PRACTICAL APPLICATION OF

HYDRAULIC TOOLS IN

URBAN STORMWATER DESIGN

By

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CHAPTER I

INTRODUCTION

Background

The design of urban stormwater channels involves applying the appropriate hydraulic tools to the problem parameters. These tools range from hand-written equations to complex computer algorithms. With today's advances in computer technology, it is expected that engineers will chose its speed and proven computational abilities over the cumbersome, time-consuming pen and paper methods.

To properly design a stormwater runoff system, the design engineer must use quality engineering judgment as well as the appropriate set of tools. The first judgment often made is identifying the appropriate technology for the problem set. For small and medium sized communities whose engineering staff performs limited design projects, this decision can include software that, although not the most technologically advanced, is known to provide acceptable results and is familiar to the engineer.

The Rolling Acres Subdivision is located in Enid, Oklahoma and was developed outside the city limits in 1962. There were no storm water regulations to consider and, as with many developments, the developer's objective probably conflicted with that of the engineer. The developer is trying to utilize as much property as possible from a profitability perspective. He hopes to minimize any improvement that will reduce the size or number of sellable lots. Two primary parameters that shape this conflict from a storm water channel perspective include width and surface type. These are also important factors to the design engineer. With a truly unbiased opinion, the engineer must evaluate all parameters using his best judgment and the appropriate hydraulic tools. However, both the developer and the engineer understand that the local reviewing agency will review the computations for appropriateness. It is, therefore, imperative that the design engineer uses the appropriate hydraulic tools so that an acceptable solution can be presented. When this approach is applied to this study, it will result in a channel and culvert design that will meet the criteria required by the local governing body.

As people decide to trade their compacted city lifestyle for larger, yet low-density, living spaces on the urban fringe, a development concept known as urban sprawl can become the unintentional default plan for local governing bodies (Daniels, 1999). This concept facilitates large developments that include commercial districts as well as residential subdivisions. Local planners are pressured into approving these site plans as the realization of an increase in revenue, or the loss thereof for not doing so, becomes closer to fruition. As these new developments begin, they can be located in what seems to be a significant distance from existing, nearby developments that may be considered rural. However, city limits expand and urban sprawl is allowed to advance.

Even if review policies advance, there is the natural tendency to apply past regulations to proposed development. Such is the case for stormwater runoff (Haase and Nuissl, 2007). During the early years of urban sprawl, the impacts of these developments are not far reaching. The nearby, once rural residents are only thinking about their life being more efficient with retail businesses getting closer. Review committees concentrate on the local controversies associated with the current development submittal, unintentionally ignoring the issue that only becomes a problem miles downstream.

Eventually, a storm event occurs that forces runoff to escape the now-too-small channel and enters the garage or patio door of the nearest home. This quickly prompts a telephone call to the local government and the mitigation process is set into motion. This process involves formulating the scope of work, deciding the priority, appropriating the funding, designing the solution, and constructing the improvement.

At this point, the more fiduciary responsible design involves sizing a channel that will offset the current development runoff while planning for how future development will compensate for its own impact. The local government has to weigh such options as adjacent-stream property purchase (often located in known flood-prone areas) versus on-site detention regulations imposed on future development. On-site detention facilities are constructed by the developer on his property that will offset the amount of increased runoff created by his development. Conversely, regional detention is constructed by the local government in a few strategic locations throughout the city using funds collected by developers at the time of their development. The design for the municipality that is governed by regional instead of on-site detention must compensate for fully developed upstream conditions (McLaughlin, 1997).

Objective of the Study

The objective of this study is to test the appropriate technology to design a storm water channel and culverts in the Rolling Acres Subdivision that will properly convey the fully developed conditions of the basin. Although technology will continue to improve the speed, user friendliness, and output options of computer software, it is less likely that they will result in much more accurate results. The principles of Manning's equation as well as gradually varied flow and culvert hydraulics have been proven over many years and will continue to function as the basic theory of future software. The technology that can provide results that properly convey these conditions will be the technology that is capable of sizing channels and culverts that protect adjacent houses from flooding.

CHAPTER II

REVIEW OF LITERATURE

Urban Runoff

An urban drainage system is an ensemble of structural elements whose purpose is to provide a defined pathway for efficient stormwater runoff collection and conveyance. These structural elements typically include open channels, detention ponds, culverts, and street inlets. As development occurs, the footprint of permeable ground surface decreases, resulting in greater amounts of stormwater runoff. Parking lots, streets, and rooftops are all contributors to this increase in impervious surface. They bring about a dramatic increase in flow volumes, peak flow, and flooding, often overwhelming the natural channels. The urban drainage system must be designed to reduce the negative impacts of this runoff while providing access to business and home properties during minor storm events (Merritt, 1983).

Open Channel Flow Tools

The open channel element provides more flow capacity than a closed pipe system. Although it is more difficult to analyze than the closed pipe, this also makes the open channel design more interesting (Bentley, 2007). There are many variables that must be determined as part of the solution. Its cross-sectional area is often large and inconsistent. While it may be prismatic just after construction, erosion, deposition, and other factors can slowly alter the channel geometry. Because this process can take many years, constructed channels may be considered prismatic for analysis purposes (Bentley, 2007). Another variable is the material that is used for the channel surface. The chosen material can make a significant impact on the flow capacity of the channel. The roughness of this surface varies from a smooth concrete surface to a channel overgrown with tall weeds, brush, and trees.

Many formulas have been developed and published for solving open channel flow conditions. Perhaps the first formula proposed was developed by Antoine Chezy, a French engineer, in 1769. This formula uses two primary assumptions for its derivation. The first assumption is that the force resisting the flow is proportional to the square of the velocity (Chow, 1959). The second assumption is based on the basic principle of uniform flow. The component of the gravity force that is parallel to the channel bottom must be equal and opposite of the total resistive force.

The Irish engineer Robert Manning developed another popular formula, first proposed in 1889, the Manning formula for open channel flow. With no formal engineering education, the practicing engineer simply calculated the average velocity for several conditions using the seven most popular formulas of that time. It has been suggested that the formula be known as the Gauckler-Manning formula, giving recognition to another engineer of that time, Philippe-Gaspard Gauckler, who separately developed a similar formula (Anderson, 2002).

The Manning formula holds its indisputable top position in the field of practical applications despite many new theoretical developments (Chow, 1959). Its simple form and ability to produce results similar to the other uniform flow formulas have made it the formula of choice for practicing engineers. The scientific community has recommended this formula for international use since 1933. Such organizations as the United States Department of Agriculture, United States Army Corp of Engineers, National Resources Conservation Service (NRCS, Open Channel, Code 582), and the Federal Emergency Management Agency have adopted technical bulletins requiring the use of this formula. Numerous local and state agencies also require the use of this formula in predicting channel flow and velocity.

Drainage Culvert Tools

Culverts can be evaluated based on their performance curve. This curve describes how headwater changes with respect to discharge. Curves are usually prepared separately for submerged and unsubmerged inlet conditions, which can result in curves that do not overlap, creating uncertainty in the transition zone. Charbeneau (2006) developed a simple, two-parameter model that provides a smooth, clearly defined transition zone between the submerged and unsubmerged flow conditions when the culvert is influenced by inlet control. Using this model, additional equations can be developed that can calculate the culvert span for multiple-barrel, low-headwater box culverts. The developed equations indicate a 17% smaller span than predicted using the Federal Highway Administration equations and coefficients. Charbeneau (2006) considers this a substantial difference due to the non-uniform flow distribution between culvert barrels as wells as the increased tendency of sediment deposition in the outer barrels due to lower discharge velocity.

The head loss associated with the culvert exit can be the largest single system energy loss component of a short culvert (Tullis, 2008). According to manuals of the Federal Highway Administration's Hydraulic Design of Highway Culverts HDS-5 (HDS-5) (Normann, Houghtalen, and Johnston, 2001) as well as that of HEC-RAS, this exit loss is defined by the difference in velocity heads at the culvert exit and downstream channel or by multiplying the culvert velocity head by a loss coefficient. Both HDS-5 and HEC-RAS software are used to determine water surface elevations in channels and culverts. Because of the significant impact of the exit configuration under outlet control conditions, a prototype culvert with varying end treatments was studied under laboratory conditions. This experiment determined exit losses that were compared with losses calculated using traditional exit loss equations and the Borda-Carnot minor loss expression, traditionally used to determine energy loss at sudden expansions in pressurized pipe flow. The Borda-Carnot expression proved to be more accurate than traditional methods for the conditions tested (Tullis, 2008).

The review of literature indicates that the evaluation of hydraulic tools for stormwater management is missing for small and medium sized communities. Based on the results of this literature review, these tools should include FlowMaster and CulvertMaster.

CHAPTER III

HYDROLOGIC SETTING

Location

The study area is located in the north-central Oklahoma town of Enid. The region is primarily rural with many small towns with populations of 2,500 or less. The 2000 census indicates the population of Enid at 47,045 while Garfield County's population is 57,813. The predominant industries include farming, livestock, and oil and gas.

Specifically, the study area is located in the northernmost developed section of Enid in the Rolling Acres Subdivision, as shown in Figure 3-1. This subdivision was developed while outside the then-current city limits. Surrounding land use includes wheat cultivation and pasture to the north, residential to the west and south, and a mixture of parks, residential, and commercial to the east.

Basin Characteristics

The study basin is referred to as the Crosslin Park Basin because the discharge point is located in the city-owned park of the same name. It consists of approximately 1.0 square miles and is outlined in the aerial photograph of Figure 3-1. The headwaters are generated one-half mile north of the intersection of Oakwood Road and Purdue Avenue. The runoff is collected in the 35-acre Crosslin Lake and sent downstream into

North Boggy Creek, one of three main channels within city limits. This runoff from this basin ultimately reaches the Arkansas River via North Boggy Creek, Boggy Creek, Skeleton Creek and the Cimarron River. As with most small channels in this region, the study channel is commonly dry for brief periods as a result of seasonal variations in precipitation (Bingham, 1980).

The topography is considered gently rolling with an elevation range of 1280 to 1330 above mean sea level. The predominant soil groups in this basin are the Pratt (PtC) and Shellabarger (SrB) series (Garfield County Soil Survey, 1980). These soils have a brown, brittle loamy fine sand top layer that is about 14-inches thick. It is classified as highly permeable and highly susceptible to water and wind erosion. While this soil type mitigates much of the runoff, it lacks the important nutrients needed to support plant life, the absence of which only contributes to the erosion.

Study Channel Characteristics

A detailed location map of the study channel can be seen in Figure 3-2. The short, dashed line represents the existing channel and arrows indicate the flow direction. The existing channel is grass-lined with a 10-foot flat bottom and 3:1 side slopes. Two of its three culverts are a single barrel 10'W x 3'H reinforced concrete box (the existing Grant Street culvert is not shown). The third culvert, located at Crosslin Park Road, is comprised of two 36-inch reinforced concrete pipes. The proposed channel is shown as long, dashed lines, representing grass-lined sections, and continuous lines, representing concrete-lined sections. The proposed culverts are labeled C1 (Grant Street), C2 (Lincoln Street), and C3 (Crosslin Park Road) and graphically represented as green 3D rectangles.

This channel has been divided into sections A, B, C, D, and E based on flow characteristics and proximity to adjacent homes. The proposed channel intercepts the existing channel at the end of channel section B. The homes are labeled a1, a2, b1, b2, etc. and correspond to the adjacent channel section. The proposed channel sections and culverts are the elements that were modeled in this study.

Climate

The climate within the study area is continental (Arndt, 2003). Warm, moist air is brought in from the Gulf of Mexico along with the prevailing south winds. The strongest winds can be expected in March and April while the calmest are July, August, and September. The mean annual temperature is 58.3 degrees Fahrenheit (Oklahoma Climatological Survey, 2010). The mean annual precipitation is 34.3 inches. The mean annual snowfall is six to nine inches. Typical weather statistics can be seen in Table 3-1. The seasons are well defined with spring characterized by frequent precipitation, severe storms, and tornados (Swafford, 1967).

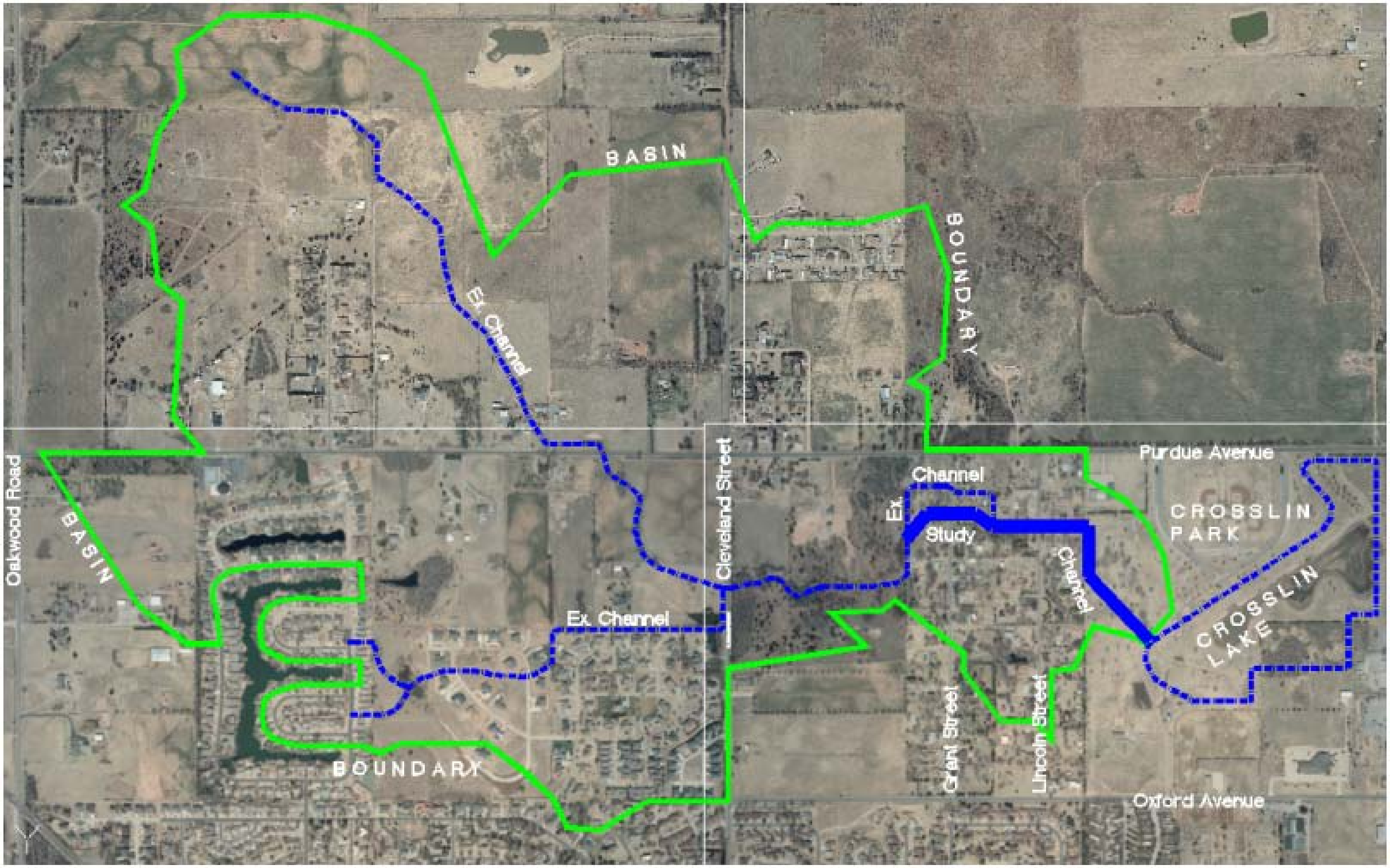


Figure 3-1 Study Channel Basin Boundary

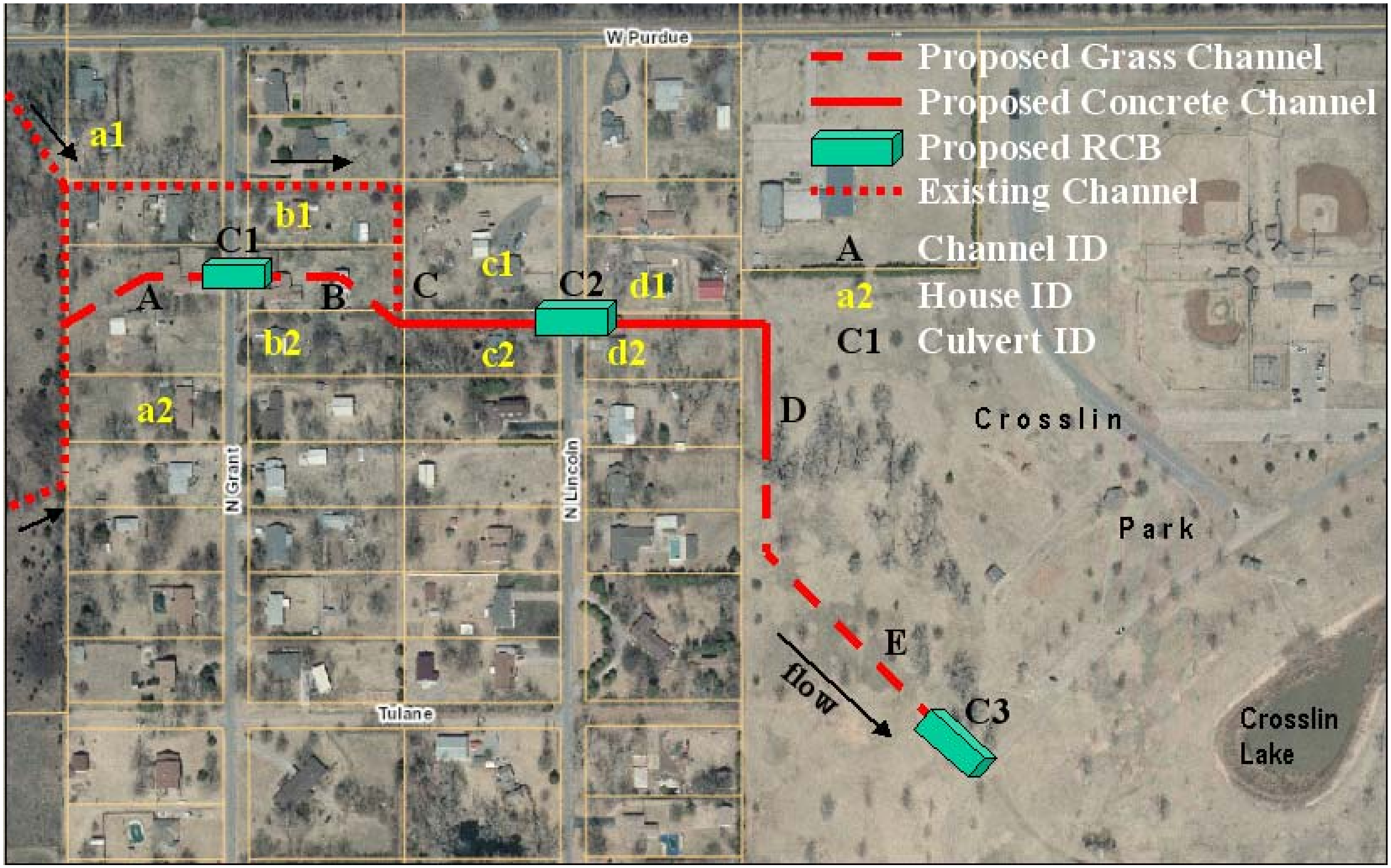


Figure 3-2 Study Channel Location Map

Table 3-1 Weather Statistics

		Temperature (^oF)		Precipitation (in.)	
Season	Month	Monthly	Average	Monthly	Average
	December	36.1		1.4	
Winter	January	33.1	35.9	1.1	1.4
	February	38.6		1.6	
	March	47.2		2.5	
Spring	April	57.3	57.4	3.2	3.5
	May	67.8		4.9	
	June	77.1		4.4	
Summer	July	82.6	80.2	2.8	3.5
	August	80.8		3.4	
	September	72.6		3.2	
Fall	October	60.5	59.7	3.4	3.0
	November	46.1		2.3	

Surface Water Flows

The surface water flows for this basin were developed in 2009 by Envirotech Engineering and Consulting (Envirotech, 2009). The unnamed channel and its tributaries were modeled using HEC-HMS software. The section of channel in this study receives flow from two tributaries. The north tributary is located in a primarily undeveloped region and, for example, contributes 338 cfs during the 100-year storm event. The south tributary is located in a heavily developed region containing both existing and proposed residential subdivisions with 1/4-acre lots. Table 3-2 shows the flows that occur throughout the study channel for the given storm recurrence intervals.

Table 3-2 Flowrates at Drainage Elements (Envirotech, 2009)

Drainage Elements	Q_{10} (cfs)	Q_{25} (cfs)	Q_{50} (cfs)	Q_{100} (cfs)
Channel A Grant Street Culvert - C1 Channel B	463	631	768	923
Channel C Lincoln Street Culvert - C2 Channel D	556	835	1,042	1,261
Channel E Crosslin Park Road Culvert - C3	566	848	1,064	1,290

CHAPTER IV

HYDRAULIC MODELS

FlowMaster

The principle use of Bentley's FlowMaster software is modeling steady, uniform flow in a prismatic channel. Recently, however, Bentley included the calculations for the gradually varied flow condition. Although the calculations necessary for this type of flow are more complex, they can be simplified with the assumption that the water pressures can be modeled as hydrostatic. One reason this is substantiated is because the differences in water surface profiles for gradually varied flow and uniform flow are small (Bentley, 2007).

Flow in a channel is considered steady when characteristics such as depth do not change at a specific point over a specified time interval (Chow, 1959). The depth and slope computed by the uniform flow formula is known as the normal depth and normal slope, respectively. It is also acceptable to assume a constant depth flow for applications in which the change in depth is small compared to the actual depth. Steady flow can be further differentiated into uniform flow or varied flow. Uniform flow occurs when those same characteristics (depth, velocity, discharge, area, etc.) do not change along the channel with respect to time. The general uniform flow equation can be described as follows (Bentley, 2007):

$$V = CR^xS^y \tag{4-1}$$

where

V = velocity (fps)

C = Flow resistance factor

R = Hydraulic radius (ft)

S = Energy slope (ft/ft)

x,y = Exponents

The flow resistance factor, C, is primarily a function of the roughness of the channel lining material. Other factors influencing this factor include channel shape, depth, and velocity. The hydraulic radius can be determined for any channel geometry by dividing the cross-sectional area by the wetted perimeter. For uniform flow condition, the energy slope can be assumed to equal that of the channel bottom.

In a practical sense, uniform flow can only exist when the channel cross sectional area does not change along the length of channel being analyzed. This type of channel is referred to as a prismatic channel. Varied flow occurs when the depth or velocity do change along the channel length. When these characteristics change slowly, the flow is described as gradually varied. Conversely, the flow is described as rapidly varied when depth and velocity change abruptly as in a hydraulic jump or flow over a weir.

