

# TUFLOW Testing and Validation

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## EXECUTIVE SUMMARY

Numerical models are often used as a method to predict changes in the natural environment. Hydrodynamic models, in particular, model the hydraulic behaviour of water bodies. During the design stage of a project, project planners and councils use hydrodynamic computer modelling programs to assess the flood impacts of a proposed development. Developed by WBM Pty Ltd, TUFLOW is a two-dimensional (2D)/ one-dimensional (1D) flood and tide simulation software program. TUFLOW can be classified as a hydrodynamic model, which is specifically orientated towards the simulation of flow patterns for coastal waters, estuaries, rivers and floodplains. Since TUFLOW is being used for major planning decisions it is essential that the modelling program be validated to ensure that the model results are consistent with expected results. TUFLOW has been extensively tested in the past. This project continues the ongoing testing and validation of TUFLOW, which is required as the model is further developed with the inclusion of new and updated features.

This study documents work undertaken testing the ability of the TUFLOW program to model:

- Culvert flow
- Weir flow
- Open channel flow.

For the cases considered as part of this study, the principal outcomes of the study are:

- TUFLOW is representing culvert flow accurately in 1D
- TUFLOW is representing weir flow accurately in 1D and 2D
- TUFLOW is representing open channel flow accurately in 1D and in 2D.

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# 1 GLOSSARY AND TERMINOLOGY

1D - One Dimensional

2D – Two Dimensional

3D - Three Dimensional

SWE - Surface Water Equation

ADI - Alternating Direction Implicit method

FDM - Finite Difference Method

FEM - Finite Element Method

HW - Headwater

TW – Tailwater

GIS – Geographic Information System, computer program

TUFLOW – Two-dimensional Unsteady FLOW, 2D/1D hydrodynamic modelling program

ESTRY – 1D hydrodynamic model utilised by TUFLOW to represent 1D flow

1D network – Terms used to describe the region represented by a 1D model

2D domain – Terms used to identify the region represented by a 2D model

Grid mesh – term describing the grid elements and nodes created when developing a 2D model

Flow Length (weir flow) – The distance across the crest of the weir. For a weir orientated perpendicular to the channel bank, the flow length is equal to the width of the channel.

## 2 STATEMENT OF ORIGINALITY

*“The material presented in this report contains all of my own work, and contains no material previously published or written by another person except where due acknowledgment is made in the report itself”.*

Christopher D Huxley

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## 3 INTRODUCTION

### 3.1 General

The impact of changes to the existing environment is often extremely difficult to predict. This is due to the, generally, large number of environmental variables contributing to the equilibrium of an environmental system. A significant amount of variability in water quality and other environmental water systems is controlled by the basic mechanism of water flow (McCutcheon, p2, 1989). Knowledge of the pathway volume and velocity of water (hydraulic behaviour) is needed to undertake any fundamental study of water quality or other water processes, including modelling investigations (Martin and McCutcheon, p1, 1999). Numerical modelling is often used as a predictive method to replicate environmental changes in an effort to assess the impact of a particular action. A numerical model used to represent the hydraulic behaviour of a water body is called a hydraulic model (Barton, p1, 2001).

The impact of such things as urban developments on fluid hydraulics and water quality can be predicted using 1-Dimensional (1D), 2-Dimensional (2D) and 3-Dimensional (3D) numerical models. The results produced by these numerical models often critically influence the decision-making processes of major and minor works.

The ability of these models to replicate natural systems is dependent on three main factors. The quality of the physical data used to establish the model, the ability of the modeller to develop a model that is representative of the system, and the numerical capability of the actual model itself (Barton, p2, 2001).

TUFLOW is a 2D/1D hydraulic modelling program developed by WBM Pty Ltd (WBM Pty Ltd, 2003). The program utilises a 1D network in conjunction with a 2D domain as a method to closely replicate the workings of natural systems. To achieve this numerically, TUFLOW uses the Stelling scheme (Syme , p38 1991).

This study aims to test TUFLOW in a structured, concise manner. TUFLOW has been extensively tested in the past. This project, however, continues the ongoing testing and validation of TUFLOW, which is required as the model is further developed with the inclusion of new and updated features. To do this a variety of test cases will be developed; the modelled results of these test cases will be checked against conventional hydraulic theories, through independent calculations. This methodology will be used to validate the results which TUFLOW produces. The validation of the model will assist in testing and proving the theoretical accuracy of the model whilst identifying any areas of the model that could possibly be further developed or improved.

## 3.2 Study Objectives

The Study objectives are as follows;

1. To provide a summary of engineering theory used to describe the hydraulic processes of culvert, weir and open channel flow.
2. To assess the theoretical accuracy of TUFLOW when modelling culvert, weir and open channel flow.
3. To identify any areas of the TUFLOW program that could possibly be further developed or improved in relation to culvert, weir and open channel flow.

### 3.3 Background/ Literature Review

Hydraulic theory has been researched and developed for hundreds of years. Developments in the field have meant that accurate estimations of hydraulic processes can today be described by a combination of experimental results and theory.

During the design of various works, hydraulic testing based on these theories and results, is often conducted. Historically, hydraulic testing was required for the sizing of structures. Today, however, hydraulic testing is also undertaken to assess the effects of various works on surrounding environments. For example, during the sizing of a culvert, the flow requirements of the culvert and the backwater effects of the culvert may be assessed. This design methodology ensures the holistic design of hydraulic structures. The structure is designed to fulfil its required role, whilst minimising its future effects on the surrounding environment.

Computationally, hydraulic analyses can be extremely complex and laborious. Over the past 20 years a variety of hydrodynamic computer modelling programs have been developed to assist project planners and councils during hydraulic analyses.

#### 3.3.1 Hydrodynamic Modelling

Quantifying the hydraulic behaviour of natural and artificial watercourses is often essential for (a) understanding hydraulic, biological and other processes, (b) predicting impacts of works and natural events, and (c) environmental management. Numerical hydrodynamic models are presently the most efficient, versatile and widely used tools for these purposes (Syme, p2, 2001).

For modelling purposes, the Shallow Water Equations (SWE) are used to calculate an approximate description of hydrodynamic processes. The SWE are used to describe water flows, which is characterised by abrupt changes in water depth and flow rates (Zoppou and Roberts, p1, 2000). The SWE represent the partial differential equations of mass and momentum conservation for unsteady free surface flow of long waves. These equations can be presented in 1D, 2D, or 3D forms.

The 2D SWE in Cartesian form is given by

$$\frac{\partial U}{\partial t} + \frac{\partial G}{\partial x} + \frac{\partial H}{\partial y} = 0$$

The vectors  $U, G$  and  $H$  can be expressed in terms of the primary variables

$$U = \begin{bmatrix} h \\ uh \\ vh \end{bmatrix}, G = \begin{bmatrix} uh \\ u^2h + \frac{gh^2}{2} \\ uvh \end{bmatrix}, H = \begin{bmatrix} vh \\ uvh \\ v^2h + \frac{gh^2}{2} \end{bmatrix}$$

in which  $g$  is the acceleration due to gravity,  $h$  is the water depth and  $u$  and  $v$  are the flow velocity in the  $x$  and  $y$  direction.

It has been found that the hyperbolic nature of SWEs makes them difficult to solve. To overcome this, various schemes have been developed to provide approximations for the SWEs. The majority of 2-D schemes that have found widespread practical application use the alternating direction implicit (ADI) finite difference method (FDM). Other schemes have utilised the finite element method (FEM), the fractional step approach, explicit FDM and fully implicit FDM (Syme, p16, 1991). On a performance basis it is difficult to differentiate between the more suitable schemes for hydrodynamic modelling given the limited information on real time application (Syme, p11, 1991). Table 1 outlines the solution schemes used by various well-known modelling programs.

**Table 1: Model Solution Schemes**

Model Name	Solution Technique	Solution Scheme	Reference
FESWMS	Finite Element	2D Implicit	FHA
Mike21	Finite Difference	2D Implicit (ADI)	DHI (1998)
RMA2	Finite Element	2D Implicit	King (1998)
TUFLOW	Finite Difference	2D Implicit (ADI)	Syme (1991)
Delft- FLS	Finite Difference	2D Implicit (ADI)	Mynett (1999)
TELEMAC	Finite Element	2D Implicit (FDM)	Leopardi <i>et al.</i> (2002)
MIKE11	Finite Difference	1D Implicit	DHI (1999)
ESTRY	Finite Difference	1D Explicit	WBM (1996)

### 3.3.1.1 1D Modelling

1D modelling is generally used for the modelling of rivers and estuaries. The flows in these circumstances are generally essentially channelled or one directional in nature. The water velocities are normally calculated as the cross sectional average in the direction of flow (Syme, p 17, 1991)

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The 1D solution can be calculated using implicit or the explicit schemes. Explicit schemes are more computationally efficient per timestep than implicit schemes, however become unstable for Courant numbers greater than 1. Implicit schemes on the other hand are unconditionally stable (Hardy *et al.*, p124, 1999). When using the explicit scheme it is often necessary to use a small timestep to achieve a Courant number less than one.

1D models do not require excessive computation power therefore many 1D-modelling programs, such as ESTRY, operate using an explicit scheme. This is the case because, with today's computer technology, the programs are generally not limited by computational power, as is the case for 2D models. When reviewing other modelling programs it can be seen that some programs such as Mike 11 use an implicit scheme. In comparison to explicit schemes, implicit schemes have the benefit of greater model stability for larger timesteps. Even though an explicit scheme uses a smaller time step both solution types can provide comparable results (NCSA, p5, 1998).

### 3.3.1.2 2D Modelling

2D modelling is widely used for flooding rivers and tidal estuaries (Syme, p16, 1991). 2D models can represent water flow in horizontal plane that are not channelled or one directional in nature. The water velocities are usually calculated as the average velocity over the depth of the water column. 2D models are more complex than 1D models and require more computation effort.

2D modelling requires significantly more computational power than 1D modelling. The 2D solution can be found using either the implicit or the explicit scheme. It is, however, generally more favourable to use an implicit scheme in 2D to reduce computational effort, by using a larger time step. This is favoured because of the limitations imposed by available computational power, especially when modelling larger systems.

Generally it is considered that for floodplain management investigations 2D modelling programs are more representative of actual flows than 1D modelling programs. Hence 2D models are generally more accurate under these circumstances. It has, however, become apparent that there are some limitations to using the 2D software. It has been found that fixed grid systems have limitations at simulating small drainage elements (eg. narrow creeks, open drains, pipes, culverts etc). If these elements are in the order of a few grid cells wide, then representation of the elements in 2D is somewhat coarse and an over-estimated or under-estimated flow capacity can be produced (Benham and Rogencamp, p1, 2003). For scenarios where a 2D model cannot accurately replicate a study area due to extensive channel networks, historically, 1D models have been used instead.

### 3.3.1.3 TUFLOW

TUFLOW is a 2D/1D modelling program developed by WBM Pty Ltd. The 2D component TUFLOW uses the Stelling scheme to implicitly solve the 2D SWE (Syme, p 17, 1991). TUFLOW is specifically orientated towards establishing flow patterns in coastal waters, estuaries, rivers, floodplains and urban areas where the flow patterns are essentially 2D in nature and cannot or would be awkward to represent using a 1D network model (WBM, p17, 2004).

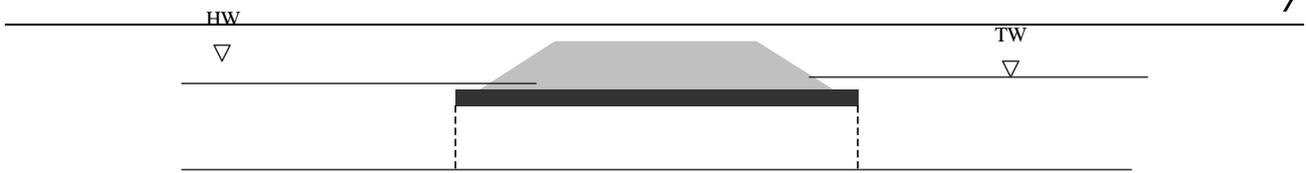
A powerful feature of TUFLOW is its ability to dynamically link to the 1D network (quasi-2D) hydrodynamic program ESTRY. ESTRY is used to model flows that are predominantly 1D in nature, such as stream and river flow. ESTRY uses the 1D St Venant Equations and standard structure equations to represent fluid flow. In practice, the user sets up a model as a combination of 1D network domains linked to 2D domains. This means the 2D and 1D domains are linked to form one model. This feature of TUFLOW overcomes the problem that 2D models often face when modelling small channels (Rogencamp & Syme, p3, 2003). This makes TUFLOW a very versatile model, suited to most model scenarios.

### 3.3.2 Culvert Flow

The flow through a culvert is complex and is the result of various parameters. Ideally there are two types of culvert flow control that occur, inlet flow control and outlet flow control. Inlet control occurs when the culvert flow is restricted by the discharge that can pass the inlet for a given headwater (CPAA, p31, 1991). Headwater (HW) is defined as the water level above the invert of the culvert inlet. Outlet control, in contrast, occurs when the culvert flow is restricted by the discharge that can pass the outlet for a given tailwater (CPAA, p31, 1991). Tailwater (TW) is defined as the water level above the invert of the culvert outlet. Figure 1 and Figure 2 illustrate basic examples of inlet control and outlet control.



**Figure 1: Culvert Flow Control: Inlet control**



**Figure 2: Culvert Flow Control: Outlet control**

Australian practice for culvert design is based on the hydraulic design manual, “Hydraulic of Precast Conduits” written by CPAA (1991). As stated in the manual,

*“it is rarely immediately obvious which pattern of flow a culvert is going to adopt, it is therefore necessary to investigate the consequences of both inlet and outlet control flow.”*  
(CPAA, p31, 1991)

Assessing whether a culvert is flowing under inlet or outlet control requires the use of culvert nomographs produced by the Concrete Pipe Association of Australasia. The nomographs have been produced as a result of various empirical studies and provide an estimation of headwater depth during culvert analysis.

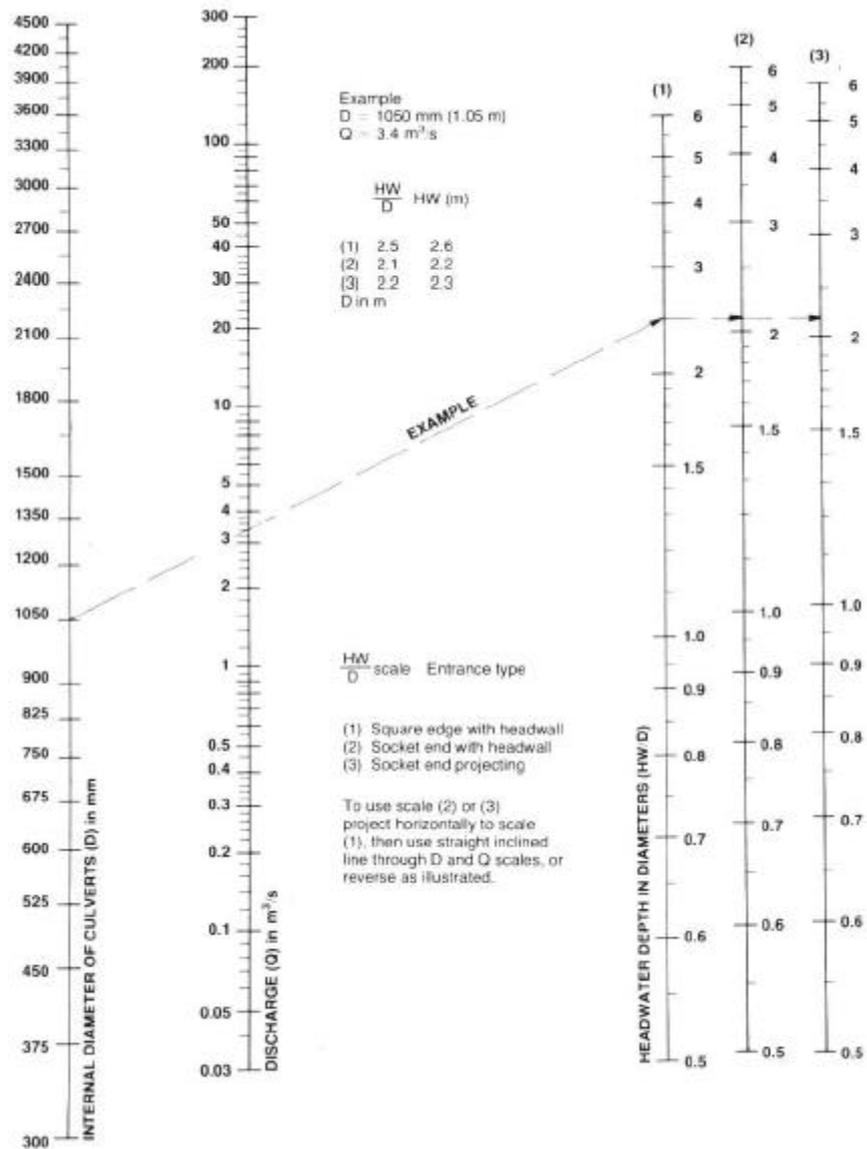


Figure 3: Headwater Depth for Concrete Pipe Culverts with Inlet Control

(CPAA, p34, 1991)

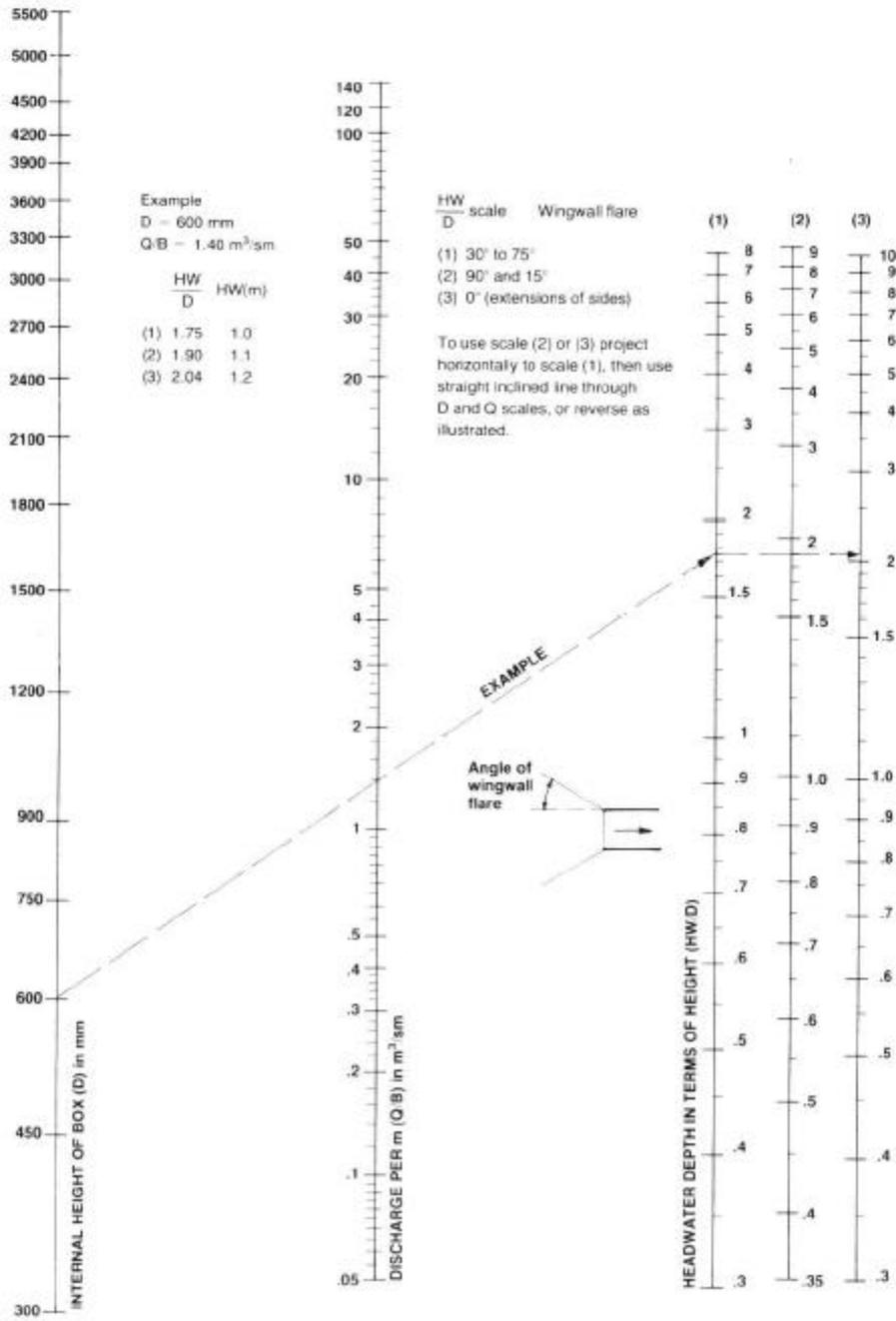


Figure 4: Headwater Depth for Concrete Box Culverts with Inlet Control

(CPAA, p35, 1991)

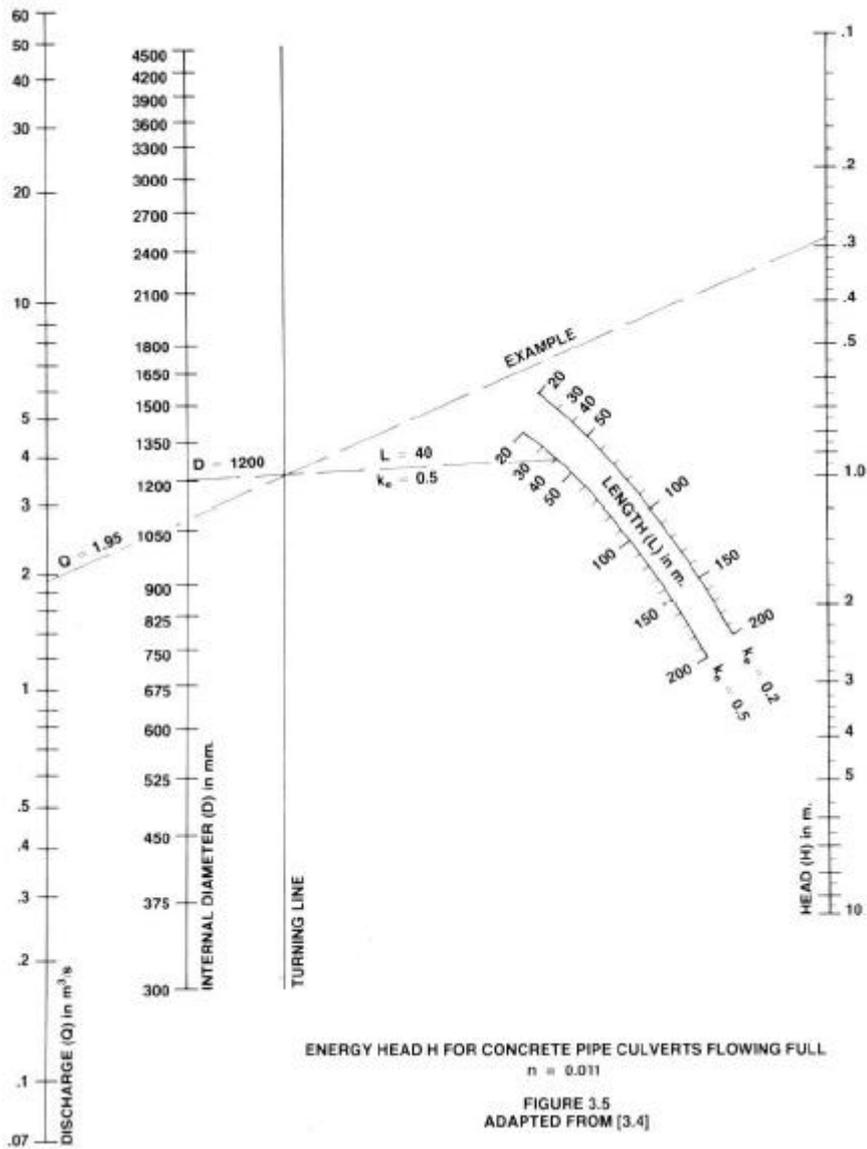


Figure 5: Energy Head H for Concrete Pipe Culverts Flowing Full

(CPAA, p36, 1991)

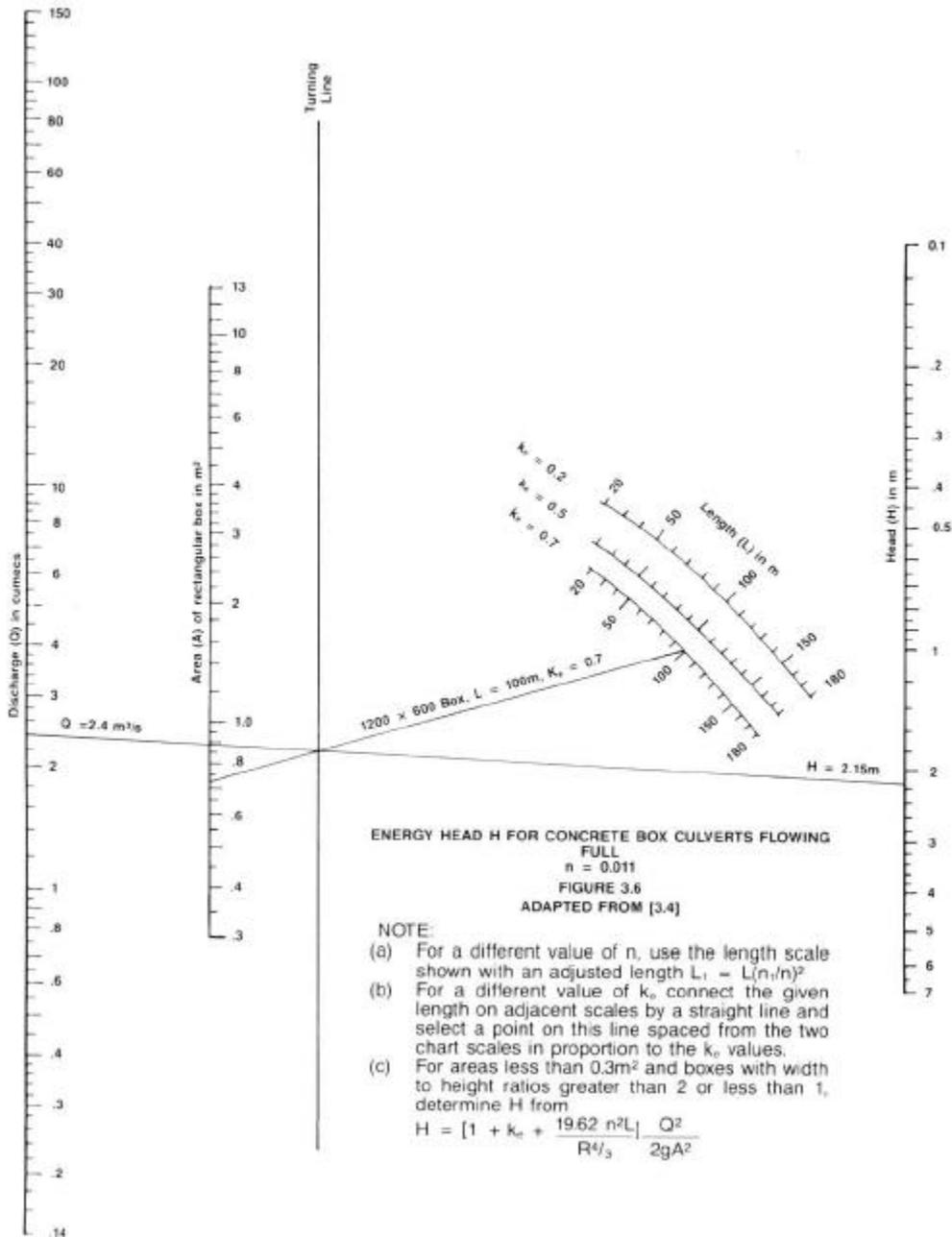


Figure 6: Energy Head H for Concrete Box Culverts Flowing Full

(CPAA, p36, 1991)

Using the nomographs, headwater depths are calculated for the design culvert assuming inlet and outlet control. For inlet control the headwater depth is taken straight from the nomograph. For outlet control, however, the energy head (H) is calculated using the outlet control nomograph, then Equation 1 is used to calculate the headwater depth.

$$\text{Equation 1: } H = HW + Ls_0 - TW \quad (\text{CPAA, p32, 1991})$$

Where: H = energy head (m)  
 HW = headwater depth (m)  
 L = culvert length (m)  
 s<sub>0</sub> = culvert gradient (m/m)  
 TW = tailwater depth (m)

For outlet control it has been found that if,

$$\text{Equation 2: } TW > \frac{d_c + D}{2} \quad (\text{CPAA, p37, 1991})$$

Where: d<sub>c</sub> = critical depth (m)  
 TW = tailwater depth (m)  
 D = culvert diameter (m)

It has been found that a good approximation of headwater level can be found by using the culvert nomographs, substituting  $\frac{d_c + D}{2}$  for TW (CPAA, p37, 1991).

