 and 0.0108. When comparing the headwater depth and discharge relationship, pipe grates had no effect on the headwater depth; however, bar grates caused an increase in headwater depth. For the pipe culvert, the pipe safety grates had a greater effect on the culvert performance because the entrance loss coefficient was increased substantially and more significant at higher discharges. It was also concluded that the entrance head loss coefficient varied with culvert slope, headwater depth, tailwater depth, and/or discharge. With regard to clogging, it was stated that the efficiency of a box culvert was decreased substantially when clogging was greater than 45 percent.

Weisman (1989) used a 1:10 scale model of a prototype 15-feet wide culvert that has a bottom circular arc, giving a height of 5.5 feet in the center and 5.0 feet at each edge, with 0-degree wingwalls (perpendicular to the flow). A distance of 30 feet between each wingwall was studied. This configuration allows for 7.5 feet of headwall on each side of the culvert barrel in the prototype. The safety grating studied was parabolic in the vertical plane and consisted of 96 bars with 3.75 inch spacing and a 1.1 inch diameter in the prototype. It was concluded that the changes measured in headwater were not significant. At high flows the water surface contains waves and other disturbances that made measurements quite difficult. It was recommended that wider spacing with smaller bars would cause even smaller increases in water surface elevation. It was also stated vortices formed at the corners where the wingwall meet the headwall at high flows in which the headwater depth exceeds the culvert height. The vortices appeared and dissipated periodically and typically alternated from one side to the other. The presence of the grate had no effect on the occurrence of vortices.

McEnroe (1994) studied pipe culverts with end sections designed specifically for collision safety. Scale models of ten safety end sections were studied. The end sections tested were the parallel and cross-drainage versions of the 24-, 36-, 48-, and 60-inch end sections with 6:1 slopes and the 60-inch end section with a 4:1 slope. It was concluded the measured inlet-control rating curves for end sections of the same size were virtually identical, regardless of the slope of the end section and the arrangement of the safety bars. Therefore, the differences in the designs of safety end sections did not affect their performance under inlet control. It was concluded that the effect of safety end sections could cause some favorable hydraulic characteristics because they force the inlet to flow full whenever the inlet was submerged even if it was hydraulically short. In cases where a culvert with a standard end section would not flow full, a safety end section would provide superior hydraulic performance. The entrance loss coefficients for safety end sections were only slightly higher than for standard manufactured sections. Therefore, installation on existing highway culverts to meet collision-safety criteria without—significantly reducing their hydraulic capacities—could occur.

2.5 PHYSICAL MODELING

Often in hydraulic engineering studies, physical models are employed to study phenomena that are difficult to model mathematically. This method is based on the principles of hydraulic similitude, which is a known and usually limited correspondence between the behavior of a physical model and that of its prototype (Warnock 1950). Complete similitude requires the physical model be geometrically, kinematically, and dynamically similar to the prototype.

The full-sized object of interest is called the prototype, represented by subscript p, whereas the scaled down version is called the model, represented by subscript m. The length ratio of prototype to model is called the geometric length ratio $L_r = L_m : L_p$. This ratio must be consistent throughout the model to maintain similarity of linear dimensions. This consistency is called geometric similarity.

Kinematic similarity is similarity of motion. It exists between two states of motion if the ratios of the components of velocity at all homologous points in two geometrically similar systems are equal. The velocity component in prototype is $(v_i)_p$, the velocity component at the homologous point in the physical model is $(v_i)_m$, and then the scale ratio is $(v)_r = (v_i)_m : (v_i)_p$.

In order for the model to behave as the prototype, the forces in the model and the prototype must also be the same, which is known as dynamic similarity. For the purposes of this hydraulic study, dynamic similarity means that the Froude numbers of the model and prototype must be equal:

$$\frac{v_p}{\sqrt{gy_p}} = \frac{v_m}{\sqrt{gy_m}} \qquad \text{(Eq 2.7)}$$

From this equation, one may solve for the ratio of velocities and combine with the area ratio to find for the discharge ratio

$$Q_r = \frac{Q_p}{Q_m} = \left(\frac{L_p}{L_m}\right)^{5/2} \qquad \text{(Eq 2.8)}$$

In this way, various characteristics between the model and prototype can be related.

3. METHODOLOGY

The objectives of this study are to evaluate the hydraulic effects of safety end treatments (SETs) on culverts through physical modeling and to provide the Texas Department of Transportation (TxDOT) with guidance on the influence of SETs in the hydraulic design of culverts. This chapter describes the physical model as well as the equipment and experimental methods used to measure discharge and water depth in this study.

3.1 PHYSICAL MODEL DESIGN

The culvert and SET models were designed using physical model similitude principles and the TxDOT SET standards presented in Sections 2.3 and 2.5. Equation 2.8 can be used to relate discharges between a prototype and model. The prototype for this research is a single-barrel box culvert with a 6 foot rise and a 10 foot span with the SET dimensioned in accordance with TxDOT specifications for a 3:1 slope and 6:1 slope.

3.1.1 Culvert Model Design

The prototype discharge for a 6-foot rise by 10-foot span box culvert is estimated using Equation (2.3) with $C_c = C_d = 2/3$ and HW = 1.3 D. The prototype discharge is calculated as

$$Q_p = \frac{2}{3}\sqrt{2\left(1.3-\frac{2}{3}\right)}\,BD\sqrt{gD} = 625\ ft^3/s \qquad \text{(Eq 3.1)}$$

The maximum discharge available for the model using the Center for Research in Water Resources (CRWR) facilities is approximately $Q_m = 8\ ft^3/s$. The model-to-prototype scale ratio can be determined using Equation 2.8:

$$L_r = \left(\frac{Q_m}{Q_p}\right)^{2/5} = \left(\frac{8}{625}\right)^{2/5} = 0.175 \qquad \text{(Eq 3.2)}$$

Thus, a scale of 1:6 was used in this research. The prototype dimensions are 6 feet high and 10 feet wide and the corresponding model dimensions are 5/3 feet wide and 1 foot high.

The project started with the simplest case, a culvert model with a vertical headwall and parallel wingwalls at a 0-degree skew. Next, a culvert model with a 3:1 mitered headwall and parallel wingwalls at a 0-degree skew was investigated. The third culvert model investigated had a 3:1 mitered headwall and parallel wingwalls at a skew angle of 30 degrees. The fourth culvert model investigated had a 6:1 mitered headwall with parallel wingwalls at a 0-degree skew. Finally, the fifth culvert model had 6:1 mitered headwall with wingwalls at 15-degrees flare and 0-degree barrel skew.

The wingwall and embankment slopes in this research were 3:1 (S=3) and 6:1 (S=6). In the vertical headwall configuration model the heights of the wingwalls (H_w) were equal to the height of the culvert opening, i.e., 1 foot. The length of the wingwalls were determined by $L_w = H_w S = 3$ ft and $L_w = 6$ ft for the two slopes. For the mitered headwall configurations the same principle was applied to calculate wingwall length. To obtain the mitered headwall configurations, the 3:1 slope was extended above the height of the culvert opening and ranged in length from 1.5 to 4.5 feet depending on the desired level of submergence.

The width of the model channel is 5 feet. The width of the embankment slope on each side of the culvert barrel was 5/3 feet. This was calculated by taking the width of the channel, subtracting the width of the culvert opening, and dividing the resulting value by two.

For the culvert model with a 30-degree skew angle the above principles were applied, but the model was developed at a 30-degree orientation to the flow of water in the channel.

The various culvert models utilized for this research are described in Section 3.2.3.

3.1.2 Safety End Treatment Design

The SET models were designed in accordance with the TxDOT standards discussed in Section 2.3. The length scale of 1:6 was applied to the SET models as determined by the culvert model design procedures discussed in Section 3.1.1. To calculate the runner length and diameter for the model, the prototype dimensions must be determined first.

With the 3:1 mitered slope, the height of the wingwall is 6 feet and the length of the wingwall is 18 feet for the prototype. The pipe runner length (P_c) was established using the TxDOT specifications, as modified below. End of pipe clearance is not an issue in the SET model components constructed for this research. Therefore, this term can be excluded from the original TxDOT equation. K1 represents the constant value based on slope (3:1 in this case) from the TxDOT specifications.

$$P_c = (L_w * K1) = (18ft * 1.054) = 18.97ft \qquad \text{(Eq 3.3)}$$

From Table 2.1, the 4-inch standard pipe with an outer diameter of 4.5 inches and an inner diameter of 4.026 inches should be used based on the maximum pipe runner length of 18.97 feet. The allowable spacing range is 2.5 feet maximum to 2.0 feet minimum measured from the center line of the pipe runners. The prototype culvert width is 10 feet, so the number of pipe runners required is four based on the maximum allowable spacing. According to TxDOT specifications, the cross pipe should be the same size as the pipe runner.

Therefore, for the corresponding physical model, there should be four pipe runners spaced every 4 inches on center and connected to a cross pipe, with an outer diameter of 0.75 inches and an inner diameter of 0.67 inches. The length of the pipe runners should be 3.16 feet long.

The various SET models utilized for this research are described in Section 3.2.3.

3.2 Description of the Physical Model

Figure 3.1 details the layout of the physical model. The channel is divided into three sections to be able to better describe water flow through the channel. These sections are described in detail in Section 3.2.2. The upstream section leads from the headbox shown to the left of the figure downstream to the model section. The model section of the channel contains the culvert and SET models, described in Section 3.2.3.

The third channel section contains the downstream channel, tailgate, and return channel with sharp-crested weir for flow measurement. The third channel section allows the water to return to the distribution reservoir creating a recycled water system.

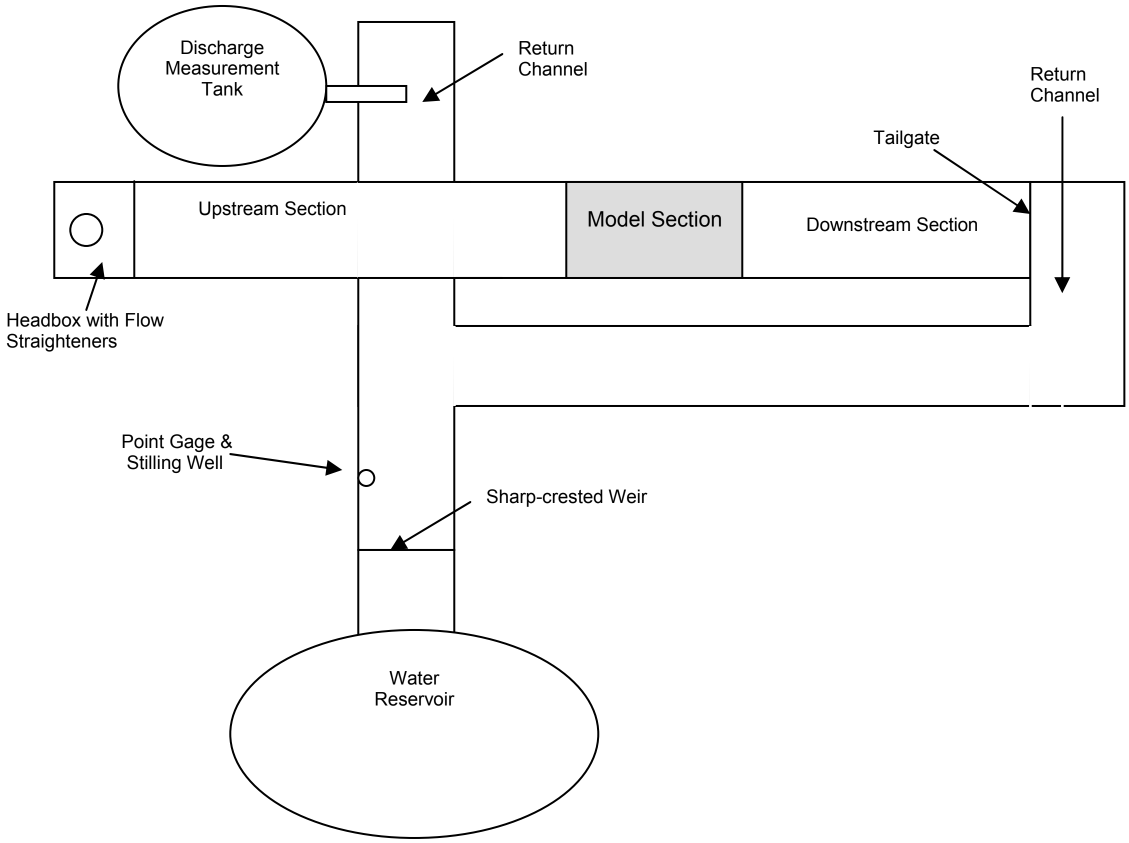


Figure 3.1: Schematic view of the physical model

3.2.1 Water Supply

The water used for the physical model experiments is pumped into the outside channel through a water distribution system from a half-million gallon reservoir located outside the CRWR laboratory. Two water supply lines lead from the distribution reservoir to the selected destination, each with its own pump. Valves located throughout the system are used to control the magnitude of channel discharge and also to direct the discharge to different destinations in the distribution system. There are three indoor destinations, the outdoor channel destination, and the discharge measurement tank destination. After flowing through the destination, the water is discharged to a return channel, which allows the water to return to the distribution reservoir creating a recycled water system.

