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MEng (Hons) in Civil Engineering

The Re-naturalisation of the River Adur on Knepp Castle Estate

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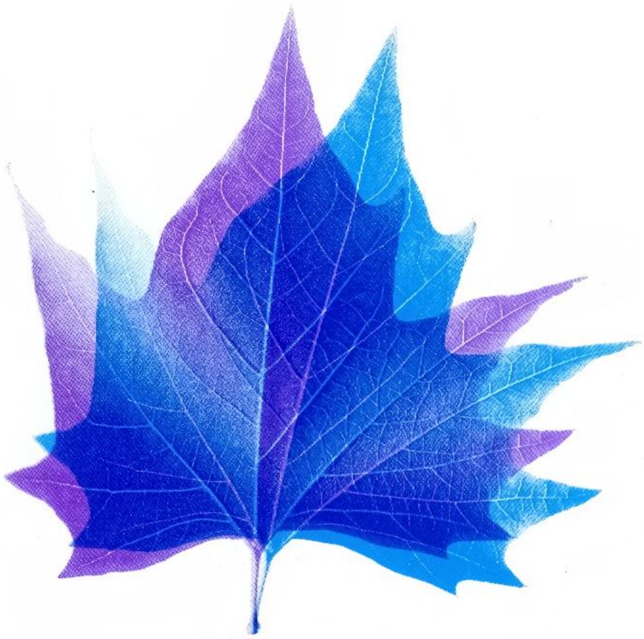

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Abstract

The following is an investigation into the feasibility of the proposed re-naturalisation scheme of the Knepp Castle Estate with specific emphasis on the alteration of the River Adur channel. The question of feasibility is considerably broad and so this report looks specifically at the impact of this proposal on the likelihood and severity of flooding. Flooding can have detrimental impacts on both the natural and manmade environment and these must be approximated beforehand. Particular consideration has been given for the A24, a main road that the river passes under through a culvert. The implications of flooding are quite serious as it could cause structural damage or submersion of the road resulting in the closure of this important transport link. With the utilisation of scale physical modelling, the impacts of creating meanders in the existing canalised channel have been explored. Readings for water depth were measured in conjunction with photographic evidence being taken of water behaviour. The resulting data and information was analysed to identify general trends which would suggest the implementation of meanders will cause a significant rise in water levels. A critical evaluation of the results gathered and overall methodology highlighted the limitations of the scale modelling approach. This has led to the suggestion of further research areas and recommendations for the improvement of the model. The main conclusion of this investigation is that the proposed re-naturalisation scheme will increase the likelihood of flooding along this stretch of the River Adur. There would subsequently be a negative impact on the natural environment surrounding the river. The worst case scenario considered in this report, a 1 in 100 year flood event, will not cause the A24 to become flooded. Even so, the increased frequency and severity of flooding may cause structural damage to the culvert over time, having a subsequent effect on the road.

1.0 Introduction

The nature of this project was to investigate the feasibility of the proposed re-naturalisation of the River Adur section that flows through the Knepp Castle Estate. Feasibility, in this report, does not refer to the cost benefits of such a development but rather the influence on water processes. More specifically, this report outlines the impacts of the new river channel design on the frequency and severity of flooding. While there can be positive effects of natural flooding on the surrounding environment, it is the close proximity to a major road (A24) that has given grounds for concern. In accordance to existing plans for a prototype channel, a scale model was designed and constructed. Observations and measurements taken in experiments have been interpreted and analysed to give indication of the real life implications of the new channel. In the light of a critical review process, the limitations of the project have been discussed and further research areas suggested. The main conclusion answers the question of feasibility for this development. In addition to this, the process of physical modelling has led to other valuable findings and insights, as was to be expected.

The course of this project can be further broken down into specific aims and objectives. First of all, any existing information from past studies carried out on this same channel has been gathered. Historical data has given light to the range of discharge rates that the river has experienced in the past which has subsequently been utilised for modelling. The accuracy of the scale model was vital to the investigation and research to this end has also been carried out. The undertaking of multiple experiments has been important in providing a good range of results that have later been analysed. Observations of the water behaviour have been invaluable for understanding the impact of flooding on a full scale and these were recorded in the form of annotated photographs. A methodology was formed for the purpose of collecting wholly accurate results though through the analysis process, possible causes for errors have been identified. This critical appraisal has been crucial for the drawing of valid conclusions from the project as a whole.

1.1 Knepp Castle Estate (Janes, et al., 2006)

The land owner at Knepp Castle Estate has a desire to improve the state of the natural landscape and river course in order to create a more thriving environment. They wish to maximise the biodiversity and see an increase in the number of flora and fauna throughout the estate. There have also been discussions with DEFRA on the steps that need to be taken in order to enter the Adur and its floodplain into an Environmental Stewardship Scheme. This has led to submission of different options that aim to enhance habitats and increase biodiversity. The most notable of these proposals is for the restoration of the river channel to its original flow path.

At present, the channel that runs through the Knepp castle estate is oversized compared to the flows it carries. The channel itself now reflects a canal due to a number of realignments and the original plan form has been lost. There are currently large weir structures in operation at various positions that have a negative impact on the landscape and hydrology. The weirs also limit the potential for fisheries to thrive as they are on the most part impassable. There is also a high demand on the Environmental Agency to provide maintenance and desilting of the structures. One of the key issues that are driving a re-naturalisation proposal is the fact that the current channel does not encourage the growth and multiplication of wildlife. This is because there is a lack of in-channel, marginal, bank-side and floodplain habitat diversity.

While there are many good reasons for going ahead with the restoration proposal, there are also a number of constraints and important issues that must be explored beforehand. The development will increase the risk of flooding of the land surrounding the river channel. More specifically, there are several buildings located in close proximity to the water course that will become more exposed to the effects of flooding. Even though these properties belong to the owner of the estate, the risk should still be noted and mitigated against. A real cause for concern is the water levels at the A24 road bridge. The water currently passes under Bay Bridge at this point and it is crucial that water levels are not increased to the point of flooding. The restoration process may also be constrained at some points along the river where access is required to Pond Farm Cottages between Capps Bridge and Trenchford Bridge. On-going maintenance will be necessary in these locations and so the re-naturalisation proposal would have to take this into account.

1.2 Historical Data

During the initial stages of this project proposal a pre-feasibility study was undertaken and a model put together. The main aim of this model was to gain a better understanding of the amount of water flowing through the channel especially at times of flooding. It was shown that the channel is oversized at its current dimensions. The restoration of the old channel would see the capacity reduced by approximately $10m^3/s$ as an average. While this would be suitable for the average flow, there would be an increased risk of flooding in times of prolonged rainfall. The study went on to estimate the effects of a 1 in 2 year flood and a 1 in 100 year flood. The table below summarises the maximum peak flow in different locations along the channel for these two flood cases:

Table 1: Peak flow data for River Adur

Location	1 in 100 year peak flow (m^3/s)	1 in 2 year peak flow (m^3/s)
River Adur, Shipley (upstream end of site)	45	13
River Adur, Tenchford (including Lancing Brook component)	71	21
River Adur, Bay Bridge (downstream end of site)	85	25

A hydrograph was also put together based on data collected for a 1 in 100 year flood:

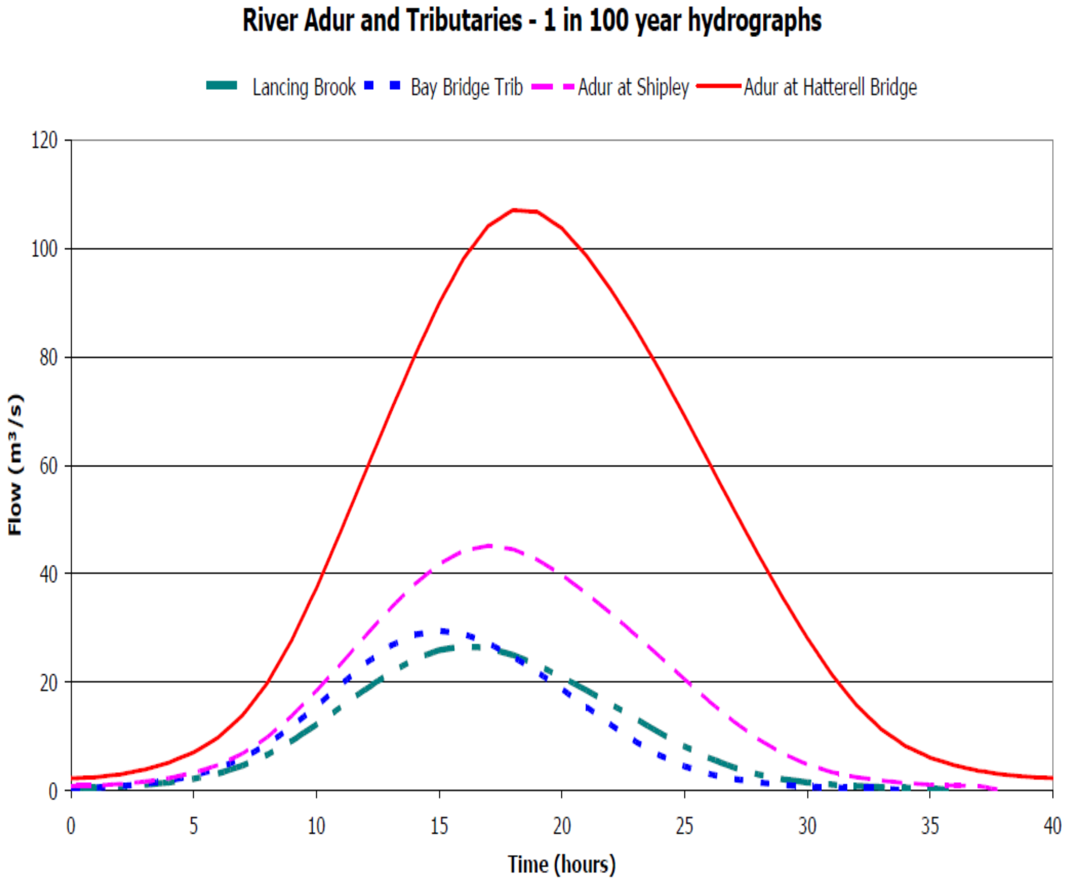


Figure 1: 1 in 100 year hydrograph for River Adur

Figure 1 shows how the flow in the different channels reacts in the event of a 1 in 100 year flood. The peaks of discharge are mostly seen between 15 and 20 hours after the event. The curves in each case are quite steep for the incline and this is also mirrored in the decline. This is caused by the geology of the area which is clay rich. This means there is little absorption of rain water into the ground but a lot of run-off that finds its way quickly into the river channels.

1.3 Scale Modelling (Novak, et al., 2001) (Peakall, et al., 1996)

Scale modelling has become increasingly more valuable to the field of hydraulics and hydraulic engineering in recent history. Significant advances were made late in the 20th century in the area of experimental methods and computational techniques leading to a more prevalent use of scale models.

There are three main types of model that engineers can adopt; namely mathematical, numerical or computational models. A mathematical model will comprise of a set of algebraic and differential equations which represent the flow. This type of model would be based upon assumptions made about the physical characteristics of the prototype environment and flow behaviour. A numerical model is only an approximation of the mathematical model and will only take into account a set number of discrete points. A computational model involves the implementation of general numerical model being applied to a specific situation. This approach requires the user to carefully analyse which general model is most suitable for the case in question as there are many to choose from. It is advantageous to use a computational model over a physical model because it is cheaper and is not subject to scale effects. That being said, this type of model requires that the physics of the case be known along with topographical data and other relevant information. There is also a risk that results are not completely accurate.

The pursuit of answers to hydraulic problems has been on-going for many centuries but it was not until the end of the 19th century that scale models were incorporated in this quest. It took a number of prominent engineers to see this experimental approach evolve and grow in credibility. In 1869 W. Froude built the first water basin model that was designed for the testing of ships. O. Reynolds also used physical modelling to analyses the tidal patterns of the Upper Mersey. At the turn of the 20th century many new laboratories were opening all over the world led by Hubert Engels in Dresden (1898) and Theodor Rehbock in Karlsruhe (1901). These two pioneering laboratories were set up to explore the area of river and hydraulic structures.

The second half of the 20th century saw an increase in the usage of mathematical techniques and computer processes in solving hydraulic problems. This has not caused a reduction in the amount of physical models being used but has adjusted the way in which they are applied. This is mainly because the size and complexity of new hydraulic schemes is increasing all the time and there are not adequate mathematical models available to apply. Physical models still provide valuable insight into physical phenomena such as the processes involved in pollutant transport, sedimentation and scour.

1.3.1 Scale Effect (Novak, et al., 2001)

In hydraulic engineering a scale model is a direct physical simulation of hydraulic phenomena that takes place in the natural or manmade environment. Models are only worthwhile if they adhere to scaling laws which ensure that a desired similarity is achieved between the model and the prototype. The ratio between a variable in the prototype and the same variable in the model is called the scale factor. It is often the case that the scale factor is not applied to every characteristic of a model and this is called distortion. By applying the scale factor to some parameters but not to others it introduces 'scale effect' into the experiment. It should be noted that distortion is often adopted into a model by design so as to highlight the action of a dominant force over less significant actions. It is vital that the modeller is aware of any distortion taking place and to what extent this affects the accuracy of the results. In particular, it should be known whether the model enhances or reduces the safety of the prototype. This requires a certain amount of knowledge and intuition which is gained through experience in modelling.

1.3.2 Basic Principles (Chanson, 2004)

When approaching physical modelling, it is of great importance to adhere to the basic theory of fluid mechanics. This will ensure the effective use of experimentation and modelling in trying to solve hydraulic problems. The theory of similarity must also be understood and incorporated in the process of forming a suitable physical model design. Figure 2 highlights some of the basic flow parameters that should be considered in physical modelling.

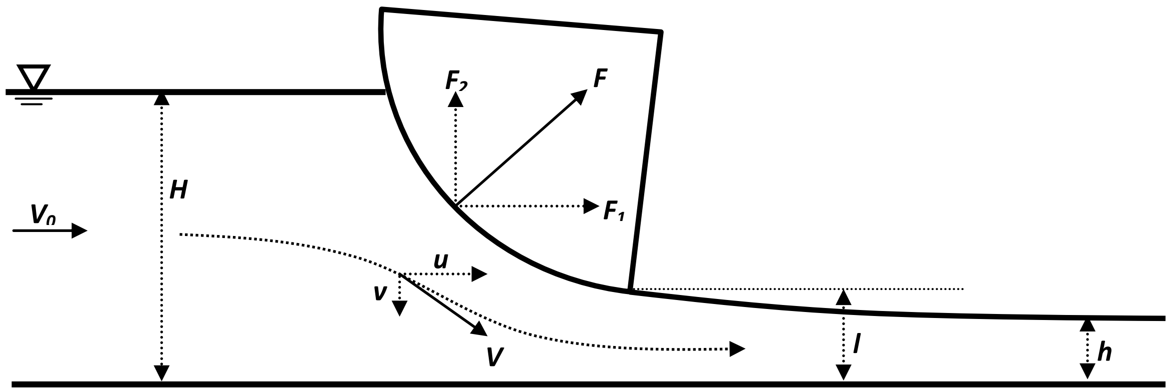


Figure 2: Basic flow parameters in physical modelling

In order to produce a valid physical model of a prototype situation, there must be continuity in three key areas. A successful model will have similarity of form (geometric similarity), similarity of motion (kinematic similarity) and similarity of forces (dynamic similarity).