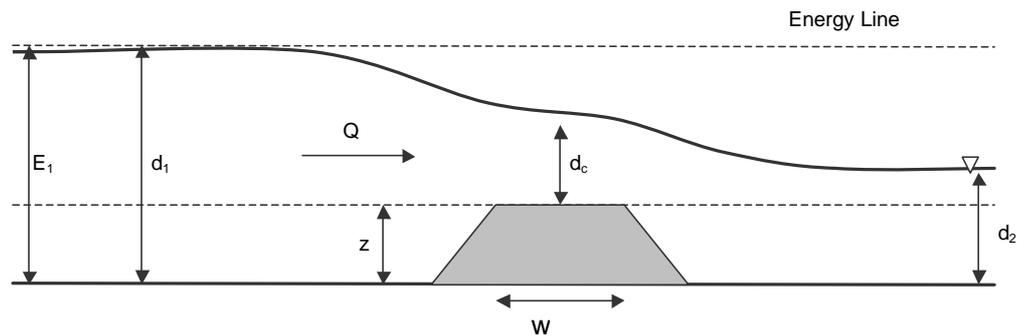
Taking all of these practices into consideration, theory states, whichever control situation produces the greater headwater depth is accepted as the governing culvert control (CPAA, p37, 1991; Chanson, p382, 1999).

Throughout the documentation presented by CPAA there are no statements outlining the assumptions made during the creation of the culvert nomographs. As such the limitations of the methodology produced cannot be deduced. Headwater calculation comparisons using a computer model created by Vienot (2004), based on empirical formulas produced by Chanson (1999), showed however, that the accuracy of the culvert nomograph method was within approximately 5%.

Since it is Australian engineering practice to use the nomograph method, it will be used for the culvert calculations during the study.

### 3.3.3 Weir Flow

Although there are many types of weir, in the context of this project, the only weirs being researched are broad crested weirs. During flood events it is often the case that structures, such as road and rail embankments or even levy banks, act as broad crested weirs. Figure 7 shows a typical schematic representation of a broad crested weir.



**Figure 7: Weir flow**

Numerous laboratory-based studies have identified that there are two basic flow regimes that determine the headwater upstream from a broad crested weir. These regimes are known as non-submerged flow and submerged flow (Hager & Schwalt, p20, 1994). Non-submerged flow is defined to occur when the weir inundation ratio is less than 0.75. Mathematically this is described in Equation 3. Submerged flow therefore occurs for weir flow with an inundation ratio greater than 0.75.

#### Non-submerged Flow

Non-submerged flow is defined to occur when the weir inundation ratio is less than 0.75. Mathematically this is described in Equation 3.

$$\text{Equation 3: } \frac{(d_2 - z)}{(d_1 - z)} < 0.75$$

Where:  $d_2$  = downstream water depth (m)  
 $d_1$  = upstream water depth (m)  
 $z$  = weir height (m)

For non-submerged flow it can be assumed that critical flow conditions occur on the crest of the broad crested weir (Finnemore and Franzini, p534, 2002). Theoretically, flow at critical depth is

dominated by critical flow. This assumption is based on the theory that for non-submerged flow, free overfall flow conditions occur and headwater depth is dependent solely on upstream conditions.

When calculating the upstream depth during non-submerged flow, minimum specific energy calculations are used.

By definition the depth corresponding to the minimum specific energy for a given flow is called the specific depth, and for a rectangular channel is given by Equation 4.

$$\text{Equation 4:} \quad d_c = \left( \frac{q^2}{g} \right)^{\frac{1}{3}}$$

Where:

- $d_c$  = critical depth (m)
- $q$  = discharge per unit width ( $\text{m}^3/\text{s}/\text{m}$ )
- $g$  = gravity ( $\text{m}/\text{s}^2$ )

Using the calculated critical depth, the minimum specific energy ( $E_{\min}$ ) for the flow can be calculated using Equation 5.

$$\text{Equation 5:} \quad E_{\min} = \frac{3}{2} d_c$$

Where:

- $E_{\min}$  = minimum specific energy (m)
- $d_c$  = critical depth (m)

Using the Minimum Specific Energy, the upstream specific energy can be calculated using Equation 6.

$$\text{Equation 6:} \quad E_1 = z + E_{\min}$$

Where

- $E_1$  = upstream specific energy (m)
- $E_{\min}$  = minimum specific energy (m)
- $z$  = height of weir (m)

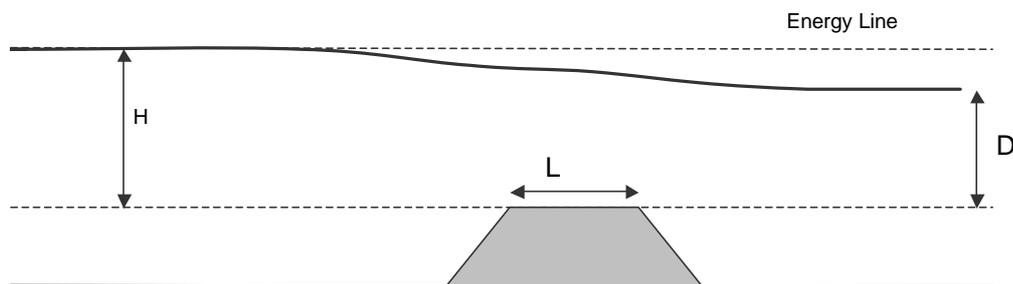
Using the calculated upstream specific energy the upstream headwater depth is found using the iterative approach given by Equation 7.

**Equation 7:** 
$$d_1 = E_1 - \frac{q^2}{2gd_1^2}$$

Where  $d_1$  = upstream depth (m)  
 $E_1$  = upstream specific energy (m)  
 $q$  = discharge per unit width ( $\text{m}^3/\text{s}/\text{m}$ )  
 $g$  = gravity ( $\text{m}/\text{s}^2$ )

### Submerged Flow

Submerged flow is defined to occur when the weir inundation ratio is greater than 0.75. During submerged flow, because free overfall flow does not occur, it cannot be assumed that critical flow conditions occur over the broad crested weir. Therefore the headwater depth is dependent on both upstream and downstream conditions. Empirical studies conducted by Bradley (1978), published by the US Department of Transport, have produced calibrated graphs that can be used for discharge coefficient estimation. Figure 9 to Figure 11 show the discharge coefficient graphs and Figure 8 defines the variables used by Bradley (1978).



**Figure 8: Weir Flow (Bradley, 1978)**

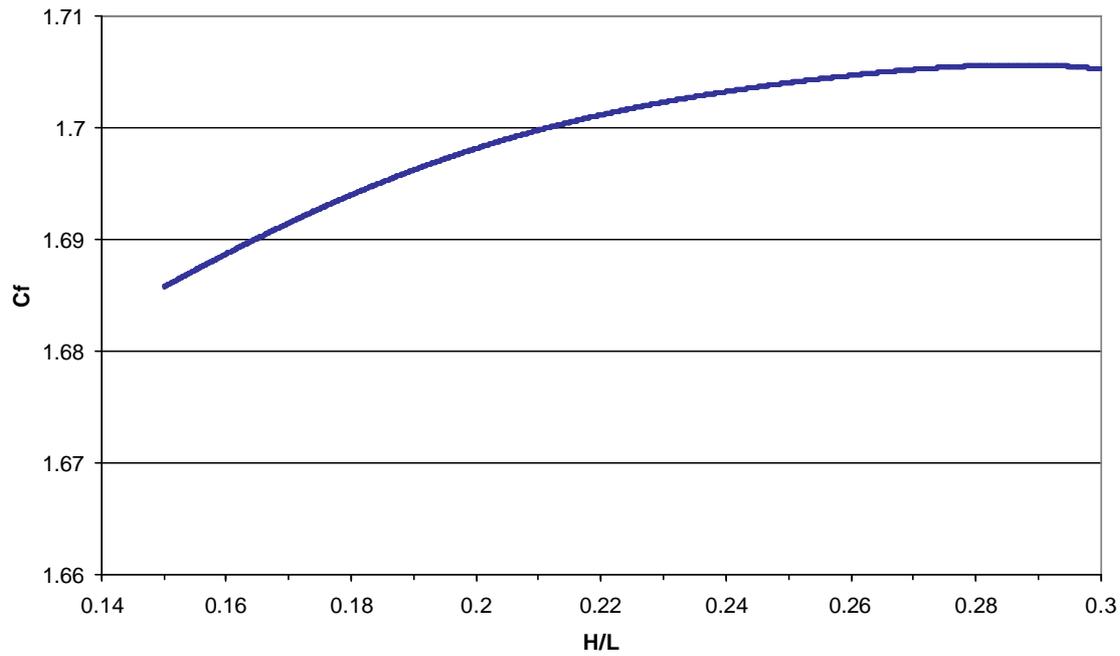


Figure 9: Discharge Coefficient  $C_f$  ( $H/L > 0.15$ )

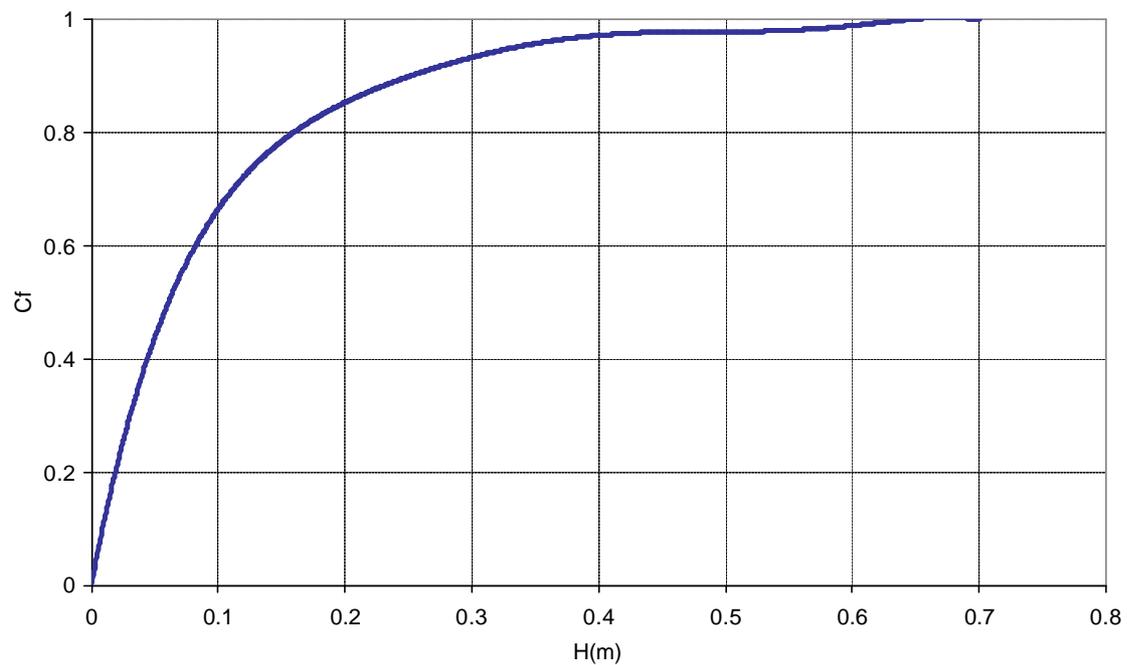
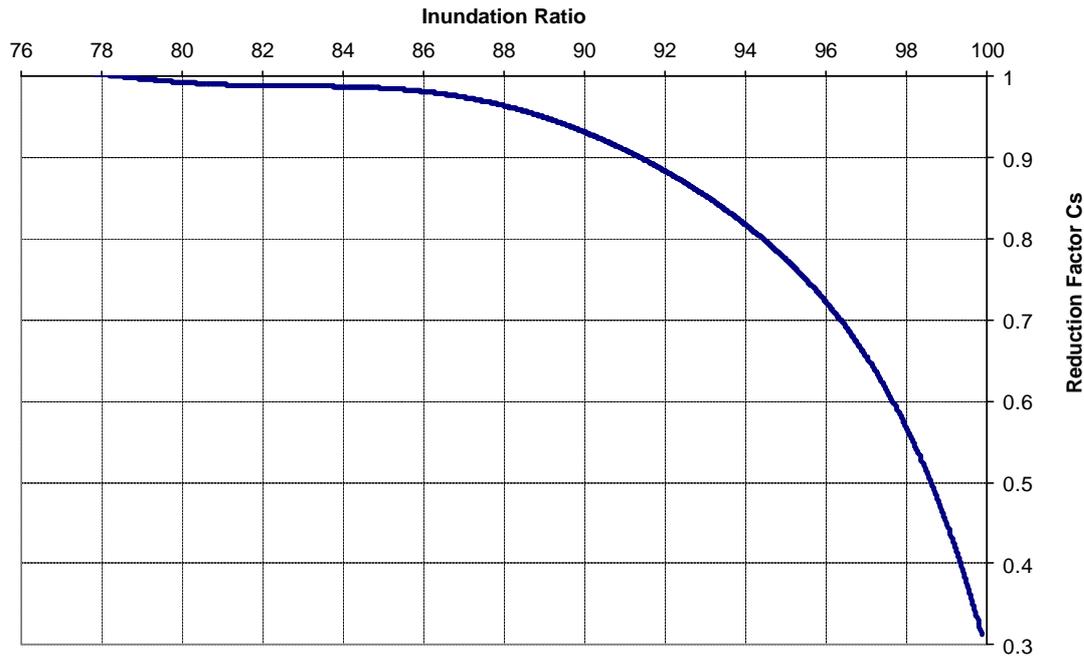


Figure 10: Discharge coefficient  $C_f$  ( $H/L < 0.15$ )



**Figure 11: Discharge coefficient  $C_s$  ( $D/H > 0.70$ )**

As Defined by Bradley (1978), Equation 8 is used to calculate the upstream water depth  $H$ .

**Equation 8:** 
$$H = \left( \frac{Q}{C_s C_f W} \right)^{\frac{2}{3}}$$

Where:  $H$  = Upstream water depth (m)

$Q$  = Upstream discharge ( $\text{m}^3\text{s}^{-1}$ )

$C_s$  = Discharge coefficient

$C_f$  = Discharge coefficient

$W$  = Weir Width (m)

Throughout the documentation presented by the Bradley (1978) there are no comments identifying the assumptions or test methodology made during the creation of the submerged weir graphs. As such the limitations of the methodology cannot be estimated. Other contemporary sources of literature, however, such as “Waterway Design – A Guide to the Hydraulic design of Bridges, Culvert and

Floodways” (AUSTROADS, p17 1994), endorse the theory produced by Bradley. AUSTROADS represents the Roads and Traffic Authorities for New South Wales, Queensland, South Australia, Australia Capital Territory and Victoria. As both the Australian and American Road and Traffic Authorities endorse the theory presented by Bradely (1978) it will also be used during this study.

### 3.3.4 Open Channel Flow

Open channel flow is characterised by a waterway, canal or conduit in which a fluid flows with a free surface, subjected to only by atmospheric pressure (Chanson, p6, 1999; Henderson, p105, 1966).

Open channel Flow can be classified or described in various ways. One classification method is based on the description of the flow, the flow is defined as being either uniform or non uniform.

Steady or uniform flow is defined to occur when the depth of flow within a channel is constant for a given time interval. Unsteady or non-uniform flow, in contrast, is characteristic of flow that changes in depth during a specified time interval.

There have been many studies based on uniform flow and as a result many uniform flow formulas are available. The Kuttat, Bazin, Powell, Chezy and Manning formulas are examples of these. Overall, the empirical formulas are based on the momentum equation, which states the exact balance between the shear forces and the gravity component along a streamline (Chanson, p79, 1999).

Of the uniform flow formulas, the Manning’s formula is the most widely used because of its simplicity and accuracy (Finnemore & Franzini, p412, 2002). The Manning’s formula is shown as

$$\text{Equation 9: } Q = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} A$$

Where: n = Manning’s n roughness coefficient

R = Hydraulic Radius (m)

S = Bed Slope (m/m)

A = Cross sectional area

Analysis of the derivation of the Manning’s formula has shown that as an empirical equation, it does not provide exact solutions, however, it can be used as an accurate estimation of normal depth. During the derivation of the formula, the exponent of the hydraulic radius R was based on experimental data taken from artificial channels. For different shapes and roughness, the average value of the exponent was found to vary from 0.6499 to 0.8395. Using these values the approximate value of 2/3 was adopted for the exponent (Chow, p99, 1959).

Authors such as Finnemore & Franzini (2002), Chow (1959) and Hamill (1995) recommend the use of Manning's formula instead other uniform flow formulas. Manning's formula was used for the open channel calculations throughout this study.

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## 4 METHODOLOGY

### 4.1 General

When discussing environmental models, it is recognised that the effectiveness of numerical models, such as TUFLOW, are dependent on three main factors. These are

- the quality of physical data used during model development,
- the competence of the modeller to produce a model that is representative of the natural system and
- the numerical capability of the model to replicate certain aspects of a given system (Barton, p1, 2001)

To minimise the possibility of physical data affecting test results, hypothetical models were created and used for all the testing. The use of hydraulic theory was used instead of real data to estimate expected flow conditions for the given model case during the testing. This testing structure was favoured for a variety of reasons. These were,

1. Elimination of possible inaccuracies in physical data.
2. The possibility for the testing of model features against an infinite number of test conditions. During the initial scoping of the project it was realised that relying on actual physical data may limit the possibility to test all flow regimes and variables.

The testing was structured so that the “test model” results were compared against independent calculations based on established engineering principles. The comparison results will be calculated in absolute terms and relative terms. The absolute variation shows the depth variation of the results in metres, the relative variation, meanwhile, can be defined as the calculated depth variation as a percent difference. The use of independent calculations based on engineering principles ensures the numerical capabilities of the solution scheme utilized by the TUFLOW program is solely tested in an organised manner.

Whilst planning the model testing, establishing the testing structure for each hydraulic feature required the consideration of two main factors. These factors were related to the modelling structure of TUFLOW, and the hydraulic principles describing the fluid flow.

Initially the modelling structure utilised by TUFLOW was considered. This ensured that each regime used by the particular feature was tested. During culvert flow, for example, the modelling structure of

TUFLOW splits the culvert flow into 12 separate flow regimes (A-J) depending on variables such as bed slope, inlet submergence and outlet submergence.

Secondly the engineering theory used to represent the hydraulic flow was analysed. This ensured that each flow regime and variable with relevance to the engineering theory for the hydraulic feature was also tested. For example, open channel flow testing was conducted for supercritical, critical and subcritical flow regimes. During each regime test, the variables of channel width, bed slope and Manning's roughness coefficient were also tested.

This testing structure has been used such that the testing of TUFLOW has been carried out in a concise, structured format.

## 4.2 Model Structure

Developing a TUFLOW model, representative of a given system, requires a variety of different data sets. These data sets define the transport of fluid within the model and the fluid volume entering and leaving the model. Factors affecting fluid transport are obtained in the form of GIS layers defining digital terrain models, hydraulic structure geometry and Manning's roughness coefficients. Fluid volumes entering and leaving the model extents are described as boundary conditions.

### 4.2.1 Fluid Transport

#### 4.2.1.1 Digital Terrain Model

A Digital Terrain Model (DTM) is a topographic map used to define all flow paths and storage areas within the 2D domain. It is recommended that the vertical accuracy of larger models be within 0.2m, whilst for fine scale urban models 0.1m is recommended (WBM Oceanics Australia, p38, 2004).

#### 4.2.1.2 Hydraulic Structures

The geometry of all hydraulic structures must be defined. This requires accurate cross section data which can often be obtained from structural plans or may need to be obtained by onsite observation and surveying.

To minimise the possibility of erroneous results during the project caused by DTM and cross section inaccuracies, hypothetical models have been used for the testing.

#### 4.2.1.3 Manning's Roughness Coefficient

Bed resistance values are defined for areas defined within the bounds of the DTM by an assigned Manning's  $n$  value.

During the testing, hypothetical models will be used, such that the full range of Manning's n values can be tested, whilst reducing possible inaccuracies of using "real" data.

#### **4.2.2 Boundary Conditions**

Boundary conditions define the amount of fluid entering and exiting the model.

All upstream boundary conditions were assigned a QT flag, defining the boundary condition to be set in a flow vs time format. All downstream boundary conditions were assigned a HT flag, defining the format of the boundary condition to be, head vs time.

### **4.3 Model Run**

Conventionally there are two types of model run which can be modelled, static and dynamic. A static run is characterised by fixed boundary conditions, simulating uniform flow conditions. In contrast, a dynamic run commonly has boundary conditions which change over time, characteristic of non-uniform flow.

During the testing, static runs were used to test the accuracy of the modelling program against independent calculations. Dynamic runs, on the other hand, were used to test the transition of flow between regimes. For example, culvert flow dynamic runs were used to test the transition between flow governed by inlet and outlet control.

## 5 INDEPENDENT TESTING

### 5.1 Culvert Analysis

A culvert is a covered channel designed to pass water through an embankment, such as under a highway, railroad or through a dam (Chanson, p365, 1999). A culvert consists of three sections, the inlet, the throat and the outlet. In cross section a culvert may be circular (pipe culvert) or rectangular (box culvert) in shape. In practice a culvert is designed to pass a specific flow rate with an associated natural flood level. Its hydraulic performances are the design discharge, the upstream depth and the maximum acceptable head loss. The hydraulic design of a culvert is basically the selection of an optimum compromise between discharge capacity and head loss (Chanson, p369, 1999).

Culvert flow is one directional in nature. In order to obtain a better representation of the hydraulic scenario and for computational efficiency TUFLOW utilises the 1D-modelling program, ESTRY, for culvert analysis. ESTRY uses various culvert regimes, based on flow characteristics, to represent culvert flow. Figure 12 and Figure 13 illustrate the culvert classifications used by ESTRY.