3.2.2 Outside Channel

The outside channel is rectangular, with a width of 5 feet, a depth of 2.6 feet, and a length of 110 feet. The channel bottom measured by a previous research team was found to be approximately horizontal (Charbeneau and Holley 2001). The side slopes of the channel are zero. At the head of the upstream section, shown to the left in Figure 3.1, water is pumped into a headbox and then through discharge straighteners before entering the upstream channel section. Baffles are located in the headbox to stabilize the discharge before entering the upstream channel section. Baffles are made of several layers of cinder blocks that are overlapped so the water must follow a tortuous path and have significant contact with the blocks before it enters the discharge straighteners. The tortuous discharge path and contact with cinder blocks helps to stabilize the discharge. Discharge straighteners, located just downstream of the headbox, are used to eliminate secondary currents as the water enters the upstream section of the channel. The discharge straighteners are made from sheet metal and extend 5 feet in the direction of the discharge and across the entire width and height of the channel. They have a lateral spacing of approximately 0.5 feet. Figure 3.2 shows the vertical delivery pipe, the downstream end of the set of baffles, and the discharge straighteners.

Figure 3.2: Channel headbox and discharge straighteners (upstream end of channel)

The model section of the channel contains the culvert model and SET model. It is located approximately 72 feet from the headbox. Detailed descriptions of the culvert and SET models used in this research can be found in Section 3.2.3 and Section 3.2.4.

The downstream section of the channel includes a tailgate that allows for easy modification of the water level by changing the gate opening (Figure 3.3). The tailgate was used in the level pool procedure discussed in Section 3.3.2.

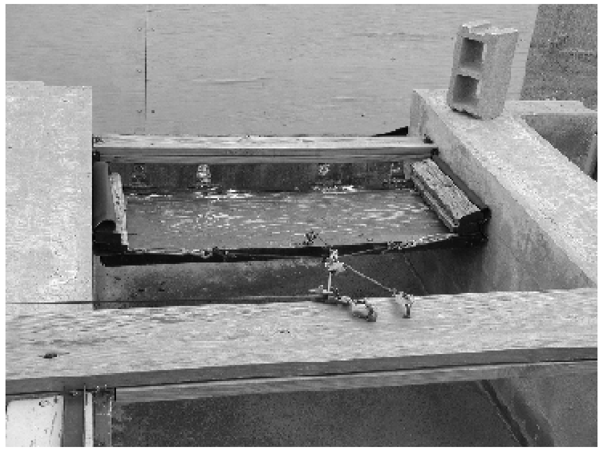


Figure 3.3: Channel tailgate (downstream end of the channel)

3.2.3 Culvert and Safety End Treatment Models

Various culvert and SET models were built using a scale of 1:6 as determined in Section 3.1.1 and were located approximately 72 feet downstream from the headbox. The models of the culvert and embankment slopes were constructed from wood, metal bracing, screws, and bolts. The models were then primed and painted to prevent water damage. Caulk was used to create a watertight seal with the model edge and rectangular channel wall. The model culvert had a 1-foot rise and 5/3 feet span. The length of the barrel was a function of the individual model configuration. Metal angle braces with one side attached to the culvert model and the other side attached to the rectangular channel wall were installed to prevent model deformation due to the force of the water movement. Model reinforcement with cinder blocks was also required to reduce the potential for deformation and sliding along the channel.

The SET models were built on a 1:6 scale, using the same scale as the culvert model. The slopes of the SET model were 3:1 and 6:1. The cross pipe and four pipe runners were made of PVC pipes with an outer diameter of 0.75 inches. The pipe runners were connected to the cross pipe using PVC T-connections in the vertical headwall and mitered headwall model configurations. The pipe runners were glued to the cross pipe using PVC end connections drilled with a hook for the 30-degree skew model configuration. The cross pipe was connected to the culvert model using hose clamps or U-brackets. The bottom anchor pipes were constructed from 90-degree PVC connections glued to the pipe runner portion of the model. The ends of the 90-degree PVC connections were beveled to have flush contact with the channel floor by cutting, filing, and filling the connections with epoxy. The PVC pipes were filled with sand to prevent vibrations. In the mitered headwall and 30-degree skew models, a plexiglass plate was added to increase stability at the base of the pipe runners.

3.2.3.1 Vertical Headwall Culvert and Safety End Treatment Model at a 0-Degree Skew

The vertical headwall model at a 0-degree skew, referred to as the vertical headwall model for the purposes of this report, was initially constructed in two pieces—the culvert model and the embankment model (Figure 3.4). After installation, it was determined the two-piece model was not feasible. Therefore, the embankment slopes were then constructed freestanding, weighed down with cinder blocks, and attached to the face of the culvert model with metal bracing (Figure 3.5). The model reinforcement is also apparent in Figure 3.5.

The rise of the culvert barrel was 1 foot and the width of the barrel was 5/3 feet. The vertical headwall extended approximately 1.5 feet above the culvert opening while the wingwall height matched the rise of the culvert barrel. The culvert barrel length was 1 foot.

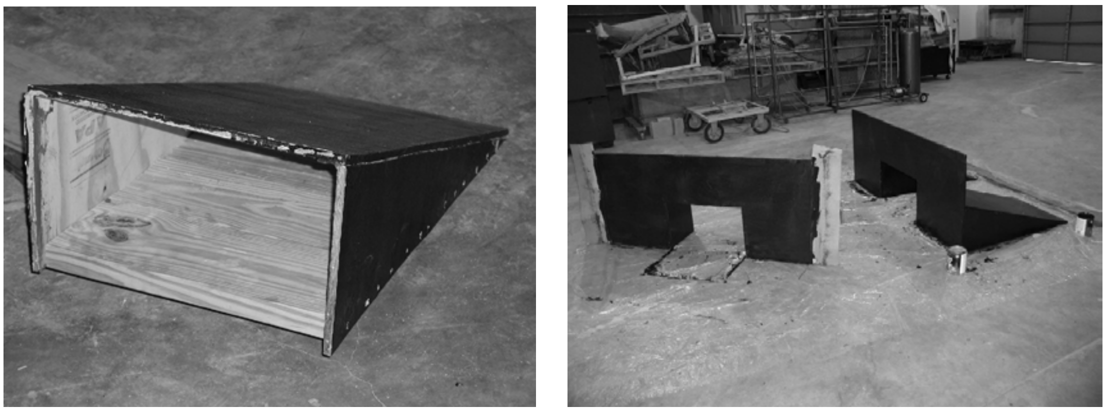


Figure 3.4: Vertical headwall model (during construction)

Figure 3.5: Vertical headwall model (during installation)

The SET model was attached to the vertical headwall using hose clamps. During model operation, the test conditions with and without the SET could be created without detachment of the entire SET model (Figure 3.6).

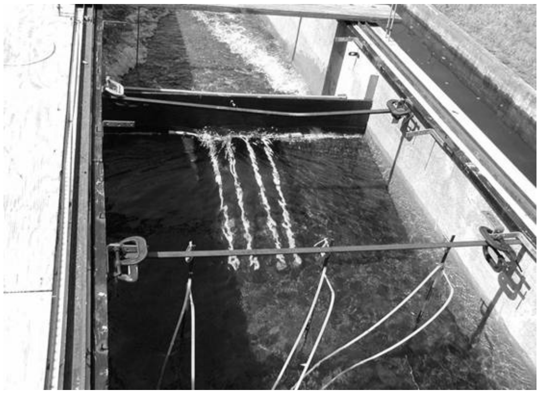


Figure 3.6: Vertical headwall model (during operation)

3.2.3.2 Mitered Headwall (3:1 Slope) Culvert and Safety End Treatment Model at 0-Degree Skew

The mitered headwall culvert model at a 0-degree skew, referred to as the mitered headwall model for the purposes of this report, was initially constructed to extend 1.5 feet above the culvert opening (Figure 3.7). After installation, it was determined the level of submergence required for the range of experiments was greater than allowed using the initial configuration. The mitered headwall was expanded to approximately 4.5 feet to obtain a greater level of submergence (Figure 3.8). A wood bracing system was developed to support the increased headwall length under the weight of water (Figure 3.8). The rise of the culvert barrel and the width of the barrel remained the same, at 1 foot and 5/3 feet respectively. The culvert barrel length was 3 feet. The SET model was attached to the mitered headwall using U-brackets and bolts. A plexiglass plate was attached to the bottom anchor pipes of the SET model to provide additional stabilization. During model operation the test conditions with and without the SET could be created with complete detachment of the entire SET model.

Figure 3.7: Initial mitered headwall model (during operation)

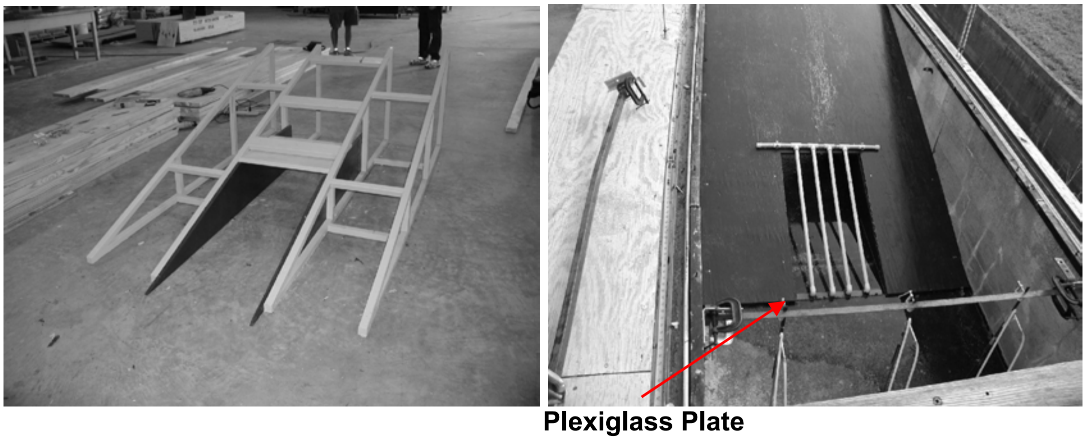


Figure 3.8: Mitered headwall model (during installation)

Vortices formed at the corners where the wingwall meet the headwall at high flows for submerged conditions. The vortices appear and dissipate periodically and typically alternate from one side to the other. The presence of the SET model component has no effect on the occurrence of vortices.

3.2.3.3 Mitered Headwall (3:1 Slope) Culvert and Safety End Treatment Model at 30-Degree Skew

The mitered headwall culvert model at a 30-degree skew, referred to as the 30-degree skew model for the purposes of this report, was constructed to extend 1.5 feet above the culvert opening (Figure 3.9). The rise of the culvert barrel was 1 foot, the width of the barrel was 5/3 feet, and the length of the culvert barrel was 3 feet. A similar bracing system was created as shown in Figure 3.8 to support the mitered headwall against deformation. The mitered headwall was limited in extent to 1.5 feet to prevent the formation of the vortex phenomenon. This limits the range of discharge rates and headwater that can be examined. The SET model was attached to the mitered headwall using U-brackets and screws. A plexiglass plate was attached to the bottom anchor pipes of the SET model to provide additional stabilization. During model operation the test conditions with and without the SET could be created with complete detachment of the entire SET model.

Figure 3.9: 30-Degree skew model (during installation and operation)

3.2.3.4 Mitered (6:1 Slope) Culvert with Curb Headwall and Safety End Treatment Model at 0-Degree Skew

The 6:1 mitered slope culvert model at a 0-degree skew and 0-degree flare, referred to as the 6:1 mitered model for the purposes of this report, was constructed to operate with headwater that could extend slightly above the barrel rise (Figure 3.10). The short headwall was vertical and represented a roadway curb. The rise of the culvert barrel was 1 foot and the width of the barrel was 5/3 feet. A similar bracing system was created as shown in Figure 3.8 to support the mitered slope against deformation. The limited height of the headwall did not allow formation of the vortex phenomena. However, this limits the range of discharge rates and headwater that can be examined. The SET model was attached to the headwall using U-brackets and screws. SET pipes were stabilized by filling PVC with copper tubing and sand. A plexiglass plate was attached to the bottom anchor pipes of the SET model to provide additional stabilization. During model operation, the test conditions with and without the SET could be created with complete detachment of the entire SET model.