Unsteady flow takes place when the depth or velocity does change at a point with respect to time. Wave action is an example of unsteady flow conditions. Most open channel problems do not exhibit unsteady flow behavior.

Uniform Flow

Because the uniform flow is comprised of steady flow conditions and discharge, equation 4.1 can be combined with the continuity equation

$$Q = VA \tag{4-2}$$

resulting in the equation

$$Q = ACR^{x}S^{y}$$

where

Q = Discharge (cfs)

A = Cross Sectional Area (sq. ft.)

There are many formulas used to solve the general uniform flow equation for open channel flow. These formulas differ from each other by the calculation of the flow resistance factor, C, and the values assigned to x and y. FlowMaster provides solutions based on formulas developed by Manning, Kutter, Hazen-Williams, and Darcy-Weisbach.

The formula chosen for this study is the Manning formula. Its calculation is simple and its results are considered satisfactory for practical applications (Chow, 1959). Its form is presented as

$$V = \frac{1.49}{n} R^{2/3} S^{1.5} \tag{4-3}$$

where

V = Velocity (fps)

n = Manning's Coefficient of Roughness

R = Hydraulic Radius (ft)

S = Friction Slope (ft/ft)

Today's computers are capable of computing even the most complex wetted perimeter for calculation of the hydraulic radius. As mentioned earlier, the friction slope is equal to the channel slope for uniform flow conditions. The most complicated factor in this equation is the determination of the roughness coefficient *n* (Chow, 1959). Because there is no exact method to select a value for *n*, the engineer must use sound engineering judgment to estimate the channel's resistance to flow. In practical applications, the value assigned to *n* can be better estimated by understanding the conditions that the flow will experience throughout the life of the channel. One factor to consider is the condition of the material that comprises the surface of the channel. For instance, material such as sand, silt, and clay are relatively small in diameter, allowing them to fit close together and form a relatively smooth surface. On the other hand, material such as gravel and boulders create a much more rough surface, whose turbulent conditions disrupt velocity streamlines. The channel surface in this study is comprised primarily of sandy material.

Another factor to consider is the type and condition of the vegetation that extends from the channel surface. Dense turf grass such as Bermuda will flatten during flow events, which causes less flow interference and, thus, a lower *n* value. Alternatively, channels with taller stalks and wider leaves, usually weeds, will result in higher *n* values.

Similarly, vegetation containing shrubs and bushes, numerous small trees, logs, and fallen trees will also cause high *n* values and lower the flow velocity. The condition of the vegetation varies highly with maintenance schedules and season. Regular mowing schedules keep grass short, causing less impact during periods of low flow and stage. In addition, mowing will prevent trees and shrubs from achieving substantial growth. Herbicide applications will eliminate troublesome weeds and other unwanted growth.

Lower *n* values can be observed during the fall and winter seasons due to negligible growth patterns. The proposed channel in this study will receive solid slab Bermuda grass sod placed on the flat bottom as well as the bottom three feet of the side slopes. Because of the channel's accessibility and proximity to Crosslin Park, this channel is expected to be mowed three times during the mowing season and receive one application of herbicide. Although there are other factors that affect *n* values, such as channel irregularity, alignment, size, and shape, they will not appreciably alter the value of *n*.

Gradually Varied Flow

As mentioned earlier, this type of flow occurs when the depth and velocity change along the length of a channel. This takes place when a flow control causes the depth to be different than the normal depth (Bentley, 2007). A flow control usually takes the form of a culvert, a change in channel slope, and channel intersections. Because a culvert does not usually convey an equal amount of flow as the influent channel, for example, the water surface elevation will increase on the upstream side of the culvert. The upstream distance needed for this higher water surface elevation to dissipate and resume normal depth characteristics is the section of channel referred to as being under the influence of gradually varied flow.

To develop the equation needed to describe the water surface profile during gradually varied flow conditions, one must recall that the total energy head H at any channel cross section equals the sum of the channel flowline elevation Z, depth of flow y, and the velocity head $V^2/2g$, which can be seen as follows:

$$H = Z + y + \left(\frac{\alpha V^2}{2g}\right) \tag{4-4}$$

Noting that $V = Q/A$ and that the energy changes along the length of the channel x, Equation 4-4 can be written in its derivative form as

$$\frac{dH}{dx} = \frac{dZ}{dx} + \frac{dy}{dx} + \frac{d}{dx}\left(\frac{\alpha Q^2}{2gA^2}\right) \tag{4-5}$$

where H = total energy head at a cross section (ft)

x = distance along channel (ft)

Z = channel flowline elevation (ft)

y = vertical flow depth (ft)

α = velocity distribution coefficient

Q = discharge (ft^3/s)

g = gravitational constant (32.2 ft^2/s)

A = cross sectional area (ft^2)

By using the calculus chain rule and because (1) the cross sectional area A depends on the depth of flow y, (2) $dH/dx = -S_f$ is the slope of the energy grade line, and (3)

$dZ/dx = -S_O$ is the slope of the channel flowline, Equation 4-5 can be further simplified as (Bentley, 2007)

$$\frac{dy}{dx} = \frac{S_O - S_f}{1 - Fr^2} \tag{4-6}$$

where

S_o = channel flowline slope (ft/ft)

S_f = friction slope or energy grade line (ft/ft)

Fr = Froude number (dimensionless)

$= \dfrac{V}{\sqrt{gD}}$

where

D = Hydraulic depth (ft)

Equation 4-6 is the governing equation for gradually varied flow. It demonstrates that the channel area and discharge are directly proportional to the change in depth along the channel.

This change in depth along the channel results in flow profiles. Flow profiles are first classified according to the relationship between the normal slope and critical slope. The critical slope, as defined by (Chow, 1959), is the slope that sustains a given discharge at a uniform and critical depth. When the normal slope is greater than the critical slope, the channel is said to have a hydraulically mild (M) slope. When the two slopes are equal, the channel is said to have a hydraulically critical (C) slope. If the normal slope is less than the critical slope, the channel is said to have a hydraulically steep (S) slope. In the rare case when the channel slope is zero, a normal slope does not exist and the channel is said to have a hydraulically horizontal (H) slope. In the most rare case when the channel flowline elevation increases in the downstream direction, the channel is said to have a hydraulically adverse (A) slope.

The second classification involves the relationship between the actual depth and the normal and critical depth. For actual depths greater than both of these, the depth corresponds to Zone 1. For actual depths less than both of these, the depth corresponds to Zone 3. Zone 2 involves actual depths between the normal and critical depth. Therefore, an M1 flow profile implies that for a specific channel cross section the actual depth is greater than the normal depth that itself is greater than the critical depth. Figure 4-1 and Figure 4-2 illustrate several examples of flow profiles that may exist in a prismatic channel along with their water surface profiles in terms of curvature (Bentley, 2007).

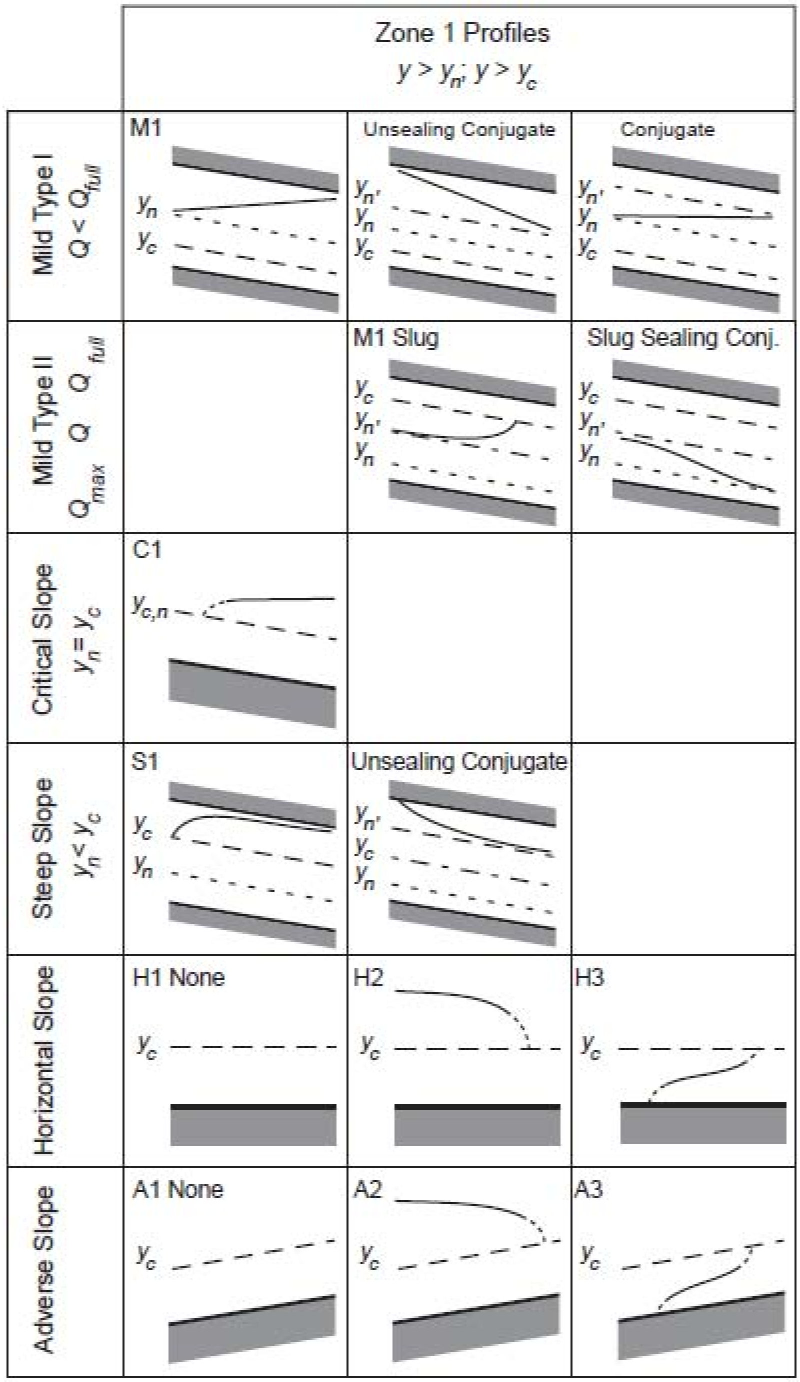


Figure 4-1 Flow Profiles for Zone 1 (Bentley, 2007)

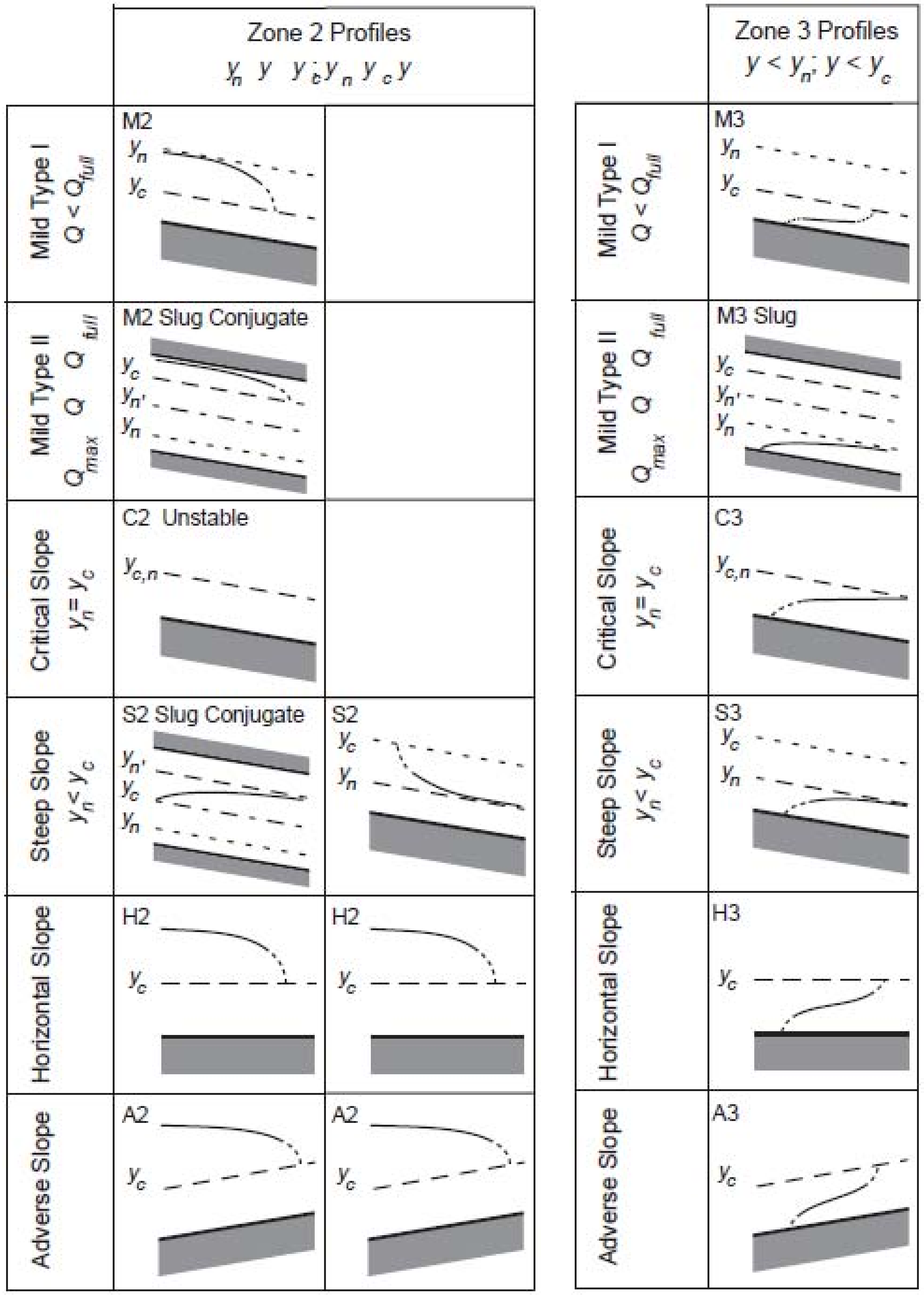


Figure 4-2 Flow Profiles for Zone 2 and Zone 3 (Bentley, 2007)

CulvertMaster

The principle use of Bentley's CulvertMaster software is designing and analyzing roadway culverts. It solves for headwater elevation, discharge, or size. Standard pipe shapes include ellipse, circular, and arch while the materials include aluminum, corrugated metal, concrete, and high-density polyethylene. Standard reinforced concrete box sizes range from 2-ft x 2-ft to 12-ft x 12-ft. It should be noted that the default setting for the flow area of the reinforced concrete box is calculated using the full height and width. Manufacturers of prefabricated reinforced concrete boxes, on the other hand, construct their sections with a chamfer in all four corners, resulting in flow area reductions of up to 2.5%.

The software analyzes the performance of the culverts using culvert hydraulics (Bentley, 2007). Culverts can create considerable restrictions in open channel flow conditions. These restrictions alter flow characteristics and can result in complicated solutions containing both gradually varied and rapidly varied flow conditions. Culverts are generally not long enough to achieve uniform flow. Bentley (2007) discusses the two generally accepted methods for predicting the hydraulic performance of a culvert. The first method utilizes a nomograph created specifically for different pipe materials, shapes, and entrance types. Figure 4-3 (Bentley, 2007) illustrates an example of a nomograph for a circular concrete pipe with entrance types of square edge with headwall, groove end with headwall, and groove end projecting.

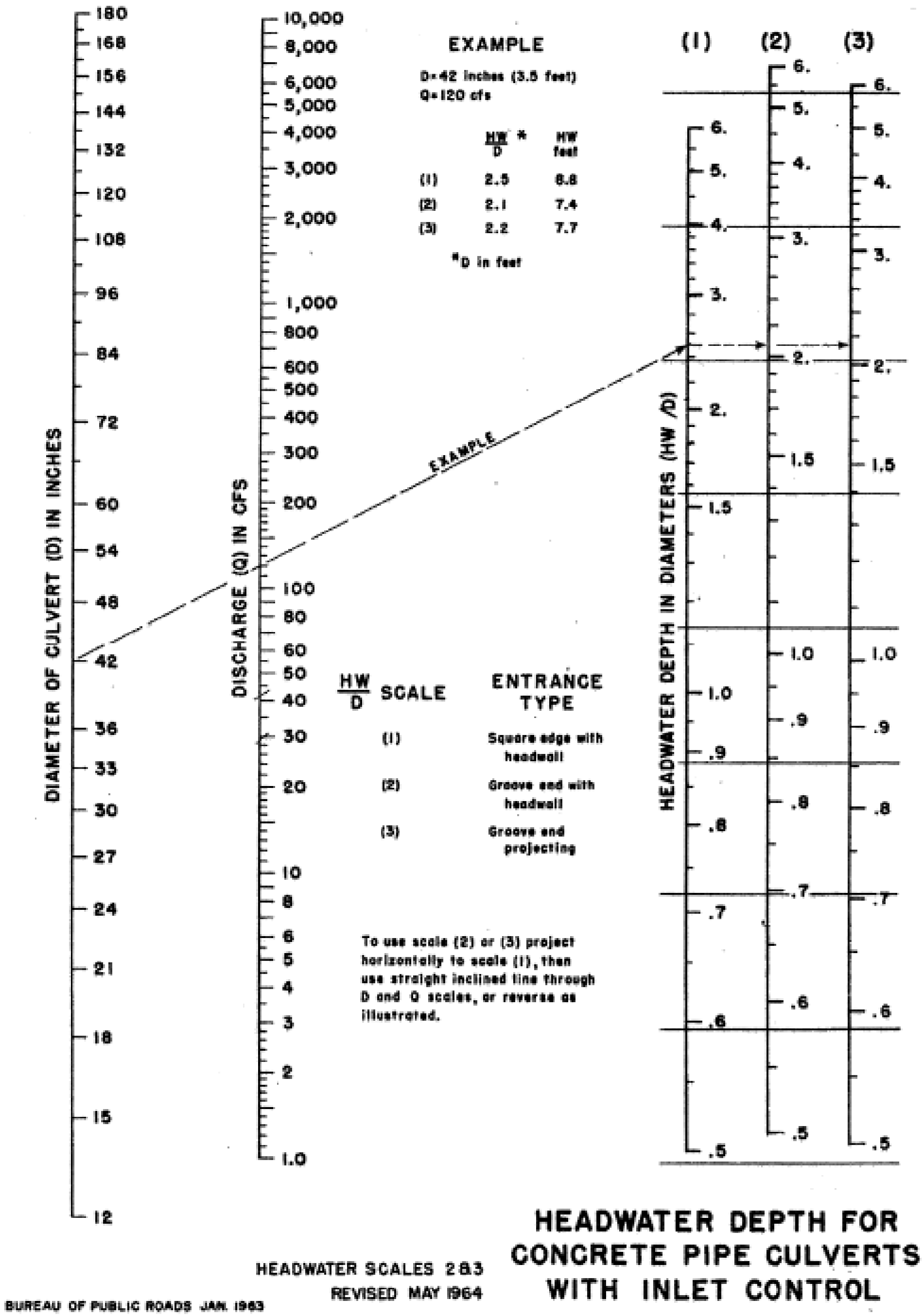


Figure 4-3 Nomograph to Compute Headwater Depth for Circular Concrete Culverts with Inlet Control (Bentley, 2007)

This method is easier to use and produces results acceptable for most designs. Using a trial and error procedure, a trial diameter is selected and a line drawn from it to the design discharge. This line is then projected to the first column in the HEADWATER DEPTH IN DIAMETERS (HW/D) scale. Then a line is lastly drawn horizontal through the other two scales. The three scale readings indicate the ratio of headwater depth to pipe diameter. Assuming the trial pipe diameter, the headwater depth can be calculated and checked against design parameters. The second method involves computing the flow profiles using gradually varied flow procedures. Although this method is very labor intensive, it produces much more accurate culvert performance. CulvertMaster uses this method to achieve results much faster than hand calculations.

CulvertMaster simplifies the flow conditions using two different assumptions developed in part through research by the National Bureau of Standards (NBS) and the Federal Highway Administration (Bentley, 2007). These assumptions are organized according to where the flow control section occurs within the culvert: inlet control and outlet control. The headwater depth is computed for both control conditions with the controlling headwater depth being the greater of the two. CulvertMaster uses procedures recommended in Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts (2001) as prepared for the U.S. Federal Highway Administration to calculate the headwater depths automatically.

Inlet Control

Under inlet control conditions, the culvert capacity is reduced due to parameters located at the entrance of the culvert. This control section is just inside the culvert barrel and it is here where the water surface drops below the critical depth and enters the supercritical region forming an S2 water surface profile like that shown in Figure 4-2. This control section is a result of available opening area, physical opening shape, and the inlet configuration. Figure 4-4 (Bentley, 2007) shows how the physical opening shape affect the flow streamlines enhances the culvert hydraulics.

Specifically, it demonstrates that square-edged inlets compress the streamlines effectively reducing the cross sectional area of the culvert. Inlet control can usually be found in applications where the culvert is installed at a steep slope and/or the downstream flow is relatively shallow. For this condition, the downstream parameters, such as pipe friction, tailwater, and other minor losses, have no effect on the culvert capacity. Figure 4-5 (Bentley, 2007) below provides four examples of a culvert operating under inlet control.