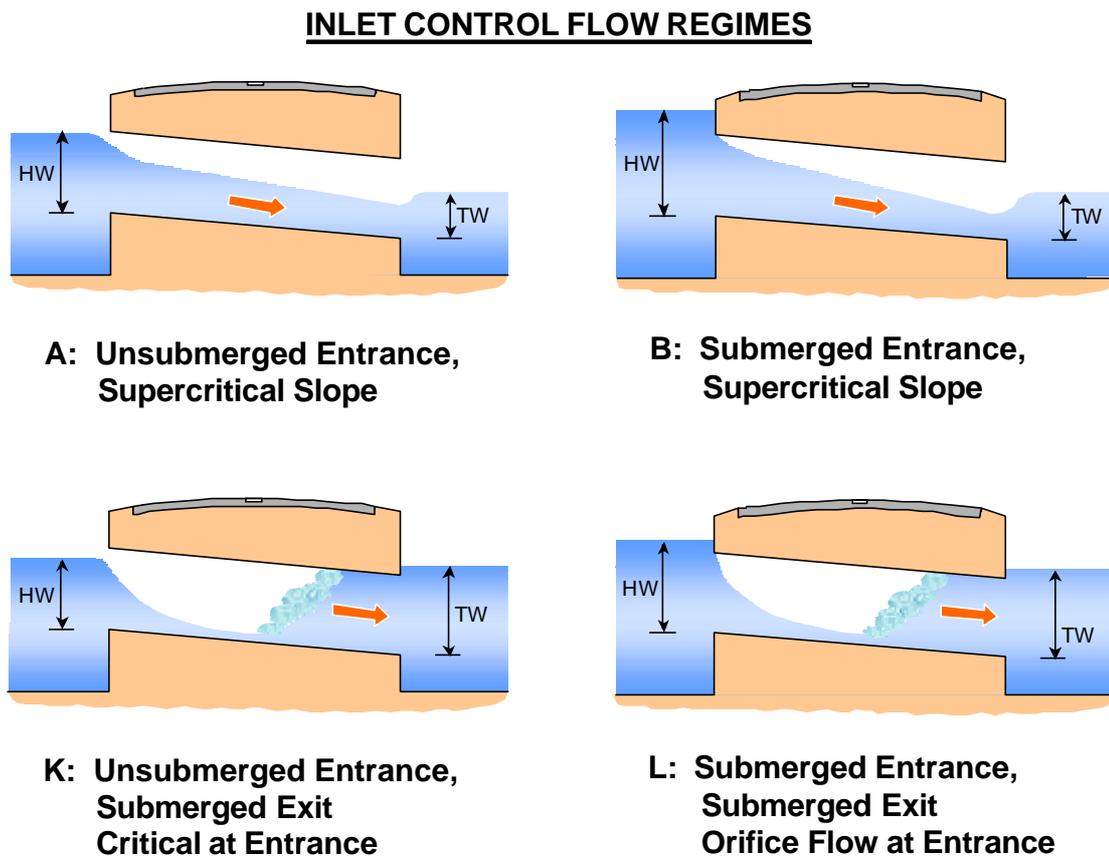
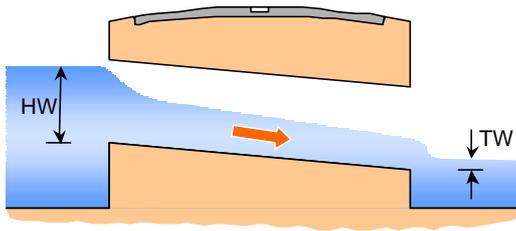


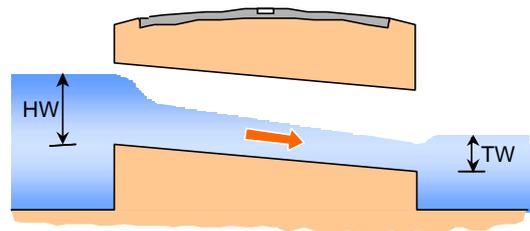
Figure 12: ID inlet Control Culvert Flow Regimes

(WBM, p44-45, 2004)

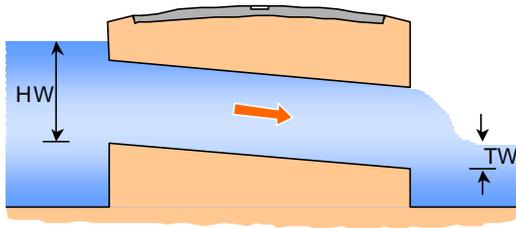
### OUTLET CONTROL FLOW REGIMES



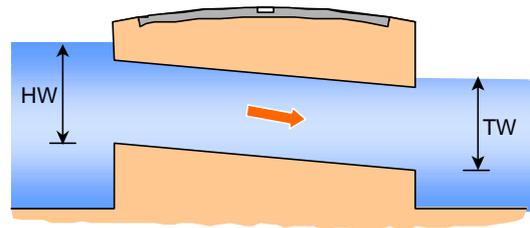
**C: Unsubmerged Entrance, Critical Exit**



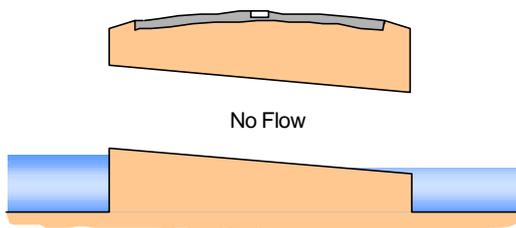
**D: Unsubmerged Entrance, Subcritical Exit**



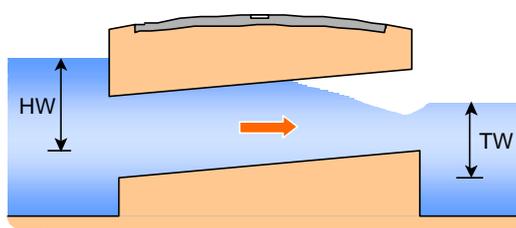
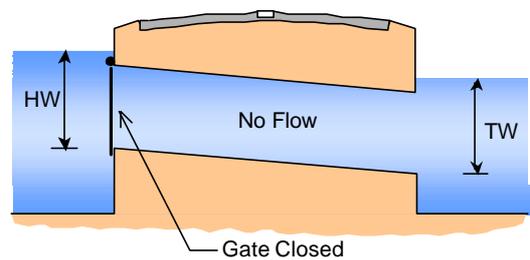
**E: Submerged Entrance, Unsubmerged Exit**



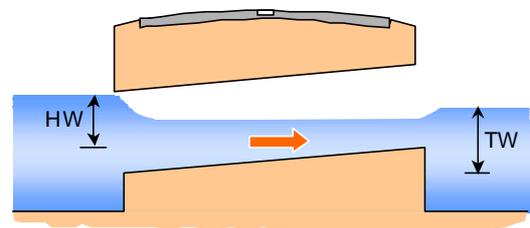
**F: Submerged Entrance, Submerged Exit**



**G: No Flow  
Dry or Flap-Gate Closed**



**H: Adverse Slope,  
Submerged Entrance**



**J: Adverse Slope,  
Unsubmerged Entrance  
(Critical or Subcritical at Exit)**

**Figure 13: 1D Outlet Control Culvert Flow Regimes**

(WBM, p44-45, 2004)

### 5.1.1 Computational Procedure

The computational procedure used to test culvert flows consisted of three tests. Initially each culvert regime (A-K), as defined by ESTRY, was tested; secondly, culvert parameters were tested. The parameters tested were;

- Culvert length
- Culvert width
- Entry loss coefficients
- Culvert number

Finally, non-uniform flow tests were used to check the ability of ESTRY to model the transition of flow between different flow regimes, based on engineering theory. This ensures the transition between inlet and outlet controlled flow is tested.

These tests were undertaken for pipe and rectangular culverts. A summary of the simulations undertaken is provided in Table 2 and Table 3.

**Table 2: Summary of Pipe Culvert Simulation – Regime test**

Run	Flow Regime	Inflow (ms <sup>-1</sup> )	Downstream Depth (m)	Diameter (mm)	Length (m)	Entry Loss (k <sub>e</sub> )	Upstream invert (m)	Downstream invert (m)	Number of culverts
1	A	2	7.1	1500	35	0.5	7.5	7	1
2	B	6.5	8.1	1500	35	0.5	10	7	1
3	C	2	6.5	1500	35	0.5	7	7	1
4	D	0.5	8	1500	35	0.5	7	7.5	1
5	E	5	6.5	1500	35	0.5	7	7	1
6	F	8	9.2	1500	35	0.5	8	7	1
7	H	5	7.7	1500	35	0.5	6.5	7	1
8	J	2	7.7	1500	35	0.5	6.5	7	1
9	K								
10	L	6.5	8.5	1500	35	0.5	10	7	1

**Table 3: Summary of Pipe Culvert Simulations - Variable Test**

Run	Inflow (ms <sup>-1</sup> )	Downstream Depth (m)	Diameter (mm)	Length (m)	Entry loss (k <sub>e</sub> )	Number of culverts	Upstream invert (m)	Downstream invert (m)	Tailwater depth (m)
11	3.5	7.4	1500	35	0.5	1	7	6.5	0.4
12	3.5	7.4	1500	50	0.5	1	7	6.5	0.4
13	3.5	7.4	1500	20	0.5	1	7	6.5	0.4
14	3.5	7.4	1500	35	0.2	1	7	6.5	0.4
15	3.5	7.4	1500	35	0.8	1	7	6.5	0.4
16	3.5	7.4	900	35	0.5	1	7	6.5	0.4
17	3.5	7.4	2100	35	0.5	1	7	6.5	0.4
18	3.5	7.4	1500	35	0.5	2	7	6.5	0.4
19	3.5	7.4	1500	50	0.5	1	7	6.5	2.0
20	3.5	7.4	1500	20	0.5	1	7	6.5	2.0

**Table 4: Summary of Rectangular Culvert Simulations - Regime Test**

Run	Flow Regime	Inflow (m <sup>3</sup> s <sup>-1</sup> )	Downstream Depth (m)	Height (mm)	Diameter (mm)	Length (m)	Entry Loss (k <sub>e</sub> )	Upstream invert (m)	Downstream invert (m)	Number of culverts
21	A	3.5	7.1	1500	1500	35	0.5	7.5	7	1
22	B	6.5	8.1	1500	1500	35	0.5	10	7	1
23	C	2	7.1	1500	1500	35	0.5	7	7	1
24	D	0.5	8	1500	1500	35	0.5	7	7	1
25	E	5	6.5	1500	1500	35	0.5	7	7	1
26	F	8	9.2	1500	1500	35	0.5	8	7	1
27	H	5	7.7	1500	1500	35	0.5	6.5	7	1
28	J	0.5	7.1	1500	1500	35	0.5	6	7	1
29	K	5.8	8.9	1500	1500	35	0.5	10	7	1
30	L	6.5	8.5	1500	1500	35	0.5	10	7	1

**Table 5: Summary of Rectangular Culvert Simulations - Variable Test**

Run	Inflow ( $\text{m}^3\text{s}^{-1}$ )	Downstream Depth (m)	Height (mm)	Width (mm)	Length (m)	Entry Loss ( $k_e$ )	Number of culverts	Upstream invert (m)	Downstream invert (m)	Tailwater depth (m)
31	3.5	7.4	1500	1500	35	0.5	1	7	6.5	0.4
32	3.5	7.4	1500	1500	50	0.5	1	7	6.5	0.4
33	3.5	7.4	1500	1500	20	0.5	1	7	6.5	0.4
34	3.5	7.4	1500	1500	35	0.2	1	7	6.5	0.4
35	3.5	7.4	1500	1500	35	0.8	1	7	6.5	0.4
36	3.5	7.4	1500	900	35	0.5	1	7	6.5	0.4
37	3.5	7.4	1500	2100	35	0.5	1	7	6.5	0.4
38	3.5	7.4	1500	1500	35	0.5	2	7	6.5	0.4
39	3.5	7.4	1500	1500	50	0.5	1	7	6.5	2
40	3.5	7.4	1500	1500	20	0.5	1	7	6.5	2

For non-uniform testing, a pipe culvert of 1.5m diameter and a rectangular culvert of the dimensions 1.5m x 1.5m was used. The following variables were assigned to the test culverts.

- Entry loss = 0.5
- Exit loss = 1.0
- Height contraction coefficient = 0.8
- Width contraction coefficients = 1.0
- Length = 35m
- Number of culverts =1
- Upstream and downstream invert = 7m

**Table 6: Culvert Flow - Non-uniform flow conditions**

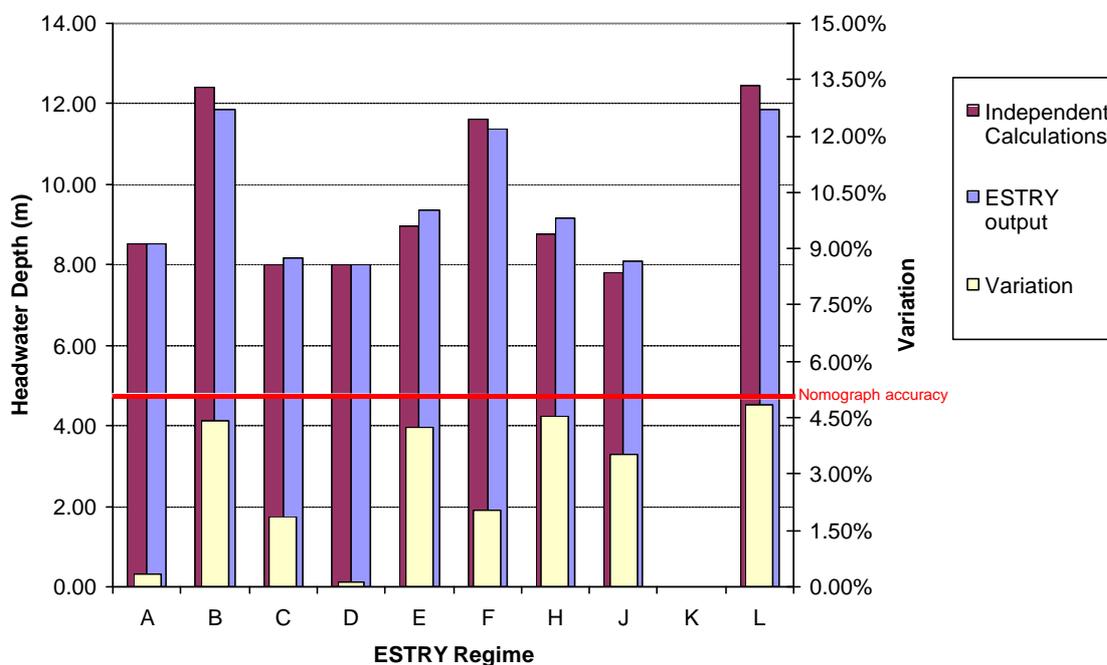
Run	Time (hours)	Inflow ( $\text{m}^3\text{s}^{-1}$ )	Downstream Depth (m)
41 (pipe culvert),	0	1.5	7.5
42 (Rectangular culvert)	1	1.5	7.5
	2	1.5	8
	3	1.5	9
	4	1.5	9.5
	5	1.5	9
	6	1.5	8
	7	1.5	7.5

## 5.1.2 Results

### 5.1.2.1 Uniform Flow

A summary of the culvert test results is given graphically Figure 14 to Figure 17.

Runs 1-10 were used to test that ESTRY was representing each culvert regime (as defined by



ESTRY) accurately for pipe culverts. The test results are shown in Figure 14.

**Figure 14: Pipe Culvert - Regime Results**

Runs 11-20 tested the variables that contribute to headwater depth calculations for circular culverts. Listed below are the variables tested for each run.

- Run 11 - base run to which all other runs could be compared against
- Run 12 and 13 - culvert length during inlet control
- Run 14 and 15 - culvert entrance losses
- Run 16 and 17 – culvert diameter
- Run 18 – number of culverts
- Run 19 and 20 – culvert length during outlet control conditions

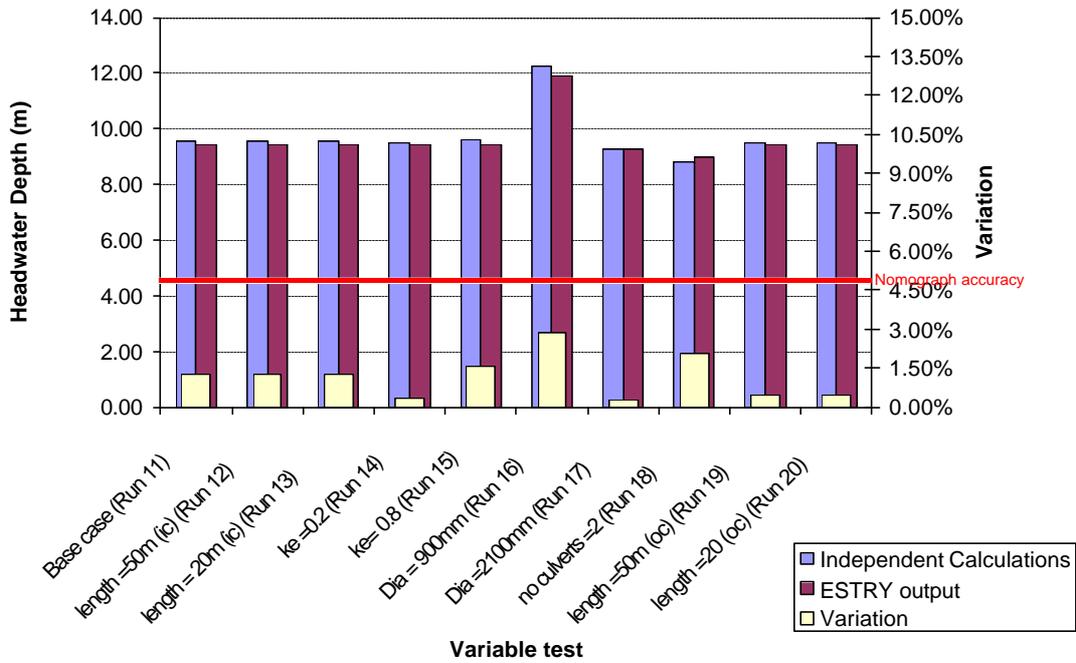


Figure 15: Pipe Culvert – Variable Results

Runs 21-30 were used to test that ESTRY was representing each culvert regime accurately for box culverts. The test results are shown in Figure 16

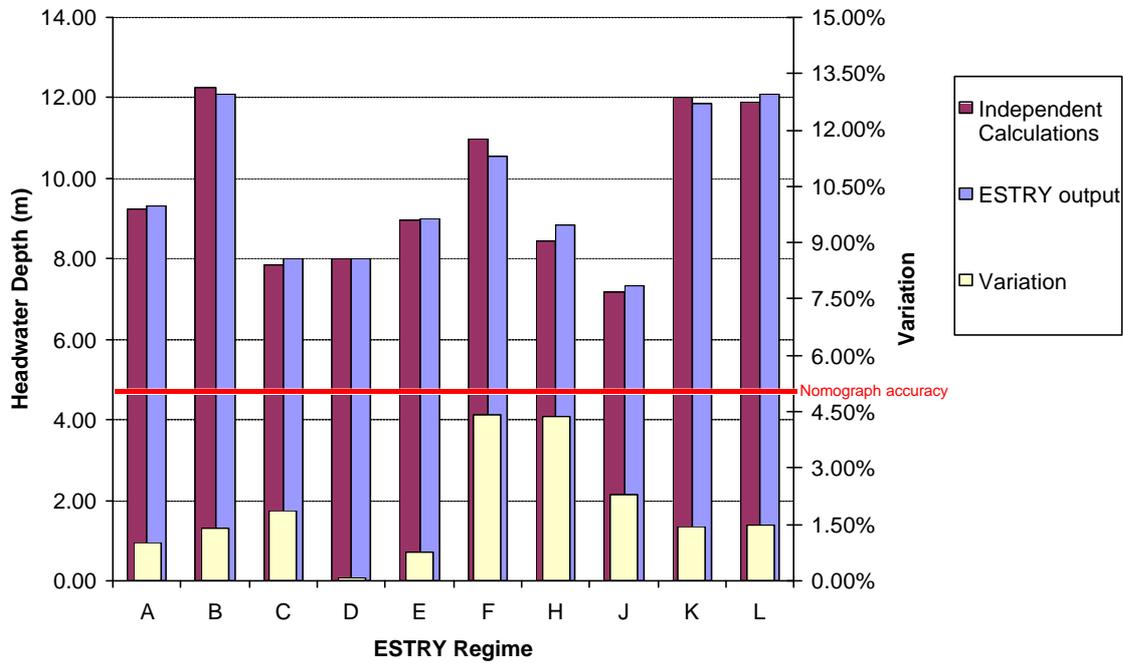


Figure 16: Box Culvert - Regime Results

Runs 31-40 tested the variables that contribute to headwater depth calculations for box culverts. Listed below are the test explanations for each run.

- Run 31 - base run to which all other runs could be compared against
- Run 32 and 33 - culvert length during inlet control
- Run 34 and 35 - culvert entrance losses
- Run 36 and 37 – culvert width
- Run 38 – number of culverts
- Run 39 and 40 – culvert length during outlet control

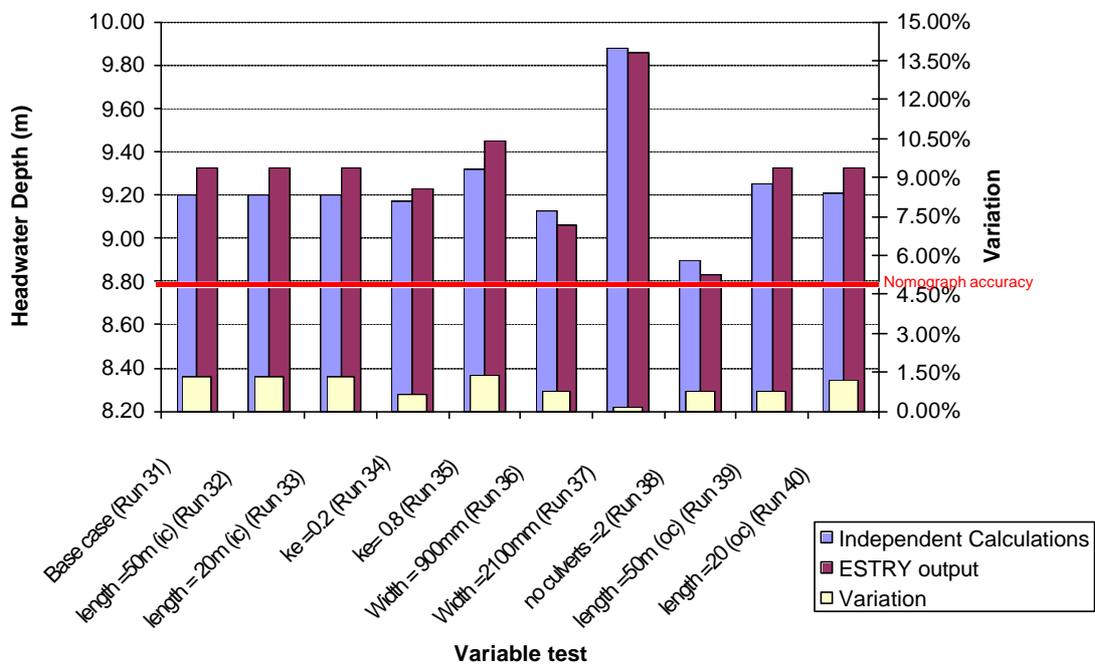


Figure 17: Box Culvert – Variable Results

5.1.2.2 Non-Uniform flow

Run 41 and 42 tests the transition of flow from inlet to outlet control, and vice versa. The testing has been conducted on pipe and box culverts.

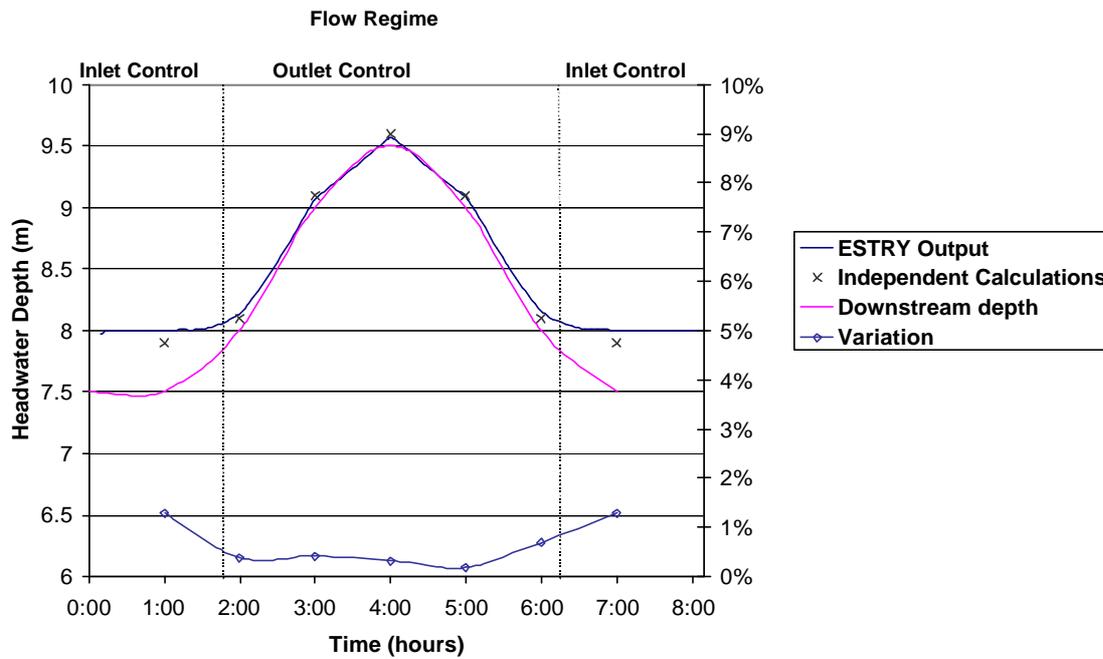


Figure 18: Pipe Culvert - Non Uniform Flow Results

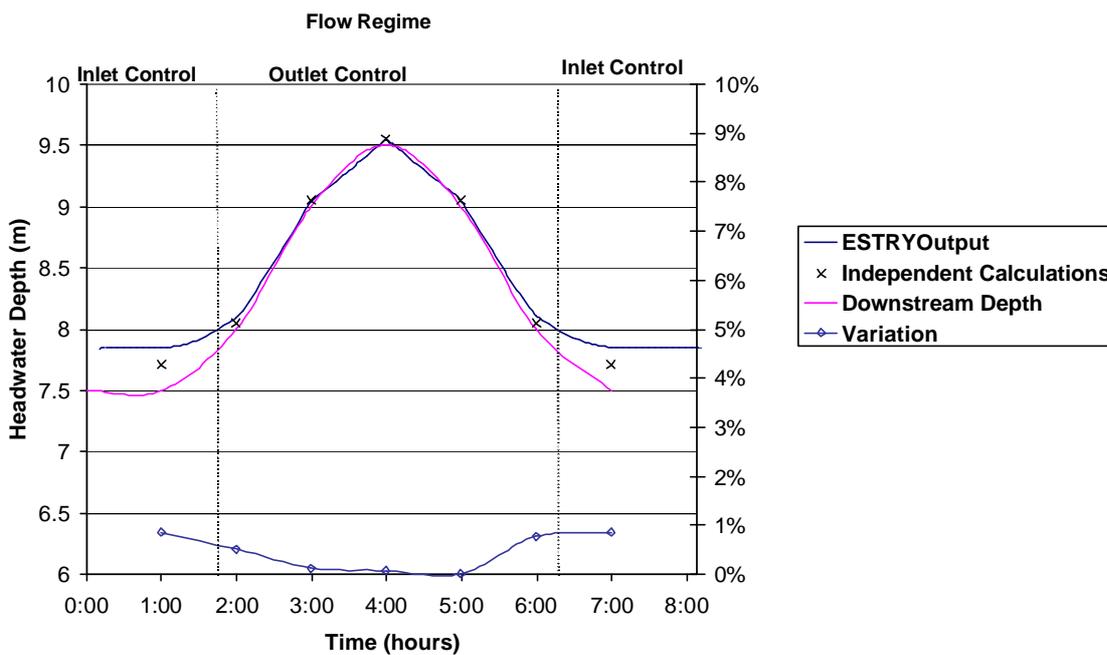


Figure 19: Box Culvert - Non Uniform Flow Results

### 5.1.3 Discussion

The culvert testing initially tested the culvert regimes identified by the ESTRY program. Secondly, variables affecting headwater depth based on engineering theory were tested. During the testing of the variables it was considered important to test culvert flow under inlet and outlet control. Finally the transition of flow between inlet and outlet control was tested.

