Extension of Rating Curves at Gauging Stations
Best Practice Guidance Manual

R&D Manual W6-061/M

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This manual should be used whenever it is necessary to extend a rating curve at a gauging station.

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LIST OF SYMBOLS

A	Intercept in rating equation
A	Area (m^2)
B	Weir crest width (across channel) (m)
B_τ	Top width of river channel (m)
β	Power in rating equation
C	Weir coefficient; constant in rating curve equation; Chezy Roughness coefficient
C_d	Coefficient of discharge
C_v	Coefficient of velocity
C_w	Weir coefficient in broad crested weir formula
C_r	Constant in rating equation
D	Bankfull depth of river (m)
Δ	A function of the boundary layer thickness (0.01) in the formula for C_d for broad crested weirs
F	Drowned flow reduction factor.
G	Acceleration due to gravity
H	Stage (m)
h_1	Upstream stage (m)
h_2	Downstream stage (m)
h_p	Pressure head on a weir crest as measured by a crest tapping
H	Total head on weir (m)
H_1	Total upstream head (m)
H_2	Total downstream head (m)
k_s	Length parameter characteristic of the surface roughness
Ks	Surface roughness parameter used in 3-D models
K	Channel conveyance
k-ε	"K-epsilon" turbulence model: a type of model for simulating 3-D flow in rivers
L	Backwater length (m)
N	Manning's resistance coefficient
P	Wetted perimeter (m)
Q	Flow (m^3/s)
R	Hydraulic radius (m)
S_f	Friction slope
S	Water surface slope
V	Velocity (m/s)

LIST OF ABBREVIATIONS

1-D	One dimensional (in space, along the watercourse)
2-D	Two dimensional (in space, either in plan or in elevation)
3-D	Three dimensional (in space)
ADCP	Acoustic Doppler Current Profiler
AOD	Above Ordnance Datum
ASCII	'American Standard Code for Information Interchange', a standard used to define the coding and format of Digital data files
BSI	British Standards Institution
CFD	Computational Fluid Dynamics

DCM	Divided Channel Method
DTM	Digital Terrain Model
FEH	Flood Estimation Handbook
Gaugeman	Environment Agency standard rating curve development software from Hydrologic, shortly to be replaced by SKED
HARP	The Agency's 'Hydrometric Archive Replacement Project'
HEC-RAS	1-D computational hydraulic model produced by the US Army Corps of Engineers (Hydrologic Engineering Center - River Analysis System)
HYDATA	Hydrological database software produced by CEH Wallingford
ISIS	1-D computational hydraulic model produced by HR Wallingford and Halcrow
ISO	International Standards Organisation
MIKE 11	1-D computational hydraulic model produced by DHI
SCS Method	Method for estimating Manning's *n* roughness coefficient developed by the US Soil Conservation Service
SKED	The rating development module associated with WISKI
WISKI	The new hydrometric archive system currently being implemented across the Environment Agency as part of the HARP project

1. INTRODUCTION

1.1 Background to the Manual

The manual is the final output produced in the Environment Agency R&D project W6-061 'Extension of Rating Curves at Gauging Stations'. The purpose of the manual is to provide best practice guidelines for the extension of rating curves at gauging stations.

Other reports produced under the project cover:

- Project Requirements and Literature Review (Report W6-061/TR1);
- Best practice for using simple hydraulic techniques for extending rating curves (Report W6-061/TR2);
- Best practice for using 1-D computational hydraulic models for extending rating curves (Report W6-061/TR3);
- Best practice for using 2-D computational hydraulic models for extending rating curves (Report W6-061/TR4); and
- Best practice for using 3-D computational hydraulic models for extending rating curves (Report W6-061/TR5).

These reports are superseded by the manual. The manual does not cover the use of physical models for the extension of rating curves.

The audience for the manual is twofold:

- Hydrometrists and project managers who will be planning, commissioning, supervising and accepting rating curve extension work; and
- Hydrometrists, hydraulic engineers and modellers, who will actually extend rating curves using the techniques.

The hydrometrist/project manager and engineer/modeller may, of course, be the same person.

1.2 Purpose of the Manual

Rating curves provide relationships between water level and flow in a river. Rating curves are used at gauging stations to produce flow estimates from recorded water levels. However for a variety of reasons rating curves do not cover the full range of levels and flows at gauging stations.

Rating curves often do not cover high flood flows because of the difficulties of measuring flows in flood conditions and the fact that gauging structures are often drowned. However the accurate estimation of flood flows is of vital importance in planning and designing flood defence measures. Rating curves often do not cover very low flows because of the difficulties of measuring low velocity flows and the poor sensitivity of some structures at low flows. However the accurate estimation of low flows is important in water resources and drought management, and for consent conditions for licensed abstractions and discharges.

The purpose of this manual is to provide guidance on how to extend rating curves to high and low flows. The manual provides:

- An introduction to rating curve extension including advice on which methods to use under different circumstances;
- Guidance on the use of simple hydraulic techniques for rating curve extension; and
- Guidance on the use of computational hydraulic modelling for rating curve extension.

1.3 Requirements of the Environment Agency

The current methods and requirements of the Environment Agency in terms of rating curve calculation and extension are detailed in the first report produced in this project, Report W6-061/TR1 of July 2001. This report indicated that the methods currently used by the Agency vary from region to region and, to some extent, by function within each region. The improvements required by the Agency, as indicated by questionnaire and discussions at a workshop in May 2001, are:

- Consistency in approach throughout the Agency;
- A robust and non-complex method of extending ratings;
- Guidance on all available methods for extending ratings; and
- Guidance on likely uncertainty values achieved by the various methods.

1.4 How to use the Manual

1.4.1 Those who are planning/supervising rating curve extension work

The manual provides the following information for those who are planning/supervising rating curve extension work:

- Advice on which method to use in different circumstances (Section 2.4);
- Overview of simple hydraulic techniques for rating curve extension (Section 4.1); and
- Overview of hydraulic modelling techniques for rating curve extension (Section 5.1).

1.4.2 Those who will extend rating curves

The manual provides the following advice and procedures for extending rating curves:

- Advice on how to undertake a rating and gauging review to decide whether the rating is suitable for extension (Section 3);
- Advice on selection of the rating curve extension method (Section 2.4);
- Step by step procedure for extension of rating curves using simple hydraulic techniques (Section 4.2);
- Step by step procedure for extension of rating curves using computational hydraulic models together with detailed guidance (Sections 5.2 and 5.3); and

- Case studies for rating curve extension using simple hydraulic techniques (Section 4.3) and computational hydraulic models (Section 5.4).

1.4.3 How not to use the manual

The manual does not:

- Replace the need to collect and utilise data for rating curve extension;
- Replace the need to understand the hydraulics of gauging stations;
- Preclude appropriate new techniques from being used to extend rating curves when they become available; and
- Include advice on the use of physical models for the extension of rating curves. This is a valid method but requires specialist laboratory facilities.

2. SELECTION OF RATING CURVE EXTENSION METHOD

2.1 Introduction

This section:

- Describes rating curves at gauging stations (in Section 2.2);
- Provides an introduction to extending rating curves (in Section 2.3); and
- Provides guidance on which methods to use to extend rating curves (in Sections 2.4 and 2.5).

Before extending a rating curve, the following work should be carried out:

- A review to see whether the rating curve is suitable for extension. A possible method for undertaking this review is outlined in Section 3.1; and
- An assessment of the hydraulics at the gauging site, in order to determine the type of extension required. A method for assessing the hydraulics is given in Section 3.2.

2.2 Rating Curves at Gauging Stations

Gauging stations consist of the two main types:

- Sites where the stage discharge relationship is controlled by hydraulic structures. Weirs are by far the most common type of structure but flumes and gated structures are also used; and
- Channel sites where there are no structures and the stage discharge relationship is controlled by the channel.

Figures 2.1 to 2.4 show examples of rating curves where extension may be required, as follows:

- Figure 2.1 shows an in-bank channel rating where the full flow range is contained in the channel, and high and low flow extensions are required;
- Figure 2.2 shows a channel rating which requires extension to overbank flow. There is normally a distinct change in the slope of the rating curve between in-bank and overbank flow;
- Figure 2.3 shows a structure rating curve which requires extension to structure-full, and drowning does not occur; and
- Figure 2.4 shows a rating curve which already extends above bankfull which requires extension to higher flows.

When drowning of structures occurs, the shape of the rating curve becomes more complex. Figures 2.5, 2.6 and 2.7 show examples of rating curves at weir sites where drowning (or non-modular) flow occurs, as follows:

- Figure 2.5 shows the effect of drowning on an in-bank structure rating curve where the downstream rating curve is constant;
- Figure 2.6 shows the effect of variations in downstream roughness or backwater effects resulting in a family of rating curves; and

- Figure 2.7 shows the effect of drowning where overbank flow occurs downstream of the structure.

These figures illustrate the importance of knowing the rating curve downstream of the structure when extending rating curves at structures in the non-modular flow range.

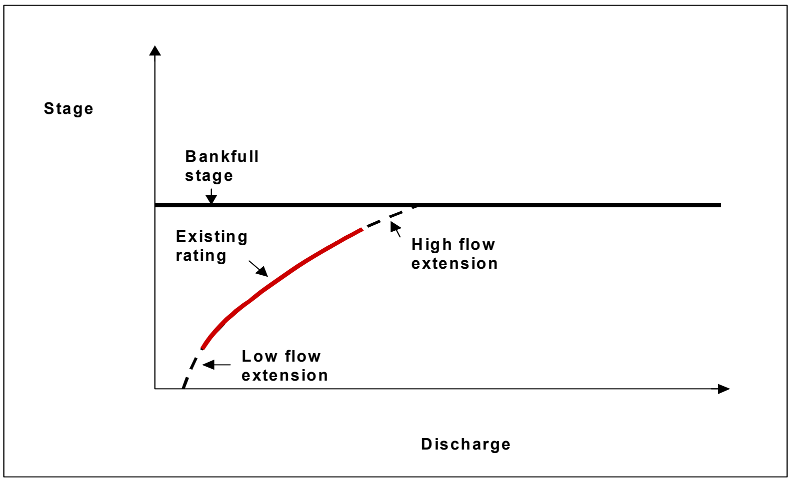

Figure 2.1: In-bank channel rating where the entire flow is contained in the channel

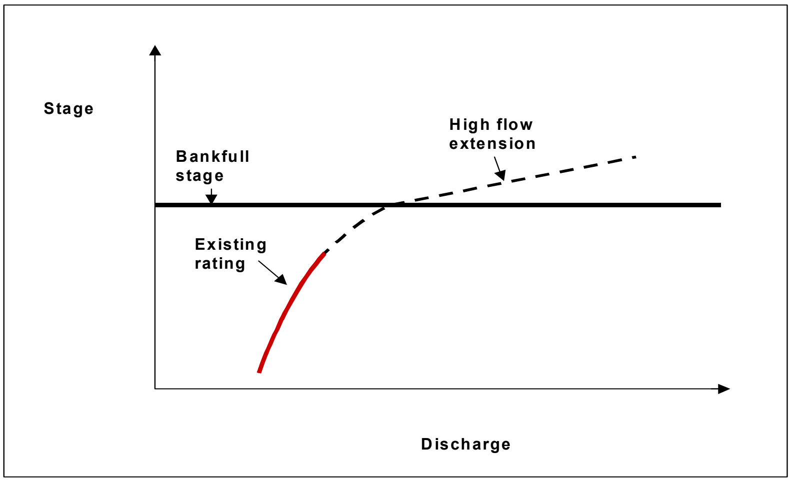

Figure 2.2: Channel rating with extension to overbank flow

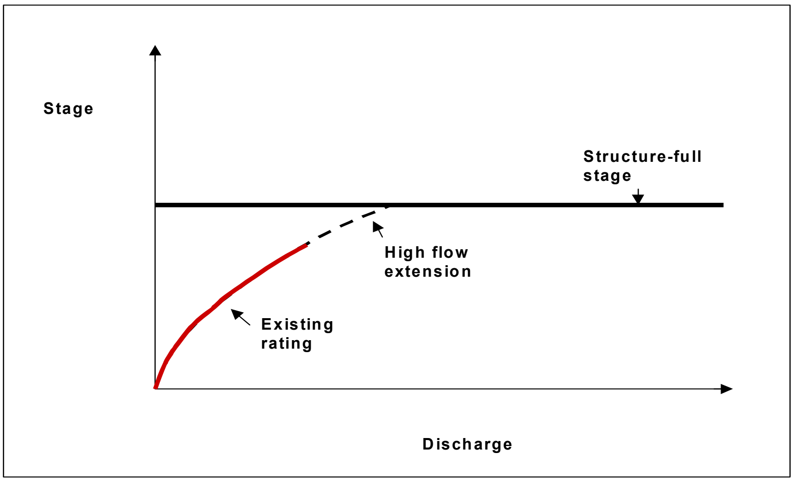

Figure 2.3: Structure rating with extension to structure-full, no drowning

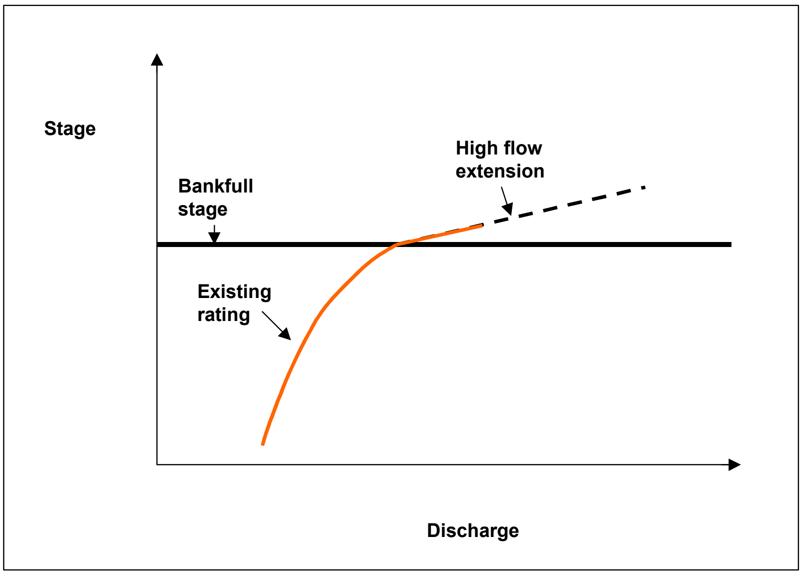

Figure 2.4: Extension of an overbank rating curve to higher flows

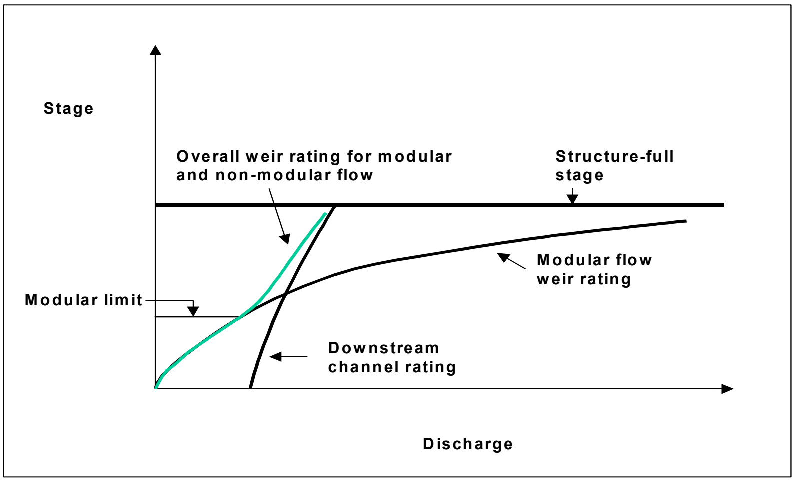

Figure 2.5: In-bank structure rating curve with drowning - constant downstream rating curve