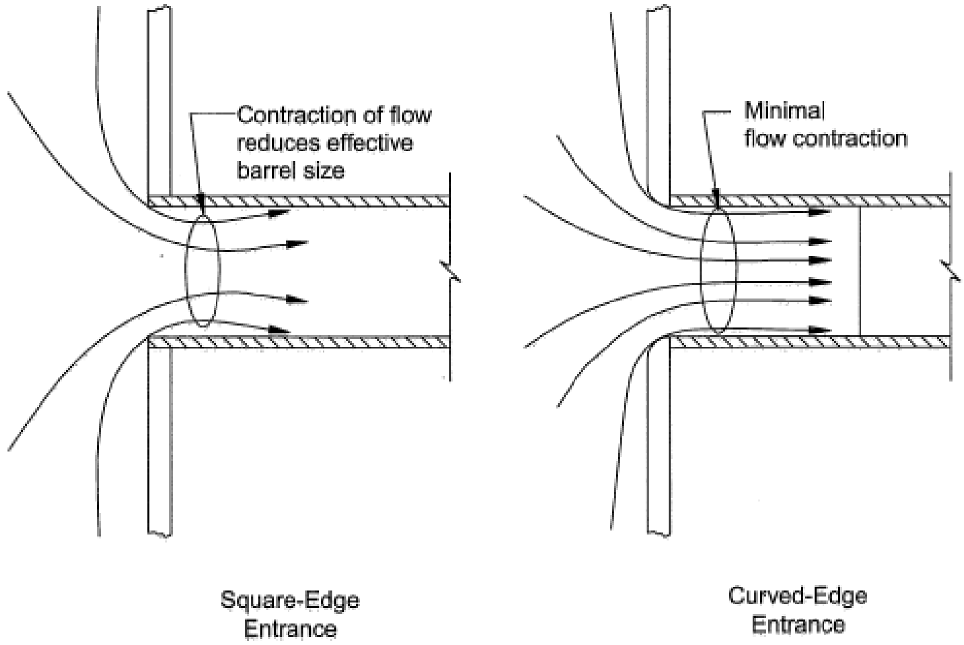


Figure 4-4 Square-Edge and Curved-Edge Culvert Entrances (Bentley, 2007)

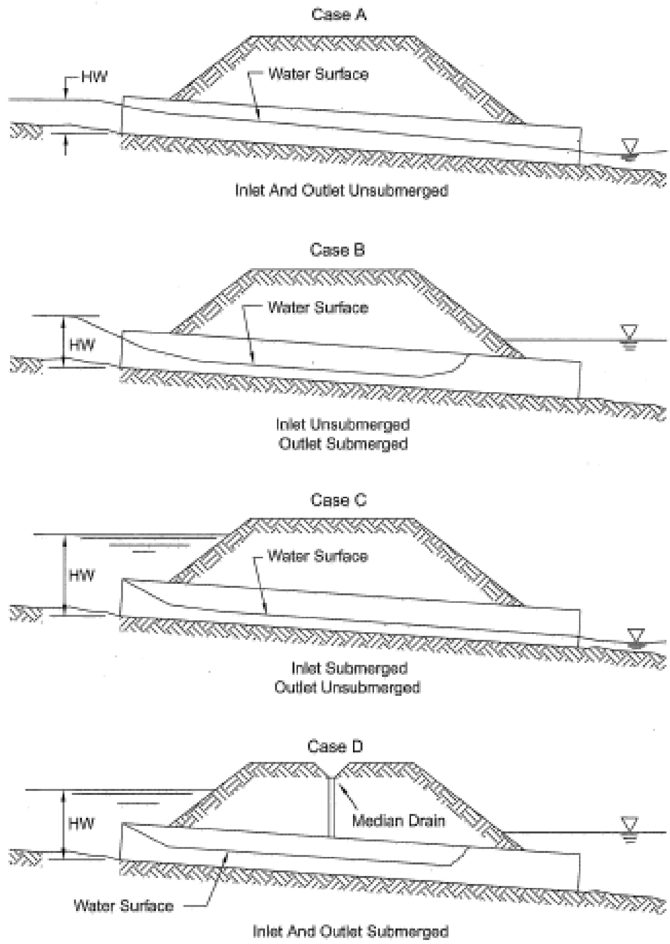


Figure 4-5 Inlet Control Flow Conditions (Bentley, 2007)

These examples illustrate the three types of culvert hydraulics in effect during inlet control conditions. The three types are unsubmerged, submerged, and transitional (Bentley, 2007). The unsubmerged effect occurs primarily in low flow events and is modeled using weir flow theory. CulvertMaster uses the following equation for this condition.

$$\frac{HW_i}{D} \equiv \frac{H_c}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S \tag{4-7}$$

where

HW_i	= Headwater depth above inlet control section invert (ft)
D	= Interior height of culvert barrel (ft)
H_c	= Specific head at critical depth ($d_c + V_c^2/2g$)(ft)
Q	= Discharge (ft^3/s)
A	= Full cross-sectional area of culvert barrel (ft^2)
S	= Culvert barrel slope (ft/ft)
K, M	= Constants from Table 4-1 (M is unitless)

Although not as theoretically correct as Equation 4-7, Equation 4-8 below is better suited for practical application (Bentley, 2007).

$$\frac{HW_i}{D} \equiv K\left[\frac{Q}{AD^{0.5}}\right]^M \tag{4-8}$$

The submerged effect occurs primarily during high flow events and is modeled using orifice flow theory. The equation used by CulvertMaster for this condition is as follows:

$$\frac{HW_i}{D} \equiv c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S \qquad (4\text{-}9)$$

where

c, Y = Constants from Table 4-1 (Y is unitless)

Equation 4-7 should be used for cases of $Q/AD^{0.5}$ = 3.5 and smaller. Equation 4-9 should be used when $Q/AD^{0.5}$ = 4.0 and larger. The transition effect occurs in between these unsubmerged and submerged zones near the crown of the culvert entrance. The headwater value for this zone must be interpolated using Figure 4-6.

Table 4-1 Constants for Inlet Control Equations (Bentley, 2007)

Culvert Shape and/or Material	Inlet Edge Description	Equation in text	Unsubmerged (Weir Flow) *K*	Unsubmerged (Weir Flow) *M*	Submerged (Orifice Flow) *c*	Submerged (Orifice Flow) *Y*
Circular, concrete	Square edge with headwall	(9.2)	.0098	2.0	.0398	0.67
	Groove end with headwall		.0018	2.0	.0292	.74
	Groove end projecting		.0045	2.0	.0317	.69
Circular, CMP	Headwall	(9.2)	.0078	2.0	.0379	.69
	Mitered to slope		.0210	1.33	.0463	.75
	Projecting		.0340	1.50	.0553	.54
Circular	Beveled ring, 45× bevels	(9.2)	.0018	2.50	.0300	.74
	Beveled ring, 33.7× bevels		.0018	2.50	.0243	.83
Rectangular box	30× to 75× wingwall flares	(9.2)	.026	1.0	.0347	.81
	90× and 15× wingwall flares		.061	0.75	.0400	.80
	0× wingwall flares		.061	0.75	.0423	.82
Rectangular box	45× wingwall flares, d = 0.043D	(9.3)	.510	.667	.0309	.80
	18× to 33.7× wingwall flares, d =		.486	.667	.0249	.83
Rectangular box	90× headwall with ¾-in. chamfers	(9.3)	.515	.667	.0375	.79
	90× headwall with 45× bevels		.495	.667	.0314	.82
	90× headwall with 33.7× bevels		.486	.667	.0252	.865
Rectangular box	45× skewed headwall; ¾-in. chamfers	(9.3)	.545	.667	.0505	.73
	30× skewed headwall; ¾-in. chamfers		.533	.667	.0425	.705
	15× skewed headwall; ¾-in. chamfers		.522	.667	.0402	.68
	10-45× skewed headwall; 45× bevels		.498	.667	.0327	.75
Rectangular box with ¾-in. chamfers	45× nonoffset wingwall flares	(9.3)	.497	.667	.0339	.803
	18.4× nonoffset wingwall flares		.493	.667	.0361	.806
	18.4× nonoffset wingwall flares; 30×		.495	.667	.0386	.71
Rectangular box w/ top bevels	45× wingwall flares, offset	(9.3)	.497	.667	.0302	.835
	33.7× wingwall flares, offset		.495	.667	.0252	.881
	18.4× wingwall flares, offset		.493	.667	.0227	.887
Corrugated metal boxes	90× headwall	(9.2)	.0083	2.0	.0379	.69
	Thick wall projecting		.0145	1.75	.0419	.64
	Thin wall projecting		.0340	1.5	.0496	.57
Horizontal ellipse, concrete	Square edge w/ headwall	(9.2)	.0100	2.0	.0398	.67
	Groove end w/ headwall		.0018	2.5	.0292	.74
	Groove end projecting		.0045	2.0	.0317	.69
Vertical ellipse, concrete	Square edge w/ headwall	(9.2)	.0100	2.0	.0398	.67
	Groove end w/ headwall		.0018	2.5	.0292	.74
	Groove end projecting		.0095	2.0	.0317	.69
Pipe arch, CM, 18-in. corner radius	90× headwall	(9.2)	.0083	2.0	.0379	.69
	Mitered to slope		.0300	1.0	.0463	.75
	Projecting		.0340	1.5	.0496	.57
Pipe arch, CM, 18-in. corner radius	Projecting	(9.2)	.0300	1.5	.0496	.57
	No bevels		.0088	2.0	.0368	.68
	33.7× bevels		.0030	2.0	.0269	.77
Pipe arch, CM, 31-in. corner radius	Projecting	(9.2)	.0300	1.5	.0496	.57
	No bevels		.0088	2.0	.0368	.68
	33.7× bevels		.0030	2.0	.0269	.77
Arch, CM	90× headwall	(9.2)	.0083	2.0	.0379	.69
	Mitered to slope		.0300	1.0	.0463	.75
	Thin wall projecting		.0340	1.5	.0496	.57

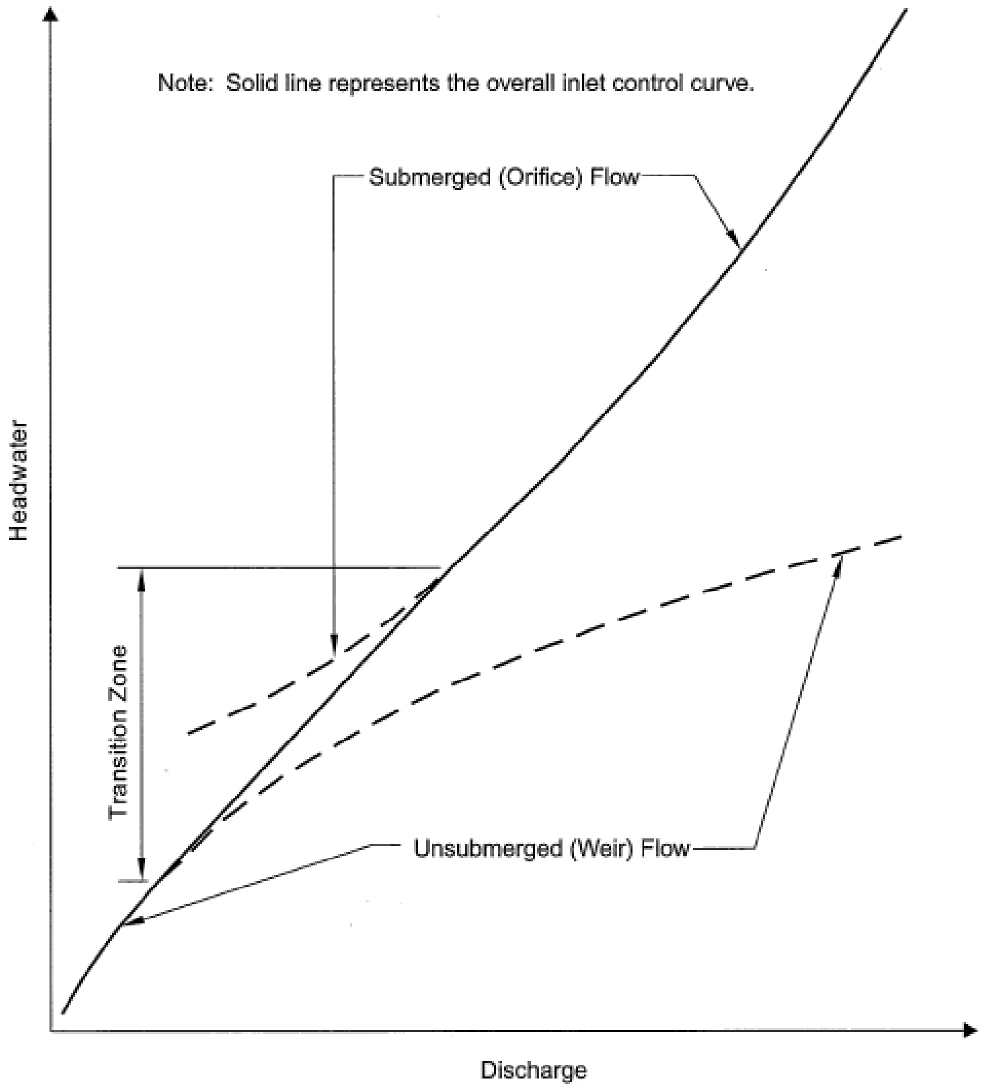


Figure 4-6 Transition from Weir to Orifice Control in Culvert (Bentley, 2007)

While each of the cases shown in Figure 4-5 follow the reasoning of inlet control conditions, case D may appear nonconforming. It would seem logical to assume that because both the inlet and outlet are submerged the entire culvert length must be submerged. The explanation is the S2 water surface profile that forms as a result of the steep grade of the culvert. At the downstream end of the culvert, the pressure flow conditions force the hydraulic jump. A structure such as a median drain should be installed to relieve the sub-atmospheric pressure before it can cause hydraulic and even structural problems.

Outlet Control

Under outlet control conditions, the culvert inlet is capable of passing a larger flow than the barrel. Its control section is located near the outlet with a flow level at critical, subcritical, or full condition. This condition frequently experiences full flow, or pressure flow, because the tailwater is sufficiently high and the channel topography is generally mild. Several examples of outlet-controlled conditions can be seen in Figure 4-7. When full flow conditions do exist, the headwater depth is affected by discharge, upstream and downstream velocity, cross-sectional area, shape, length, roughness, slope, inlet and outlet edge configuration, and tailwater depth (Bentley, 2007). Consequently, the losses associated with these parameters have to be computed. CulvertMaster uses the following equation to calculate the headwater for outlet control conditions.

$$HW_o + \frac{{V_u}^2}{2g} = TW + \frac{{V_d}^2}{2g} + H_L \qquad (4\text{-}10)$$

where

HW_o = Headwater depth above outlet invert (ft)

V_u = Upstream velocity (ft/s)

TW = Tailwater depth above the outlet invert (ft)

Vd = Downstream velocity (ft/s)

H_L = Sum of all losses including entrance (H_e), friction loss (H_f), exit loss (H_o), and other losses

g = Gravitational acceleration constant (ft/s^2)

The minor loss associated with the entrance of the culvert can be expressed as follows:

$$H_e = k_e\left(\frac{V^2}{2g}\right) \tag{4-11}$$

where

H_e = Entrance loss (ft)

k_e = Entrance loss coefficient (unitless)

V = Velocity inside of barrel entrance (ft/s)

g = Gravitational acceleration constant (ft/s^2)

The entrance loss coefficient, k_e, is a function of inlet arrangement. Table 4-2 provides several values for different pipe materials and entrances.

Table 4-2 Entrance Loss Coefficients for Outlet Control Conditions (Bentley, 2007)

Structure Type and Entrance Condition	k_e
Concrete pipe	
Projecting from fill, socket or groove end	0.2
Projecting from fill, square edge	0.5
Headwall or headwall and wingwalls	
Socket or groove end	0.2
Square edge	0.5
Rounded (radius = *D*/12)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Beveled edges (33.7° or 45° bevels)	0.2
Side- or slope-tapered inlet	0.2
Corrugated metal pipe or pipe-arch	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls (square edge)	0.5
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Beveled edges (33.7° or 45° bevels)	0.2
Side- or slope-tapered inlet	0.2
Reinforced concrete box	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 sides	0.5
Rounded or beveled on 3 sides	0.2
Wingwalls at 30° to 75° from barrel	
Square-edged at crown	0.4
Crown edge rounded or beveled	0.2
Wingwalls at 10° to 25° from barrel	
Square-edged at crown	0.5
Wingwalls parallel (extensions of box sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

The minor loss associated with the exit of the culvert is a function of the difference between the velocity just inside the culvert exit and the velocity in the outfall channel. It can be expressed as follows:

$$H_o = 1.0\left(\frac{V^2}{2g} - \frac{V_d^{\,2}}{2g}\right) \tag{4-12}$$

where

H_o = Exit loss (ft)

V = Velocity inside of barrel exit (ft/s)

V_d = Velocity in channel outfall (ft/s)

g = Gravitational acceleration constant (ft/s^2)

The minor losses associated with friction are calculated using a gradually varied flow analysis. The analysis usually starts at the tailwater elevation and proceeds upstream. Within the culvert, the water surface elevation will move above and below the crown of the culvert and CulvertMaster will automatically move in and out of pressure mode. For the section of culvert that is submerged, the friction loss will be computed using the full flow friction slope.

CulvertMaster automatically determines whether the culvert is operating under inlet or outlet control conditions and performs the appropriate gradually varied flow (frontwater or backwater profile) analysis.

Gradually varied flow profiles denote the water surface depth curve along the length of the culvert and are developed in an upstream or downstream direction depending on the slope of the culvert and the controlling water surface elevation (Bentley, 2007). For culverts installed on a slope less than critical slope, this mild slope causes the depth of flow to increase gradually if the downstream water surface is less than the normal depth, resulting in an M2 drawdown curve. An M1 drawdown curve will result when the downstream water surface is greater than the normal depth and the depth of flow gradually decreases. Both of these water surface profiles indicate flow is operating in the subcritical region.

For culverts installed on a slope greater than critical slope, this steep slope causes the gradually varied flow profile to develop in the upstream direction when the controlling tailwater elevation is well above critical depth and subcritical flow exists at the culvert exit. These pipes usually will experience a hydraulic jump somewhere within the culvert length when the tailwater depth is greater than the critical depth.

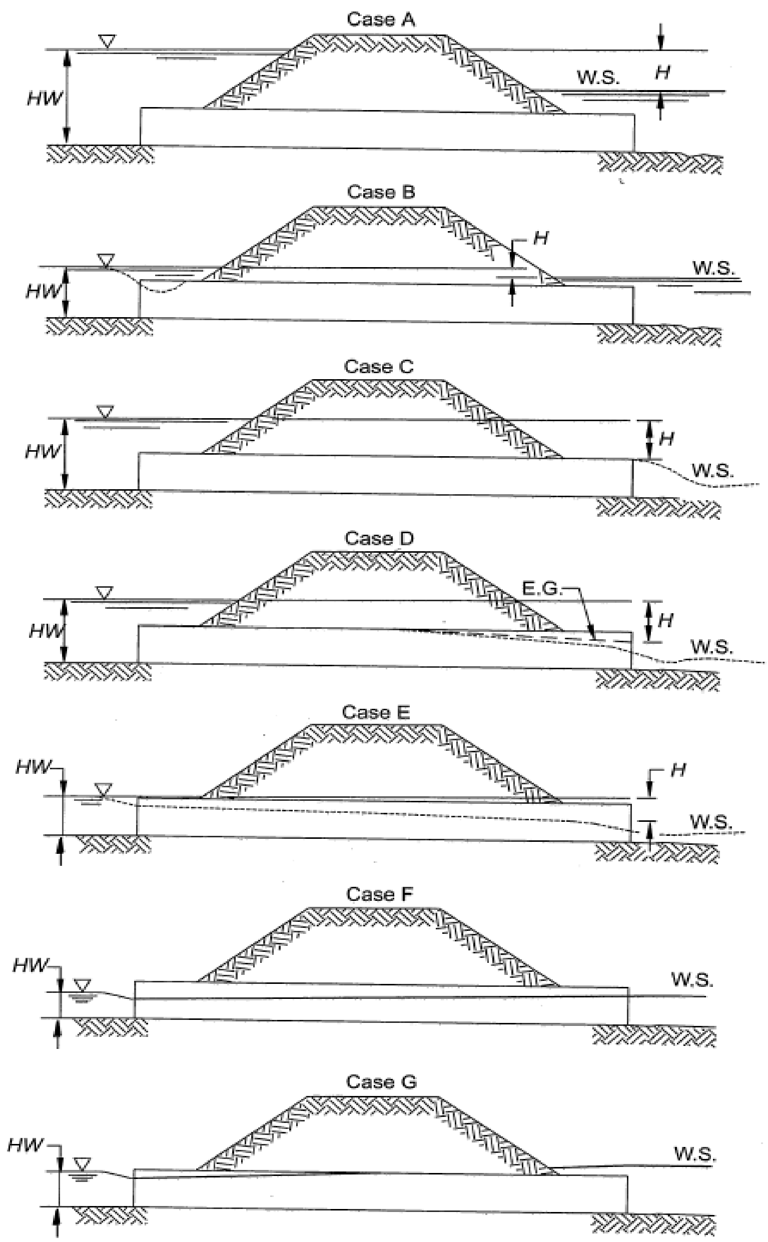


Figure 4-7 Outlet Control Flow Conditions (Bentley, 2007)

CHAPTER V

METHODOLOGY

The general steps utilized for solving typical channel hydraulics problems involve the following series of actions:

1. Problem Definition
2. Data Acquisition
3. Channel Modeling
4. Culvert Modeling

Therefore, these steps were used to solve the problems related to this study.

Problem Definition

The first step of problem definition actually involves two components. The first is investigating the location of the problem by observing physical evidence and discussing conditions with local residents. The primary physical evidence includes high water marks and house elevations with respect to waterway banks. After numerous site visits, it became clear that the local, intermittent stream was flowing out of its banks more often during 10-year or less rain events and threatening to inundate houses in the subdivision. This waterway, locally known as Rolling Acres Channel, was constructed in 1959 when the subdivision was platted. It is a dry waterway that only flows during rain events and is shown but unnamed on United States Geological Survey (USGS) quadrangle maps. The second component is to identify the cause of the problem. Upstream waterways were inspected for dam breaches and non-permitted diversion channels. Upon finding none of these, it was concluded that the large, upstream development had finally began to produce an amount of runoff that exceeded the capacity of the existing channel. Although the developer would share in the cost of an improvement, it would be the responsibility of the local government to acquire property and/or easement, design, construct, and maintain a solution that would prevent further damage to life and property.

Data Acquisition

This step involves collecting the data necessary for the design of an improved channel and roadway culverts. The type of data to be collected included topographic surveys, aerial photographs, and a geotechnical investigation. Because the focus of this study was hydraulic modeling, the associated hydrologic data was also needed. The topographic surveys are valuable and provide land contours and such features as buildings, fences, trees, driveways, roadways, existing waterway details, and utilities. Accurate land contours are critical in determining earthwork volumes and excavation daylight lines. A representative sample of the topographic survey, including trees, roads, culverts, houses, and surface contours, can be seen in Figure 5-1. The type of data collected for a building consists of its outline, relationship to other buildings, and finished floor elevation. This data is needed to show the proximity of proposed improvements to adjacent homes, which can prove helpful when discussing construction plans with adjacent homeowners. Probably most important is the accurate measurement of the finished floor elevation. These elevations are compared to the model output to verify their relationship to the proposed water surface elevations. Another key factor is the location of all existing utilities because they can financially impact the proposed improvement if they have to be relocated. Although the topographic survey can locate the cables and pipes horizontally, the utility owner must provide the depths. It is these depths that can conflict with proposed excavation daylight lines as well as flow line slope. When the utility is located in a private easement, the cost for relocation will be borne by the local governing body.

Aerial photographs can be used to track the progression of development as well as provide a wide-angle view of the basin and its characteristics. They often contain ground contours that can be compared with topographic surveys and known waterways. The primary purpose of the geotechnical investigation is to establish soil characteristics and ground water table depth.

Channel Modeling

The third step involves using FlowMaster to model design parameters in a channel. To begin, a worksheet must be opened as shown in Figure 5-2. Both the Uniform Flow tab and the Gradually Varied Flow (GVF) tab will be used for this study. Although the channel reaches will be prismatic, depth and velocity will change with respect to time upstream of the culverts and at changes in liner material. The Manning friction method was chosen for uniform flow modeling because of its simplicity and widespread industry usage. In addition, the software allows the user to select from a list the desired unknowns for which to solve. These parameters can be seen in Figure 5-3 and include discharge, normal depth, side slopes, bottom width, channel slope, and roughness.