Geometric similarity means that the ratio of lengths between the prototype and the model are equal:

$$L_r = \frac{l_p}{l_m} = \frac{d_p}{d_m} = \frac{H_p}{H_m} \qquad \text{Length}$$

Kinematic similarity means that the ratio of velocities between the prototype and the model are equal:

$$V_r = \frac{V_p}{V_m} = \frac{(V_1)_p}{(V_1)_m} = \frac{(V_2)_p}{(V_2)_m} \qquad \text{Velocity}$$

Dynamic similarity means that the ratio of forces between the prototype and the model are equal:

$$F_r = \frac{(F_1)_p}{(F_1)_m} = \frac{(F_2)_p}{(F_2)_m} \qquad \text{Force}$$

Subscripts *p* and *m* refer to the full size prototype and the model respectively. The subscript *r* refers to the prototype-to-model ratio.

These three scale ratios are the key ratios that help to establish other subsequent ratios:

$$M_r = \rho_r L_r^3 \qquad \text{Mass}$$

$$t_r = \frac{L_r}{V_r} \qquad \text{Time}$$

$$Q_r = V_r L_r^2 \qquad \text{Discharge}$$

$$P_r = \frac{F_r}{L_r^2} \qquad \text{Pressure}$$

1.3.3 Free-surface Flow (Chanson, 2004)

When modelling rivers or wave motion it is called free-surface modelling. In such experiments, gravity forces are the most predominant. For this reason prototype-to-model similarity is achieved with a Froude similitude:

$$Fr_p = Fr_m$$

If the gravitational acceleration is the same both in the model and the prototype it implies that:

$$V_r = \sqrt{L_r} \qquad \text{Froude Similitude}$$

When modelling rivers, a Froude similitude is appropriate as the gravity effects are predominant. What should also be considered are the viscous effects which are of a comparable magnitude. Therefore it is important to minimise these effects by creating a turbulent or fully rough flow with the same relative roughness as in the prototype:

$$Re_m > 5000$$

$$(k_s)_r = L_r$$

1.3.4 Distorted Models (Chanson, 2004)

It is possible and sometimes preferable to create a model in which the geometric scale is different between each main direction. When modelling rivers it is often the case that the scale in the horizontal direction is a lot higher than in the vertical direction. This type of distortion does not have severe effects on the flow of water and has proved to be successful in giving good results. In the case of a distorted model the following expressions are adopted:

$$\text{Horizontal} \qquad X_r > Z_r \qquad \text{Vertical}$$

$$(k_s)_r = Z_r$$

The following relationships are now slightly different than in non-distorted models:

$$V_r = \sqrt{Z_r} \qquad \text{Velocity}$$

$$t_r = \frac{X_r}{V_r} = \frac{X_r}{\sqrt{Z_r}} \qquad \text{Time}$$

$$Q_r = V_r X_r Z_r = X_r Z_r^{3/2} \qquad \text{Discharge}$$

$$(\tan\theta)_r = \frac{Z_r}{X_r} \qquad \text{Longitudinal bed slope}$$

There are a number of advantages to using a distorted model in solving hydraulic problems in the lab. The time scale is reduced while velocities and turbulence are increased compared to non-distorted models. There is a great dynamic similarity between the prototype and model because the Reynolds number is higher. The greater vertical depth also allows for a higher level of accuracy when taking depth readings. As well as all these, it is clear that distorted models provide practical and economic benefits.

The model distortion is recommended to be less than 5-10:

$$X_r / Z_r < 5 - 10$$

1.3.5 Scale Selection Process (Chanson, 2004)

1. Select the smallest horizontal-scale ratio X_r to fit within the constraints of the laboratory.
2. Determine the possible range of vertical-scale Z_r such as:
   a. The smallest-scale $(Z_r)_1$ is that which gives the limit of the discharge scaling ratio, based upon the maximum model discharge $(Q_m)_{max}$
   b. The largest-scale $(Z_r)_2$ is that which gives the feasible flow resistance coefficient (i.e. feasible f_m or $(n_{manning})_m$)
   c. Check the distortion ratio X_r/Z_r (X_r/Z_r should be less than 5-10)
3. Check the model Reynolds number Re_m for the smallest test flow rate. This might provide a new largest vertical-scale ratio $(Z_r)_3$. Check the distortion ratio X_r/Z_r
4. Select a vertical-scale ratio which satisfies: $(Z_r)_1 < Z_r < \min[(Z_r)_2, (Z_r)_3]$. If this condition cannot be satisfied, a smaller horizontal-scale ratio must be chosen.
5. Chose the convenient scales (X_r, Z_r)

1.4 Culverts (Butler, et al., 2011)

Culverts are widely used in urban drainage systems and also to enable natural watercourses to flow under roads, railways or other manmade structures. A culvert is made up of three parts called the intake or inlet; the barrel or throat; and the diffuser or outlet. There are many different cross-sectional shapes that can be incorporated into culvert design, most commonly a circle, or rectangle.

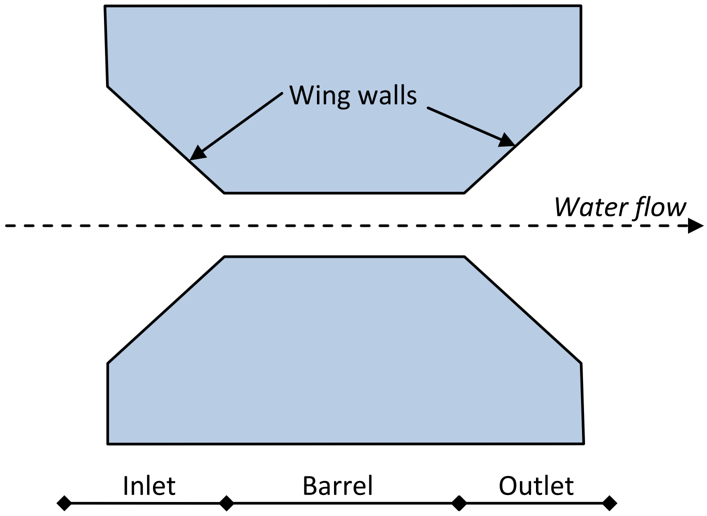
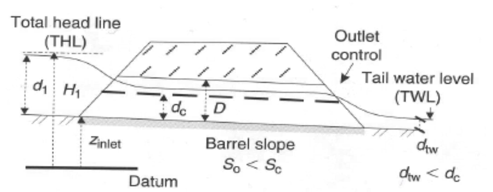
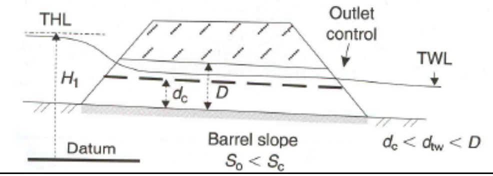
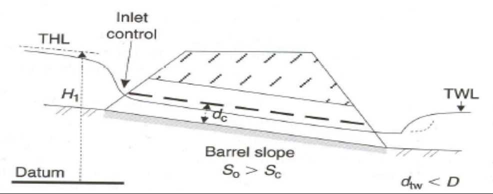
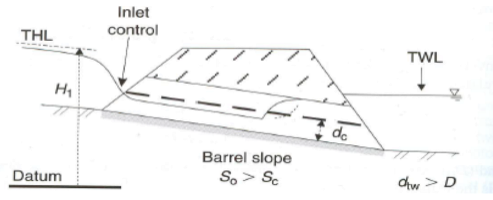
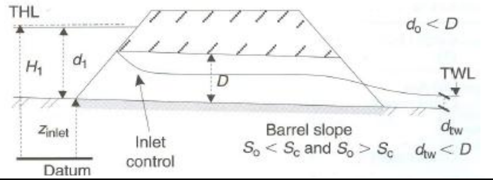
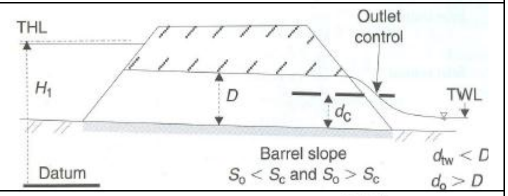
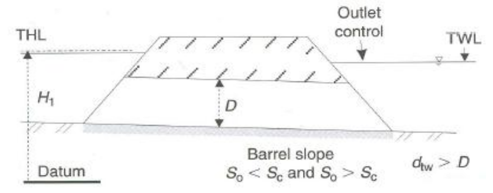
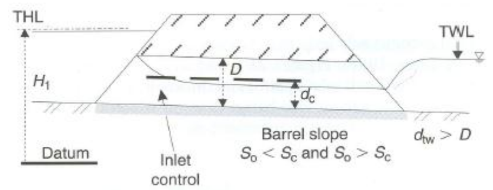


Figure 3: Basic culvert form

When it comes to understanding the hydraulic processes associated with culverts, the type of flow must be identified. There are a number of different longitudinal water surface profiles that can occur within a culvert depending on the conditions. First of all, if the water depth upstream of the culvert is less than 1.2 times the culvert height then the culvert will behave as an open channel. If the depth upstream is more than 1.2 times the culvert height then the flow-rate is likely to be limited. This is because the inlet will be behaving like an orifice or because of friction and local losses in the culvert. These two possibilities are called 'inlet-controlled' or 'losses-controlled'. Another variable in culvert conditions is whether the slope is steep or mild. Finally, if the downstream conditions have an impact on the depth of water within the culvert, known as 'downstream surcharge', then the flow of water will be altered. In the case of a 'losses-controlled' situation there is also the possibility of 'downstream surcharge'.

Table 2: Culvert flow conditions (Butler, et al., 2011) (Chanson, 2004)

Flow Condition	Control location	Remarks	Flow Conditions	Surface Profile
Free-surface inlet flow	Outlet control		$(H_1 - z_{inlet}) \leq 1.2D$ $d_{tw} < d_c$ $S_o < S_c$	
Free-surface inlet flow	Outlet control		$(H_1 - z_{inlet}) \leq 1.2D$ $d_c < d_{tw} > D$ $S_o < S_c$	
Free-surface inlet flow	Inlet control	Hydraulic jump takes place at outlet	$(H_1 - z_{inlet}) \leq 1.2D$ $d_{tw} < D$ $S_o \geq S_c$	
Free-surface inlet flow	Inlet control	Hydraulic jump takes place in the barrel	$(H_1 - z_{inlet}) \leq 1.2D$ $d_{tw} > D$ $S_o \geq S_c$	
Sub-merged entrance	Inlet control		$(H_1 - z_{inlet}) > 1.2D$ $d_{tw} < d_c$ $d_o < D$ $S_o < S_c$ or $S_o > S_c$	
Sub-merged entrance	Outlet control	Drowned barrel; critical flow depth is at outlet	$(H_1 - z_{inlet}) > 1.2D$ $d_{tw} < d_c$ $d_o > D$ $S_o < S_c$ or $S_o > S_c$	
Sub-merged entrance	Outlet control	Drowned barrel	$(H_1 - z_{inlet}) > 1.2D$ $d_{tw} > D$ $S_o < S_c$ or $S_o > S_c$	
Sub-merged entrance	Inlet control	Hydraulic jump takes place at the outlet	$(H_1 - z_{inlet}) > 1.2D$ $d_{tw} > D$ $S_o < S_c$ or $S_o > S_c$	

$D = barrel\ height\quad d_c = critical\ depth\quad d_o = uniform\ equilibrium\ depth\quad d_{tw} = tailwater\ depth$

$H_1 = total\ head\ at\ inlet\quad S_c = critical\ slope\ of\ barrel\quad S_o = barrel\ slope\quad z_{inlet} = inlet\ elevation$

2.0 Methods

2.1 Chosen Scales

Table 3: Chosen scales

Parameter	Units	Scale ratio with Froude similitude		
		Non-distorted model	Distorted model	Chosen scales
Geometric Properties				
Length	m	L_r	X_r	200
Depth	m	L_r	Z_r	50
Area	m^2	L_r^2	$X_r Z_r$	10000
Slope	degrees	-	Z_r/X_r	0.25
Kinematic Properties				
Velocity	m/s	$\sqrt{L_r}$	$\sqrt{Z_r}$	7.0710678
Discharge per unit width	m^2/s	$L_r^{3/2}$	$Z_r^{3/2}$	353.55339
Discharge	m^3/s	$L_r^{5/2}$	$Z_r^{3/2}X_r$	70710.678
Time	s	$\sqrt{L_r}$	$X_r\sqrt{Z_r}$	1414.2136
Dynamic Properties				
Force	N	$\rho_r L_r^3$	-	
Pressure	Pa	$\rho_r L_r$	$\rho_r Z_r$	50
Density	kg/m^3	ρ_r	ρ_r	1
Dynamic viscosity	Pa s	$L_r^{3/2}\sqrt{\rho_r}$	-	
Surface tension	N/m	L_r^2	-	

2.2 Experiment

The aim of the experiment was to observe the behaviour of water flowing through a scale model channel and culvert. While the entire channel could not be considered because of the limitations of the laboratory, the final stage of the water course and the culvert were modelled. The first model was made as a replica of the existing canalised channel that flows through the Knepp Estate. The second model was made to reflect the proposed design for a naturalised river channel with meanders, see Appendix V. In both cases readings of the water depth were taken at regular intervals along the channel. This gave a picture of how water depth altered at various points with particular reference to the culvert. Four different discharge rates were imposed on the models and observations were recorded of how the water behaviour changed. Any significant differences between the canalised and naturalised channels were noted and interpreted in order to evaluate the potential impact on the real scale river.

The discharge rates were chosen because of the historical data that was available, as seen in Table 1. More specifically, the figures recorded for the site near Bay Bridge were utilised for this investigation. A 1 in 2 year flood was recorded as having a discharge of 25m^3/s and a 1 in 100 year flood a discharge of 85m^3/s. For the purpose of having regular intervals between flow rates, 45m^3/s and 65m^3/s were also considered for the investigation. Table 4 gives a summary of the flow rates for the real life scale and for the model.