During inlet control, theory states that conditions downstream of the culvert inlet do not contribute to the headwater depth. This means that entry losses and culvert diameter are the dominant variables contributing to headwater depth. Culvert length, however, has no effect on headwater depth. The results show that ESTRY represents inlet control correctly.

Under outlet control downstream conditions influence headwater depth. In contrast to inlet controlled flow conditions, culvert length affects headwater depth. When experiencing outlet control the results show that ESTRY operates correctly, and as expected, the culvert length influences the headwater depth.

The steady state model runs testing inlet and outlet controlled flow regimes indicate that ESTRY provides accurate estimations for headwater depth. To complete the culvert analysis it was considered necessary to also assess the capabilities of ESTRY to estimate flow that is experiencing a flow regime change. Dynamic models were created, modelling a test culvert that was flowing initially under inlet control. The flow was varied such that the culvert flow developed into outlet-controlled flow, after a period of time, the culvert flow was then returned to inlet control. The testing indicated that ESTRY effectively models the transition of flow between inlet and outlet control, for both, pipe and rectangular culverts.

Overall, the results show that ESTRY produces results similar to predicted headwater depths using the culvert nomograph method developed by the CPAA (1991). The variation in the results ranged from 0.11% to 5%, equalling a maximum flow depth variation of 0.4m. The average variation of the results was calculated to be 1.69%, which in absolute terms, is an average depth variation of 0.14m.

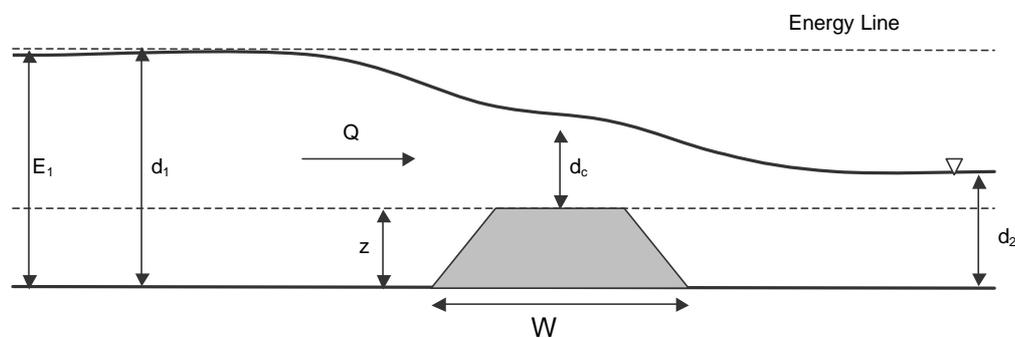
During the study it was expected that the ESTRY output would vary from the calculated depth by up to 5%. This was assumed based on the creation of the culvert nomographs used for the testing. The nomographs were produced from empirical laboratory studies and it is recognised that they are only a method of headwater depth estimation.

Since the ESTRY results all produce values with a variation less than 5%, the testing shows that ESTRY is representing culvert flow consistent with current engineering depth estimation practices.

## 5.2 Weir Flow

A weir is an obstruction in a channel that raises the upstream water level, whilst allowing fluid flow to pass over or through the obstruction (Streeter, p416, 1958). In practice, a weir is an accurate flow-measuring device (Chow, p360, 1973). As such they are often designed for that purpose.

Although there are many types of weirs, in the context of this project, the only weirs being researched are broad crested weirs. During flood events it is often the case that structures, such as road and rail embankments or even levee banks, act as broad crested weirs. Other weir types such as the Sharp Crested weir and the Ogee weir vary rarely occur in the natural or urbanised environments unless they were designed for a specific purpose, such as for flow measurement. Figure 3 shows a section of a broad crested weir.



**Figure 20: Broad Crested Weir**

A broad crested weir is a flat-crested structure with a flow length ( $L$ ), represented by the distance across the flow path at the weir crest, large compared with the flow thickness. The ratio of flow length of the weir to upstream head over crest must typically be greater than 1.5 to 3 (Chow, p410, 1973).

Research has found there are typically two types of flow encountered during weir flow; these are described as submerged and non-submerged flow.

### 5.2.1 Computational Procedure

TUFLOW has been designed such that a weir can be represented either as a 1D or 2D structure. Basically, the size of the weir structure relative to the chosen grid size of the 2D domain defines whether the weir is better suited to 1D or 2D modelling. Ideally, this means that weirs of small flow length are often represented in 1D and weirs of greater flow length are modelled in 2D. During the testing, both 1D and 2D structures were analysed for flow conditions which were:

- Non-submerged
- Submerged
- Transitional (Moving from submerged into non-submerged flow conditions).

The non-submerged and submerged flow was tested using steady state flow conditions. The transitional testing was conducted using non-uniform flow.

### 5.2.1.1 1D Weir Flow

Non-submerged flow was tested varying; upstream discharge, downstream depth and the flow length were varied. During the testing a weir with a height of 5 metres was used.

Table 7 shows the variables that were tested for free overfall flow conditions.

**Table 7: 1D Non-submerged Flow**

Run	Upstream Discharge, Q ( $\text{m}^3\text{s}^{-1}$ )	Downstream Depth, $d_2$ (m)	Flow Length, L (m)
1	1	4	5
2	3	4	5
3	5	4	5
4	7	4	5
5	1	4.5	5
6	3	4.5	5
7	5	4.5	5
8	7	4.5	5
9	1	5	5
10	3	5	5
11	5	5	5
12	7	5	5
13	1	4.99	5
14	3	4.99	5
15	5	4.99	5
16	7	4.99	5
17	1	4.99	10
18	3	4.99	10
19	5	4.99	10
20	7	4.99	10
21	1	4.99	20

22	3	4.99	20
23	5	4.99	20
24	7	4.99	20

During submerged flow testing; upstream discharge and the weir width were varied. The downstream depth was maintained constant, at 5.5 metres, to ensure that the flow conditions remained as submerged flow. Table 8 shows the variables that were tested for submerged flow conditions.

**Table 8: 1D Submerged Flow**

Run	Upstream Discharge Q ( $\text{m}^3 \text{s}^{-1}$ )	Downstream Depth $d_2$ (m)	Flow Length, L (m)
25	1	5.5	5
26	3	5.5	5
27	5	5.5	5
28	7	5.5	5
29	1	5.5	10
30	3	5.5	10
31	5	5.5	10
32	7	5.5	10
33	1	5.5	20
34	3	5.5	20
35	5	5.5	20
36	7	5.5	20

The transition of flow from submerged to non-submerged flow was tested using a dynamic model, under non-uniform flow conditions. A flow length of 5 metres and a downstream depth of 5.5 metres were used for the testing. The upstream discharge was increased from  $1\text{ m}^3\text{ s}^{-1}$  to  $16\text{ m}^3\text{ s}^{-1}$ ; this ensured that the weir was initially flowing under submerged flow before developing into non-submerged flow. Table 9 shows the variables that were used for transitional flow testing.

**Table 9: 1D Flow Transition Test**

Run	Upstream Discharge, $Q\text{ (m}^3\text{ s}^{-1}\text{)}$	Time, $t$ (hours)	Downstream Depth, $d_2\text{ (m)}$	Flow Length, $L$ (m)
37	1	0	5.5	5
	1.5	1	5.5	5
	2	2	5.5	5
	2.5	3	5.5	5
	3	4	5.5	5
	3.5	5	5.5	5
	4	6	5.5	5
	4.5	7	5.5	5
	5	8	5.5	5
	5.5	9	5.5	5
	6	10	5.5	5
	6.5	11	5.5	5
	7	12	5.5	5
	16	13	5.5	5

#### 5.2.1.2 2D Weir Flow

During the testing a weir with a height of 5 metres was used.

Non-submerged flow was tested varying the; upstream discharge, downstream depth and the flow length. Table 10 shows the variables that were tested.

**Table 10: 2D Non-submerged Flow**

Run	Upstream Discharge Q (m <sup>3</sup> s <sup>-1</sup> )	Downstream Depth d <sub>2</sub> (m)	Flow Length, L (m)
1	10	4	20
2	15	4	20
3	20	4	20
4	10	4.5	20
5	15	4.5	20
6	20	4.5	20
7	10	5	20
8	15	5	20
9	20	5	20
10	10	5	30
11	15	5	30
12	20	5	30
13	10	5	40
14	15	5	40
15	20	5	40

During submerged flow testing; upstream discharge and the weir width were varied. The downstream depth was maintained constant, at 5.5 metres, to ensure that the flow conditions remained as submerged flow. Table 11 shows the variables that were tested for submerged flow conditions.

**Table 11: 2D Submerged Flow**

Run	Upstream Discharge, Q (m <sup>3</sup> s <sup>-1</sup> )	Downstream Depth, d <sub>2</sub> (m)	Flow Length, L (m)
16	10	5.5	20
17	15	5.5	20
18	20	5.5	20
19	10	5.5	30
20	15	5.5	30
21	20	5.5	30
22	10	5.5	40
23	15	5.5	40
24	20	5.5	40

The transition of flow from submerged to non-submerged flow was tested using a dynamic model. The model used a flow length of 20 metres and a constant downstream depth of 5.5 metres. The upstream discharge was increased from  $10\text{m}^3\text{s}^{-1}$  to  $40\text{m}^3\text{s}^{-1}$ . The change in upstream discharge, with constant downstream depth ensured that the weir was initially flowing under submerged flow before developing into non-submerged flow. Table 12 shows the variables that were used for transitional flow testing.

**Table 12: 2D Flow Transition Test**

Run	Upstream Discharge, $Q\ (\text{m}^3\text{s}^{-1})$	Time, t (hours)	Downstream Depth, $d_2\ (\text{m})$	Flow Length, L (m)
25	10	0	5.5	20
	11	1	5.5	20
	12	2	5.5	20
	13	3	5.5	20
	14	4	5.5	20
	15	5	5.5	20
	16	6	5.5	20
	17	7	5.5	20
	18	8	5.5	20
	19	9	5.5	20
	20	10	5.5	20
	40	11	5.5	20

## 5.2.2 Results

### 5.2.2.1 ID Weir Results

A summary of the 1D weir test results is given in Figure 21 to Figure 24.

Runs 1-12 were used to test that ESTRY was representing non-submerged flow accurately. During non-submerged flow, because of free overfall conditions, the downstream depth has no influence on the upstream depth. As expected all runs with identical upstream discharges produced identical upstream depth estimations, independent of downstream depth. The model runs that were found to have identical results are listed;

- Runs 1,5,9
- Runs 2,6,10
- Runs 3,7,11
- Runs 4,8,12.

The results can be seen in Table 13 this has also been graphically shown in Figure 21.

**Table 13: 1D Non-submerged Flow – Results (Downstream Depth)**

Run	Upstream Depth Independent Calculation (m)	Upstream Depth ESTRY Output (m)	Variation (%)
1,5,9	5.24	5.24	0.04%
2,6,10	5.50	5.50	0.09%
3,7,11	5.70	5.71	0.13%
4,8,12	5.87	5.89	0.18%

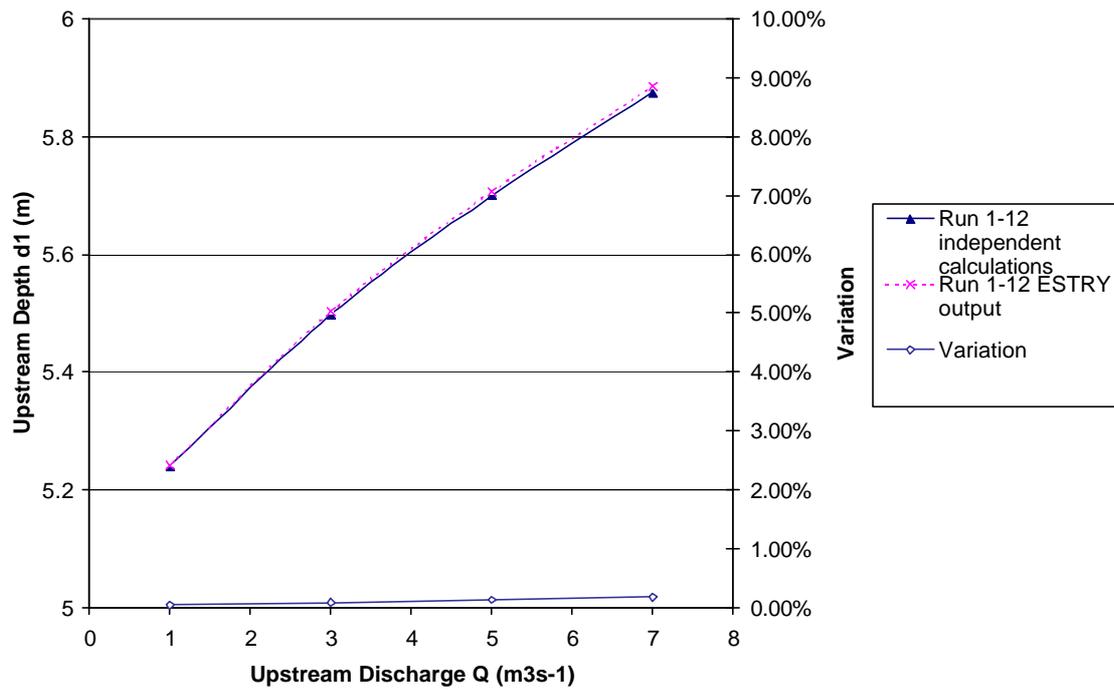


Figure 21: 1D Non-submerged Flow – Results (Downstream Depth)

Runs 13-24 were used to test that ESTRY was representing non-submerged flow accurately for varying weir widths. Since upstream conditions influence downstream depth during non-submerged flow it was expected that for wider weir widths, as defined by the continuity equation, lower upstream depths would be produced. The results are presented in Figure 22.

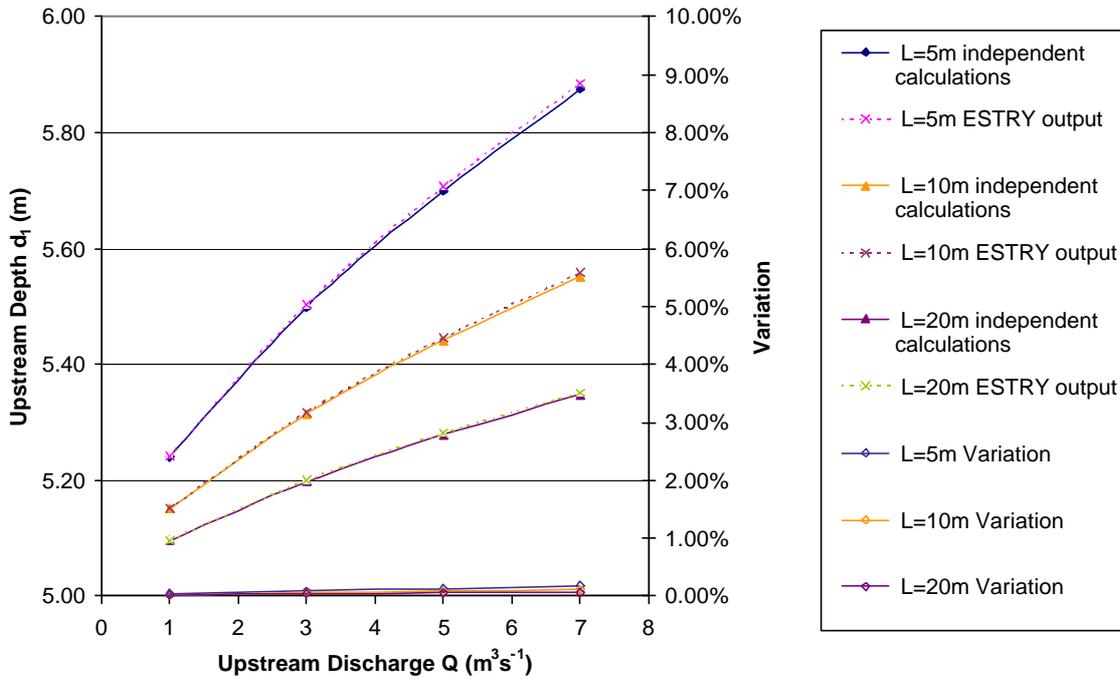


Figure 22: 1D Non-submerged Flow – Results (Flow Length)

Runs 25-36 were used to test that ESTRY was representing submerged flow accurately for varying weir flow lengths. Since both upstream and downstream conditions influence upstream depth during submerged flow it was expected that for greater flow lengths, with a constant downstream level, lower upstream depths would be produced. The results are presented in Figure 23.

In addition to submerged flow it was also tested that ESTRY could function for flow conditions which were conventionally in the reverse direction. To achieve this, the upstream discharge used was designed to be small enough in magnitude to provide an upstream head less than the downstream depth. The conservation of momentum defines that water flows from a greater elevation to a lower elevation due to gravitational forces. Conventionally this means that flow is expected to move from downstream, upstream, in the reverse direction. For varying discharges this was tested for all flow lengths and can be seen when the upstream depth is equal to the downstream depth, 5.5m. The results are presented in Figure 23.

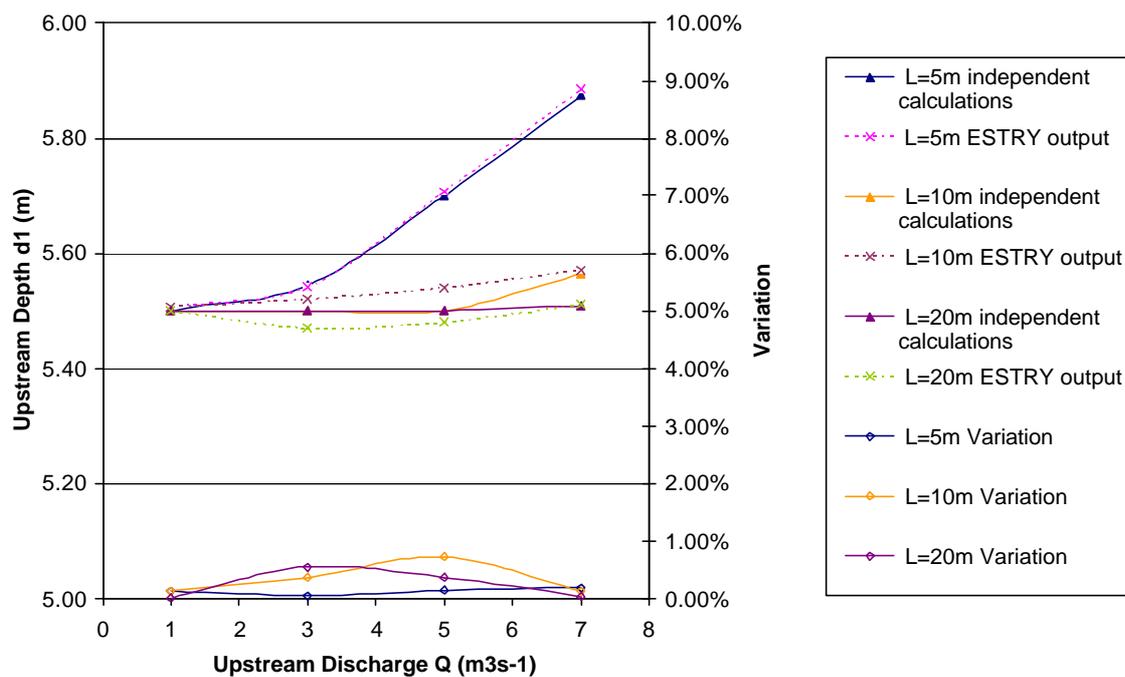
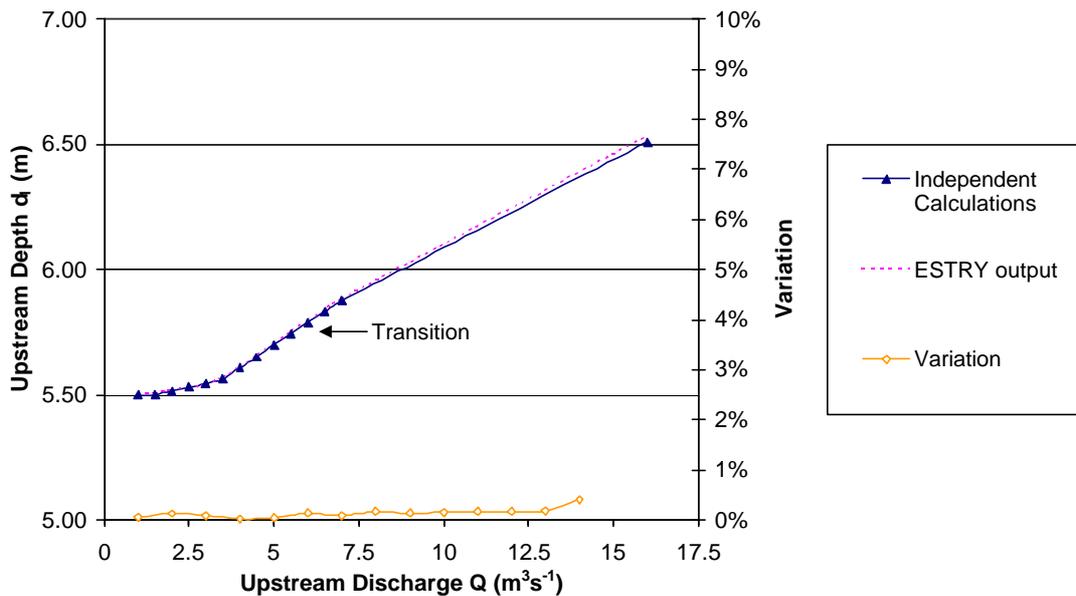


Figure 23: 1D Submerged Flow - Results

Run 37 was used to test that ESTRY was representing submerged flow for varying inundation ratios accurately. This testing method assesses whether ESTRY replicates the transition from submerged flow to unsubmerged flow at an inundation ratio of approximately 0.75. Since both upstream and downstream conditions influence upstream depth during submerged flow it was expected that for smaller flow rates, hence higher inundation ratios, the upstream depth would be at its least. The results are presented in Figure 24.



**Figure 24: 1D Flow Transition – Results**

#### 5.2.2.2 2D Weir Results

A summary of the 2D weir test results is given in Figure 25 to Figure 27.

Runs 1-12 were used to test that TUFLOW was representing non-submerged flow accurately. During non-submerged flow downstream depth has no influence on the upstream depth. As expected all runs replicating identical upstream discharges produced similar upstream depth estimations, independent of downstream depth. The model runs that were found to have similar results are listed;

- Runs 1,4,7
- Runs 2,5,8
- Runs 3,6,9.

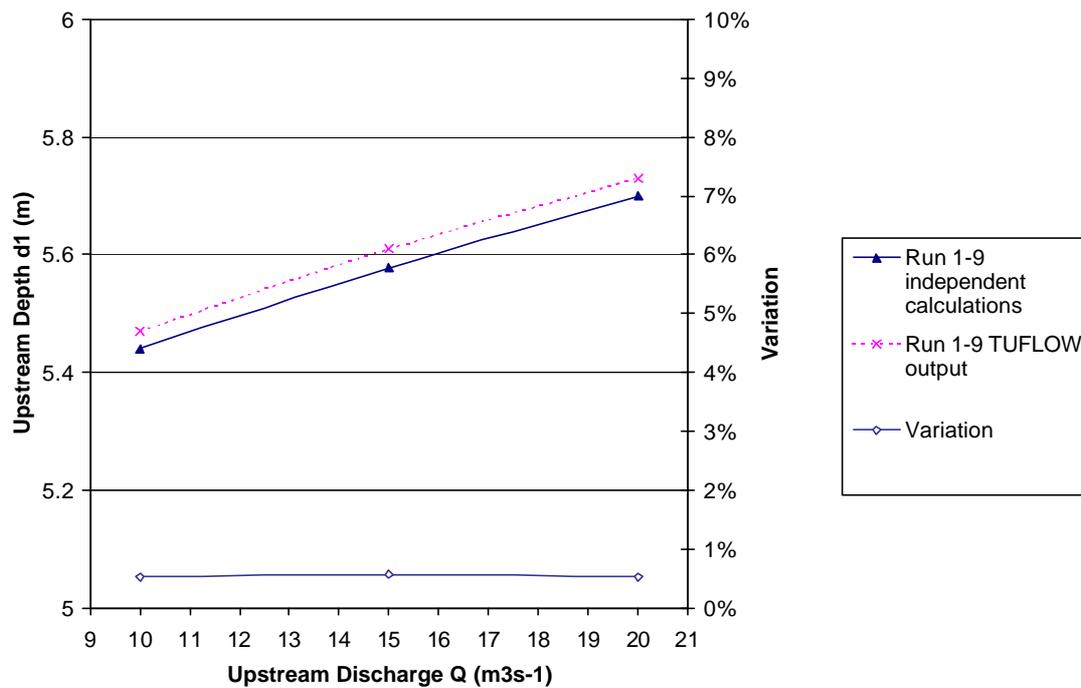
The results can be seen in

Table 14 and graphically in Figure 25.

Figure 23 uses the average upstream depth and variation of runs 1-9 for the given discharge.