The next operation in this step is to determine which of these parameters is unknown. As discussed in Chapter III, the discharge rates for this study were taken from a previous hydrologic analysis (Envirotech, 2009). Side slopes are also known as they are typically set to a minimum of 3:1 to facilitate safe maintenance operations. During the data acquisition phase of this study, it was discovered that sanitary sewer pipelines were located perpendicular to the channel and adjacent to both Grant Street and Lincoln Street. Because these pipelines are gravity systems, they cannot be relocated. Therefore, these pipelines, as well as the spillway elevation in Crosslin Park Lake, set the channel flowline elevations and slope for this study.

After conducting site visits, it was clear that the channel would encounter very sandy soils and narrow easements. The sandy soil conditions can result in severe sediment transport and erosion during even small storm events. Therefore, any grass channel sections would require a material that would hold the soil in place such as solid slab sod with a tight wood staple pattern. The narrow easements were caused by the close proximity of existing residential structures such as houses and sheds. To minimize the impact to these structures, these channel sections would require a concrete slab to minimize cross sectional area. In addition, the local municipality requires a 10-foot wide concrete trickle channel in the center of all grass-lined channels. Using this information, the channel roughness could be eliminated as an unknown. It was obvious that the channel's bottom width would need to be adjusted to provide a water surface elevation that was below the finished floor elevations of the nearby residential houses. Thus, the normal depth was chosen as the unknown parameter for which to solve.

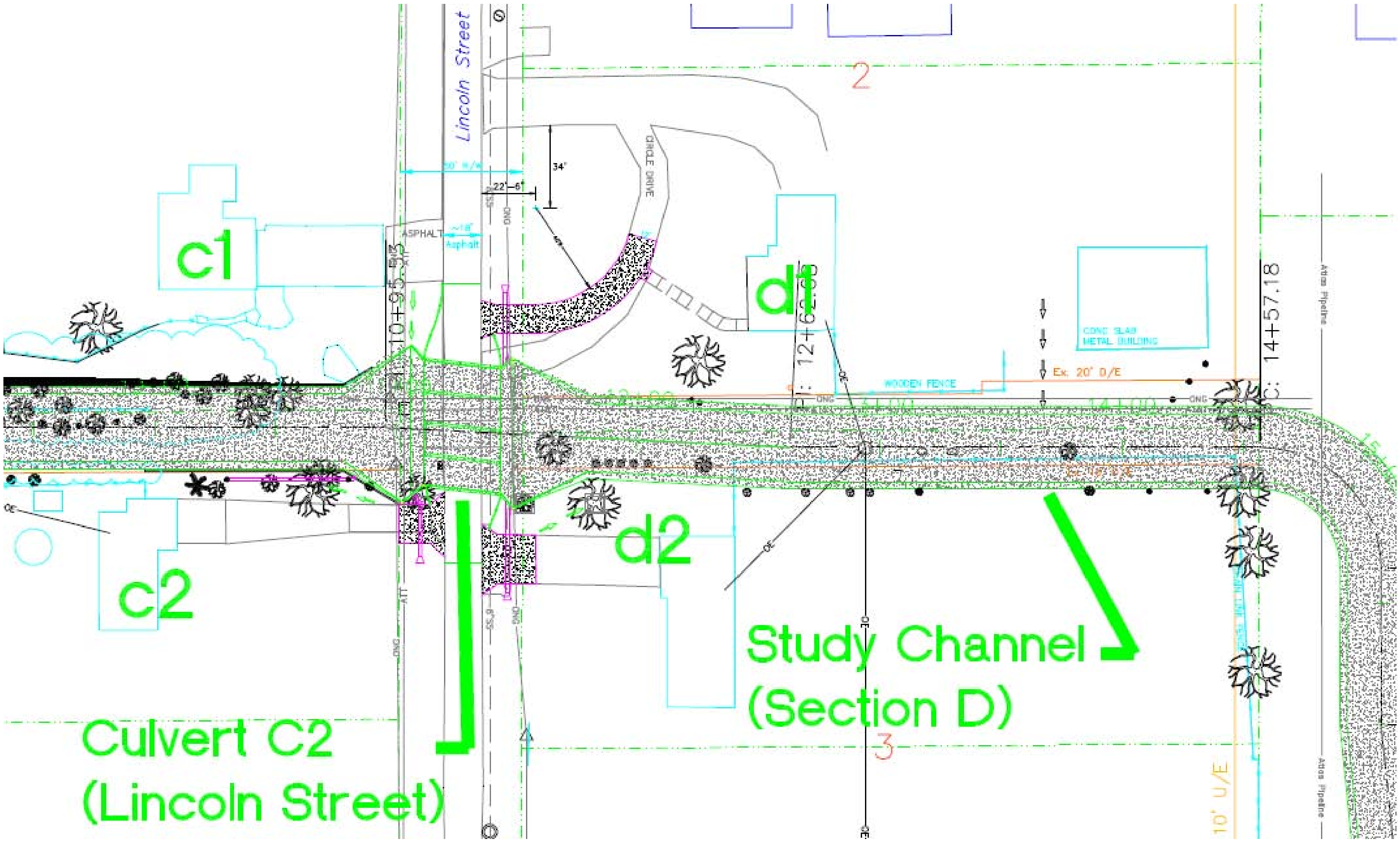


Figure 5-1 Topographic Survey Sample

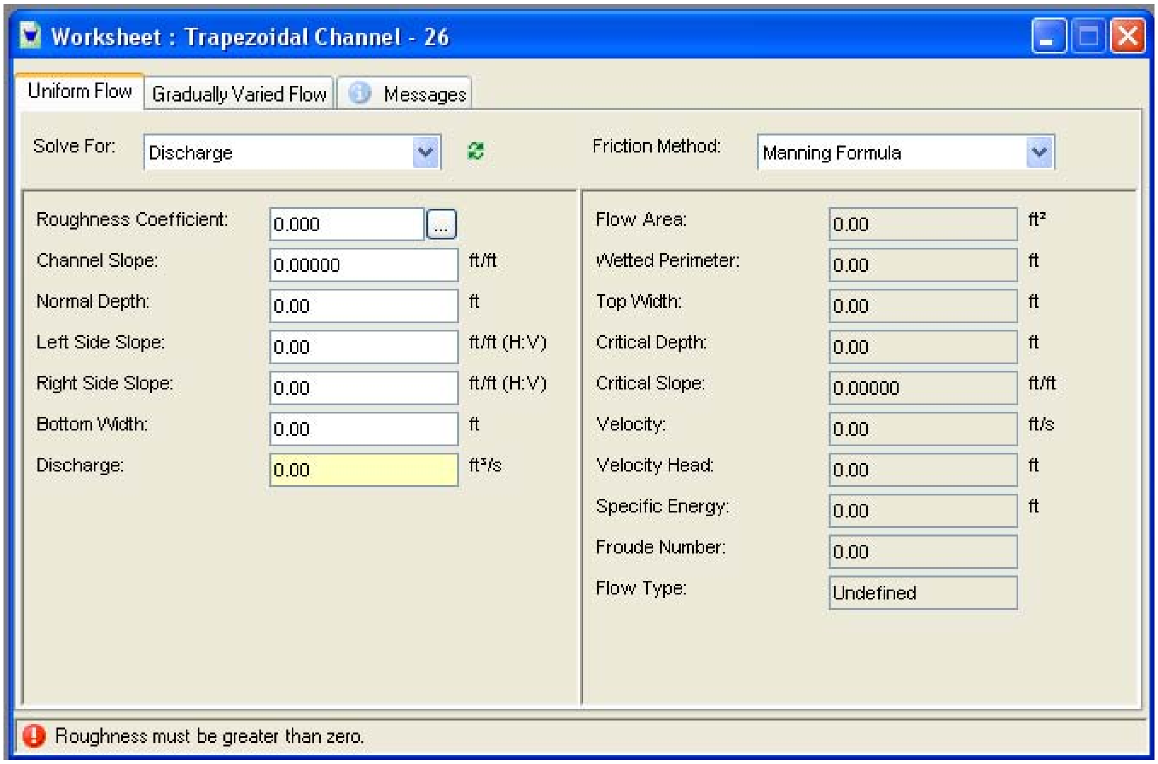


Figure 5-2. FlowMaster Worksheet Layout

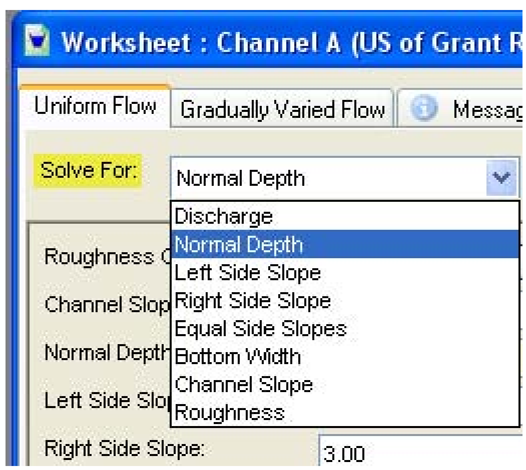


Figure 5-3. Available Parameters for Unknowns - FlowMaster

The calculation of the channel roughness would be the final operation necessary before solving for the chosen unknown. For standard channel sections, FlowMaster provides a materials list with associated roughness coefficients from which to choose. This list was used to select the roughness coefficient for the section of channel with a concrete surface (Figure 5-4). Because the grass-lined channel sections would also contain a concrete trickle channel, a composite roughness coefficient would have to be calculated. Table 5-1 shows the calculation for this composite coefficient.

Now that all of the input variables have been determined, FlowMaster can solve for the unknown parameter. An example calculation can be seen in Figure 5-5 while the entire study calculations are shown in Appendix A.

Figure 5-5 illustrates how conveniently FlowMaster displays several characteristics related to the channel hydraulics on the right side of the worksheet. Two of these characteristics are worth reviewing immediately after calculation: velocity and Froude number. It is very desirable to develop a velocity greater than the cleansing velocity of the industry standard 3 feet per second to keep particles in suspension and minimize sediment deposition. There is also a benefit to achieving subcritical flow, especially for grass-lined channels. A Froude number less than one will reduce the risk of channel scour and other risks associated with turbulent conditions.

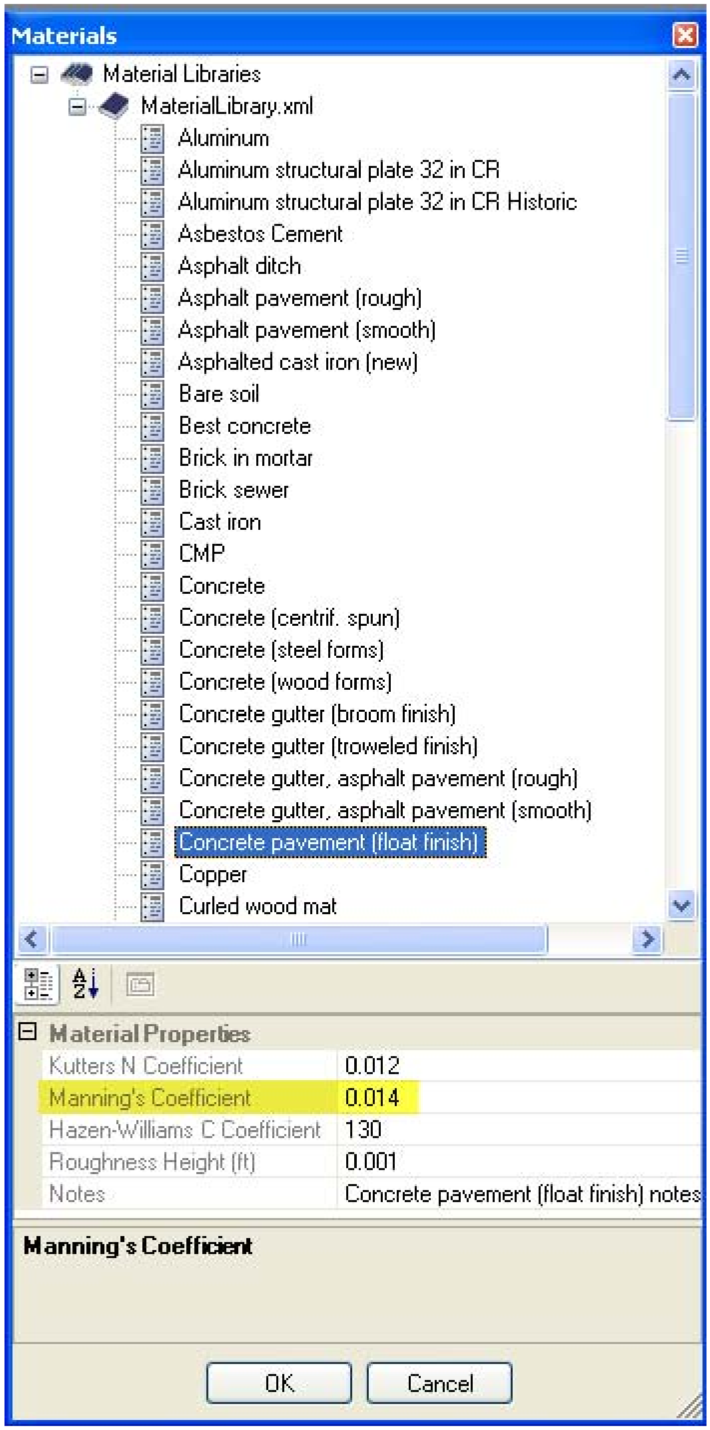


Figure 5-4. Material List for Roughness Coefficient Selection

Table 5-1. Composite Roughness Coefficient Calculation

Grass channel in Crosslin Park		
total length of ground surface	59	35' FB w/ 4:1 slopes that are 12'W and w/ 10'W FB pcc trickle channel w/ 1'W curbs
grass length	47	12+(35-(10+1+1))+12
grass n	0.045	
concrete length	12	10+1+1
concrete n	0.014	
composite n	0.0387	
Grass channel upstream and downstream of Grant Street		
total length of ground surface	74	50' FB w/ 4:1 slopes that are 12'W and w/ 10'W FB pcc trickle channel w/ 1'W curbs
grass length	62	12+(50-(10+1+1))+12
grass n	0.045	
concrete length	12	10+1+1
concrete n	0.014	
composite n	0.0400	
	0.04	**Use for all grass channels**

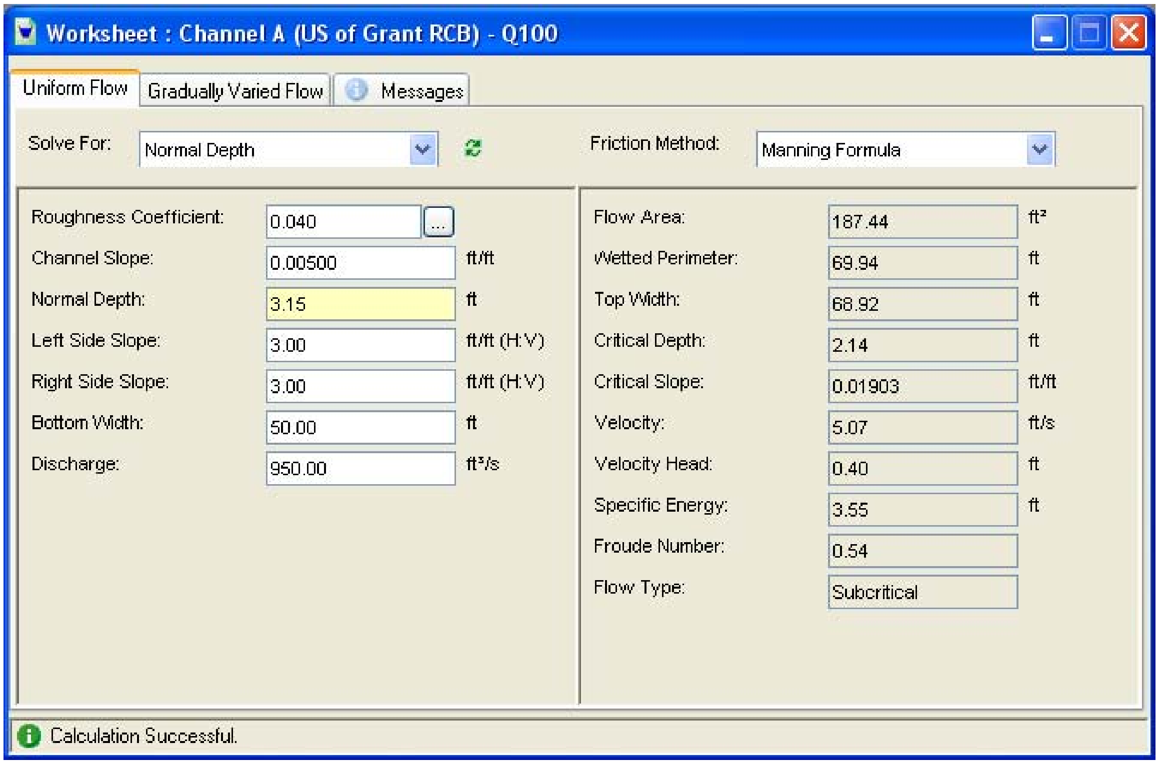


Figure 5-5. FlowMaster Example Input/Output Results

Culvert Modeling

The last step involves using CulvertMaster to model design parameters for drainage culverts that will serve as roadway channel crossings. The waterway in this study includes three roadway crossings that must be sized to adequately convey the runoff while also considering the relationship between the generated headwater and the adjacent house's finished floor. To begin, a worksheet must be opened as shown in Figure 5-6.

Similar to the worksheet in FlowMaster, this worksheet offers a list of properties for which to solve. The list is shown in Figure 5-7 and includes headwater elevation, discharge, and size.

As in FlowMaster, the first decision involves selecting the property to be used as the unknown. As mentioned previously, the discharge has been determined from a previous research (Envirotech, 2009). Keeping the study objective in perspective, the headwater elevation was chosen as the unknown for this study. Therefore, the size of culvert would be varied until a headwater elevation was found to meet the criteria of the objective.

Figure 5-6 shows that there are four components that require data input. The Culvert component requires the known discharge, a maximum allowable headwater elevation, and a tailwater elevation. The maximum allowable headwater was chosen as the lowest, adjacent, finished floor elevation so that a direct comparison between this and the computed headwater elevation could be made. If the computed headwater elevation is less than the finished floor elevation, the adjacent houses are protected from flooding. The tailwater elevation was chosen based on the uniform depth calculations for the downstream channel as well as the detention characteristics of Crosslin Lake. The Inverts component calculates the culvert slope after inputting the proposed inlet and outlet flowlines of the culvert. The Section component requires data input relative to the type of culvert proposed. CulvertMaster offers many shapes of culvert including arch, box, circular, and ellipse. Based on the large flows in this study, a reinforced concrete box would be the most practical culvert shape. A Manning's *n* value of 0.013 was selected as the roughness coefficient to represent a standard concrete surface. The size and number of barrels for the reinforced concrete box are the actual properties that would be adjusted until the computed headwater was lower than adjacent finished floor elevations. The Inlet component would identify the type of entrance geometry for the barrels. As mentioned in a previous chapter, this geometry can greatly affect the flow capacity of the culvert. The culverts for this study would be perpendicular to channel flow and the standard construction practice of using ¾-inch chamfers would be required. The typical input parameters for a culvert can be seen in Figure 5-8. A typical reinforced concrete box (RCB) culvert is shown in Figure 5-9.