Table 4: Discharge rates

Prototype Discharge (m^3/s)	Model Discharge (m^3/s)	Model Discharge (l/s)	Label
25	3.5355E-04	0.354	Scenario 1
45	6.3640E-04	0.636	Scenario 2
65	9.1924E-04	0.919	Scenario 3
85	1.2021E-03	1.202	Scenario 4

2.3 Equipment and Materials

Table 5: Equipment and materials

Item	Purpose	Notes	Health & Safety
Tape measure	-Measuring out dimensions of channel profile -Making regular markings for depth readings	-Be aware of units and ensure consistency	N/A

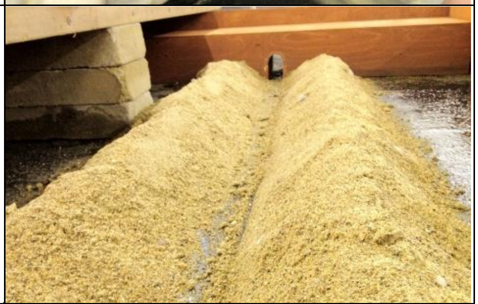
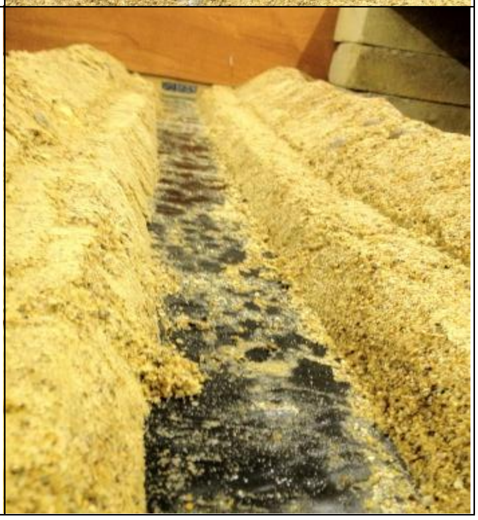
Item	Purpose	Notes	Health & Safety
Handheld shovel	-Pouring sand onto foundational stones for channel banks	-Ensure small gaps between stones are filled -Compact sand afterwards to strengthen embankments	-Be careful not to cause splinters when in contact with stones
Large shovel	-Clearing sand from experiment area -Collecting additional sand for banks	-Do not damage floor of test area with heavy contact	-Be aware of heavy lifting procedure
Trowel	-Smoothing of embankments -Compacting embankments and channel profile	-Ensure sharp corners do not pierce waterproof floor covering	-Be careful not to cause splinters when in contact with stones
Point gauge	-Taking depth readings at regular intervals along channel	-Ensure gauge is set to zero at the base of the channel	-Ensure gauge is level and secure before taking readings
Scissors	-Cutting geotextile for fitting to channel profile	-Long strips to be cut with same direction of fibres in textile for better fitting to channel	-Care to be taken while cutting
Wood saw	-Cutting wood board needed to direct water flow to start of channel	-Using pencil and tape measure, draw guidelines of cut	-Care to be taken when sawing wood -Used a secure work top for cutting on -Goggles to protect against sawdust

Item	Purpose	Notes	Health & Safety
Silicone	-Securing geotextile to surface	-Clear area before applying silicone for best outcome	-Be aware of strong fumes
Stones	-Large stones used for channel banks foundations -Smaller stones filling in gaps and adding strength	-Be careful not to damage waterproofing layer on floor -Fill gaps between larger stones with small stones	-Be aware of heavy lifting procedure
Sand	-Forming of channel embankments	-Easy to pour and fill gaps when dry -Use water to dampen when forming channel profile	-Be careful not to get in eyes -Be aware of heavy lifting procedure
Wooden Beams	-Supporting Point gauge at range of points	-Make marks of exact positions to ensure consistency in results for each test -Make sure they are level	-Take care lifting long objects in confined spaces
Bricks	-Used to support wooden runners and point gauge	-Clear sand from beneath the desired position -Ensure they are secure between large stones	-Be aware of heavy lifting procedure
Geotextile	-Covering channel profile to prevent erosion of sand and maintain shape of embankment	-Use all off-cuts and reduced wastage -Determine the best direction to lay onto channel profile for a close fit	N/A

2.4 Canalised Channel Setup

Table 6: Canalised channel setup

<table>
<tr><td rowspan="2">**Canalised Channel – Setup**</td><td>• Enclosed space for channel construction approximately 2 x 1.5 metres
• Made from wood and treated with protective coating
• Platform put in place to allow access to all points of the channel model</td><td></td></tr>
<tr><td>• Different rock and sand sizes used for construction of the model</td><td></td></tr>
<tr><td rowspan="2">**Canalised Channel – Setup**</td><td>• Larger stones used initially to construct the banks of the channel</td><td></td></tr>
<tr><td>• A second row of larger stones used to reinforce the foundation of the channel banks</td><td></td></tr>
</table>

Canalised Channel – Setup	• Smaller stones added alongside the larger stones to further build up the foundations of the channel banks • Both banks covered with smaller stones to fill in gaps and strengthen	
	• Using a small handheld shovel, sand was poured onto the stones • The gaps were initially targeted to ensure a stable bank structure • Dry sand easy to pour into small spaces	
	• Sand poured until the entire channel bank had been covered • A natural bank slope forming according to the properties of the dry sand	
	• The sand was made damp with water so it could be moulded to the desired dimensions • Using a trowel and small pieces of wood, the channel was moulded to meet the required dimensions • 8cm base and 4cm depth	

	• Geotextile cut to fit the channel and banks	
	• Bricks added into the banks structure but not altering the channel design • Two pieces of wood positioned as runners for the point gauge • Marks made at 10cm intervals for depth readings the be taken	

2.5 Methodology for Measurements and Observations

Four different discharge rates were considered on the model and for each one the following methodology of taking measurements and observations was employed:

- Using a point gauge resting on two wooden runners, the depth of the water was measured at 10cm intervals.
- Observations in and around the culvert were made to see if a hydraulic jump was present.
- Photos were taken at various points along the channel to ensure a comprehensive record of water behaviour was kept.
- The profile of the channel was monitored to ensure no significant erosion had taken place helping to keep consistency and accuracy in the results.

2.6 Canalised Channel – Test 1

Table 7: Canalised channel - Test 1

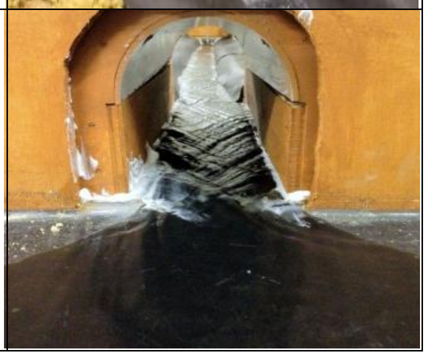
Taking readings	• Readings taken for depth at 10cm intervals starting from the culvert back upstream • Point guage able to give readings in terms of milemetres • Fine adjustment component on point guage allowed measurements to be taken at exact point of contact with water surface	
Observations made	• As the geotextile was cut as one piece to fit the entire channel it allowed water to pass underneath • Difficult to ensure that all the water was flowing on top of the geotextile surface	
	• Removal of geotextile revealed the erosion of the banks that took place • This was caused by water flowing underneath the geotextile • Banks not keeping their initial profile	
Observations made	• Water forming outside of the channel which could affect the strength of the embankment if the level rose significantly	

Observations made	• Water flowing through culvert model • The culvert section did not sit in complete contact with the floor and water was able to flow underneath • Not showing an accurate picture of water interaction in a culvert	

This preliminary test was very valuable in highlighting some problems with the model that were unforeseen. The pictures above give evidence of this and led to improvements being made for the next run. The test also only included one discharge rate while observations were made. Even though this did not show the effects of increasing the discharge, the depth readings demonstrated to an extent how the water behaves while flowing down the channel and through the culvert.

2.7 Canalised Channel – Test 2

Table 8: Canalised channel - Test 2

Improvements made	• Geotextile now cut into two separate pieces for each bank • No geotextile on base surface to prevent water from flowing underneath • Edge of geotextile fixed to the channel surface with silicone sealant	
Improvements made	• Silicone sealant used to seal the culvert section to the base surface • Water now unable to flow underneath giving a more realistic water behaviour in the culvert model	

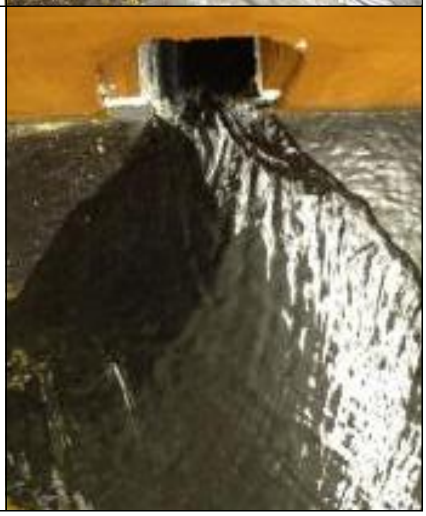
Channel inlet	• The pictures show an increase in turbulence as the discharge was increased • The small pebble was washed away by the force of the water in Scenario 3 and 4 • The geotextile was eventually undermined in Scenario 4	   
Culvert inlet	• The depth at the culvert inlet increased with greater discharge • Free-surface flow upon entering the culvert as seen in Table 2 • The water became more turbulent	

<table>
<tr>
<td></td>
<td>• For Scenario 3 and 4 it can be seen that the banks of the channel had started to erode beneath the geotextile</td>
<td>
Scenario 2

Scenario 3

Scenario 4</td>
</tr>
<tr>
<td>Culvert outfall</td>
<td>• As the discharge increased so too did the turbulence of the water in the culvert
• The water came out with increasing force
• For Scenario 4 the depth of the water was visibly higher as it left the culvert
• No hydraulic jump present and so the flow was outlet controlled according to Table 2</td>
<td>
Scenario 1

Scenario 2</td>
</tr>
</table>

		 
Observations made	• The picture shows the shape of the water dispersal upon leaving the culvert • This is not a true reflection of the culvert in real life as the channel would continue afterwards and this would affect the behaviour of the water	

The second test was very successful and readings for each of the discharge rates were able to be taken. The improvements made following the preliminary test meant the water flow was more true to real life and consistent throughout the test. There was far less erosion of the sand banks as the water was not able to pass underneath the geotextile. The sealant on the inside of the culvert also allowed the water to behave in a more realistic way. As table 8 shows, there were still improvements to be made and these were implemented in the next test.

2.8 Canalised Channel – Test 3

Table 9: Canalised channel - Test 3

Improvements made	• Bricks added at the outfall of the culvert to create a channel profile • This caused the water to behave in a more realistic way when passing through the culvert	
Channel inlet	• The depth of the water entering the channel was higher with greater discharge • The turbulence of the water also increased • The profile of the embankment was slightly eroded with the increasing discharge	Scenario 1  Scenario 2  Scenario 3  Scenario 4 

Culvert outfall	• The speed and force at which the water left the culvert increased with greater discharge • The turbulence was also made greater as the discharge increased • Outlet controlled flow and no hydraulic jump occurred	   

2.9 Meandering Channel Setup

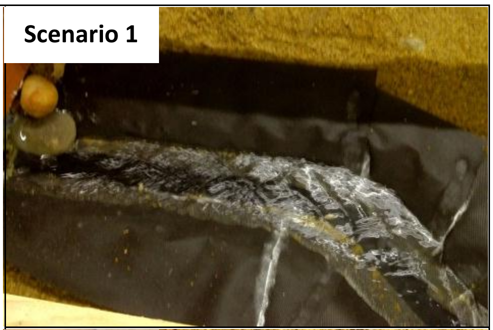
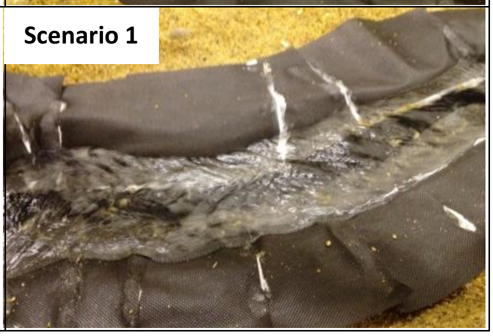
Table 10: Meandering channel setup

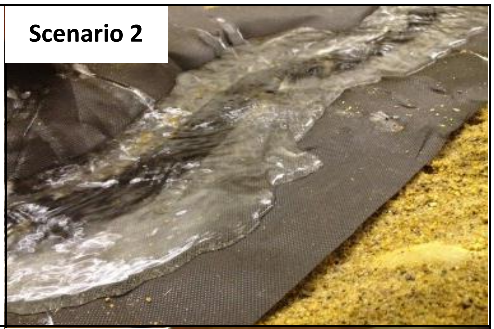
Meandering Channel – Set up	• Tape measure and protractor used to mark out the shape of the meander • Mid-section of channel removed for meander construction • Larger stones used to form foundations of new embankments	
	• Bricks put in place within the embankment to provide support for wooden runners for point gauge • Smaller stones placed alongside and on top of larger stones • Shape of banks built up according to markers of silicone on base surface	
	• Small hand held shovel used to pour sand onto the stones • Meander banks formed and compacted by hand • Damp sand used to make sure the profile was held in place	

Meandering Channel – Set up	• Geotextile used to cover each individual bank • Base of the channel left uncovered • Geotextile cut into smaller pieces in order to follow the curves of the channel without leaving gaps • Geotextile secured in place by silicone sealant	
	• Additional brick columns put in place to allow for multiple positions for the point gauge in accordance with the meander profile	
	• String marked at 10cm intervals • String laid out following the flow path of the channel	
	• Point gauge positioned at each mark on the string • Using a stright rod, measurements were marked on the outer wall • River chainage and equivalent linear distance marked for comparison with straigth channel	 

2.10 Meandering Channel – Test 1

Table 11: Meandering channel - Test 1

Channel inlet	• Initially small amount of turbulence • In Scenario 2 the force of the water caused the geotextile to pull away from the bank • Water undermining the geotextile and eroding sand embankment	 Scenario 1  Scenario 2
Midchannel meander	• Water slowing as it went around main meander • As discharge was increased the depth of water increased notibly • Banks of the channel model almost overtopped in Scenario 2	 Scenario 1  Scenario 2
Last meander	• Increased depth with greater discharge • More turbulence visible as discharge increased • The banks were almost at the	 Scenario 1

	point of overtopping for Scenario 2	
Culvert outfall	• Similar water behaviour for both Scenario 1 and 2 at the culvert outfall • Slightly increased amount of turbulence in the water as discharge increased • No hydraulic jump visible for each Scenario • Outlet controlled flow as shown by Table 2	 

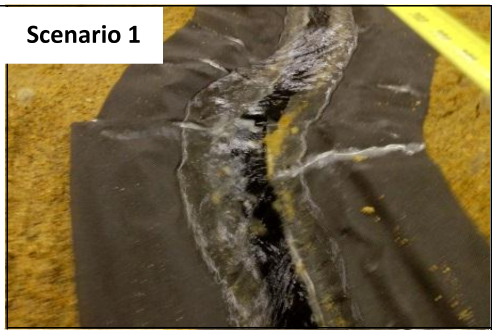
2.11 Meandering Channel – Test 2

Table 12: Meandering channel - Test 2

Improvements	• Geotextile fixed more securely at the channel inlet • Additional strips of geotextile used to fix on outside of inlet	
Change in method	• Point gauge not efficient to take depth readings • Cocktail sticks dipped at regular intervals and marked with pen to be later measured	