Figure 3.10: Mitered slope (6:1) with 0-degree flare

3.2.3.5 Mitered (6:1 Slope) Culvert and Safety End Treatment Model with 15-Degree Flare

The 6:1 mitered slope culvert model at a 0-degree skew and 15-degree flare, referred to as the 6:1 mitered/flared model for the purposes of this report, was constructed to operate with headwater that could extend slightly above the barrel rise (Figure 3.11). The short headwall was vertical and represented a roadway curb. The rise of the culvert barrel was 1 foot and the width of the barrel was 5/3 feet. A similar bracing system was created as shown in Figure 3.8 to support the mitered slope against deformation. Copper tubing was used inside the PVC to stabilize the SET. The limited height of the headwall did not allow formation of the vortex phenomena. However, this limits the range of discharge rates and headwater that can be examined. The SET model was attached to the headwall using U-brackets and screws. A plexiglass plate was attached to the bottom anchor pipes of the SET model to provide additional stabilization. During model operation, the test conditions with and without the SET could be created with complete detachment of the entire SET model.

Figure 3.11: Mitered slope (6:1) with 15-degree flare

3.3 DATA ACQUISITION

The data collected in this investigation are the channel discharge and the upstream water depth. From the data collected, water velocity, velocity head, specific energy, and minor loss coefficients were calculated. The following sections describe the methods used to collect these data and the calculations that were performed.

3.3.1 Channel Discharge

Accurately determining the flow rate was an integral part of the experiments because this value would be used to compute many other parameters (e.g., flow velocity). Channel discharge was typically measured with a normal sharp-crested weir calibrated against tank measurements. These methods are described below.

The sharp-crested weir was constructed of a thin plate of metal and erected perpendicular to the flow near the downstream end of the reservoir return channel. The upstream face of the weir plate is smooth except for a small bypass door (1 feet wide and 8 inches high) located in the center of the bottom of the weir. This door allows for complete drainage of water in the return channel when necessary. The weir extends horizontally, the full 5 feet width of the channel. A weir height of 2 feet was utilized to allow for adequate freeboard. The weir was attached to the return channel walls using angle iron. The weir is reinforced on the downstream side with a horizontal brace to prevent deformation.

The sharp-crested design caused the nappe to spring free. Upstream water levels were measured using a point gage, with the lower half of the point gage placed in a stilling well (transparent plastic tube) to reduce the impacts of water waves while measurements were taken. The point gage and stilling well were located about 16 feet upstream from the weir to ensure it was beyond the zone of appreciable surface curvature. Nappe aeration tests were performed using a 4 inch diameter PVC pipe to ensure this was an adequate distance for the point gage and stilling well placement.

The weir flow was computed from (Bos 1989)

$$Q = C_d \frac{2}{3}\sqrt{2g}LH^{3/2} \tag{3.4}$$

where C_d = 0.618, L = width of weir crest = 5 feet, and H = measured head on the weir crest.

The measured head was determined by point gage measurements. This tool is a marked pointer that can measure the elevation of the water surface by adjusting the tip of the pointer to the water surface and reading the dial configuration (thousandth decimal place accuracy). The datum weir crest height was determined by averaging a series of point gage measurements taken when the water just reached the weir crest. This datum value is subtracted from discharge point gage measurements to determine the measured head value. Each discharge point gage measurement was taken at least 30 minutes after any change in discharge rate to ensure stabilization had occurred.

An alternative method of measuring channel discharge was employed to calibrate the sharp-crested weir. This method involves redirecting all channel discharge into the large tank reservoir shown in Figure 3.12. The tank reservoir has a capacity of 9,300 gallons with dimensions of 12 feet in diameter and 11 feet in height. The channel discharge is redirected to the inlet pipe located at the top of the tank and discharged through the outlet pipe controlled by a valve. The water level inside the tank can be viewed through a vertically mounted spyglass on the outside of the tank. When the water level in the tank (meniscus in the spyglass) is no longer fluctuating, the system has reached steady state. Once steady state is achieved, the outlet valve is closed quickly and the elapsed time is recorded for the meniscus in the spyglass to move from the steady state level to a level near the top of the spyglass. Using the change in water level, cross sectional area of the tank, and the lapsed time, the discharge rate is determined. This process was performed at several discharge rates and used to calibrate the weir. Calibration results are shown in Figure 3.13.

Figure 3.12: Large tank reservoir used for weir calibration

When discharging from the large tank reservoir a strong wave existed near the point gage that impacted the discharge point gage measurement. To minimize this effect, a 1.3 feet high by 3 feet wide dam constructed of cinder blocks was built between the outlet pipe and the point gage. This dam allowed for enough energy dissipation that the wave no longer existed; therefore, the discharge point gage measurement was accurate.

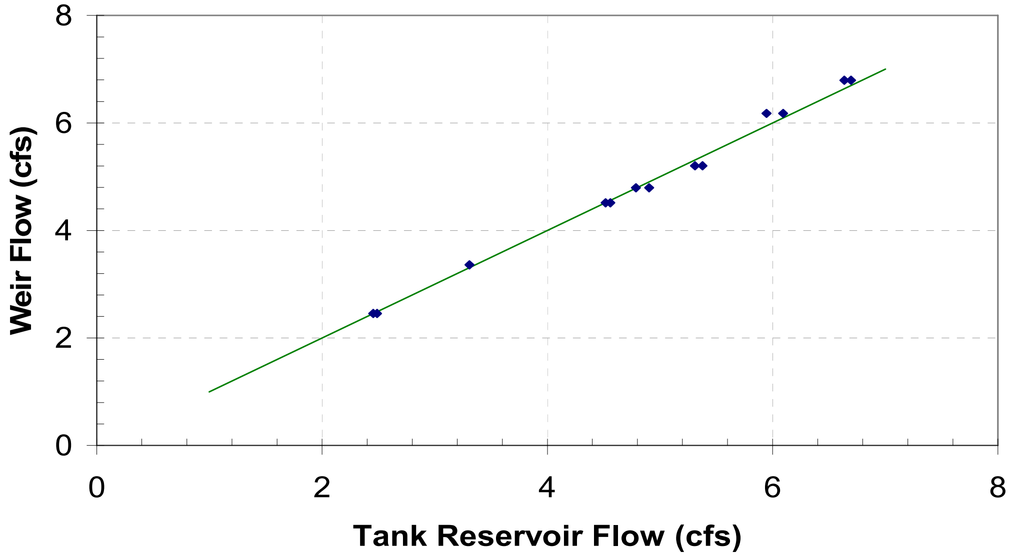


Figure 3.13: Weir calibration

Figure 3.13 shows results from a calibration run. This figure shows the discharge measured in the storage tank from change in water level versus the discharge measured by the weir. The solid line marks exact one-to-one correspondence. This line is a good fit to the data points; showing the weir and tank reservoir discharge measurements do indeed correspond to each other and therefore, measuring discharge using the weir equation is an accurate method. The standard deviation of the weir flow rate minus the tank flow rate divided by the tank flow rate is 0.0205.

3.3.2 Water Depth

It was essential to this research to obtain accurate measurements of small changes in water depth. For the purpose of this report, the terms "water depth," "water level," and "static head" will be considered synonymous and the terms will be used interchangeably. The static ports of Pitot tubes were connected to an inclined manometer board via flexible plastic tubing to obtain the accurate measurements of the small changes in water level required (Figure 3.14). The inclined manometer board allows precise measurement of water depth. A Pitot tube can measure both total hydraulic head and static hydraulic head. Only the static hydraulic head measurements were utilized in this research.

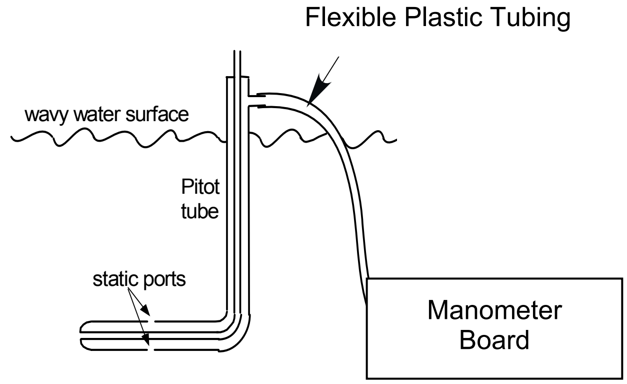


Figure 3.14: Schematic of water depth measurement system

The inclined manometer board and flushing tube were constructed for the purposes of this research using both new and existing materials from the CRWR hydraulics laboratory (Figure 3.15). The flushing tube is a PVC pipe that is approximately 4 feet in length, 3 inches in diameter, and has barbs for the attachment of the flexible plastic tubing and a water source (provided by a water hose). By forcing water through the system, the air is "flushed" out of the flexible plastic tubing as well as the hard plastic tubes on the inclined manometer board. The inclined manometer board consists of twenty-eight transparent hard plastic tubes with barbs for flexible tubing connection at each end and thirteen measurement tapes mounted on a 1 inch thick plexiglass sheet attached to an adjustable metal frame. Each hard plastic tube has an inner diameter of 3/8 inch and a length of 4.46 feet. A water depth reading is determined by correlating the meniscus of the water column in the hard plastic tube with the measurement tape located to the right of the hard plastic tube.

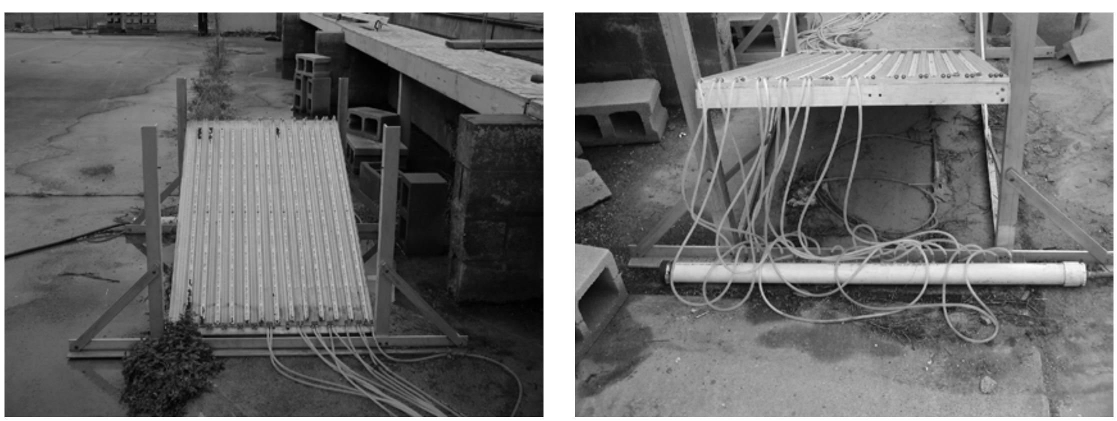


Figure 3.15: Manometer board and flushing tube

The slope of the manometer board determines the amplification of water depth readings. However, the slope of the manometer board must be shallow enough that the meniscus of the water column can be read throughout the desired range of experiments. For this research, the desired range of water depth is approximately 2 feet; therefore, the slope of the manometer board was fixed at 1 : 2.41 (vertical : hypotenuse), which means that vertical water readings were amplified 2.41 times. Thus,

$$H_v = \frac{H_A}{2.41} \qquad \text{(Eq 3.5)}$$

where H_v = the vertical position of the water surface and H_A = the reading from the inclined manometer board.

Six Pitot tubes were installed at two cross sections upstream from the culvert and SET model (Figure 3.16). Two pieces of angle iron were fixed across the channel at positions 5 feet and 9 feet upstream from the culvert entrance. One set of three Pitot tubes was attached to each angle iron by C-clamps. In each set of Pitot tubes, one was attached in the middle of the channel and the other two were offset 1.5 feet from the center Pitot tube. All the Pitot tube bottoms were approximately 2 inches from the floor of the channel.

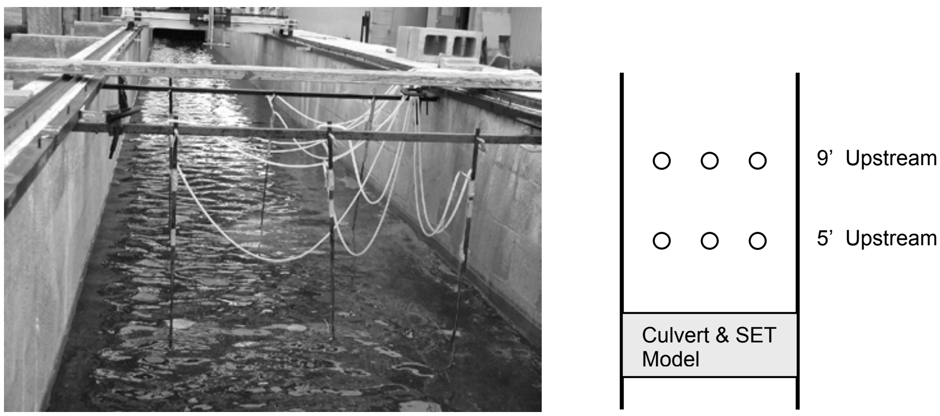


Figure 3.16: Pitot tube configuration

The static head values measured by a Pitot tube must be referenced to a datum in order to be converted into water depth. In order to establish the datum a level pool calibration must be performed. A level pool was created in the channel by passing a very low discharge rate through the channel and allowing the system to reach steady state. A small discharge was required because of leakage through the tailgate. Velocities in the pool were very low so that every point on the horizontal surface of the pool had approximately the same elevation for the entire channel length. Thus, it was considered a level pool.