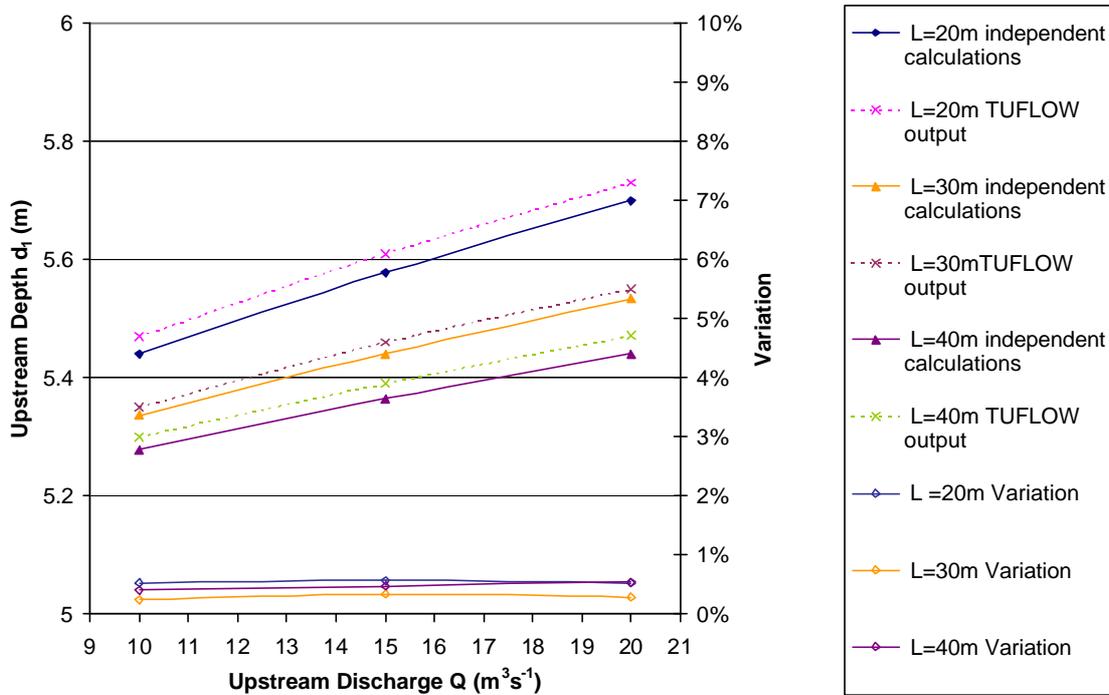
**Table 14: 2D Non-submerged Flow – Results (Downstream Depth)**

Run	Upstream Depth, $d_1$ Independent Calculation (m)	Upstream Depth, $d_1$ TUFLOW Output (m)	Variation (%)
1,4,7	5.24	5.470 – 5.475	0.53% - 0.62%
2,5,8	5.50	5.610 – 5.615	0.58% - 0.66%
3,6,9	5.70	5.725 – 5.732	0.45% - 0.53%



**Figure 25: 2D Non-submerged Flow – Results (Downstream Depth)**

Runs 7-15 were used to test that TUFLOW was representing non-submerged flow accurately for varying weir widths. Since upstream conditions influence downstream depth during non-submerged flow it was expected that for wider weir widths, as defined by the continuity equation, lower upstream depths would be produced. The results are presented in Figure 26.



**Figure 26: 2D Non-submerged Flow – Results (Flow Length)**

Runs 16- 24 were used to test that TUFLOW was representing submerged flow for varying inundation ratios. For each weir width, the designed discharge was varied such that for at least one case the inundation ratio was greater than one. This implies the downstream depth is greater than the upstream depth, hence, the weir flow is in the reverse direction. Figure 27 shows the 2D submerged flow weir results.

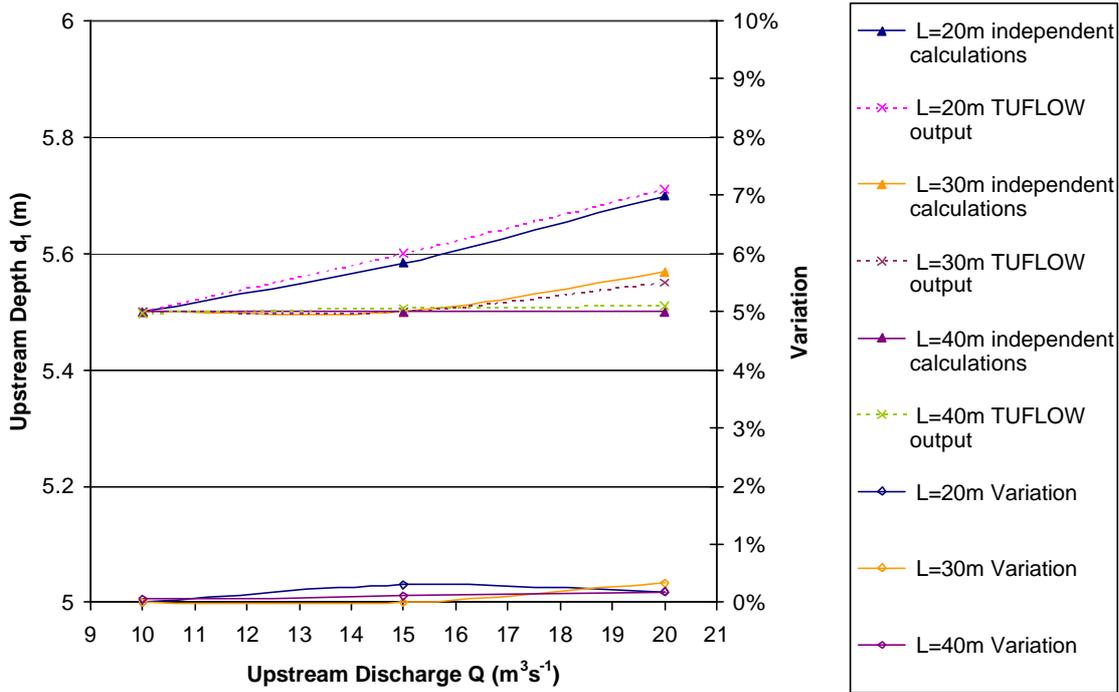


Figure 27: 2D Submerged Flow - Results

Run 25 tested the transition of flow from submerged to non-submerged flow, dynamically, in 2D. For lower discharges it was expected that lower upstream discharges would be experienced.

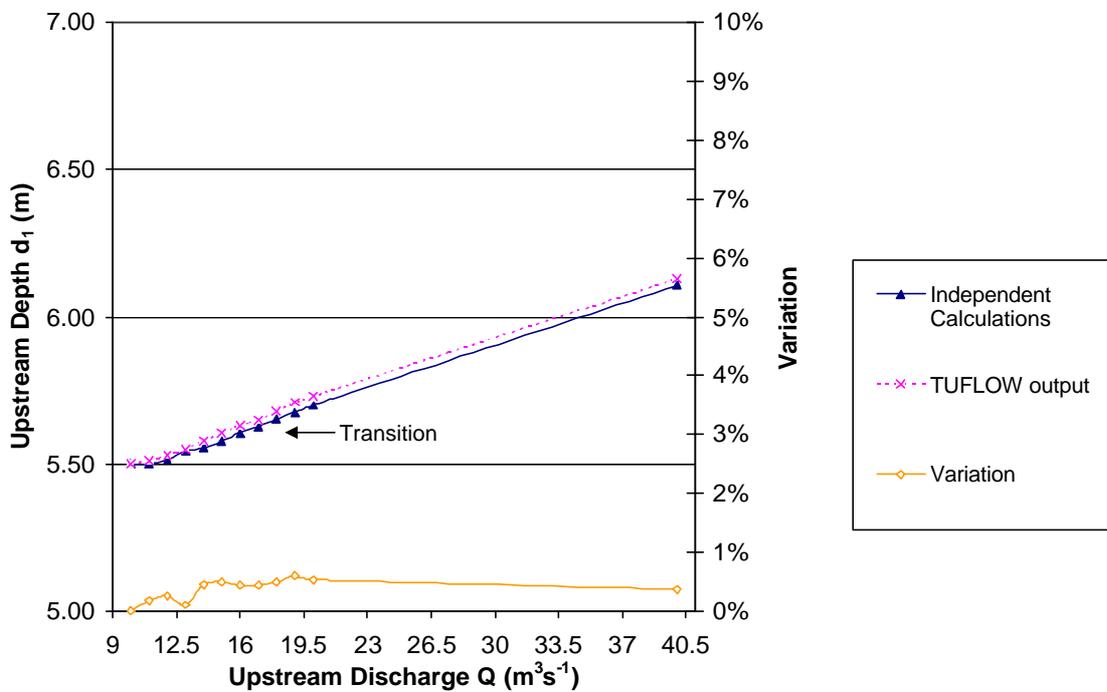


Figure 28: 2D Flow Transition – Results

### 5.2.3 Discussion

TUFLOW is able to represent weir flow in 2D, whilst, ESTRY is used to represent weir flow in 1D. Theoretically there are two regimes of flow that can occur at a weir, submerged and non-submerged flow. These factors required that the weir flow be tested in 1D and 2D, for non-submerged, submerged flow and the transition between the flow regimes.

During non-submerged flow, conditions downstream of the weir do not contribute to the upstream water depth. Therefore, the dominant variables contributing to headwater depth are flow length and upstream discharge. The testing was structured to test non-submerged flow for a variety of downstream depths, upstream discharge and flow length. The results show that ESTRY and TUFLOW represent non-submerged weir flow correctly, when compared against calculations based on critical depth theory. The variation in the 1D results ranged from 0.04% to 0.18%, the 2D results ranged from 0.25% - 0.66%.

Under submerged flow conditions, critical depth calculations cannot be used; instead empirical studies by Bradley (1978) have produced engineering theory that is consistent for submerged flow. During submerged flow conditions, upstream discharge, downstream depth and flow length are all dominant variables contributing to upstream water depth. When comparing the ESTRY and TUFLOW output results against the submerged flow calculations, the results show that ESTRY and TUFLOW are working extremely efficiently. The variation in the 1D results ranged from 0.00% to 0.55%, the 2D results ranged from 0.00% - 0.34%.

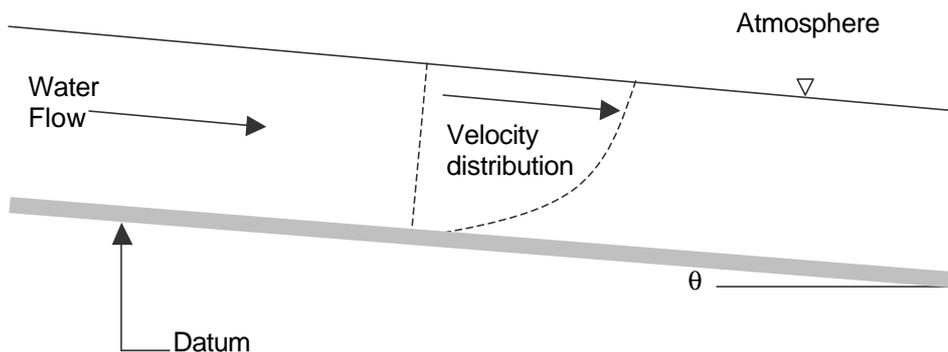
The transition of flow from submerged to non-submerged flow was tested using a dynamic model operating under non-uniform flow conditions. Steady state calculations were used to check the validity of the ESTRY and TUFLOW output results. These were calculated at hourly intervals for specified upstream discharges. Results show that ESTRY and TUFLOW are representing the transition from submerged to non-submerged flow proficiently. Output results compared against the steady state calculations show the calculated variation for 1D flow ranged from 0.02% - 0.4%, whilst the variation for 2D flow was calculated to range from 0.00% - 0.6%.

Overall, the ESTRY and TUFLOW results produced depth estimations with a variation of less than 1% from independently calculated depths using current engineering depth estimation practices. In absolute terms the maximum variation in depth for all the weir testing was found to be 0.04 metres. The testing illustrates that ESTRY and TUFLOW are operating efficiently, representing weir flow correctly.

## 5.3 Open Channel Flow

Open channel flow is characterised by a waterway, canal or conduit in which a fluid flows with a free surface, subjected to atmospheric pressure (Chanson, p6, 1999; Finnemore & Franzini, p407, 2002).

During open channel flow, flow is caused by the gravity component along the slope of the channel; not by some external head, as is the case with pipe flow.



**Figure 29: Open Channel Flow**

Open channel flow typically occurs within river systems and on floodplains during flood events.

### 5.3.1 Computational Procedure

TUFLOW has the ability to represent open channel flow in 1D and 2D. Conventionally 1D channels are used to represent smaller drainage elements, which cannot be accurately represented by the use of a 2D grid mesh. The computational procedure used to test open channel flow consisted of four tests. These were:

- 2D channel flow (ie; River flow)
- 2D floodplain flow
- 2D floodplain/ 2D channel flow
- 2D floodplain/ 1D channel flow.

For each test scenario, runs were developed testing the accuracy for a variety of:

- Channel bed slopes

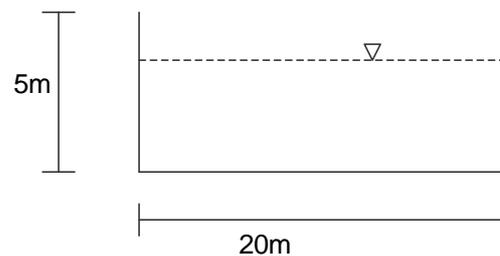
- Manning's n
- Discharges.

Throughout testing, channel bed slope was varied from 10.4%, representing a “channel of large slope”, to 0.02%, representing a “channel of small slope” as defined by Chow (p33, 1959). Manning's n values ranging from 0.02 to 0.12 were used. These values are the maximum and minimum estimated Manning's n values for both river and floodplain flows quoted by Chow (pp110-113, 1959).

Finally flow discharges ranging from  $800\text{m}^3/\text{s}$  to  $10\text{m}^3/\text{s}$  were used such that supercritical, critical and subcritical flow conditions were tested.

### 5.3.1.1 2D Channel Flow

A basic rectangle channel, as shown in Figure 30, was used for the testing.



**Figure 30: 2D Channel Flow Model**

Table 15 shows the variables that were tested for 2D channel flow.

- Runs 1-18 use a bed slope of 10.4% testing super critical flow
- Runs 19-36 use a bed slope of 5.2% testing super critical and critical flow
- Runs 37-54 use a bed slope of 2.6% testing super critical to sub critical flow
- Runs 55-72 use a bed slope of 1.3% testing super critical to sub critical flow
- Runs 73-90 use a bed slope of 0.2% testing sub critical flow
- Runs 91-108 use a bed slope of 0.02% testing sub critical flow

**Table 15: 2D Channel Flow Variables**

Mannings n	Discharge ( $\text{m}^3\text{s}^{-1}$ )		
	10	20	30
0.02	Run 1,19,37,55,73,91	Run 7,25,43,61,80,97	Run 13,31,49,67,86,103
0.04	Run 2,20,38,56,74,92	Run 8,26,44,62,81,98	Run 14,32,50,68,87,104
0.06	Run 3,21,39,57,75,93	Run 9,27,45,63,82,99	Run 15,33,51,69,88,105
0.08	Run 4,22,40,58,76,94	Run 10,28,46,64,83,100	Run 16,34,52,70,89,106
0.10	Run 5,23,41,59,78,95	Run 11,29,47,65,84,101	Run 17,35,53,71,90,107
0.12	Run 6,24,42,60,79,96	Run 12,30,48,66,85,102	Run 18,36,54,72,91,108

### 5.3.1.2 2D Floodplain Flow

A simple rectangular floodplain, as shown in Figure 30, was used for the testing. The width of the floodplain was made sufficiently large enough to ensure that the width is greater than 10 times the flow depth. As defined by Chow (p27, 1957) this classifies the floodplain as a “wide open channel”.

**Figure 31: 2D Floodplain Flow Model**

Table 15 shows the variables that were tested for 2D floodplain flow.

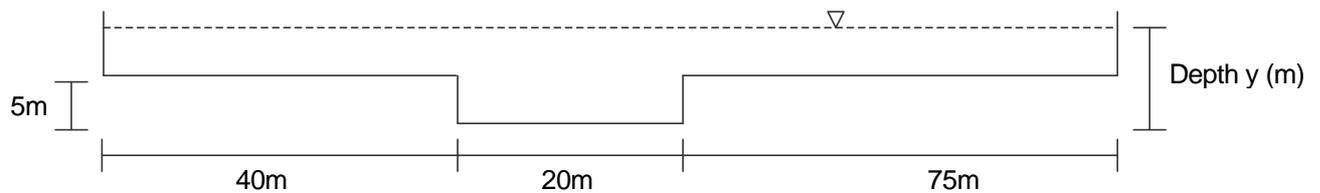
- Runs 1-18 use a bed slope of 10.4% testing super critical flow
- Runs 19-36 use a bed slope of 5.2% testing super critical and critical flow
- Runs 37-54 use a bed slope of 2.6% testing super critical to sub critical flow
- Runs 55-72 use a bed slope of 1.3% testing super critical to sub critical flow
- Runs 73-90 use a bed slope of 0.2% testing sub critical flow
- Runs 91-108 use a bed slope of 0.02% testing sub critical flow

**Table 16: 2D Channel Flow Variables**

Mannings n	Discharge ( $\text{m}^3\text{s}^{-1}$ )		
	100	200	300
0.02	Run 1,19,37,55,73,91	Run 7,25,43,61,80,97	Run 13,31,49,67,86,103
0.04	Run 2,20,38,56,74,92	Run 8,26,44,62,81,98	Run 14,32,50,68,87,104
0.06	Run 3,21,39,57,75,93	Run 9,27,45,63,82,99	Run 15,33,51,69,88,105
0.08	Run 4,22,40,58,76,94	Run 10,28,46,64,83,100	Run 16,34,52,70,89,106
0.10	Run 5,23,41,59,78,95	Run 11,29,47,65,84,101	Run 17,35,53,71,90,107
0.12	Run 6,24,42,60,79,96	Run 12,30,48,66,85,102	Run 18,36,54,72,91,108

### 5.3.1.3 2D Floodplain Flow/ Channel Flow

Tests models were developed to represent compound channel flow, when the flow is not solely confined to the channel. A simple floodplain/channel scenario, as shown in Figure 32 was used for the testing. The testing was modelled using steady and non-steady state flow conditions.



**Figure 32: 2D Floodplain/ Channel Flow Model**

The models were created to replicate both steep and flat-sloped floodplain/channel cases.

- Steady state runs 1-18 and non steady state runs 37-42 use a bed slope of 1.3%
- Steady state runs 19-36 and non steady state runs 43-48 use a bed slope of 0.02%

Throughout the testing the Manning's n roughness coefficient for the channel was fixed at 0.05. The Manning's n coefficient of the floodplain and the upstream discharge were varied during the testing, as is shown in Table 17 and Table 18. For the non-steady state testing the upstream discharge was varied from  $300\text{m}^3/\text{s}$  to  $800\text{m}^3/\text{s}$  over a period of 3 hours. The flows agreement was defined as:

- Time = 0 (hours), Discharge =  $300\text{m}^3/\text{s}$
- Time = 1 (hours), Discharge =  $500\text{m}^3/\text{s}$
- Time = 2 (hours), Discharge =  $800\text{m}^3/\text{s}$ .

The model tests were conducted for 2D floodplain/2D channel and 2D floodplain/1D channel configurations.

**Table 17: 2D Floodplain/ Channel Flow Variables (Steady State)**

<b>Manning's n</b>	<b>Discharge (m<sup>3</sup>s<sup>-1</sup>)</b>		
	<b>300</b>	<b>500</b>	<b>800</b>
0.02	Run 1, 19	Run 7, 25	Run 13, 31
0.04	Run 2, 20	Run 8, 26	Run 14, 32
0.06	Run 3, 21	Run 9, 27	Run 15, 33
0.08	Run 4, 22	Run 10, 28	Run 16, 34
0.10	Run 5, 23	Run 11, 29	Run 17, 35
0.12	Run 6, 24	Run 12, 30	Run 18, 36

**Table 18: 2D Floodplain/ Channel Flow Variables (Non-Steady State)**

<b>Manning's n</b>	<b>Run</b>
0.02	Run 37, 43
0.04	Run 38, 44
0.06	Run 39, 45
0.08	Run 40, 46
0.10	Run 41, 47
0.12	Run 42, 48

## 5.3.2 Results

### 5.3.2.1 2D Channel Flow and 2D Floodplain Flow

2D channel and 2D floodplain flow was tested for a variety of bed slopes, upstream discharge and Manning's n roughness coefficients. The testing structure was

- Runs 1-18 test for a steep bed slope ( $>10\%$ ), typical of the upper reaches of a coastal river or stream. The tests check whether TUFLOW can represent supercritical flow conditions in 2D channel flow situations. The test results are presented in Figure 33 for 2D channel flow Figure 39 and for 2D floodplain flow.
- Runs 19-36 test for a mild bed slope, typical of the upper reaches of a coastal river or stream. The testing checks that TUFLOW can represent supercritical and critical flow conditions when replicating 2D channel flow cases. The test results are presented in Figure 34 for 2D channel flow Figure 40 and for 2D floodplain flow.
- Runs 37-54 were designed to test the 2D channel flow for a small slope ( $<10\%$ ), typical of the middle reaches of a coastal river or stream. The testing checks that TUFLOW can represent supercritical, critical and subcritical flow conditions for 2D channel flow cases. The test results are presented in Figure 35 for 2D channel flow and Figure 41 for 2D floodplain flow.
- Runs 55-72 were designed to test the 2D channel flow for a small slope, typical of the middle reaches of a coastal river or stream. The testing checks that TUFLOW can represent supercritical, critical and subcritical flow conditions for 2D channel flow cases. The test results are presented in Figure 36 for 2D channel flow and Figure 42 for 2D floodplain flow.
- Runs 73-108 test for a very small bed slope, typical of the middle/lower reaches of a coastal river or stream. The testing checks that TUFLOW can represent subcritical flow conditions for 2D channel flow cases. The test results are presented in Figure 37 and Figure 38 for 2D channel flow and Figure 43 and Figure 44 for 2D floodplain flow.

It was expected that for greater flow rates and increased Manning's n roughness coefficients, the corresponding flow depth would increase. The test results are presented in Figure 33.

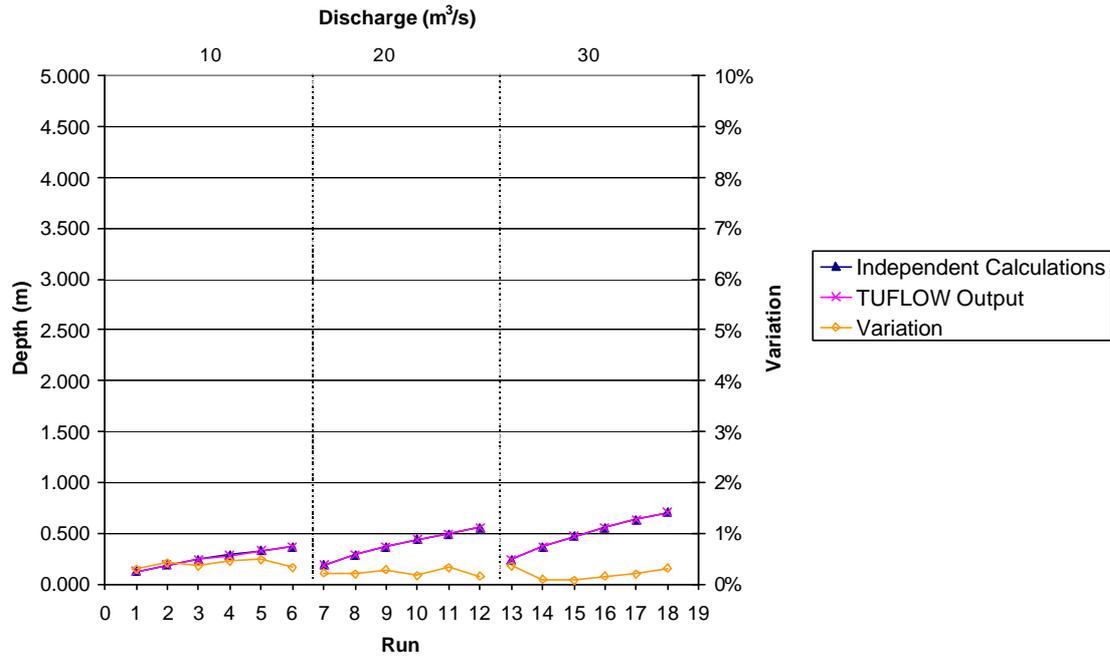


Figure 33: 2D Channel Flow Results- Slope= 10.4%

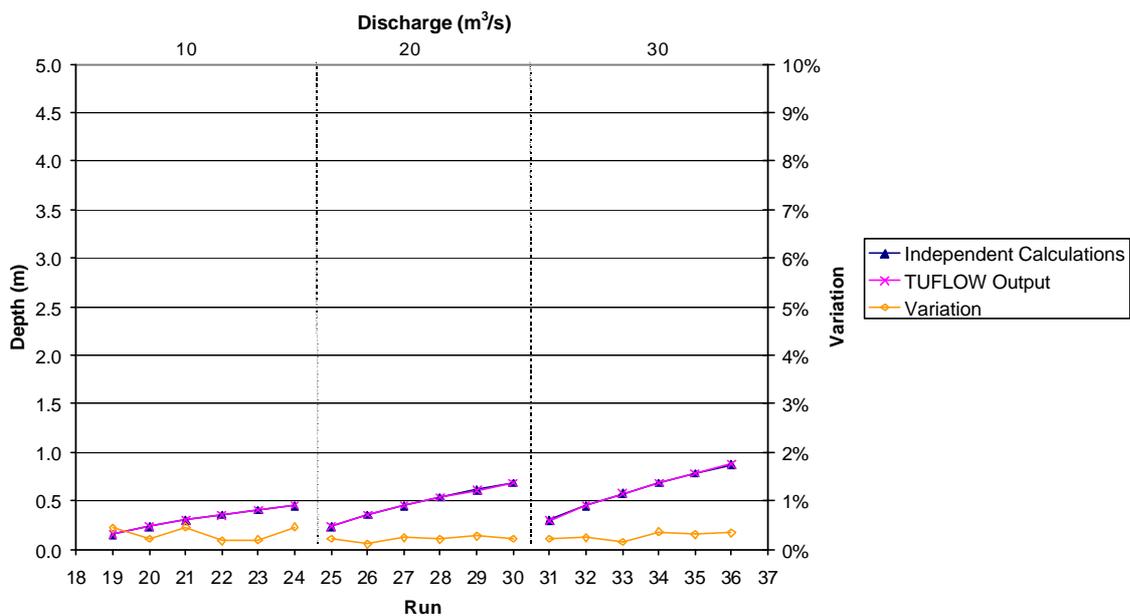
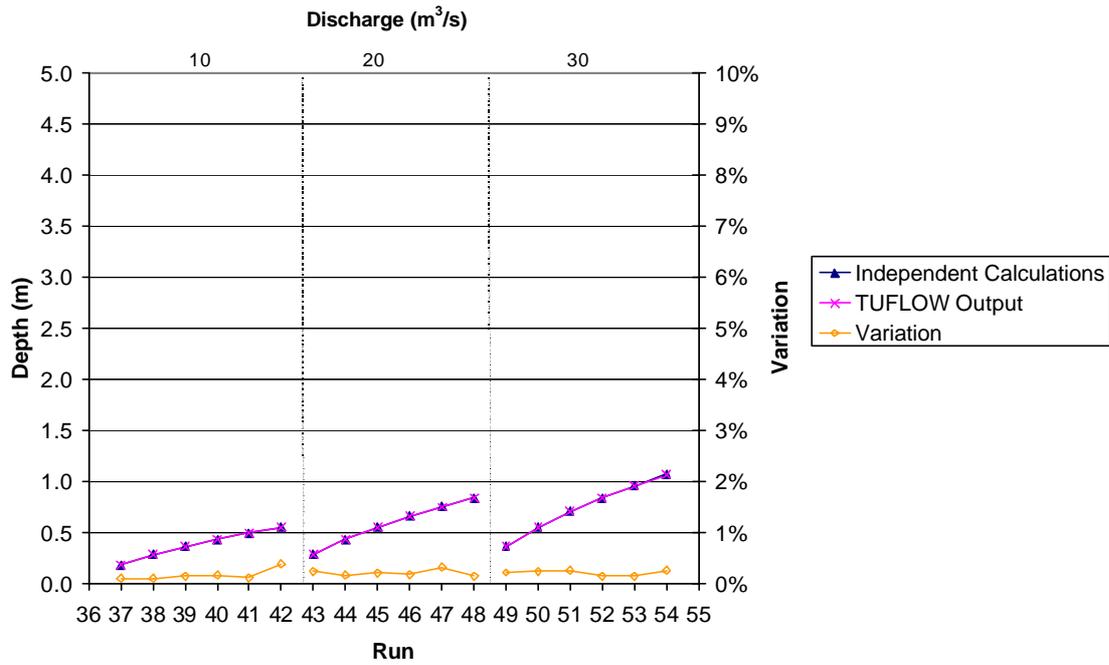