<table>
<tr>
<td rowspan="3">First meander</td>
<td rowspan="3">

Increased depth with higher discharge
Turbulence also increased with the discharge
For Scenario 3 the water was almost at the point of overtopping the banks of the channel

</td>
<td>
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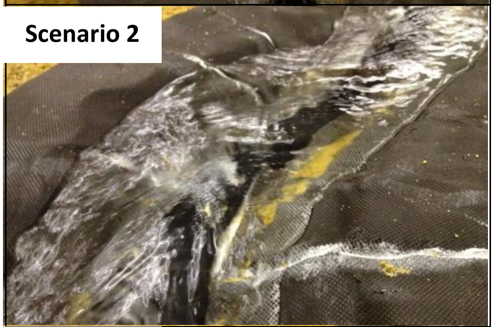
<td>
</td>
</tr>
<tr>
<td>
</td>
</tr>
<tr>
<td rowspan="2">Before main meander</td>
<td rowspan="2">

Depth increased with great discharge
By the time Scenario 4 was in effect, the banks had been overtopped
Overtopping occurred on the outside of bends

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</tr>
</table>

Midchannel meander	• Water depth increased greatly with more discharge • Flow more turbulent with higher discharge • For Scenario 3 the water was close to going over the banks • Banks on the outside of the main meander were overtopped in Scenario 4	   
After main meander	• Increased depth and turbulences with greater discharge • Water overtopping banks for	

	Scenario 3 and 4 • Even in Scenario 2 the water is almost too much for the channel capacity • Sand quickly eroded by water when beyond the geotextile	  
Last meander	• Water flowed more quickly beyond main meander • Water depth and turbulence increased with the higher discharges • Water not overtopping banks of channel but closer on the outside of curves	 

3.0 Results

3.1 Canalised Channel

Table 13: Canalised channel - Test 1 results

Experiment		Date	Time			
1		13/03/2012	13:00:00			
Prototype Discharge [m³/s]			25	45	65	85
Model scale [m³/s]			3.5355E-04	6.3640E-04	9.1924E-04	1.2021E-03
Model scale [l/s]			0.354	0.636	0.919	1.202
			Depth [cm]			
Distance upstream from culvert [m]		0.000	1.04	/	/	/
		0.100	1.14	/	/	/
		0.200	1.14	/	/	/
		0.300	1.05	/	/	/
		0.400	0.99	/	/	/
		0.500	1.07	/	/	/
		0.600	1.17	/	/	/
		0.700	1.20	/	/	/
		0.800	1.10	/	/	/
		0.900	1.04	/	/	/
		1.000	1.15	/	/	/
		1.100	1.04	/	/	/
		1.200	1.10	/	/	/
		1.300	1.10	/	/	/
		1.400	1.05	/	/	/
		1.500	1.25	/	/	/
		1.600	1.21	/	/	/
		1.700	1.27	/	/	/
		1.800	1.05	/	/	/
		1.900	1.09	/	/	/
		2.000	0.93	/	/	/

Table 14: Canalised channel - Test 2 results

Experiment	Date	Time			
2	14/03/2012	13:00:00			
Prototype Discharge [m^3/s]		25	45	65	85
Model scale [m^3/s]		3.5355E-04	6.3640E-04	9.1924E-04	1.2021E-03
Model scale [l/s]		0.354	0.636	0.919	1.202
		Depth [cm]			
Distance upstream from culvert [m]	0.000	1.06	1.49	1.83	2.00
	0.100	1.15	1.79	1.88	2.43
	0.200	1.23	2.00	2.20	2.55
	0.300	1.40	1.70	2.18	2.59
	0.400	1.13	1.50	2.15	2.49
	0.500	1.22	1.64	2.04	2.45
	0.600	1.34	1.72	2.03	/
	0.700	1.30	1.72	2.03	/
	0.800	1.25	1.49	1.89	2.29
	0.900	1.11	1.55	1.90	2.24
	1.000	1.23	1.58	2.17	2.35
	1.100	1.08	1.46	1.88	2.35
	1.200	1.19	1.75	2.02	2.28
	1.300	1.19	1.74	2.14	2.52
	1.400	1.25	1.70	1.91	2.41
	1.500	1.41	1.82	2.15	2.77
	1.600	1.42	1.89	2.45	2.63
	1.700	1.26	1.90	1.99	2.50
	1.800	1.11	1.75	1.92	/
	1.900	1.00	1.48	1.98	2.59
	2.000	0.96	1.23	1.49	2.91

Table 15: Canalised channel - Test 3 results

Experiment	Date	Time			
3	15/03/2012	13:00:00			
Prototype Discharge [m³/s]		25	45	65	85
Model scale [m³/s]		3.5355E-04	6.3640E-04	9.1924E-04	1.2021E-03
Model scale [l/s]		0.354	0.636	0.919	1.202
		Depth [cm]			
Distance upstream from culvert [m]	0.000	1.24	1.47	1.70	2.19
	0.100	1.15	1.69	1.90	2.21
	0.200	1.06	1.74	2.05	2.34
	0.300	1.10	1.56	2.03	2.50
	0.400	1.09	1.53	1.88	2.30
	0.500	1.14	1.46	1.89	2.18
	0.600	1.09	1.49	1.80	2.16
	0.700	1.15	1.70	1.78	2.15
	0.800	1.04	1.74	1.89	2.26
	0.900	1.23	1.70	2.03	2.23
	1.000	1.34	1.79	2.09	2.65
	1.100	1.39	1.81	2.15	2.34
	1.200	1.38	1.80	2.19	2.59
	1.300	1.40	1.92	2.24	2.69
	1.400	1.23	1.90	2.23	2.22
	1.500	1.45	1.78	2.20	2.73
	1.600	1.36	1.84	2.25	2.40
	1.700	1.57	1.67	1.97	2.32
	1.800	1.25	1.70	1.78	2.36
	1.900	1.14	1.50	1.87	1.73
	2.000	0.99	1.50	1.90	2.54

3.2 Meandering Channel

Table 16: Meandering channel - Test 1 results

Experiment		Date	Time			
1		26/03/2012	11:00:00			
Prototype Discharge [m^3/s]			25	45	65	85
Model scale [m^3/s]			3.5355E-04	6.3640E-04	9.1924E-04	1.2021E-03
Model scale [l/s]			0.354	0.636	0.919	1.202
	Linear	*River*	*Depth [cm]*			
Distance upstream from culvert [m]	0.000	0.000	1.07	/	/	/
	0.100	0.100	1.29	/	/	/
	0.200	0.200	1.28	/	/	/
	0.300	0.300	1.55	/	/	/
	0.400	0.400	1.54	/	/	/
	0.495	0.500	2.17	/	/	/
	0.590	0.600	2.19	/	/	/
	0.660	0.700	1.82	/	/	/
	0.730	0.800	1.90	/	/	/
	0.810	0.900	1.98	/	/	/
	0.890	1.000	1.84	/	/	/
	0.980	1.100	1.80	/	/	/
	1.050	1.200	1.78	/	/	/
	1.130	1.300	2.09	/	/	/
	1.230	1.400	2.08	/	/	/
	1.290	1.500	2.04	/	/	/
	1.370	1.600	1.93	/	/	/
	1.460	1.700	1.79	/	/	/
	1.550	1.800	1.49	/	/	/
	1.640	1.900	1.14	/	/	/
	1.710	2.000	1.79	/	/	/
	1.780	2.100	1.75	/	/	/
	1.850	2.200	1.59	/	/	/
	1.950	2.300	1.00	/	/	/
	2.050	2.400	1.18	/	/	/

Table 17: Meandering channel - Test 2 results

Experiment		Date	Time			
2		28/03/2012	14:00:00			
Prototype Discharge [m³/s]			25	45	65	85
Model scale [m³/s]			3.5355E-04	6.3640E-04	9.1924E-04	1.2021E-03
Model scale [l/s]			0.354	0.636	0.919	1.202
	Linear	*River*	*Depth [cm]*			
Distance upstream from culvert [m]	0.000	0.000	1.37	2.03	2.52	2.78
	0.100	0.100	1.38	2.08	/	/
	0.200	0.200	1.60	1.94	/	/
	0.300	0.300	1.64	2.46	/	/
	0.400	0.400	1.48	2.37	/	/
	0.495	0.500	2.00	2.55	/	/
	0.590	0.600	2.05	2.39	3.23	3.30
	0.660	0.700	2.10	2.56	/	/
	0.730	0.800	1.90	2.47	/	/
	0.810	0.900	1.85	2.37	/	/
	0.890	1.000	1.89	2.77	3.43	3.81
	0.980	1.100	1.91	2.44	/	/
	1.050	1.200	1.95	2.55	/	/
	1.130	1.300	1.96	2.61	3.55	4.05
	1.230	1.400	1.99	2.77	/	/
	1.290	1.500	2.07	2.85	3.10	3.70
	1.370	1.600	1.94	2.65	/	/
	1.460	1.700	1.50	2.28	2.90	3.60
	1.550	1.800	1.65	2.21	/	/
	1.640	1.900	1.40	2.33	/	/
	1.710	2.000	1.98	2.62	/	/
	1.780	2.100	1.89	2.00	2.82	3.00
	1.850	2.200	1.90	2.18	/	/
	1.950	2.300	2.10	2.12	/	/
	2.050	2.400	1.78	/	/	/

3.3 Canalised Channel Depth Graphs

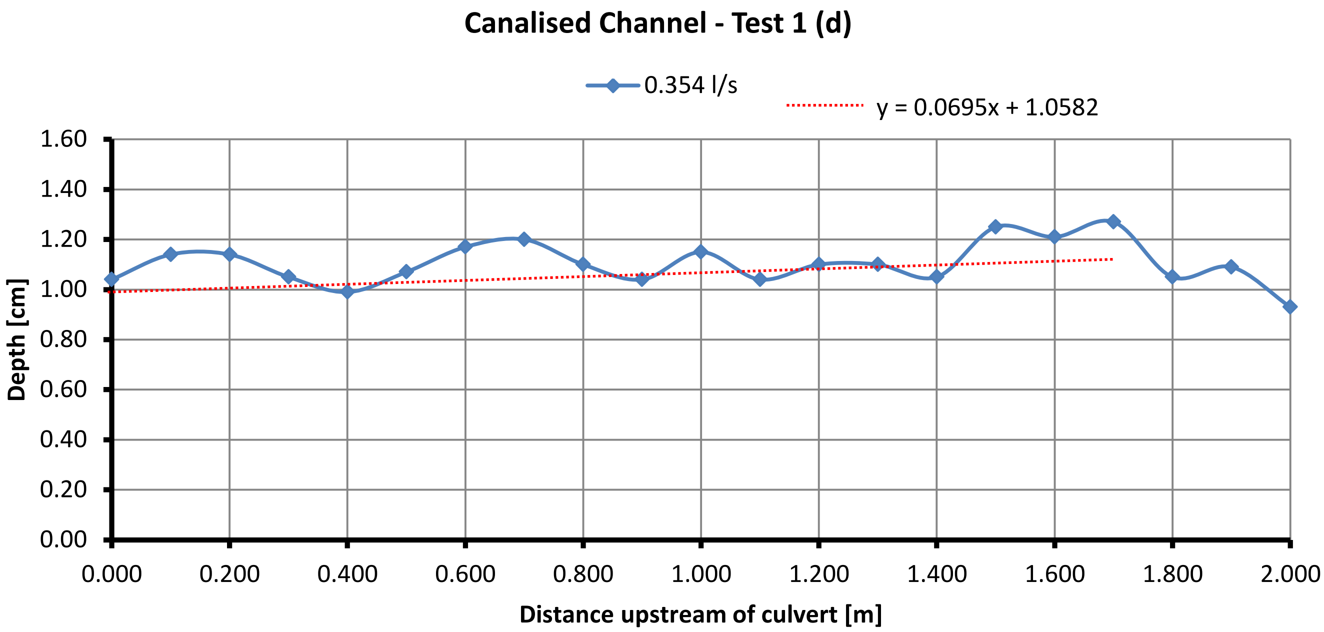

Figure 4: Canalised channel - Test 1 (d)

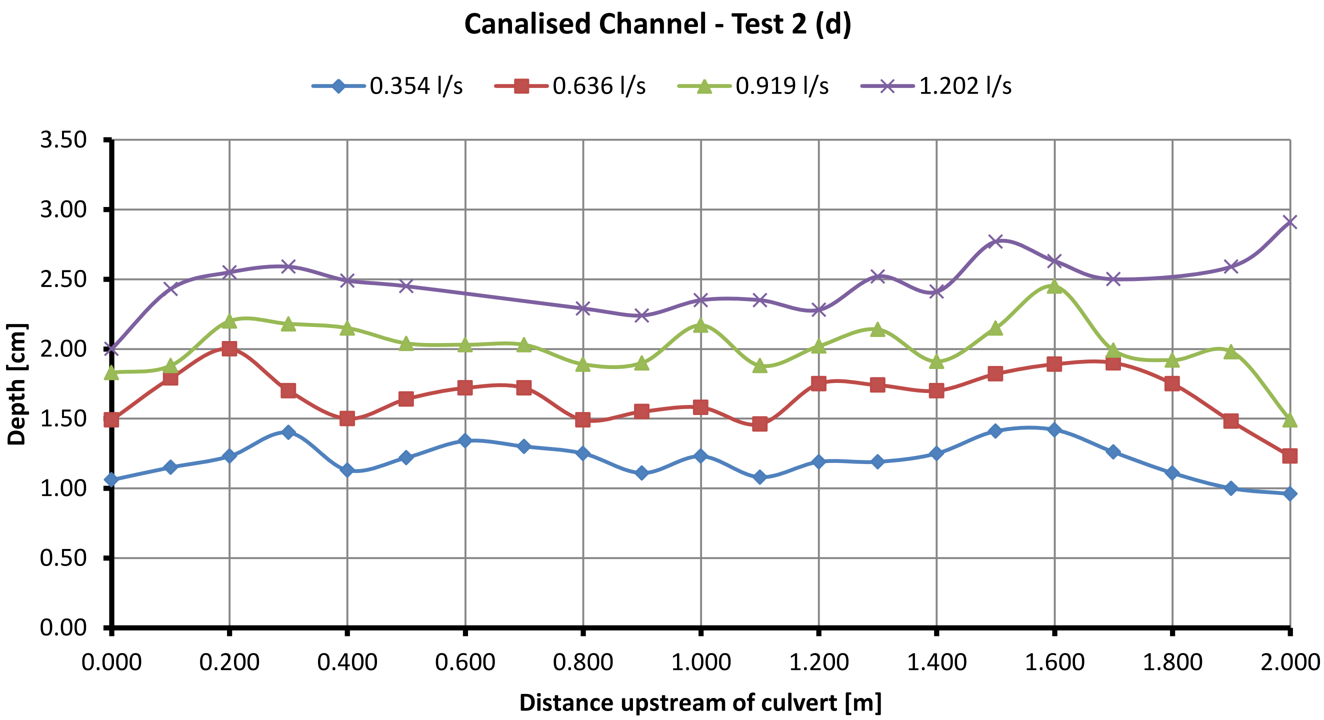

Figure 5: Canalised channel - Test 2 (d)

