UNIVERSITY OF CAMBRIDGE
Department of Engineering

Surveying and Hydraulic Modelling of River Cam

Author Name: Cheuk Yu (Jackie), YIM

College: Newnham College

Supervisor: Dr. Dongfang, LIANG

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I hereby declare that, except where specifically indicated, the work submitted herein is my own original work.

Signed _______________________________ *Date* ______________________________

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1. Summary

In this project, a 4km river model of River Cam was built, 1D steady flow analysis was performed using HEC-RAS software and the model was verified by surveyed data. During the model construction process, different hydraulic parameters were investigated. Firstly, the sensitivity of model to geometric specification was tested and it was concluded that, for low flowrate simulations, the shape of river cross-sections has low influence, a rectangular shape of river can be assumed. Secondly, the significance of contraction and expansion effect on energy loss were examined. It was found that, the greatest contribution of expansion loss is at the section with a substantial increase in cross-sectional area, the energy loss also increases with flowrate. Finally, the computation result was compared with the surveyed data. The discrepancies between the two results laid within the acceptable range of error and therefore it was concluded that the river model accurately represents the actual river network which can be used for further study.

2. Background

2.1 River Cam

River Cam is the main river flowing through Cambridge city located in Eastern England. As a significant river for rowers, punters, students and tourists in Cambridge, the flow behaviour of this river is of great interest and it also affects the well-being of the neighbouring residents and asset owners.

Looking at the past record, River Cam has a history of moderate flooding and in fact, the most significant flood that has been recorded in recent years was the one occurred in October 2001. From 21st to 23rd October, there was a heavy rain in Cambridge. On 21st October, 3.5 inches of rainfall was recorded in a single day and it caused severe flooding in the downstream of River Cam. 31 houses were flooded and a lot more properties were damaged by this flood event. Regarding to this flood event, different parties had carried out flood analysis on River Cam. For instance, the Environment Agency produced a flood map[1] for reference and the University of Cambridge also cooperated with the Ambiental Technical

Solutions Ltd. to develop a raster based flood inundation modelling tool[2]. Results produced from the Ambiental Report suggested a few limitations on their tools, such as capturing the hydraulic structures along the river. This suggested that there is a need to build an improved model of River Cam and to carry out effective river analysis, which is the main aim of this project.

2.2 Hydraulic Modelling Software – HEC-RAS

Two free river modelling and simulation software are available online for download, namely Flood Modeller and HEC-RAS. The Flood Modeller is a software that the Environment Agency is currently using for its ongoing River Cam Flood Mapping Improvements project. From the River Cam Flood Mapping Report[3] published by the Environment Agency in February 2012, a full river model of the Cam is provided in the format of Flood Modeller. In order to explore another form of river model, HEC-RAS was chosen to be used in this project and a river model was built and tested. HEC-RAS (Hydrologic Engineering Center's-River Analysis System) developed by the Hydrologic Engineering Center[4] can perform one-dimensional steady flow, one and two-dimensional unsteady flow calculations, sediment transport/ mobile bed computations, and water temperature/ water quality modelling. It could be used for further assessment of River Cam after a valid river model is established.

The aim of the project is to build an accurate model of River Cam in HEC-RAS, so as to model flood events or carry out different scenario studies such as riverbed dredging or malfunction of hydraulic structures in future.

3. Hydraulic Modelling

3.1 Model Building

The modelling software HEC-RAS Version 5.0 was downloaded from its official website in October 2016. Two river models were built with HEC-RAS in this project. The first model was a simplified version and was built to investigate the necessity of an accurate representation of channel cross-sections. The second model was a detailed version with a more precise cross-section data based on the available model in Flood Modeller format from River Cam Flood Mapping Report by Environment Agency. The hydraulic structures built in both models were identical.

First of all, the method of building a river model with this software is to first draw a river reach and define points of cross-sections along the reach, which are called river stations. At each river station, cross-section data should be specified. It includes the position of left-hand bank, right-hand bank, bed level, reach lengths to next river station, manning's n-value and expansion coefficient, etc., the details of these parameters are covered in section 3.2 where the principles of modelling are explained. After specifying a river reach, hydraulic structures such as bridges, weirs, gates, culverts, etc. at different locations can also be specified to give greater details to the river model.

In this project, 1D steady flow simulation was carried out on the two models to investigate the water surface profile along River Cam. The theoretical principles, the specification of parameters and computation procedures will be covered in the following sections in order.

3.2 Theoretical Basis

3.2.1 Energy Equation

After forming a river model, a 1D steady flow analysis was performed to obtain water surface profiles. Comparing the simulated and measured values, the accuracy of the model could be determined. HEC-RAS uses Energy Equations for basic water surface calculations. The relevant equations are extracted from the HEC-RAS Hydraulic Reference Manual[5]. It should be noted that all equations presented in this report are in standard US units.

The Energy equation:

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e \quad (3\text{-}1)$$

$$h_e = L\overline{S}_f + C\left|\frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g}\right| \quad (3\text{-}2)$$

Where:
Z_1, Z_2 = elevation of the main channel inverts
Y_1, Y_2= depth of water at cross-sections
V_1, V_2= average velocities (total discharge per flow area)
a_1, a_2= velocity weighting coefficients
g= gravitational acceleration
h_e= energy head loss
L= discharge weighted reach length
$\bar{S}_f$ = representative friction slope between two sections
C= expansion/ contraction loss coefficient

The Energy head loss (h_e) is comprised of both friction loss and contraction or expansion losses. From the equation (3-2), it is shown that, the first half contains the friction effect between two sections and the second half contains the contraction and expansion effect.

HEC-RAS requires the flow to be subdivided into units that the velocity is uniformly distributed. It subdivides flow in the overbank areas at locations where our input Manning's n-values change, and by solving equation (3-3) and (3-4), the flow and conveyance can be calculated in each subdivision.

$$Q = KS_f^{1/2} \quad (3\text{-}3)$$

$$K = \frac{1.486}{n} A R^{2/3} \qquad (3\text{-}4)$$

Where:
Q= flowrate
K= conveyance for subdivision
n= Manning's roughness coefficient for subdivision
A= flow area for subdivision
R= hydraulic radius for subdivision (area/ wetted perimeter)

The total conveyance for the cross-section is the sum of the three subdivisions conveyances on left overbank, right overbank and main channel. However, in this river model, overbank is not modelled therefore the overbank parts could be omitted and just the main channel calculation is considered. HEC-RAS has several ways to determine the Manning's coefficient n, for the river channel. It computes a single composite n-value for the whole channel depending on the n-value specified at every cross-section.

The contraction or expansion coefficient, C, defines the degree of loss. If the velocity head downstream is greater than that upstream, a contraction is assumed. Similarly, if the velocity head upstream is greater than that downstream, an expansion is assumed in the flow. Typical values are provided by HEC-RAS and appropriate values are chosen and input at each river station.

3.2.2 Hydraulic Structures

There are several hydraulic structures along the river network. Most of them are bridges and the most significant structures that affect the water surface profile are the sluice gates at Mill Pond and the sluice gates and lock at the back of Jesus Green.

Bridges

Bridges that span across the bank such as Silver Street Bridge, do not have its structures submerged in water. They do not have direct interference with water flow so they have less effect on water surface profile in the simulation. Three arched bridges, such as Trinity College Bridge, which have pillars extending through the riverbed at the middle of cross-sections, can cause contractions and expansions to flows.

Four cross-sections have to be defined in order to compute the energy losses when modelling a bridge structures in HEC-RAS. A picture of these four cross-sections is shown in Fig.3.1. Cross section 1, is located sufficiently downstream such that the flow has fully expanded and not affected by the bridge structure at all. Cross section 2, is located a short distance downstream of the bridge. It should be placed far enough from the downstream face of the bridge deck such that flow expansion is accommodated. Cross section 3 is located a short distance upstream from the bridge. It should be placed far enough from the upstream face of bridge deck such that abrupt contraction of flow can occur to reach the bridge openings. Cross section 4 is located upstream from the bridge where the flow lines are considered to be parallel, in other words, at a location upstream where the bridge structure has not caused contraction of flow.

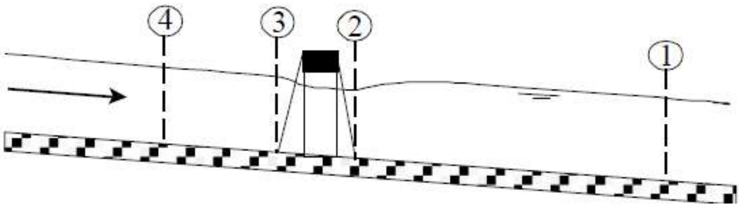

Fig. 3.1. The cross-section locations for bridge modelling[5]

As shown in Fig. 3.1, the flow contracts from section 4 to 3 and expands from section 2 to 1 when it passes through a bridge. The contraction and expansion losses, are captured by specifying the Contraction and Expansion Coefficient. The loss due to expansion of flow usually is greater than that due to contraction, and losses from abrupt transition is usually greater than that from gradual transition. HEC-RAS user manual suggests that typical bridge sections in subcritical flow, which is the case in this river model, should use a contraction coefficient of 0.3 and expansion coefficient of 0.5.

For the losses through the bridge (from section 3 to 2), Energy Equation is used for computation. The method of calculation is essentially similar to the Energy Equation described in equation (3-1) for natural river cross-section. The area of bridge is subtracted from the total area and the wetted perimeter is increased due to the extra pillars from bridge structures. The programme also formulates two additional cross-sections within the bridge structure, section BD (bridge downstream) and BU (bridge upstream) based on the bridge geometry and section 3 and 2 (See Fig. 3.2). Standard energy equations are then solved from section 2 to BD, BD to BU, and then BU to 3.

The Manning's coefficient and expansion/ contraction coefficient have to be specified for energy-based calculation. The n value and C values as mentioned above are determined from a set of suggested values.

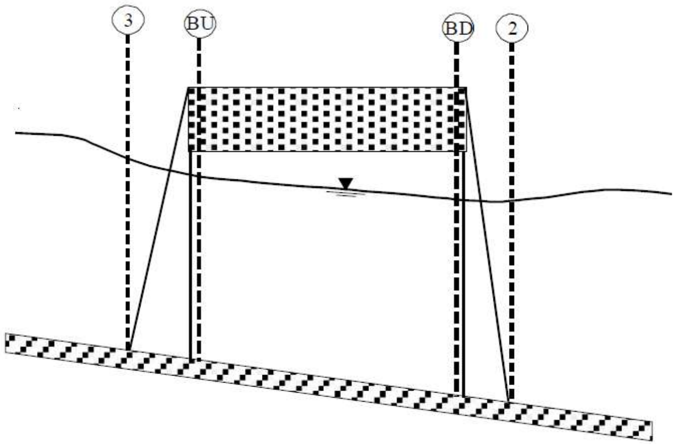

Fig. 3.2. Cross-sections near and inside a bridge structure[5]

Sluice Gates and Weir

HEC-RAS can model gated spillways by setting the correct geometric data. Sluice gates are modelled as vertical lift gates. A schematic view of sluice gate with a broad crest is shown in Fig. 3.3.