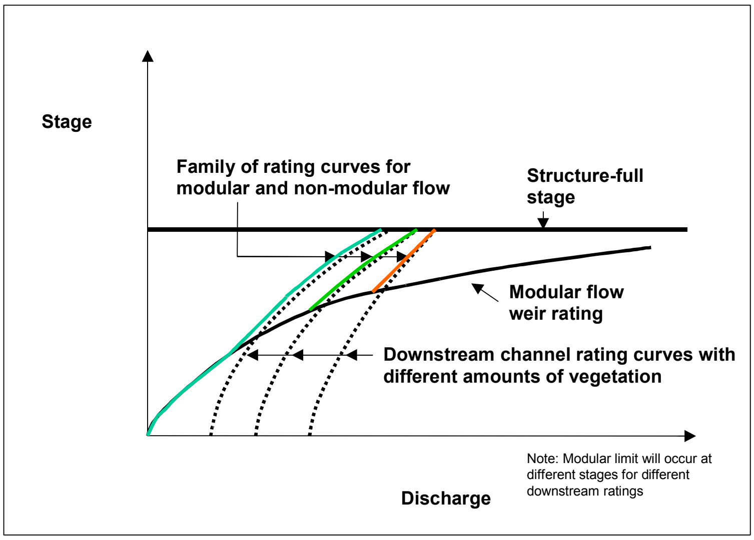

Figure 2.6: In-bank structure rating curves with variable downstream roughness

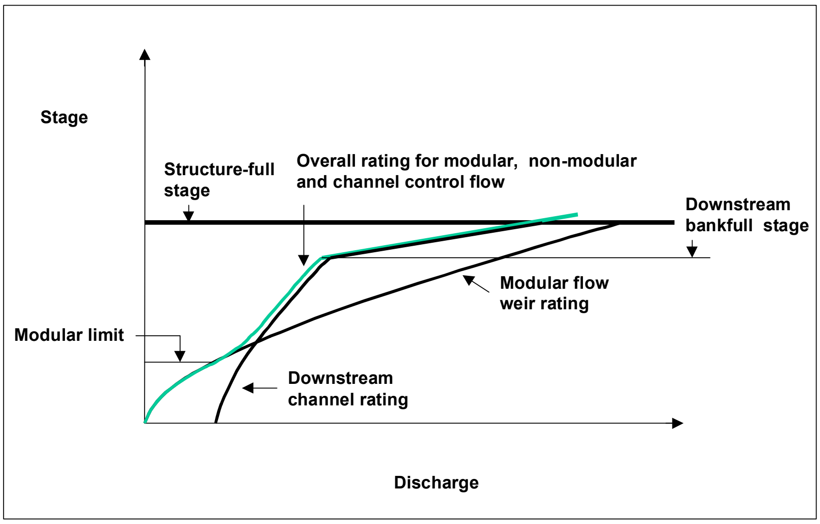

Figure 2.7: Structure rating curve with drowning and overbank flow downstream

Rating curves are divided into segments, each representing a different flow regime. For example, the rating curve on Figure 2.7 has three segments, representing modular weir flow, non-modular weir flow and channel control respectively. Each segment is expressed as an equation of the following form in the Agency's hydrometric archive system:

$$Q = C(h + a)^{\beta}$$

where: h = river stage (m)
Q = river flow (m^3/s)
C, a, β = constants

It should be remembered while working with the power law coefficients C, a and β, that these coefficients do have a physical significance as they were originally derived from hydraulic theory.

The coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases. Thus, if an open channel site is known to suffer from weed growth, the correct amendment to the coefficients should be to reduce the C coefficient as the channel roughness will be higher. It can be shown that for simple geometries the β coefficient is constant. In the example of an open channel site with weed growth, the β coefficient should remain unchanged. a is related to the elevation of the bed, and should also remain unchanged in this example.

2.3 Introduction to Methods for Extending Rating Curves

Rating curves at gauging stations are usually derived either from a hydraulic analysis of a gauging structure, or from a series of paired stage and discharge measurements using current meters, Acoustic Doppler Current Profilers (ADCP) and other measurement techniques. In either case, the rating curve will have upper and lower limits to its range of applicability. Simple hydraulic techniques and computational models offer a range of techniques to extend rating curves, under certain hydraulic conditions, to higher or lower flows for which current meter gaugings have not, or cannot, be obtained.

The approach does, of course, assume that a rating curve can be defined. This will not generally be the case where the stage - discharge relationship at the gauging station is controlled by a variable backwater effect, such as weed growth or tidal drowning, or the flow dynamics cause significant hysteresis in the rating. In such cases, the techniques could be adapted to produce a family of rating curves covering the different conditions (see Figure 2.6). This will only be worthwhile if the system used to calculate flows from measured stage values can support such processing. The functionality to process weed-affected stage data to flows will be available within the WISKI system.

The methods for deriving rating curves at river level and flow measurement stations are well established and documented in various British and International Standards, whether based on hydraulic principles or an empirical fit to gauging data. However, the extrapolation of rating curves to higher flows is hindered mainly by difficulties in actually measuring extreme flood flows to verify any such extensions. These problems relate to access, heath and safety and timing of measurements to coincide with flood peaks.

Hydraulic methods exist for extrapolating medium flow ratings to the high flow range as summarised in British and International Standards (ISO 1100/2:1998 Annex D). These methods are generally reasonable where the watercourse is confined within the channel by the river banks. However, there are many cases where the methods can be inaccurate, particularly when a flooded watercourse inundates floodplains or flow bypasses gauging sites. Although the Standards referred to above provide a description of the various methods available, there are no details of the calculations required, the potential pitfalls in using the various methods or worked examples. The manual is aimed to address these gaps.

Extension of rating curves to low flows is problematic because the rating will be affected by relatively minor local changes including roughness variation, changes in bed profile and changes in the condition of structures. Ratings for Standard structures developed using standard weir formulae already cover the full flow range. Whilst the gauge zero for weirs is normally at the zero flow stage (i.e. the lowest point on the weir crest), this is not true of open channel sections where the gauge zero may be above or below the current bed level.

2.4 Selection of Rating Curve Extension Method

The primary criterion for selecting the rating curve extension strategy should be the hydraulic correctness of the method. The methods covered in this manual are as follows:

- Simple hydraulic techniques:
  - Simple extension of the existing rating curve;
  - Logarithmic extrapolation of the existing rating curve;
  - Weir formulae for modular and non-modular flow;
  - Velocity extrapolation method 1: extrapolation of velocity against stage;

- Velocity extrapolation method 2: extrapolation of velocity against hydraulic radius;
- Velocity extrapolation method 3: extrapolation of flow against geometric properties of the Manning equation;
- Slope-area method; and
- Divided Channel Method (DCM), which is a variation of the Slope-Area method for sites with overbank flow.

- Computational hydraulic modelling:
  - 1-D
  - 2-D
  - 3-D

Tables 2.1 and 2.2 are provided to assist with selection of the rating curve extension method. Table 2.1 summarises the approaches that should be considered under different circumstances, and Table 2.2 provides guidance on other factors that should be considered when planning rating curve extension work. These factors include staff experience, budget, time to undertake the work, data availability, accuracy and risks.

The procedure for selecting the rating curve extension method is given below:

1. Identify the hydraulic conditions at the gauging station in terms of the descriptions given in the first column of Table 2.1. It may be necessary to treat gauging stations with complex flow paths as if they are two (or more) stations within the terms of these guidelines and produce separate ratings for each flow path.

2. For the hydraulic conditions identified in Step 1, read off the corresponding possible methods from the second column of Table 2.1. Also read the remarks in the third column of Table 2.1 and discount any of the possible methods for which the remarks indicate that the method is not applicable to the conditions at your specific gauging station.

3. If Step 2 leads to only one possible method for extending the rating curve, continue with Step 4, otherwise go on to Step 7.

4. From Table 2.2, read off the various factors that are required to implement the selected method so as to get a feel for what is required.

5. Read the appropriate material in either Section 4.1 (simple hydraulic techniques) or Section 5.1 (hydraulic modelling techniques) to confirm the applicability of your chosen method.

6. If Step 5 does not confirm the choice of method, then the reasons for this should be identified and a more appropriate method selected based upon the detailed material in Sections 4.1 and 5.1. Once the method selection is finalised, go to Step 9.

7. Where more than one method has been identified, use the factors set out in Table 2.2 to produce a provisional ranking of the methods in order of preference, giving preference to simpler, cheaper, quicker methods. Do not however discount any of the methods at this step.

8. Read the more detailed descriptions of the methods in Section 4.1 (simple hydraulic techniques) and / or Section 5.1 (computational models), and revise the provisional ranking from Step 7 in the light of this to give a final ranking. The top ranked method is the one to use. In some cases it may be appropriate to apply more than one method and compare the results.

9. Once the method(s) has been selected, plan the rating extension work (Sections 4.1 or 5.1) and carry out the work using the step by step procedures given in Section 4.2 for simple hydraulic techniques or Sections 5.2 and 5.3 for hydraulic modelling techniques.

As the simple hydraulic techniques are relatively quick to apply, detailed guidance on which method(s) to use are given as part of the step by step procedure in Section 4.2.3. The choice of computational modelling method should ideally be made at the planning stage, and information to assist with method selection is given as part of the overview in Tables 5.1 and 5.2.

2.5 Combinations of Methods

It is possible to combine these methods under certain circumstances, including the following:

- The use of the simple techniques as part of an initial analysis to assess whether a more complex technique is needed and, if so, what technique should be used;
- A combination of 3-D and 1-D computational models at complex structure sites. The 3-D computational model is used to determine the detailed hydraulic performance of the structure. The results are then incorporated in a less detailed 1-D model of a longer length of river; and
- The possible use of 1-D model at low flows and a 2-D model at higher flows where significant bypassing or floodplain inundation occurs.

Table 2.1: Selection of methods for rating curve extension

Type of rating extension required	Recommended methods	Remarks and guidance on selection where there is a choice
Open channel sites (channel control)		
Note: The rating curve at an open channel site may be affected by variable backwater effects (for example, vegetation growth and structure operation). In such cases a family of rating curves may be needed for different downstream conditions.		
Low flow extension	Simple extension; Velocity extrapolation methods 1 and 2.	Manning's equation generally unsuitable at low flows.
High flow extension to bankfull	Simple extension; Velocity extrapolation; Slope-area; 1-D model	Slope-area theoretically best where there are changes in the cross-section shape (for example, flood berms) but is very sensitive to slope measurement and Manning's n estimation. Velocity techniques may be better in practice. 1-D model where backwater effects are important.
High flow extension above bankfull: simple floodplains without embankments where the flow paths are known.	DCM; 1-D model.	Use DCM for uniform cross-section, and no backwater effects. Use model for non-uniform cross-section and/or backwater effects.
High flow extension above bankfull: embanked floodplains; floodplain flow parallel to river and flow paths known.	1-D model.	
High flow extension above bankfull: complex floodplains and flow paths not known.	2-D model.	Steep sided channels can be problematic for 2-D models.

Table 2.1: Selection of methods for rating curve extension (continued)

Type of rating extension required	Recommended methods	Remarks and guidance on selection where there is a choice
Structure sites and natural section controls		
Notes: (1) When using the weir formula for multiple-crested weirs, apply the formula to each section and combine the results. (2) Where a family of rating curves needs to be produced to allow for variable backwater effects (for example, vegetation growth or tidal effects) a greater number of gaugings will be required for calibration.		
High flow extension to structure full, no drowning.	Weir formula for modular flow.	
High flow extension to structure full with drowning.	Weir formula for modular and drowned flow; 1-D model. 3-D model	Need information on downstream rating. If no information, use 1-D model and calibrate using drowned flow gaugings, if any. 3-D model may be used to evaluate the drowned flow characteristics of a non-standard structure.
Overbank extension, no drowning	Weir formula for channel with DCM for floodplains; 1-D model; 3-D model.	Use 3-D model only if channel / floodplain interaction significantly affects the rating.
Overbank extension with drowning	Weir formula for modular and non-modular flow in channel with DCM for floodplains; 1-D model; 2-D model (wide floodplains); 3-D model.	Need information on downstream rating. If no information, use model and calibrate using drowned flow gaugings, if any. Use 3-D model only if channel / floodplain interaction significantly affects the rating or to evaluate the drowned flow characteristics of a non-standard structure.
Complex structure including gates	1-D model; 3-D model.	Need information on downstream rating. If no information, use 1-D model and calibrate using drowned flow gaugings, if any. 3-D model particularly useful for non-standard structure without much gauge data.

Table 2.2: Factors affecting choice of rating curve extension strategy

Method	Staff experience	Budget	Time to undertake the work	Data requirements	Risk of inaccurate results (1)	Complexity
Simple extension	Basic	Very low	Short	Low	High	Low
Logarithmic extrapolation	Basic	Very low	Short	Low	High	Low
Weir formulae	Basic	Low	Short	Medium	High	Low
Velocity extrapolation: Method 1 Method 2 Method 3	Basic	Low	Medium	Medium	High	Low
Slope-area	Basic	Low	Medium	Medium	High	Low /Medium
Divided channel (DCM)	Basic	Low	Medium	Medium	High	Medium
1-D model	Modeller	Medium(2)	Long	High	Medium	Medium
2-D model	Specialist	High	Long	High	Medium	High
3-D model	Specialist	High	Long	High	Low	High

Notes:
(1) All methods will produce reasonably accurate results when applied appropriately
(2) Could be high for very complex sites

Explanation of terms used in Table 2.2

Staff experience

Basic:	Competent hydrometric staff
Modeller:	Competent user of 1-D modelling software
Specialist:	Competent user of 2-D and 3-D modelling software

Budget (external costs only)

Very low:	< £1,000 to £2,000
Low	£1,000 to £3,000
Medium	£3,000 to £12,000
High	£12,000 to £40,000

Duration of work

Short	< 1 day
Medium	1 to 3 days
Long	3 to 14 or more days

Data requirements

Low	Standard hydrometric data
Medium	Standard hydrometric data plus local cross-section surveys (one to three) and/or structure details
High	As Medium plus more extensive channel and floodplain surveys

Risk of inaccurate results

Low	Most of relevant physics included in the model
Medium	Interpretation and calibration required to achieve good results
High	Considerable scope for inappropriate application of methods

Hydraulic complexity

Low	Simple, approximately prismatic channel local to open channel gauge or British Standard measuring structure
Medium	Possibly non-prismatic channel or overbank flow, but flow paths well understood
High	Non-standard structure or significant overbank flow

3. RATING REVIEW AND HYDRAULIC ASSESSMENT

Before extending a rating curve, the following work is required:

- A review to see whether the rating curve is suitable for extension. This can be done by undertaking a rating and gauging review as described in Section 3.1; and
- Assessing the hydraulics at the gauging site, in order to determine the type of extension required. A method for assessing the hydraulics is given in Section 3.2.