The Pitot tube datum was established by creating a level pool in the channel and marking the static head readings on the manometer board under the level pool condition. Next, the depth was measured at the location of all Pitot tubes with a point gage. These depths corresponded to the static head readings obtained from the manometer board reading. The differences in those readings were measured by the deviation of the meniscus from the level pool readings on the manometer board.

During the model experiments, two sets of water depth measurements were taken at each discharge rate for each scenario at the six Pitot tubes located as shown in Figure 3.16. The measured water level values were averaged arithmetically. By comparing the average measured values for scenarios with SETs present with those without SETs present, the effect of SETs could be determined.

At the beginning of each experiment, there was no water in the channel. The experiment started by flushing water through the Pitot tubes for about 20 minutes to eliminate air bubbles from the flexible plastic tubing as well as the hard plastic tubes on the inclined manometer board. Air bubbles could create an error in water depth measurement. Figure 3.15 shows the configuration used for the flushing process. While the flushing process was occurring, the Pitot tubes were checked and, if necessary, cleaned with a needle to ensure that all the static ports of each Pitot tube were not clogged (shown in Figure 3.14).

While the flushing process was still taking place and after verifying that each Pitot tube was clean, the required discharge rate was set by configuring the valves and turning on the appropriate pumps. When the water depth in the channel rose above the Pitot tubes, the flushing process continued for approximately 20 to 30 minutes to prevent air from reentering any tubing. At least 30 minutes after the flushing process was completed, the first set of water depth and flow measurements was obtained. Once the first set of readings was finished, the scenario was changed according to what was required (i.e., the removal/insertion of the SET model component or the removal/insertion of the anti-vortex plate). As before, the second group of measurements was performed at least 30 minutes later, allowing enough time for the flow conditions to stabilize. Two sets of water level readings were taken for each measurement set and averaged arithmetically to ensure accuracy of the water level readings obtained before changing the discharge.

3.4 DATA PROCESSING

From the acquired data of channel discharge and water level, many other components were calculated. These calculations included headwater, barrel velocity head, and SET minor loss coefficient.

3.4.1 Headwater

The upstream (approach) water depth (y_a) is determined from the calibrated Pitot tube measurements. The headwater (HW) is the approach specific-energy. The headwater may be calculated using

$$HW = y_a + \frac{{v_a}^2}{2g} = y_a + \frac{Q^2}{2g(B_a y_a)^2} \qquad \text{(Eq 3.6)}$$

In Equation 3.6, B_a is the approach channel width (5 feet) and the channel discharge is provided through weir measurements.

3.4.2 Culvert Barrel Velocity Head

The barrel velocity (velocity head) is not measured directly. Instead, it is calculated from assumptions that are made in development of the performance equations for box culverts (Equations 2.2 and 2.3). The first assumption is that the culvert entrance-specific energy is equal to the headwater (head losses are negligible). The second assumption depends on whether the culvert is operating under unsubmerged or submerged conditions. For unsubmerged conditions it is assumed that the culvert entrance chokes the channel (Henderson 1966) so that critical flow occurs within the culvert entrance (the culvert entrance acts as a control). At the control section the effective channel width is C_b B, the depth is critical (y_c), as is the velocity ($v_c = \sqrt{g\,y_c}$). Combining these, the discharge is calculated from

$$Q = C_b B y_c v_c = C_b B \sqrt{g y_c^{\;3}} \qquad \text{(Eq 3.7)}$$

Thus, critical depth may be calculated from

$$y_c = \left(\frac{Q^2}{g C_b^{\;2} B^2} \right)^{1/3} \qquad \text{(Eq 3.8)}$$

The corresponding barrel velocity head is then given by

$$\frac{v_b^{\;2}}{2g} = \frac{v_c^{\;2}}{2g} = \frac{\left(\sqrt{g y_c}\right)^2}{2g} = \frac{y_c}{2} = \left(\frac{Q^2}{8 g C_b^{\;2} B^2} \right)^{1/3} \qquad \text{(Eq 3.9)}$$

Calculation of the culvert barrel velocity head requires estimation of the side contraction coefficient, C_b.

When the culvert operates under submerged conditions, the specific energy within the barrel entrance is again assumed to equal the headwater, and the depth of flow is determined by the soffit (ceiling) contraction coefficient. Equating the headwater and barrel entrance-specific energy one finds

$$HW = C_c D + \frac{v_b^{\;2}}{2g}$$

Thus the barrel velocity head may be calculated from

$$\frac{v_b^{\;2}}{2g} = HW - C_c D \qquad \text{(Eq 3.10)}$$

The barrel velocity may also be calculated from the discharge divided by the flow area within the culvert entrance. Including both side and ceiling contraction coefficients, this gives for the barrel velocity head

$$\frac{v_b^{\;2}}{2g} = \frac{\left(Q/\left(C_b B C_c D\right)\right)^2}{2g} = \frac{Q^2}{2 g C_d^{\;2} B^2 D^2} \qquad \text{(Eq 3.11)}$$

Equations 3.10 and 3.11 are combined to give the performance, Equation 2.3. The transition from unsubmerged to submerged performance conditions is specified by Equations 2.5 and 2.6.

3.4.3 Minor Loss Coefficient

The minor loss coefficient is defined by Equation 2.1. To calculate the minor loss coefficient associated with SETs, the difference in the approach depth of flow is measured for conditions with and without the SET in place. The reference barrel velocity head is that calculated for conditions without the SET present. The minor head loss associated with SETs is the difference in headwater. If y_a is the depth without SET present and $(y_a + \Delta y)$ is the depth with the SET present, the head loss is calculated from

$$h_m = \Delta y + \frac{Q^2}{2gB_a{}^2}\left(\frac{1}{(y_a + \Delta y)^2} - \frac{1}{y_a{}^2}\right) \qquad \text{(Eq 3.12)}$$

Because Δy is small compared with y_a, Equation 3.12 can be approximated using (Taylor's series)

$$h_m = \Delta y\left(1 - \frac{Q^2}{gB_a{}^2 y_a{}^3}\right) \qquad \text{(Eq 3.13)}$$

With h_m calculated from Equation 3.13 and the barrel velocity head from Equation 3.9 for unsubmerged conditions and 3.10 for submerged conditions, Equation 2.1 is used to calculate the SET minor loss coefficient, K_m.

4. RESULTS

The overall goal of this research is to evaluate the hydraulic effects of safety end treatments (SETs) on culverts through physical modeling and to provide the Texas Department of Transportation guidance on the influence of SETs in the hydraulic design of culverts. Important specific research objectives are to study the nature of water level difference upstream of the culvert due to SET presence; to provide minor loss coefficients due to the presence of SETs that may be used in design procedures, and, finally to compare the headwater-level discharge relationships with and without SET presence to show how SETs with different end configurations influence the hydraulics of culverts by developing performance curves for box culverts operating under inlet control, based on the experiments performed during this study, and compare them to performance curves developed from earlier work reported in "Hydraulics of Channel Expansions Leading to Low-Head Culverts" by Charbeneau et al. (2002). The analysis of collected data is presented in this chapter.

4.1 WATER LEVEL DIFFERENCE

The water level difference upstream of the culvert can be examined in two ways: distribution across the channel for a specified discharge, and average water level difference due to the presence of SETs. Benson (2004) has examined the former issue by evaluating the water level difference distributions across the channel at two channel cross sections—one for the three Pitot tubes that are installed 5 feet upstream of the culvert model and one for the three Pitot tubes that are installed 9 feet upstream of the culvert model, as shown in Figure 4.1. These cross sections were compared to see if trends exist under different flow conditions for different model configurations. The bed of the channel is essentially horizontal; this indicates that the depth at a cross section should be approximately equal. The water level across the cross section 9 feet upstream of the culvert entrance is consistent but the water level across the cross section 5 feet upstream from the culvert entrance is somewhat variable. The water level is slightly higher at the cross section 5 feet upstream of the culvert entrance compared to the cross section 9 feet upstream of the culvert entrance. The water level is always slightly higher when the SET model component is present. Benson (2004) concluded that the small variance in water level at the cross section 5 feet upstream of the culvert entrance could be due to unevenness of the bed channel or the slight increase in depth, especially near the channel center, and could be associated with the conversion of kinetic energy to depth in order to build up the energy to accelerate the flow through the culvert entrance section. However, the differences in water level depths across the channel are so slight that they can be neglected. Thus, the water levels of the 5 foot cross section and the 9 foot cross section can be viewed as approximately static and the arithmetic average of these six water depths measured by the Pitot tubes can be taken to represent the water level for each model scenario.

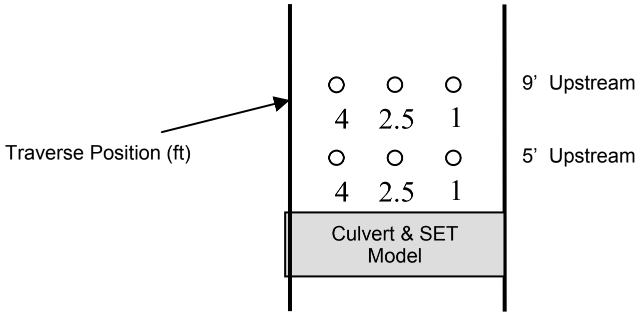


Figure 4.1: Pitot tube locations

It was expected that the presence of SETs would increase the water level upstream of the culvert because of the added drag force imposed on the flow. As shown in Figure 4.2, the experimental data confirms that for most conditions of interest with low-head culverts, the water depth increases when SETs are present. The depth change (Δy) is the water level difference between conditions with SETs present and SETs removed. For larger depths (specific energy) with the 3:1 mitered slope, a large vortex would form that would dominate the presence of SETs and generally improve culvert performance (decrease in upstream depth for a given discharge). While the effect of the vortex was more significant than the effect of SETs, upstream water depths would sometimes be smaller with SETs present, which gives negative depth changes as shown in Figure 4.2. While the presence or absence of the vortex is of hydraulic interest, it is not of direct interest to the scope of this study.

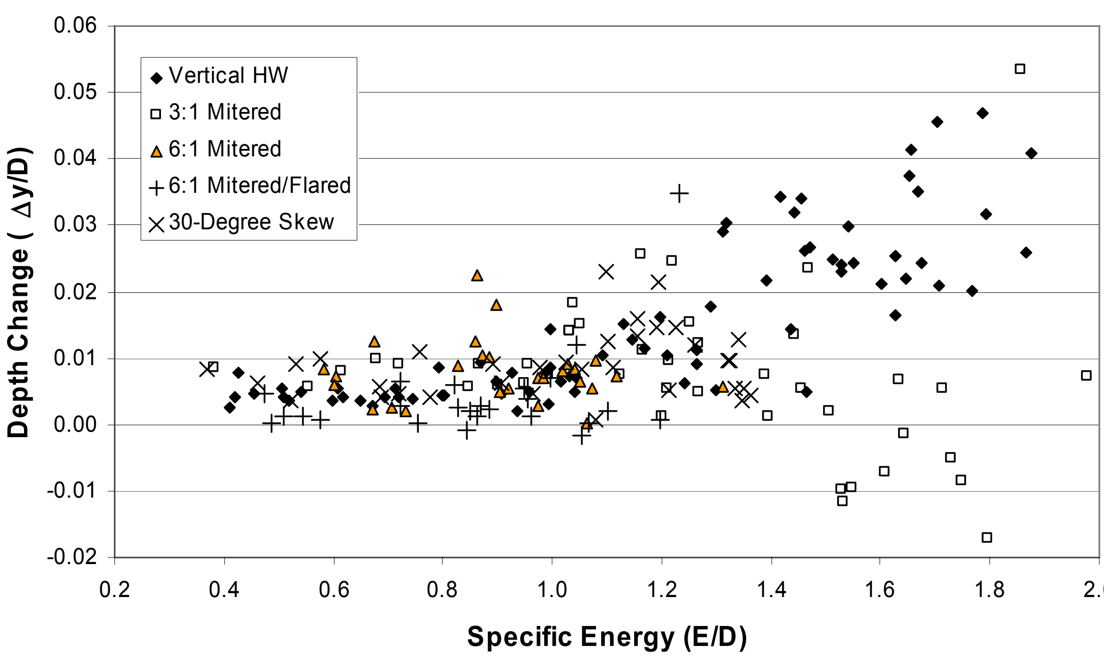


Figure 4.2: Water depth difference due to SET presence

Figure 4.2 clearly indicates that there is a trend in water level difference increasing as the upstream depth or specific energy increases, except for the model culvert with 3:1 mitered slope (because of the development of the vortex). A point of specific interest is that the relative change in depth (compared with the culvert rise) is small, generally being less than a few percent. This suggests that the backwater effects of SETs are also small for all culvert end configurations considered. These data are used to estimate minor loss coefficients for SETs with different culvert end configurations. First, however, performance curves are calibrated so that the necessary performance parameters can be estimated.