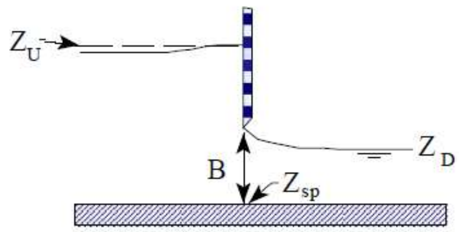

Fig. 3.3. The cross-section of sluice gate modelling[5]

$$Q = C W B \sqrt{2gH} \qquad (3\text{-}5)$$

Where:
C= Discharge Coefficient

B= Height of gate opening
H= Upstream energy head above spillway crest (Z_u-Z_{sp}).

The calculation for flow across a sluice gate is shown in equation (3-5). When the downstream tailwater increases to a level where the gate is not flowing freely, the programme switches to equation (3-6).

$$Q = CWB\sqrt{2g3H} \tag{3-6}$$

Where:
H= Z_u-Z_D

When tailwater depth above the spillway divided by the headwater energy above the spillway H is greater than 0.67, submergence is treated to be occurring. This equation is used for transition from free flow of water to fully submerged flow over sluice gate.

Usually, sluice gates are located over a weir, when specifying dimensions for the gate itself, the weir should be included as well. If the water surface upstream is lower than the top of gate opening, water flows through gate as a weir flow and this is modelled by weir equation:

$$Q = CLH^{3/2} \tag{3-7}$$

Where:
C= Weir Coefficient
L= Length of weir
H= Upstream energy head above weir

HEC-RAS defines a range of values for Weir Coefficient depending on the shape of weir. For Broad Crested weir, the range is 2.6 -3.1; for Ogee Crested weir, it is 3.2 - 4.1 and for Sharp Crested weir, it is 3.1 -3.3.

Culverts

The energy losses due to culverts are separated into three parts. Firstly, it is the loss in downstream from the culvert due to expansion. Secondly, it is the loss due to friction through the culvert. Thirdly, it is the losses at upstream of culvert where contraction occurs. In HEC-RAS, the computation of upstream and downstream losses are similar to that of bridges as mentioned in Bridge section. The cross-sections and loss coefficients are specified in the same way. The programme can model different shapes of culverts such as elliptical, circular or box. The total head loss through culvert is the sum of entrance losses, friction losses and exit losses. The equation for each loss is shown in equation (3-8), (3-9) and (3-10) respectively.

For entrance loss,

$$h_{en} = k_{en} \frac{V_{en}^2}{2g} \quad (3\text{-}8)$$

Where:
h_{en}= Entrance head loss
k_{en}= Entrance loss coefficient
V_{en}= Flow velocity inside culvert at entrance

In equation (3-8), the entrance loss is the multiple of velocity head and the loss coefficient k_{en}, suggested values for k_{en} could be found in Reference Manual[5] Table 6-3 to 6-5 and this value is dependent on the type of structure and entrance design of the culvert chosen.

For friction loss,

$$h_f = L\left(\frac{Qn}{1.486AR^{2/3}}\right)^2 \quad (3\text{-}9)$$

Where:
h_f= Friction loss
L= Culvert length

Equation (3-9) is the Manning's equation with suggested n-values provided by HEC-RAS Reference Manual[5], the choice of n-value has to be adjusted according to culvert condition.

For exit loss,

$$h_{ex} = k_{ex}\left(\frac{a_{ex}V_{ex}^2}{2g} - \frac{a_2V_2^2}{2g}\right) \quad (3\text{-}10)$$

Where:
h_{ex}= Exit energy loss
k_{ex}= Exit loss coefficient
V_{ex}= Velocity inside of culvert at exit
V_2= Velocity outside of culvert at downstream cross-section

The exit coefficient k_{ex} is set to 1.0 for sudden expansion. This value can vary from 0.3 to 1.0 and a lower value should be adopted for less abrupt outlet expansion.

3.3 Data Input

3.3.1 Geometric

Manning's roughness coefficient, n-value

As suggested in section 3.2, an appropriate n-value is needed for HEC-RAS to calculate the water surface elevation using Energy Equations. HEC-RAS provides a table of typical values of Manning's coefficient in different channel conditions which can be found in Chapter 3 Table 3-1 in HEC-RAS Reference Manual[5]. The table of suggested values is shown in Appendix A. The Manning's coefficient adopted was 0.05, this corresponds to a channel that is clean, winding, with some pools, shoals, weeds and stones, which is believed to be the right description of the bed of River Cam. An n-value of 0.05 was used throughout the whole river network on the majority of cross-sections. According to previous literatures on the Ambiental Report[2] and the River Cam Flood Mapping Report[3] by Environment Agency, adjustments of n-value were made on certain cross-sections.

Expansion/ contraction loss coefficient, C- values

As mentioned in section 3.2.1, the choice of C values depends on the how the flow contracts or expands through the cross-section, it will affect the energy head loss and therefore the water surface elevation calculated. HEC-RAS suggests a few coefficients value depending on the degree of transition, it is shown in Table 3-3 in the Reference Manual[5]. The table of suggested values is shown in Appendix A. A gradual transition is chosen in this model so a contraction coefficient of 0.1 and expansion coefficient of 0.3 were input.

3.3.2 Steady Flow Data

In order to run a steady flow analysis, a set of data has to be input. Boundary conditions should be set-up in a way that it allows water surface to be established at both the upstream and downstream end of the river network. There are four types of boundaries conditions that can be set in HEC-RAS. Firstly, it is the Known Water Surface Elevations, the water surface level at the ends of the river reach can be inserted and the programme calculates the rest of the profile. Secondly, it is the Critical Depth, it means the programme automatically calculates the critical depth and applies it to the end of reach as boundary condition. Thirdly, it is the Normal Depth, an energy slope has to be input and the programme uses Manning's formula to compute the normal depth. Fourthly, it is Rating Curve, a set of elevation versus flowrate data is inserted and a rating curve will be plotted. This curve gives the relationship between flowrate and water level by interpolation so the programme can pick corresponding elevation from a given flowrate.

In the River Cam Flood Mapping Report by Environment Agency, standard water level retained upstream of each sluice gate is suggested. Since the water surface elevation is known, the first type of boundary condition are used and elevations at upstream and downstream end are specified.

Then, the flow data is to be input in the programme. The Shoothill GaugeMap[6] website contains information of all available gauges in the UK that measures flowrates of rivers. However, main stem of River Cam that is being investigated does not have a gauging station. Instead, gauging stations are available in three tributaries of River Cam, namely River Rhee, River Granta and River Cam. The location and official station number are tabulated in Table. 3.1.

Gauging Station	**Station No. (Shoothill GaugeMap)**
Rhee at Burnt Mill	33021
Cam at Dernford	33024
Granta at Stapleford	33053

Table. 3.1. Information of the three gauging stations at tributaries of River Cam

It is assumed that the flowrate of River Cam is sum of flowrates of the three tributaries. The flowrate at a particular hour of a particular day could be obtained from Shoothill GaugeMap for each gauging station. Averaging out, it was found that the usual flowrate of River Cam ranges from 0.8 m^3/s to 2.5 m^3/s. The average flowrate on the survey day was found to be 1.6 m^3/s. So, a flowrate of 1.6 m^3/s was first used in the simulation so as to produce a result for comparison with surveyed data.

3.4 Computation Procedure

HEC-RAS runs the 1-D steady flow simulation in five steps and iterates. First, it assumes a value for the water surface level at an upstream cross-section. It then calculates the total conveyance and velocity head from the assumed water surface level. Substituting the values into equation (3-2), the total energy head loss is calculated. Finally, using the energy equation (3-1), the water surface is computed. The programme compares the computed value with the initial assumed value of the water surface, and iterates until they agree to within 0.003m. The programme has a maximum number of iterations for balancing the water surface in order to keep a reasonable computation time. If the maximum number of iterations has exceeded before reaching a balanced water surface elevation, the programme will calculate the critical depth. While it is iterating, it keeps a record of the smallest error obtained between the assumed and computed values, the corresponding water surface level is called the minimum error water surface. The program will then check the whether this error obtained is less than a predefined tolerance, which is 0.1m by default. If the minimum error falls within this tolerance, this minimum value is accepted to be the final answer. On the other hand, if the minimum error is greater than our tolerance, the programme automatically uses the critical value as the final answer for water surface.

Neither the minimum error water surface nor the critical water depth gives an ideal solution, they are applied in order to complete the computation so a warning message will be prompted when either is used. When the programme is not able to balance the energy equation, it is mainly due to insufficient number of cross-sections. When this happens, extra cross-sections with shorter intervals will be added by interpolation.

3.5 River Models

3.5.1 Simplified Model

In the first model, previously surveyed data provided by Environment Agency and Cam Conservancy was used[7]. In these drawings, the river bed level, left-hand bank level and right-hand bank level and bankwidth are given at each cross-section at 50m interval along the reach. With these data, a basic rectangular shape was assumed and an 8km reach spanning from the South of Coe Fen upstream to Baits Bite Lock downstream was built, consisting of 166 cross-sections. Each cross-section was numbered from upstream to downstream end.

Digital Elevation Models (DEM) information was needed in order to georeference this river model. Light Detection and Ranging (LiDAR) based DEM values were obtained from the Environment Agency[8]. In the LiDAR system, light pulses from laser travelled from aircraft to the ground and rebounded back to sensor to allow distance to be measured. According to the Environment Agency, this technique produces up to 100,000 measurements per second, allowing highly detailed terrain models to be generated at a range of spatial resolutions. The Digital Terrain Model (DTM) data for Cambridge area was available at 1m or 2m spatial resolution. Since DTM data at 2m spatial resolution had a greater coverage, the data at 2m resolution was used. DTM data for Cambridge and nearby areas, such as Chesterton and Grantchester, was obtained and a Terrain model was built using a geographic information system (GIS) software, QGIS. The terrain model was then inserted into HEC-RAS. The 8 km-long section of River Cam was then aligned with the terrain and gave a clearer image. The terrain was used for pinpointing river stations and is useful in further study for flood mapping. The schematic view of the river network with terrain background inserted is shown in Fig. 3.4 below. A simulation was performed on this model.