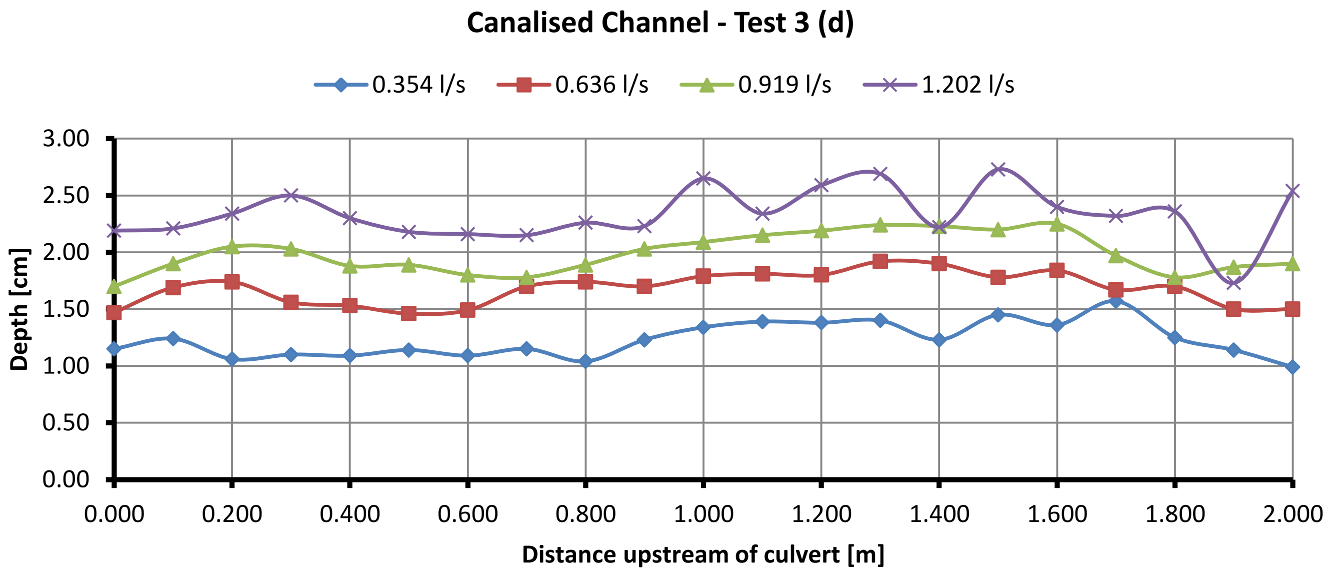

Figure 6: Canalised channel - Test 3 (d)

3.4 Canalised Channel Froude Number Graphs

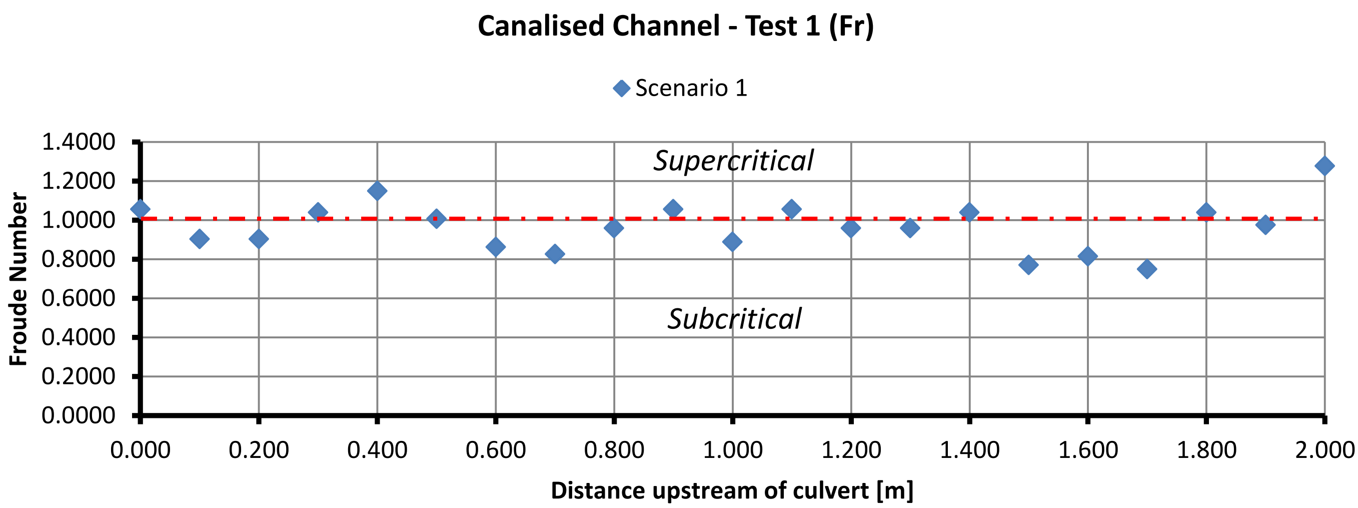

Figure 7: Canalised channel - Test 1 (Fr)

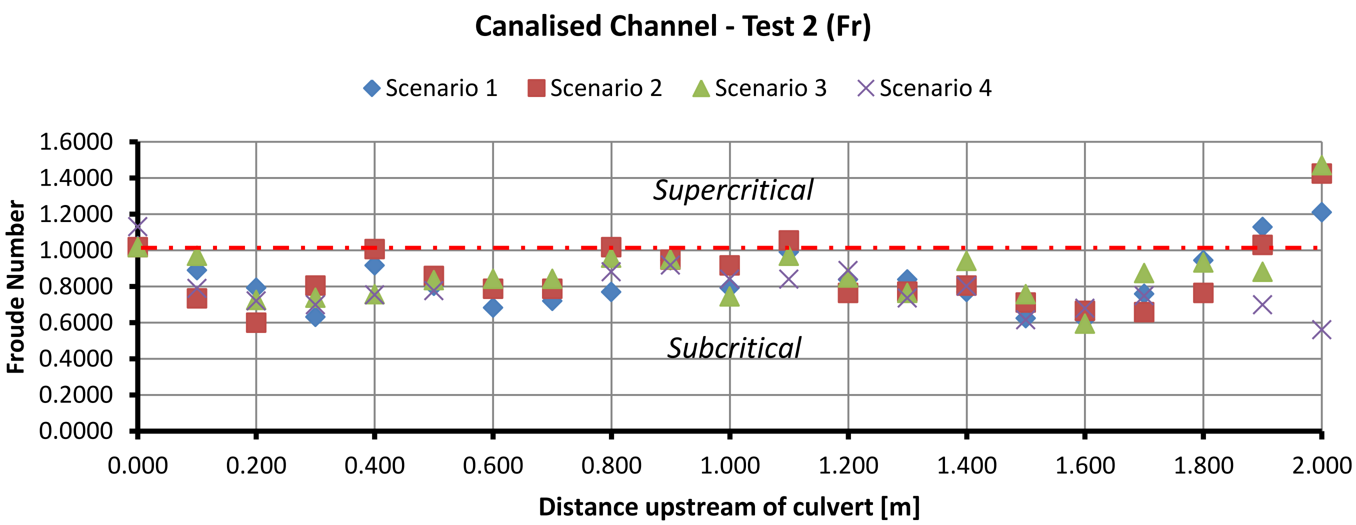

Figure 8: Canalised channel - Test 2 (Fr)

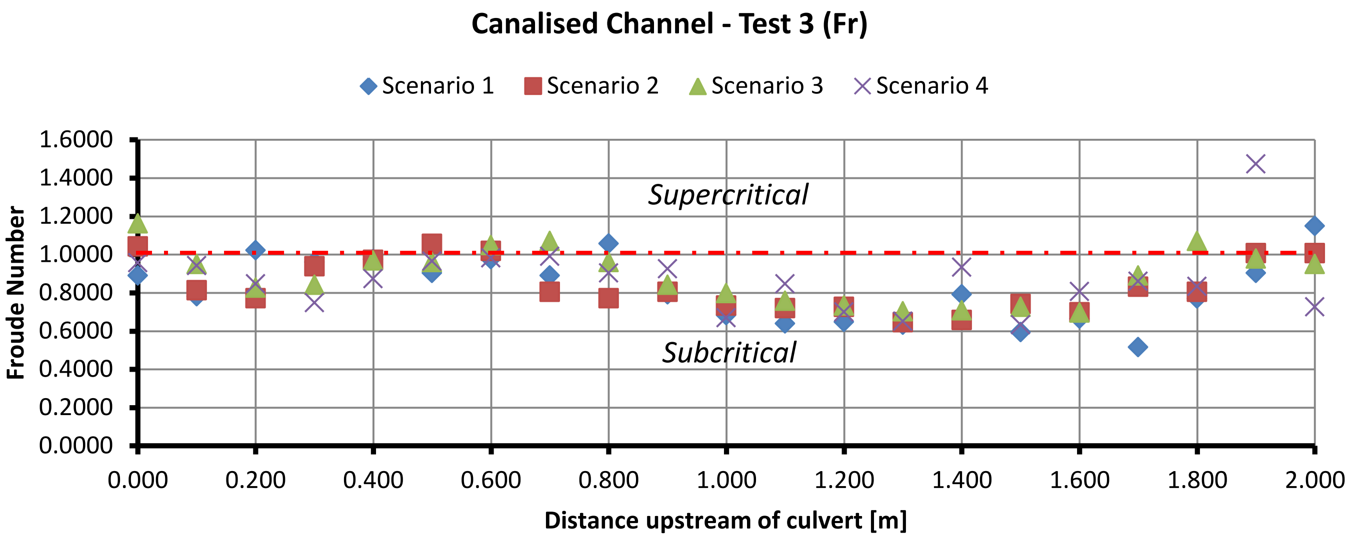

Figure 9: Canalised channel - Test 3 (Fr)

3.5 Meandering Channel Depth Graphs

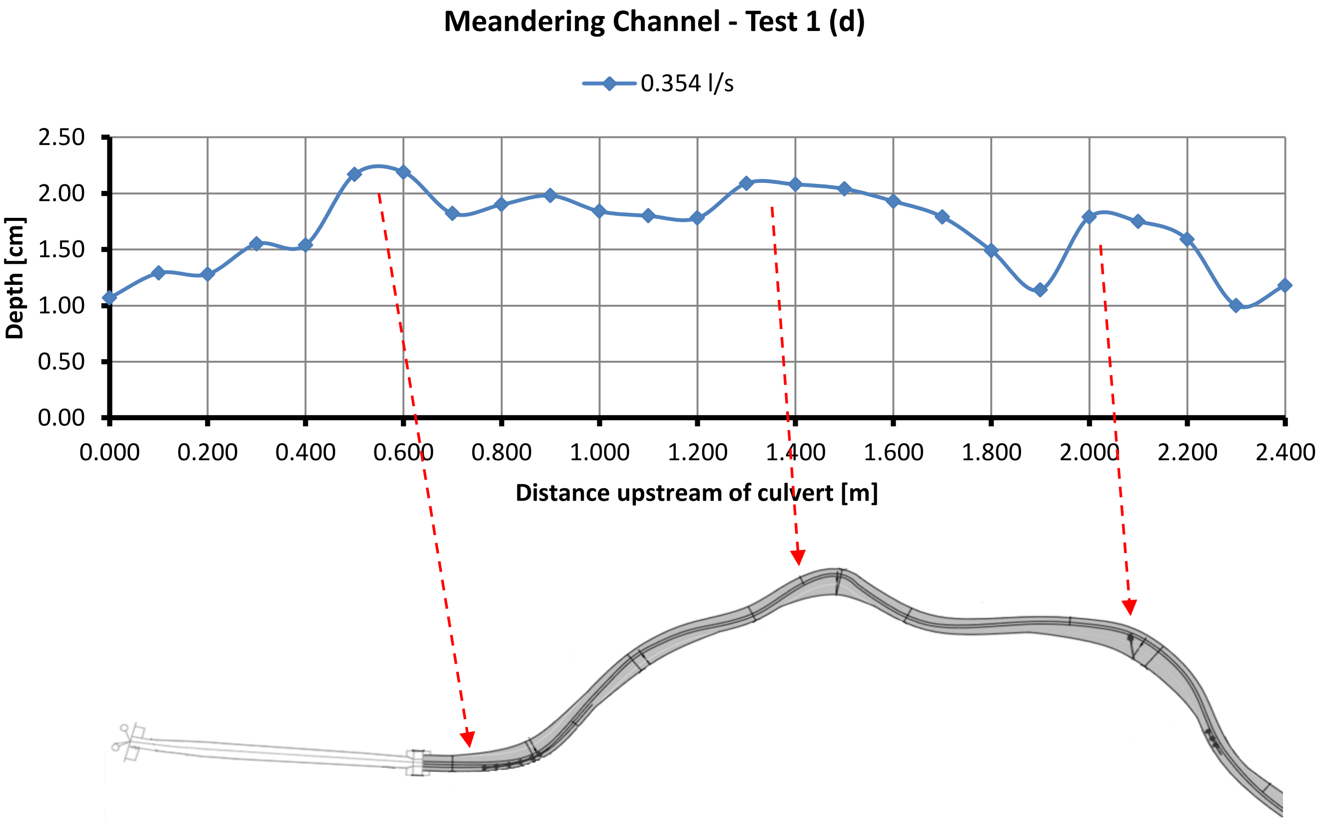

Figure 10: Meandering channel - Test 1 (d)