4.2 CALIBRATION OF PERFORMANCE CURVE PARAMETERS

The culvert performance curves are specified by Equations 2.2 and 2.3 for unsubmerged and submerged conditions, respectively. The performance curve parameters are the side and ceiling contraction coefficients, C_b and C_c. The third discharge coefficient, C_d, is related through Equation 2.4.

The parameters are estimated using the least-squares method. Both the discharge and headwater are measured for each experimental run (where the headwater is calculated using Equation 3.6, and either may be used as the regression variable. Consider the case of regression of headwater against discharge. For each measured value of the variable $\left(\frac{Q}{BD\sqrt{gD}}\right)_m$, there is a measured value $\left(\frac{HW}{D}\right)_m$ and a value $\left(\frac{HW}{D}\right)_c$ calculated from Equation 2.2 or 2.3. The calculated value requires estimation of the regression parameters C_b, C_c, and C_d. The normalized standard error (SE) for the entire data set may be calculated from the SE statistic which is defined by the following equation:

$$SE = \frac{1}{N}\sum_{j=1}^{N}\frac{\sqrt{\left((HW/D)_{m,j} - (HW/D)_{c,j}\right)^2}}{(HW/D)_{c,j}} \qquad \text{(Eq 4.1)}$$

In Equation 4.1, N is the number of data points. The objective of the least-squares method is to determine the regression parameters that minimize the normalized standard error. Estimated values of C_b and C_c (with $C_d = C_b\ C_c$) are adjusted until a minimum value of SE is achieved. This adjustment in parameter values is accomplished using the *Microsoft Excel "Solve"* function. The resulting values are shown in Table 4.1. The parameter values for the box culvert (vertical headwall with no wingwalls and 0-degree skew) were presented in Charbeneau et al. 2002.

Table 4.1: Parameters determined through regression of $\left(\frac{HW}{D}\right)_m$ on $\left(\frac{Q}{BD\sqrt{gD}}\right)_m$

	C_b	C_c	C_d
Box culvert	1.000	0.667	0.667
Vertical headwall	0.910	0.629	0.572
3:1 mitered	0.898	0.569	0.511
6:1 mitered	0.875	0.703	0.615
6:1 flared	0.895	0.814	0.728
30-degree skew	0.802	0.614	0.492

The same procedure can be used in regression of the measured discharge on the measured headwater. The results of this regression are shown in Table 4.2.

Table 4.2: Parameters determined through regression of $\left(\frac{Q}{BD\sqrt{gD}}\right)_m$ on $\left(\frac{HW}{D}\right)_m$

	C_b	C_c	C_d
Box culvert	1.000	0.667	0.667
Vertical headwall	0.918	0.620	0.569
3:1 mitered	0.898	0.569	0.511
6:1 mitered	0.876	0.702	0.615
6:1 flared	0.899	0.695	0.625
30-degree skew	0.802	0.614	0.492

Comparison of the parameter values in Tables 4.1 and 4.2 shows that the only parameter that differs significantly between the two tables is the ceiling contraction coefficient, C_c, for the 6:1 mitered/flared configuration. The configuration shown in Figure 3.11 does not allow for significant submergence of the culvert, so estimation of the ceiling contraction coefficient is uncertain. Because there is no apparent reason to prefer one regression variable over another, the estimated parameters are averaged for use in subsequent analysis. The resulting representative parameter values are provided in Table 4.3.

Table 4.3: Representative parameter values for culvert performance

	C_b	C_c	C_d
Box culvert	1.000	0.667	0.667
Vertical headwall	0.914	0.624	0.571
3:1 mitered	0.898	0.569	0.511
6:1 mitered	0.876	0.703	0.615
6:1 flared	0.897	0.755	0.677
30-degree skew	0.802	0.614	0.492

The different performance parameters do have a significant influence on the resulting culvert performance curves. The performance data for the different culvert end configurations are shown in Figure 4.3, along with the performance curve for the box culvert. It is clear from this figure that additional energy (headwater) is required to support a given discharge through the different culvert systems, as compared with the box culvert.

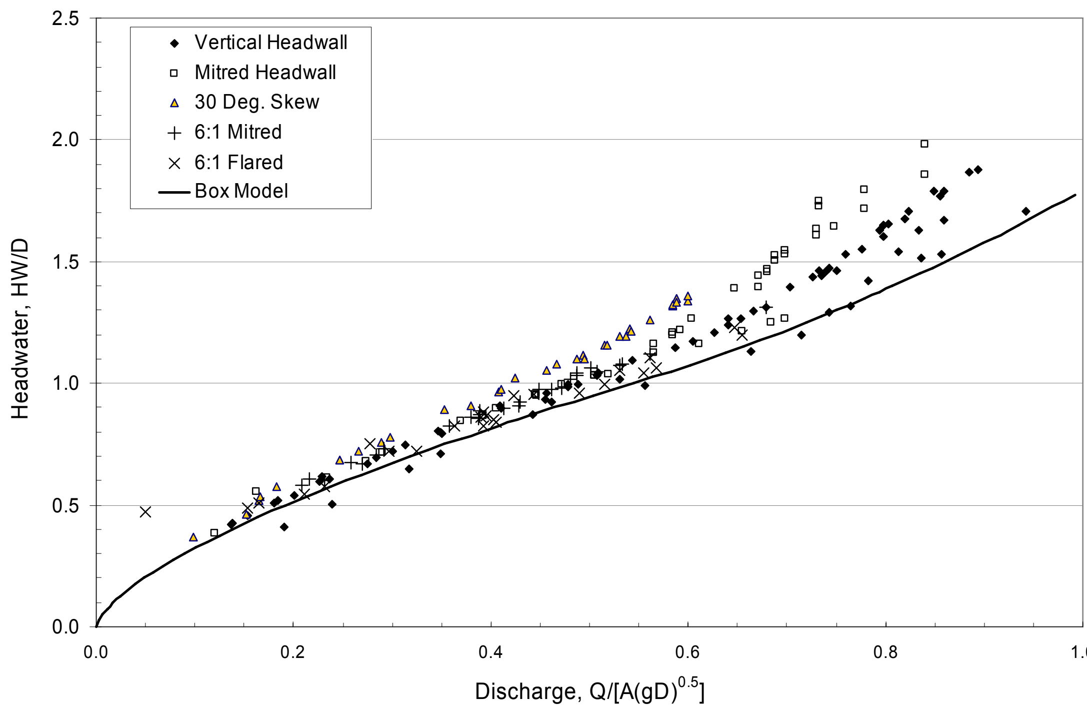


Figure 4.3: Performance data for different culvert end configurations

Figure 4.4 shows the data for the vertical headwall and 3:1 mitered configurations, along with their fitted performance curves. The parameters are provided in Table 4.3. For unsubmerged conditions, the curves are nearly identical, corresponding to their similar C_b values, 0.911 and 0.898. This is expected because the embankment had a 3:1 slope for both model configurations. The difference for submerged conditions is associated with the headwall and its influence on the soffit contraction coefficient, C_c. This parameter decreases from 0.624 for the vertical headwall to 0.569 for the 3:1 mitered headwall. This can be understood physically from the momentum of the flow down the mitered slope and around the culvert soffit, for submerged conditions, as compared with flow down the vertical headwall.

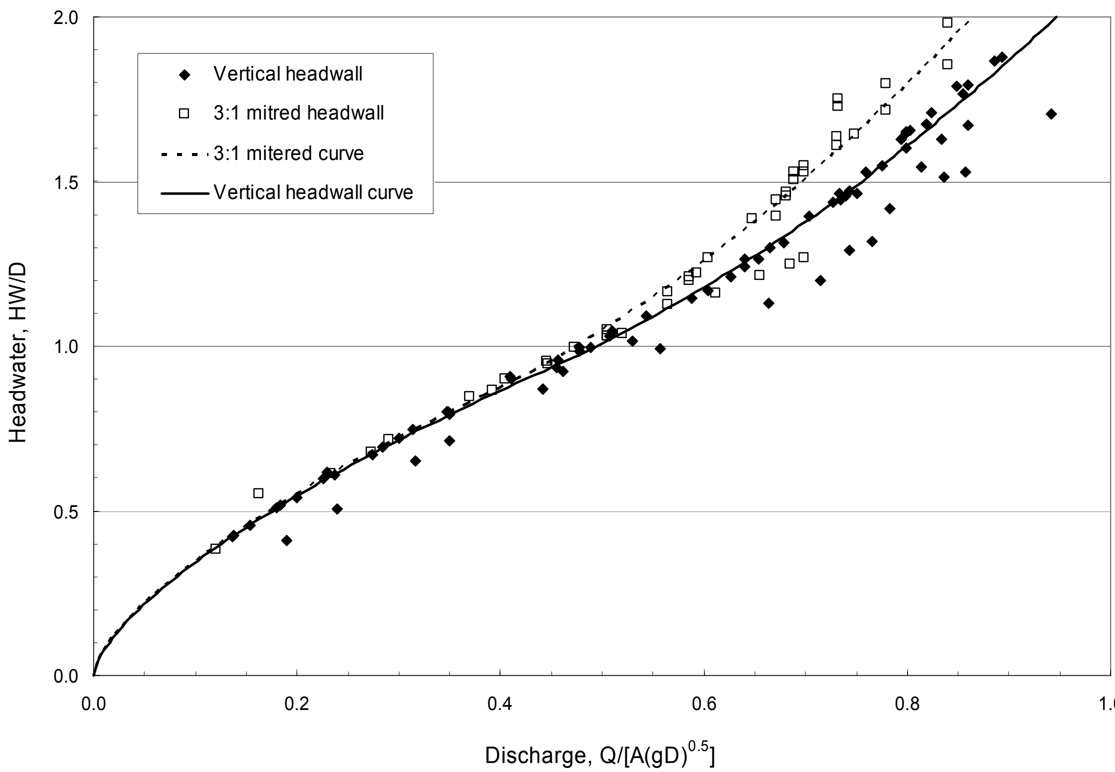


Figure 4.4: Influence of headwall configuration on culvert performance

Figure 4.5 shows the performance data and fitted curves for the 6:1 mitered slope, 6:1 mitered slope with 15-degree flares, and 3:1 mitered slope with 30-degree skew configurations. There is very little difference between the two 6:1 mitered curves, except that the flared configuration is slightly more efficient, requiring slightly less headwater for the same discharge. The performance of the skewed configuration is less efficient both for unsubmerged and submerged configurations. This is primarily associated with the smaller side contraction coefficient, C_b = 0.802. Separation of the flow from the side walls near the culvert entrance is more severe when the culvert barrel is skew to the headwall and wingwalls.

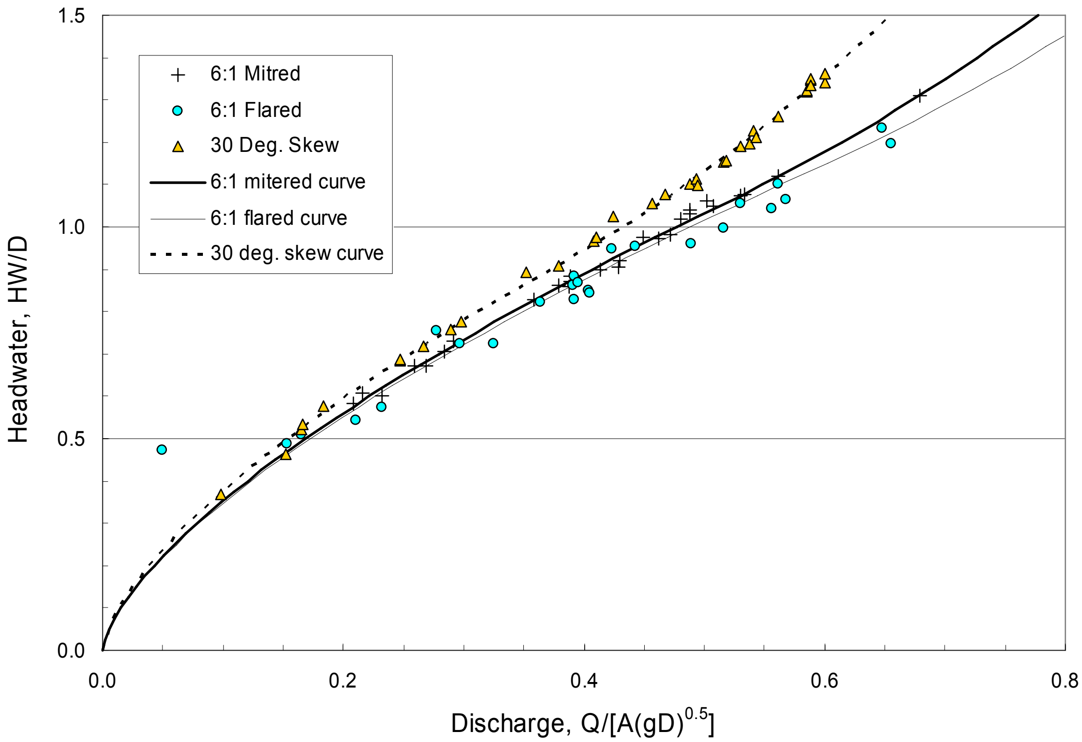


Figure 4.5: Performance data and curves for the 6:1 mitered, 6:1 mitered/flared, and skewed culvert configurations

Figure 4.6 provides a summary view of the six performance curves with parameter values from Table 4.3. In terms of headwater required for a given discharge, they range in decreasing efficiency from a standard box culvert to one whose barrel is skewed to the roadway embankment. For unsubmerged conditions, there is little difference between a 3:1 and 6:1 wingwall slope. Nor does providing a slight flare to the wingwalls appear to have a significant effect. The curves diverge for submerged conditions, with the flared configuration taking on highest efficiency. Throughout, the skewed configuration has the lowest efficiency in culvert performance. However, it is important to note that only the vertical headwall and 3:1 mitered configurations have numerous data for submerged configurations, so the performance of other configurations at larger headwater is uncertain.