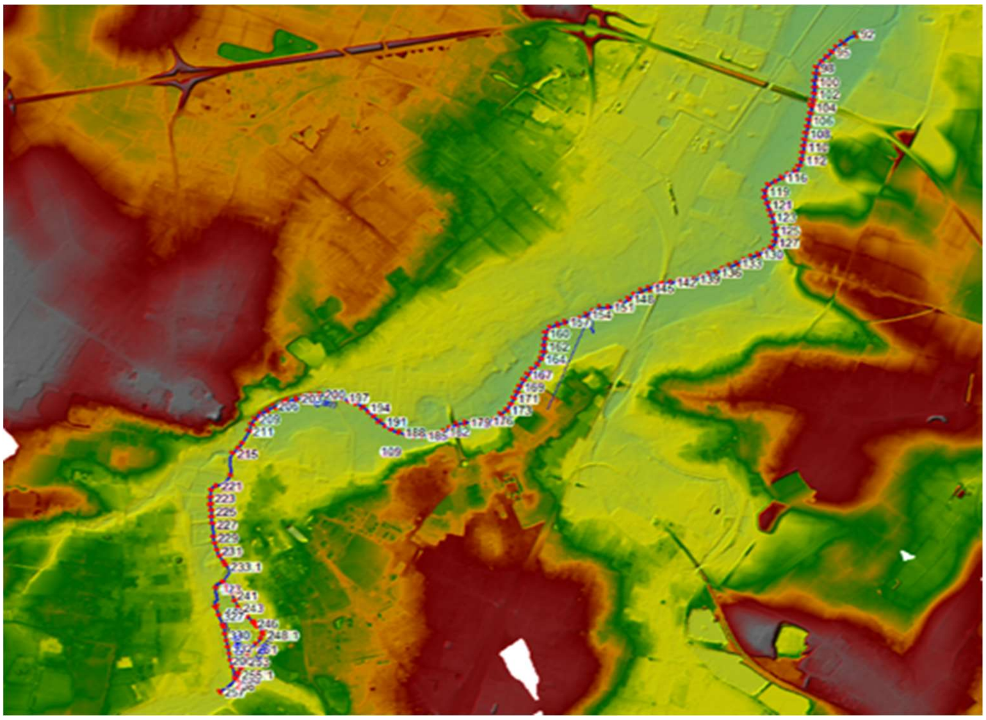

Fig. 3.4. The Simplified Model of 8km reach in HEC-RAS with terrain

3.5.2 Detailed model

Another model was built based on the model of River Cam provided by the Environment Agency in their River Cam Flood Mapping Report. This river model is in the format of another hydraulic modelling software, Flood Modeller. It consists of information of the channel with greater details. Using information of these cross-sections, a river network was rebuilt in HEC-RAS. The river network consists of seven reaches, namely Cam 1, Cam 3, Cam 2, Millpond, Millpond 1, Millpond 2 and Backs. The exact locations of these reaches are shown in Fig. 3.5. The whole river network is a 4km channel spanning from Southern End of Coe Fen at upstream to Elizabeth Way at downstream. The cross-section of river station No. 22770, which is located at Fen Causeway in Coe Fen is shown in Fig. 3.7. Comparing to the same cross-section in the simplified model in Fig. 3.6, the shape of cross-section here is more precise, rather than a simple rectangular shape. Another simulation would be run using this model and compared with the previous simplified model. The simplified model is an 8km reach, therefore the extra 4km at downstream was extracted to keep the consistency of total channel distance.

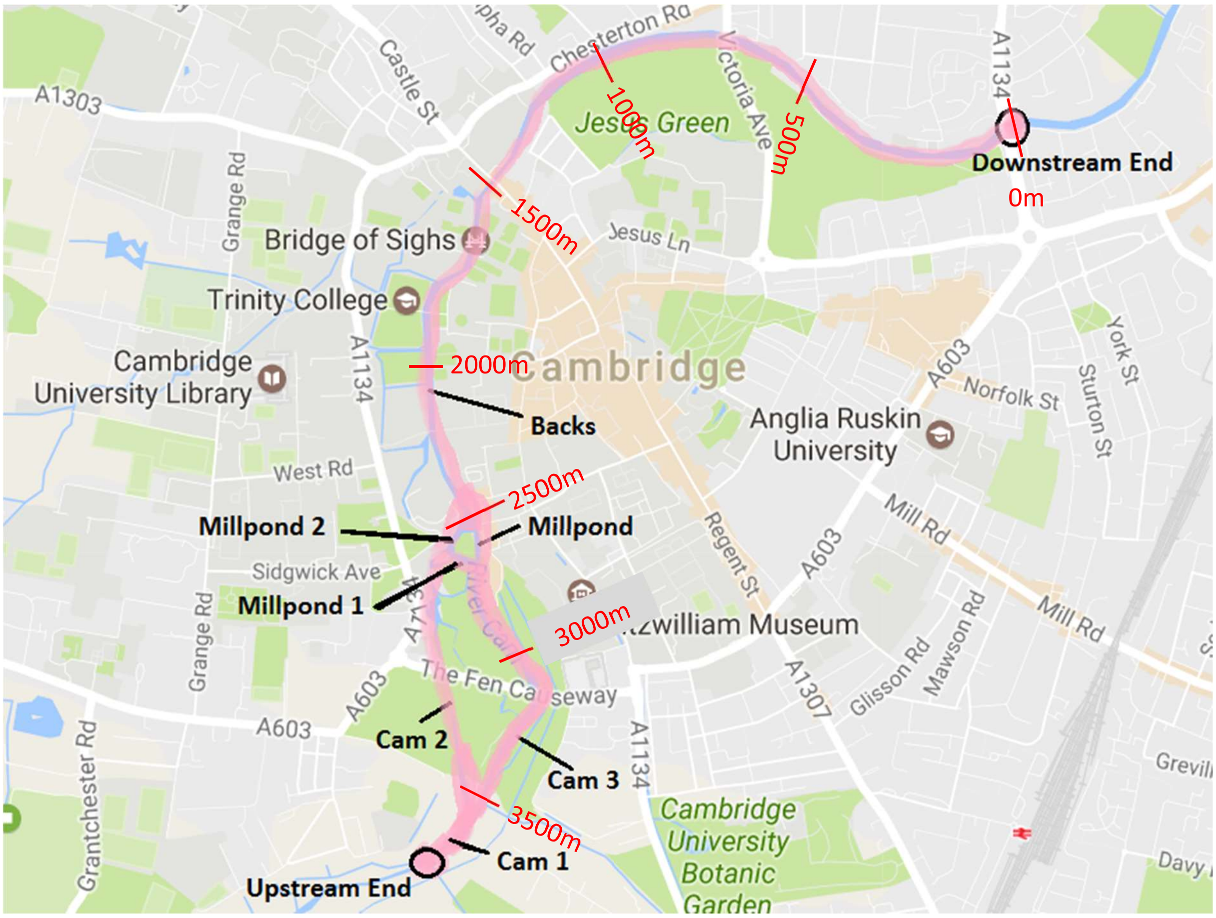

Fig. 3.5. The 4km river network, names of the reaches and main channel distance

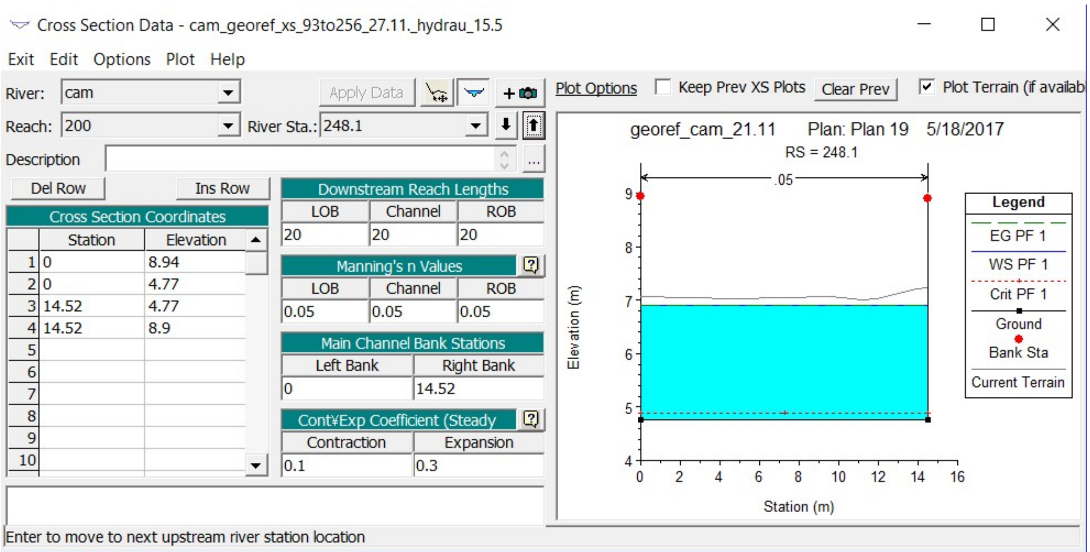

Fig. 3.6. The simplified rectangular cross-section geometry of river station No.22770 at Fen Causeway

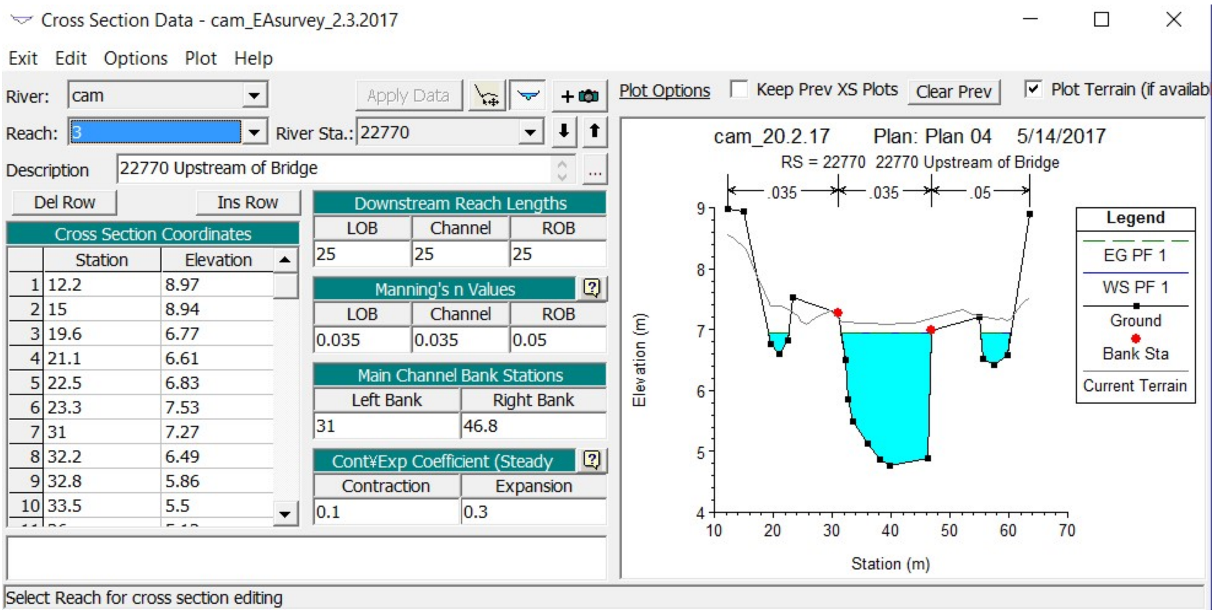

Fig.3.7. The detailed cross-section geometry of river station No.22770 at Fen Causeway

3.5.3 Parameter Setting

From the reach Millpond to Backs, there is a weir followed by a culvert. This weir was modelled as broad crested, with a weir coefficient of 2.6. The culverts consist of two arch openings with concrete linings. The entrance loss coefficient suggested was 0.5 for concrete pipe with a square cut end. The exit loss coefficient was 1 since the outlet expansion was observed to be relatively abrupt. The Manning's n-value used for the culverts was 0.01 as the sides have a lower roughness comparing to the sandy riverbed. The image of the weir and the cross-section of the weir in HEC-RAS are shown in Fig. 3.8 and that of culverts are shown in Fig. 3.9.