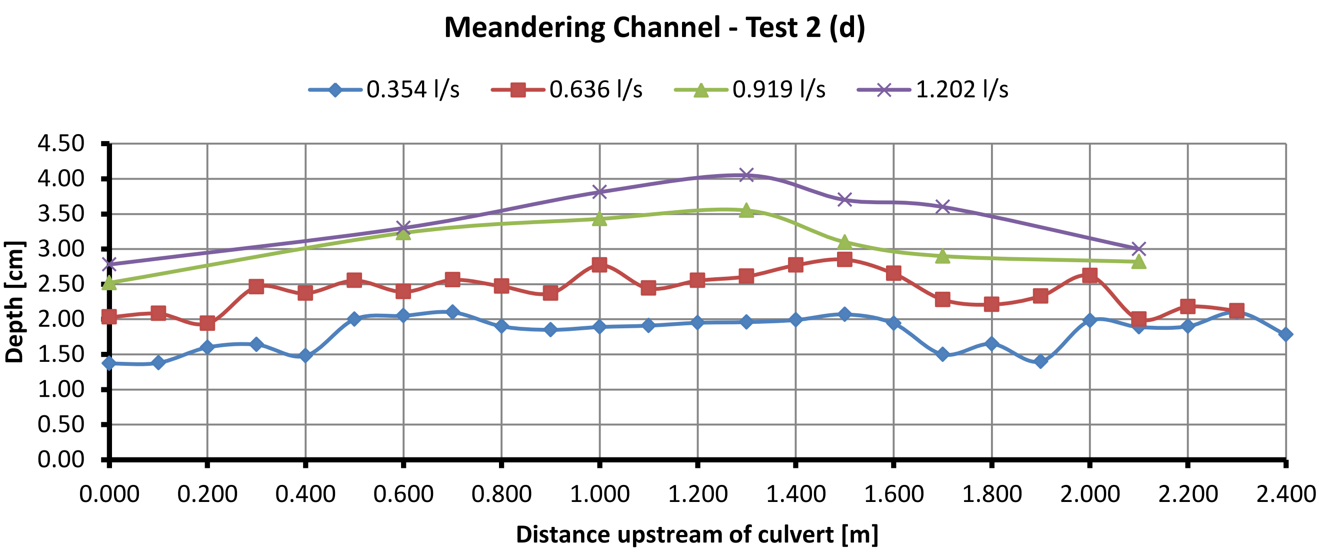

Figure 11: Meandering channel - Test 2 (d)

3.6 Meandering Channel Froude Number Graphs

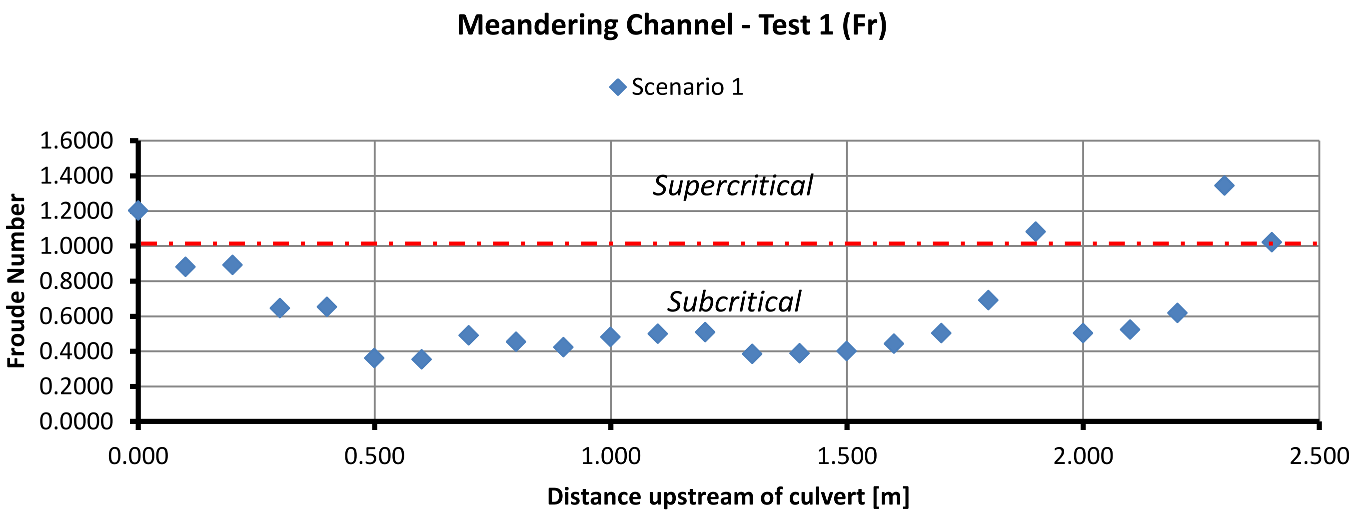

Figure 12: Meandering channel - Test 1 (Fr)

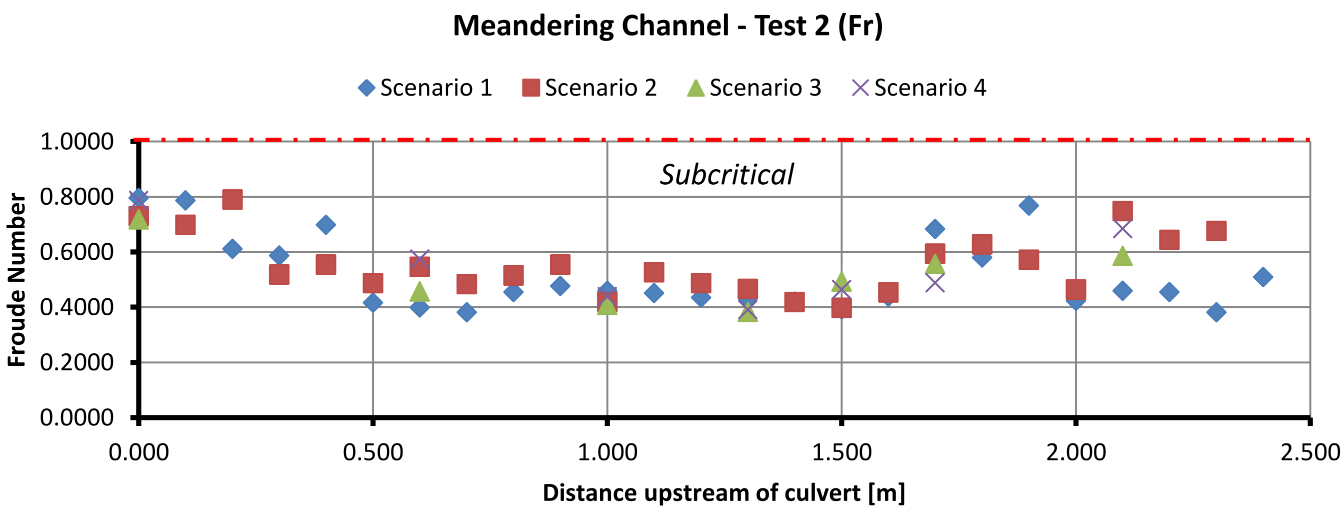

Figure 13: Meandering channel - Test 2 (Fr)

4.0 Discussion

4.1 Analysis of Graphs

By displaying the results for depth readings in the form of a graph it has been easier to recognise general trends and individual anomalies. Each of the tests has been tabulated and plotted and an analysis of these follows.

The canalised channel results were collected from a total of three separate tests. Test 1 was a preliminary run and does not compare different discharge rates though it did give a picture of how the water behaves. The depth did not alter too much with a range of only 0.34cm. Figure 4 shows that just before the culvert the water depth dropped to 1.04cm which is one of the lowest depth readings. As Figure 7 shows, the flow at this point was supercritical which resulted in the relatively shallow and fast flow. There was free surface flow at the inlet to the culvert and no hydraulic jump after the culvert or in the barrel. Table 2 describes this as outlet controlled flow. The lowest depth recorded was seen at the entrance to the channel. Again Figure 7 shows that the water was in supercritical flow at this point which led to a faster and shallower flow. Figure 4 shows that the peak of the water depth occurred between 1.5m and 1.7m from the culvert entrance. The water was as deep as 1.27cm at this point of the channel and the flow was subcritical, as seen on Figure 7. Figure 4 also shows a trend line which has a positive inclination and therefore indicates that the depth of the water gradually increased moving back from the culvert. This trend line was formed from all of the data apart from the last three points. This is because the readings are influenced by the water rushing into the channel.

The second graph, Figure 5, shows results for depths taken in the case of four different discharge rates. It can be clearly seen that as the discharge was increased that the whole range of depth readings also increased. There was a variation in the range of results that were taken for each individual discharge. For Scenario 1 the range of depths was just 0.46cm and for Scenario 2 the range was 0.77cm. There was a greater range of results for the higher discharges with Scenario 3 giving a range of 0.96cm and Scenario 4 a range of 0.91cm. The water behaviour is more volatile when there is a great amount flowing down a narrow channel. This was particularly obvious at the entrance of the channel where water rushed through a small opening. A trend that is seen for each of the discharges is the fact that the depth of water always dropped before entering the culvert. There was no instance of a submerged inlet at the culvert but always a free surface flow. As Figure 8 shows, each of the Scenarios experienced a supercritical flow at this point. This was characterised by a relatively shallower and faster flow as the water entered the culvert.

The third test for the canalised channel also encompassed all four discharge rates. The range of depths measured for each Scenario was quite similar. It can be shown that Scenario 1, Scenario 2 and Scenario 3 had ranges of 0.58cm, 0.46cm and 0.55cm respectively. The measurements taken for Scenario 4 were far more varied with a range of 1.0cm. As Figure 6 shows, the general trend that was seen in the second test is also present here. The water depth decreased before entering the culvert in each case but there was not supercritical flow for every Scenario. As Figure 9 shows, Scenario 2 and 3 experienced a supercritical flow upon entering the culvert but Scenario 1 and 4 were in subcritical flow.

Two tests were undertaken for the meandering channel which was constructed according to the proposed re-naturalisation scheme. The results begin to give a picture of how the water behaviour would be affected if the development goes ahead on the real scale. There is also a contrast between these results and those taken for the canalised channel and any notable comparisons have been highlighted.

The initial test only incorporated the lowest discharge rate, Scenario 1, to give an indication of how the water would interact with the meanders. What stands out is the large variation of depths measured at different points along the channel, especially compared to those taken from the canalised channel at this discharge.

The smallest depth was 1.0cm and the largest was 2.19cm and therefore the range was 1.19cm. A focus on the main peaks in water depths has allowed the corresponding position on the channel to be identified. The graph has been annotated alongside a diagram of the channel profile as can be seen in Figure 10. The three highest peaks for water depths occurred where the main meanders are. More specifically, the peaks took place towards the end of each meander. This is because the water slowed down in changing direction as a result of the meanders. Figure 12 identifies the type of flow that was occurring at these points. In every case the flow was subcritical which is characterised by a relatively deeper and slower flow. Just as in the canalised channel, there was a more shallow depth right at the start of the channel. At this point the water was in supercritical flow and therefore a comparatively fast and shallow flow was evident. As Figure 10 shows, the water depth also dropped before entering the culvert. Figure 12 demonstrates that the flow at this point was supercritical and this was observed as shallower and faster flow. Similarly to the canalised channel, the flow through the culvert was outlet controlled with no hydraulic jump present, as seen in Table 2. The only other occurrence of supercritical flow was at 1.9m from the culvert just after the first of the meanders. The water at this point was increasing in velocity after leaving the bend and now travelling in a straighter part of the channel.

The second test carried out for the meandering channel included all four of the discharge Scenarios. Observations were recorded of the changing water behaviour and the areas most susceptible to flooding. Figure 11 shows that each of the flow rates resulted in a similar profile of depth readings throughout the channel. It can be seen that in each case the peak depth was located between 1.2m and 1.5m upstream of the culvert entrance. This distance corresponds to the position of the main meander in the channel as can be seen on figure 10. Again this increase in depth can be attributed to the fact that the water had to slow at this point of changing direction. The direction alteration is most severe at this location along the channel which would result in the greatest reduction in water velocity. Figure 13 shows that the water was in subcritical flow at this point which is typified by a deep and slow flow. Other trends that are visibly present for each of the discharge rates is the drop in water depth at the start of the channel and before entering culvert. As previously discussed, this is due to the water increasing in velocity at these points. Though, unlike the previous test, the water was in subcritical flow at this point. The inlet did not become submerged and there was no evidence of a hydraulic jump. Table 2 describes this as outlet controlled flow.

4.2 Meandering Channel Diagrams

Throughout the course of the experiments, observations and photographs were taken of the water flow through the channel. These observations have been presented in the following diagrams (Figures 14 to 17) which give a clearer image of where the water depths became critical or flooding took place. This insight is not necessarily easy to interpret from the tables or graphs.

('Critical' used to describe the case of the water being close to overtopping the banks rather than the type of water flow)