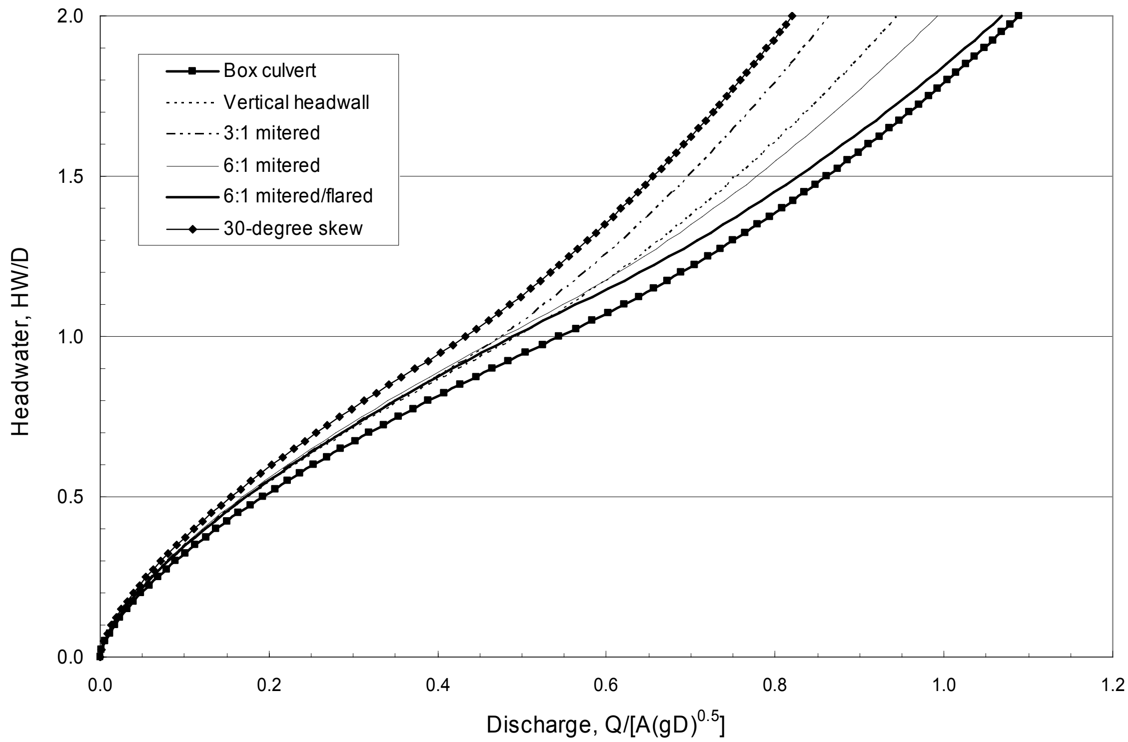


Figure 4.6: Culvert performance curves predicted using parameters from Table 4.3

4.3 SAFETY END TREATMENT MINOR LOSS COEFFICIENTS

In this section, the minor loss coefficients are provided. The data shown in Figure 4.2 are used with Equation 3.13 to calculate the head loss associated with SETs. The culvert barrel velocity head is calculated using Equation 3.9 for unsubmerged conditions. For submerged conditions, the barrel velocity head can be calculated using either Equation 3.10 or 3.11, which should give equivalent results. However, one depends on the measured headwater (based on upstream depth) while the other depends on the measured discharge. These are independent measurements. Figure 4.7 compares the normalized velocity head values ($v_b^2/2gD$) determined from Equations 3.10 and 3.11 for all data resulting in submerged conditions (as determined by Equations 2.5 and 2.6). Generally, there is very good correspondence between the two methods for estimating the barrel velocity head. If anything, there is a tendency for velocity heads estimated from headwater (Equation 3.10) to be smaller than velocity heads estimated from discharge (Equation 3.11). The barrel velocity head calculated using Equation 3.11 is used to calculate the minor loss coefficient associated with SETs. This choice is made because both Equations 3.9 and 3.11 depend on the discharge, and the criteria from Equation 2.6 can be used consistently to distinguish unsubmerged from submerged conditions.

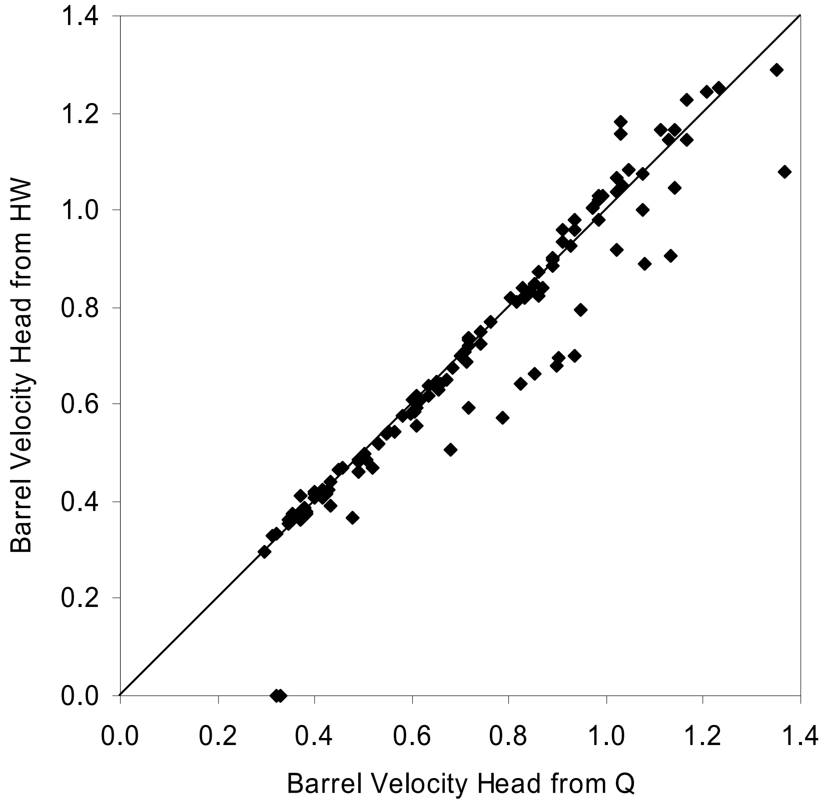


Figure 4.7: Comparison of velocity heads measured using Equations 3.10 and 3.11

The minor loss coefficients that are calculated using Equation 2.1 are shown in Figure 4.8. Except for a couple of values calculated for the 3:1 mitered configuration, all K_m values are greater than zero and most are less than 0.05. The values less than zero are for submerged conditions and are associated with the presence of a large vortex that dominates the hydraulic performance of the culvert entrance. Considering all of the data as one population, there does not appear to be a trend of increasing K_m-value with increasing headwater. This observation was confirmed by fitting a regression line through all of the data and testing the hypothesis that the slope was different from zero. Additional statistical tests were performed on the data set and are described in Trub (in press).

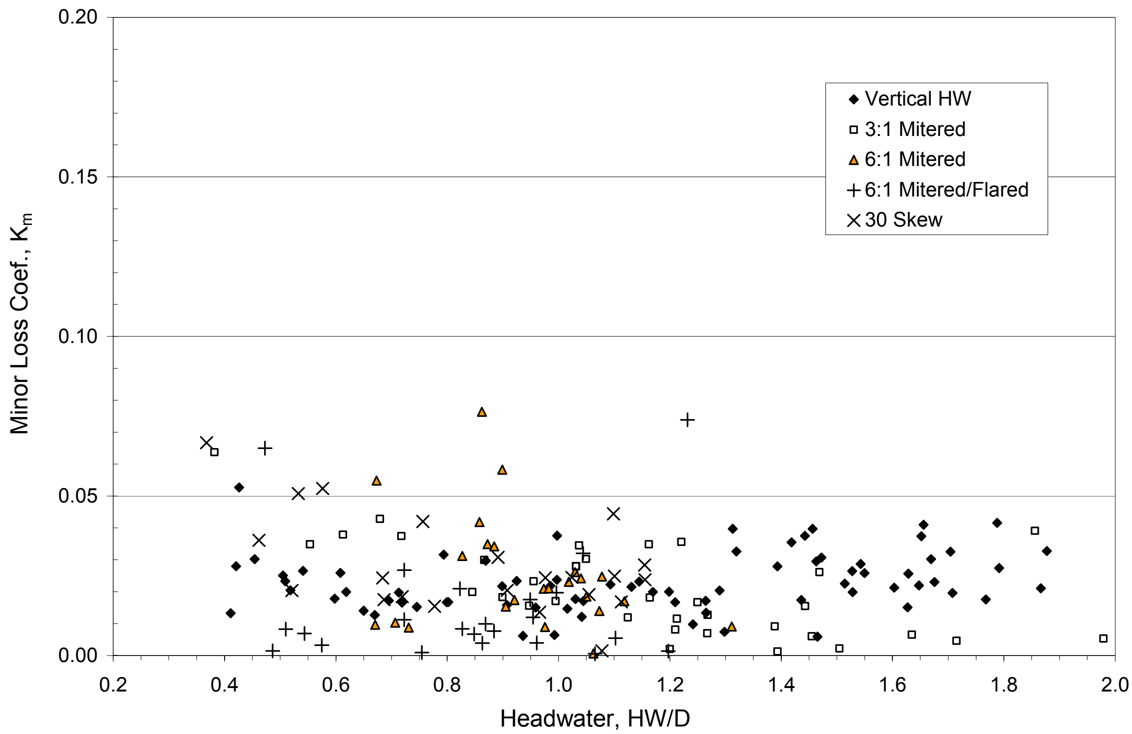


Figure 4.8: Minor loss coefficients associated with SETs for various culvert end configurations

The SET minor loss coefficients for culverts in general and for specific end configurations are presented in Table 4.4, along with a measure of their uncertainty. As a measure of uncertainty, the standard deviation of K_m-values is used. The values for the general culvert consider all of the data as one population. In calculation of the average value and standard deviation for the 3:1 mitered configuration, the negative values of K_m were excluded. Overall, there is a small variation in SET K_m-values between the different end configurations, and a representative value of K_m = 0.021 is probably appropriate for most SET applications.

Table 4.4: SET minor loss coefficients and variability

	K_m	Std. Dev.
General Culverts	0.021	0.0151
Vertical Headwall	0.023	0.0091
3:1 Mitered	0.021	0.0145
6:1 Mitered	0.026	0.0173
6:1 Mitered/Flared	0.014	0.0190
30 Deg. Skew	0.023	0.0146

According to the TxDOT Hydraulic Manual (2002), the entrance loss coefficient for a reinforced box culvert with wingwalls parallel (extension of sides) with a square-edge at crown is 0.7, and for a reinforced box culvert with wingwall at 10 to 25 degrees to barrel square-edged at crown is 0.5. The TxDOT Hydraulic Manual (traffic safety section) also states that mitered end sections should be used carefully because they may increase hydraulic head losses. Compared with these values, the minor loss coefficient values calculated for SETs can be considered insignificant.

5. SUMMARY AND CONCLUSIONS

This chapter presents a summary and conclusions of the research contained in this report. Section 5.1 contains a summary of the approach and progress made on the research objectives. Section 5.2 presents conclusions, while Section 5.3 discusses the implications of this research.