3.1 Rating and Gauging Review

Before a rating curve is extended it should be reviewed to ensure it is sufficiently accurate and represents the hydraulic conditions at the control. This can be achieved by comparing each rating with all available gaugings and undertaking both a qualitative and quantitative review. It is prudent to undertake such a review for built as well as natural controls since even purpose built weirs can suffer from sediment accumulation, weed growth or structural movement. The procedure is given in Sections 3.1.1 and 3.1.2 below.

3.1.1 Qualitative review of ratings

An initial assessment of the rating should be carried out to assess whether there is any evidence to suggest that the gauge has changed with time. In such cases checks should include:

- Reviewing evidence of structural change; and
- Reviewing evidence of upstream and downstream change.

This will be particularly important at sites that do not have any flow gaugings. This is usually because they are standard structures and the rating is based on the standard flow formulae. In such cases it is also important to assess whether the rating may be affected by drowning of the structure.

A qualitative examination of available gauging data will allow any changes with time in the control to be determined. The following will assist to determine whether a rating is stable.

1. Plot gaugings on a rising and falling stage to see if hysteresis or a different relationship between stage and flow occurs.

2. Plot gaugings in 2 to 5 year time periods (depending on the length of the gauging record) against the rating. This can reveal whether changes in the control have occurred due to erosion, settlement, sedimentation etc. Where changes in a control are known to have occurred, the periods should be selected taking account of the dates of the changes.

3. Plot gaugings in winter and summer time periods – the definition of which will vary depending mainly on geographic location of the gauge. This can reveal whether there is a seasonal change in the control which could suggest weed growth, vegetation and/or sedimentation.

3.1.2 Quantitative review of gaugings

A quantitative review of gaugings and ratings can indicate whether the existing rating is sufficiently accurate for an extension to be undertaken. The following is recommended and further details can be found in the various British Standards (BS):

1. Calculate the deviation (as a value and percentage) of the gauged flow from the theoretical rated flow for each gauging.

2. An investigation of the accuracy of individual gaugings that either show significant deviation from the other gaugings and/or have a significant impact on the upper end of the rating curve should be carried out. 'Significant' in this context is not defined here as it needs to be left to the judgement of the person undertaking the review. In order to guide this judgement, examples of data points that should be investigated are shown on Figures 3.1 and 3.2.

   On Figure 3.1, the flow gauging of 15 cumecs at a stage of 0.43m requires investigation. On Figure 3.2, the flow gaugings of less than 30 cumecs at a stage of about 2.5m require investigation, although from knowledge of the site the cause is likely to be vegetation in the channel. If the investigation reveals doubts about the accuracy of these gaugings, they should not be used and the rating curve should be re-calculated. The removal of data points must be rational, fully justifiable and undertaken in full consultation with the Agency Hydrometric Team since each point could form a valid part of the gauging data set.

3. Calculate the standard error (SE) of the rating and individual limbs and compare with the indicative thresholds to determine whether the rating is acceptable. The method for calculating SE is given in BS 3680 Part 3C. As an indication of thresholds for SE, hydraulic structures conforming to British Standard should have an SE of better than ±10%, other weirs and structures better than ±20% and natural controls better than ±25%. These thresholds are arbitrary but define an approximate level at which recalculation of the rating may be required.

4. Plot the deviations calculated in step 1 against time and stage. This can reveal much about the performance of a rating throughout the history of the control and any tendencies to over or underestimate flows at given stage ranges.

5. The SE gives the accuracy of the rating but not its tendency to consistently under or over estimate flows as the square of the deviations is used in the calculations. This can be examined by calculating the mean deviation of the gauging flow from the rating flow. Calculate the mean deviation and compare to an arbitrary limit, chosen from experience to indicate which ratings warrant further consideration. The arbitrary limit could be a percentage (say 10%) of the bankfull discharge for all gaugings above a certain threshold.

6. For each successive gauging calculate and plot the cumulative deviation with time. A plot of cumulative deviation with time can reveal whether the tendency to under or over estimate has a historical significance which could suggest gradual changes in the control have occurred.

Only when a rating and gauging review has been undertaken, and the rating confirmed as suitably accurate for the control, should the various methods for rating extension be considered. A methodology for rating review is given in Figure 3.3.

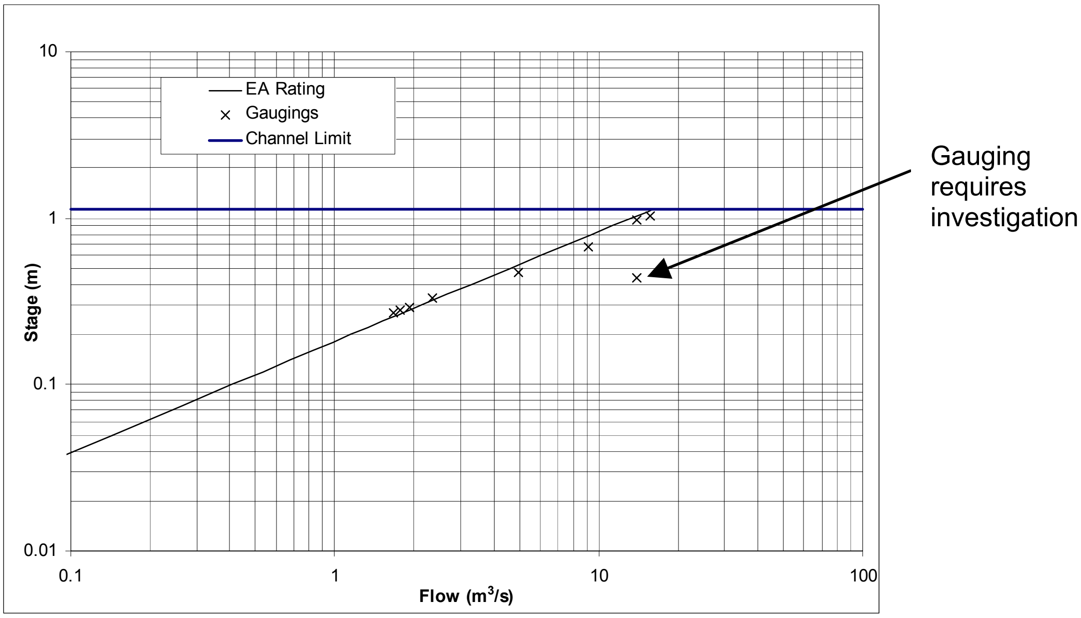

Figure 3.1: Rating curve and gaugings at a Crump Weir gauging station

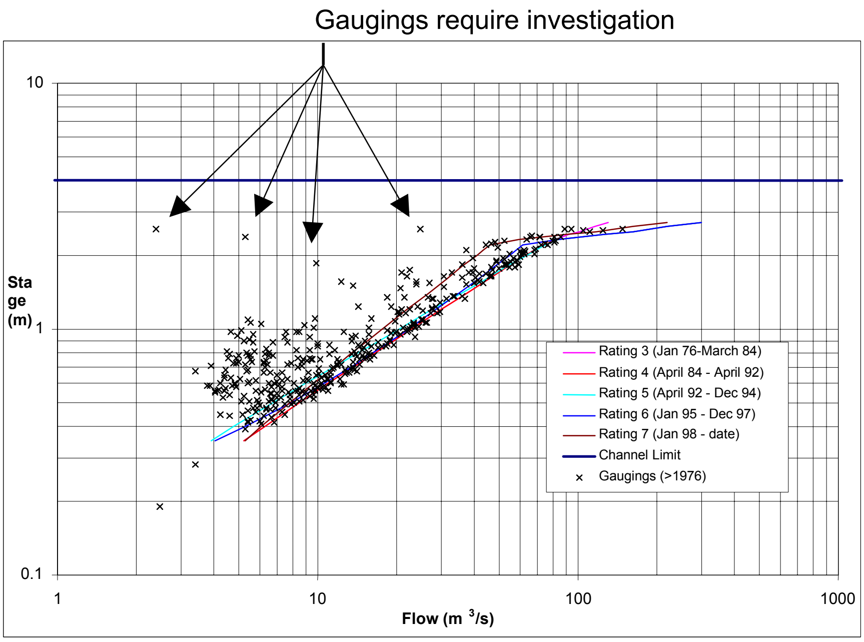

Figure 3.2: Rating curve and gaugings at an open channel gauging station

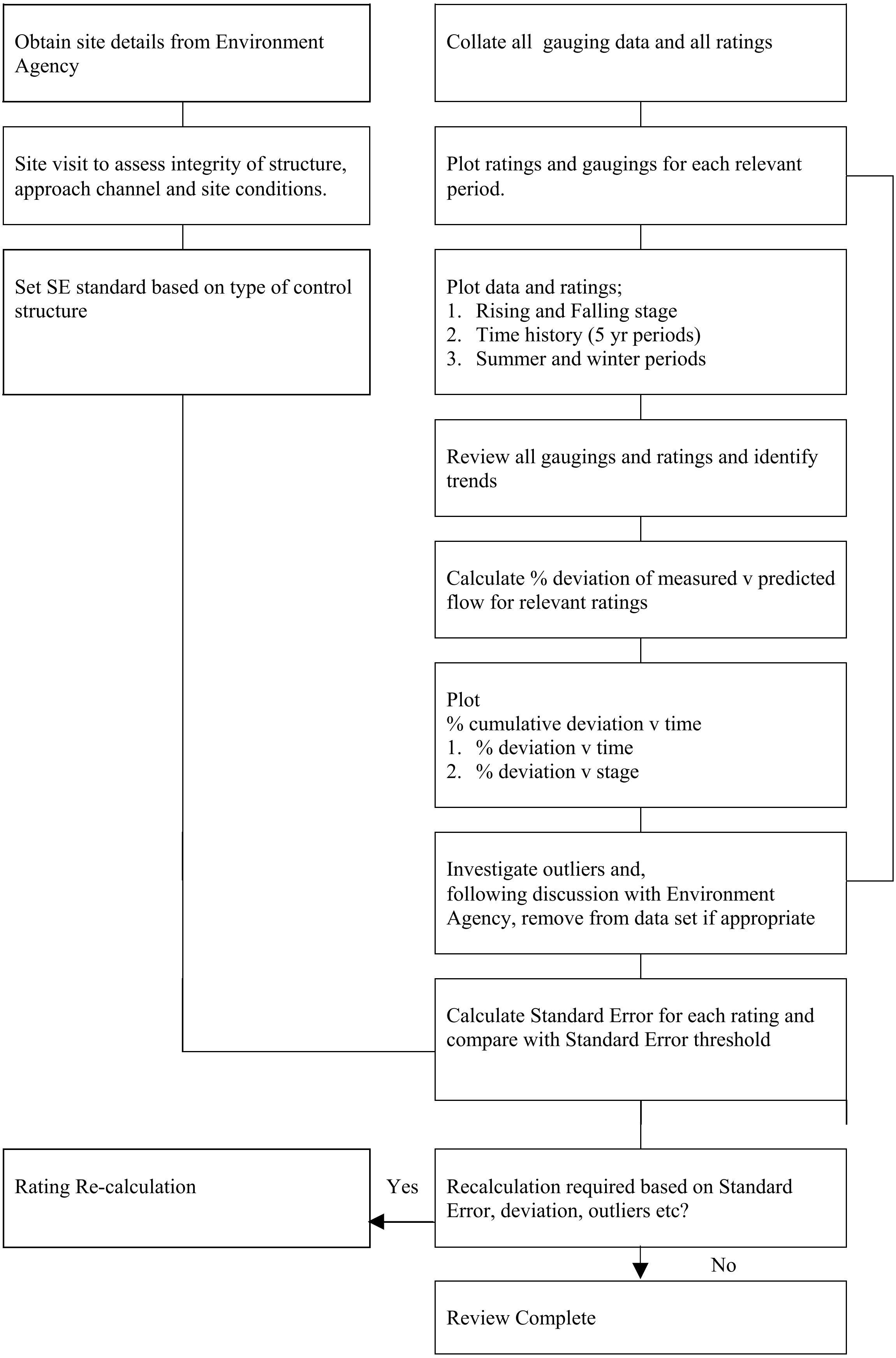

Figure 3.3: Methodology for Rating Review

3.2 Understand the Hydraulics

An assessment of the hydraulics is needed in order to:

- Determine the type of rating extension needed; and
- Develop a more detailed understanding of the site hydraulics in order to select the most appropriate approach.

The steps needed to understand the hydraulics are as follows:

- Obtain a map of the gauging site at a suitable scale for planning purposes (1:1250 or 1:2500 are suggested, depending on the size of river);
- Visit the site at normal flows to appreciate the topography and other relevant features;
- If possible, visit the site at high flows to identify hydraulic behaviour;
- Inspect any available photographs and video of the site, particularly for high flows;
- Discuss the site with the Agency's Area Hydrometric Team, particularly staff who have visited the site over many years in all conditions;
- Inspect data on the site which is held by the National River Flow Archive at CEH Wallingford, or Agency hydrometric archives including paper based information;
- Establish the range of the existing rating, and stages where changes in the cross-section occur;
- Establish the range of flows for which the rating extension is required; and
- If possible, establish the modular limit for structures and the stage/flow when channel control occurs.

In addition, where the gauging site or river has been modelled in the past, contact should be made with Agency staff and / or consultants who either undertook or supervised the modelling.

A cross-section of the river at the gauging site should be obtained either from existing information or by survey. The ranges covered by each segment of the existing rating curve should be shown on the cross-section. This cross-section is also needed to assess the range of applicability of the simple hydraulic techniques as it will show the stages at which the cross-section changes shape, for example at bankfull stage.

A brief planning document should be prepared which includes a base plan of the site and a statement of:

- The type of section and leading dimensions;
- Information on changes in cross-section shape with stage, which will affect the choice of method;
- In-bank and out-of-bank flow paths for the full range of stages / flows for which the extension is required. The estimated flow paths should be shown on a copy of the plan;
- The maximum flow, which should generally be at least equal to the estimated 1 in 200-year flow;
- The geometry and roughness characteristics of the channels and floodplains;
- Likelihood and impact of sediment movement, if any;

- Structures and natural controls that will affect the rating:
    - Structures may include bridges, weirs, flumes, sluices, gates, culverts
    - Controls may include rapids and features of the channel;
- Drowning of structures;
- Any variable backwater effects, for example tides, weed growth or backing up from a gated structure or confluence;
- Flood banks and other floodplain features that will affect the flow, for example roads and railway embankments;
- Whether unsteady flow will affect the rating; and
- Any relevant natural or man made storage areas.

4. EXTENSION OF RATING CURVES USING SIMPLE HYDRAULIC TECHNIQUES

4.1 Overview

This section of the report is aimed primarily at staff involved in planning, commissioning, supervising and accepting rating curve extension work.

4.1.1 Background to the methods

The approach for extending rating curves using simple hydraulic techniques can be divided into 5 methods:

- Simple extension (Section 4.1.3);
- Log extrapolation (Section 4.1.4);
- The Weir equation (Section 4.1.5);
- Velocity extrapolation (Section 4.1.6); and
- Slope-Area methods (Section 4.1.7).

These methods are described in the following sections. However, on the basis that extension almost always involves the use of an existing rating it is considered essential that a rating and gauging review should precede any rating extension exercise. This is described in Section 3.

4.1.2 Theory and assumptions behind simple hydraulic techniques

Simple hydraulic techniques are primarily concerned with extending an existing rating curve in a number of different ways, as follows:

- Simple extensions and log extrapolations extend the rating curve using the rating equation for the highest limb of the existing rating curve;
- The weir equation extends the rating curve for weirs using standard weir flow formulae for modular and drowned flows;
- Velocity extrapolation methods calculate the rating curve extension by extrapolating key parameters (velocity or geometric properties of the section at the site) beyond the limits of the existing rating; and
- Slope-Area methods extrapolate the rating curve using a flow resistance equation.