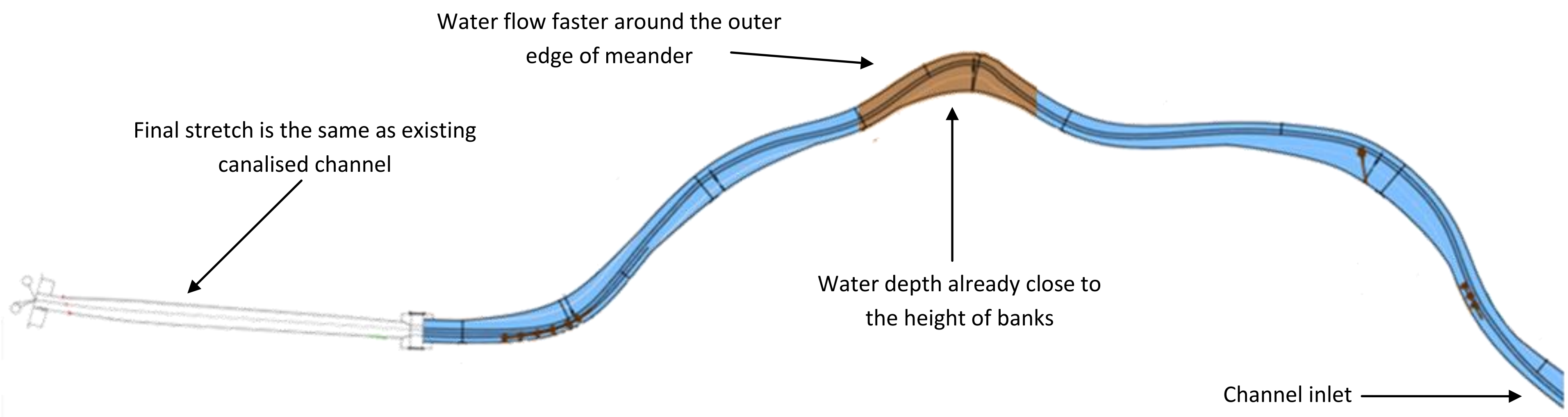

Figure 14: Channel diagram - Scenario 1

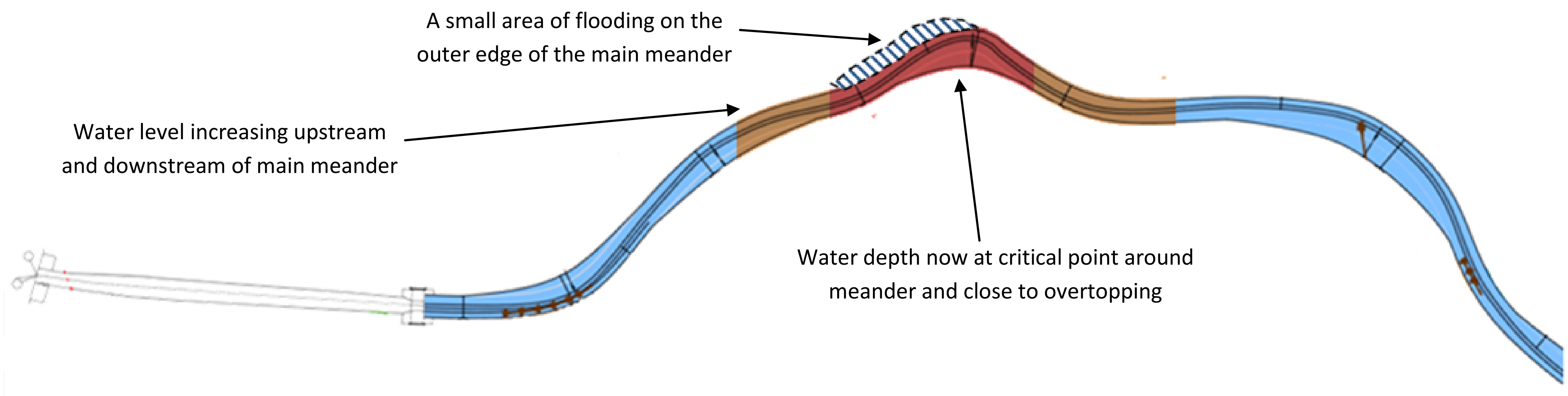

Figure 15: Channel diagram - Scenario 2

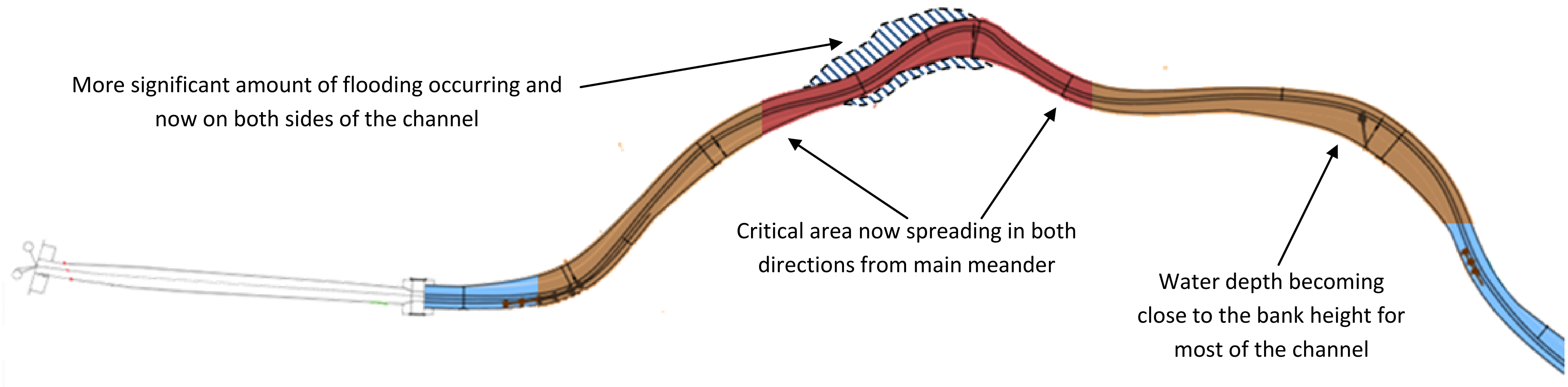

Figure 16: Channel diagram - Scenario 3

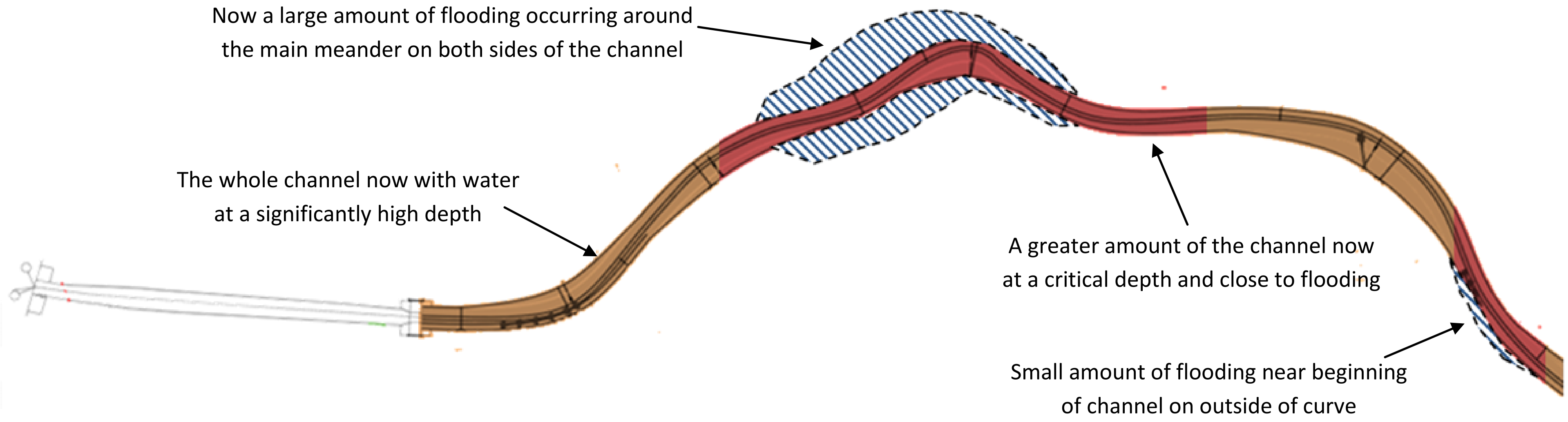

Figure 17: Channel diagram - Scenario 4

4.3 Critical Review of Results

The implications of undertaking physical modelling are not to be underestimated and a satisfactory outcome is dependent on careful planning and practice. The critical review of results is an important process and has been vital in establishing their validity in this project. A range of factors has been considered before the formulation of final conclusions.

4.3.1 Model Limitations

The success of a scale model depends on just how accurate it can be in reflecting the form of the real sized prototype. Any inaccuracies must be identified along with the magnitude of their impact. The model in this investigation certainly had aspects that were not able to be precisely formed in accordance with the prototype.

A geotextile was chosen for the lining of the channel in the model. This was to prevent the transfer of sediment occurring which would have led to the channel profile changing during the course of a test. The consistency of results was protected by maintaining the channel profile for multiple discharge rates. A disadvantage of incorporating the geotextile was the manning's number which was not similar to the real life situation. A river channel of this type would be expected to have a manning's number of approximately 0.035 whereas the geotextile's value would be approximately 0.025 (Hall, et al., 2012). The equation below shows how the manning's number influences the velocity:

$$V = \frac{1.49(R^{0.66}S^{0.5})}{n}$$

$$V = velocity \;\; R = hydraulic\ radius \;\; S = water\ surface\ slope$$

$$n = mannings\ number$$

So it can be deduced that as the manning's number increases the velocity decreases. This would suggest that the model created a situation that caused the water velocity to be greater than it should be.

By using a geotextile lining some other scenarios associated with sediment transport have been ignored. The amount of erosion due to flooding has not been considered which could be significant. The limitations of the model also meant that the channel profile did not naturally evolve. The meanders may have altered over time or the channel may have even reverted to a canal form. While a certain stability was required for consistency in the modelling, this does not reflect the river processes on the real scale.

Another area which led to inaccuracies was the cross-sectional profile of the meandering channel. There was an in depth proposal document put together for the re-naturalisation of Knepp Castle Estate and this included plans for the new channel. From this a design for the exact scale model profile was put together and can be seen in Figure 18:

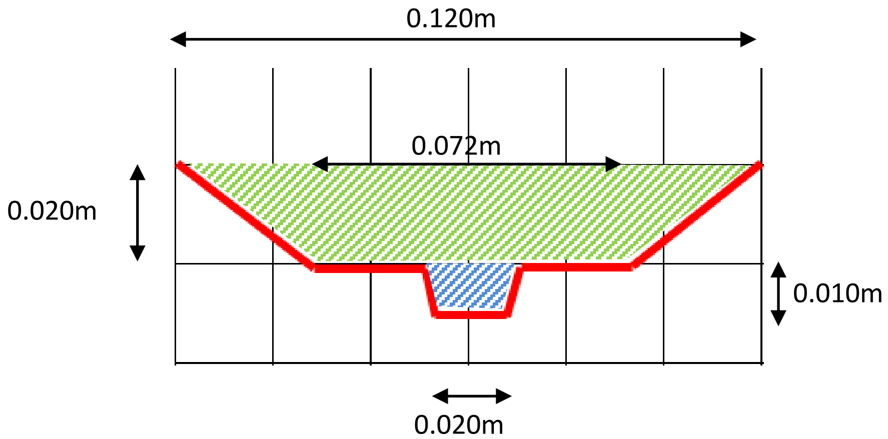

Figure 18: Cross section of meandering channel

This was reasonable in theory but when it came to the construction of the channel it was found that the exact profile could not be produced. Working with sand it would have been extremely difficult to create the central low flow channel which is shaded blue in Figure 18. This would only be 2cm wide and 1cm deep. In conjunction with this very intricate formation process, there would be the matter of fitting the geotextile to this profile exactly without allowing water to pass underneath. For these reasons, it was decided to model only the high flow part of the cross section which is shaded green on the Figure 18. The end result of this alteration was a reduced capacity for water in the meandering channel. The total volume of this reduction has been calculated by multiplying the entire length of the channel by the cross-sectional area.

$$0.020 * 0.010 * 2.4 = 0.0005\ m^3$$

This value has then been scaled up to determine the impact on the real channel:

$$0.0005 * {Z_r}^2 X_r = 0.0005 * 50^2 * 200 = 240\ m^3$$

This information has been considered when forming conclusions about whether or not the proposed channel would cause significant flooding.

It should also be noted that only a small portion of the proposed channel was modelled because of the limitations of time and laboratory size. The lower stretch of the river channel was considered to be the most crucial part to model because of its proximity to the road. As has been previously mentioned, some of the trends that can be seen in the results are caused by the way the water enters the channel model. The narrow entrance opening caused the water to rush in with a higher velocity than would be expected, usually in supercritical flow. This resulted in a lower depth and the first portion of the model would have been influenced by this. If the entire river channel or even a significant amount more was modelled, then the water's behaviour would have been more true to the real life situation.

The report outlining the proposed scheme also indicates that there is an existing weir in place at the farm access crossing. This weir is to be kept in place in the proposed scheme as a means of controlling the flow of water for the final stretch of the channel before the culvert. There was a limit to the complexity of model that could be constructed and so a scale model of this weir structure was not included. The focus was to create the correct profile and flow path to reflect the prototype. The fact is that by excluding a scale version of the weir structure from the model the validity of the results has been affected. The purpose of the weir structure was to hold water back to a certain extent. It therefore could be suggested that there would have been a greater amount of flooding had it been included in the model. This has been considered when drawing the final conclusions.

There are many environmental factors that have not been modelled in this investigation. For example, the influence of rain has not been fully explored. While it was possible to simulate different discharge rates with a steady flow from one source, this does not capture the effects of rain. The Knepp Castle Estate has a geology that is rich in clay. The impact of this is very little absorption of rain water and a high amount of surface run-off. This quickly finds its way to the river channel from a number of different directions. The physical model did not demonstrate this process and so does not completely reflect the real life conditions. It is likely that the flooding would have been more severe if this unsteady flow situation could have been reproduced.

4.3.2 Errors in Results

Another area worthy of discussion is the methodology of taking depth readings. While every effort was taken to ensure accuracy and consistency in the measurements, it is important to appraise whether errors have taken place.

The first method for taking the depth readings was using the point gauge supported above the channel by two wooden beams. The gauge offered a degree of accuracy to a tenth of a millimetre which had the potential to give very exact measurements. The difficulty came in operating and reading the point gauge during the course of the experiment. Because of the size of the model, the depths were very small and close to the ground. Viewing the gauge was uncomfortable and this became more apparent after 20 readings for each of the different discharge rates were taken. The awkward nature of this measuring process may have led to errors in the depth readings.

The second method that was used to measure the depth of the water was dipping cocktail sticks and marking the point where the water reached. This process was less uncomfortable than using the point gauge but still required leaning down to dip and mark the cocktail stick. The accuracy of this method would have to be less than using the point gauge because it was more difficult to find the exact point at which the cocktail stick emerged from the water. To help counter this, two readings for each of the intervals were taken and an average was calculated. This would have reduced the impact of any errors incurred during this process.

The issue affecting both of the measuring methods was the need to take results as quickly as possible. The necessity for speed while taking depth readings had not been anticipated but as the experiments progressed it was realised that this was the case. This was driven by the desire to have consistency in the results across the different discharges. Although a geotextile was used to line the entire channel and prevent sediment transport, there was still an erosion of the channel banks taking place over the course of each test. Of course this was slowed by the geotextile but was still evident, especially as the discharge increased. As a result, there was a need to gather the depth readings as quickly as possible so that the channel profile had not been altered too greatly. While this rapid approach was considered necessary, it may have influenced the accuracy of the values measured.

Repetition is very important in practical experiments as it helps to reduce to impact of errors. It would have been preferable to have done another test for the meandering channel but the time restraints of the project inhibited this. Two tests were adequate but it can be recognised that a third test would have reinforced any overarching trends. This is particularly relevant to Scenario 3 and Scenario 4 as measurements for these were only taken once.

The different issues that have been discussed above may have only led to an error of a millimetre or even a centimetre but this must be scaled up to appreciate the real significance. The vertical scale used for the model was 1/50 and so a 1mm error would equate to a 0.05m error at the real scale. An error of 1cm would equate to an error of 0.5m for the prototype which is certainly significant for a river channel that is designed to be only 1.5m deep.

5.0 Conclusions

The primary purpose of this investigation has been to explore whether or not the proposed re-naturalisation of the River Adur on Knepp Castle Estate is feasible. The focus has not been on the financial aspects of such a development but the influence on the frequency and severity of flooding. The concern of flooding in close proximity to the A24 was the main motivation for the investigation. After a process of physical modelling, a range of data was taken as well as observations being recorded of water behaviour. Following the critical review of such results the following conclusions have been formulated.

The re-naturalisation of the channel, with the addition of meanders, will increase the probability and severity of flooding. The curves in the river result in a slower moving flow with greater depths, which is attributed to subcritical flow. The new scheme includes a shallower river profile with meanders and this combination will cause more flooding.

There would be negative effects on the local natural environment with an increased likelihood and severity of flooding. Vegetation on the surrounding floodplains may become submerged including trees. A constant repetition of this flooding may cause trees to fall over and enter the water course. Animal life may also suffer from more severe flooding. These impacts are the opposite of the desired outcomes expressed by the Knepp Castle Estate owner.