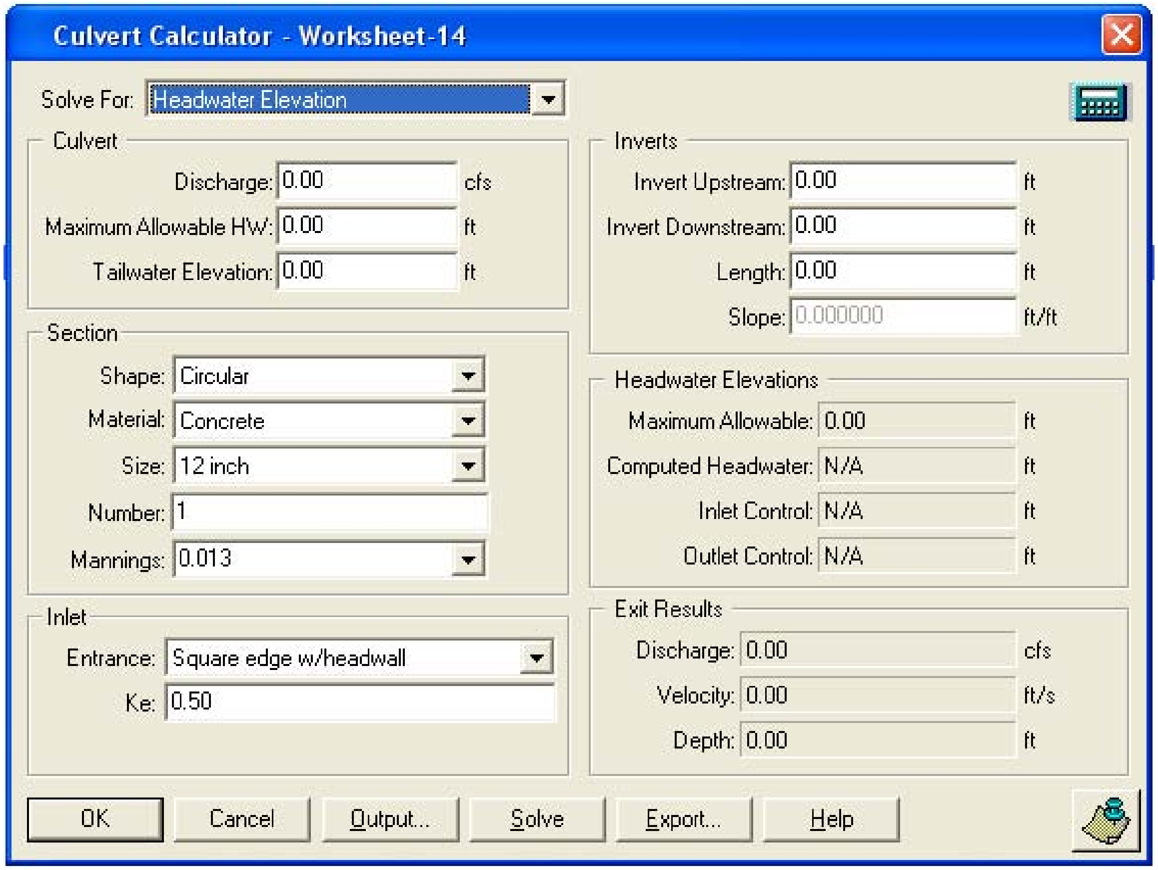


Figure 5-6. CulvertMaster Worksheet Layout

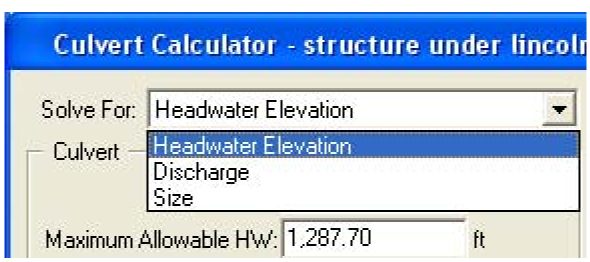


Figure 5-7. Available Properties for Unknowns - CulvertMaster

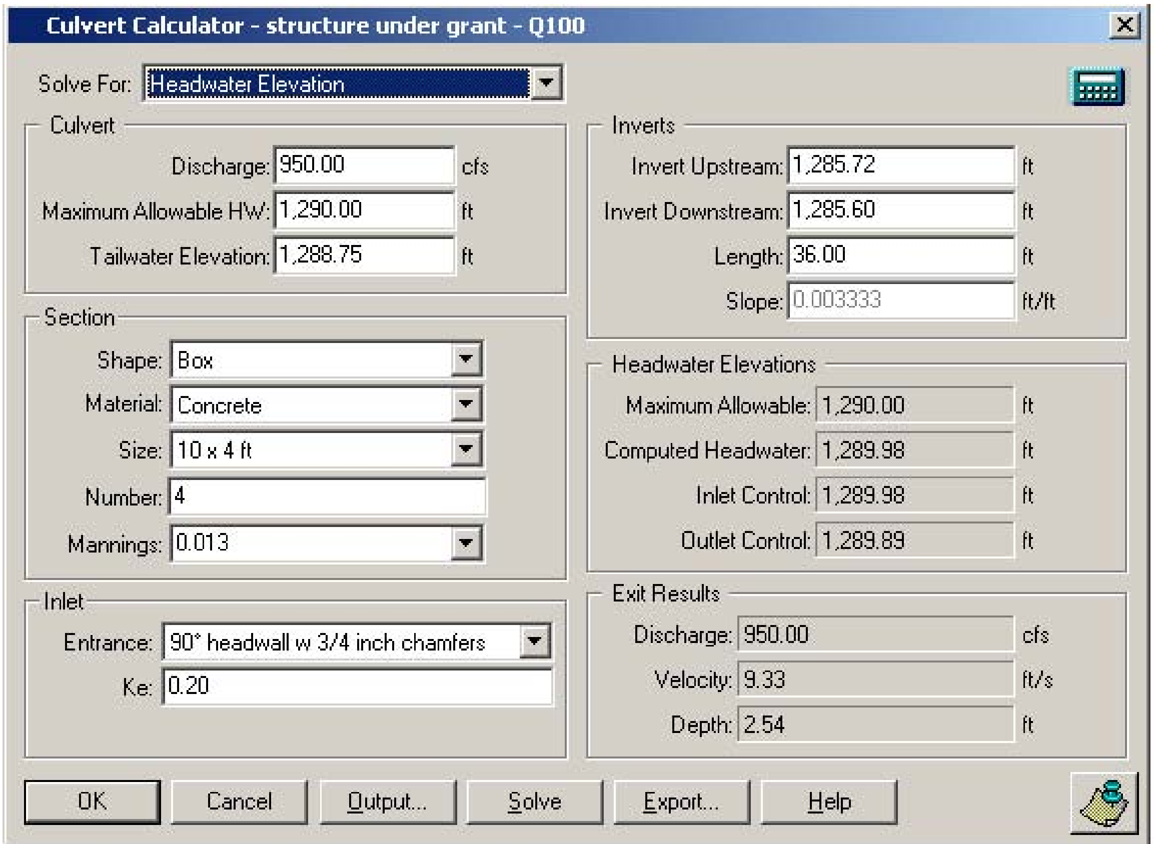


Figure 5-8. CulvertMaster Example Input/Output Results

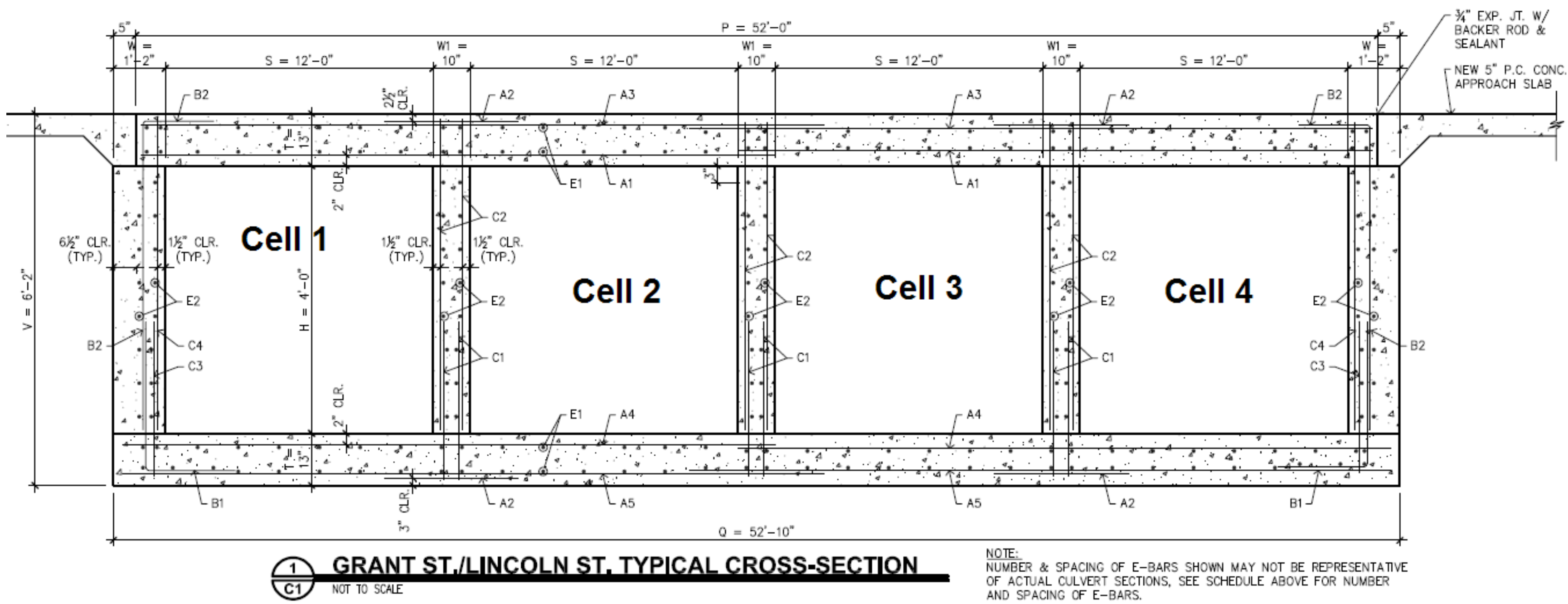


Figure 5-9 Typical Reinforced Concrete Box (RCB) Culvert

CHAPTER VI

RESULTS

Computed water surface elevations for the Q_{10}, Q_{25}, Q_{50}, and Q_{100} rainfall events are shown in Table 6-1 through Table 6-4. These elevations, as well as the proposed channel flowline, were computed at the channel station adjacent to the houses. To determine these water surface elevations, both the normal depth and the depth due to gradually varied flow conditions were computed. The depths due to gradually varied flow were a result of the headwater created by the three culverts that were necessary to provide safe vehicular and pedestrian traffic flow over the channel. Once these depths were known, the larger depth was added to the channel flowline to obtain the water surface elevation at that channel station. The point of these tables is to show the direct comparison between those water surface elevations (WSEL) and the finished floor (FF) elevations of the adjacent houses.

The results of the channel sizing calculated by FlowMaster are provided in Appendix A and shown in Figure 6-1. Typical cross sections for the concrete lined channel and the grass-lined channel can be seen in Figure 6-2 and Figure 6-3, respectively. These figures also show the elevation of the studied rainfall events in relation to the top and bottom of the channel. Because the flows in the concrete lined channel fall within the supercritical region, they are all contained within the channel. The flows in the grass-lined channel, on the other hand, fall within the subcritical region. As expected, the water surface elevations in this channel are higher than those in the concrete lined channel. Figure 6-3 shows that all of the rain events are confined within the channel except for the 100-year event. The requirement of the local government is that the water surface elevation associated with the 50-year event be held within the channel and that of the 100-year event be lower than adjacent houses. Both of these requirements are met.

As Figures 6-2 and 6-3 indicate, the proposed channel will have a considerably larger flow capacity than the existing 10-foot flat bottom channel with 3:1 side slopes. This existing channel will remain in-place and parallel to proposed channel sections A and B. The remaining length of the existing channel will be modified to meet the proposed model conditions.

The results of the culvert sizing calculated by CulvertMaster are provided in Appendix B and shown in Figure 6-4. A typical cross section of the proposed culvert was shown in Figure 5-9. Given the large flow rates and the presence of existing 3-ft high x 10-ft wide reinforced concrete boxes at culvert locations C1 and C2, it was obvious that the proposed culverts would need to be larger in both size and number of cells. Although the same reason would apply to C3, this proposed culvert would replace two 36-in. reinforced concrete pipes.

Table 6-1 House Finished Floor Elevation vs. Q_{10} Water Surface Elevation

Channel Section	Adjacent House			Channel Flowline	Q_{10} (ft)				Height of FF above WSEL (ft)
	ID *	Address	FF Elevation **		Normal Depth	Critical Depth	GVF Depth	WSEL ***	
A	a1	1515 W. Purdue	1294.28	1286.59	2.08	1.35	2.44	1289.03	5.25
	a2	3628 N. Grant	1294.62						5.59
B	b1	3901 N. Grant	1291.32	1284.77	2.08	1.35	2.10	1286.87	4.45
	b2	3715 N. Grant	1290.29						3.42
C	c1	3802 N. Lincoln	1289.33	1283.20	1.90	2.24	Beyond Flow Profile	1285.44	3.89
	c2	3716 N. Lincoln	1288.79						3.35
D	d1	3801 N. Lincoln	1292.00	1281.95	1.58	2.24	Beyond Flow Profile	1284.19	7.81
	d2	3719 N. Lincoln	1287.07						2.88

* See Figure 3-2

** FF = Finished Floor

*** Water Suface Elevation based on shaded depths.

Table 6-2 House Finished Floor Elevation vs. Q_{25} Water Surface Elevation

Channel Section	Adjacent House			Channel Flowline	Q_{25} (ft)				Height of FF above WSEL (ft)
	ID *	Address	FF Elevation **		Normal Depth	Critical Depth	GVF Depth	WSEL ***	
A	a1	1515 W. Purdue	1294.28	1286.59	2.49	1.65	3.17	1289.76	4.52
	a2	3628 N. Grant	1294.62						4.86
B	b1	3901 N. Grant	1291.32	1284.77	2.49	1.65	2.59	1287.36	3.96
	b2	3715 N. Grant	1290.29						2.93
C	c1	3802 N. Lincoln	1289.33	1283.20	2.42	2.92	Beyond Flow Profile	1286.12	3.21
	c2	3716 N. Lincoln	1288.79						2.67
D	d1	3801 N. Lincoln	1292.00	1281.95	2.01	2.92	Beyond Flow Profile	1284.87	7.13
	d2	3719 N. Lincoln	1287.07						2.20

* See Figure 3-2
** FF = Finished Floor
*** Water Suface Elevation based on shaded depths.

Table 6-3 House Finished Floor Elevation vs. Q_{50} Water Surface Elevation

Channel Section	Adjacent House			Channel Flowline	Q_{50} (ft)				Height of FF above WSEL (ft)
	ID *	Address	FF Elevation **		Normal Depth	Critical Depth	GVF Depth	WSEL ***	
A	a1	1515 W. Purdue	1294.28	1286.59	2.79	1.87	3.46	1290.05	4.23
	a2	3628 N. Grant	1294.62						4.57
B	b1	3901 N. Grant	1291.32	1284.77	2.79	1.87	2.96	1287.73	3.59
	b2	3715 N. Grant	1290.29						2.56
C	c1	3802 N. Lincoln	1289.33	1283.20	2.77	3.36	Beyond Flow Profile	1286.56	2.77
	c2	3716 N. Lincoln	1288.79						2.23
D	d1	3801 N. Lincoln	1292.00	1281.95	2.30	3.36	Beyond Flow Profile	1285.31	6.69
	d2	3719 N. Lincoln	1287.07						1.76

* See Figure 3-2
** FF = Finished Floor
*** Water Suface Elevation based on shaded depths.

Table 6-4 House Finished Floor Elevation vs. Q_{100} Water Surface Elevation

Channel Section	Adjacent House			Channel Flowline	Q_{100} (ft)				Height of FF above WSEL (ft)
	ID *	Address	FF Elevation **		Normal Depth	Critical Depth	GVF Depth	WSEL ***	
A	a1	1515 W. Purdue	1294.28	1286.59	3.10	2.10	3.94	1290.53	3.75
	a2	3628 N. Grant	1294.62						4.09
B	b1	3901 N. Grant	1291.32	1284.77	3.10	2.10	3.35	1288.12	3.20
	b2	3715 N. Grant	1290.29						2.17
C	c1	3802 N. Lincoln	1289.33	1283.20	3.10	3.80	Beyond Flow Profile	1287.00	2.33
	c2	3716 N. Lincoln	1288.79						1.79
D	d1	3801 N. Lincoln	1292.00	1281.95	2.58	3.80	Beyond Flow Profile	1285.75	6.25
	d2	3719 N. Lincoln	1287.07						1.32

* See Figure 3-2
** FF = Finished Floor
*** Water Suface Elevation based on shaded depths.

	Channel A	Channel B	Channel C	Channel D	Channel E
Length (ft)	430	269	375	734	637
Bottom Width (ft)	50	50	28	28	35
Side Slope (H:V)	4:1	4:1	4:1	4:1	4:1
Min. Depth (ft)	3	3	3	3	3
Slope (%)	0.5	0.5	0.42	0.78	0.42
Surface Lining	Bermuda Grass	Bermuda Grass	Concrete	Concrete	Bermuda Grass
Manning's n	0.04	0.04	0.013	0.013	0.04
Flow Condition	Subcritical	Subcritical	Supercritical	Supercritical	Subcritical
Additional	10' Wide Concrete Trickle Channel	10' Wide Concrete Trickle Channel	12' High Vegetated Retaining Wall	NA	10' Wide Concrete Trickle Channel

Figure 6-1. Summary of Channel Design Results

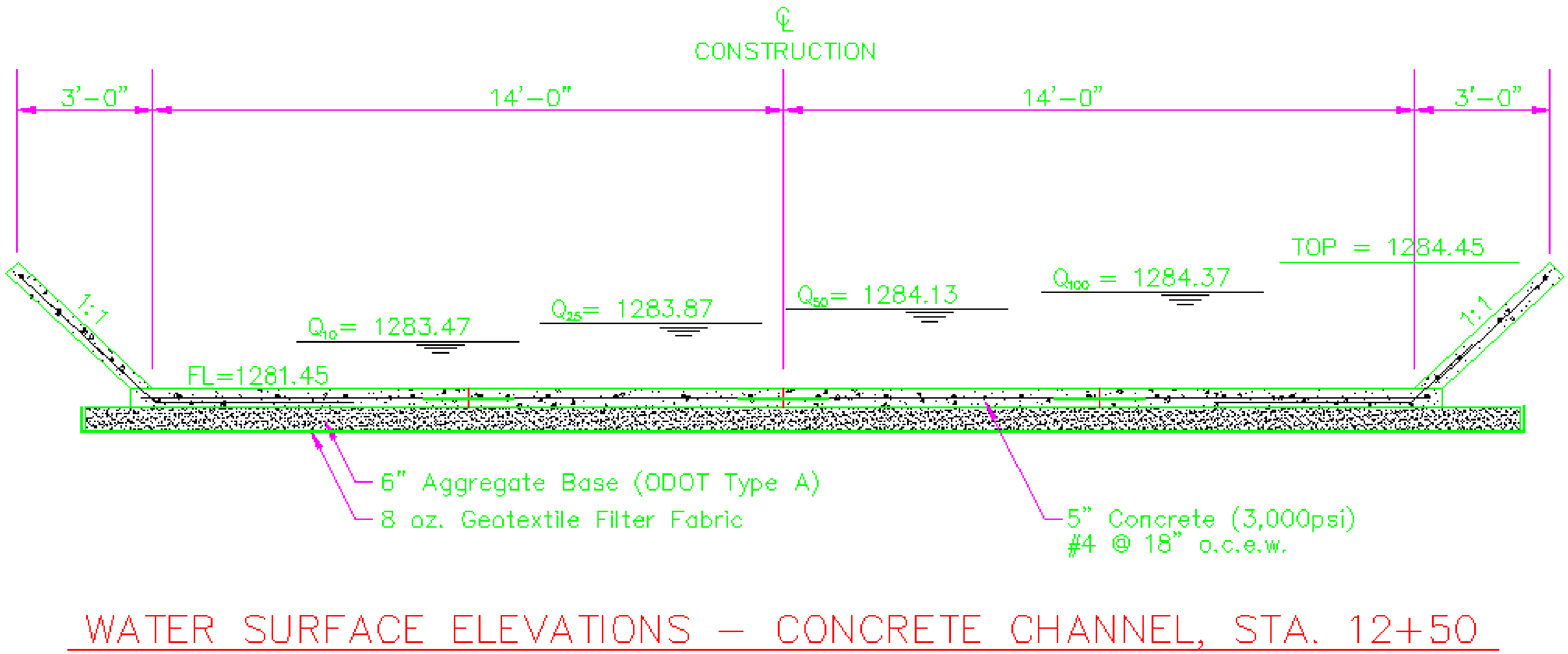
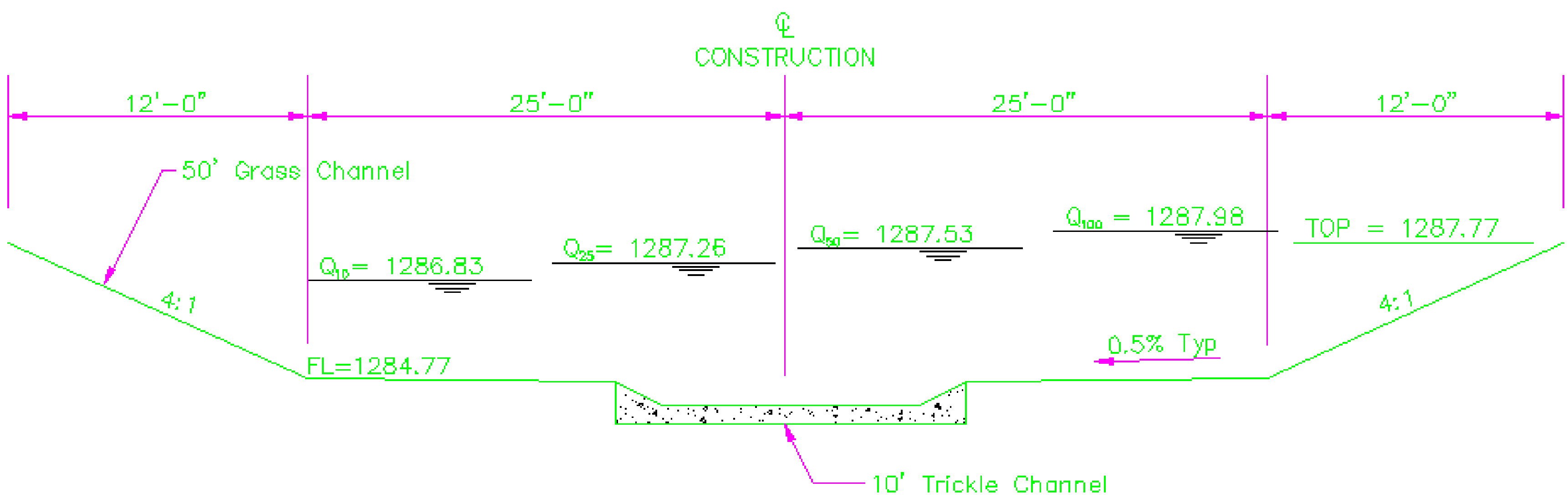


Figure 6-2 Water Surface Elevations in a Concrete Lined Channel



WATER SURFACE ELEVATIONS – GRASS LINED CHANNEL, STA. 5+50

Figure 6-3 Water Surface Elevations in a Grass Lined Channel

	C1 Grant Street Culvert	C2 Lincoln Street Culvert	C3 Crosslin Park Road Culvert
Shape	Box	Box	Box
Material	Concrete	Concrete	Concrete
No. of Openings	4	4	2
Opening Width (ft)	10	12	12
Opening Height (ft)	4	4	4
Length (ft)	36	32	55
Slope (%)	0.33	0.19	0.51

Figure 6-4. Summary of Culvert Design Results

Discussion of Results

Tables 6-1 through 6-4 indicate that the finished floor of every house is above the corresponding water surface elevation. This translates to the houses being protected from every rainfall event in the study. Notably, this was accomplished without the need to move or purchase adjacent houses. This fact was unexpected for two reasons. The first reason is that these houses were originally built in a subdivision outside of the city limits and thus not subject to any stormwater policies or regulations. This meant that houses would not be elevated above future runoff elevations. The second reason is that both Grant Street and Lincoln Street contain a sanitary sewer pipeline that could not be lowered without a significant capital improvement cost. This limited how low the channel flowline could be excavated.

The final geometry of the channel and culverts was determined using a random procedure. The bottom width of the channel was varied in FlowMaster until a water surface elevation was found that did not flood adjacent houses. Upon inputting a bottom width, the model immediately calculated the depth corresponding to uniform flow conditions. This depth was added to the channel flowline elevation and compared to the finished floor elevation of the adjacent house. For the culvert modeling, the size and number of concrete boxes were varied in CulvertMaster until the computed headwater resulted in the same outcome as above except using the gradually varied flow model in FlowMaster. This process was considered optimized when the corresponding water surface elevation fell just below the adjacent house's finished floor elevation for the 100-year rainfall event.

Another observation to make of the above tables is which depth is used in the calculation of the water surface elevation. First, the governing depth varies between the uniform flow condition and gradually varied flow condition. When the depth is governed by the gradually varied flow condition, it is a result of a downstream culvert and its corresponding flow profile. CulvertMaster allows the user to determine the distance, by trial and error, from the culvert for which these flow profiles are operational. Secondly, it can be seen that the governing depth is not always the lower depth. This is a result of the flow profile associated with the gradually varied flow condition extending from the downstream culvert. When this flow profile extends beyond an adjacent house, the governing depth is that associated with the gradually varied flow condition.

The houses along this channel are also in close proximity to the roadway culverts. Therefore, the headwater produced by these culverts must also be compared to the finished floor elevations of the immediate downstream houses. If the adjacent roadway is not located at an elevation higher than this headwater, the house may still experience flooding. This is the case for the houses located at 3715 N. Grand and 3719 N. Lincoln. During the 100-year storm event, the headwater produced by the adjacent, upstream culverts will be higher than the house. A check of survey data shows that the roadway is higher than the finished floor elevation.