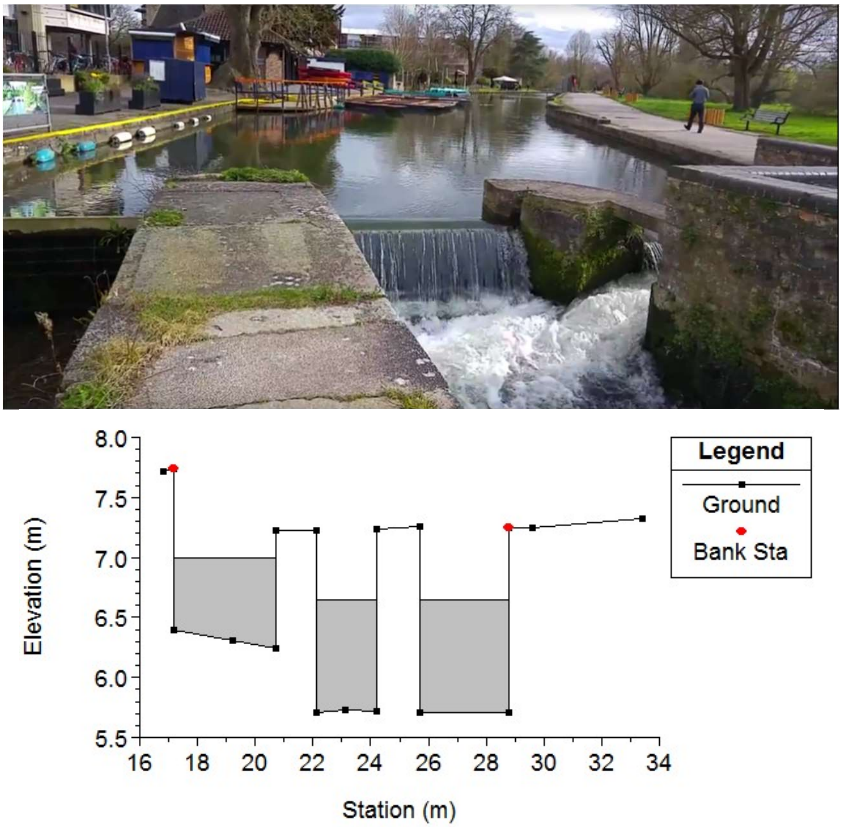

Fig.3.8. The image of weir at Mill pond (above) and the model of it in HEC-RAS (below)

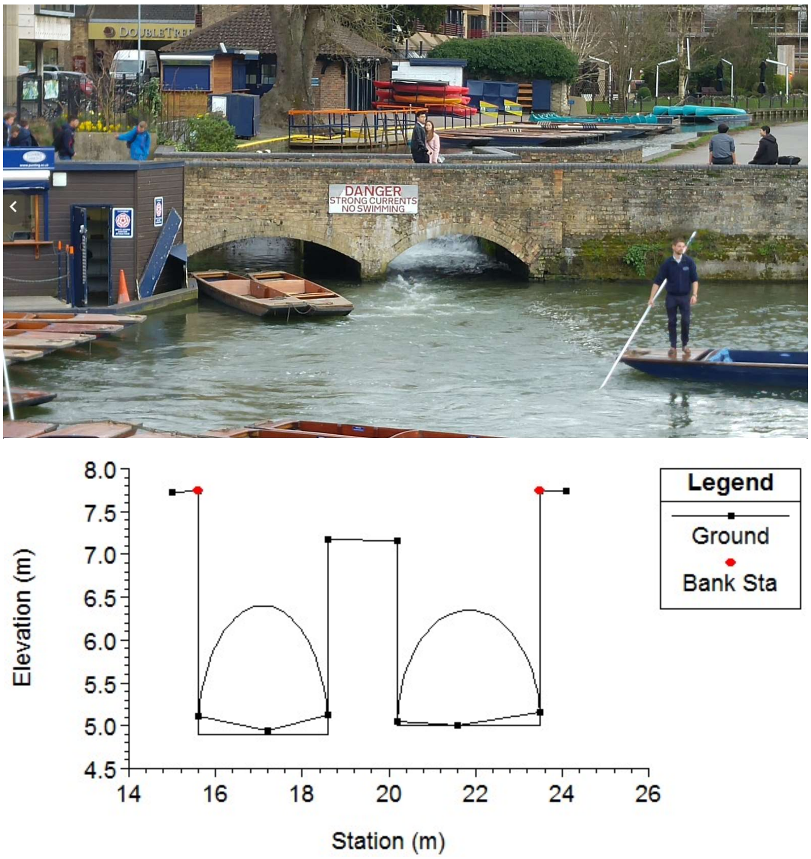

Fig.3.9. The image of culvert at Mill pond (above) and the model of it in HEC-RAS (below)

From the reach Cam 3 to Millpond 1, there is a sluice gate structure with four openings. The sluice discharge coefficient is a measure of how efficiently the flow enters the gate openings, the typical range is from 0.5 to 0.7. 0.7 was used to model these four gates. Again, the weir at the sluice gate was modelled as broad crested, with a weir coefficient of 2.6. The image of sluice gate and its model in HEC-RAS is shown in Fig. 3.10.

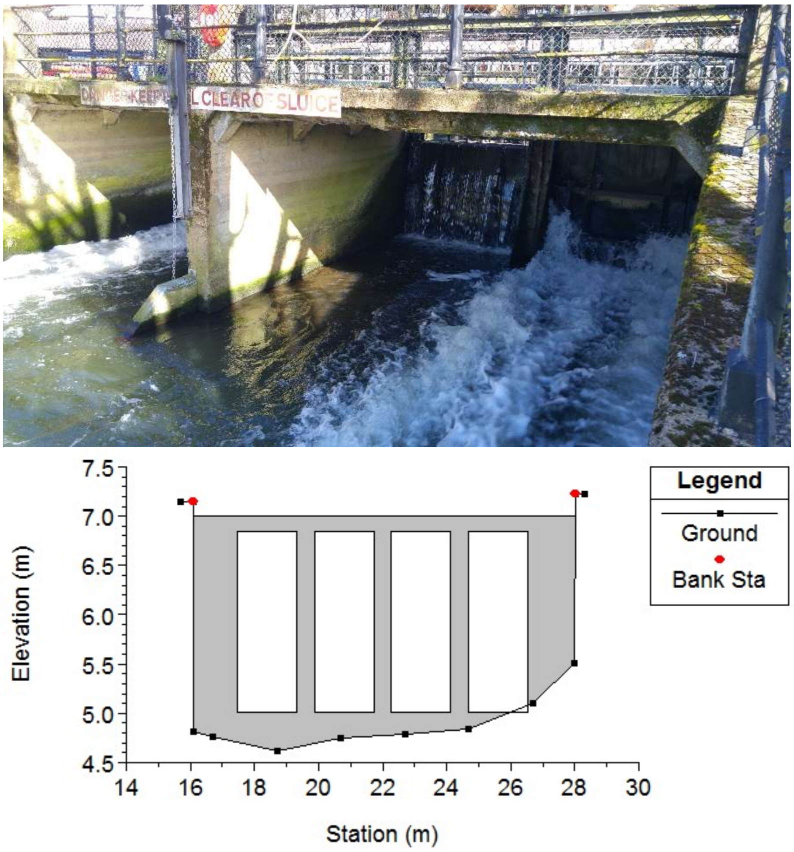

Fig. 3.10. The image of sluice gate at Mill pond (above) and the model of it in HEC-RAS (below)

Along the reach Backs, another sluice gate is encountered behind Jesus Green, together with Jesus Lock. This hydraulic structure consists of two sluice gates and a lock, with an ait in the middle that sits on the riverbed and was modelled as a big block of object that diverts the water into two streams. The two gates on the left are sluice gates, which have their openings varied according to the flowrate of the river. The gate on the right is a lock which is kept shut at all time regardless of flowrate. The lock is only open when transporting boats from one side to another. Since there is a water level difference across the lock, vessels intending to travel to another side have to be raised or lowered through this lock. This lock contains two gates. Initially, both gates are shut and the space within the two gates are adjustable. When a boat has to travel from downstream to upstream, it first enters through the lower gate, then the gate is shut and the boat is “locked”. The space is filled with water from upstream. When the water level of the lock reaches upstream level, the upper gate is opened and the boat exits through that gate. The picture of Jesus Lock with a vessel exiting the upper gate, and the corresponding cross-section of this hydraulic structure in the model is shown in Fig. 3.11.

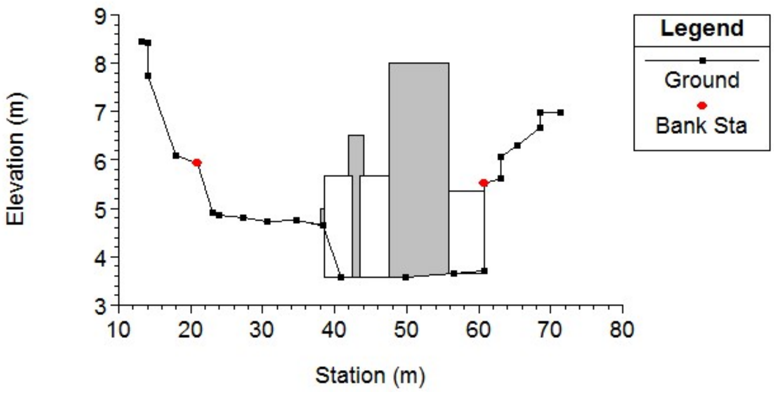

Fig.3.11. The image of Jesus Lock and a moving vessel (above) and the model of it in HEC-RAS (below)

3.6 Modelling Results and Comparison

Determination of gate openings

First of all, the settings of the gate openings that could best represent the hydraulic structures were investigated. The boundary conditions at upstream and downstream are both set to the known water surface level. The openings of sluice gates at Mill Pond and Jesus Lock should be specified. They can be fully or partially opened and the height of each opening can be set. Simulation was first done on the detailed model since it could generate a more precise result. Different combinations of the gate openings were tried and the results were compared in order to obtain the ideal configuration on a normal flowrate day with inflow of 1.6m^3/s.

A steady flow simulation was run with the Mill Pond sluice gates and Jesus Lock sluice gates all closed. The water surface profile was plotted by the programme and is shown in Fig. 3.12. The names of the reaches are presented on the top. The graph reads from right to left, upstream to downstream. The profiles for reaches Cam 1, Cam 3, Millpond 1, Millpond 2 and Backs are plotted. Reach Cam 2 and Millpond are overlapped with Cam 3 and Millpond 2 therefore they are not shown to maintain readability of the following graphs. The blue area represents the water level, the red dotted line represents the left-hand bank level, the green dotted line represents the right-hand bank level, and the spikes represent the bridges that are modelled along the river. The main hydraulic structures, Mill Pond weir and culverts, are located at main channel distance, x= 2800m. Jesus Lock is located at x= 1100m. Three major water levels are observed from the simulation. The first one, from upstream end to Mill Pond (main channel distance 2800m to 3800m), water level is computed to be 7.92m. As shown in the plot, this upstream water level will lead to flooding in Coe Fen region as it is higher than the bank levels. Across the sluice gates at Mill Pond, a substantial drop of 2.72m is calculated and the subsequent water level is found to be 5.20m all the way along the backs of colleges (main channel distance 1100m to 2800m). Across Jesus Lock, the water level drops further by 1.29m from 5.18m to 3.89m (main channel distance 0m to 1100m).

Another simulation was done with all sluice gates fully opened. The result is shown in Fig. 3.13. This time, the upstream water level is computed to be 6.92m along Coe Fen region.