All of these methods, except a variation of the Slope-Area method called the Divided Channel Method (DCM), extrapolate from properties of the existing rating. They are therefore generally not suited to a change in hydraulic conditions, for example from in-channel to overbank flow. The DCM is able to take account of changes in hydraulic conditions to some degree although it contains only a very simple hydraulic representation.

Further background to each individual method is given in Sections 4.1.3 to 4.1.7.

4.1.3 Simple extension

Background
The simplest form of rating curve extension consists of applying the upper segment of the existing rating curve to stages higher than its current maximum limit, or conversely for the lowest segment for lower stages. Thus for a rating in Agency format the same equation and parameters are assumed to apply for a wider range of river stage and flows.

Theory
The basic equation is:

$$Q = C\left(h + a\right)^{\beta}$$

Where: h = river stage (m)
Q = river flow (m^3/s)
C, a, β = constants

The method involves calculating the flow for stages which are higher than the existing rating. If 'a' in the rating is equal to 0.0 this method is effectively the same as extending a rating linearly on a log-log plot (Section 4.1.4).

Applicable hydraulic conditions
Simple extension can be used where the channel section rises reasonably uniformly above the highest stage on the existing rating. Thus it is suitable for extending the rating for:

- In-channel flows including modular flows at structures or drowned flow where the rating to be extrapolated already represents drowned flow, unless the degree of drowning changes in an irregular way; and
- Floodplain flows where the existing rating includes floodplain flow on the full width of the floodplain, and the flooded width will not increase significantly. In practice there are very few rating curves that fulfil this criterion because of the difficulties of floodplain flow gauging.

The method should not be used in the following cases:

- The transition from in-channel to overbank flow;
- Above bankfull level with the exception of the floodplain flow case referred to above;
- The transition from modular to drowned flow;
- The transition from drowned structure flow to channel control; and
- The method does not make allowances for changes due to vegetation, cross-section changes, etc.

Data requirements
Data requirements are as follows:

- The parameters of the existing rating are essential;

- A cross-section of the site is desirable showing the range of stages covered by the existing rating. This is used to identify transitions such as the bankfull limit, above which the method should not be applied; and
- Where the method is applied to structures, information on the stages where drowning and channel control occurs.

Available software
Gaugeman/Hydrolog use this Simple Extension approach when the 'rating curve extrapolation' option is selected, but without specifying an upper limit to the extended rating curve. However, the preferred method of implementing the technique is to set an appropriate, higher limit to the existing curve's limit of applicability, and switch-off the rating curve extrapolation option above this limit.

4.1.4 Logarithmic extrapolation

Background
Logarithmic Extrapolation is a similar approach to the simple extrapolation method but with stage and flow plotted on log-log paper. The method is described here as it is included in the British Standard but it is recommended that it is NOT to be used as it has the potential to generate very large errors.

Theory
The basic theory is similar to simple extension (Section 4.1.3). The existing rating is plotted on log-log paper and the extension sketched by hand, by extending the upper part of the rating. This can be inaccurate if the curvature of the rating is not equal to 0 and in many ways is inferior to simple extension.

The benefit of logarithmic extrapolation occurs when the current rating goes as high as the highest gaugings, but the highest gaugings indicate a different slope to the upper segment of the current rating curve. The method would allow this change of slope to be taken into account.

However this situation suggests that the existing rating requires improvement to take account of gauged flows. Extrapolation may not be appropriate as it could lead to large errors. It is recommended that the technique is generally not used as it requires considerable expertise to be used successfully.

Applicable hydraulic conditions
Logarithmic extrapolation can, like simple extension, be used where the channel section rises reasonably uniformly above the highest stage on the existing rating. Thus it is suitable for extending the rating for:

- In-channel flows including modular flows at structures or drowned flow where the rating to be extrapolated already represents drowned flow; and
- Floodplain flows where the existing rating includes floodplain flow on the full width of the floodplain, and the flooded width will not increase significantly. In practice there are very few rating curves that fulfil this criterion because of the difficulties of floodplain flow gauging.

Like simple extension, the method should not be used in the following cases:

- The transition from in-channel to overbank flow;
- Above bankfull level with the exception of the floodplain flow case referred to above;
- The transition from modular to drowned flow;
- The transition from drowned structure flow to channel control; and
- The method does not make allowances for changes due to vegetation, cross-section changes, etc.

BS ISO 1100/2:1998 suggests that large errors can result from inappropriate application and recommends that:

- If the shape of the control does not change significantly, and the channel roughness remains fairly constant, then a straight-line extrapolation on a log-log plot is reasonable;
- The method is particularly suited to channel control conditions for medium and high flows;
- Logarithmic extrapolation should not be used to extrapolate more than about 1.5 times the highest measured discharge; and
- For extensions to very low flows, where section control exists, it is essential that the shape of the control is accounted for, and that the gauge height of zero flow is known. In this situation it is often better to plot the rating on arithmetic axes so that the zero-flow gauge height can be plotted.

As with the simple extension the upper limit may also be constrained by the bankfull limit or the next or any change in the cross-sectional shape, where a different stage-discharge relation may apply. Extension beyond this limit is not recommended.

Data requirements
Data requirements are as follows:

- The parameters of the existing rating are essential;
- A cross-section of the site is desirable showing the range of stages covered by the existing rating. This is used to identify transitions such as the bankfull limit, above which the method should not be applied; and
- Where the method is applied to structures, information on the stage where drowning occurs.

4.1.5 The weir equation

Background
Many standard gauging structures consist of weirs. These have standard rating curves based on the general weir equation but with particular coefficients for each type (e.g. Crump weir, Flat-V weir, Broad Crested weir etc). Where the control at a gauging station consists of a weir, it is possible to extend the rating curve using the general weir equation with the coefficients adopted according to the type of weir control.

Theory
Weirs operate under two conditions – modular and non-modular flow. Where there more than one weir crest (for example, at a compound weir) separate calculations for each weir crest should be carried out and the results combined.

Modular flow

The general equation for modular flow at a weir with a horizontal crest (e.g. Crump weir) is:

$$Q = C\ bH^{1.5}$$

where: Q = flow (m^3/s)
C = a weir coefficient
b = crest width (m)
H = the total head, relative to the zero-flow stage (m)

The weir coefficient (C) is often expressed as;

$$C = C_d \left(\frac{2}{3}\right)^{1.5} \sqrt{g}$$

where: C_d = a coefficient of discharge based on weir geometry and head
g = acceleration due to gravity

Values of C can be estimated by plotting calculated C values in the current rating range against stage and extrapolating this curve upwards. Alternatively C_d can be calculated from equations given in the relevant British or International Standards.

It is important to note that H is the total head, not the gauged head. The total head is the sum of the gauged head (h) and the velocity head ($V^2/2g$). Thus when calculating C for the current rating, it is necessary to calculate the velocity at the cross-section where the water level is gauged and add the velocity head to the gauged head to obtain total head (H).

The method may be used to extend ratings to the structural limit (e.g. the top of any wing walls) or to the next or any change in the cross-sectional topography providing the modular limit is not exceeded (i.e. the weir does not become drowned by high downstream water levels).

There is a different power law for Flat-V weirs where the crest is sloping.

Non-modular flow

The general weir equation is only valid if the tail water level is low enough for there to be a free discharge over the weir. Once downstream water levels interfere with upstream water levels the weir is said to be operating under drowned conditions and the general weir equation will not apply. For most weirs the modular flow relationship continues to apply until the submergence ratio (i.e. the ratio of downstream to upstream total head over the weir crest, H_2/H_1) exceeds a factor, the 'modular limit', which varies depending on the type of weir. For a broad crested weir the modular limit is 0.66 whilst for a Crump weir it is 0.75.

Standard weirs have a modified equation for drowned flow which includes the drowned flow reduction factor (f). The general weir equation for drowned flow is:

$$Q = C\,b\,f\,H^{1.5}$$

where: Q = flow (m^3/s)
C = weir coefficient
b = crest width (m)
f = the drowned flow reduction factor
H = the total head, relative to the zero-flow stage (m)

For this equation to be applied it is necessary to calculate the submergence ratio and the drowned flow reduction factor, which requires records of both upstream and downstream water levels for a range of flows. This assumes a stable downstream rating for the river channel although, in practice, the downstream water levels may be affected by variable factors such as vegetation.

The drowned flow reduction factor f is calculated from the submergence ratio. The formula for the drowned flow reduction factor depends on the type of weir, and can be found in the British Standards for certain types of weir.

If no downstream water levels are available but cross-sectional data exist, it is possible to calculate downstream water levels either manually or using simple hydraulic models. This will provide an estimated stage discharge curve for downstream of the structure. This is used to identify the flow at which the modular limit is reached. It is also used to calculate the drowned flow reduction factor for the weir and hence the drowned flow rating curve. However this approach is subject to a high uncertainty if there are no water level data downstream of the structure for calibration.

It is advisable to plot the downstream rating on the structure rating in order identify when drowned flow is likely to occur and the likely shape of the drowned flow section of the rating. This will also help to calibrate the drowned flow performance of the structure where data on flows and water levels are only available upstream of the structure.

Crump and Flat-V weirs may be fitted with crest tappings and drowned flows calculated from double gaugings.

Applicable hydraulic conditions

The weir equation is suitable for extending the rating for weirs under both modular and drowned flow. This equation can also be used for rating extensions under section control (BS ISO 1100-2: 1998).

The method generally cannot be used above structure-full level (or the next change in the cross-sectional shape if any) although it is possible to make an estimate of overbank flow by combining the weir equation with the divided channel method. In this case the weir equations are applied to the channel and the Slope-Area method to the floodplains.

Care is also needed when applying the drowned flow equation because the control at a site often changes from structure to channel control at high stages. Once channel control occurs, the weir no longer has any significant effect, and the drowned flow equation is no longer applicable. The point at which channel control starts to affect the rating can be estimated by plotting the downstream channel control rating curve and the modular flow weir rating curve on the same plot. The required point is approximately where the curves cross, bearing in mind that they are taken at different sections and adjustment may be needed to take account of the channel slope between the sections.

Data requirements
Essential data requirements are as follows:

- Weir level(s) and zero flow gauge level (normally lowest crest level);
- Weir width(s);
- Cross-section at the head measurement section to permit the calculation of upstream total head;
- Gauged calibration data and/or calculated value of C_d , the coefficient of discharge;
- Modular limit; and
- Formula or graph for the drowned flow reduction factor f.

One of the greatest problems with standard structures is that they do not always conform to the Standards. For example, siltation may have occurred upstream of the weir thus affecting the weir height, or the tapping points may be in the wrong place. If the standard values are to be applied, a check should be made of conformance of the structure to the relevant Standard.

Thus, whilst C_d has a standard value for standard structures, calibration using actual gauged flows is advisable. With regard to drowning, standard structures have standard modular limits and submergence factors. However, it is advisable to check these values at each particular site using data on upstream and downstream water levels and corresponding gauged flows.

Where the weir formula is applied to non-standard structures, it is necessary to calibrate C_d under modular flow conditions using gauged data, and both the modular limit and drowned flow reduction factor using data on upstream and downstream water levels and corresponding gauged flows.

4.1.6 Velocity extrapolation methods

Background
The velocity extrapolation methods provide three approaches to finding a best-fit equation between various channel or hydraulic parameters, and extending this relationship to the high flow region on the assumption that the same relationship applies. The three methods are;

- The Simple Approach, where velocity is extrapolated against stage;
- Hydraulic Radius Approach, where the velocity is extrapolated against hydraulic radius (i.e. the channel dimensions are assumed to be similar in the extended range); and
- Manning's Approach, where the flow is extrapolated against the geometric properties of the Manning equation.

These are described below.

Theory: Simple approach
This method requires calculation of the cross-sectional area and velocity at incremental river stages based on the flow calculated from an existing rating curve. The rating curve extension is based on an extrapolation of velocity against stage. To apply the method, one cross-section is required at the gauging site to calculate the change in cross-sectional area with stage. This method is based on the assumption that the velocity-stage curve usually exhibits little curvature. Once the cross-sectional area and velocity are known in the extended range, the flow is calculated from the continuity equation ($Q=VA$).

Theory: Hydraulic radius approach
A plot of velocity against the hydraulic radius ($R=A/P$) often shows a linear relationship and this too can be used to provide values of velocity in the extended range. On the basis of the derived velocity and cross-sectional area, the flow in the extended range can be calculated using the continuity equation.

Theory: Manning's approach
Another variation of this method is to use Manning's equation:

$$Q = AV = \frac{AR^{2/3}s^{1/2}}{n}$$

Where: Q = flow (m^3/s)
A = cross-sectional area (m^2)
V = mean velocity (m/s)
R = hydraulic radius (m)
s = water surface slope
n = Manning's resistance coefficient

Assuming $s^{1/2}$ and Manning's n remain constant across the full range of flows, a curve can be prepared for Q against $AR^{2/3}$ for the rating curve. This is extrapolated to provide values of flow against stage beyond the limits of the existing rating.

This approach can also be used with the Chezy roughness formulation instead of the Manning formula. In this case, the approach is known as Steven's Method.

Applicable hydraulic conditions
Velocity extrapolation can be used where the channel section rises reasonably uniformly above the highest stage on the existing rating. Thus it is suitable for extending the rating for:

- In-channel flows including modular flows at structures and drowned flow where the rating to be extrapolated already represents drowned flow.

The method should not be used in the following cases:

- The transition from in-channel to overbank flow;
- Above bankfull level;
- The transition from modular to drowned flow;
- The transition from drowned structure flow to channel control; and

- The method does not make allowances for changes due to vegetation, cross-section changes, etc.

The current Standard (BS ISO 1100-2) states that it is often difficult to estimate the velocity-stage and velocity-hydraulic radius relationships accurately in the extended range and Manning's method relies on a constant slope and *n* value. For this reason the three methods detailed above, although simple to use, are often regarded as inferior to the Slope-Area method described below. They do, nevertheless, form an approach for the extension of rating curves where only a single cross-section is available.

As with simple and logarithmic extensions described in Sections 4.1.3 and 4.1.4 the Velocity Extrapolation approach is only valid to the bankfull stage and any floodplain or bypass channel flow should be calculated separately.

Data requirements
Essential data requirements are:

- A single cross-section at the rating site covering the full range of required water levels (and flows); and
- The parameters and limits of the existing rating.

4.1.7 Slope-Area methods

Background
The Slope-Area methods are based upon estimating the cross-sectional area, hydraulic radius, Manning's *n* and water surface slope of the channel at the gauging station. Full descriptions of the method can be found in standard hydraulics textbooks, e.g. Chadwick and Morfett (1998).

There are two approaches, one for simple channels and a variation for compound channels (the divided channel method) but these essentially rely on the same approach. BS ISO 1100-2 states that these methods can be used to extrapolate the high end of rating curves under channel control and, when properly applied, are the most hydraulically 'correct' of all the simple techniques. As such these methods are often preferred.