A review of Figure 6-2 and Figure 6-3 shows the channel geometry needed to adequately convey the 100-year storm event without flooding any houses. One abnormality is that downstream Channel E is more narrow than the upstream Channels A and B. This is due to Channel E not having adjacent houses and the fact it is located in a park, which is allowed to flood.

A review of the CulvertMaster output in Appendix B shows that Crosslin Park

Road will experience overtopping. During the 100-year rainfall event, the headwater for this culvert is 18-inches higher than the top of curb. According to local government officials, this is acceptable since the park should not be in use during such large storm events. This smaller culvert size also represents a decrease of approximately 50% in construction cost.

A last remark is that both FlowMaster and CulvertMaster hydraulic models sized the proposed channels and culverts using flow rates that were established for fully developed basin conditions. This translates to these drainage elements being well oversized for current as well as short- to medium-term development conditions while providing the necessary capacity for the basin's maximum development plan.

CHAPTER VII

CONCLUSIONS

The following conclusions can be made based on the results of this study.

1. Both FlowMaster and CulvertMaster software can successfully be applied in solving urban stormwater problems for small and medium sized communities.
2. The results of this hydraulic analysis are reasonable. Channels with 50-foot wide flat bottoms are not common in small and medium sized communities unless constructed in post-developed conditions. In addition, the culverts sized in this study are only slightly larger than the existing, downstream culverts. However, there is detention between these culverts in the form of Crosslin Lake.
3. Both FlowMaster and CulvertMaster software are easily manipulated, user friendly, and involve a small learning curve. They allow the user to obtain acceptable results while maintaining constraints such as minimum backwater effects that do not flood adjacent houses.

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APPPENDIX A

FLOWMASTER RESULTS
10-, 25-, 50-, AND 100-YEAR DESIGN FLOWS

Worksheet for Channel A (US of Grant RCB) - Q10

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient		0.040	
Channel Slope		0.00500	ft/ft
Left Side Slope		3.00	ft/ft (H:V)
Right Side Slope		3.00	ft/ft (H:V)
Bottom Width		50.00	ft
Discharge		463.00	ft³/s

Results

Normal Depth		2.08	ft
Flow Area		116.92	ft²
Wetted Perimeter		63.15	ft
Top Width		62.47	ft
Critical Depth		1.35	ft
Critical Slope		0.02184	ft/ft
Velocity		3.96	ft/s
Velocity Head		0.24	ft
Specific Energy		2.32	ft
Froude Number		0.51	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth		2.64	ft
Length		70.00	ft
Number Of Steps		50	

GVF Output Data

Upstream Depth		2.44	ft
Profile Description	M1		
Profile Headloss		0.15	ft
Downstream Velocity		3.03	ft/s
Upstream Velocity		3.31	ft/s
Normal Depth		2.08	ft
Critical Depth		1.35	ft
Channel Slope		0.00500	ft/ft
Critical Slope		0.02184	ft/ft

Worksheet for Channel A (US of Grant RCB) - Q25

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	631.00	ft³/s

Results

Normal Depth	2.49	ft
Flow Area	143.04	ft²
Wetted Perimeter	65.74	ft
Top Width	64.93	ft
Critical Depth	1.65	ft
Critical Slope	0.02056	ft/ft
Velocity	4.41	ft/s
Velocity Head	0.30	ft
Specific Energy	2.79	ft
Froude Number	0.52	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	3.24	ft
Length	70.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.02	ft
Profile Description	M1	
Profile Headloss	0.13	ft
Downstream Velocity	3.26	ft/s
Upstream Velocity	3.54	ft/s
Normal Depth	2.49	ft
Critical Depth	1.65	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.02056	ft/ft

Worksheet for Channel A (US of Grant RCB) - Q50

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	768.00	ft³/s

Results

Normal Depth	2.79	ft
Flow Area	162.80	ft²
Wetted Perimeter	67.64	ft
Top Width	66.74	ft
Critical Depth	1.87	ft
Critical Slope	0.01980	ft/ft
Velocity	4.72	ft/s
Velocity Head	0.35	ft
Specific Energy	3.14	ft
Froude Number	0.53	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	3.69	ft
Length	70.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.46	ft
Profile Description	M1	
Profile Headloss	0.12	ft
Downstream Velocity	3.41	ft/s
Upstream Velocity	3.68	ft/s
Normal Depth	2.79	ft
Critical Depth	1.87	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01980	ft/ft

Worksheet for Channel A (US of Grant RCB) - Q100

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	923.00	ft³/s

Results

Normal Depth	3.10	ft
Flow Area	183.88	ft²
Wetted Perimeter	69.61	ft
Top Width	68.60	ft
Critical Depth	2.10	ft
Critical Slope	0.01913	ft/ft
Velocity	5.02	ft/s
Velocity Head	0.39	ft
Specific Energy	3.49	ft
Froude Number	0.54	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	4.18	ft
Length	70.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.94	ft
Profile Description	M1	
Profile Headloss	0.11	ft
Downstream Velocity	3.53	ft/s
Upstream Velocity	3.79	ft/s
Normal Depth	3.10	ft
Critical Depth	2.10	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01913	ft/ft

Worksheet for Channel B (DS of Grant RCB) - Q10

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	463.00	ft³/s

Results

Normal Depth	2.08	ft
Flow Area	116.92	ft²
Wetted Perimeter	63.15	ft
Top Width	62.47	ft
Critical Depth	1.35	ft
Critical Slope	0.02184	ft/ft
Velocity	3.96	ft/s
Velocity Head	0.24	ft
Specific Energy	2.32	ft
Froude Number	0.51	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	2.24	ft
Length	269.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	2.09	ft
Profile Description	M1	
Profile Headloss	1.19	ft
Downstream Velocity	3.64	ft/s
Upstream Velocity	3.94	ft/s
Normal Depth	2.08	ft
Critical Depth	1.35	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.02184	ft/ft

Worksheet for Channel B (DS of Grant RCB) - Q25

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	631.00	ft³/s

Results

Normal Depth	2.49	ft
Flow Area	143.04	ft²
Wetted Perimeter	65.74	ft
Top Width	64.93	ft
Critical Depth	1.65	ft
Critical Slope	0.02056	ft/ft
Velocity	4.41	ft/s
Velocity Head	0.30	ft
Specific Energy	2.79	ft
Froude Number	0.52	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	2.92	ft
Length	199.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	2.59	ft
Profile Description	M1	
Profile Headloss	0.67	ft
Downstream Velocity	3.68	ft/s
Upstream Velocity	4.21	ft/s
Normal Depth	2.49	ft
Critical Depth	1.65	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.02056	ft/ft

City of Enid

Worksheet for Channel B (DS of Grant RCB) - Q50

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	768.00	ft³/s

Results

Normal Depth	2.79	ft
Flow Area	162.80	ft²
Wetted Perimeter	67.64	ft
Top Width	66.74	ft
Critical Depth	1.87	ft
Critical Slope	0.01980	ft/ft
Velocity	4.72	ft/s
Velocity Head	0.35	ft
Specific Energy	3.14	ft
Froude Number	0.53	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	3.36	ft
Length	199.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	2.96	ft
Profile Description	M1	
Profile Headloss	0.59	ft
Downstream Velocity	3.80	ft/s
Upstream Velocity	4.41	ft/s
Normal Depth	2.79	ft
Critical Depth	1.87	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01980	ft/ft

Worksheet for Channel B (DS of Grant RCB) - Q100

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	923.00	ft³/s

Results

Normal Depth	3.10	ft
Flow Area	183.88	ft²
Wetted Perimeter	69.61	ft
Top Width	68.60	ft
Critical Depth	2.10	ft
Critical Slope	0.01913	ft/ft
Velocity	5.02	ft/s
Velocity Head	0.39	ft
Specific Energy	3.49	ft
Froude Number	0.54	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	3.80	ft
Length	269.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.26	ft
Profile Description	M1	
Profile Headloss	0.80	ft
Downstream Velocity	3.96	ft/s
Upstream Velocity	4.74	ft/s
Normal Depth	3.10	ft
Critical Depth	2.10	ft
Channel Slope	0.00500	ft/ft
Critical Slope	0.01913	ft/ft

Worksheet for Channel C (US of Lincoln RCB) - Q10

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.014	
Channel Slope	0.00420	ft/ft
Left Side Slope	1.00	ft/ft (H:V)
Right Side Slope	1.00	ft/ft (H:V)
Bottom Width	28.00	ft
Discharge	556.00	ft³/s

Results

Normal Depth	1.90	ft
Flow Area	56.73	ft²
Wetted Perimeter	33.37	ft
Top Width	31.80	ft
Critical Depth	2.24	ft
Critical Slope	0.00241	ft/ft
Velocity	9.80	ft/s
Velocity Head	1.49	ft
Specific Energy	3.39	ft
Froude Number	1.29	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	2.67	ft
Length	40.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	2.24	ft
Profile Description	Composite S1 -> S2	
Profile Headloss	-0.26	ft
Downstream Velocity	6.79	ft/s
Upstream Velocity	8.20	ft/s
Normal Depth	1.90	ft
Critical Depth	2.24	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.00241	ft/ft

Worksheet for Channel C (US of Lincoln RCB) - Q25

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient		0.014	
Channel Slope		0.00420	ft/ft
Left Side Slope		1.00	ft/ft (H:V)
Right Side Slope		1.00	ft/ft (H:V)
Bottom Width		28.00	ft
Discharge		835.00	ft³/s

Results

Normal Depth		2.42	ft
Flow Area		73.68	ft²
Wetted Perimeter		34.85	ft
Top Width		32.84	ft
Critical Depth		2.92	ft
Critical Slope		0.00226	ft/ft
Velocity		11.33	ft/s
Velocity Head		2.00	ft
Specific Energy		4.42	ft
Froude Number		1.33	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth		3.56	ft
Length		65.00	ft
Number Of Steps		50	

GVF Output Data

Upstream Depth		2.92	ft
Profile Description	Composite S1 -> S2		
Profile Headloss		-0.37	ft
Downstream Velocity		7.43	ft/s
Upstream Velocity		9.26	ft/s
Normal Depth		2.42	ft
Critical Depth		2.92	ft
Channel Slope		0.00420	ft/ft
Critical Slope		0.00226	ft/ft

City of Enid

Worksheet for Channel C (US of Lincoln RCB) - Q50

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.014	
Channel Slope	0.00420	ft/ft
Left Side Slope	1.00	ft/ft (H:V)
Right Side Slope	1.00	ft/ft (H:V)
Bottom Width	28.00	ft
Discharge	1042.00	ft³/s

Results

Normal Depth	2.77	ft
Flow Area	85.09	ft²
Wetted Perimeter	35.82	ft
Top Width	33.53	ft
Critical Depth	3.36	ft
Critical Slope	0.00219	ft/ft
Velocity	12.25	ft/s
Velocity Head	2.33	ft
Specific Energy	5.10	ft
Froude Number	1.36	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	4.12	ft
Length	75.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.36	ft
Profile Description	Composite S1 -> S2	
Profile Headloss	-0.44	ft
Downstream Velocity	7.87	ft/s
Upstream Velocity	9.88	ft/s
Normal Depth	2.77	ft
Critical Depth	3.36	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.00219	ft/ft

Worksheet for Channel C (US of Lincoln RCB) - Q100

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.014	
Channel Slope	0.00420	ft/ft
Left Side Slope	1.00	ft/ft (H:V)
Right Side Slope	1.00	ft/ft (H:V)
Bottom Width	28.00	ft
Discharge	1261.00	ft³/s

Results

Normal Depth	3.10	ft
Flow Area	96.41	ft²
Wetted Perimeter	36.77	ft
Top Width	34.20	ft
Critical Depth	3.80	ft
Critical Slope	0.00213	ft/ft
Velocity	13.08	ft/s
Velocity Head	2.66	ft
Specific Energy	5.76	ft
Froude Number	1.37	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	4.71	ft
Length	95.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.80	ft
Profile Description	Composite S1 -> S2	
Profile Headloss	-0.51	ft
Downstream Velocity	8.18	ft/s
Upstream Velocity	10.45	ft/s
Normal Depth	3.10	ft
Critical Depth	3.80	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.00213	ft/ft

Worksheet for Channel D (DS of Lincoln RCB) - Q10

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient		0.014	
Channel Slope		0.00780	ft/ft
Left Side Slope		1.00	ft/ft (H:V)
Right Side Slope		1.00	ft/ft (H:V)
Bottom Width		28.00	ft
Discharge		556.00	ft³/s

Results

Normal Depth		1.58	ft
Flow Area		46.60	ft²
Wetted Perimeter		32.46	ft
Top Width		31.15	ft
Critical Depth		2.24	ft
Critical Slope		0.00241	ft/ft
Velocity		11.93	ft/s
Velocity Head		2.21	ft
Specific Energy		3.79	ft
Froude Number		1.72	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth		3.05	ft
Length		50.00	ft
Number Of Steps		50	

GVF Output Data

Upstream Depth		2.24	ft
Profile Description	Composite S1 -> S2		
Profile Headloss		-0.42	ft
Downstream Velocity		5.87	ft/s
Upstream Velocity		8.20	ft/s
Normal Depth		1.58	ft
Critical Depth		2.24	ft
Channel Slope		0.00780	ft/ft
Critical Slope		0.00241	ft/ft

Worksheet for Channel D (DS of Lincoln RCB) - Q25

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.014	
Channel Slope	0.00780	ft/ft
Left Side Slope	1.00	ft/ft (H:V)
Right Side Slope	1.00	ft/ft (H:V)
Bottom Width	28.00	ft
Discharge	835.00	ft³/s

Results

Normal Depth	2.01	ft
Flow Area	60.37	ft²
Wetted Perimeter	33.69	ft
Top Width	32.02	ft
Critical Depth	2.92	ft
Critical Slope	0.00226	ft/ft
Velocity	13.83	ft/s
Velocity Head	2.97	ft
Specific Energy	4.98	ft
Froude Number	1.78	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	4.21	ft
Length	83.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	2.92	ft
Profile Description	Composite S1 -> S2	
Profile Headloss	-0.65	ft
Downstream Velocity	6.16	ft/s
Upstream Velocity	9.26	ft/s
Normal Depth	2.01	ft
Critical Depth	2.92	ft
Channel Slope	0.00780	ft/ft
Critical Slope	0.00226	ft/ft

Worksheet for Channel D (DS of Lincoln RCB) - Q50

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.014	
Channel Slope	0.00780	ft/ft
Left Side Slope	1.00	ft/ft (H:V)
Right Side Slope	1.00	ft/ft (H:V)
Bottom Width	28.00	ft
Discharge	1042.00	ft³/s

Results

Normal Depth	2.30	ft
Flow Area	69.61	ft²
Wetted Perimeter	34.50	ft
Top Width	32.59	ft
Critical Depth	3.36	ft
Critical Slope	0.00219	ft/ft
Velocity	14.97	ft/s
Velocity Head	3.48	ft
Specific Energy	5.78	ft
Froude Number	1.81	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	5.51	ft
Length	165.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.36	ft
Profile Description	Composite S1 -> S2	
Profile Headloss	-0.86	ft
Downstream Velocity	5.64	ft/s
Upstream Velocity	9.88	ft/s
Normal Depth	2.30	ft
Critical Depth	3.36	ft
Channel Slope	0.00780	ft/ft
Critical Slope	0.00219	ft/ft

Worksheet for Channel D (DS of Lincoln RCB) - Q100

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.014	
Channel Slope	0.00780	ft/ft
Left Side Slope	1.00	ft/ft (H:V)
Right Side Slope	1.00	ft/ft (H:V)
Bottom Width	28.00	ft
Discharge	1261.00	ft³/s

Results

Normal Depth	2.58	ft
Flow Area	78.76	ft²
Wetted Perimeter	35.29	ft
Top Width	33.15	ft
Critical Depth	3.80	ft
Critical Slope	0.00213	ft/ft
Velocity	16.01	ft/s
Velocity Head	3.98	ft
Specific Energy	6.56	ft
Froude Number	1.83	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	7.44	ft
Length	375.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.80	ft
Profile Description	Composite S1 -> S2	
Profile Headloss	-0.72	ft
Downstream Velocity	4.78	ft/s
Upstream Velocity	10.45	ft/s
Normal Depth	2.58	ft
Critical Depth	3.80	ft
Channel Slope	0.00780	ft/ft
Critical Slope	0.00213	ft/ft

Worksheet for Channel E (US of Park Road RCB) - Q10

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00420	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	35.00	ft
Discharge	566.00	ft³/s

Results

Normal Depth	2.97	ft
Flow Area	130.31	ft²
Wetted Perimeter	53.77	ft
Top Width	52.81	ft
Critical Depth	1.90	ft
Critical Slope	0.02002	ft/ft
Velocity	4.34	ft/s
Velocity Head	0.29	ft
Specific Energy	3.26	ft
Froude Number	0.49	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	4.24	ft
Length	637.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	3.05	ft
Profile Description	M1	
Profile Headloss	1.48	ft
Downstream Velocity	2.80	ft/s
Upstream Velocity	4.21	ft/s
Normal Depth	2.97	ft
Critical Depth	1.90	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.02002	ft/ft

Worksheet for Channel E (US of Park Road RCB) - Q25

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00420	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	35.00	ft
Discharge	848.00	ft³/s

Results

Normal Depth	3.72	ft
Flow Area	171.82	ft²
Wetted Perimeter	58.54	ft
Top Width	57.33	ft
Critical Depth	2.45	ft
Critical Slope	0.01864	ft/ft
Velocity	4.94	ft/s
Velocity Head	0.38	ft
Specific Energy	4.10	ft
Froude Number	0.50	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	6.08	ft
Length	637.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	4.21	ft
Profile Description	M1	
Profile Headloss	0.80	ft
Downstream Velocity	2.62	ft/s
Upstream Velocity	4.23	ft/s
Normal Depth	3.72	ft
Critical Depth	2.45	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.01864	ft/ft

Worksheet for Channel E (US of Park Road RCB) - Q50

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00420	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	35.00	ft
Discharge	1064.00	ft³/s

Results

Normal Depth	4.22	ft
Flow Area	201.04	ft²
Wetted Perimeter	61.68	ft
Top Width	60.31	ft
Critical Depth	2.81	ft
Critical Slope	0.01793	ft/ft
Velocity	5.29	ft/s
Velocity Head	0.44	ft
Specific Energy	4.65	ft
Froude Number	0.51	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	7.76	ft
Length	637.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	5.51	ft
Profile Description	M1	
Profile Headloss	0.42	ft
Downstream Velocity	2.35	ft/s
Upstream Velocity	3.75	ft/s
Normal Depth	4.22	ft
Critical Depth	2.81	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.01793	ft/ft

Worksheet for Channel E (US of Park Road RCB) - Q100

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.040	
Channel Slope	0.00420	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	35.00	ft
Discharge	1290.00	ft³/s

Results

Normal Depth	4.69	ft
Flow Area	229.94	ft²
Wetted Perimeter	64.64	ft
Top Width	63.12	ft
Critical Depth	3.17	ft
Critical Slope	0.01737	ft/ft
Velocity	5.61	ft/s
Velocity Head	0.49	ft
Specific Energy	5.18	ft
Froude Number	0.52	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	9.92	ft
Length	637.00	ft
Number Of Steps	50	

GVF Output Data

Upstream Depth	7.44	ft
Profile Description	M1	
Profile Headloss	0.20	ft
Downstream Velocity	2.01	ft/s
Upstream Velocity	3.02	ft/s
Normal Depth	4.69	ft
Critical Depth	3.17	ft
Channel Slope	0.00420	ft/ft
Critical Slope	0.01737	ft/ft

APPENDIX B

CULVERTMASTER RESULTS
10-, 25-, 50-, AND 100-YEAR DESIGN FLOWS

Culvert Calculator Report
Grant Street Culvert – Q10

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,290.00	ft	Headwater Depth/Height	0.66	
Computed Headwater Elevation	1,288.36	ft	Discharge	463.00	cfs
Inlet Control HW Elev.	1,288.36	ft	Tailwater Elevation	1,287.69	ft
Outlet Control HW Elev.	1,288.33	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,285.72	ft	Downstream Invert	1,285.60	ft
Length	36.00	ft	Constructed Slope	0.003333	ft/ft

Hydraulic Profile					
Profile	S1		Depth, Downstream	1.56	Ft
Slope Type	Steep		Normal Depth	1.56	Ft
Flow Regime	Subcritical		Critical Depth	1.61	Ft
Velocity Downstream	7.40	ft/s	Critical Slope	0.003049	ft/ft

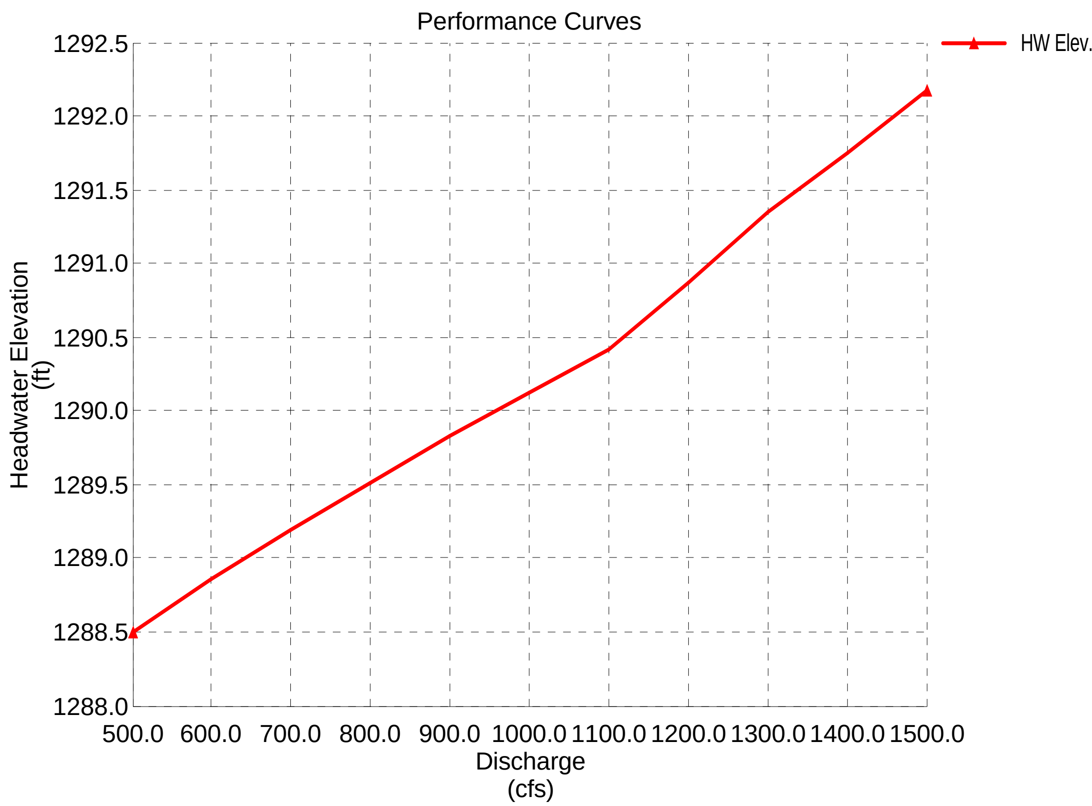
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	10.00	Ft
Section Size	10 x 4 ft		Rise	4.00	Ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,288.33	ft	Upstream Velocity Head	0.80	Ft
Ke	0.20		Entrance Loss	0.11	Ft