Figure 34: 2D Channel Flow Results- Slope= 5.2%

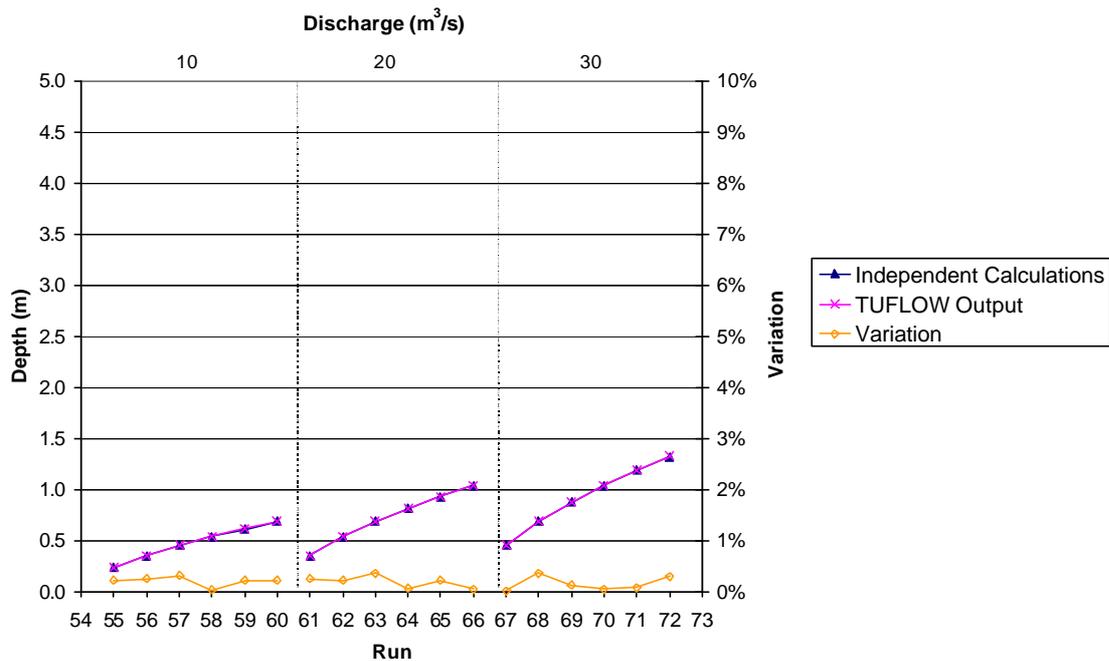
Supercritical flow = Runs 1-10, 13-14, 20-23, 25-26, 31

Critical flow = Runs 15, 24, 27, 32

Subcritical flow = Runs 11-12, 16-19, 28-30, 33-36



**Figure 35: 2D Channel Flow Results- Slope= 2.6%**



**Figure 36: 2D Channel Flow Results- Slope= 1.3%**

Supercritical flow = Runs 36-39, 43-44, 49, 54-56, 61, 67

Critical flow = Runs 40, 57, 67

Subcritical flow = Runs 41-42, 45-48, 50-55, 58-60, 62-66, 68-72

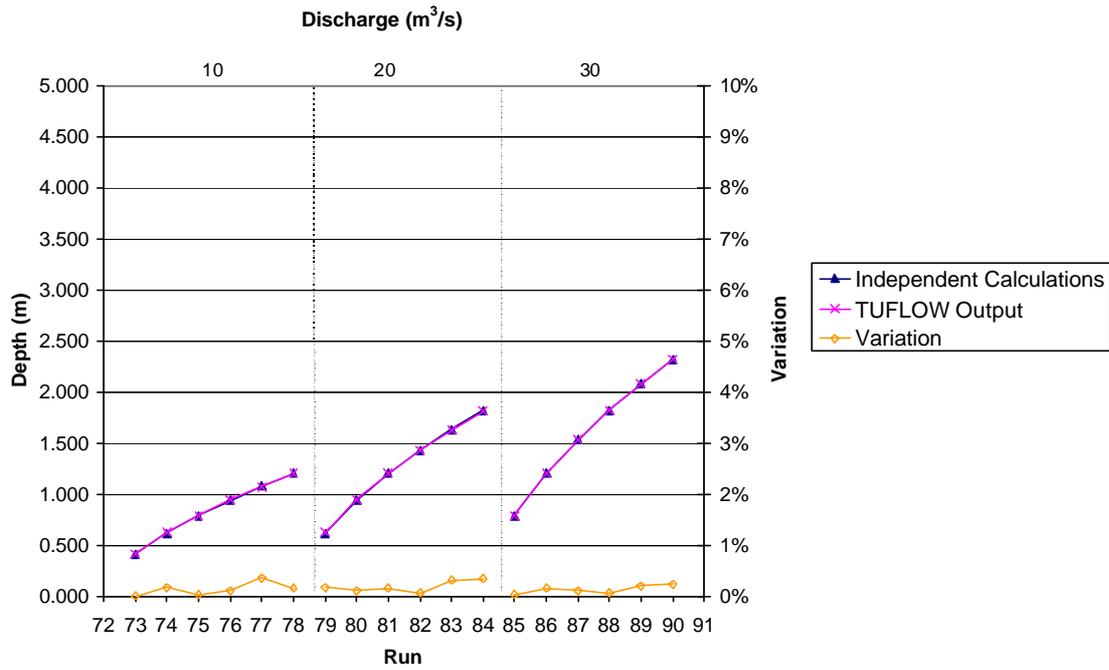


Figure 37: 2D Channel Flow Results- Slope= 0.2%

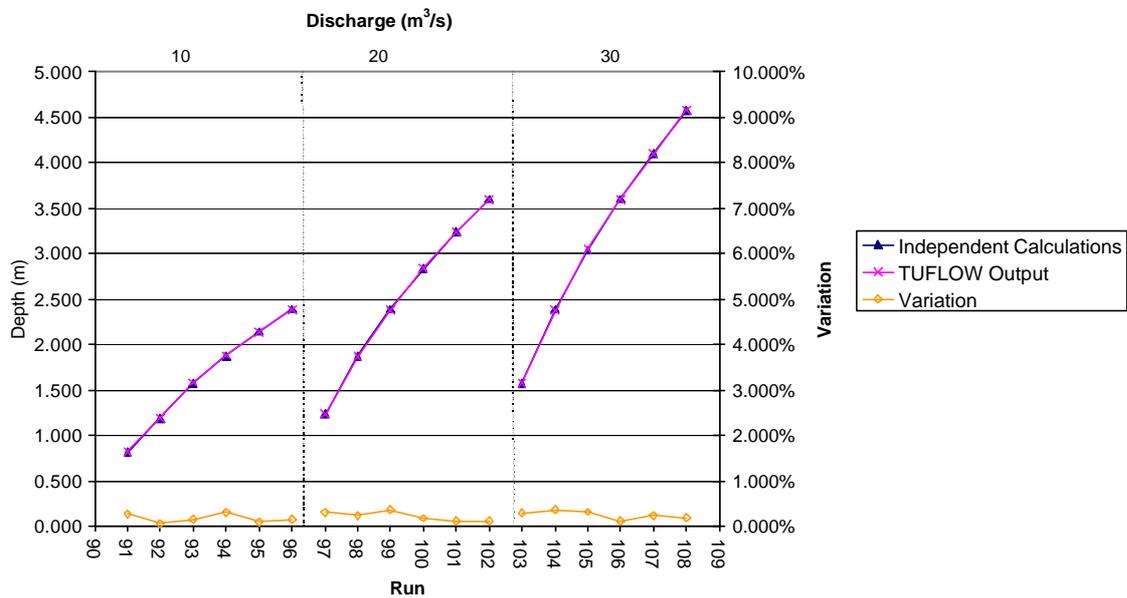


Figure 38: 2D Channel Flow Results- Slope= 0.02%

Supercritical flow = Run 73

Subcritical flow = Runs 74-108

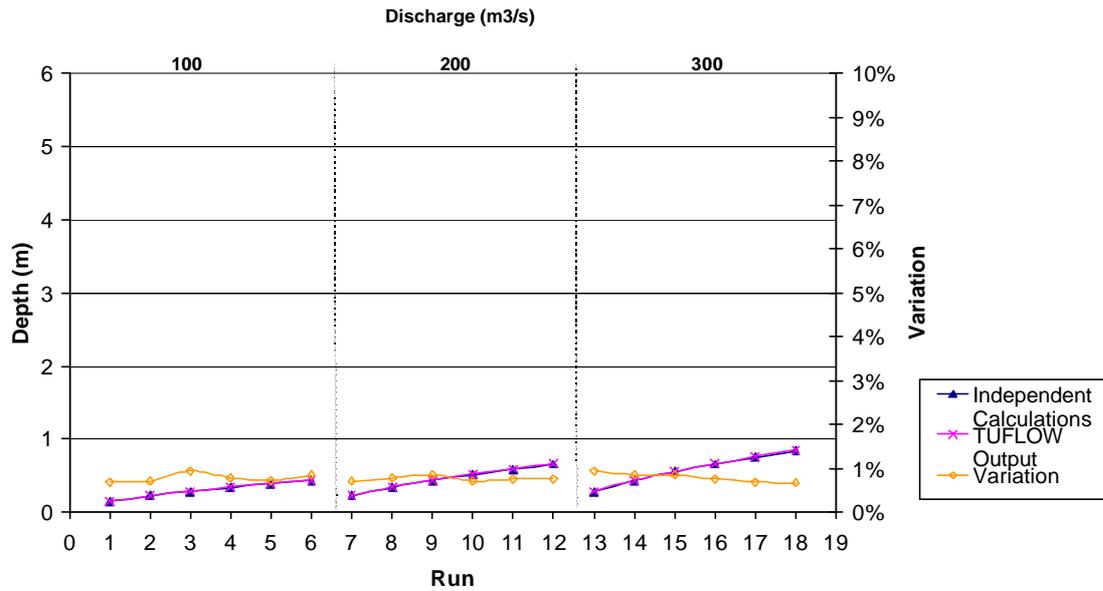


Figure 39: 2D Floodplain Flow Results- Slope= 10.4%

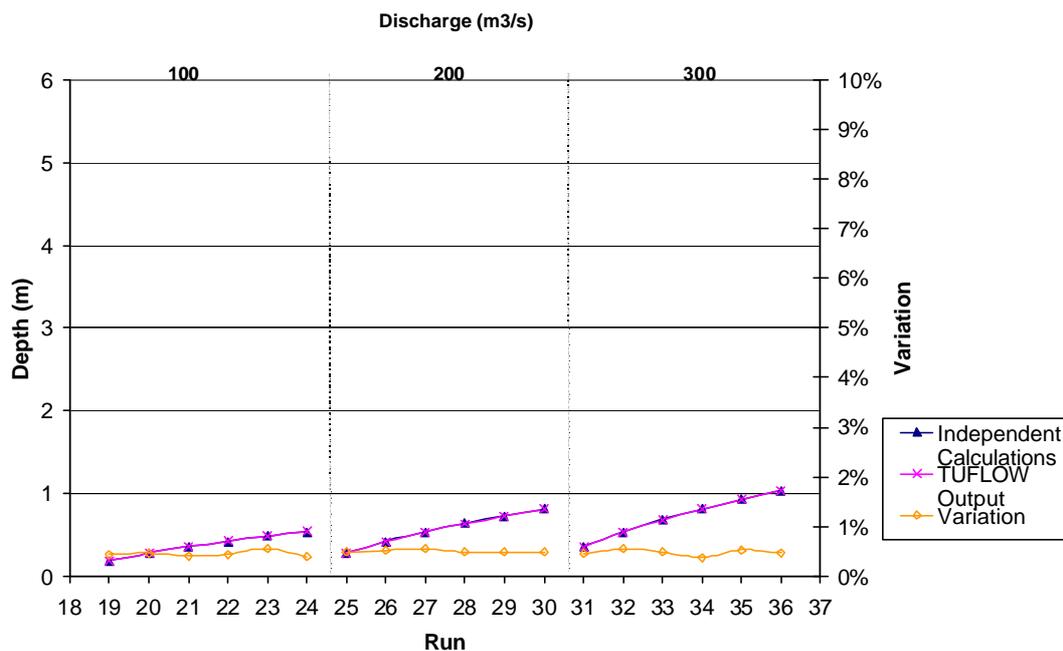


Figure 40: 2D Floodplain Flow Results- Slope= 5.2%

Supercritical flow = Runs 1-3, 7-10, 13-16, 19-20, 25-26, 31-33

Critical flow = Runs 4, 11, 17, 21, 27

Subcritical flow = Runs 5-6, 12, 18-19, 22-24, 28-30, 34-36

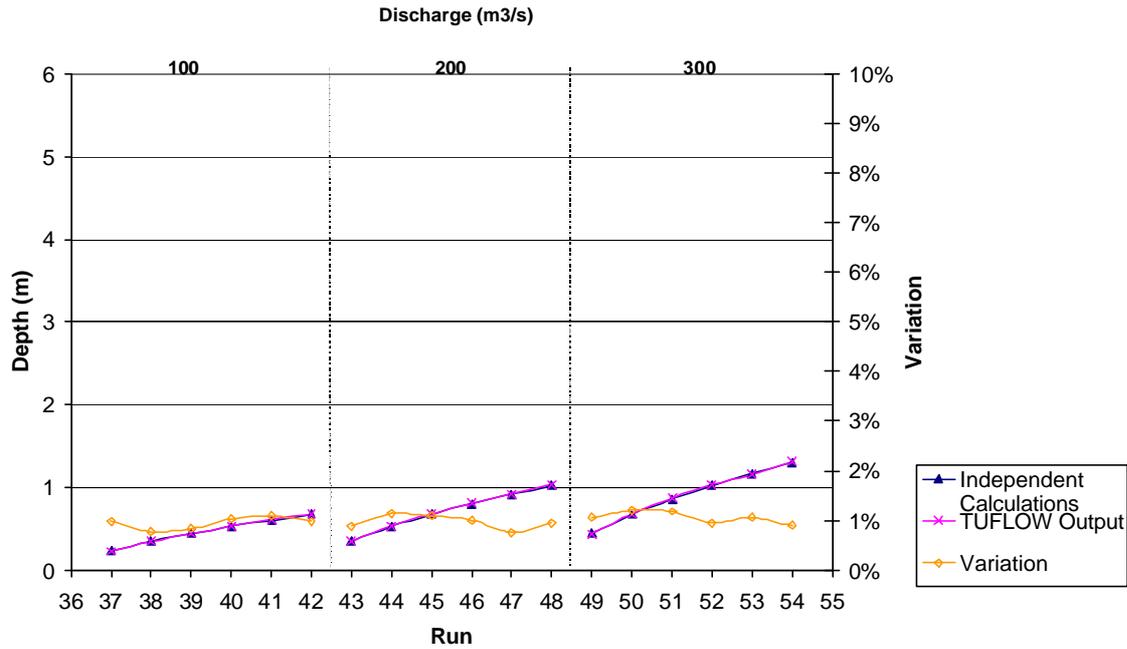


Figure 41: 2D Floodplain Flow Results – Slope = 2.6%

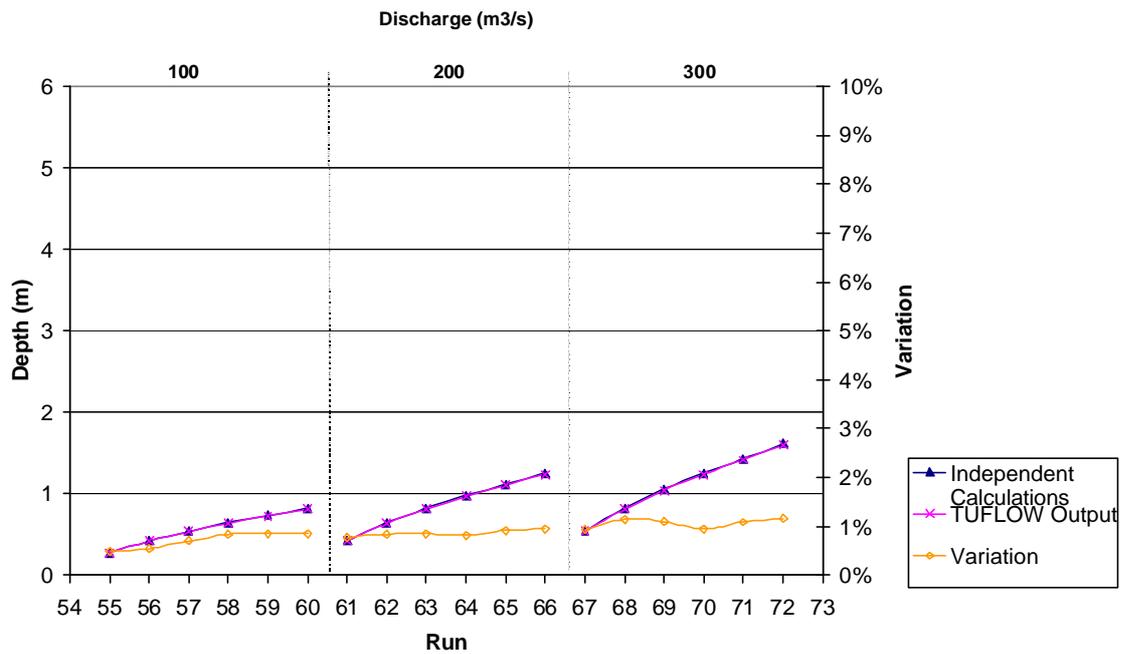


Figure 42: 2D Floodplain Flow Results – Slope = 1.3%

Supercritical flow = Runs 37, 43-44, 49-50, 55, 61, 67

Critical flow = Run 38

Subcritical flow = Runs 39-42, 45-48, 51-54, 56-60, 62-66, 68-72

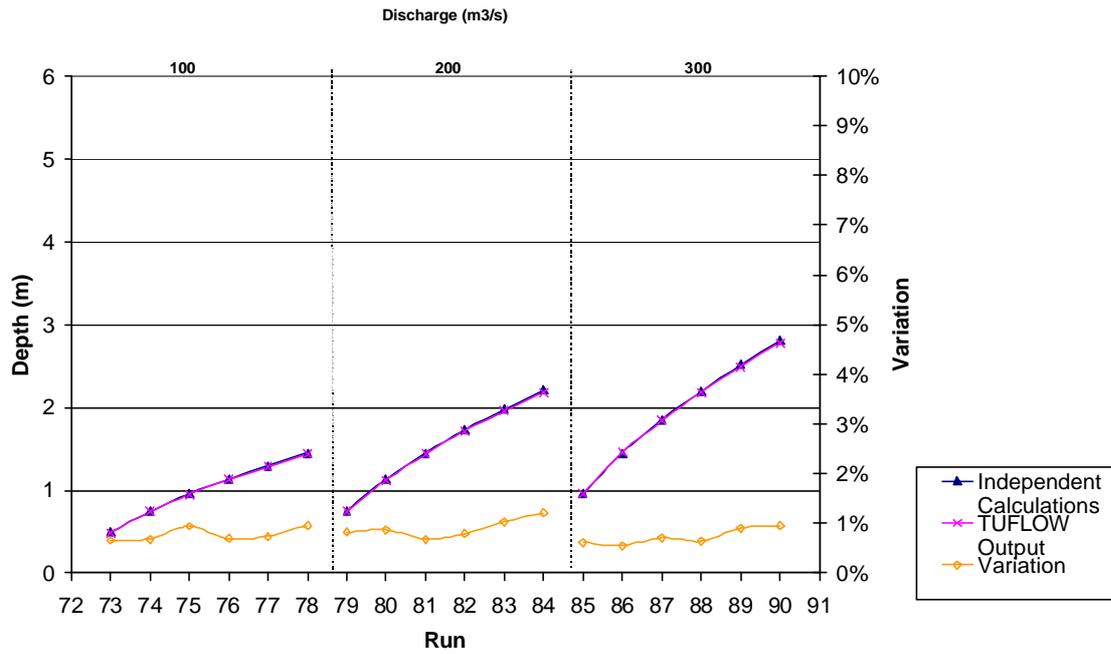


Figure 43: 2D Floodplain Flow Results – Slope = 0.2%

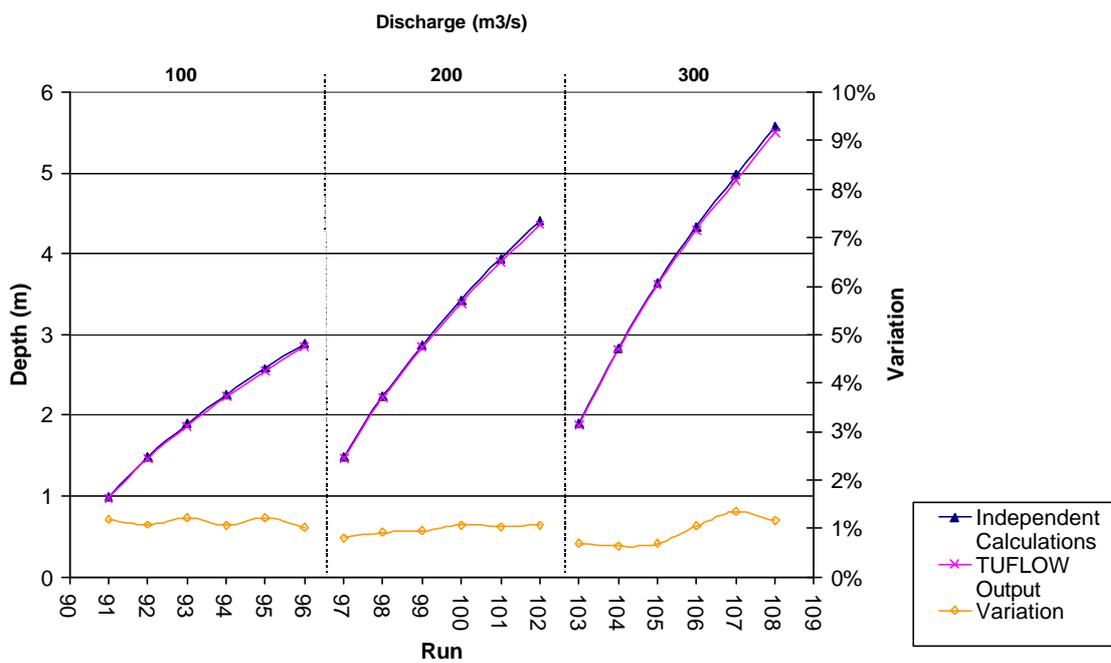
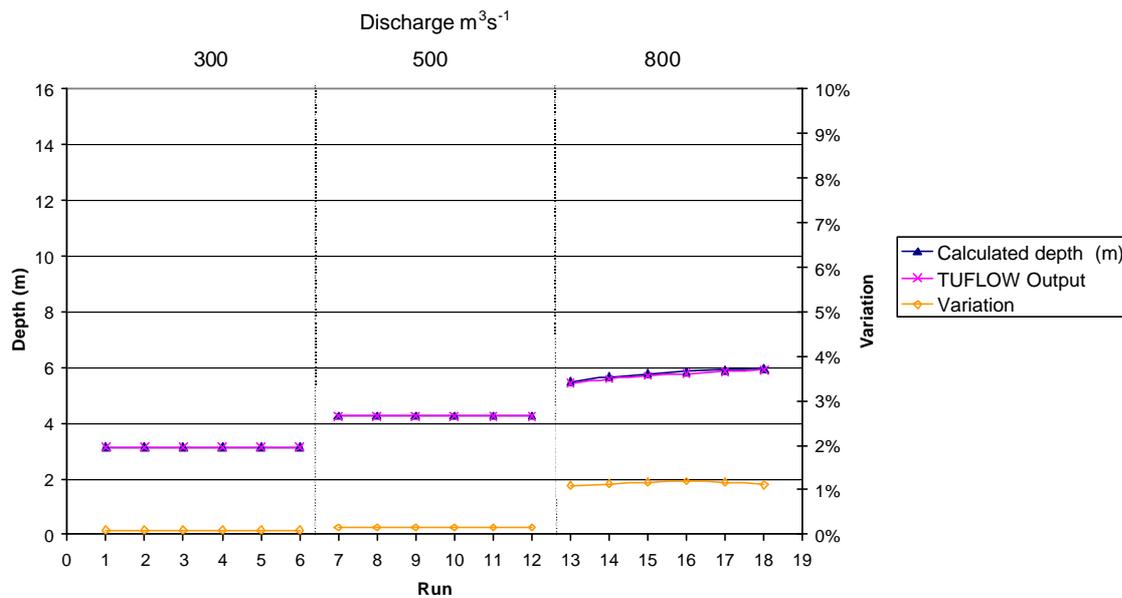


Figure 44: 2D Floodplain Flow Results – Slope = 0.02%

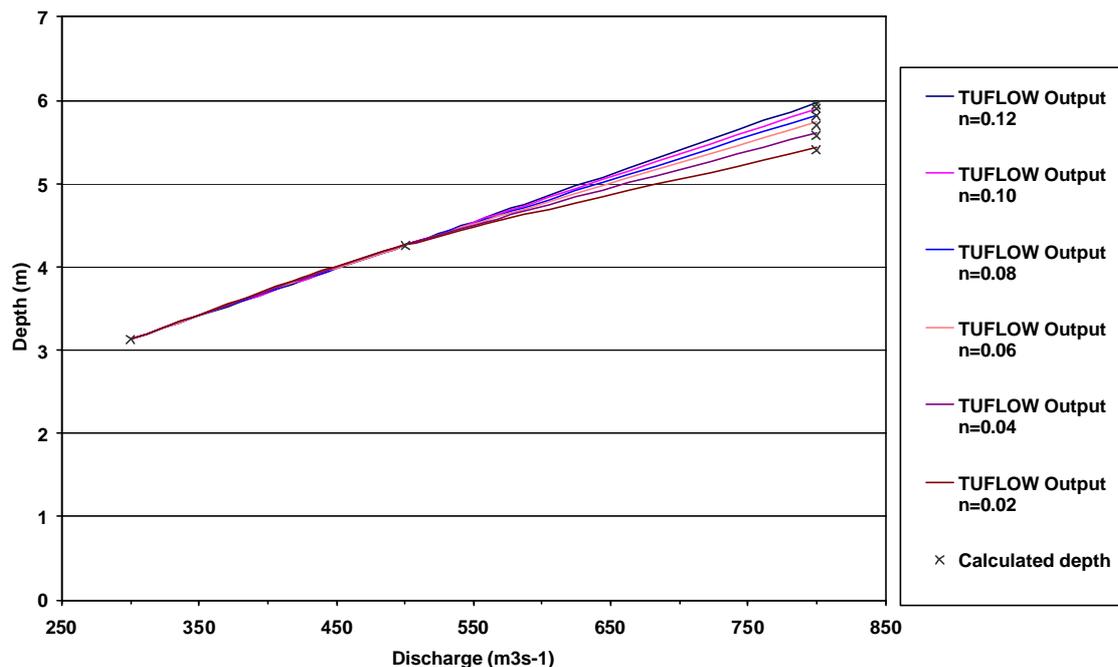
Subcritical flow = Runs 73-108

### 5.3.2.2 2D Floodplain Flow/ 2D Channel Flow

Runs 1-18 were used to test the transition from 2D channel flow, to 2D floodplain/ 2D channel flow. Runs 1-12 were designed to use a flow rate insufficient to require floodplain flow. Runs 13-18, however, were assigned a flow rate large enough to require floodplain flow. All runs were designed to reach steady state. The results are presented in Figure 45. Figure 46 shows the results of the non-steady state testing.