Theory: Simple approach
The principle of the approach is that cross-sectional area and water surface slope of the measuring reach are determined for a range of flows and the mean velocity calculated using Manning's equation. Other friction laws, such as Chezy's equation may also be used, but Manning's is generally preferred. The discharge is then computed using the Manning equation for different river stages. Manning's equation is:

$$Q = AV = \frac{AR^{2/3}s^{1/2}}{n}$$

Definitions are given in Section 4.1.6. The cross-sectional area and hydraulic radius are calculated for different values of stage based on a cross-sectional survey of the gauging station. BS/ISO 1070 recommends that at least three sections should be used in order to provide an average section for the reach.

Both Manning's n and the water surface slope will vary with stage, so the method requires that both are estimated for different values of stage. The corresponding flows are then calculated and a rating equation can be fitted to the calculated set of flow and stage values.

Accurate measurements of the water surface slope s are difficult to obtain, either because it is not measured or water level differences between measurement points are small and the measurement error is significant. It can be taken from water level recorders or gauges boards at two different locations upstream and downstream of the gauge site. The friction slope S_f is often used as an approximation to s. This approximation is generally valid in uniform river reaches which are unaffected by backwater and drawdown effects. The friction slope is assumed to be equal to the average slope of the river channel. Hence the average bed or bank slope can be used as an approximation to water surface slope in uniform flow.

Theory: Compound channels – the divided channel method

The simple approach to the Slope-Area method described above generally applies to cross-sections where there are no abrupt changes in the channel cross-section. The simple approach does not permit accurate extrapolation of ratings for over-bank conditions where sharp changes in the wetted perimeter, and hence hydraulic radius may occur. This will affect the calculated rating extensions above the bankfull or structure limits. In addition, the simple Slope-Area approach cannot be used for overbank flow because values of n on the floodplains will differ from in-channel values.

Much research has been undertaken on flow in compound channels, and several methods for estimating overbank rating curves have been developed in the last 10 to 15 years. These include methods for straight channels (Ackers method) and for meandering channels (James & Wark method) although these are somewhat complex for hand calculation (NRA 1994). As a first approximation, the "Divided Channel Method" (DCM) is suggested and is detailed by Ramsbottom (1989), particularly at sites where over-bank gaugings exist. This is simply an extension of the Slope-Area approach.

The DCM requires a surveyed cross-section of the river channel and floodplain. The channel and floodplain are then divided into three separate components using vertical division lines – the channel, the left bank floodplain and the right bank floodplain. The channel rating is extrapolated above bankfull assuming there is no friction on the vertical boundaries above bankfull. Separate ratings are then calculated for the floodplain components using the Slope-Area approach and using estimated values of n. The rating curves for the channel and floodplain components are added to obtain a rating for the section.

Applicable hydraulic conditions

The Slope-Area method may be applied for open channel sites. The method can be used to predict the effects of changes in roughness and cross-section shape.

The Slope-Area method is the most hydraulically correct of the simple methods for channel control situations. However it takes no account of longitudinal changes along the river valley, including changes in channel shape and backwater effects. It is therefore generally applicable to uniform reaches. Uncertainties will increase where the method is applied to non-uniform reaches (for example, where the section changes significantly along the reach or there are bends, meanders or structures).

The divided channel method may be applied in the following additional cases:

- Straight or gradually curved channels with parallel floodplains; and
- The method can be applied in combination with other methods to provide an approximate overbank rating (for example, weirs with bypassing floodplain flow can be represented using the weir formulae for the channel section and Manning's equation for the floodplains).

The divided channel method should not be used for meandering channels, channels which cross the floodplain on a skew angle, or bends. The method is sometimes applicable locally to short straight reaches in an otherwise meandering river, where the floodplain flow is parallel to the river channel.

The divided channel method does not take account of interference between the fast moving channel flow and the slow moving floodplain flow, and this can be significant particularly at low depths of floodplain flow.

Data requirements
Essential data requirements are:

- At least one and preferably three cross-sections of the river channel (and floodplains where the DCM is to be applied);
- The existing rating and/or flow gaugings at the site in order to calibrate the channel roughness; and
- Estimates of the water surface slope. This can be taken from water level recorders, gauges boards or trash marks at two different locations, upstream and downstream of the gauge site. If none of these data are available, the water surface slope is assumed to be parallel to the bed or river banks. Information on these slopes can be derived from longitudinal sections of the river or an average river bank slope based on contours. The latter is subject to a high degree of uncertainty.

4.1.8 Potential accuracy

Accuracy of the methods depends very much on individual circumstances. Generally speaking all the methods can provide reasonable accuracy (within +/-10%) where the extension takes place within the applicable hydraulic conditions. However, large errors can occur as soon as the methods are extended outside the applicable hydraulic conditions, and they should therefore not be used under these circumstances.

Specific examples of uncertainties estimated in the case studies are as follows:

Open channels for flows which are up to twice the current rating maximum:

> From +/-10% to +/-30% depending on the match between the existing rating and gaugings, the uniformity of the section, and the impact of vegetation on the control.

Weirs where drowning occurs and there are no data on downstream water levels:

> +/- 50% in the region where drowning occurs.
> +/- 30% in the region where flow is under channel control.

The overbank rating of a weir where a 'within structure' rating exists:

> +/- 30%

Complex structure site where the performance of structural components has been determined by physical modelling or detailed analysis:

> +/- 20%

These uncertainty values should not be treated as having general applicability, but may be used as a guide to the general degree of accuracy that can be attained by applying these methods.

Generally the applicable ranges are as follows:

> Simple extension: This can be used within channel where the shape of the channel above the maximum stage of the existing rating is similar to the shape covered by the upper segment of the rating. If the shape changes significantly (for example, the banks widen out and the channel top width increases), errors will immediately be introduced using this method.
>
> The weir equation: This can be used for weirs for both modular and drowned flows up to the structure-full limit or the point at which the weir becomes so heavily drowned that the rating curve is effectively controlled by the downstream channel, whichever is the lower.
>
> Velocity extrapolation: These methods have similar limitations to the simple extension and logarithmic extrapolation. However, because the methods take some account of the physical features of the channel, the errors associated with changes in shape of the river channel above the limit of the existing rating will be smaller.
>
> Slope-Area methods: These methods take account of the physical shape of the river channel above the limit of the existing rating. Errors associated with significant changes in shape above the limit of the existing rating will therefore be smaller than for the other simple methods.

4.1.9 Staff experience required

Whilst the methods are relatively simple, a knowledge of hydraulics is needed for implementation as it is important to understand each step in the process. It is also important that staff are able to know whether or not the answer looks right, and be able to make an independent check if required.

The most important single factor is familiarity with methods. Staff who are regularly involved in applying hydraulic analysis techniques will have little difficulty with the methods. It is therefore important that staff have reasonable continuity in hydraulic analysis to ensure the knowledge is available when needed.

4.1.10 Indicative time and cost of using method

The methods are all simple and easy to apply, and can be applied semi-automatically by software. Thus the actual costs of applying each method at each site will be less than £1,000 and in most cases less than one day's work for an experienced member of staff.

The main cost is likely to be in data collection and, where needed, survey work. Indicative costs of applying the methods including survey work are as follows:

• Simple extension	£200 where the rating curve is available and no cross-section is used £1,200 where a cross-section is surveyed in order to determine limits of the rating.
• The weir equation	£1,000 assuming that all data are available but calibration of the parameters is needed.
• Velocity extrapolation	£1,500 including the survey of a single cross-section.
• Slope-area (simple) method	£2,000 including the survey of three channel cross-sections.
• Slope-area (divided channel) method	£3,000 including the survey of three channel and floodplain cross-sections.

The above costs are indicative for a small river that can be surveyed by a two-man team. Clearly survey costs could vary significantly depending on size and type of river.

The costs do not include the tasks of understanding the hydraulics and the rating and gauging review, as these are a general requirement for all methods for extending rating curves. An indicative cost for these tasks is £3,000 although this will vary depending on the location of the site and the availability of data. All the above indicative costs are for the year 2002.

4.1.11 Risks in using the methods

When using the simple hydraulic techniques to extend rating curves, there are risks that the quality of the rating will be less than that which is required, and that the work takes longer or costs more than expected.

Risks to quality

The principal unsatisfactory outcome will be that the extension will be unacceptably inaccurate or that an extension is found that appears to be acceptably accurate but actually is not. The most important risks that need to be appreciated and minimised in order to avoid these unsatisfactory outcomes are as follows:

- Application beyond the range for which the hydraulic conditions for the method are applicable, for example, by applying simple extension without consideration of the hydraulics;

- A parallel problem is application of the extrapolation methods without reference to a cross-section to identify the limits of the method;
- Errors arising from the use of a single cross-section. For example, if the bankfull level upstream is lower than the gauging site, overbank flow and bypassing could occur which would not be predicted by the simple methods;
- Existing rating curves which do not match flow gaugings at the site. In such cases any extrapolation of the existing rating will compound the error;
- Use of roughness values based on low flows for high flows. Roughness values usually decrease with increasing stage;
- Structures which are believed to be standard structures but do not actually conform with British Standards. Asset surveys of gauging sites have identified a range of problems including incorrect construction and inadequate maintenance (HR Wallingford, 1995 and 1997). These problems can result in the use of incorrect discharge coefficients, etc.;
- Lack of knowledge of drowning or other changes in flow state, for example the onset of overbank flow. The case studies demonstrated that a lack of knowledge of downstream water levels and hence drowning of structures is a very significant source of error; and
- Lack of information on other factors which affect the rating, for example bed stability during high flows.

Risks leading to cost and time increases
The risks that need to be appreciated and minimised in order to avoid cost and time overruns are:

- Insufficient assessment of the hydraulics leading to the application of an incorrect method;
- Need to revise existing rating curves because they do not match flow gaugings at the site;
- Higher than expected costs and/or delays for survey work because of high flows or other unanticipated problems at the site;
- Insufficient calibration data for the site requiring additional data collection or sensitivity tests; and
- Use of inexperienced or incompetent staff.

4.1.12 Where the methods have and have not been found to work

There have been many cases where the simple hydraulic techniques have not worked, and the most common reasons are as follows:

- Extrapolation into zones where the hydraulic conditions are not applicable, including:
  - transition to overbank flow
  - transition from modular flow to drowned flow
  - transition from drowned flow structure control to channel control
  - changes in the cross-section shape at a stage above the limit of the existing rating
- Changes in the cross-section since the rating was developed, or during individual flood events;

- Impacts of vegetation; and
- Other changes in the river which affect the rating, for example new engineering works or dredging downstream.

4.2 Step by Step Procedure

4.2.1 Overview

This section provides a step by step procedure for applying each of the simple hydraulic techniques for extending rating curves, and is intended for use by suitably trained staff to implement the techniques.

The main steps in the application of simple hydraulic techniques to extend a rating curve are shown on the flow chart in Figure 4.1. These tasks are described in more detail in the following sections.

4.2.2 Understand the hydraulics

This is initially done at a high level in order to decide which general approach to rating curve extension is to be used. Having chosen to use simple hydraulic techniques, the purpose of this task is to develop a more detailed understanding of the site hydraulics in order to select the most appropriate approach. The steps needed to understand the hydraulics are described in Section 3.2, including the preparation of a brief planning document.

The planning document should include a cross-section of the river at the gauging site, obtained either from existing information or by survey. The ranges covered by each segment of the existing rating curve should be shown on the cross-section. This cross-section is particularly important for extending rating curves using simple hydraulic techniques, because the range of application is often limited by features of the cross-section. In particular, the cross-section is needed to:

- Assess the range of applicability of a particular method, as it will show the stages at which the cross-section changes shape, for example at bankfull stage; and
- Assess the accuracy of the rating curve extension, as it will show the uniformity of the section outside the limits of the existing rating curve.

4.2.3 Select rating curve extension method(s)

The factors to be considered in method selection include:

- Quality of the representation of the hydraulics;
- Data requirements and, in particular, new survey;
- Usability;
- Cost; and
- Time to carry out the work.

It is important that full consideration be given to the hydraulics of the channel sections. For example, extrapolation methods are unsuitable beyond break points (e.g. bank top) whereas methods based on actual cross-sections can be modified to account for overbank flow. Cross-section data are needed to identify these hydraulic features.

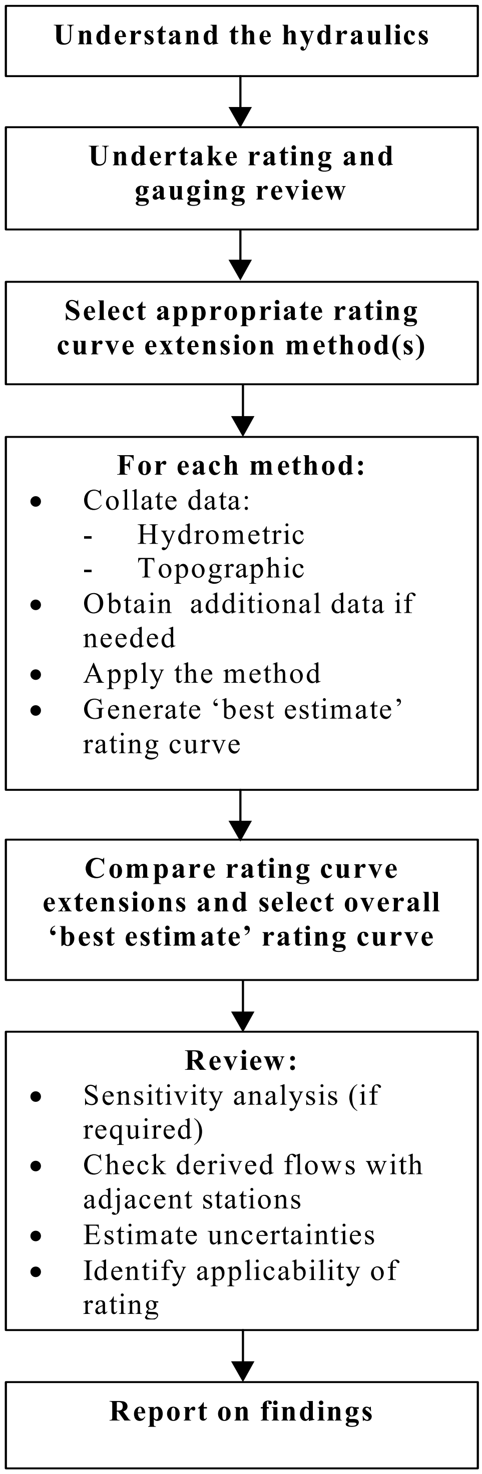

Figure 4.1: Use of simple hydraulic techniques: Step by step procedure

The steps to take when selecting a rating curve extension method are:

- Review the range of stages and flows covered by the existing rating, the range of flows required for the extended rating, and information on channel shape in order to select the most appropriate methods;
- All appropriate methods should be applied and the results compared to select the 'best estimate' rating and estimate uncertainties; and
- Where channel parameters vary along the channel (i.e. A, P or R) then bear in mind the limitations of extensions using simple hydraulic techniques at a single section.

The first question when deciding on the choice of method is to determine whether the gauging site is a section control or a natural control.

For section controls (including weirs and natural control sections) the first and most obvious choice of method will be the use of the theoretical weir equations. These should be based on the relevant British or International Standards for the type of weir to derive the flow equations and C_d relationship. The main issue is to determine the modular limit, so that the applicability of the modular and non-modular flow equations can be determined. This should preferably be based on upstream and downstream water levels or crest tapping measurements.