The impact on the A24 cannot be judged fully by the scope of this investigation. A 1 in 100 year flood will not cause the road to become submerged but a more sever flood event has not been explored.

The water flow through the culvert will be impacted and evidence suggests that this will become less supercritical and even subcritical. The effects of a completely submerged culvert have not been explored and a more severe flood event would need to be modelled.

The use of physical scale modelling is invaluable in understanding the behaviour of rivers and hydraulic processes. While creating a wholly perfect model is unattainable, as long as the limitations can be identified, the results can still be utilised.

5.1 Recommendations

This project has been valuable in exploring the feasibility of the Knepp Castle Estate re-naturalisation project but its limitations are also evident. The following are recommendations for further investigation:

- A scale model including weir structure to investigate its impact on the likelihood and severity flooding.
- An incorporation of woody debris log jams in scale model in accordance with prototype.
- A more extensive model in terms of the length of channel and also the surrounding landscape including accurate topography.
- A model with consideration for sediment transport processes in the proposed river channel. A movable bed model to explore whether the profile of the river would be altered by the water flow and if the channel would revert back to a straight channel over time.
- A model with the capability of simulating the effects of rain, run-off and unsteady flow.
- An investigation of a more sever flood event such as 1 in 250 or 1 in 500 years.

6.0 References

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7.0 Appendices

Appendix I - Preliminary Hydrology and Hydraulics study

Preliminary Hydrology and Hydraulics study

As a result of initial hydraulic assessment and discussion of the restoration concepts for the geomorphology of the channel and floodplain it was decided that preliminary modelling work on the hydraulics and hydrology of the site should be undertaken to try and "narrow down" the options as early in the process as possible.
This work has included:

- Some initial hydrological analysis to determine the volumes of water in a 1 in 100 year flood and 1 in 2 year flood;
- Some initial modelling work on the channel and floodplain areas to determine the existing flow capacity;
- an initial assessment of the "excess" volume of water needing to be stored if the bed level was raised by 1 to 1.5 m (to the estimated practical depth of 1.0m gained from the topographic survey information – Section River Channel and Floodplain) along the entire channel.

The hydrological work was carried out using FEH methodology. The peak flows generated used in a simplified hydraulic model of the site. The results of both sets of work are described here briefly.

Hydrology

Some Flow Estimation Handbook (FEH) hydrological modelling has been carried out, to estimate the hydrology and provide some hydrographs and flood events at the upstream end, upstream of Capps Bridge, on the Lancing Brook and tributary draining Southwater.
The downstream boundary of such a model would be the gauging station at Hatterell Bridge. There is another tributary which joins the Adur just upstream of the Hatterell Bridge gauging station which drains Southwater. The hydrology of this tributary to the north was assessed using FEH to determine the contribution which that area makes to the flows going through the gauging station.
The following were generated using the FEH method:

1 in 100 year flow hydrograph
1 in 20 or year flow hydrograph
1in 10 year flow hydrograph
1 in 2 year flow hydrograph

These hydrographs were generated for the following locations:

514200 121750 Upstream of the village of Shipley
515050 120900 Lancing Brook – a tributary
516800 120800 The tributary which comes in just downstream of the A24 bridge

517850 119700 Hatterell Bridge gauging station

Figure D1 1 in 100 year hydrographs from FEH analysis

The 2 km study reach is on the River Adur between Shipley and upstream of where the Bay Bridge Tributary joins the River Adur. From the FEH analysis the peak flows at Shipley (the upstream end of the site), just downstream of the Lancing Brook confluence at Tenchford and at Bay Bridge (at the downstream end of the site) are given in table? Location	1 in 100 year peak flow (m3/s)	1 in 2 year peak flow (m3/s)
River Adur, Shipley (upstream end of site)	45	13
River Adur, Tenchford (including Lancing Brook component)	71	21
River Adur, Bay Bridge (downstream end of site)	85	25

Hydraulic analyses

Cross-section data (topographic survey information undertaken by Maltby Land Surveys) was used and overlain with LiDAR data for the floodplain areas. This cross-section data was put into INFOWORKS hydraulic model and a simple model of the river, without bridges, sluices or tributaries constructed to enable a steady state model to be run. A steady state, simple model allows the capacity of the channel to be investigated as well as the impact of, for example, raising the bed level on the water levels. This allows a 'broad brush' investigation, without structures, to see what might be possible.

Based on the simple cross-sections in the model, the bankful capacity of the River Adur between the Lancing Brook tributary and the Bay Bridge is, on average 26 m3/s. Upstream of the Lancing Brook confluence the bankful channel capacity is 23m3/s. Bankful discharge is often approximated to the 1 in 2 year flood. When comparing this value with the 1 in 2 year flood estimate at Bay Bridge of 25 m3/s and just downstream of Lancing Brook, the estimate of 21 m3/s, shows that this model predicts that the channel carries slightly more than the 1 in 2 year flood.

By raising the bed to a depth of 1m (see section 3.2.1) on average 10m3/s conveyance from the channel would be lost which would need to be either stored in an alternative location or placed into another channel, or the water attenuated on the floodplain by planting of floodplain forests etc.

Analysis of the hydrographs show that the volume of water being carried by the **channel** in a 1 in 100 year flood between the Lancing Brook and the Bay Bridge on the River Adur is 2.5Mm3. Therefore the volume on the **floodplain** is 2.3 Mm3 and the total is 4.9Mm3, as stated above. The "extra" volume of water which would be on the floodplain by raising the bed to a depth of 1m would be approximately 0.5Mm3 over the 2km in a 1 in 100 year flood. Therefore the channel would carry 0.5Mm3 less, and the floodplain 0.5Mm3more, water for the 19 hours that the floodwater would be on the floodplain in a 1 in 100 year flood.

This can be considered as a depth of water using Manning's equation, assuming a floodplain width of 100m and a roughness of 0.05. The additional depth of water on the floodplain to carry the 'lost' capacity of 10m3/s would be a uniform 0.19m. This is a very simple approach and in some areas the water would be spread over a larger area. In addition to these hand calculations, INFOWORKS was run for pre and post restoration cases to look at the predicted increase in level when the bed level in the channel is raised by approximately 1.2m. Table 5.4.5.2 shows the increases in maximum water level at the sections for a 1 in 100 year flood.

Section (from survey)	Post Restoration Max Stage (m AD)	Pre Restoration Max Stage (m AD)	Differences Max Stage (m AD)
1.001	7.846	7.846	0
1.002	7.883	7.883	0
1.003	7.886	7.883	0.003
1.004	7.902	7.897	0.005
1.005	7.921	7.914	0.007
1.006	7.944	7.937	0.007
1.007	7.966	7.953	0.013
1.008	7.999	7.981	0.018
1.009	8.028	8.006	0.022
1.010	8.066	8.040	0.026
1.011	8.038	8.021	0.017
1.012	8.239	8.115	0.124
1.013	8.330	8.205	0.125
1.014	8.330	8.205	0.125
1.015	8.396	8.304	0.092
1.016	8.491	8.415	0.076
1.017	8.587	8.520	0.067
1.018	8.743	8.684	0.059
1.019	8.735	8.670	0.065
1.020	8.851	8.798	0.053
1.021	8.938	8.875	0.063
1.022	9.017	8.962	0.055
1.023	9.114	9.062	0.052
1.024	9.175	9.142	0.033
1.025	9.300	9.267	0.033

Table C2. 1 in 100 year increases in water level post works (modelled).

The maximum rise is 0.125m just downstream of the Lancing Brook confluence, with a lower rise of approx. 0.05m between Tenchford and Capps Bridge.

These results are both preliminary and indicative as the model does not include any bridges or structures and it is steady state so does not include any storage areas.

Appendix II - Physical Scale Model Materials

Distorted Froude-Scaled Flume Analysis of Large Woody Debris (Wallerstein, et al., 2001)
In each run, the flume bed was covered with sand 0D8 mm in diameter and a trapezoidal channel was created by pre-wetting the sand and then cutting the trapezoidal section into the length of the flume with a scraper plate. The scraper was machined to dimensions scaled down from the prototype (see below), and attached to a carriage mounted on top of the flume. Excess sand was carefully removed and the bed slope was adjusted to the desired level using the calibrated jack system. Prior to each test run, the movable-bed channel was reshaped and smoothed, and the design discharge run for two hours prior to testing to ensure that the boundary was stable.

Bed load transfer and channel morphology in braided streams (Ashmore, 1987)
The model bed material was coarse sand and gravel with D of 1.16 mm, D of 4 mm, D of 0.35 mm and D /D equal to 6.4. The sediment sorting is comparable with that of intermediate size alluvial gravel and the approximate geometric scale is 1:30. The flume used is 10 m long by 2 m wide, with adjustable slope. The water supply was circulated from the laboratory sump, while sediment was recirculated within the flume using a small diaphragm pump. Sediment deposited in the tail box of the flume was withdrawn, along with some water, and piped to the head of the flume where it was injected into the flow via four nozzles arranged across the width of the head section of the channel.

The "unreasonable effectiveness" of stratigraphic and geomorphic experiments (Paola, et al., 2009)
The first system for producing experimental stratigraphy with controlled, spatially variable subsidence was developed at St Anthony Falls Laboratory beginning in 1996. The experimental stratigraphy basin is called the Experimental EarthScape (XES) system, though it is more often referred to as "Jurassic Tank". In XES subsidence is produced by filling an experimental basin with granular material and extracting the material through a honeycomb of hexagonal cells in the basin floor (Paola, 2000; Paola et al., 2001) (Fig. 8). The internal friction of the granular material (pea gravel in this case) prevents uncontrolled release and allows the basement to support high lateral subsidence gradients, but its fluid-like properties smooth out the cell boundaries, producing a continuous basement surface. The pea gravel is extracted through the bottoms of the cells by precisely controlled fluid pulses that knock aliquots of gravel out of the cell base and draw the basement surface down. The pulses are continuously calibrated to produce about 0.1 mm of subsidence per pulse. A pulse pattern is fired about every 2min, so that the maximum subsidence rate is several mm/h. The granular basement is covered with a flexible membrane that stretches and unfolds as the basement deforms. The experimental deposit is developed on top of this membrane by supplying the system with water and sediment from one or more input points and, if desired, manipulating base level.

Appendix III - River Blackwater

Physical Model of River Blackwater (Rameshwaran, et al., 2005)

Only brief details of the 1:5 scale physical model is given here; a full description of the experimental set up has been described by Lambert and Sellin (1996). The undistorted 1:5 scale model of the River Blackwater was constructed in the 56 m long and 10 m wide UK Flood Channel Facility flume, as shown in Figure 1. The detail of the channel geometry and the location of the cross sections (sections 3, 4 and 5) where flow field measurements were taken, are illustrated in Figures 2 and 3. The channel surfaces were composed of smooth cement mortar. The main channel had a sinuosity of 1.18, whilst the flood channel had a sinuosity of 1.06. The longitudinal valley slope was 1×10-3. The shape of the main channel cross-section was trapezoidal as shown in Figure 3. The main channel bank slopes were 45o with a bank-full depth of 150 mm. Experiments were carried out with different roughness conditions. The roughened main channel and floodplain surfaces were obtained by placing a layer of gravel on the smooth channel surfaces. The inclined walls were left smooth for all experimental runs. The floodplains were either horizontal or at an inclination of 1 in 30 (Figure 3). Only horizontal floodplain cases are considered in this study.

Figure 1. The River Blackwater physical model.

Appendix IV - Calculating Froude Numbers

Example of table from Excel used for calculating Froude numbers

Q [m^3/s]	d [cm]	d [m]	B [m]	b [m]	α	Sin(α)	Bank [m]	A [m^2]	F	P_w [m]	R [m]	V [m/s]
0.000354	1.04	0.01040	0.122	0.080	26.565	0.447	0.023	0.00105	1.0559	0.127	8.29E-03	3.37E-01
0.000354	1.14	0.01140	0.126	0.080	26.565	0.447	0.025	0.00117	0.9021	0.131	8.95E-03	3.02E-01
0.000354	1.14	0.01140	0.126	0.080	26.565	0.447	0.025	0.00117	0.9021	0.131	8.95E-03	3.02E-01
0.000354	1.05	0.01050	0.122	0.080	26.565	0.447	0.023	0.00106	1.0387	0.127	8.35E-03	3.33E-01
0.000354	0.99	0.00990	0.120	0.080	26.565	0.447	0.022	0.00099	1.1482	0.124	7.95E-03	3.58E-01
0.000354	1.07	0.01070	0.123	0.080	26.565	0.447	0.024	0.00108	1.0058	0.128	8.49E-03	3.26E-01
0.000354	1.17	0.01170	0.127	0.080	26.565	0.447	0.026	0.00121	0.8626	0.132	9.14E-03	2.92E-01
0.000354	1.2	0.01200	0.128	0.080	26.565	0.447	0.027	0.00125	0.8257	0.134	9.34E-03	2.83E-01
0.000354	1.1	0.01100	0.124	0.080	26.565	0.447	0.025	0.00112	0.9592	0.129	8.68E-03	3.15E-01
0.000354	1.04	0.01040	0.122	0.080	26.565	0.447	0.023	0.00105	1.0559	0.127	8.29E-03	3.37E-01
0.000354	1.15	0.01150	0.126	0.080	26.565	0.447	0.026	0.00118	0.8887	0.131	9.01E-03	2.98E-01
0.000354	1.04	0.01040	0.122	0.080	26.565	0.447	0.023	0.00105	1.0559	0.127	8.29E-03	3.37E-01
0.000354	1.1	0.01100	0.124	0.080	26.565	0.447	0.025	0.00112	0.9592	0.129	8.68E-03	3.15E-01
0.000354	1.1	0.01100	0.124	0.080	26.565	0.447	0.025	0.00112	0.9592	0.129	8.68E-03	3.15E-01
0.000354	1.05	0.01050	0.122	0.080	26.565	0.447	0.023	0.00106	1.0387	0.127	8.35E-03	3.33E-01
0.000354	1.25	0.01250	0.130	0.080	26.565	0.447	0.028	0.00131	0.7692	0.136	9.66E-03	2.69E-01
0.000354	1.21	0.01210	0.128	0.080	26.565	0.447	0.027	0.00126	0.8139	0.134	9.40E-03	2.80E-01
0.000354	1.27	0.01270	0.131	0.080	26.565	0.447	0.028	0.00134	0.7483	0.137	9.79E-03	2.64E-01
0.000354	1.05	0.01050	0.122	0.080	26.565	0.447	0.023	0.00106	1.0387	0.127	8.35E-03	3.33E-01
0.000354	1.09	0.01090	0.124	0.080	26.565	0.447	0.024	0.00111	0.9744	0.129	8.62E-03	3.19E-01
0.000354	0.93	0.00930	0.117	0.080	26.565	0.447	0.021	0.00092	1.2765	0.122	7.54E-03	3.86E-01

Appendix V - River Adur Restoration Design (Lower Section)