Inlet Control Properties					
Inlet Control HW Elev.	1,288.36	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	160.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Grant Street Culvert – Q10

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Grant Street Culvert – Q25

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,290.00	ft	Headwater Depth/Height	0.81	
Computed Headwater Elevation	1,288.96	ft	Discharge	631.00	cfs
Inlet Control HW Elev.	1,288.96	ft	Tailwater Elevation	1,288.15	ft
Outlet Control HW Elev.	1,288.93	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,285.72	ft	Downstream Invert	1,285.60	ft
Length	36.00	ft	Constructed Slope	0.003333	ft/ft

Hydraulic Profile					
Profile	S1		Depth, Downstream	1.92	ft
Slope Type	Steep		Normal Depth	1.92	ft
Flow Regime	Subcritical		Critical Depth	1.98	ft
Velocity Downstream	8.20	ft/s	Critical Slope	0.003060	ft/ft

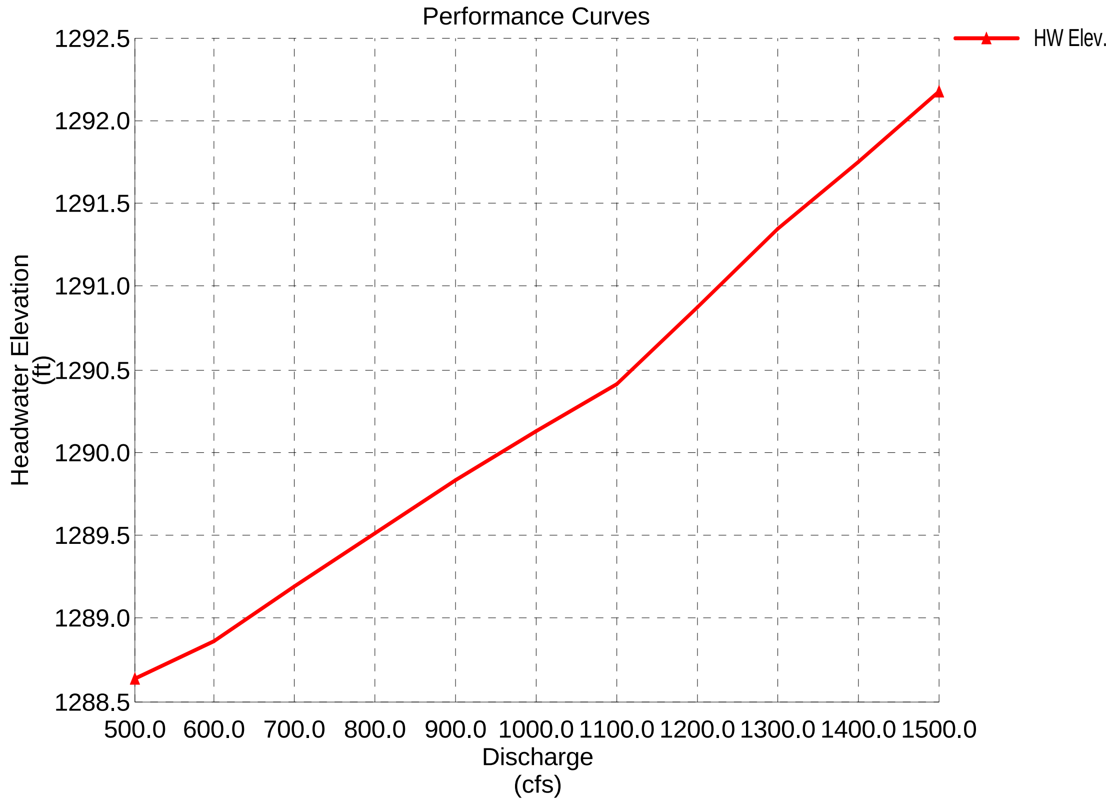
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	10.00	ft
Section Size	10 x 4 ft		Rise	4.00	ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,288.93	ft	Upstream Velocity Head	0.99	ft
Ke	0.20		Entrance Loss	0.13	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,288.96	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	160.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Grant Street Culvert – Q25

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Grant Street Culvert – Q50

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,290.00	ft	Headwater Depth/Height	0.92	
Computed Headwater Elevation	1,289.41	ft	Discharge	768.00	cfs
Inlet Control HW Elev.	1,289.41	ft	Tailwater Elevation	1,288.49	ft
Outlet Control HW Elev.	1,289.38	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,285.72	ft	Downstream Invert	1,285.60	ft
Length	36.00	ft	Constructed Slope	0.003333	ft/ft

Hydraulic Profile					
Profile	S1		Depth, Downstream	2.20	ft
Slope Type	Steep		Normal Depth	2.20	ft
Flow Regime	Subcritical		Critical Depth	2.25	ft
Velocity Downstream	8.73	ft/s	Critical Slope	0.003085	ft/ft

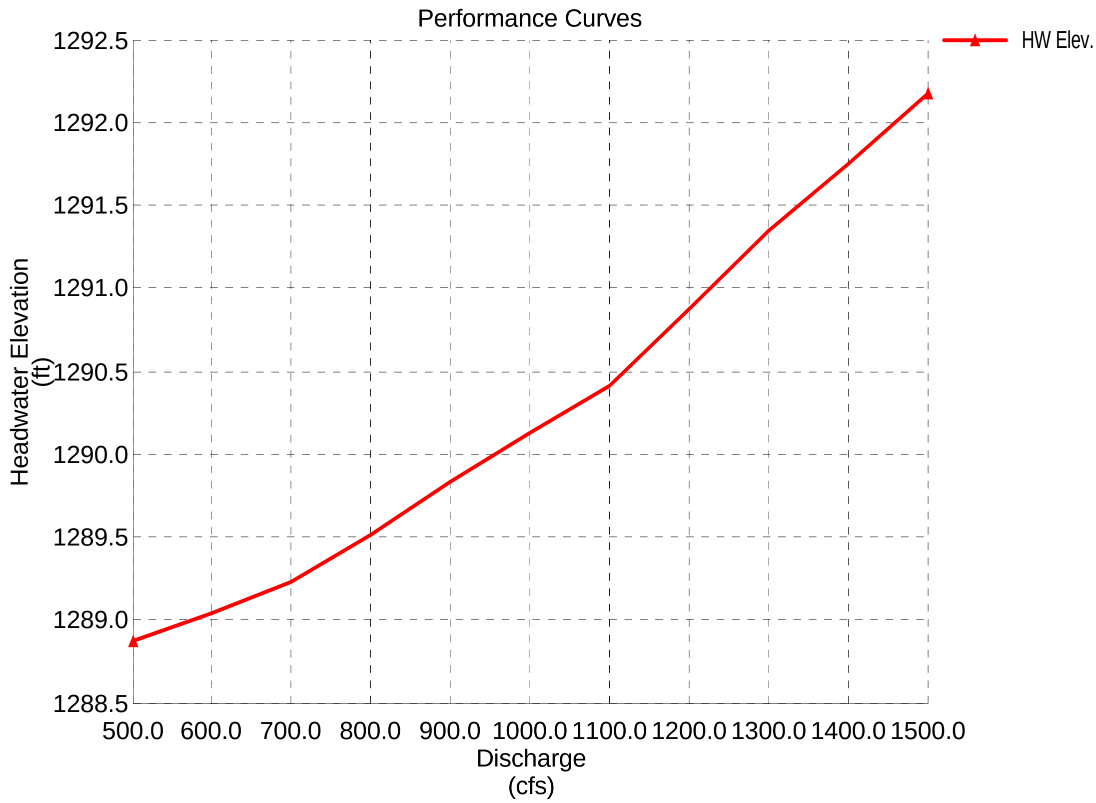
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	10.00	ft
Section Size	10 x 4 ft		Rise	4.00	ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,289.38	ft	Upstream Velocity Head	1.13	ft
Ke	0.20		Entrance Loss	0.15	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,289.41	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	160.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Grant Street Culvert – Q50

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	Cfs



Culvert Calculator Report
Grant Street Culvert – Q100

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,290.00	ft	Headwater Depth/Height	1.04	
Computed Headwater Elevation	1,289.90	ft	Discharge	923.00	cfs
Inlet Control HW Elev.	1,289.90	ft	Tailwater Elevation	1,288.86	ft
Outlet Control HW Elev.	1,289.87	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,285.72	ft	Downstream Invert	1,285.60	ft
Length	36.00	ft	Constructed Slope	0.003333	ft/ft

Hydraulic Profile					
Profile	S1		Depth, Downstream	2.49	ft
Slope Type	Steep		Normal Depth	2.49	ft
Flow Regime	Subcritical		Critical Depth	2.55	ft
Velocity Downstream	9.25	ft/s	Critical Slope	0.003122	ft/ft

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	10.00	ft
Section Size	10 x 4 ft		Rise	4.00	ft
Number Sections	4				

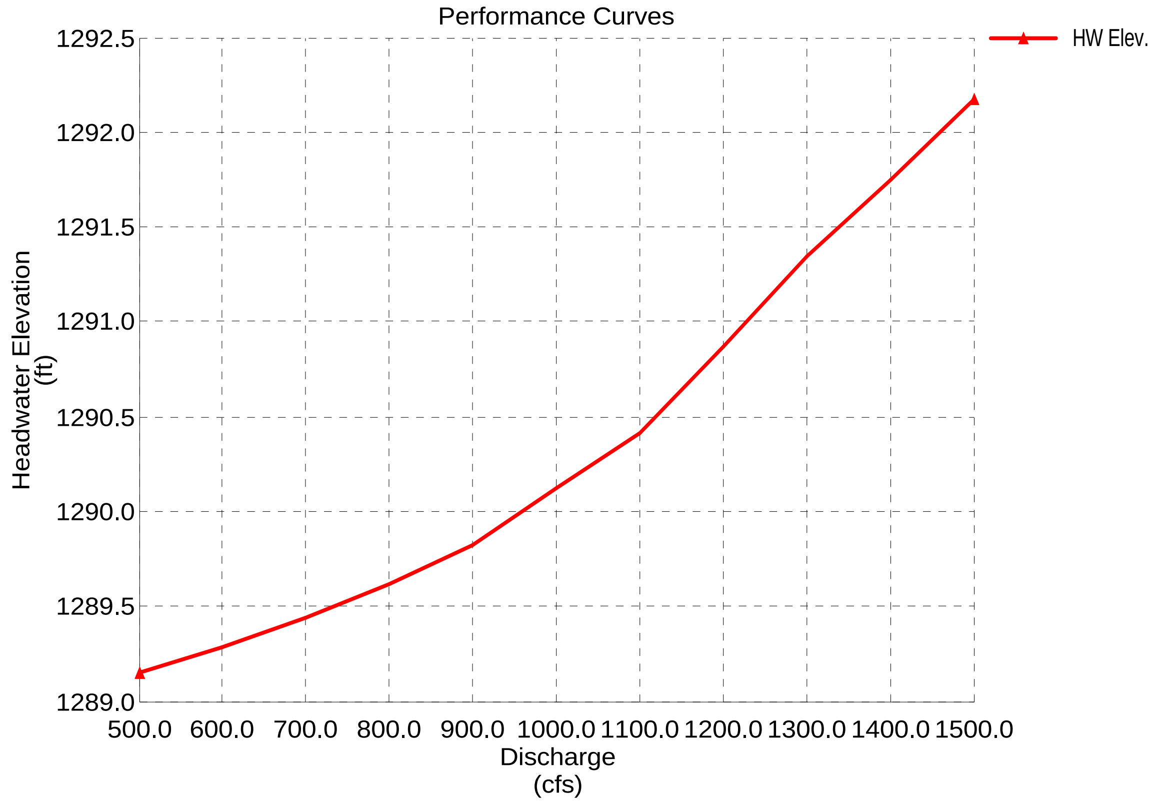
Outlet Control Properties					
Outlet Control HW Elev.	1,289.87	ft	Upstream Velocity Head	1.27	ft
Ke	0.20		Entrance Loss	0.17	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,289.90	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	160.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Grant Street Culvert – Q100

Range Data:

	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Lincoln Street Culvert – Q10

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,287.07	ft	Headwater Depth/Height	0.66	
Computed Headwater Elevation	1,285.38	ft	Discharge	556.00	cfs
Inlet Control HW Elev.	1,285.38	ft	Tailwater Elevation	1,284.26	ft
Outlet Control HW Elev.	1,285.31	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,282.74	ft	Downstream Invert	1,282.68	ft
Length	32.00	ft	Constructed Slope	0.001875	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	1.61	ft
Slope Type	Mild		Normal Depth	1.86	ft
Flow Regime	Subcritical		Critical Depth	1.61	ft
Velocity Downstream	7.20	ft/s	Critical Slope	0.002885	ft/ft

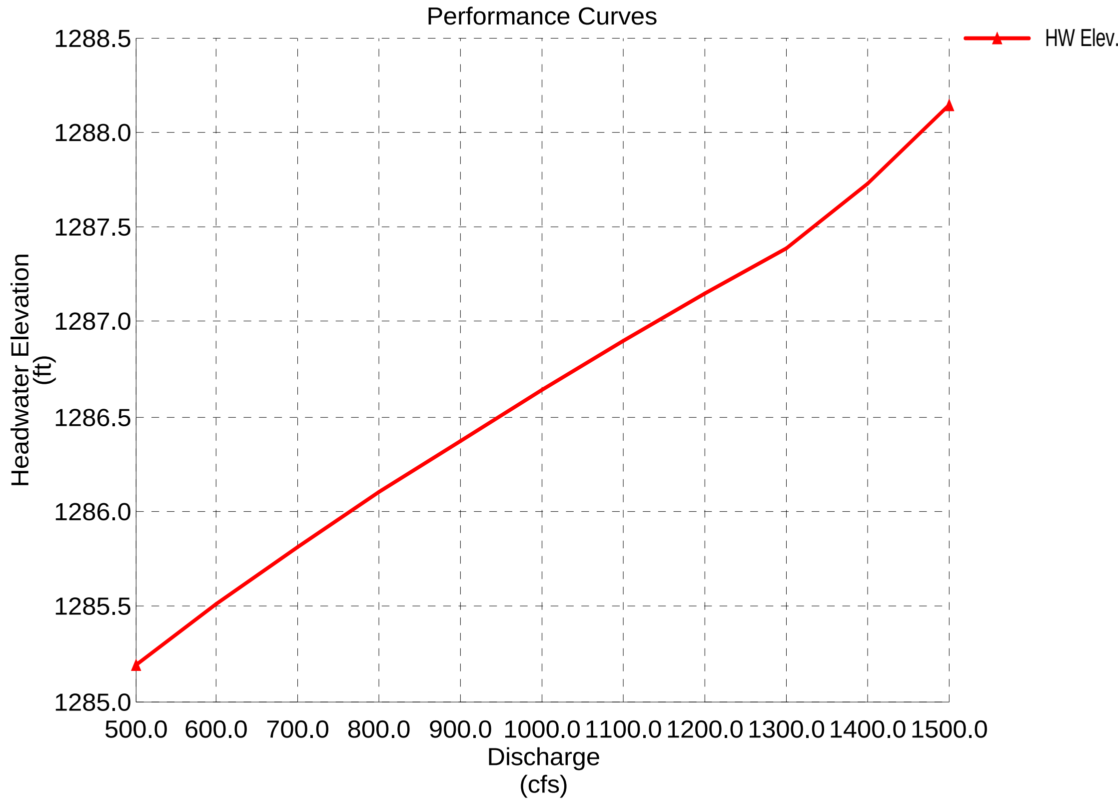
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,285.31	ft	Upstream Velocity Head	0.68	ft
Ke	0.20		Entrance Loss	0.14	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,285.38	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	192.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Lincoln Street Culvert – Q10

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	Cfs



Culvert Calculator Report
Lincoln Street Culvert – Q25

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,287.07	ft	Headwater Depth/Height	0.89	
Computed Headwater Elevation	1,286.30	ft	Discharge	835.00	cfs
Inlet Control HW Elev.	1,286.20	ft	Tailwater Elevation	1,285.60	ft
Outlet Control HW Elev.	1,286.30	ft	Control Type	Outlet Control	

Grades					
Upstream Invert	1,282.74	ft	Downstream Invert	1,282.68	ft
Length	32.00	ft	Constructed Slope	0.001875	ft/ft

Hydraulic Profile					
Profile	M1		Depth, Downstream	2.92	ft
Slope Type	Mild		Normal Depth	2.44	ft
Flow Regime	Subcritical		Critical Depth	2.11	ft
Velocity Downstream	5.96	ft/s	Critical Slope	0.002869	ft/ft

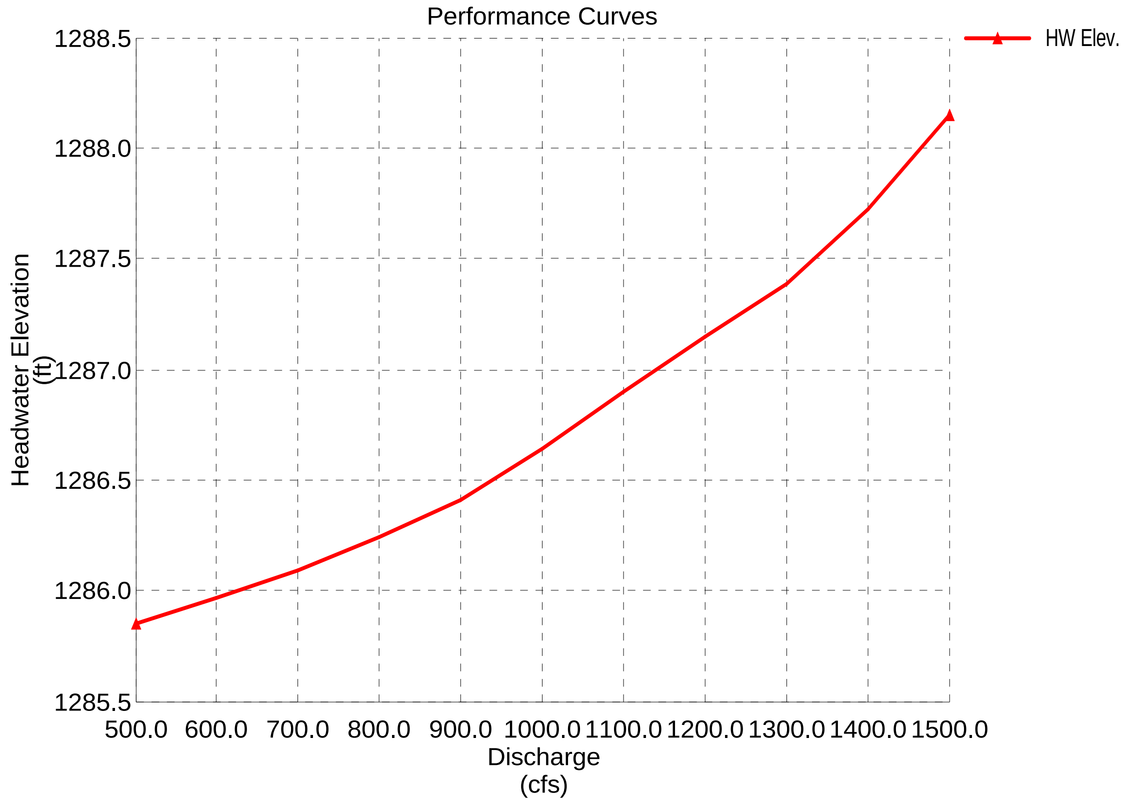
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,286.30	ft	Upstream Velocity Head	0.57	ft
Ke	0.20		Entrance Loss	0.11	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,286.20	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	192.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Lincoln Street Culvert – Q25

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Lincoln Street Culvert – Q50

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,287.07	ft	Headwater Depth/Height	1.03	
Computed Headwater Elevation	1,286.86	ft	Discharge	1,042.00	cfs
Inlet Control HW Elev.	1,286.75	ft	Tailwater Elevation	1,286.04	ft
Outlet Control HW Elev.	1,286.86	ft	Control Type	Outlet Control	

Grades					
Upstream Invert	1,282.74	ft	Downstream Invert	1,282.68	ft
Length	32.00	ft	Constructed Slope	0.001875	ft/ft

Hydraulic Profile					
Profile	M1		Depth, Downstream	3.36	ft
Slope Type	Mild		Normal Depth	2.83	ft
Flow Regime	Subcritical		Critical Depth	2.45	ft
Velocity Downstream	6.46	ft/s	Critical Slope	0.002884	ft/ft

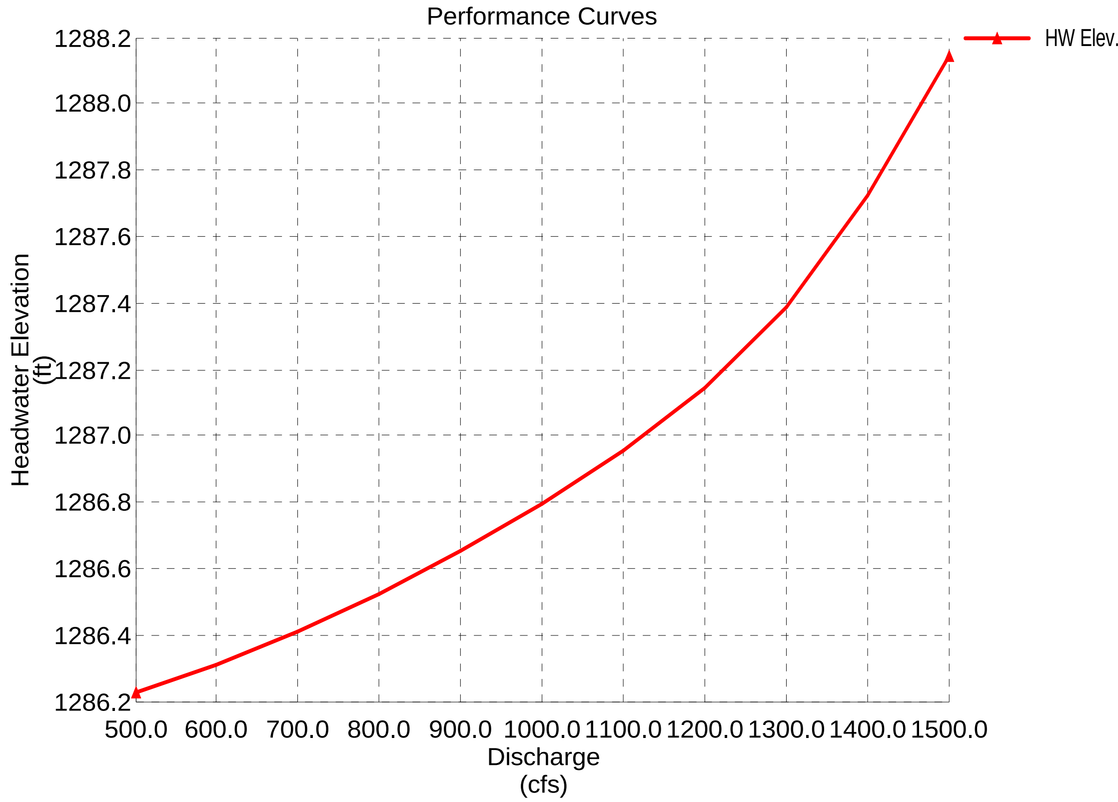
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,286.86	ft	Upstream Velocity Head	0.66	ft
Ke	0.20		Entrance Loss	0.13	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,286.75	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	192.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Lincoln Street Culvert – Q50