For flow above the structure limits or wing walls, it is likely that the channel will become the major control. The most suitable simple method for extension in this range is the Divided Channel Method (DCM) as this allows for a combination of the theoretical weir equations for the main channel and the slope area method for the floodplains. However, the method has limitations and a modelling approach will be more suitable in many circumstances.

Table 4.1 provides details of the requirements and issues to be addressed for extension below and above the limits of the existing rating at structure sites.

For non-section control sites, i.e. channel controls, there is more choice available in selecting the most appropriate method. Essentially any method except the weir equations can be used. The most important consideration is to note when abrupt changes in the cross-section occur, such as the bankfull limit or flood berms. The rapid change in the wetted perimeter for only a small increase in cross-sectional area can make the sudden change or even drop in hydraulic radius produce unrealistic extensions. The first thing to do is therefore to obtain and plot a cross-section at the site of interest to determine where changes occur, and then plot cross-sectional parameters against stage to determine if sharp changes in A, P and R occur.

If an extension is required below the bankfull limit and there are no sharp changes in cross-section, the simple extension, log extrapolation or velocity extrapolation methods may be the simplest and most appropriate methods to adopt. If there are changes in the cross-section the Slope-Area method may be preferred but this requires estimates of water surface slope and roughness in the range of water levels where the extension is required.

The second issue is to determine if sharp changes in channel cross-section occur, for example over floodplains. In these circumstances the DCM method will be more appropriate than the Slope-Area method.

Table 4.2 provides details of the requirements, suggested methods and issues to be addressed for rating curve extension at non-structure sites.

The simple hydraulic techniques should be combined under certain circumstances. For example:

- A weir that becomes fully drowned would require the weir equation at lower flows but the Slope-Area method at higher flows where the gauging site becomes a channel control; and
- In the case of a channel section where overspill that outflanks the gauge occurs at high flows, the rating curve for the channel may be extended using the Slope-Area method, but a separate calculation is required to estimate the overspill.

Table 4.1: Requirements, suggested methods and issues for structure sites

Requirements	Suggested Method	Issues to address	How to address issues
Extension within the structure confines.	Derive formula from BS and extrapolate to structure limits.	Where non-modular flow occurs.	Obtain water level measurements for a section immediately downstream of the structure and/or crest tapping levels to determine modular limit, which depends on type of structure.
		Calibration of coefficients against gauged flow data.	Use formulae for modular and drowned flow depending on the modular limit.
		Non-modular flow where downstream levels and/or crest tapping levels are not measured.	Normal depth or backwater calculations and/or hydraulic model to determine downstream rating and hence modular limit. Calibrate against gaugings where available.
		Effect of seasonal vegetation and weed growth on the stage at which non-modular flow occurs.	Calculate water levels for different downstream channel conditions (or use model) with different *n* values, to identify the modular limit in each case.
			Use modular and drowned flow weir formulae to produce a family of rating curves for different conditions
Extension above the existing rating maximum and structure confines.	Use DCM method based on Slope-Area method for floodplain areas, and weir formulae for channel (or Slope-Area method for channel control).	Determine areas of floodplain flow and no flow.	Photographic evidence preferable to identify which parts of the floodplain are no flow areas during floods.

Table 4.2: Requirements, suggested methods and issues for non-structure sites

Requirements	Suggested Method	Issues to address	How to address issues
Extending rating for high flows and low flows within bankfull limits	Any method except weir formulae	Changes in channel cross-section and channel roughness	Use Slope-Area method
Extending high flow rating beyond bankfull	Slope-Area or DCM methods (or hydraulic modelling)	Floodplain flow	DCM preferable to Slope-Area method
		Effect of seasonal vegetation growth and weed on conveyance and hence the rating under normal depth conditions.	Slope-Area method, with estimates of the impact of vegetation on roughness.

4.2.4 Apply method

Simple extension

Collect data

Parameters for the existing rating are essential for the analysis and must be collected from the appropriate Environment Agency Hydrometric Team. In addition, information on the stage values where the cross-section changes (e.g. bankfull) should be obtained. A cross-section of the site is desirable showing the range of stages covered by the existing rating. This is used to identify transitions such as the bankfull limit, above which the method should not be applied. Where the method is applied to structures, information on the stage where drowning and transition to channel control occur is required.

Apply method

The principles for simple extension were given in Section 4.1.3. For a rating equation of the form:

$$Q = C(h + a)^{\beta}$$

where: h = river stage (m)
Q = River flow (m^3/s)
C, a, β = constants

The procedure is as follows:

1. Obtain the rating equation constants C, a, β for the upper (or lower) segment of the existing rating curve, and the upper (or lower) limit of the current rating.

2. Identify the revised upper (or lower) limit that the extended rating will be deemed to apply to. When extending a rating to higher flows, this limit should not be beyond:

   i. The next change in cross-section (e.g. bankfull, top of flood banks).

   ii. The start of by-passing or outflanking flow.

   iii. The onset of drowning, if the current limit is at or below the modular limit.

   iv. The next change in the drowning state, if the current rating includes drowning.

   In many cases, the revised limit may need to be lower than these thresholds, and in some cases it may become apparent that the limit should be reduced from its current value, rather than increased. In all cases, sound judgement will need to be applied in order to identify an appropriate limit, taking account of all available data.

An example of the application of this method is given in Appendix C, Section C.2.

Generate 'best estimate' rating curve
The rating curve is computed by the method described above. The whole of the rating curve should then be plotted, along with any gaugings and appropriate thresholds, such as bankfull, in order to define zones where reasonable accuracy might be expected and also where the method should not be applied.

Logarithmic extrapolation

This method is generally not recommended for the reasons given in Section 4.1.4.

Collect data
Parameters for the existing rating are essential for the analysis and must be collected from the appropriate Environment Agency Hydrometric Team. In addition, information on the stage values where the cross-section changes (e.g. bankfull) should be obtained. A cross-section of the site is desirable showing the range of stages covered by the existing rating. This is used to identify transitions such as the bankfull limit, above which the method should not be applied. Where the method is applied to structures, information on the stages where drowning and transition to channel control occur is required.

Apply method
The principles for logarithmic extrapolation were given in Section 4.1.4. The procedure is as follows.

1. Convert the stage and flows of the existing rating to logarithmic values (or plot stage and flow on log-log paper).

2. Plot the log of stage and flow on an arithmetic scale and extend by eye.

3. Apply this line beyond (or below) the rating maximum (minimum) and convert the extended log values using anti logarithms.

4. Where the limit of the current rating is below bankfull, show the bankfull stage on the plot and ensure the rating is not extended beyond this value. Once over-bank flow occurs, this method will not be valid as the rating slope will change. It is also unwise to apply the method to a region where the channel shape changes, for example where flood berms occur.

Large errors can result from inappropriate application of this approach. It is recommended that:

- If the shape of the control does not change significantly, and the channel roughness remains fairly constant, then a straight-line extrapolation on a log-log plot is reasonable;
- The method is particularly suited to channel control conditions for medium and high flows;
- It should not be used to extrapolate more than about 1.5 times the highest measured discharge; and
- For extensions to very low flows, where a section control exists, it is essential that the shape of the control is accounted for and that the gauge height of zero flow is known. In this situation it is often better to plot the rating on arithmetic axes so that the zero-flow gauge height can be plotted.

Generate 'best estimate' rating curve
The rating curve is computed by the method described above and plotted. The limits such as bankfull are also plotted on the rating curve in order to define zones where reasonable accuracy might be expected and also where the method should not be applied.

The weir equation

Collect data
Data for the weir should be collected from the appropriate Environment Agency Hydrometric Team including weir level(s) and width(s), zero flow gauge level (normally lowest crest level), gauged flow data and/or the value of C_d appropriate to the site, the modular limit and the formula for the drowned flow reduction factor f.

For drowned flow it is also essential to collect data that will permit the calculation of a downstream rating curve at the structure. If no downstream water level data are available, one or more cross-sections will be required. Ideally upstream and downstream levels and associated flow gaugings are needed to assess the drowned flow performance of a structure.

Apply method

Modular flow
The principles of the weir equation were given in Section 4.1.5. The procedure is as follows.

1. Identify the appropriate weir equation for the structure or control being considered from the appropriate British or International Standard. For non-standard structures, the general broad crested weir equation may be used:

   $$Q = C\ bH^{1.5}$$

   where: Q = flow (m^3/s)
   C = a weir coefficient
   b = crest width (m)
   H = the total head, relative to the zero-flow stage (m)

   The weir coefficient (C) is often expressed as:

   $$C = C_d \left(\frac{2}{3}\right)^{1.5} \sqrt{g}$$

   where: C_d = a coefficient of discharge based on weir geometry and head
   g = acceleration due to gravity

Steps 2 – 6 assume that the general broad crested weir equation is being used to extend a rating upwards. If a different weir equation is being used, the procedure is essentially the same, but will need to be modified to account for the specific form of the equation.

2. Identify the appropriate value of b.

3. Plot the values of the weir coefficient (C) or the coefficient of discharge (C_d) against h over the range of the existing rating, using flows derived from the existing rating. These values should be compared with values derived from gaugings and theoretical equations, if any, from the appropriate British or International Standards.

   It is important to note that H is the total head, not the gauged head. The total head is the sum of the gauged head (h) and the velocity head ($V^2/2g$). Thus when calculating C or C_d for the existing rating, it is necessary to calculate the velocity at the cross-section where the water level is gauged and add the velocity head to gauged head to obtain total head (H).

4. Identify the upper limit that the extended rating will be deemed to apply to. This limit should not be beyond:

   i. The next change in cross-section (e.g. structure-full).

ii. The start of by-passing or outflanking flow, unless this is dealt with separately.

iii. The onset of drowning.

In some cases, the revised limit may need to be lower than these thresholds, particularly if the relationship between *C* and *h* is not well defined. In some cases it may become apparent that the limit should be reduced from its current value, rather than increased. In all cases, sound judgement will need to be applied in order to identify an appropriate limit, taking account of all available data.

5. Extrapolate the curve of *C* against *h* upwards to the identified limit, taking account of the comparisons with the values derived from gaugings and theoretical equations.

6. Use the weir equation with the extrapolated values of *C* to calculate pairs of flows and discharges, and fit a standard rating curve equation to these points. As the weir equation is based on total head, it will be necessary to calculate the velocity at the cross-section where the water level is gauged and add this to the gauged head to obtain total head. As the total flow is the factor being calculated, and iterative approach is required.

An example of the application of this method is given in Appendix B, Section B.4.

Drowned flow conditions

This method assumes constant backwater and no current crest tapping or downstream level measurements available (otherwise, use the drowned flow equation directly). It also assumes that a weir equation based rating curve for the modular flow range, as described above, has already been defined.

The calculations rely on the 'Submergence Ratio', which is defined as either:

$$H_2 / H_1$$

or

$$h_p / H_1$$

Where: H_1 = Total head upstream of the weir, relative to the zero-flow stage (m)
H_2 = Total head downstream of the weir, relative to the zero-flow stage (m)
h_p = Head as measured by a crest tapping, relative to the zero-flow stage (m)

These two definitions will not give the same value, so it is essential to be sure which one you are working with.

This relies upon the same weir equation identified for modular flow, but with an extra coefficient to allow for drowning. The general broad crested weir equation becomes:

$$Q = f\,C\;bH^{1.5}$$

where: Q is the flow (m^3/s)

C is the weir coefficient
b is the weir or cross-section width (m)
H is the total head, relative to the zero-flow stage (m)
f is the drowned flow reduction factor

The equations for other weirs are dealt with in a similar way. The drowned flow reduction factor is a function of the submergence ratio, and the specific equations can be found in the British or International Standards. Note, however, that it will not generally be possible to use this technique with drowned flow reduction factor formulae that are based upon crest tapping levels, as there is no way to estimate them independently.

Applying the weir equation under drowned flow conditions is a two-stage process. Firstly the modular limit must be identified, and then the rating must be generated. The procedure is:

1. Identify the modular limit. There are three possible ways of doing this, as described below, with (i) being the preferred method and (iii) the least preferable.

   i. If suitable current meter gaugings are available, plot them on the same stage – discharge graph as the weir equation rating. The expected pattern is for these gaugings to be in agreement with the rating below a certain threshold, but for the gauged flows to progressively reduce compared to the rated flows above this threshold. This threshold is the modular limit.

   ii. If historic downstream or crest tapping levels are available, and these apply to the current control and downstream channel conditions, then plot a graph of the submergence ratio against upstream stage. The submergence ratio at the modular limit for each weir type can be found from the relevant British or International Standards, and the stage at the modular limit read off from the graph.

   iii. If no gaugings or appropriate historic levels are available, then downstream levels will need to be estimated either by normal depth calculations, a backwater profile, or a 1-D hydraulic model. The choice of method should be based upon the hydraulics of the downstream reach. Once this has been done, the modular limit can be identified as in (ii) above.

2. Generate the rating. This is done in the same way as for modular flow, but also requires f, and therefore the submergence ratio, to be estimated for the range of the extension. In order to do this:

   i. Generate a rating curve for the downstream levels, either based upon observed flows and downstream levels, or by the methods outlined in (iii) above.

   ii. For each of a number of flows in the non-modular rating curve extension range, use the downstream rating curve to estimate the downstream total head, H_2.

iii. Substitute each of the pairs of Q and H_2 into the appropriate weir equation, and calculate the corresponding values of H_1. This will need to be done using an iterative technique.

3. Identify the upper limit that the extended rating will be deemed to apply to. This limit should not be beyond:

   i. The next change in cross-section (e.g. structure full).

   ii. The start of by-passing or outflanking flow, unless this is dealt with separately.

   iii. The transition to full channel control, when the fall in water level across the weir becomes very small.

   In some cases, the revised limit may need to be lower than these thresholds. It may also become apparent that the limit should be reduced from its current value, rather than increased. In all cases, sound judgement will need to be applied in order to identify an appropriate limit, taking account of all available data.

4. In order to derive a curve of C against h_1 (upstream gauged head) it will be necessary to calculate the velocity at the location where the water level is gauged and apply the weir equation. Extrapolate the curve of C against h_1 upwards to the identified limit, taking account of the comparisons with the values derived from gaugings and theoretical equations.

5. Use the weir equation with the extrapolated values of C to calculate pairs of flows and discharges, and fit a standard rating curve equation to these points. As the weir equation is based on total head, it will be necessary to calculate the velocity at the cross-section where the water level is gauged and add this to the gauged head to obtain total head. As the total flow is the factor being calculated, an iterative approach is required.

An example of the application of this method is given in Appendix B, Section B.4.

Generate 'best estimate' rating curve

Produce a combined rating extension for both the modular and non-modular flow ranges, using the best estimates of the parameters to generate the 'best estimate' curve. The whole of the rating should then be plotted along with any gaugings and appropriate thresholds, such as bankfull, in order to define zones where reasonable accuracy might be expected and also where the method should not be applied.

Velocity extrapolation methods

Collect data

The parameters and limits of the existing rating should be obtained from the appropriate Environment Agency Hydrometric Team together with the cross-section of the site. Where no cross-section exists, a survey must be carried out. The cross-section should cover the full range of required water levels (and flows).

Apply method

There are three approaches to the velocity extrapolation method which are based on finding a best-fit equation between velocity and various hydraulic channel parameters, and extending this relationship to the high flow region on the assumption that the same relationship applies. The theory behind these three methods is described in Section 4.1.6 and the three methods are:

- The Simple Approach, where velocity is extrapolated against stage;
- Hydraulic Radius Approach, where the velocity is extrapolated against hydraulic radius (i.e. the channel dimensions are assumed to be similar in the extended range); and
- Manning's Approach, where the flow is extrapolated against the geometric properties in the Manning equation.