5.1 SUMMARY

The overall goal of this research is to evaluate the hydraulic effects of safety end treatments (SETs) on culverts through physical modeling and to provide the Texas Department of Transportation (TxDOT) with guidance on the influence of SETs in the hydraulic design of culverts. Important specific research objectives are the following:

1. To study the nature of water level differences upstream of the culvert due to SET presence.
2. To evaluate and compare the headwater-discharge relationships (performance curves) for different end configurations with culverts operating under inlet control, and to compare them to performance curves developed from earlier research reported in "Hydraulics of Channel Expansions Leading to Low-Head Culverts" (Charbeneau et al. 2002).
3. To provide minor loss coefficients (K_m) due to the presence of SETs that may be used in design procedures.

To meet the research objectives, physical models of a single-barrel box culvert (prototype dimensions of 10 feet by 6 feet) were constructed and studied on a scale of 1:6. The configurations included (1) a vertical headwall model configuration, (2) a 3:1 mitered headwall model configuration with a 0-degree skew angle, (3) a 3:1 mitered headwall model configuration with a 30-degree skew angle, (4) a 6:1 mitered embankment with curb headwall, and (5) a 6:1 mitered embankment with curb headwall and 15-degree flare. Conditions with and without SETs installed at the inlet end of the culvert model were examined. Experiments were performed under inlet control for both submerged flow and unsubmerged flow. Discharge and water level measurements were acquired for various scenarios. The collected data (discharge and water level) were utilized to evaluate culvert performance parameters and SET minor loss coefficients.

5.2 CONCLUSIONS

The following conclusions were made based on the research conducted and presented in this investigation.

The conclusions related to Objective 1 are the following:

- The water level can be taken as approximately uniform across the channel.
- The water level increases when there are SETs present. In general, the water level difference increases as the discharge under submerged conditions increases. The water level difference remains approximately constant for unsubmerged conditions.
- When the vortex phenomenon is present, the headwater level is typically less than when the vortex is not present for the same discharge. In fact, results obtained from this experimental program indicate the impact of

SETs on the water levels is significantly less than the impact of the vortex phenomenon. When a strong vortex is present, the velocity field is strongly multidimensional. Minor loss coefficients are used in a one-dimensional analysis of a flow system. These experiments show that the multidimensional effects of the vortex are equally, if not more significant than the one-dimensional effects of SETs on upstream water levels.

- Values for the range of data collected indicate a less than 3 percent increase in water level due to the presence of the SET model component. Therefore, it seems that the overall impact of SETs on water level is small for the range of experiments performed for this research.

The conclusions related to Objective 2 are the following:

- The inlet control equation describes the flow under both conditions, with and without SETs.
- When the differences in the performance relationship for the same model configuration due to SET presence are compared, the overall decrease in the performance due to SET presence can be considered minor.
- There is an apparent decrease in overall culvert performance for the five culvert configurations of the current research when compared to the culvert configuration of the previous research (box culvert studies reported in Charbeneau et al. 2002), especially under submerged conditions.
- The culvert inlet configurations studied in this research have a greater impact on culvert performance than SET presence. This is especially true under submerged conditions. This conclusion is consistent with the constants used for inlet control design equations presented by the Federal Highway Administration (FHWA) (Normann et al. 1985).

The conclusions related to Objective 3 are the following:

- The individual experiment SET minor loss coefficients range from less than zero to 0.076, and average approximately 0.021 for the entire data set, representing a value for general culverts (Table 4.4). The barrel minor loss coefficient due to the presence of SETs is very small; therefore, the overall impact of SETs on culvert performance is small.
- The SET minor loss coefficients do not vary with headwater or discharge.

5.3 IMPLICATIONS

The overall conclusion of this research is that the impacts of SETs on culvert performance is small and may not be significant for most applications. Retrofitting of existing culverts with SETs should not significantly impact culvert hydraulic performance, unless SETs become clogged with debris.

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APPENDIX A: DATA TABLES

Table A.1
Data Summary for Vertical Headwall Configuration

Date	Discharge	Condition	Average depth w/ SET (ft)	Average depth w/o SET (ft)	K_m	$Q/[A(gD)^{0.5}]$	E/D
08/12/03	1.296	unsubmerged	0.4190	0.4149	0.0280	0.1368	0.4210
07/28/03	1.307	unsubmerged	0.4278	0.4201	0.0526	0.1379	0.4261
08/01/03	1.458	unsubmerged	0.4524	0.4477	0.0302	0.1539	0.4543
08/01/03	1.708	unsubmerged	0.5058	0.5018	0.0233	0.1802	0.5089
07/31/03	1.743	unsubmerged	0.5149	0.5113	0.0205	0.1839	0.5185
05/21/03	1.801	unsubmerged	0.4006	0.3981	0.0133	0.1900	0.4109
08/12/03	1.897	unsubmerged	0.5382	0.5332	0.0265	0.2002	0.5411
08/01/03	2.144	unsubmerged	0.5934	0.5897	0.0178	0.2263	0.5980
08/13/03	2.169	unsubmerged	0.6149	0.6108	0.0199	0.2289	0.6186
08/15/03	2.246	unsubmerged	0.6049	0.5995	0.0259	0.2370	0.6082
05/21/03	2.269	unsubmerged	0.4972	0.4918	0.0251	0.2394	0.5050
08/14/03	2.601	unsubmerged	0.6633	0.6603	0.0127	0.2744	0.6700
07/28/03	2.696	unsubmerged	0.6897	0.6857	0.0172	0.2844	0.6953
08/13/03	2.847	unsubmerged	0.7128	0.7087	0.0166	0.3004	0.7188
08/14/03	2.973	unsubmerged	0.7390	0.7352	0.0152	0.3137	0.7454
05/21/03	3.001	unsubmerged	0.6395	0.6359	0.0140	0.3167	0.6497
08/14/03	3.289	unsubmerged	0.7958	0.7913	0.0168	0.3471	0.8020
07/23/03	3.304	unsubmerged	0.7927	0.7881	0.0167	0.3487	0.7991
05/21/03	3.311	unsubmerged	0.7047	0.6992	0.0198	0.3494	0.7132
07/31/03	3.319	unsubmerged	0.7913	0.7827	0.0316	0.3502	0.7939
08/13/03	3.877	unsubmerged	0.9006	0.8958	0.0158	0.4091	0.9074
07/29/03	3.892	unsubmerged	0.8933	0.8868	0.0217	0.4108	0.8987
05/21/03	4.189	unsubmerged	0.8639	0.8544	0.0297	0.4420	0.8693
06/04/03	4.315	unsubmerged	0.9243	0.9223	0.0062	0.4553	0.9359
06/09/03	4.329	unsubmerged	0.9508	0.9458	0.0150	0.4568	0.9588
06/03/03	4.372	unsubmerged	0.9184	0.9105	0.0234	0.4613	0.9248
06/06/03	4.530	unsubmerged	0.9797	0.9718	0.0218	0.4781	0.9853
08/12/03	4.531	unsubmerged	0.9926	0.9840	0.0237	0.4781	0.9972
06/09/03	4.633	unsubmerged	0.9978	0.9836	0.0376	0.4889	0.9974
07/29/03	4.812	unsubmerged	1.0238	1.0166	0.0177	0.5078	1.0305
07/30/03	4.829	unsubmerged	1.0331	1.0281	0.0121	0.5096	1.0418
08/05/03	4.829	unsubmerged	1.0379	1.0309	0.0171	0.5096	1.0445
05/20/03	5.025	submerged	1.0069	1.0003	0.0147	0.5302	1.0160
08/05/03	5.151	submerged	1.0897	1.0793	0.0222	0.5435	1.0934
05/13/03	5.274	submerged	0.9777	0.9745	0.0064	0.5566	0.9927
08/04/03	5.568	submerged	1.1426	1.1299	0.0232	0.5876	1.1450
08/05/03	5.728	submerged	1.1659	1.1544	0.0199	0.6044	1.1697
08/15/03	5.943	submerged	1.2046	1.1942	0.0167	0.6271	1.2096
07/23/03	6.070	submerged	1.2326	1.2263	0.0098	0.6405	1.2415
07/30/03	6.070	submerged	1.2609	1.2498	0.0171	0.6405	1.2645
08/06/03	6.197	submerged	1.2587	1.2496	0.0134	0.6540	1.2649
05/23/03	6.288	submerged	1.1263	1.1112	0.0215	0.6636	1.1311
07/30/03	6.308	submerged	1.2883	1.2831	0.0074	0.6656	1.2981
08/15/03	6.437	submerged	1.3267	1.2978	0.0397	0.6793	1.3131
07/31/03	6.661	submerged	1.4003	1.3785	0.0279	0.7029	1.3930
05/23/03	6.768	submerged	1.1944	1.1781	0.0200	0.7142	1.1986
07/25/03	6.888	submerged	1.4358	1.4213	0.0174	0.7269	1.4359
08/06/03	6.946	submerged	1.4559	1.4509	0.0059	0.7329	1.4652
07/28/03	6.965	submerged	1.4598	1.4279	0.0375	0.7349	1.4426
08/04/03	7.003	submerged	1.4758	1.4417	0.0397	0.7390	1.4563
07/23/03	7.041	submerged	1.4844	1.4577	0.0307	0.7430	1.4722
05/22/03	7.044	submerged	1.2881	1.2702	0.0203	0.7433	1.2893
06/09/03	7.118	submerged	1.4740	1.4478	0.0295	0.7511	1.4628
08/06/03	7.195	submerged	1.5376	1.5136	0.0265	0.7593	1.5276
05/22/03	7.249	submerged	1.3304	1.3001	0.0326	0.7650	1.3194
07/24/03	7.351	submerged	1.5600	1.5355	0.0258	0.7757	1.5498
05/20/03	7.420	submerged	1.4356	1.4012	0.0355	0.7830	1.4186
08/08/03	7.527	submerged	1.6403	1.6149	0.0256	0.7943	1.6284
08/08/03	7.527	submerged	1.6403	1.6149	0.0256	0.7943	1.6284
08/04/03	7.566	submerged	1.6104	1.5891	0.0212	0.7984	1.6032
08/07/03	7.566	submerged	1.6559	1.6339	0.0220	0.7984	1.6473
08/11/03	7.566	submerged	1.6760	1.6387	0.0374	0.7984	1.6519
08/07/03	7.606	submerged	1.6839	1.6425	0.0410	0.8026	1.6559
06/04/03	7.709	submerged	1.5568	1.5269	0.0287	0.8135	1.5428
07/25/03	7.764	submerged	1.6860	1.6618	0.0230	0.8193	1.6753
08/11/03	7.804	submerged	1.7151	1.6943	0.0196	0.8235	1.7075
06/10/03	7.905	submerged	1.6285	1.6120	0.0151	0.8342	1.6269
05/23/03	7.925	submerged	1.5215	1.4966	0.0225	0.8363	1.5140
08/08/03	8.044	submerged	1.8219	1.7751	0.0415	0.8488	1.7878
08/11/03	8.104	submerged	1.7744	1.7543	0.0176	0.8552	1.7675
05/22/03	8.124	submerged	1.5335	1.5104	0.0199	0.8573	1.5284
06/12/03	8.144	submerged	1.6894	1.6543	0.0302	0.8594	1.6694
07/28/03	8.145	submerged	1.8102	1.7785	0.0274	0.8595	1.7915
07/25/03	8.389	submerged	1.8794	1.8536	0.0210	0.8852	1.8663
08/07/03	8.471	submerged	1.9054	1.8644	0.0328	0.8939	1.8773
05/23/03	8.927	submerged	1.7321	1.6866	0.0325	0.9421	1.7040

Table A.2
Data Summary for 3:1 Mitered Headwall Configuration

Note: Submerged condition is before anti-vortex plate (fin) was developed.