Water level drops substantially by 2.81m to 4.11m across Mill Pond sluice. Further down at Jesus Lock, the water level is dropped by another 0.22m to 3.89m across the lock. The actual setting of the openings should lie between these two scenarios.

Optimal setting of gates openings was applied based on the information provided by River Cam Flood Mapping Report[3]. The openings of the four sluice gates at Mill Pond should be set in a way such that upstream water level is retained at 6.90m above datum. In this model, the openings are set to 1.6m. For the two sluice gates at Jesus Lock, they should be set to retain upstream water level of 5.23m above datum. This corresponds to an opening height of 0.7m in my model. Based on this setting, the upstream end has an elevation of 6.90m. Water level drops to 5.23m across Mill Pond sluice. This level is maintained along the reach until Jesus Lock, then it drops to 3.89m till the downstream end. This is shown in Fig. 3.14.

This set of results suggests a few findings. The sluice gate at Mill Pond cannot be fully closed as it provides a major control of water level upstream. If the gate is closed, the upstream Coe Fen region is likely to be flooded even on a low flow day. The second finding is that, in order to maintain a fixed water level difference upstream and downstream of Jesus Lock, the sluice gate there should maintain at a limited opening height. The setting in the third simulation leads to an output that is closest to the actual water surface profile of the river on a normal day with flowrate= 1.6m^3/s, suggested by Environment Agency. The results will again be verified by actual surveyed data in section 4.

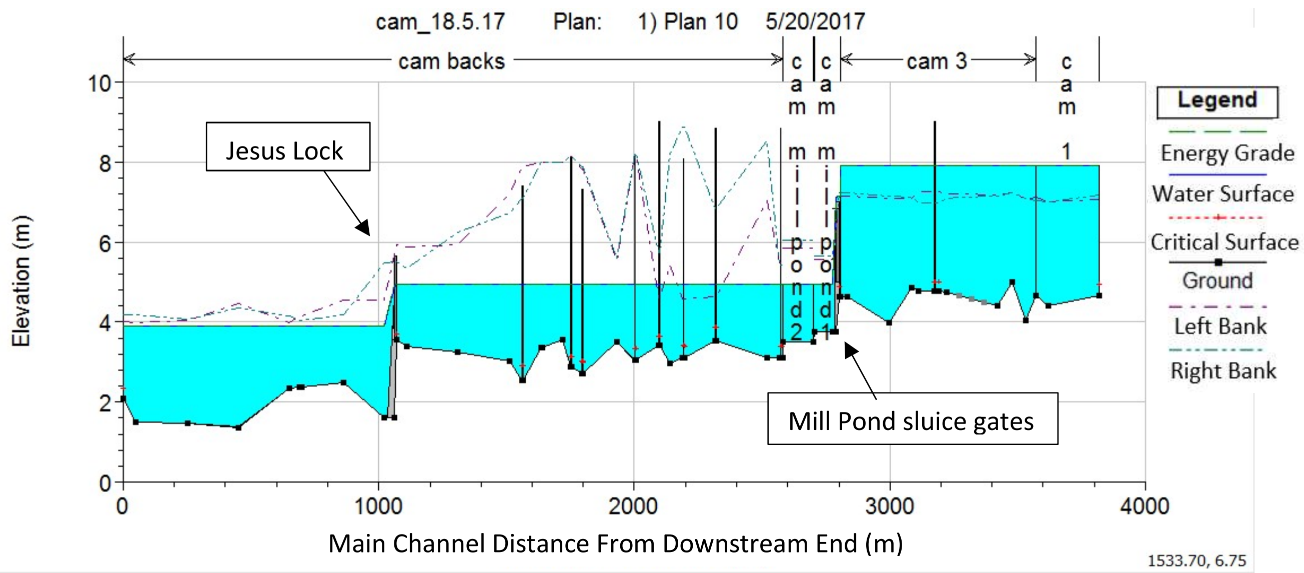

Fig. 3.12. Simulation Result of fully closed gates

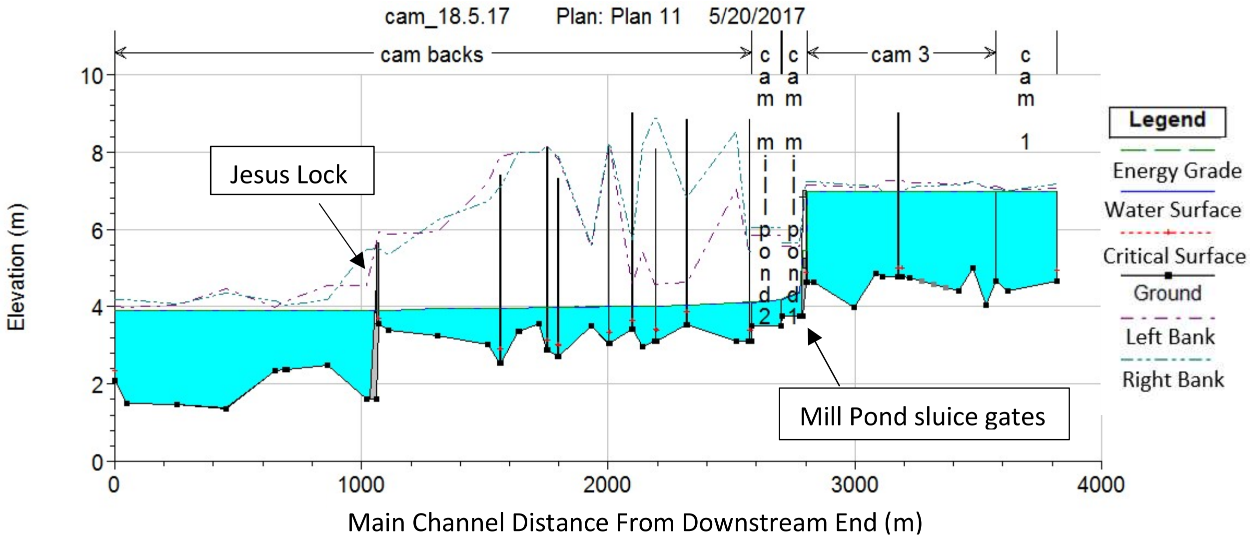

Fig. 3.13. Simulation Result of fully opened gates

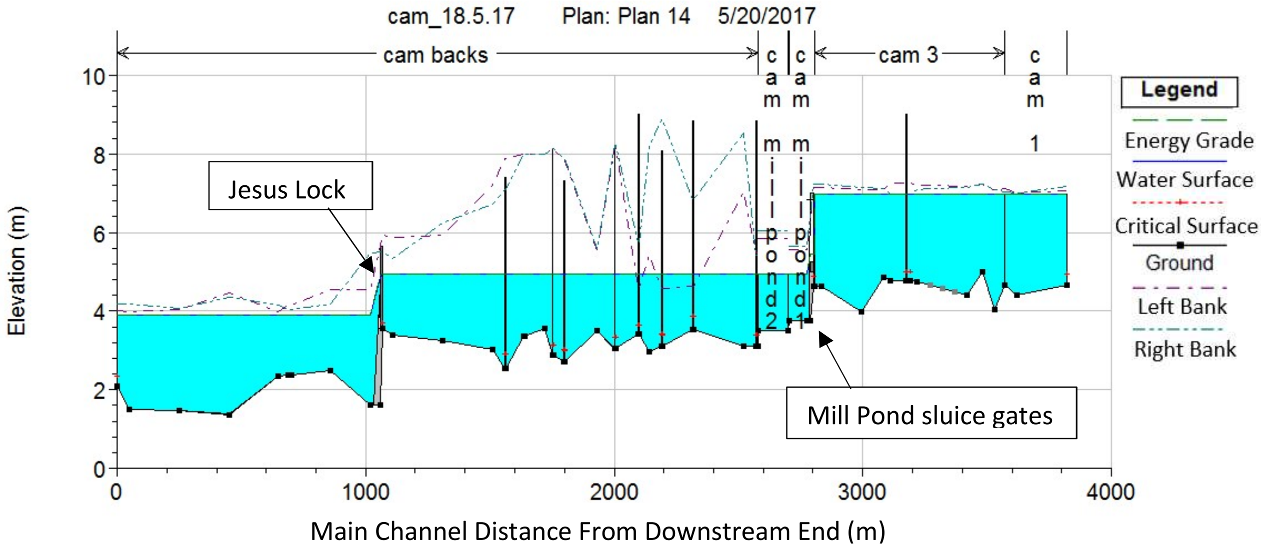

Fig. 3.14. Simulation Results of normal flowrate day

Comparison between Simplified and Detailed Model

The ideal settings of the gates that best represent a normal flowrate day were found from the computation above. The aim for the next simulation is to determine the difference between the simplified and detailed model. As mentioned, the simplified model has the same hydraulic structures as the detailed model but it assumes rectangular cross-sections along the river. This assumption affects the cross-sectional area of flow. It was proposed that the difference in flow area would affect the flow velocity according to continuity. Therefore, the velocity head is affected and from the energy equation (3-1), the water level should be affected. In order to quantify this difference and assess its significance on the accuracy of modelling, a simulation was performed on the simplified model. The openings of the four sluice gates at Mill Pond were set to 1.6m, and the two sluice gates at Jesus Lock are opened with an opening height of 0.7m. As before, an inflow of 1.6m^3/s was specified. The results of the computed water levels for simplified and detailed model are tabulated in Table. 3.1.

Location	Reach	Water Level (m)		
		Simplified Model	Detailed Model	Difference
Upstream end (Coe Fen)	Cam 1	6.91	6.90	0.01
Upstream of Mill Pond	Cam 3	6.91	6.90	0.01
Downstream of Mill Pond	Backs	5.21	5.23	0.02
Upstream of Jesus Lock	Backs	5.21	5.23	0.02
Downstream of Jesus Lock	Backs	3.89	3.89	0
Downstream End	Backs	3.89	3.89	0

Table 3.1. Results of water surface profile for simplified and detailed model

From Table 3.1. It is shown that, the difference in water level between the two models at the reach Cam 1 and Cam 3 was 1cm and difference at reach Backs was 2cm. The biggest discrepancy between the two models is 2cm which is relatively small. This suggests that the difference in water level computation due to the difference in cross-sectional area between the two models is insignificant. This result can be explained by the fact that the flowrate specified is too low. A flowrate of 1.6m^3/s leads to an average velocity of flow in main channel of 0.03 m^2/s (when it is far away enough from hydraulic structures). This is a small velocity head and therefore the energy balance is dominated by potential head effect. In order to see the effect of change in cross-sectional area, the kinetic energy should be prominent so a lager flow velocity is needed. Therefore, a larger inflow should be specified. In the next set of simulations, flowrate is increased gradually to differentiate the two models.

Four high inflow profiles were applied to the two models. An inflow of 5 m^3/s, 10 m^3/s, 20 m^3/s, and finally 44.6 m^3/s which is the peak flowrate measured in River Cam during the 2001 flood, were applied. The results were tabulated in Table 3.2 and Table 3.3 below.