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Lincoln Street Culvert – Q100

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,287.07	ft	Headwater Depth/Height	1.18	
Computed Headwater Elevation	1,287.45	ft	Discharge	1,261.00	cfs
Inlet Control HW Elev.	1,287.29	ft	Tailwater Elevation	1,286.48	ft
Outlet Control HW Elev.	1,287.45	ft	Control Type	Outlet Control	

Grades					
Upstream Invert	1,282.74	ft	Downstream Invert	1,282.68	ft
Length	32.00	ft	Constructed Slope	0.001875	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	3.80	ft
Slope Type	Mild		Normal Depth	N/A	ft
Flow Regime	Subcritical		Critical Depth	2.78	ft
Velocity Downstream	6.91	ft/s	Critical Slope	0.002910	ft/ft

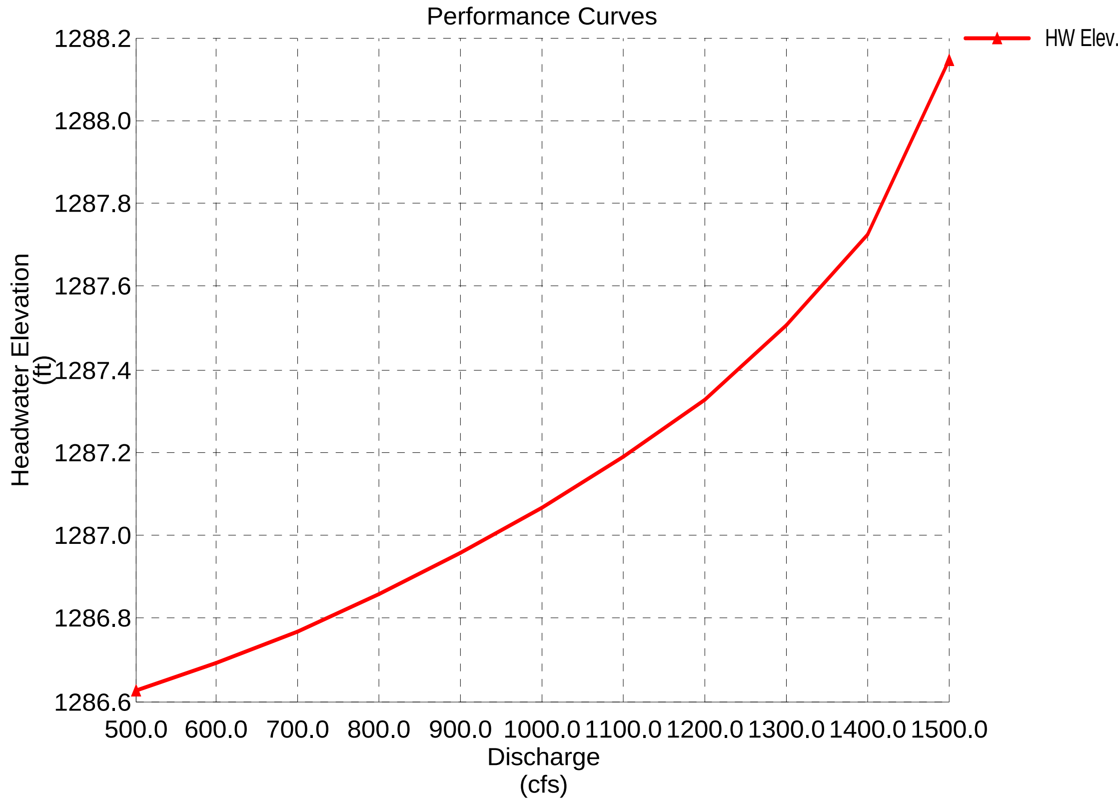
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	4				

Outlet Control Properties					
Outlet Control HW Elev.	1,287.45	ft	Upstream Velocity Head	0.73	ft
Ke	0.20		Entrance Loss	0.15	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,287.29	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	192.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Lincoln Street Culvert – Q100

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Crosslin Park Road Culvert – Q10

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,282.64	ft	Headwater Depth/Height	1.06	
Computed Headwater Elevation	1,278.52	ft	Discharge	566.00	cfs
Inlet Control HW Elev.	1,278.52	ft	Tailwater Elevation	1,275.72	ft
Outlet Control HW Elev.	1,278.42	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,274.28	ft	Downstream Invert	1,274.00	ft
Length	55.00	ft	Constructed Slope	0.005091	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	2.27	ft
Slope Type	Steep		Normal Depth	2.14	ft
Flow Regime	Supercritical		Critical Depth	2.59	ft
Velocity Downstream	10.39	ft/s	Critical Slope	0.002893	ft/ft

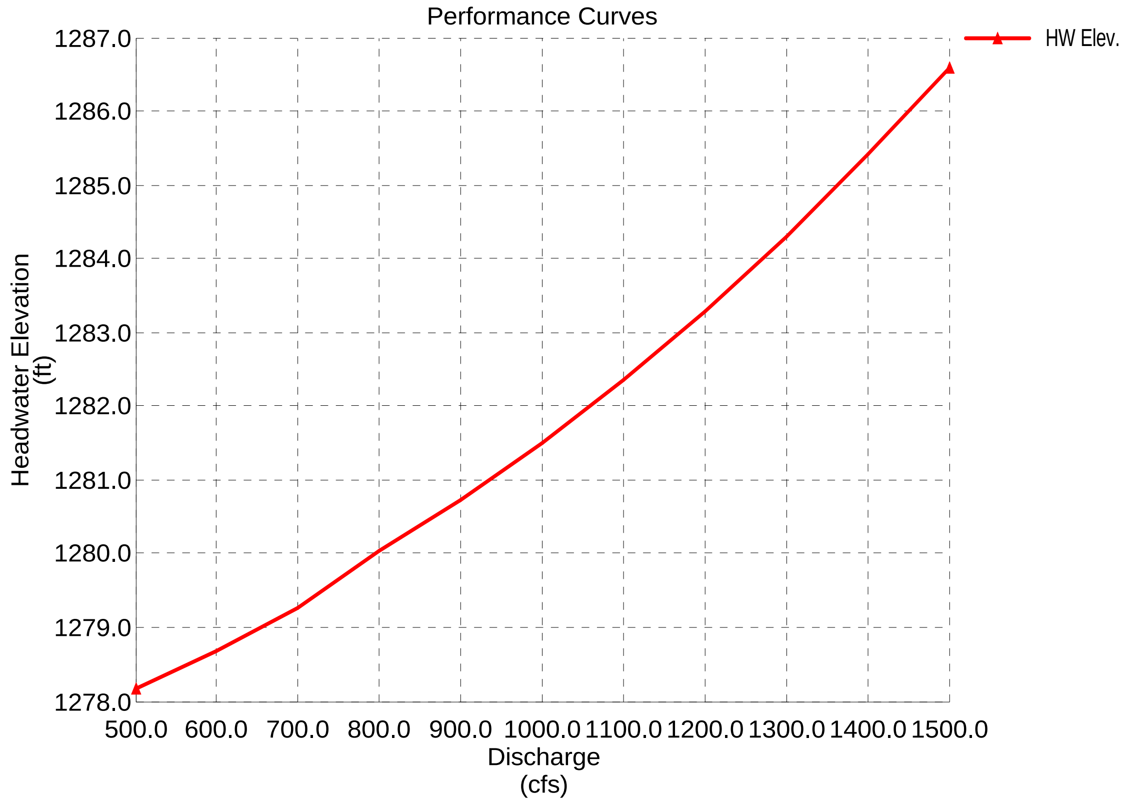
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	1,278.42	ft	Upstream Velocity Head	1.29	ft
Ke	0.20		Entrance Loss	0.26	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,278.52	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	96.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Crosslin Park Road Culvert – Q10

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Crosslin Park Road Culvert – Q25

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,282.64	ft	Headwater Depth/Height	1.52	
Computed Headwater Elevation	1,280.36	ft	Discharge	848.00	cfs
Inlet Control HW Elev.	1,280.36	ft	Tailwater Elevation	1,275.75	ft
Outlet Control HW Elev.	1,279.70	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,274.28	ft	Downstream Invert	1,274.00	ft
Length	55.00	ft	Constructed Slope	0.005091	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	3.01	ft
Slope Type	Steep		Normal Depth	2.81	ft
Flow Regime	Supercritical		Critical Depth	3.39	ft
Velocity Downstream	11.72	ft/s	Critical Slope	0.002978	ft/ft

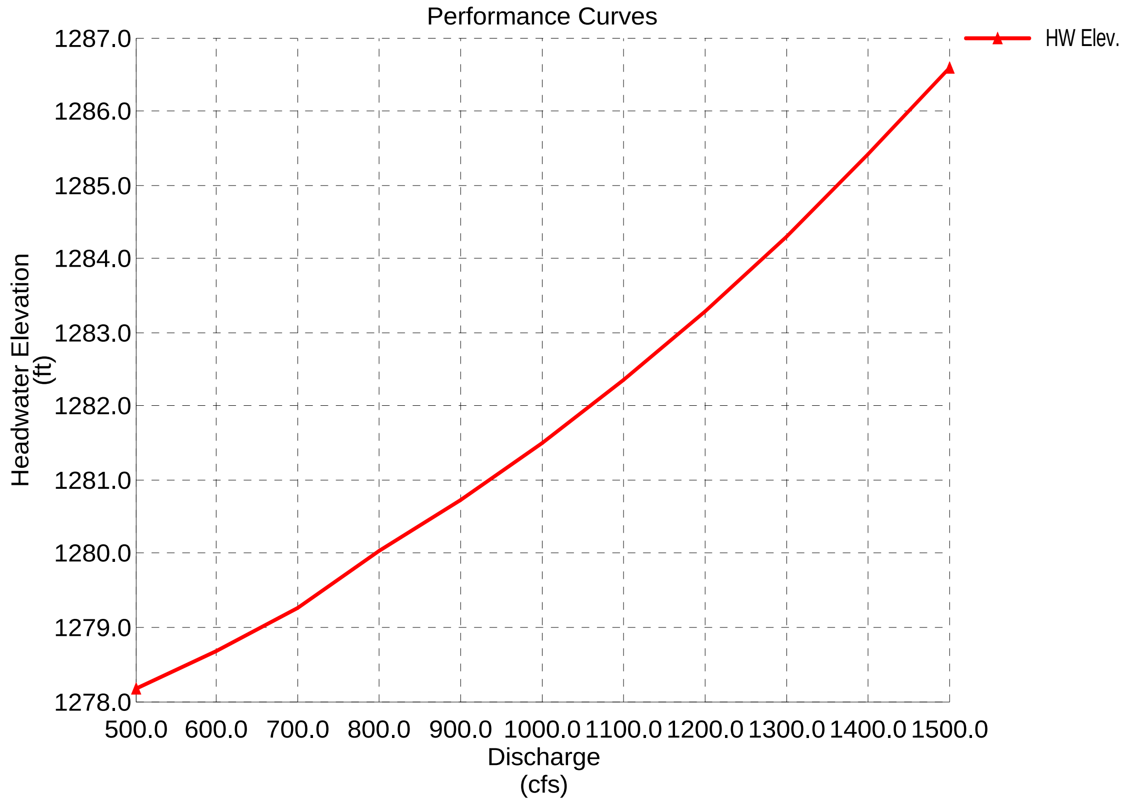
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	1,279.70	ft	Upstream Velocity Head	1.69	ft
Ke	0.20		Entrance Loss	0.34	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,280.36	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	96.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Crosslin Park Road Culvert – Q25

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Crosslin Park Road Culvert – Q50

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,282.64	ft	Headwater Depth/Height	1.94	
Computed Headwater Elevation	1,282.04	ft	Discharge	1,064.00	cfs
Inlet Control HW Elev.	1,282.04	ft	Tailwater Elevation	1,276.26	ft
Outlet Control HW Elev.	1,280.58	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,274.28	ft	Downstream Invert	1,274.00	ft
Length	55.00	ft	Constructed Slope	0.005091	ft/ft

Hydraulic Profile					
Profile	S2		Depth, Downstream	4.00	ft
Slope Type	Steep		Normal Depth	N/A	ft
Flow Regime	Supercritical		Critical Depth	3.94	ft
Velocity Downstream	11.08	ft/s	Critical Slope	0.003056	ft/ft

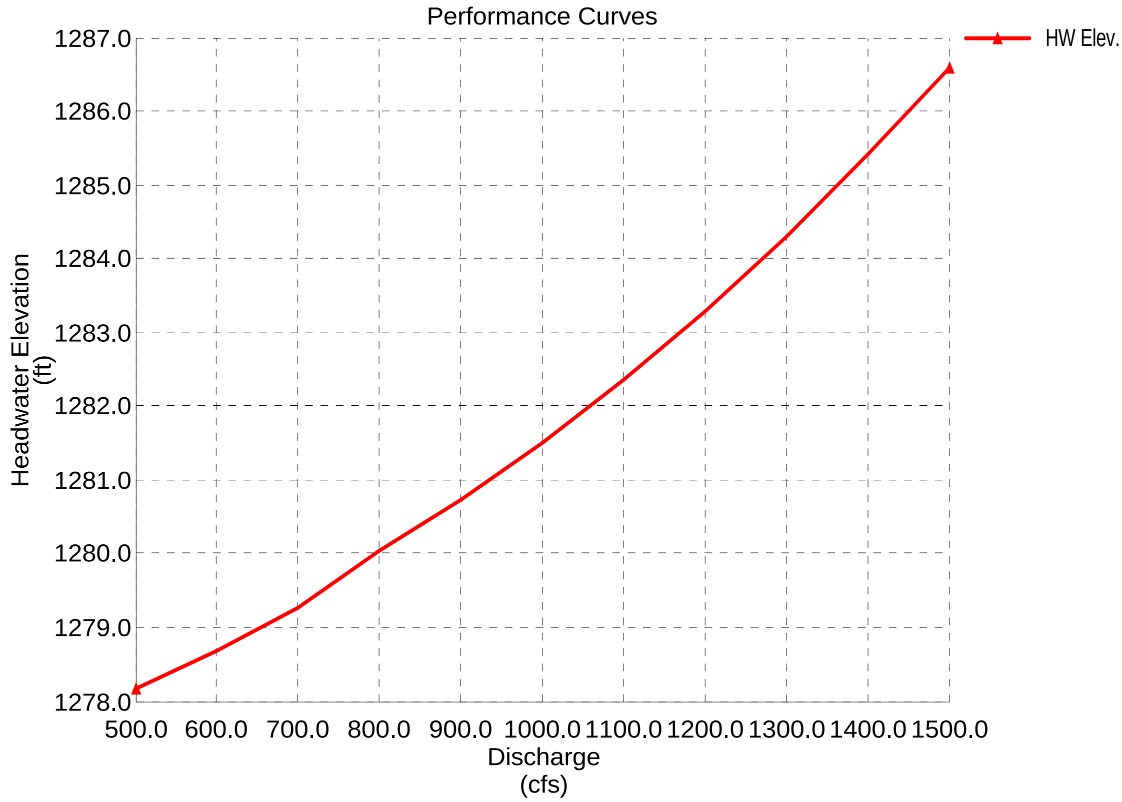
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	1,280.58	ft	Upstream Velocity Head	1.97	ft
Ke	0.20		Entrance Loss	0.39	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,282.04	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	96.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Crosslin Park Road Culvert – Q50

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



Culvert Calculator Report
Crosslin Park Road Culvert – Q100

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	1,282.64	ft	Headwater Depth/Height	2.48	
Computed Headwater Elevation	1,284.20	ft	Discharge	1,290.00	cfs
Inlet Control HW Elev.	1,284.20	ft	Tailwater Elevation	1,276.78	ft
Outlet Control HW Elev.	1,281.81	ft	Control Type	Inlet Control	

Grades					
Upstream Invert	1,274.28	ft	Downstream Invert	1,274.00	ft
Length	55.00	ft	Constructed Slope	0.005091	ft/ft

Hydraulic Profile					
Profile	Pressure Profile		Depth, Downstream	4.00	ft
Slope Type	N/A		Normal Depth	N/A	ft
Flow Regime	N/A		Critical Depth	4.00	ft
Velocity Downstream	13.44	ft/s	Critical Slope	0.008049	ft/ft

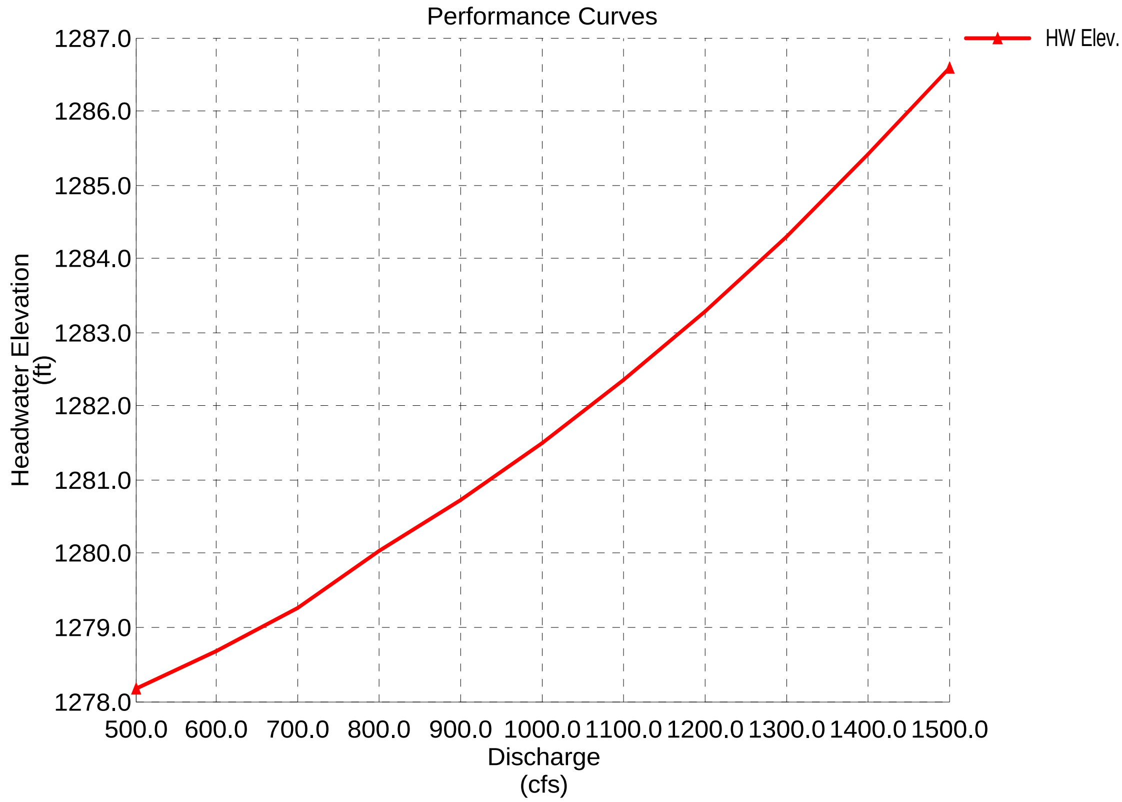
Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	12.00	ft
Section Size	12 x 4 ft		Rise	4.00	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	1,281.81	ft	Upstream Velocity Head	2.81	ft
Ke	0.20		Entrance Loss	0.56	ft

Inlet Control Properties					
Inlet Control HW Elev.	1,284.20	ft	Flow Control	N/A	
Inlet Type	90° headwall w 3/4 inch chamfers		Area Full	96.0	ft2
K	0.51500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	1	
C	0.03750		Equation Form	2	
Y	0.79000				

Performance Curves Report
Crosslin Park Road Culvert – Q100

Range Data:				
	Minimum	Maximum	Increment	
Discharge	500.00	1,500.00	100.00	cfs



VITA

Jason Charles Brinley

Candidate for the Degree of

Master of Science

Thesis: PRACTICAL APPLICATION OF HYDRAULIC TOOLS IN URBAN STORMWATER DESIGN

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Torrance, California, on January 3, 1970, the son of Dwain and Betty Brinley.

Education: Received a Bachelor of Science degree in Civil Engineering from Oklahoma State University in December 1993. Completed the requirements for the Master of Science in Civil Engineering degree at Oklahoma State University, Stillwater, Oklahoma in July, 2010.

Experience: Employed by Envirotech Engineering and Consulting as a Project Engineer from 1995 to 1997. Employed by City of Enid as the Engineering Administrator from 1997 to present.

Professional Memberships: American Society of Civil Engineers, National Society of Professional Engineers

Name: Jason Brinley Date of Degree: July, 2010

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: PRACTICAL APPLICATION OF HYDRAULIC TOOLS IN URBAN STORMWATER DESIGN

Pages in Study: 120 Candidate for the Degree of Master of Science

Major Field: Civil Engineering

Scope and Method of Study: The purpose of this study was to identify appropriate hydraulic modeling technologies to aid in the design of urban stormwater systems for small and medium sized communities. The unnamed channel used in this research is approximately 2,500-feet long and located in the Rolling Acres Subdivision in Enid, Oklahoma. The channel and eight adjacent houses were surveyed to establish elevations. Flowrates were established using HEC-HMS model from a previous study of this basin. Bentley's FlowMaster software was used to determine the channel geometry while Bentley's CulvertMaster software was used to determine the culvert size at three roadway crossings.

Findings and Conclusions: Both FlowMaster and CulvertMaster were found to be very easy to use and provided acceptable results. Because the subdivision was developed outside prior city limits, this channel and adjacent houses were not constructed according to local stormwater regulations or with future, upstream development considered. These nearby, low lying houses as well as existing utilities provided difficult constraints that could not be modified without significant costs. Therefore, the FlowMaster model required both wide channels and concrete-lined channels to prevent house flooding. The CulvertMaster model was able to provide culvert sizes that, while smaller than ideal, resulted in headwaters that would not flood adjacent houses.

ADVISER'S APPROVAL: Avdhesh K. Tyagi