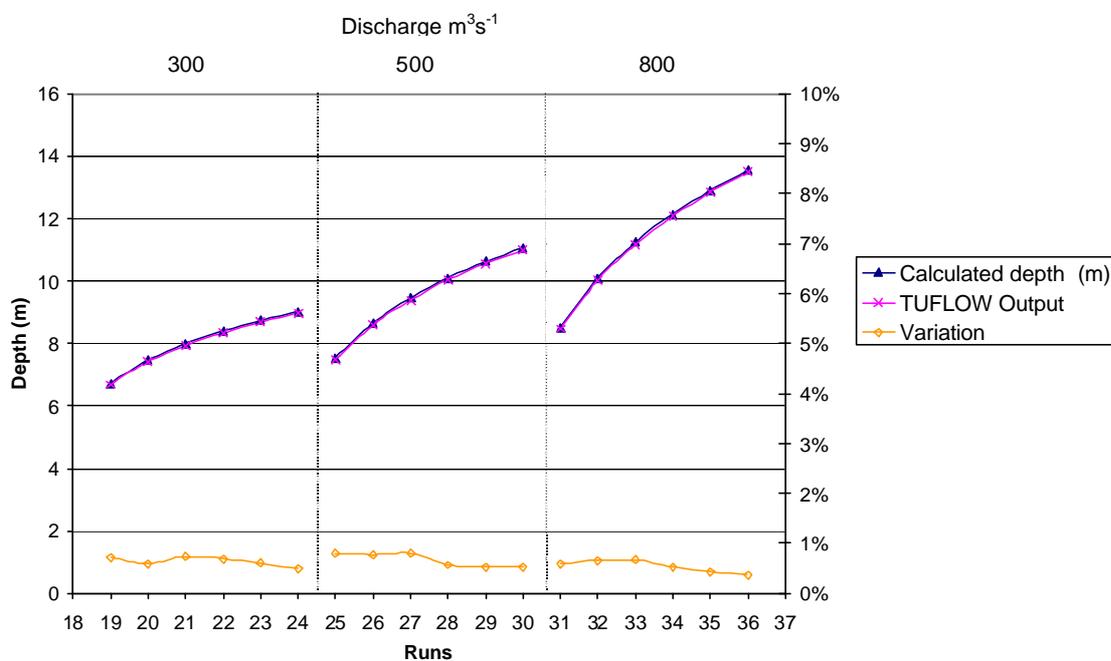


**Figure 45: 2D Floodplain/2D Channel Flow Results – Slope = 1.3% (Steady State)**



**Figure 46: 2D Floodplain/2D Channel Flow Results – Slope = 1.3% (Non-Uniform Flow)**

Runs 19-36 were used to test compound channel flow for a very small bed slope, typical of a coastal floodplain/river. The results are presented in Figure 47. Figure 48 illustrates the results of the non-uniform flow testing.



**Figure 47: 2D Floodplain/2D Channel Flow Results – Slope = 0.02% (Steady State)**

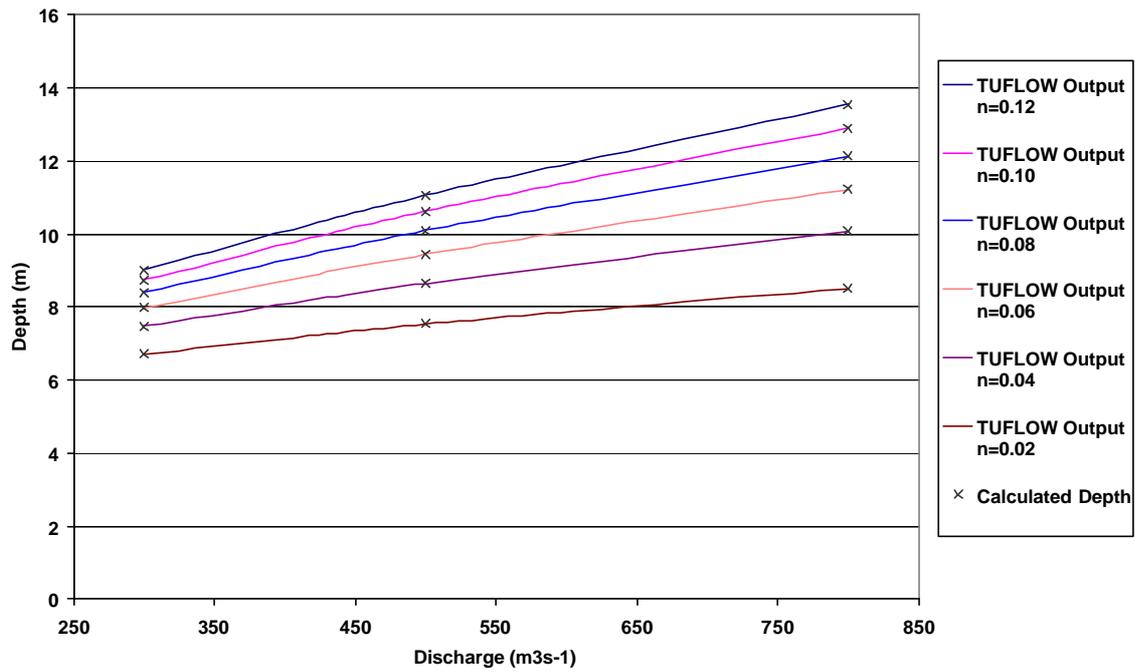


Figure 48: 2D Floodplain/2D Channel Flow Results – Slope = 0.02% (Non-Uniform Flow)

### 5.3.2.3 2D Floodplain Flow/ 1D Channel Flow

Runs 1-18 were used to test the 2D floodplain/ 1D channel flow. Runs 1-12 were designed to use a flow rate insufficient to require floodplain flow, as such the flow is confined to the channel. Runs 13-18, were assigned a flow rate great enough to require floodplain flow. All runs were designed for uniform flow conditions. The results are presented in Figure 45. Figure 46 shows the results of the non-uniform testing. During the non-uniform flow testing the upstream discharge ranged from  $300\text{m}^3\text{s}^{-1}$  to  $800\text{m}^3\text{s}^{-1}$  over a 3 hour period.

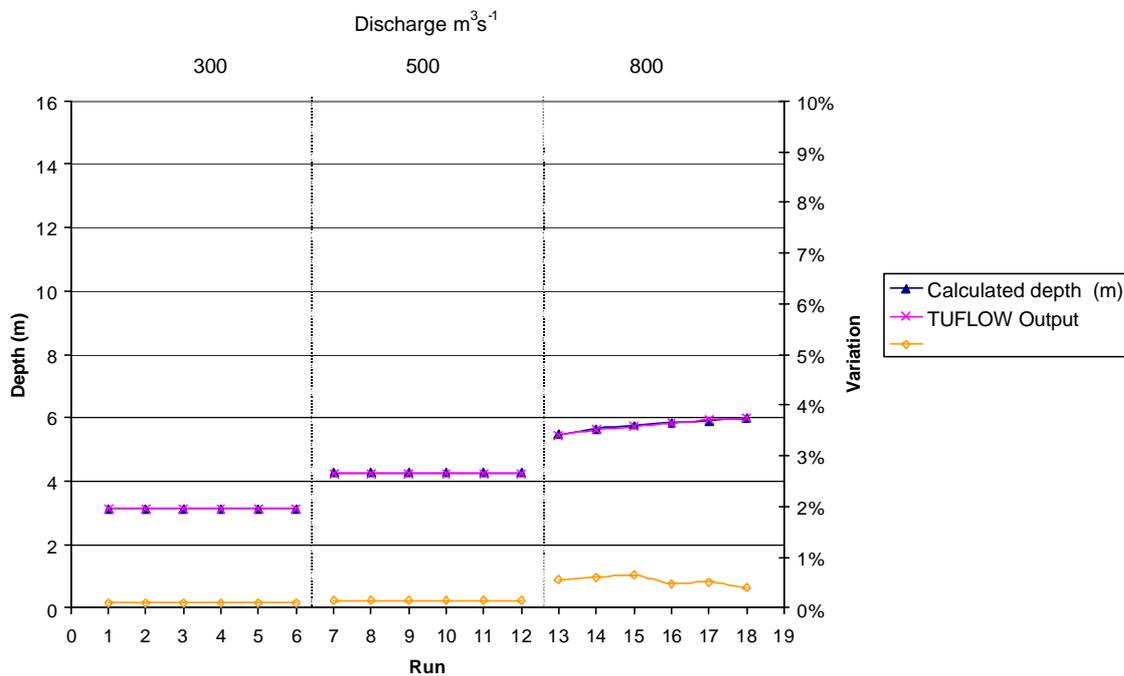
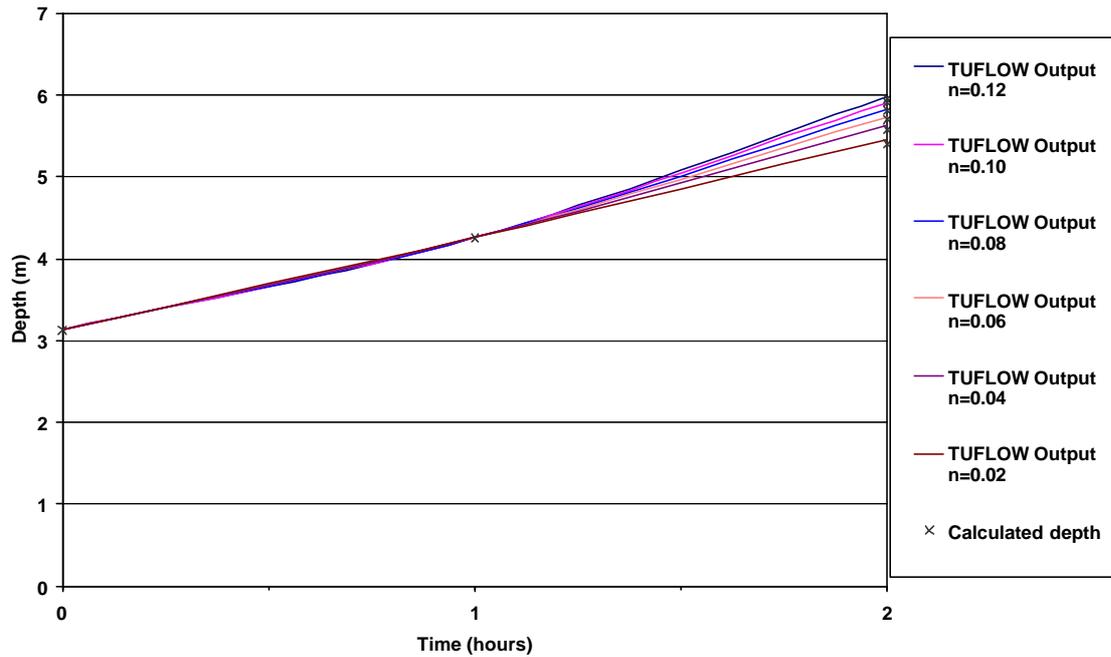


Figure 49: 2D Floodplain/1D Channel Flow Results – Slope = 1.3% (Steady State)



**Figure 50: 2D Floodplain/2D Channel Flow Results – Slope = 1.3% (Non-Uniform Flow)**

Runs 19-36 were used to test compound channel flow for a very small bed slope, typical of a coastal floodplain/river. The results are presented in Figure 47. Figure 48 illustrates the results of the non-uniform flow testing.

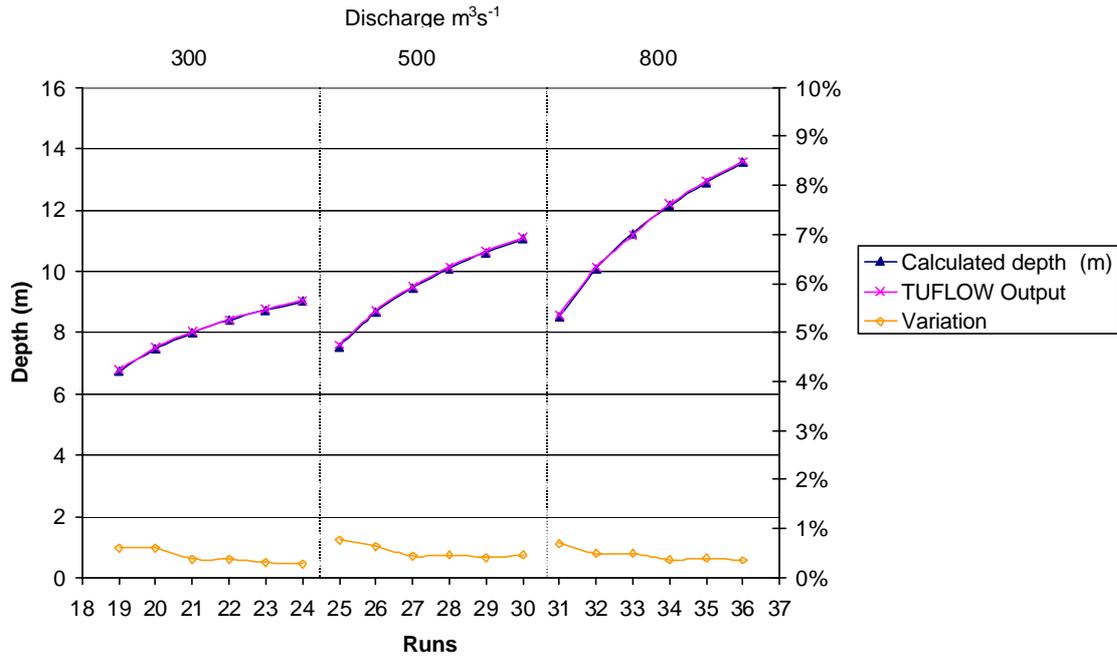


Figure 51: 2D Floodplain/1D Channel Flow Results – Slope = 0.02% (Steady State)

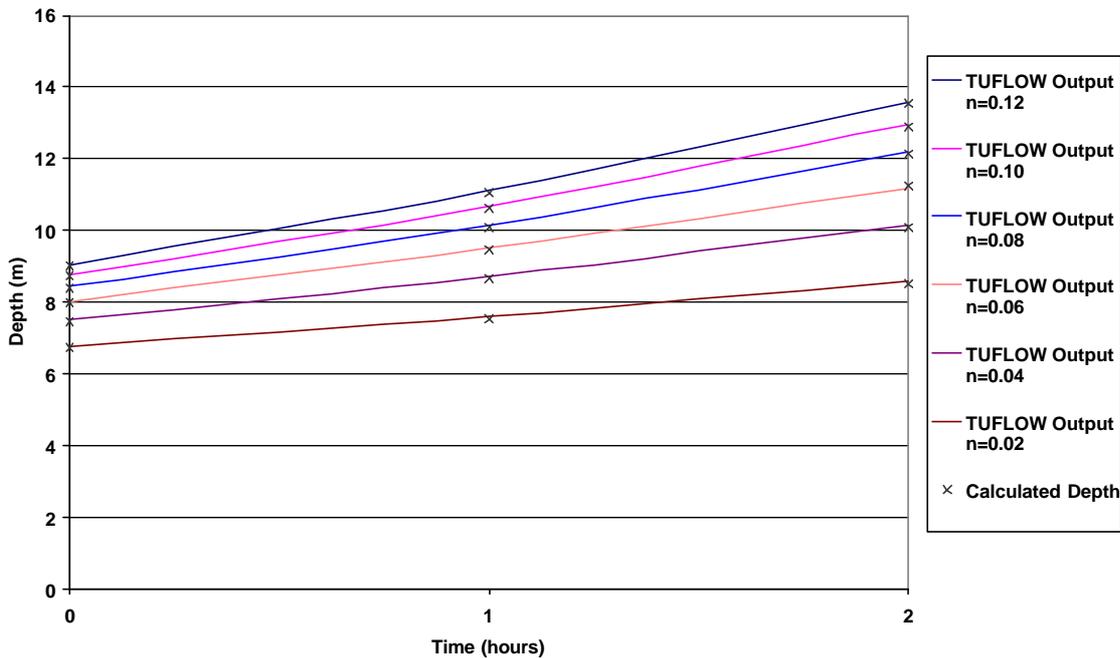


Figure 52: 2D Floodplain/1D Channel Flow Results – Slope = 0.02% (Non-Uniform Flow)

### 5.3.3 Discussion

As defined by the Froude number, open channel flow can be described by three flow regimes, subcritical, critical and supercritical flow. These flow regimes were tested during the floodplain and channel testing. Based on the modelling structure utilised by TUFLOW, compound channel flow was tested using steady state and dynamic boundary conditions. The deeper main channel was represented in both 1D and 2D.

Studies have shown that various implicit solution schemes for the 2D SWE experience problems when used for hydraulic calculations during supercritical flow (Kutijar & Hewett, p2, 2002). For this reason, it was considered important to test open channel flow for supercritical flow. To achieve this, floodplains and channels with steep bed slopes were used. It should be noted, however, the geometry of the test models used during the testing did not model the representation of hydraulic situations such as hydraulic jumps or the surcharging of water against obstructions. As these situations are characterised by 3D localised effects, further study is required to assess TUFLOW's capabilities to model these situations.

The results of the 2D floodplain and 2D channel testing show that the solution scheme utilised by the TUFLOW program operates accurately for all flow conditions.

The calculated depths using the Manning equation the 2D channel results showed a variation in depth ranging from 0.48% to 0.00%. The 2D floodplain testing showed a variation in water depth ranging from 1.35% to 0.27%.

Compound channel configurations were tested using uniform and non-uniform boundary conditions. In both cases steady state calculations were used to check the validity of the TUFLOW output results. For the non-uniform flow conditions these were calculated at hourly intervals for specified upstream discharges. Results show TUFLOW represents compound channel flow correctly. Overall the results from the fully 2D model and the 1D/2D model were comparable. The fully 2D model produced water depth results with a variation ranging from 1.201% to 0.097%. The 1D/2D models in comparison produced a variation between 0.779% and 0.92%.

The TUFLOW results produce depth estimations with a variation of less than 1.5% from independently calculated depths using theory based on the Manning equation. Overall, the greatest variation in depth was found to be 0.06m for the open channel flow test cases. The testing shows that TUFLOW represents open flow correctly for the test cases examined.

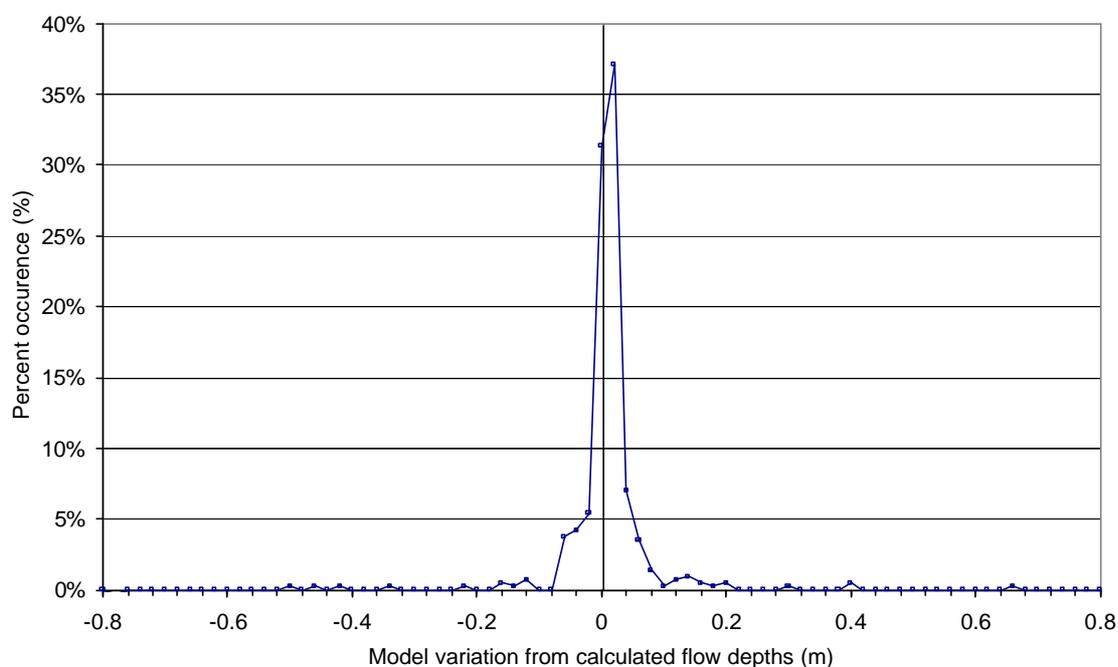
## 5.4 TUFLOW Performance

### 5.4.1 Computational Procedure

The combined data for culvert, weir and open channel flow was pooled to assess the overall performance of the TUFLOW program for the cases considered as part of this study.

### 5.4.2 Results

The combined data results, cumulatively presenting the total variation in flow depth, is shown in Figure 53, a summary of these results is also presented in Table 19

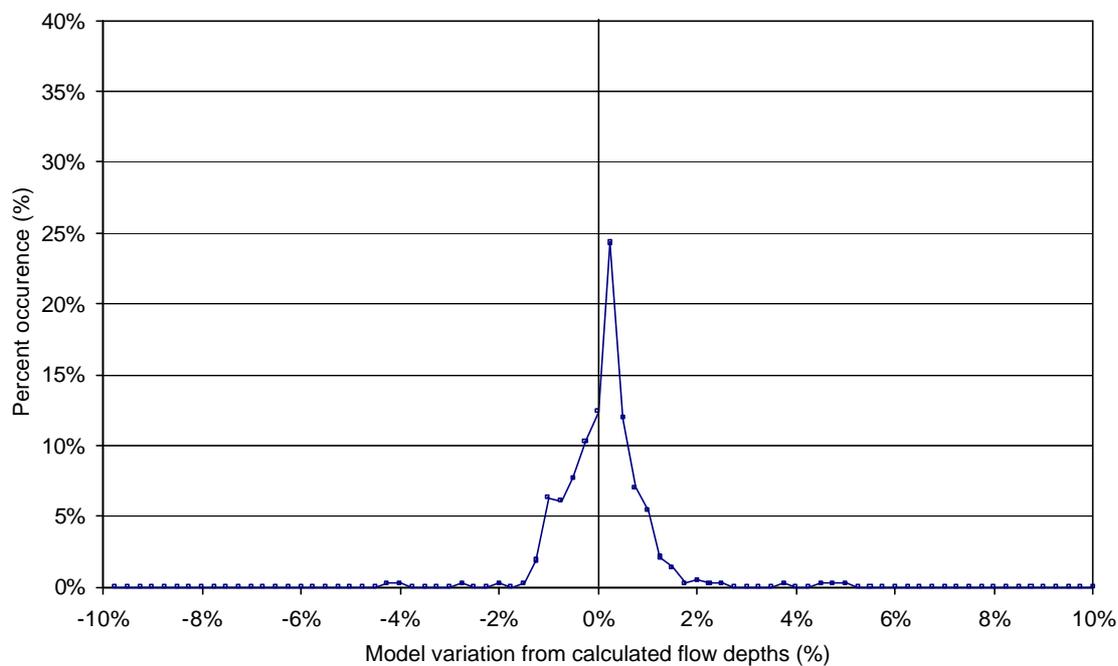


**Figure 53: TUFLOW Performance – Variation in Flow Depth**

**Table 19: TUFLOW Performance – Variation in Flow Depth (Summary)**

Variation in Flow Depth (m)	Percent occurrence (%)
Less than $\pm 0.05\text{m}$	88.3%
Less than $\pm 0.10\text{m}$	94.7%
Less than $\pm 0.15\text{m}$	96.3%
Less than $\pm 0.20\text{m}$	97.9%

The combined data results for variation in flow depth, as a function of percent variation, is graphically shown in figure 54, a summary of the results is also presented in Table 20. The percent variation is calculated using the variation in flow depth relative to the expected calculated depth.