Location	Reach	Flowrate = 5 m^3/s			Flowrate = 10 m^3/s		
		Water Level (m)			Water Level (m)		
		Simplified Model	Detailed Model	Difference	Simplified Model	Detailed Model	Difference
Upstream End (Coe Fen)	Cam 1	8.22	8.20	0.02	8.42	8.48	-0.06
Upstream of Mill Pond	Cam 3	7.22	7.12	0.1	7.38	7.30	0.08
Downstream of Mill Pond	Backs	5.27	5.16	0.11	5.52	5.36	0.16
Upstream of Jesus Lock	Backs	5.21	5.11	0.1	5.32	5.22	0.1
Downstream of Jesus Lock	Backs	3.98	3.91	0.07	4.16	3.97	0.19
Downstream End	Backs	3.93	3.90	0.03	4.04	3.93	0.11

Table 3.2. Results for inflow= 5 m^3/s and 10 m^3/s

Location	Reach	Flowrate = 20 m^3/s			Flowrate = 44.6 m^3/s		
		Water Level (m)			Water Level (m)		
		Simplified Model	Detailed Model	Difference	Simplified Model	Detailed Model	Difference
Upstream End (Coe Fen)	Cam 1	8.75	8.88	-0.13	9.47	9.69	-0.22
Upstream of Mill Pond	Cam 3	7.57	7.49	0.08	7.90	7.68	0.22
Downstream of Mill Pond	Backs	6.01	5.71	0.30	7.06	6.4	0.66

Upstream of Jesus Lock	Backs	5.51	5.38	0.13	5.84	5.66	0.18
Downstream of Jesus Lock	Backs	4.61	4.15	0.46	5.62	4.66	0.96
Downstream End	Backs	4.38	4.05	0.33	5.27	4.47	0.80

Table. 3.3. Results for inflow= 20 m^3/s and 44.6 m^3/s

From the above results, it is shown that the difference between the two models gets more obvious as flowrate increases, which agrees with the initial assumption. By gradually increasing the input inflow, it is found that for inflow lower than 5 m^3/s, the difference between simplified and detailed model is not significant. The river can be modelled by the simplified model assuming rectangular cross-sections. For inflow higher than 5 m^3/s, the simplified model result starts to deviate from the detailed model result and gives inaccurate simulation. Here comes another question, which is how rare a flowrate of 5 m^3/s exactly is. This can be found out by looking at return period of difference flowrates at River Cam. From the flood frequency analysis report by Prof. K. Richards in 2008, a power relationship between discharges and return period of River Cam is obtained[9]. This graph is shown in Fig. 3.15.

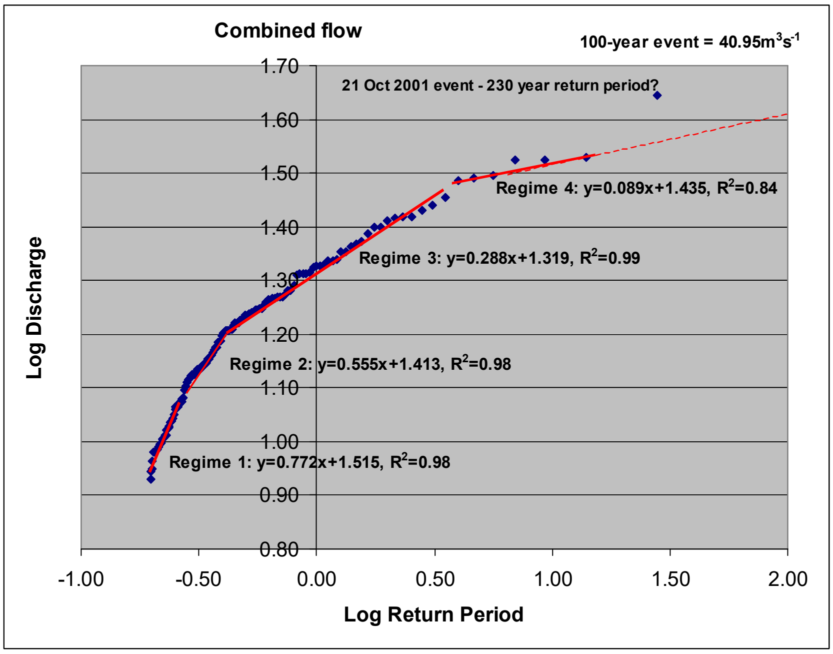

Fig. 3.15. River Cam- Discharge against Return Period in Log Scale

From extrapolation, applying a discharge of 5 m^3/s to the equation in Regime 1, the corresponding return period is 0.088 years, which is about one month. In other words, assuming this relationship can be extrapolated, in every month, there is one day that the flowrate of River Cam will reach 5 m^3/s which is considered to be relatively frequent.

However, it should be noted that in the flood frequency analysis report, the accuracy of this power relationship was not fully verified and it might not be appropriate to extrapolate the equation for Regime 1. The report also has pointed out that, the combined flow from the three tributaries might not be enough. The ungagged river Bourn Brook also joins River Cam at upstream so this fourth catchment should be considered (see Fig. 3.16). Due to limited information about Bourn Brook flowrate, its contribution to flowrate in River Cam would be omitted in this project.

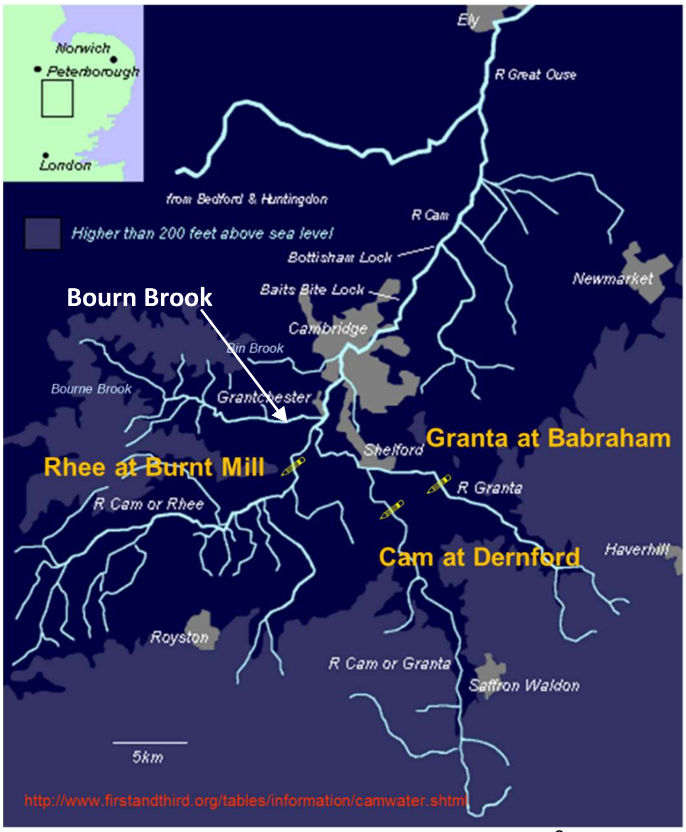

Fig. 3.16. Tributaries of River Cam[9]

It should also be noted that, the above simulation was done based on an unchanged gate openings setting. In practice, the opening of gates varies on different days. The two sluice gates at Jesus Lock are operated automatically to retain a fixed water level upstream.

Contraction and Expansion effect on energy loss due to hydraulic structures

Another investigation was about the effect of bridges on energy loss. Along the reach Backs, several bridges are presented and all of them are arch bridges. An initial assumption was made that, arch bridge with a single arch that does not have pillars submerged into rivers should have little effect on the water flow. On the other hand, bridges with three arches, having two pillars submerged in water, split the flow and induce contraction loss and expansion loss at inlet and outlet respectively. The contraction and expansion losses were computed for five inflow profiles. Profile 1 to 5 corresponds to inflow of 1.6m^3/s, 5 m^3/s, 10 m^3/s, 20 m^3/s, and 44.6 m^3/s, respectively. The losses are plotted along the main channel and is shown in Fig. 3.17. It can be seen that, along the Backs, the losses due to contraction and expansion are in fact insignificant no matter which type of bridge is encountered at all five flows. The only location that shows an expansion effect, is at channel distance 2800m to 3000m, which corresponds to the cross-sections at Mill Pond. The reason for this expansion loss is not due to bridges as proposed. At that location, there is a substantial increase of bank width when the flow enters from Cam 2 and Cam 3, into Mill Pond. This abrupt increase in cross-sectional area leads to a huge expansion. As the flowrate increase, the loss increases. This can be explained by the head loss equation. Referring back to equation (2-2). It is shown that the head loss due to the expansion effect is proportional to the square of velocity difference between two cross-sections. From Cam 2 to Mill Pond 2, and from Cam 3 to Mill Pond 1, the sudden change in cross-section causes a sudden change in velocity. This huge velocity difference is multiplied to the expansion coefficient to give the head loss in metre. Therefore, the losses increase as flowrate.

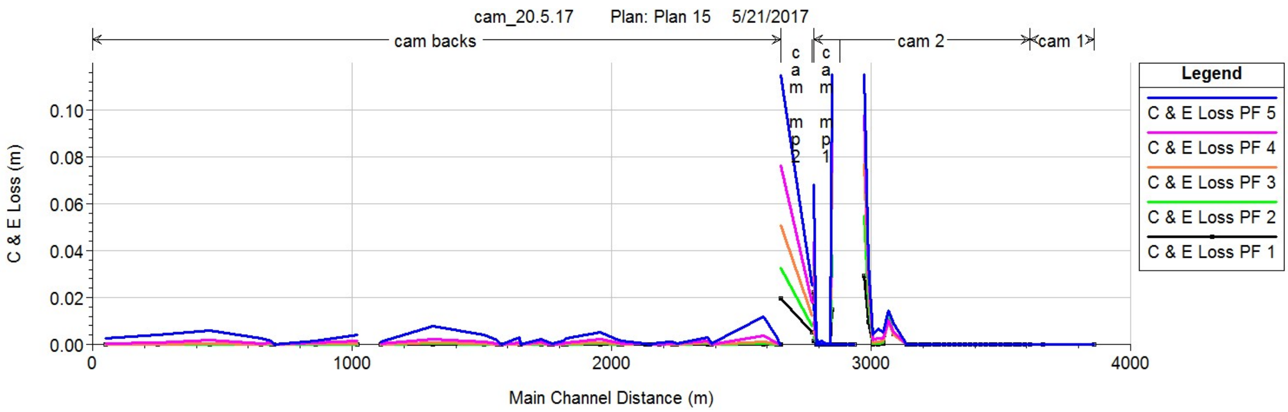

Fig. 3.17. Contraction/ Expansion Loss plotted along river network

4. Surveying

4.1 Levelling

Another major part of this project is the surveying of River Cam. In order to verify the river model in HEC-RAS, the results obtained from computer simulation were compared with the surveyed water surface profile. Levelling technique was used to capture the water surface profile at upstream and downstream of the river channel. Levelling determines the height of point of interest by comparing the respective heights of two points above datum. In the British Isles, the datum used is the mean sea level at Newlyn in Cornwall, which is called the Newlyn Datum. Benchmarks are points with known heights above the Newlyn Datum and they are provided by the Ordnance Survey. Adequate number of benchmarks is necessary to establish a complete survey network. A full set of benchmark data could be found on the Ordnance Survey database[10]. However, a major issue was discovered. According to the Ordnance Survey data, the benchmarks available in Cambridge were last verified in 1956 so the heights could have changed due to subsidence over such a long period of time. The accuracy of these benchmarks were doubtful. Besides, the benchmarks were mostly found on buildings located far away from River Cam, so the direct use of Ordnance Survey benchmarks was not adopted. Alternatively, useful benchmark data were provided by the Cambridge University Engineering Department (CUED). CUED has established further benchmarks which are located in Coe Fen region based on Ordnance Survey data, these benchmarks are verified every year by CUED students for their surveying project so the data are relatively accurate. The benchmarks are brass studs which have a larger brass disc around their base (Fig.4.1). The benchmark locations and altitudes are given in Extension Activity- Map of Coe Fen (Appendix B). The benchmarks A1 and B3 were used. The information of benchmarks are listed in Table 4.1.