These are described below.

Simple approach

This method requires calculation of the cross-sectional area and velocity at different river stages using the existing rating. The rating curve is extended based on an extrapolation of velocity plotted against stage based on the assumption that the velocity-stage curve usually exhibits little curvature. The procedure is:

1. Obtain an up-to-date survey of the cross-section at which stage is measured, covering the full range of stage values up to the intended revised rating maximum.

2. Calculate the cross-sectional area (A) at incremental stage values for the current rating to the rating maximum. Also calculate the cross-sectional area at incremental stage values from the current rating maximum to the to the intended revised rating maximum.

3. Calculate the flow (Q) at the same stage values using the existing rating to the current rating maximum.

4. Calculate the velocity (V) from the flow (Q) and cross-sectional area (A) of the channel using the continuity equation, $V = Q/A$. It is useful to tabulate stage, cross-sectional area and flow for the existing rating to the rating maximum.

5. Plot the Velocity-Stage relationship for the current rating flows, and assuming little curvature occurs, plot a best-fit line through the data points.

6. Using this best-fit line calculate the velocity for stage values from the current rating maximum to intended revised rating maximum.

7. On the basis of the derived velocity (V) and surveyed cross-sectional area (A), calculate the flow using the continuity equation ($Q=VA$) for stages above the current rating maximum.

8. Identify the revised upper (or lower) limit that the rating will be deemed to apply to. When extending a rating to higher flows, this limit should not be beyond:

i. The next change in cross-section (e.g. bankfull, top of flood banks).

ii. The start of by-passing or outflanking flow.

iii. The onset of drowning, if the current limit is at or below the modular limit.

iv. The next change in the drowning state, if the current rating includes drowning.

In many cases, the revised limit may need to be lower than these thresholds, and in some cases it may become apparent that the limit should be reduced from its current value, rather than increased. In all cases, sound judgement will need to be applied in order to identify an appropriate limit, taking account of all available data.

9. Fit a standard power law rating equation to the pairs of Q and H values above the current rating maximum.

An example of the application of this method is given in Appendix C, Section C.5.

Hydraulic radius approach
A plot of velocity against the hydraulic radius often shows a linear relationship and hence provides values of velocity in the extended range from the channel cross-sectional parameters. The procedure is:

1. Obtain an up-to-date survey of the cross-section at which stage is measured, covering the full range of stage values up to the intended revised rating maximum.

2. Calculate the cross-sectional area (A) and wetted perimeter (P) at incremental stage values at the cross-section to the current rating maximum, and calculate the hydraulic radius ($R=A/P$). Also calculate the hydraulic radius between this maximum and the intended revised rating maximum.

3. Calculate the known flow (Q) at the same stage values using the current rating to the rating maximum.

4. Calculate the velocity (V) from the rated flow (Q) and cross-sectional area (A) of the channel using the continuity equation, $V = Q/A$. Tabulate stage, cross-sectional area, wetted perimeter, flow and hydraulic radius for the existing rating to the rating maximum.

5. Plot the hydraulic radius – velocity relationship, and assuming little curvature occurs calculate a best-fit line for this relationship.

6. Using this best-fit line calculate the velocity for hydraulic radius values above the rating maximum to the intended revised rating maximum.

7. On the basis of the derived velocity (V) and surveyed cross-sectional area (A), calculate the flow using the continuity equation ($Q=VA$) for stages above the current rating maximum.

8. Identify the revised upper (or lower) limit that the rating will be deemed to apply to. When extending a rating to higher flows, this limit should not be beyond:

   i. The next change in cross-section (e.g. bankfull, top of flood banks).

   ii. The start of by-passing or outflanking flow.

   iii. The onset of drowning, if the current limit is at or below the modular limit.

   iv. The next change in the drowning state, if the current rating includes drowning.

   In many cases, the revised limit may need to be lower than these thresholds, and in some cases it may become apparent that the limit should be reduced from its current value, rather than increased. In all cases, sound judgement will need to be applied in order to identify an appropriate limit, taking account of all available data.

9. Fit a standard power law rating equation to the pairs of Q and H values above the current rating maximum.

An example of the application of this method is given in Appendix C, Section C.5.

Manning's approach
A third approach to the velocity extrapolation method is to use Manning's equation:

$$Q = AV = \frac{AR^{2/3}s^{1/2}}{n}$$

Where: Q = flow (m^3/s)
A = cross-sectional areas (m^2)
V = mean velocity (m/s)
R = hydraulic radius (m)
s = water surface slope
n = Manning's resistance coefficient

Assuming $s^{1/2}$ and Manning's n remain constant, plotting values of Q against $AR^{2/3}$ will give a straight line that can be extrapolated upwards. Since A and R are both functions of stage, this line can then be used to generate pairs of Q and stage values to which a power law rating curve can be fitted. The procedure is as follows:

1. Obtain an up-to-date survey of the cross-section at which stage is measured, covering the full range of stage values up to the intended revised rating maximum.

2. Calculate the cross-sectional area (A) and wetted perimeter (P) at incremental stage values at the cross-section to the current rating maximum, and calculate the

hydraulic radius ($R=A/P$). Also calculate the hydraulic radius between this maximum and the intended revised rating maximum.

3. Calculate the flow (Q) at the same stage values to the current rating maximum using the current rating. Tabulate stage, cross-sectional area, wetted perimeter, hydraulic radius, $AR^{(2/3)}$ and flow for the existing rating to the rating maximum.

4. Plot the flow against $AR^{(2/3)}$ and assuming little curvature occurs fit a best-fit line to this relationship.

5. Using this best-fit line calculate the flow for $AR^{(2/3)}$ values above the rating maximum to the cross-section maximum.

6. Identify the revised upper (or lower) limit that the rating will be deemed to apply to. When extending a rating to higher flows, this limit should not be beyond:

   i. The next change in cross-section (e.g. bankfull, top of flood banks).

   ii. The start of by-passing or outflanking flow.

   iii. The onset of drowning, if the current limit is at or below the modular limit.

   iv. The next change in the drowning state, if the current rating includes drowning.

   In many cases, the revised limit may need to be lower than these thresholds, and in some cases it may become apparent that the limit should be reduced from its current value, rather than increased. In all cases, sound judgement will need to be applied in order to identify an appropriate limit, taking account of all available data.

7. Fit a standard power law rating equation to the pairs of Q and H values above the current rating maximum.

An example of the application of this method is given in Appendix C, Section C.5.

Generate 'best estimate' rating curve
If more than one rating curve extension has been generated by the above methods, then one should be selected as the best estimate. The whole of this extended rating should then be plotted along with any gaugings and appropriate thresholds, such as bankfull, in order to define zones where reasonable accuracy might be expected and also where the method should not be applied.

Slope-Area methods

Collect data
The existing rating and flow gaugings at the site should be obtained from the appropriate Environment Agency Hydrometric Team together with a cross-section of the site. The cross-section should include the floodplains where the divided channel method is to be applied. The cross-section should be obtained by survey where there is

no existing cross-section. It should be noted that BS/ISO 1070 recommends that at least three sections should be used. In this methodology it is assumed that only one section is used but the section should ideally be representative of the river reach to avoid errors arising from variations in the river cross-section.

The method requires an estimate of the water surface slope. Accurate measurement of water surface slope and the variation with stage is difficult, and a simplified approach is normally adopted in which a single value is used for all stages. Measurements of the water surface slope may be subject to errors but it can be taken from two or more water level recorders, gauges boards or trash marks at two different locations, upstream and downstream of the gauge site.

In the absence of any recorded information, and since rating curves ideally apply to steady flow, the water surface slope may be assumed to be the same as the friction slope S_f in uniform river reaches which are unaffected by backwater and drawdown effects. This is approximately equal to the bed or bank slope if the flow is uniform. These should be shown on a longitudinal section of the river, if available. Alternatively the bank slope may be estimated from contour maps of the river valley although this approach is very approximate.

For non-uniform flow, the water surface slope will need to be estimated. Some possible ways of doing this are:

- Estimating n and calculating the water surface slope – stage curve from the range of the existing rating and extrapolating;
- Using high water marks (wrack marks) at the peak flow although this method does not give accurate water levels. Longitudinal changes in velocity head may also need to be considered; and
- Peak level gauges, where installed, should be used in preference to wrack marks as they give more accurate water level measurement.

Apply method

Simple Slope-Area method

The principle of the approach is that the cross-sectional area, hydraulic radius and water surface slope of the measuring reach is determined and the mean velocity calculated using Manning's equation:

$$Q = AV = \frac{AR^{2/3}s^{1/2}}{n}$$

1. For the cross-section calculate the cross-sectional area (A), wetted perimeter (P) and hence hydraulic radius (R) at incremental levels of stage up to the limit of the existing rating and to the limit of the intended revised rating.

2. Calculate the water surface slope (s), either directly from water level recordings or indirectly using the river bed or bank slope. A single value of s is normally used for all stages, as discussed under *Collect data* above.

3. Estimate Manning's n using the existing rating to the rating maximum.

4. For extension of the rating curve to higher stages, either the value of *n* at the rating maximum should be used or the plot of *n* against stage can be used to predict values of *n* at higher stages. Ideally *n* should be estimated from actual flow gaugings.

5. The corresponding flows are then calculated by applying the above equation to stages above the rating maximum. *A* and *R* are obtained from step 1 above, *s* from step 2 and *n* from steps 3 and 4. A rating equation is fitted to the calculated set of flow and stage values.

Compound channels - the divided channel method
The Slope-Area method is not suitable where there are abrupt changes in the channel cross-section with stage, for example at bankfull or the top of structure walls. This is because the method cannot account for abrupt changes in channel geometry or lateral changes in channel roughness where *n* on the floodplains will differ from in-channel values.

The Divided Channel Method (DCM) is an adaptation of the Slope-Area method for compound channels, particularly at sites where over-bank gaugings exist. The method provides a simple approach to compound channel flow estimation. In particular, it assumes that the river channel and floodplain cross-section is constant upstream and downstream. The DCM is applied as follows:

1. The cross-section of the river channel (or flow measurement structure) and floodplain is divided into compartments (left floodplain, main channel and right floodplain) using vertical division lines. The division lines are normally located at the tops of river banks or structure walls.

2. Calculate a rating using the Slope-Area method for the channel section using the method described under *Simple Slope-Area Method* above.

3. Calculate the cross-sectional area (*A*), wetted perimeter (*P*) and hence hydraulic radius (*R*) for the channel at incremental levels of stage above bankfull assuming there is no friction on the vertical boundaries above bankfull.

4. Using bankfull values of *n* and *s*, extend the in-channel rating above bankfull.

5. Calculate separate ratings using the Slope-Area method for the left and right bank floodplain components. *A*, *P* and *R* should be calculated for the same increments of level as used for the channel in step 3 above. It should be assumed that there is no friction on the vertical boundaries with the channel above bankfull. *s* should be the same as for the channel as specified in step 4 above. Values of *n* should be estimated using floodplain values (from Appendix A).

6. Add the flows for the channel and floodplains at each value of stage to obtain the total flow and therefore rating for the section.

An example of the application of this method is given in Appendix C, Section C.6.

Whilst the use of estimated values of *n* for the floodplains is an approximation, the proportion of total flow on the floodplains is often relatively small and therefore the error associated with this approximation is also relatively small. Care should be taken to identify the extent of the floodplain where active flow is likely to occur, as only this area is needed for the calculation. Flood storage and other no-flow areas should therefore be excluded from the calculation. The limits of the active flow areas should be obtained from observations and photographs of real events, if available.

Generate 'best estimate' rating curve

The rating curve is computed by the methods described above and plotted. Limits such as bankfull are also plotted on the rating curve in order to define zones where reasonable accuracy might be expected for the different methods, and where the methods should not be applied. Generally the simple Slope-Area method is applicable up to bankfull, and the DCM is applicable for compound channels.

4.2.5 Carry out sensitivity analysis, if required

The purpose of sensitivity analysis is to identify the likely range of uncertainty associated with a rating extrapolation. Possible sources of uncertainty include:

- Inaccuracies in parameter values;
- Incorrect assumptions;
- Vertical changes in the cross-section at the gauging site;
- Longitudinal changes along the river, for example in section shape; and
- Variable downstream influences, for example vegetation growth.

Errors will also arise if the rating is used beyond recommended limits, for example bankfull. This can be avoided by showing the limits on the stage discharge plots.

The most likely causes of uncertainty should be identified from the hydraulic review and the rating and gauging review in order to provide a basis for sensitivity tests. Possible sensitivity tests include the following:

- Varying the values of the constants C, a and β in the simple extension method. This should be based on the correlation between flow gaugings and the existing rating curve;
- Varying the coefficient of discharge for the weir equation under modular flow conditions. This should be based on the correlation between flow gaugings and the rating curve if these data are available;
- Varying the downstream water levels and therefore the stage at which drowning occurs in the weir equation for drowned flow conditions. This is often a source of considerable uncertainty because of lack of knowledge of the stage at which drowning begins to occur, and the drowned flow performance of structures;
- Varying the values of velocity in the simple and hydraulic radius approaches to velocity extrapolation; and
- Varying the values of water surface slope and Manning's n in the Slope-Area method.

Sensitivity tests will result in the generation of additional rating curve extensions for comparison with the 'best estimate'. This in turn will contribute to the estimate of uncertainty in the rating, discussed in Section 4.2.7 below.

A brief statement should be provided giving the results of sensitivity tests.

4.2.6 Check derived flows with adjacent stations

It is also advisable to check that the flows used for the extended rating curve at the gauging station are consistent with flows elsewhere in the catchment. The purpose of this check is to ensure that:

- The extended rating curve covers the required range of flows; and
- Flows predicted by the extended rating curve for the highest water levels recorded at the gauging site are consistent with flows elsewhere in the catchment.

If the derived flows using the extended rating curve are inconsistent with established 'correct' flows, the 'best estimate' rating curve may require amendment. In this case the results of the sensitivity tests referred to in Section 4.2.5 should be used to ensure that any adjustments are hydraulically reasonable.

A brief statement should be provided stating how consistent the derived flows are with flows at upstream and downstream gauging stations, and whether any adjustment was required to the extended rating curve.

4.2.7 Estimate uncertainties in the rating

The sensitivity tests results, consistency checks and any other relevant information should be used to estimate the uncertainty in the rating, and the impact this would have upon derived flows, particularly the flows in the annual maximum series.

A brief statement should be provided of uncertainty in the rating, and the uncertainty in the annual maximum series flows.

4.2.8 Identify applicability of the revised rating

The applicability of the rating should be stated in terms of:

- Maximum and minimum flows and stages; and
- The time period that the rating applies to.

4.2.9 Report on findings

A report should be produced on the work undertaken for the rating curve extension, based upon the statements generated in the previous tasks. The report should be addressed to two audiences: people who will have to implement the revised rating and people who will have to approve the work at its completion, or audit the work in the future.

It must include clear statements of:

- The revised rating, in a form suitable for incorporation in the flow processing system; and
- The uncertainties in flows derived from the extended rating.

In addition, the report should cover:

- The hydraulics of the site;
- The site plan and cross-section(s) at the site; and
- An audit trail.

4.3 Case Studies

4.3.1 General approach to the analysis

The purpose of the case studies is to demonstrate the application of the different methods, identify some of the problems that might be encountered, and provide estimates of the likely uncertainties in the results.