Date	Discharge	Condition	Average depth w/ SET (ft)	Average depth w/o SET (ft)	K_m	$Q/[A(gD)^{0.5}]$	E/D
09/10/03	1.141	unsubmerged	0.3848	0.3762	0.0637	0.1206	0.3819
10/17/03	1.536	unsubmerged	0.5544	0.5488	0.0348	0.1624	0.5536
09/26/03	2.208	unsubmerged	0.6127	0.6048	0.0379	0.2334	0.6130
10/10/03	2.587	unsubmerged	0.6800	0.6701	0.0428	0.2736	0.6793
10/07/03	2.750	unsubmerged	0.7175	0.7084	0.0374	0.2908	0.7178
10/13/03	3.496	unsubmerged	0.8401	0.8345	0.0199	0.3697	0.8454
09/05/03	3.708	unsubmerged	0.8639	0.8549	0.0300	0.3920	0.8666
09/25/03	3.831	unsubmerged	0.8937	0.8878	0.0183	0.4050	0.8993
09/26/03	4.207	unsubmerged	0.9522	0.9431	0.0233	0.4449	0.9554
09/05/03	4.223	unsubmerged	0.9408	0.9347	0.0156	0.4465	0.9474
09/26/03	4.465	unsubmerged	0.9893	0.9819	0.0171	0.4721	0.9947
09/04/03	4.779	unsubmerged	1.0320	1.0179	0.0280	0.5053	1.0316
09/04/03	4.779	unsubmerged	1.0320	1.0179	0.0280	0.5053	1.0316
10/07/03	4.779	unsubmerged	1.0515	1.0363	0.0303	0.5053	1.0495
09/09/03	4.913	unsubmerged	1.0410	1.0227	0.0345	0.5195	1.0370
10/17/03	5.341	submerged with fin	1.1177	1.1102	0.0119	0.5647	1.1246
10/17/03	5.341	submerged without fin	1.1621	1.1508	0.0181	0.5647	1.1642
10/08/03	5.533	submerged with fin	1.2020	1.1966	0.0081	0.5850	1.2099
10/08/03	5.533	submerged without fin	1.1878	1.1864	0.0020	0.5850	1.1999
10/20/03	5.603	submerged	1.2317	1.2073	0.0356	0.5925	1.2206
10/22/03	5.710	submerged	1.2594	1.2544	0.0070	0.6037	1.2673
09/08/03	5.781	submerged	1.1721	1.1465	0.0348	0.6113	1.1623
10/10/03	6.124	submerged	1.3837	1.3762	0.0091	0.6476	1.3885
09/08/03	6.197	submerged	1.2059	1.1961	0.0115	0.6553	1.2128
10/15/03	6.345	submerged with fin	1.3816	1.3805	0.0013	0.6708	1.3936
10/15/03	6.345	submerged without fin	1.4444	1.4308	0.0155	0.6708	1.4431
10/07/03	6.437	submerged with fin	1.4803	1.4567	0.0261	0.6806	1.4688
10/07/03	6.437	submerged without fin	1.4481	1.4426	0.0060	0.6806	1.4550
09/08/03	6.474	submerged	1.2481	1.2327	0.0167	0.6846	1.2498
10/16/03	6.512	submerged with fin	1.5059	1.5157	-0.0106	0.6885	1.5271
10/16/03	6.512	submerged without fin	1.4950	1.4930	0.0022	0.6885	1.5048
09/08/03	6.605	submerged	1.2628	1.2506	0.0127	0.6984	1.2679
10/01/03	6.605	submerged with fin	1.5268	1.5363	-0.0100	0.6984	1.5478
10/01/03	6.605	submerged without fin	1.5070	1.5186	-0.0122	0.6984	1.5303
10/01/03	6.907	submerged with fin	1.6304	1.6236	0.0066	0.7304	1.6348
10/01/03	6.907	submerged without fin	1.5891	1.5961	-0.0068	0.7304	1.6078
10/06/03	6.927	submerged with fin	1.7313	1.7397	-0.0081	0.7324	1.7495
10/06/03	6.927	submerged without fin	1.7132	1.7181	-0.0048	0.7324	1.7282
09/26/03	7.080	submerged	1.6299	1.6313	-0.0012	0.7486	1.6430
10/03/03	7.370	submerged with fin	1.7680	1.7853	-0.0146	0.7793	1.7959
10/03/03	7.370	submerged without fin	1.7086	1.7032	0.0046	0.7793	1.7148
10/02/03	7.944	submerged with fin	1.9762	1.9689	0.0053	0.8399	1.9790
10/02/03	7.944	submerged without fin	1.8980	1.8444	0.0391	0.8399	1.8560

Table A.3
Data Summary for 3:1 Mitred 30-Degree Skew Configuration

Date	Discharge	Condition	Average depth w/ SET (ft)	Average depth w/o SET (ft)	K_m	$Q/[A(gD)^{0.5}]$	E/D
03/03/04	0.925	unsubmerged	0.3721	0.3637	0.0667	0.0978	0.3677
03/03/04	1.436	unsubmerged	0.4619	0.4558	0.0361	0.1519	0.4620
02/09/04	1.559	unsubmerged	0.5191	0.5154	0.0204	0.1648	0.5211
01/23/04	1.570	unsubmerged	0.5363	0.5272	0.0508	0.1660	0.5327
02/05/04	1.731	unsubmerged	0.5803	0.5703	0.0524	0.1830	0.5760
02/16/04	2.336	unsubmerged	0.6823	0.6766	0.0243	0.2470	0.6841
03/03/04	2.336	unsubmerged	0.6835	0.6794	0.0175	0.2470	0.6867
02/09/04	2.520	unsubmerged	0.7159	0.7113	0.0185	0.2665	0.7191
02/05/04	2.737	unsubmerged	0.7590	0.7481	0.0420	0.2894	0.7564
01/23/04	2.819	unsubmerged	0.7730	0.7689	0.0154	0.2981	0.7773
02/19/04	3.333	unsubmerged	0.8916	0.8825	0.0308	0.3525	0.8914
02/17/04	3.586	unsubmerged	0.9043	0.8980	0.0205	0.3792	0.9079
03/01/04	3.861	unsubmerged	0.9608	0.9560	0.0136	0.4083	0.9661
03/02/04	3.877	unsubmerged	0.9751	0.9664	0.0243	0.4099	0.9764
02/06/04	4.017	unsubmerged	1.0234	1.0141	0.0245	0.4248	1.0238
02/18/04	4.319	unsubmerged	1.0526	1.0442	0.0191	0.4567	1.0548
01/27/04	4.416	unsubmerged	1.0680	1.0674	0.0015	0.4670	1.0780
01/23/04	4.613	unsubmerged	1.1020	1.0896	0.0249	0.4877	1.1007
01/28/04	4.662	unsubmerged	1.1102	1.1016	0.0168	0.4930	1.1127
02/05/04	4.679	unsubmerged	1.1100	1.0871	0.0444	0.4947	1.0986
03/02/04	4.879	unsubmerged	1.1592	1.1433	0.0283	0.5159	1.1546
02/17/04	4.896	unsubmerged	1.1574	1.1440	0.0237	0.5177	1.1554
02/18/04	5.014	submerged	1.1941	1.1794	0.0249	0.5302	1.1906
03/01/04	5.082	submerged	1.2050	1.1837	0.0351	0.5374	1.1951
01/27/04	5.117	submerged	1.2293	1.2145	0.0240	0.5410	1.2255
02/09/04	5.134	submerged	1.2070	1.2018	0.0084	0.5428	1.2132
02/09/04	5.306	submerged	1.2617	1.2497	0.0182	0.5610	1.2609
02/16/04	5.533	submerged	1.3186	1.3088	0.0136	0.5850	1.3199
02/16/04	5.533	submerged	1.3218	1.3120	0.0136	0.5850	1.3231
02/16/04	5.568	submerged	1.3401	1.3365	0.0050	0.5888	1.3473
02/18/04	5.568	submerged	1.3456	1.3401	0.0075	0.5888	1.3509
03/03/04	5.568	submerged	1.3283	1.3229	0.0075	0.5888	1.3339
01/28/04	5.674	submerged	1.3410	1.3283	0.0168	0.6000	1.3397
02/17/04	5.674	submerged	1.3547	1.3503	0.0057	0.6000	1.3613

Table A.4
Data Summary for 6:1 Mitred Headwall Configuration

Date	Discharge	Condition	Average Depth w/ SET (ft)	Average Depth w/o SET (ft)	K_m	$Q/[A(gD)^{0.5}]$	E/D
6/21/2004	1.9704	unsubmerged	0.5837	0.5754	0.0420	0.2083	0.5827
7/13/2004	2.0443	unsubmerged	0.6061	0.5989	0.0360	0.2162	0.6061
7/1/2004	2.1948	unsubmerged	0.5986	0.5927	0.0278	0.2321	0.6013
6/21/2004	2.4473	unsubmerged	0.6773	0.6649	0.0548	0.2588	0.6733
6/28/2004	2.5469	unsubmerged	0.6637	0.6615	0.0097	0.2693	0.6707
7/6/2004	2.6819	unsubmerged	0.6998	0.6973	0.0103	0.2836	0.7065
6/21/2004	2.7571	unsubmerged	0.7239	0.7218	0.0087	0.2915	0.7308
7/13/2004	3.3923	unsubmerged	0.8254	0.8166	0.0312	0.3587	0.8273
7/7/2004	3.5865	unsubmerged	0.8737	0.8513	0.0764	0.3792	0.8623
7/3/2004	3.6621	unsubmerged	0.8587	0.8463	0.0418	0.3872	0.8579
7/7/2004	3.6773	unsubmerged	0.8714	0.8610	0.0349	0.3888	0.8723
7/12/2004	3.6773	unsubmerged	0.8837	0.8735	0.0342	0.3888	0.8845
6/30/2004	3.9080	unsubmerged	0.9050	0.8869	0.0582	0.4132	0.8989
7/1/2004	4.0488	unsubmerged	0.8974	0.8925	0.0153	0.4281	0.9053
6/28/2004	4.0645	unsubmerged	0.9138	0.9083	0.0174	0.4298	0.9207
6/22/2004	4.2472	unsubmerged	0.9664	0.9635	0.0090	0.4491	0.9756
7/3/2004	4.3679	unsubmerged	0.9678	0.9608	0.0210	0.4618	0.9736
6/28/2004	4.4653	unsubmerged	0.9765	0.9694	0.0210	0.4721	0.9826
7/3/2004	4.5470	unsubmerged	1.0138	1.0059	0.0231	0.4808	1.0186
7/8/2004	4.6128	unsubmerged	1.0361	1.0277	0.0242	0.4877	1.0402
7/13/2004	4.6128	unsubmerged	1.0261	1.0170	0.0261	0.4877	1.0298
7/7/2004	4.7453	unsubmerged	1.0503	1.0501	0.0006	0.5017	1.0628
7/6/2004	4.7953	unsubmerged	1.0433	1.0367	0.0184	0.5070	1.0500
6/22/2004	5.0144	partially submerged	1.0649	1.0595	0.0140	0.5302	1.0734
6/22/2004	5.0399	partially submerged	1.0735	1.0639	0.0247	0.5329	1.0779
6/30/2004	5.3060	submerged	1.1111	1.1039	0.0170	0.5610	1.1182
7/8/2004	6.4186	max submerged	1.3016	1.2959	0.0091	0.6787	1.3112

Table A.5
Data Summary for 6:1 Mitred Flared Configuration

Date	Discharge	Condition	Average Depth w/ SET (ft)	Average Depth w/o SET (ft)	Km	$Q/[A(gD)^{0.5}]$	E/D
8/3/2004	0.474	unsubmerged	0.477	0.472	0.0650	0.0502	0.4727
8/4/2004	1.447	unsubmerged	0.481	0.481	0.0014	0.1530	0.4866
8/4/2004	1.559	unsubmerged	0.506	0.504	0.0082	0.1648	0.5102
8/5/2004	1.995	unsubmerged	0.536	0.535	0.0069	0.2109	0.5433
8/5/2004	2.195	unsubmerged	0.566	0.565	0.0032	0.2321	0.5747
8/4/2004	2.628	unsubmerged	0.747	0.747	0.0010	0.2778	0.7546
8/4/2004	2.805	unsubmerged	0.720	0.713	0.0268	0.2966	0.7228
8/5/2004	3.073	unsubmerged	0.714	0.711	0.0112	0.3249	0.7229
8/3/2004	3.437	unsubmerged	0.817	0.812	0.0209	0.3634	0.8227
8/3/2004	3.693	unsubmerged	0.852	0.851	0.0038	0.3904	0.8629
8/5/2004	3.708	unsubmerged	0.817	0.814	0.0084	0.3920	0.8270
8/9/2004	3.708	unsubmerged	0.875	0.873	0.0077	0.3920	0.8842
8/4/2004	3.738	unsubmerged	0.860	0.857	0.0099	0.3953	0.8687
8/2/2004	3.815	partially submerged	0.838	0.836	0.0067	0.4034	0.8486
8/2/2004	3.831	partially submerged	0.829	0.829	-0.0030	0.4050	0.8427
8/9/2004	4.002	partially submerged	0.943	0.938	0.0175	0.4231	0.9490
8/3/2004	4.191	partially submerged	0.946	0.942	0.0120	0.4432	0.9543
8/5/2004	4.629	partially submerged	0.947	0.946	0.0039	0.4895	0.9607
8/5/2004	4.879	partially submerged	0.988	0.981	0.0197	0.5159	0.9963
8/9/2004	5.014	submerged	1.039	1.040	-0.0044	0.5302	1.0548
8/2/2004	5.254	submerged	1.040	1.028	0.0320	0.5555	1.0441
8/3/2004	5.306	submerged	1.090	1.088	0.0054	0.5610	1.1023
8/2/2004	5.375	submerged	1.049	1.049	0.0006	0.5684	1.0655
8/9/2004	6.124	submerged	1.251	1.216	0.0739	0.6476	1.2316
8/2/2004	6.197	submerged	1.181	1.180	0.0014	0.6553	1.1972