Fig. 4.1. Benchmark B3 established by CUED

Benchmark	Elevation (AoD)	Descriptions
A1	6.458m	By end post of fence North of punt rollers, in North corner of area A
B3	7.961m	By first fencepost on North side of raised approach to bridge, on North East corner of area B

Table 4.1. Benchmark Data

Levelling was performed at three locations, Coe Fen, Mill Pond and downstream region along Midsummer Common and Jesus Green. At the first two locations, an electronic Leica Sprinter 150M Level (Fig. 4.2) and levelling staff (Fig. 4.3) were used together with benchmarks A1 and B3 to obtain precise levelling results. Since surveying equipment could not be transported to downstream of the river for logistical reason, an estimation of water level using tape measurement and known bank level was done instead, at downstream end region.

Fig. 4.2. Leica Sprinter 150M Level

Fig. 4.3. Levelling Staff for Leica Sprinter

4.2 Procedures

Coe Fen

At Coe Fen, the upstream and downstream of Cam 3 were measured. The first measurement was taken on 10th March, 2017 from 11am to 12pm. The flowrate of the river was about $1.6m^3/s$. The aim of this measurement was to assess the drop of water surface elevation along the reach Cam 3. There was a difficulty in placing and levelling the staff on top of water surface as it was not a hard surface. It would be more sensible to place the staff on a stable flat surface for taking observations. The approach used here was to first measure the elevation of the concrete river bank, after that, the distance between the river bank and water surface was read-off from a measuring tape. Taking into account of this height difference, the actual water level above datum could be determined. First of all, the level was set-up at a location that was equidistant to B3 and the concrete river bank. The staff was placed on top of the B3 brass stud and a back-sight was measured. The staff was then moved to the concrete river bank, with the level directed onto it, foresight reading was obtained. Five points were measured at 5m intervals at upstream. Similarly, moving the equipment to the downstream and making use of benchmark A1, five points were measured at 5m intervals at downstream. Results are presented in Table 4.2. Another measurement using purely measuring tape was done on 19th May, 2017 from 6pm. The flowrate at that period was about 1.8 m^3/s. The results are shown in Table 4.3.

Mill Pond

At Mill Pond, measurements were taken both upstream and downstream of the sluice gate. Levelling was carried out on 14th March, 2017 from 11:40am to 12:40pm. First, the level was set-up at a point on the green field opposite to the Mill, from where the benchmark A1, upstream of sluice and downstream of sluice were all visible. Same as above, the staff was first placed on top of A1 to obtain the reference back-sight. It was then placed at five different points along the bank downstream and five points upstream respectively. The corresponding bank to water surface distances were also measured with measuring tape. The results are shown in Table 4.4.

Apart from the water surface level. It was attempted to obtain the flowrate of the river by measuring water velocity and multiplying it by the cross-sectional area.

At the upstream of sluice gate, flow velocity is estimated by dropping a leaf on river and measuring its travel time over a certain distance. This gave a rough estimation of water velocity. Results are shown Table 4.5.

Midsummer Common and Jesus Green

At Midsummer Common and Jesus Green, a rather different method was used for measurement. It was carried out on 27th February, 2017 from 5:00pm to 6:30pm. A measuring tape was used to measure the distance between river bank and water surface level. Measurement was taken at each river station and the corresponding bank elevation data was obtained from the Environment Agency river model[3]. Deducting the measured distance from estimated bank elevation, the water level was then obtained. The corresponding river stations measured were from No.19800 to No.20800 spanning a channel length of about 1.2 km. The water level difference upstream and downstream of Jesus Lock was also captured. The results are tabulated in Table 4.6.

4.3 Results Interpretation

Coe Fen

Location	Point	Bank Elevation (m)	Distance from water surface to bank (cm)	Water Level (m)	Average Water Level (m)
Upstream of Coe Fen	1	7.173	39.6	6.777	
	2	7.192	42.2	6.770	
	3	7.177	40.5	6.772	
	4	7.155	38.6	6.769	
	5	7.143	37.3	6.770	6.77
Downstream of Coe Fen	6	7.107	33.7	6.770	
	7	7.165	39.5	6.770	
	8	7.156	38.7	6.769	
	9	7.154	38.2	6.772	
	10	7.153	39.5	6.758	6.77

Table. 4.2. Levelling Results at Coe Fen along River Cam when Q= $1.6m^3/s$

Location	Point	Bank Elevation (m)	Distance from water surface to bank (cm)	Water Level (m)	Average Water Level (m)
Upstream of Coe Fen	1	7.173	35.2	6.82	
	2	7.192	34.5	6.85	
	3	7.177	33.6	6.84	
	4	7.155	32.5	6.83	
	5	7.143	30.5	6.84	6.84
Downstream of Coe Fen	6	7.107	34.8	6.76	
	7	7.165	34.8	6.82	
	8	7.156	35.0	6.81	
	9	7.154	35.1	6.80	
	10	7.153	34.9	6.80	6.80

Table. 4.3. Levelling Results at Coe Fen along River Cam when Q= $1.8m^3/s$

Points 1-5 are at upstream. Points 6-10 are at downstream. The water flow at the period of measurement is steady and relatively low by observation and the combined flowrate obtained from the Shoothill GaugeMap was around $1.6m^3/s$. From Table 4.2, it is shown that, throughout the 750m span along Cam 3, the water level has not dropped, and was maintained at the level 6.77m above datum, to the nearest 0.01m. On the other measurement day, the flowrate obtained was around $1.8m^3/s$. This time, although the bank to water surface distance was still measured by tape measure, the exact elevation of bank was not levelled. It should be noted that, the elevation of bank at points of measurement was assumed to stay the same. This assumption would lead to a less accurate result. The water levels measured at the upstream end was 6.84m and 6.80m at downstream. The difference is 0.04m which is believed to fall within the limit of error. Therefore, it is concluded that, the water level drop of Cam 3 is insignificant on a low-flowrate day and is treated as a flat water surface profile.

Mill Pond

Location	Point	Bank Elevation (m)	Distance from water surface to bank (cm)	Water Level (m)	Average Water Level (m)
Downstream of Mill Pond	1	6.086	88.0	5.206	
	2	6.046	83.9	5.207	
	3	5.926	71.5	5.211	
	4	5.860	64.9	5.211	
	5	6.110	89.7	5.213	5.210
Upstream of Mill Pond	6	7.233	35.4	6.879	
	7	7.215	33.5	6.880	
	8	7.208	32.3	6.885	
	9	7.374	49.3	6.881	
	10	7.357	47.5	6.882	6.881

Table. 4.4. Levelling Results at Mill Pond along River Cam when Q= $1.6m^3/s$

Points 1-5 are at downstream of sluice gate. Points 6-10 are at upstream of sluice gate. During the period of measurement, the opening of sluice gate was at its standard setting. The levelling results gave an average upstream water level of 5.21m above datum, and a downstream water level of 6.881m. Water level drop over the sluice gate at Mill Pond was 1.671m.

For velocity measurement, it was taken upstream of the sluice gate at Mill Pond. The distance travelled by the leaf was 2m. Five trials were done and the time taken to travel was recorded in Table 4.5.

Measurement	1	2	3	4	5	Average
Time(s)	14.00	13.95	12.85	14.00	15.30	14.02

Table 4.5. Time taken for a leaf to travel through 2m along the river

The average time for travel was 14.02s. The velocity of flow is deduced to be 2/14.02= 0.14m/s. From this, we can work out the flowrate in cubic metre per second by multiplying velocity with flow area. The flow area can be found in our river model in HEC-RAS. From HEC-RAS, the corresponding river station is No.22400 and its flow area is 25.03m^2, the flowrate calculated is 3.5m^3/s which was more than double of our input value 1.6 m^3/s. The flowrate of 3.5m^3/s should be an unusual high flow which was unrealistic. Therefore, this result is believed to be incorrect both by inspection and comparison to computation result.

The reason for this is, again, due to the low flowrate of river. The Froude number is not big enough to let the internal velocity of river flow outweigh the external velocity. It was relatively windy on the day of measurement, so the surface flow was highly affected by strong wind. A large inconsistency of time recorded was also shown in Table 4.6. It was concluded that the velocity estimation here was not reliable and the use of this set of data was not adopted.

Midsummer Common and Jesus Green

Location	River Station	Measurement Time	Bank Elevation (m) (GIVEN)	Distance from water surface to bank (cm)	Water Level (m)	Average Water Level (m)
Downstream of Jesus Lock	19800	17:26	4.18	34.5	3.81	
	20000	17:34	4.08	17.0	3.91	
	20175	17:42	4.34	36.0	3.98	
	20440	17:49	4.03	8.0	3.95	
	20440.2	17:51	4.03	15.0	3.88	
	20600	17:58	4.19	33.0	3.86	
	20760	18:08	5.48	163.0	3.85	3.89
Upstream of Jesus Lock	20760.2	18:15	5.52	34.5	5.18	
	20800	18:17	5.33	30.0	5.03	5.11

Table 4.5. Levelling Results at Midsummer Common and Jesus Green along River Cam

The water level downstream of Jesus Lock had an average elevation of 3.89m while the upstream side of the lock was measured to be 5.11m. The water level drop through the Jesus Lock was 1.22m. It should be noted that, since the measurement was not done by precise level, the margin of error of these results is big. The actual value of water level is believed to be plus or minus a centimetre to the measured value.