**Figure 54: TUFLOW Performance – Percent Variation**

**Table 20: TUFLOW Performance – Percent Variation (Summary)**

Variation in Flow Depth (%)	Percent occurrence (%)
Less than $\pm 0.5\%$	66.4%
Less than $\pm 1.0\%$	91.1%
Less than $\pm 1.5\%$	96.7%
Less than $\pm 2.0\%$	97.2%

### 5.4.3 Discussion

The absolute variation of the TUFLOW results is represented by the variation in depth, measured in metres. Meanwhile, the relative variation of the TUFLOW results is presented as variation in depth, measured by percent. The culmination of the test results show TUFLOW is producing accurate results for the cases considered as part on this study.

The results show that approximately 98% of the time TUFLOW produced results within  $\pm 0.2\text{m}$  of the predicted water depth values, estimated using current engineering theories and principles. Relative to the estimated water depth, TUFLOW produced results within  $\pm 2\%$  of the estimated depth approximately 97% of the time.

## 6 CONCLUSIONS

A summary of the conclusions made, based on the results of this study are provided below;

On the basis of this study TUFLOW has been found to model culvert flow, weir flow and open channel flow accurately.

- The Culvert flow results showed the computed headwater depths calculated by ESTRY to have a maximum variation of 5%, which is equivalent to 0.4m, when compared against calculated headwater depths based on culvert nomographs produced by the CPAA (1991). The average variation was found to be 1.69% or, in terms of flow depth 0.14m. Due to inaccuracies in the culvert nomograph method these are considered acceptable results. The testing proved that ESTRY can estimate culvert headwater depths for inlet and outlet controlled situations.
- The weir flow results showed that ESTRY and TUFLOW accurately predict upstream water depths for submerged and non-submerged weir flow. The maximum variation of 0.66%, in absolute terms, 0.04m, was calculated during a comparison between the upstream depth computed by TUFLOW, and calculation based on the specific energy equation. The average variation in results was found to be 0.2%, which is equal to 0.014m. The testing validates TUFLOW and ESTRY's capabilities to estimate upstream depth for weir flow, during submerged and non-submerged flow.
- Water depths are accurately estimated by TUFLOW for open channel flow. The results show the computed water depths calculated by TUFLOW to have a maximum variation of 1.35%, or 0.06m, compared with water depth estimations based on the Manning's equation. The average depth variation was calculated to be 0.15%, which is also equal to 0.015m

Overall the results show that

- The Stelling solution scheme utilised by the TUFLOW program solves the 2D SWE, which mathematically represents the unsteady free surface flow of long waves, accurately. This implies, and the testing proves, that TUFLOW estimates hydraulic process accurately.
- The hydraulic structure equations used by ESTRY replicate hydraulic processes accurately.

This study has found no areas where the TUFLOW program could possibly be further developed or improved, increasing the accuracy of the programs hydrodynamic/ hydraulic flow estimation capabilities.

## 7 RECOMMENDATIONS

To complete the testing/validation of the TUFLOW program it is recommended that further testing is required. Listed below are the various tests that are recommended for further testing/ validation of the TUFLOW program.

- The testing of the interaction of fluid flow between hydraulic features. Such as, open channel flow combined with culvert flow.
- Global sensitivity analysis of all features modelled by the TUFLOW program.

## APPENDIX A: CULVERT FLOW

### Results

**Table 21: Pipe Culvert - Regime Results**

Run	ESTRY Regime	ESTRY Headwater Depth Output (m)	Nomograph Headwater Depth Calculation (m)	Headwater Difference (m)	Variation (%)
1	A	8.541	8.513	0.029	0.33%
2	B	11.873	12.300	-0.427	3.60%
3	C	8.164	8.013	0.152	1.86%
4	D	8.010	8.000	0.010	0.12%
5	E	9.346	8.950	0.396	4.24%
6	F	11.366	11.600	-0.234	2.06%
7	H	9.167	8.750	0.642	4.55%
8	J	8.090	7.805	0.285	3.52%
9	K	11.870	12.400	-0.530	4.77%
10	L	8.541	8.513	0.029	0.33%

**Table 22: Pipe Culvert - Variable Results**

Run	ESTRY Headwater Depth Output (m)	Nomograph Headwater Depth Calculation (m)	Headwater Difference (m)	Variation (%)
11	9.454	9.58	0.126	1.28%
12	9.454	9.58	0.126	1.28%
13	9.454	9.58	0.126	1.28%
14	9.454	9.49	0.036	0.33%
15	9.454	9.61	0.156	1.60%
16	11.933	12.28	0.347	2.87%
17	9.257	9.28	0.023	0.26%
18	8.971	8.79	-0.181	2.05%
19	9.544	9.450	-0.094	0.98%
20	9.454	9.350	-0.104	1.1%

Table 23: Box Culvert Regime Test Results

Run	ESTRY Regime	ESTRY Headwater Depth Output (m)	Nomograph Headwater Depth Calculation (m)	Headwater Difference (m)	Variation (%)
21	A	8.82	8.85	0.03	0.34%
22	B	12.08	12.25	0.17	1.39%
23	C	8.01	7.86	-0.15	1.87%
24	D	8.01	8.00	-0.01	0.11%
25	E	9.02	8.95	-0.07	0.79%
26	F	10.54	11.00	0.46	4.41%
27	H	8.84	8.45	-0.39	4.38%
28	J	7.34	7.17	-0.17	2.32%
29	K	11.85	12.03	0.18	1.46%
30	L	12.08	12.25	0.17	1.39%

Table 24: Rectangular Culvert Variable Test Results

Run	ESTRY Headwater Depth Output (m)	Nomograph Headwater Depth Calculation (m)	Headwater Difference (m)	Variation (%)
31	9.32	9.20	-0.12	1.31%
332	9.32	9.20	-0.12	1.31%
33	9.32	9.20	-0.12	1.31%
34	9.23	9.17	-0.06	0.68%
35	9.45	9.32	-0.13	1.42%
36	9.06	9.13	0.07	0.75%
37	9.86	9.88	0.02	0.16%
38	8.83	8.90	0.07	0.76%
39	9.322	9.25	-0.07	1.36%
40	9.35	9.19	-0.16	1.42%

## APPENDIX B: WEIR FLOW

### 1D Weir Results

**Table 25: 1D Non-submerged Flow – Results (Downstream Depth)**

Run	Upstream Depth Independent Calculation (m)	Upstream Depth ESTRY Output (m)	Upstream Depth Difference (m)	Variation (%)
1,5,9	5.24	5.24	0	0.04%
2,6,10	5.50	5.50	0	0.09%
3,7,11	5.70	5.71	0.01	0.13%
4,8,12	5.87	5.89	0.02	0.18%

**Table 26: 1D Non-submerged Flow – Results (Flow Length)**

Run	Upstream Depth Independent Calculation (m)	Upstream Depth ESTRY Output (m)	Upstream Depth Difference (m)	Variation (%)
13	5.24	5.24	0	0.04%
14	5.50	5.50	0	0.09%
15	5.70	5.71	0.01	0.13%
16	5.87	5.86	-0.01	0.18%
17	5.15	5.15	0	0.02%
18	5.31	5.32	0.01	0.06%
19	5.44	5.45	0.01	0.09%
20	5.55	5.56	0.01	0.11%
21	5.10	5.10	0	0.02%
22	5.20	5.20	0	0.04%
23	5.28	5.28	0	0.06%
24	5.35	5.35	0	0.06%

Table 27: 1D Submerged Flow - Results

Run	Upstream Depth Independent Calculation (m)	Upstream Depth ESTRY Output (m)	Upstream Depth Difference (m)	Variation (%)
25	5.50	5.51	0.01	0.13%
26	5.54	5.54	0	0.04%
27	5.70	5.71	0.01	0.13%
28	5.87	5.89	0.02	0.18%
29	5.50	5.51	0.01	0.13%
30	5.50	5.52	0.02	0.36%
31	5.50	5.54	0.04	0.72%
32	5.56	5.57	0.01	0.12%
33	5.50	5.50	0	0.00%
34	5.50	5.47	-0.03	0.55%
35	5.50	5.48	-0.02	0.36%
36	5.51	5.51	0	0.03%

Table 28: 1D Flow Transition - Results

Run	Inundation Ratio	Upstream Depth Independent Calculation (m)	Upstream Depth ESTRY Output (m)	Upstream Depth Difference (m)	Variation (%)
37	99.25%	5.50	5.50	0	0.05%
38	98.50%	5.50	5.51	0.01	0.13%
39	97.25%	5.51	5.52	0.01	0.09%
40	95.30%	5.53	5.53	0	0.02%
41	92.25%	5.54	5.54	0	0.04%
42	87.50%	5.56	5.57	0.01	0.13%
43	81.56%	5.61	5.61	0	0.09%
44	75.72%	5.65	5.66	0.01	0.17%
45	70.65%	5.70	5.71	0.01	0.13%
46	66.32%	5.75	5.75	0	0.16%
47	62.58%	5.79	5.80	0.01	0.17%
48	59.35%	5.83	5.84	0.01	0.16%
49	56.46%	5.87	5.89	0.02	0.18%

50	32.55%	6.51	6.54	0.03	0.40%
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## 2D Weir Results

**Table 29: 2D Non-submerged Flow – Results (Downstream Depth)**

Run	Upstream Depth Independent Calculation (m)	Upstream Depth TUFLOW Output (m)	Upstream Depth Difference (m)	Variation (%)
1	5.44	5.48	0.03	0.62%
2	5.58	5.62	0.04	0.66%
3	5.70	5.73	0.03	0.53%
4	5.44	5.47	0.03	0.53%
5	5.58	5.61	0.03	0.61%
6	5.70	5.73	0.03	0.45%
7	5.44	5.47	0.03	0.53%
8	5.58	5.61	0.03	0.58%
9	5.70	5.73	0.03	0.53%

**Table 30: 2D Non-submerged Flow – Results (Flow Length)**

Run	Upstream Depth Independent Calculation (m)	Upstream Depth TUFLOW Output (m)	Upstream Depth Difference (m)	Variation (%)
7	5.44	5.47	0.03	0.53%
8	5.58	5.61	0.03	0.58%
9	5.70	5.73	0.03	0.53%
10	5.34	5.35	0.01	0.25%
11	5.44	5.46	0.02	0.35%
12	5.53	5.55	0.02	0.29%
13	5.28	5.30	0.02	0.41%
14	5.36	5.39	0.03	0.48%
15	5.44	5.47	0.03	0.55%

Table 31: 2D Submerged Flow - Results

Run	Upstream Depth Independent Calculation (m)	Upstream Depth TUFLOW Output (m)	Upstream Depth Difference (m)	Variation (%)
16	5.50	5.50	0.00	0.00%
17	5.58	5.60	0.02	0.30%
18	5.70	5.71	0.01	0.19%
19	5.50	5.50	0.00	0.00%
20	5.50	5.50	0.00	0.00%
21	5.57	5.55	-0.02	0.34%
22	5.50	5.50	0.00	0.05%
23	5.50	5.51	0.01	0.11%
24	5.50	5.51	0.01	0.18%

Table 32: 2D Flow Transition - Results

Run	Upstream Depth Independent Calculation (m)	Upstream Depth TUFLOW Output (m)	Upstream Depth Difference (m)	Variation (%)
25	5.50	5.50	0.00	0.00%
26	5.50	5.51	0.01	0.18%
27	5.52	5.53	0.01	0.26%
28	5.54	5.55	0.01	0.11%
29	5.55	5.58	0.03	0.45%
30	5.58	5.61	0.03	0.49%
31	5.61	5.63	0.02	0.43%
32	5.63	5.65	0.02	0.43%
33	5.65	5.68	0.03	0.49%
34	5.68	5.71	0.03	0.60%
35	5.70	5.73	0.03	0.53%
36	6.11	6.13	0.02	0.37%

## APPENDIX C: OPEN CHANNEL FLOW

### 2D Channel Results

Table 33: 2D Channel Flow Results – Slope = 10.4%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
1	0.125	0.124	-0.001	0.300%
2	0.189	0.188	-0.001	0.424%
3	0.241	0.240	-0.001	0.361%
4	0.286	0.285	-0.001	0.460%
5	0.327	0.329	0.002	0.485%
6	0.365	0.364	-0.001	0.333%
7	0.189	0.189	0.000	0.211%
8	0.286	0.287	0.001	0.203%
9	0.365	0.364	-0.001	0.278%
10	0.434	0.435	0.001	0.166%
11	0.497	0.495	-0.002	0.327%
12	0.554	0.555	0.001	0.151%
13	0.241	0.240	-0.001	0.361%
14	0.365	0.365	0.000	0.087%
15	0.466	0.467	0.001	0.076%
16	0.554	0.555	0.001	0.151%
17	0.634	0.635	0.001	0.199%
18	0.707	0.705	-0.002	0.306%

Table 34: 2D Channel Flow Results – Slope = 5.2%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
19	0.153	0.154	0.001	0.439%
20	0.232	0.233	0.001	0.216%
21	0.297	0.298	0.001	0.459%
22	0.353	0.352	-0.001	0.174%
23	0.403	0.404	0.001	0.192%
24	0.450	0.452	0.002	0.458%
25	0.232	0.233	0.001	0.216%

26	0.353	0.353	0.000	0.110%
27	0.450	0.451	0.001	0.237%
28	0.535	0.536	0.001	0.209%
29	0.612	0.610	-0.002	0.276%
30	0.683	0.684	0.001	0.210%
31	0.297	0.296	-0.001	0.213%
32	0.450	0.451	0.001	0.237%
33	0.574	0.575	0.001	0.150%
34	0.683	0.685	0.002	0.356%
35	0.781	0.783	0.002	0.309%

Table 35: 2D Channel Flow Results – Slope = 2.6%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
37	0.189	0.189	0.000	0.105%
38	0.286	0.286	0.000	0.111%
39	0.365	0.366	0.001	0.159%
40	0.434	0.435	0.001	0.166%
41	0.497	0.496	-0.001	0.126%
42	0.554	0.552	-0.002	0.390%
43	0.286	0.286	0.000	0.251%
44	0.434	0.435	0.001	0.166%
45	0.554	0.553	-0.001	0.210%
46	0.659	0.660	0.001	0.180%
47	0.753	0.751	-0.002	0.321%
48	0.841	0.842	0.001	0.152%
49	0.365	0.365	0.000	0.224%
50	0.554	0.556	0.002	0.241%
51	0.707	0.709	0.002	0.259%
52	0.841	0.842	0.001	0.152%
53	0.961	0.960	-0.001	0.149%

Table 36: 2D Channel Flow Results – Slope = 1.3%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
55	0.232	0.232	0.000	0.214%
56	0.353	0.354	0.001	0.251%
57	0.450	0.449	-0.001	0.318%
58	0.535	0.535	0.000	0.022%
59	0.612	0.613	0.001	0.214%
60	0.683	0.684	0.001	0.210%
61	0.353	0.354	0.001	0.251%
62	0.535	0.536	0.001	0.209%
63	0.683	0.685	0.002	0.356%
64	0.811	0.811	0.000	0.057%
65	0.928	0.930	0.002	0.217%
66	1.035	1.035	0.000	0.047%
67	0.450	0.450	0.000	0.016%
68	0.683	0.685	0.002	0.356%
69	0.871	0.870	-0.001	0.117%
70	1.035	1.035	0.000	0.047%
71	1.184	1.185	0.001	0.076%
72	1.321	1.325	0.004	0.290%

Table 37: 2D Channel Flow Results – Slope = 0.2%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
73	0.408	0.408	0.000	0.001%
74	0.619	0.620	0.001	0.171%
75	0.790	0.790	0.000	0.021%
76	0.939	0.940	0.001	0.107%
77	1.074	1.070	-0.004	0.353%
78	1.198	1.200	0.002	0.155%
79	0.619	0.620	0.001	0.171%
80	0.939	0.940	0.001	0.107%
81	1.198	1.200	0.002	0.155%
82	1.424	1.425	0.001	0.062%

83	1.628	1.623	-0.005	0.318%
84	1.816	1.810	-0.006	0.341%
85	0.790	0.790	0.000	0.021%
86	1.198	1.200	0.002	0.155%
87	1.528	1.530	0.002	0.102%
88	1.816	1.815	-0.001	0.066%
89	2.076	2.080	0.004	0.204%
90	2.315	2.320	0.005	0.230%

**Table 38: 2D Channel Flow Results – Slope = 0.02%**

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
91	0.815	0.818	0.003	0.274%
92	1.189	1.190	0.001	0.066%
93	1.578	1.580	0.002	0.157%
94	1.874	1.880	0.006	0.297%
95	2.142	2.140	-0.002	0.101%
96	2.389	2.385	-0.004	0.150%
97	1.237	1.233	-0.004	0.296%
98	1.874	1.870	-0.004	0.236%
99	2.389	2.380	-0.009	0.360%
100	2.835	2.840	0.005	0.179%
101	3.236	3.240	0.004	0.121%
102	3.604	3.600	-0.004	0.114%
103	1.578	1.573	-0.005	0.287%
104	2.389	2.380	-0.009	0.360%
105	3.040	3.050	0.010	0.320%
106	3.604	3.600	-0.004	0.114%
107	4.109	4.100	-0.009	0.229%
108	4.572	4.580	0.008	0.181%

## Floodplain Results

**Table 39: 2D Floodplain Flow Results – Slope = 10.4%**

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
1	0.148	0.149	0.001	0.686%
2	0.224	0.226	0.002	0.715%
3	0.286	0.289	0.003	0.942%
4	0.340	0.343	0.003	0.784%
5	0.389	0.392	0.003	0.723%
6	0.434	0.438	0.004	0.855%
7	0.224	0.226	0.002	0.715%
8	0.340	0.343	0.003	0.784%
9	0.434	0.438	0.004	0.855%
10	0.516	0.520	0.004	0.712%
11	0.590	0.595	0.005	0.757%
12	0.659	0.664	0.005	0.754%
13	0.286	0.289	0.003	0.942%
14	0.434	0.438	0.004	0.855%
15	0.554	0.559	0.005	0.856%
16	0.659	0.664	0.005	0.754%
17	0.754	0.759	0.005	0.688%
18	0.841	0.847	0.006	0.672%

**Table 40: 2D Floodplain Flow Results – Slope = 5.2%**

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
19	0.353	0.354	0.001	0.403%
20	0.419	0.421	0.002	0.440%
21	0.479	0.482	0.003	0.550%
22	0.535	0.537	0.002	0.388%
23	0.276	0.275	-0.001	0.479%
24	0.419	0.417	-0.002	0.512%
25	0.535	0.532	-0.003	0.546%
26	0.636	0.633	-0.003	0.478%

27	0.728	0.724	-0.004	0.483%
28	0.812	0.808	-0.004	0.489%
29	0.353	0.351	-0.002	0.446%
30	0.535	0.532	-0.003	0.546%
31	0.683	0.680	-0.003	0.481%
32	0.812	0.809	-0.003	0.366%
33	0.929	0.924	-0.005	0.524%
34	1.037	1.032	-0.005	0.467%
35	0.229	0.227	-0.002	0.979%
36	0.348	0.343	-0.005	1.351%

Table 41: 2D Floodplain Flow Results – Slope = 3.6%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
37	0.444	0.438	-0.006	1.283%
38	0.528	0.533	0.005	1.027%
39	0.603	0.610	0.007	1.090%
40	0.673	0.680	0.007	0.977%
41	0.347	0.350	0.003	0.884%
42	0.527	0.533	0.006	1.137%
43	0.673	0.680	0.007	1.087%
44	0.800	0.808	0.008	1.005%
45	0.914	0.921	0.007	0.762%
46	1.020	1.030	0.010	0.949%
47	0.443	0.438	-0.005	1.058%
48	0.672	0.680	0.008	1.202%
49	0.858	0.868	0.010	1.183%
50	1.020	1.030	0.010	0.949%
51	1.167	1.155	-0.012	1.056%
52	1.303	1.315	0.012	0.896%
53	0.427	0.422	-0.005	1.195%
54	0.280	0.279	-0.001	0.494%

Table 42: 2D Floodplain Flow Results – Slope = 1.3%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
55	0.425	0.423	-0.002	0.546%
56	0.543	0.539	-0.004	0.703%
57	0.645	0.640	-0.005	0.844%
58	0.738	0.732	-0.006	0.851%
59	0.824	0.817	-0.007	0.852%
60	0.425	0.422	-0.003	0.781%
61	0.645	0.640	-0.005	0.844%
62	0.824	0.817	-0.007	0.852%
63	0.980	0.972	-0.008	0.826%
64	1.121	1.111	-0.010	0.924%
65	1.252	1.240	-0.012	0.949%
66	0.545	0.540	-0.005	0.933%
67	0.827	0.818	-0.009	1.145%
68	1.057	1.045	-0.012	1.107%
69	1.252	1.240	-0.012	0.949%
70	1.433	1.417	-0.016	1.091%
71	1.619	1.600	-0.019	1.160%
72	0.492	0.489	-0.003	0.659%

Table 43: 2D Floodplain Flow Results – Slope = 0.2%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
73	0.747	0.742	-0.005	0.689%
74	0.954	0.945	-0.009	0.944%
75	1.135	1.127	-0.008	0.692%
76	1.299	1.289	-0.010	0.739%
77	1.450	1.436	-0.014	0.958%
78	0.747	0.741	-0.006	0.823%
79	1.135	1.125	-0.010	0.868%
80	1.450	1.440	-0.010	0.683%
81	1.726	1.712	-0.014	0.790%
82	1.975	1.955	-0.020	1.038%

83	2.207	2.180	-0.027	1.205%
84	0.954	0.960	0.006	0.625%
85	1.450	1.458	0.008	0.556%
86	1.853	1.840	-0.013	0.716%
87	2.194	2.180	-0.014	0.643%
88	2.513	2.490	-0.023	0.904%
89	2.808	2.781	-0.027	0.947%
90	3.427	3.390	-0.037	1.071%

**Table 44: 2D Floodplain Flow Results – Slope = 0.02%**

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
91	0.988	0.976	-0.012	1.177%
92	1.481	1.465	-0.016	1.079%
93	1.893	1.870	-0.023	1.219%
94	2.254	2.230	-0.024	1.069%
95	2.582	2.550	-0.032	1.221%
96	2.885	2.855	-0.030	1.026%
97	1.481	1.469	-0.012	0.809%
98	2.241	2.220	-0.021	0.918%
99	2.867	2.840	-0.027	0.949%
100	3.427	3.390	-0.037	1.071%
101	3.941	3.900	-0.041	1.042%
102	4.407	4.360	-0.047	1.074%
103	1.893	1.880	-0.013	0.691%
104	2.838	2.820	-0.018	0.635%
105	3.635	3.610	-0.025	0.685%
106	4.336	4.290	-0.046	1.050%
107	4.973	4.906	-0.067	1.348%
108	5.565	5.500	-0.065	1.164%

## 2D Floodplain/2D Channel Results

**Table 45: 2D Floodplain/2D Channel Flow Results – Slope = 1.3%**

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
1	3.133	3.130	-0.003	0.097%
2	3.133	3.130	-0.003	0.097%
3	3.133	3.130	-0.003	0.097%
4	3.133	3.130	-0.003	0.097%
5	3.133	3.130	-0.003	0.097%
6	3.133	3.130	-0.003	0.097%
7	4.257	4.250	-0.007	0.157%
8	4.257	4.250	-0.007	0.157%
9	4.257	4.250	-0.007	0.157%
10	4.257	4.250	-0.007	0.157%
11	4.257	4.250	-0.007	0.157%
12	4.257	4.250	-0.007	0.157%
13	5.480	5.420	-0.060	1.104%
14	5.654	5.590	-0.064	1.135%
15	5.767	5.700	-0.067	1.170%
16	5.850	5.780	-0.070	1.201%
17	5.914	5.845	-0.069	1.173%
18	5.966	5.900	-0.066	1.106%

**Table 46: 2D Floodplain/2D Channel Flow Results – Slope = 0.02%**

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
19	6.728	6.680	-0.048	0.719%
20	7.465	7.420	-0.045	0.600%
21	7.989	7.930	-0.059	0.738%
22	8.398	8.340	-0.058	0.693%
23	8.733	8.680	-0.053	0.603%
24	9.014	8.970	-0.044	0.488%
25	7.541	7.480	-0.061	0.807%
26	8.654	8.587	-0.067	0.773%

27	9.458	9.382	-0.076	0.803%
28	10.092	10.035	-0.057	0.569%
29	10.616	10.560	-0.056	0.524%
30	11.059	11.000	-0.059	0.534%
31	8.520	8.470	-0.050	0.589%
32	10.090	10.024	-0.066	0.657%
33	11.235	11.160	-0.075	0.669%
34	12.145	12.080	-0.065	0.534%
35	12.899	12.843	-0.056	0.436%
36	13.542	13.491	-0.051	0.374%

## 2D Flooplain/1D Channel Results

Table 47: 2D Floodplain/1D Channel Flow Results – Slope = 1.3%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
1	3.133	3.130	-0.003	0.097%
2	3.133	3.130	-0.003	0.097%
3	3.133	3.130	-0.003	0.097%
4	3.133	3.130	-0.003	0.097%
5	3.133	3.130	-0.003	0.097%
6	3.133	3.130	-0.003	0.097%
7	4.257	4.250	-0.007	0.157%
8	4.257	4.250	-0.007	0.157%
9	4.257	4.250	-0.007	0.157%
10	4.257	4.250	-0.007	0.157%
11	4.257	4.250	-0.007	0.157%
12	4.257	4.250	-0.007	0.157%
13	5.480	5.450	-0.030	0.556%
14	5.654	5.620	-0.034	0.604%
15	5.767	5.730	-0.037	0.650%
16	5.850	5.822	-0.028	0.483%
17	5.914	5.945	0.031	0.515%
18	5.966	5.990	0.024	0.401%

Table 48: 2D Floodplain/1D Channel Flow Results – Slope = 0.02%

Run	Depth Independent Calculations (m)	Depth TUFLOW Output (m)	Depth Difference (m)	Variation (%)
19	6.728	6.770	0.042	0.615%
20	7.465	7.510	0.045	0.602%
21	7.989	8.020	0.031	0.387%
22	8.398	8.430	0.032	0.377%
23	8.733	8.760	0.027	0.312%
24	9.014	9.040	0.026	0.288%
25	7.541	7.600	0.059	0.779%
26	8.654	8.710	0.056	0.644%
27	9.458	9.500	0.042	0.443%
28	10.092	10.140	0.048	0.469%
29	10.616	10.660	0.044	0.416%
30	11.059	11.110	0.051	0.459%
31	8.520	8.580	0.060	0.697%
32	10.090	10.140	0.050	0.490%
33	11.235	11.180	-0.055	0.491%
34	12.145	12.190	0.045	0.370%
35	12.899	12.950	0.051	0.391%
36	13.542	13.590	0.048	0.356%

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