Surveyed and Modelled Result

These surveyed data are compared to the computed water level to verify the model in HEC-RAS. The water surface profiles presented in Table 4.2, 4.4 and 4.5 are plotted onto a graph together with the computation result by HEC-RAS with the detailed model (See Fig. 4.4)

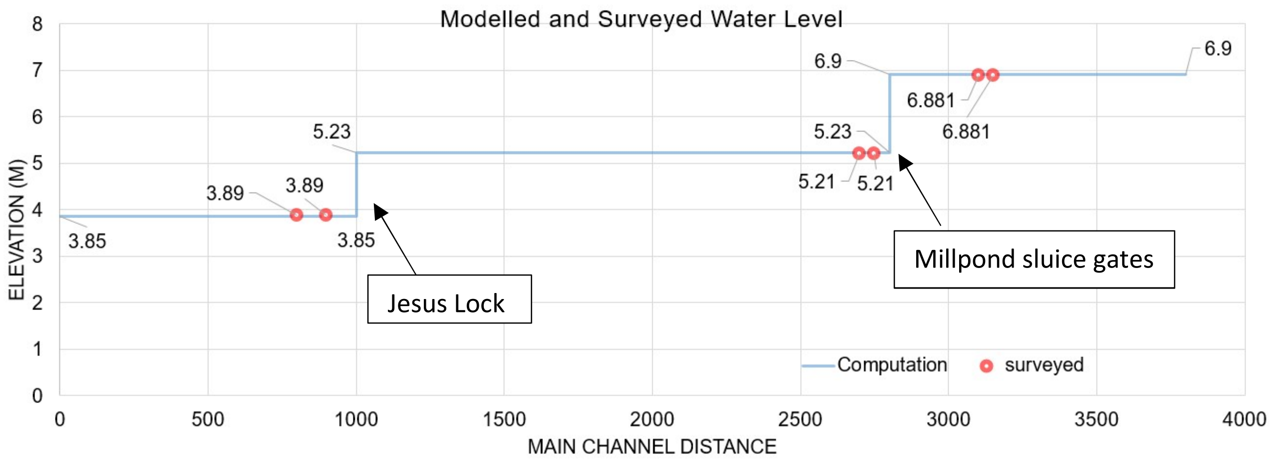

Fig. 4.4. The modelled and surveyed water surface profile when Q=1.6m^3/s

In upstream region, the measured value is 6.88m while the computed value is 6.9m which is 0.02m higher. Passing the sluice gate at Mill Pond, the measured water level is 5.21m while the computed value is 5.23m. Again, the computed value is 0.02m higher. The measured water level drop across Mill Pond sluice is 6.881-5.210 = 1.67m and the modelled water drop is 6.9-5.23 = 1.67m. This suggests that the water level difference is maintained at 1.67m and the model has correctly simulated it. Although there is a 0.02m upward shift in absolute value, this discrepancy is relatively low and accepted as normal fluctuation in water level.

At downstream, across the sluice gate at Jesus Lock, modelled water level drops by 1.38m from 5.23m to 3.85m, while measured water level dropped by 1.32m from 5.21m to 3.89m.

The discrepancy obtained is 0.06m. This is greater than the discrepancy obtained at upstream, but is still within an acceptable range. It can be concluded that the river model can accurately represent the actual river network to a certain degree. Further work in unsteady flows can then be done using this river model built in HEC-RAS.

5. Limitations and Suggestions

In the survey part of this project. Several limitations were encountered. First of all, due to the limit of resources, it was impossible to transport electronic level and staff to the downstream end of the river model to measure its water level. Therefore, a tape measure was used to measure only the difference between bank and water level, the exact bank elevation was an unknown. The coordinates of the location of measurement were determined by a GPS software on smartphone, this could induce a few metre error between exact and detected location. Based on this location recorded by GPS, the corresponding cross-section was searched from the Environment Agency river model. This cross-section could be an interpolated cross-section which was also an estimation. The river bank level was then read-off from their river model and by deducting the measured distance, the water level was computed. This method accumulated errors and the accuracy of water level computed was doubtful. Especially for low inflow rate (Q= 1.6m^3/s and 1.8 m^3/s), the water level variation was subtle, the observed as well as simulated results had small variations and an error of a few centimetres could be very significant.

For the surveying activities, two suggestions are made. Level and staff should be used and a detailed survey should be done in order to obtain the bank elevation and thus the precise water surface elevation. Another suggestion is that, the same surveying measurement should be done in days with bigger flowrate, for instance a day when rain is heavy and flowrate of the river is significantly higher than usual. In that case, the water level drop along the 4km reach would become obvious and therefore the error mentioned above will become relatively insignificant.

6. Conclusions and Future Work

Two models were built in HEC-RAS, a simplified version and a detailed version. The simplified model assumed a rectangular cross-section while the detailed model consisted of precise shape of riverbed. A steady flow analysis was carried out on both models and the water surface profiles were computed. It was found that, under low flowrate (below $5m^3/s$), the deviation between the two models was insignificant. The effect of cross-sectional difference became noticeable under high flowrate. Therefore, for simple modelling involving low flowrate, the simplified model could be used.

Besides, the effect of hydraulic structures on energy loss was also investigated. It was found that, the head losses caused by bridges and sluice gates were insignificant under low flowrate. Their energy loss increases with flowrate as head loss is proportional to the velocity between two cross-sections. However, the most significant energy loss found was not in any hydraulic structures but at the transition from Cam 2 to Mill Pond. In fact, the major frictional and expansion loss was discovered at Mill Pond, where the two reaches Cam 2 and Cam 3 flow into Mill Pond, experiencing a substantial and abrupt increase in cross-sectional area.

Surveying activities were carried out to measure the water surface elevation. The measured data was used to verify the hydraulic model. The two profiles were plotted on Fig. 4.4 and it was found that, the model obtained agrees well with the actual measured water level with a small degree of discrepancies. Therefore, it was concluded that a representative model was constructed in HEC-RAS for further analysis in future.

An accurate model of River Cam is obtained based on 1D steady flow analysis. The next stage is to perform unsteady flow analysis with this model to further verify it. Future work is to make use of other functions of HEC-RAS, and to carry out other scenario studies on River Cam. One of the possible studies will be the investigation of dredging effect. Due to the popularity of rowing among students in Cambridge, congestions happened often near boathouses region (downstream of Jesus Lock). An interesting proposal was made to widen the river or excavate another river reach to mitigate the heavy traffic for rowers. This will involve dredging of sediment deposits. Bed sediment properties will have to be determined and specified in the model. HEC-RAS then can compute the sediment transport potential by modifying channel geometry and water discharge.

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Appendix A – Extract from HEC-RAS 5.0 Reference Manual

Type of Channel and Description	Minimum	Normal	Maximum
A. *Natural Streams*			
1. Main Channels			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy. deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.070	0.100	0.150
2. Flood Plains			
a. Pasture no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2. Same as above, but heavy sprouts	0.050	0.060	0.080
3. Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4. Same as above, but with flow into branches	0.100	0.120	0.160
5. Dense willows, summer, straight	0.110	0.150	0.200
3. Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

Manning's n-values suggested by HEC-RAS

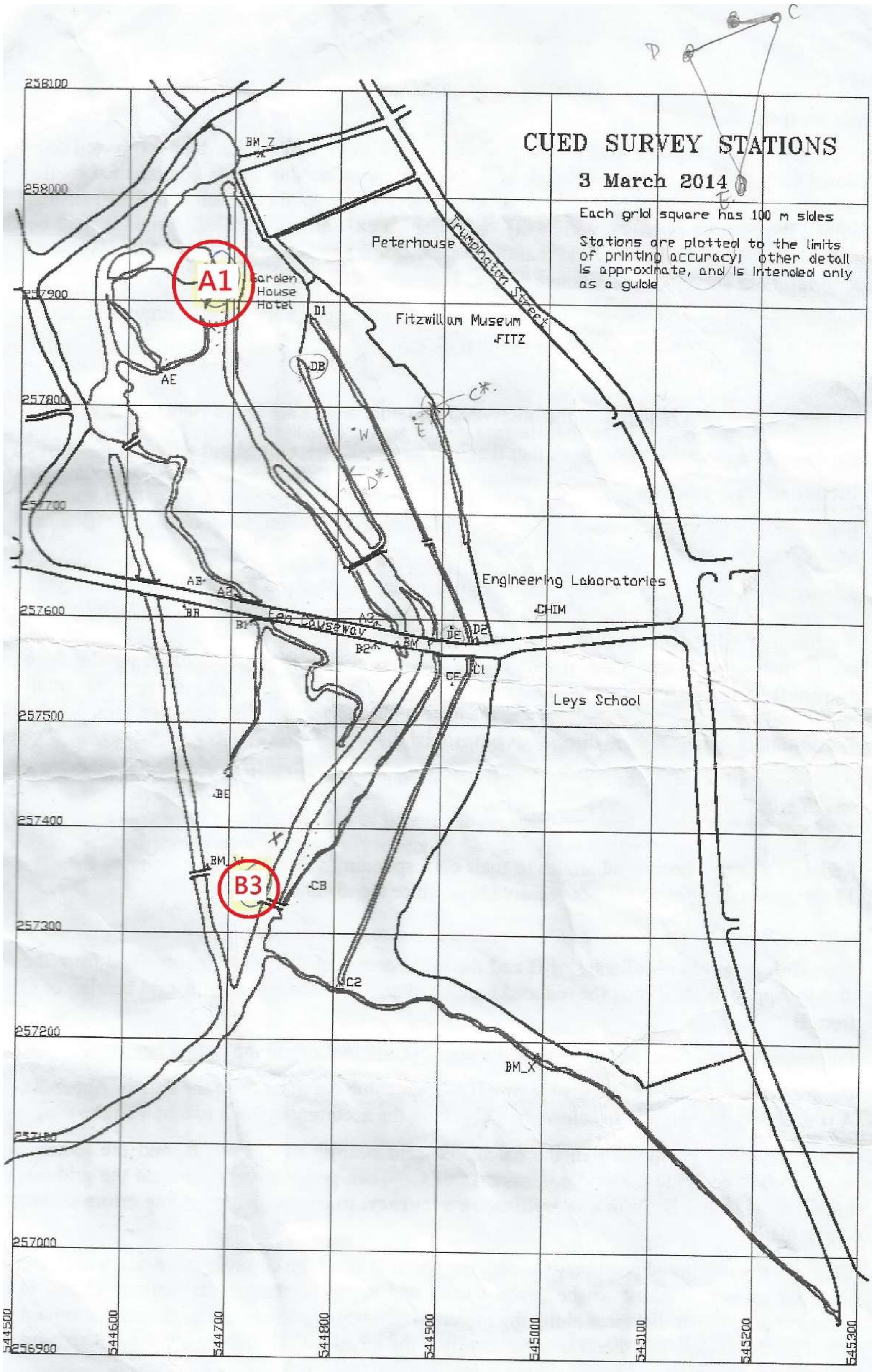
Subcritical Flow Contraction and Expansion Coefficients

	Contraction	Expansion
No transition loss computed	0.0	0.0
Gradual transitions	0.1	0.3
Typical Bridge sections	0.3	0.5
Abrupt transitions	0.6	0.8

Contraction/ expansion coefficients suggested by HEC-RAS

Appendix B

CUED Extension Activity- Map of Coe Fen



Appendix C

Risk Assessment Retrospective

Before the start of project, a risk assessment was submitted taking into account of all the possible hazards that may occur during the surveying of the project. At the early stage, a more complicated surveying technique was proposed. Surveying was planned to be performed by mounting a sonar at the bottom of punt then punt along the river to capture the cross-sections of riverbed. This involves placing electronic devices close to water which may lead to electrical hazard. Related safety measures were provided in the risk assessment. As the project progressed, this methodology was abandoned and was replaced by placing level and staff on the river bank to measure water level. The risk of getting electrified has largely reduced. Besides, the level was placed far away from the river during the measurement so the possible electrical hazard was very low.

The other risk concerned was slips or falls into river while doing measurements. This risk was well-prevented as non-slip shoes were worn every time when measurements were carried out. Slips or falls were not encountered during the course of project. Besides, detailed survey involving more equipment such as staff and level was not done individually. An exchange student from Dr. Liang's laboratory kindly offered help with the levelling activities. Therefore the equipment could be handled with greater care and students could keep an eye on each other during the fieldwork to ensure safety.

There were some stinging grasses in Coe Fen region. Although long sleeves and trousers were worn during measurements, ankle, wrist and fingers were still stung by these plants and this has caused rashes. This suggested that more protective clothing or footwear should be considered for risk assessment in future.