The simple methods have been applied to three case study sites. The analysis and results are provided in detail in Appendices B to D respectively. This section summarises the results of the case studies and considers the most suitable method to apply in each case. Reference is made to the appendices for detailed application of the recommended method(s).

The case study sites are listed in Section 4.3.2 below. The general approach for each case study has been to:

- Apply different extension methods to the existing rating curves;
- Use gauged flows to assess the accuracy of each method for the extended rating curves where possible;
- Where gauged flows do not exist, the method used for extending the rating curve should be as hydraulically correct as possible; and
- Provide recommendations for which method to adopt in different circumstances based on hydraulic considerations. Other considerations including data availability and cost should also be taken into account in rating curve extension work.

In deciding which method to recommend, a balance is needed between the importance of reducing uncertainty in flood flow estimates and cost. For example, it may be better to invest in survey work or additional hydrometric data collection and accept the associated cost in order to get a better rating.

4.3.2 Description of the case studies

Details of the three case studies are given in Table 4.3.

Table 4.3: Case Studies

Case Study No	Type of Control	Appendix
1	Non Standard Weir	B
2	Open Channel with floodplains	C
3	Compound Sluice	D

The case studies do not include a rigorous hydraulic assessment or rating and gauging review, but concentrate on the application of the simple hydraulic techniques for illustrative purposes.

4.3.3 Case Study 1: Non-standard weir

Understanding the hydraulics

The gauging station consists of a compound structure of broad crested weirs with no divide piers. The length of the weirs parallel to the direction of flow is assumed to be 1.0m. The low crest is 12.31m wide with a crest elevation of 14.76m AOD; the high crest is 11.62m wide with a crest elevation of 15.22m AOD. The weir is located in an incised river channel that has a width of 24m at structure full level

The channel opens out from a width of 24m and a level of 16.2m AOD, at the top of the weir wing walls, to a width of 37m at 18.3m AOD. A horizontal berm at 18.3m AOD opens the channel further to a width of 41m before the channel slopes upwards to give a top width of 50m at an elevation of 20.07m AOD giving a maximum head above the weir crest of 5.31m before overbank flow occurs. The gauge zero is 14.87m AOD. A cross-section of the weir is shown on Figure 4.2.

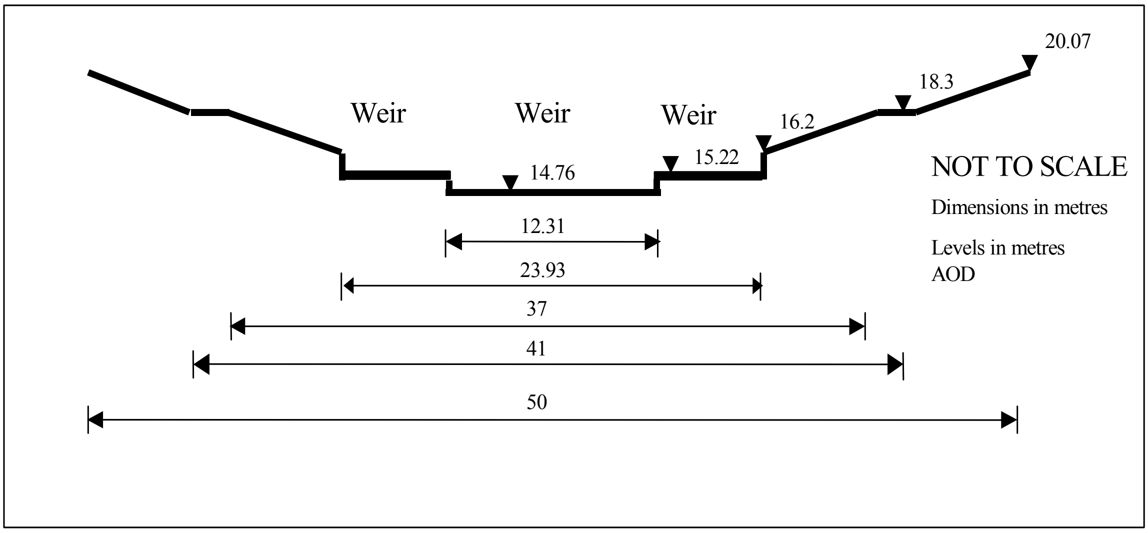

Figure 4.2: Case Study 1: Weir cross-section

The approach channel to the weir is straight with the channel showing a sinuous plan form downstream of the weir. The bed slope of the river channel upstream of the weir is approximately 1 in 700. Downstream of the weir, for a distance of approximately 400m, the profile of the river bed is uneven with an average bed slope close to horizontal. Beyond 400m downstream of the weir the bed slope increases significantly to approximately 1 in 200.

The weir is modular to a stage and discharge of approximately 1.6m above gauge zero and 100 m^3/s respectively. Above a water surface level of 1.6m the weir starts to drown and becomes fully drowned at a water surface level of approximately 3.1m.

The flow remains almost entirely in-bank up to the 100-year flow which is in excess of 400 cumecs. A rating extension is required for flows up to 500 cumecs.

The biggest difficulty associated with the hydraulics at the site is the fact that the flow gaugings show very little increase in flow between stages of 1.6 and 3.1m. Reasons for this could be:

- Drowning of the structure;
- Interference between slow moving flow on the sloping bank and faster moving flow in the main channel. The discontinuity in the gauged data occurs when the flow goes out-of-bank, suggesting that it could be caused by this interaction. However the differences are greater than would be expected from this source; and
- The flow gaugings only use a single velocity measurement in the vertical. The river is over 5 metres deep and the single measurement may not fully identify the average velocity in the vertical.

The analysis suggests that drowning of the weirs is the primary cause of the discontinuity in the flow gaugings.

Rating and gauging review

Three ratings have been used with the rating maximum increasing with each rating. The current rating (GM109303) extends to a flow of about 600 cumecs. The two historic ratings and the current rating are shown with relevant gaugings on Figure 4.3.

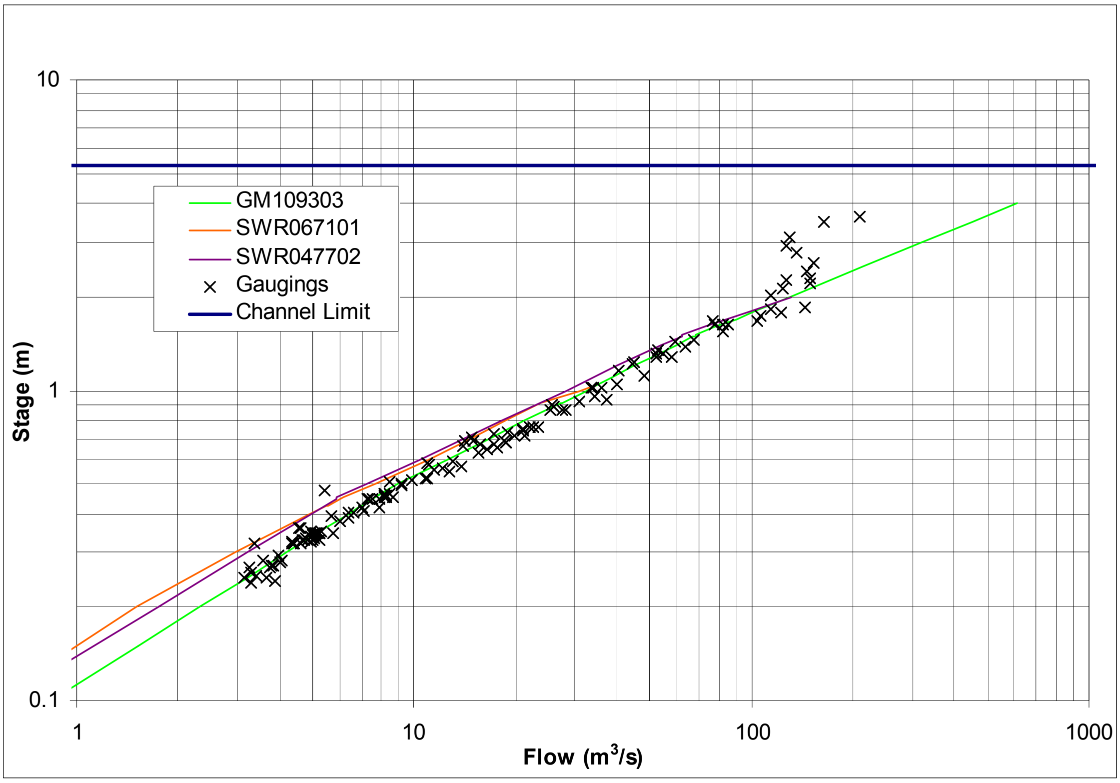

Figure 4.3: Case Study 1: Environment Agency Ratings and gaugings

Whilst a thorough rating and gauging review has not been carried out for the site, it is apparent from Figure 4.3 that there is a departure of the gaugings from the existing rating at about 150 cumecs. The rating should therefore be extrapolated from this point (i.e. for the range 150 to 500 cumecs). Thus the rating extension strategy is as follows:

- There is reasonable correlation between the existing rating and gauged flows up to a level of about 1.9m above gauge zero, and a flow of about 150 cumecs;
- The existing rating does not match the gaugings above 150 cumecs. The reason for the deviation is discussed above; and
- Any rating extension should begin from the limit of acceptability of the existing rating (i.e. 1.9m above gauge zero, and a flow of about 150 cumecs).

Select method(s)
The most appropriate simple method for rating extrapolation is the use of the modular flow and drowned weir equations. This is because:

- The gauging site consists of a compound weir which operates under modular and drowned flow conditions; and
- The required flow range is contained within bank, although there is a discontinuity in the cross-section at a level of about 1.4m (flow about 60 cumecs).

As discussed below, a point will be reached where the control changes from a drowned weir control to an open channel control. Thus an open channel method is needed to complement the weir equation. It is recommended that the Slope-Area method is used as this provides maximum flexibility in varying parameters. In particular, values of Manning's *n* can be varied in order to calibrate the method against gauged flows.

Other appropriate simple methods have also been applied for illustrative purposes, as shown in Appendix B.

Collect data
The following data were collected for the site:

- The existing rating curves;
- Weir levels and widths;
- Site cross-section;
- The gauge zero level;
- Gauged flow data;
- Drowning characteristics of the site; and
- A minimum of one cross-section is required downstream of the structure. This is needed to determine a rating curve downstream of the structure for calculating the modular limit and drowned flow part of the structure rating curve. In this case, a steady state hydraulic model was used to develop the downstream rating which used 10 sections covering about 1,100m of river, as no downstream water level measurements were available.

Apply method(s)
Application of the weir equation is described in detail in Section B.4 of Appendix B. The results are shown on Figure 4.4.

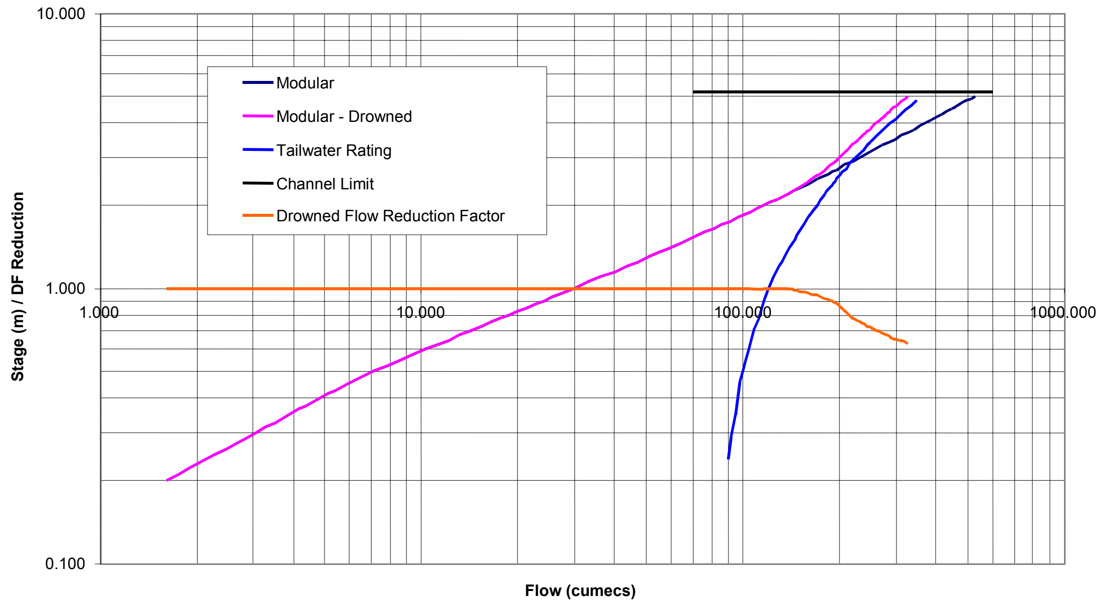

Figure 4.4: Case Study 1: Rating curve predictions from weir equations

Generate 'best estimate' rating curve

The gaugings show that the shape of the rating curve changes at a stage of about 2m. This is understood to be caused by drowning of the structure. As the flow increases, there is a transition from drowned structure flow to control by the river channel. The suitability of the simple methods for extension of the rating curve at this site is discussed below.

- The simple extension of the current rating is not suitable because the current rating assumes the flow at the weir is modular and it does not take account of drowning at the structure. If the current rating were based on the high flow gaugings, this approach might be appropriate to bankfull stage;
- The modular and drowned weir equations identify the transition to drowned flow and the shape of the drowned flow curve. It is however essential to have a downstream rating curve to accurately predict the drowned flow rating;
- The velocity extrapolation methods are not suitable although they could be applied at the section where the water level is recorded, upstream of the weir;
- The Slope-Area methods are not applicable at this site; and
- The hydraulic model did not identify the drowned flow performance of the weir, although the application of 1-D and 3-D hydraulic models to this site is covered in Case Studies 5 and 7 respectively.

From the above discussion, the 'best estimate' rating curve is as follows:

- The current rating can be applied for the modular flow range which extends to a stage of about 2m and a flow of about 150 cumecs;
- The drowned weir formula should be used above a stage of 2m, but requires a downstream rating curve that is calibrated using the drowned flow gaugings;
- The derived rating curve will be applicable to the bankfull stage of 5.31m;

- The above discussion assumes that the gaugings were not available. If the gaugings were available, any rating extension should be taken from the highest gauging of 210 cumecs at a stage of 3.62m; and
- Overbank flow will occur above a stage of 5.31m and cannot be predicted by the simple methods. However, this is likely to be above the highest required flow of 500 cumecs.

Uncertainties in the rating
The 'best estimate' rating identified above assumes that the flow gaugings are not available. The results indicate the wide range of error that can occur without knowledge of the hydraulics of the site. Thus it is essential to have an appreciation of the drowning characteristics of the site before a meaningful rating extension can be derived.

In the case where the flow gaugings were available, the rating extension would apply from the maximum gauging. If it is assumed that the flow gaugings are accurate to +/- 10%, then this uncertainty would apply to the maximum gauged flow of 210 cumecs at a stage of 3.62m. An extrapolation of this rating curve to a stage of 5.31m (bankfull) would have an uncertainty at this stage of the order of +/- 20%.

Applicability of rating
The applicability of the 'best estimate' rating identified above would cover the full range of in-channel flow to the bankfull stage of 5.31m. However, it is essential for information on the drowned flow performance of the structure in order to apply the simple methods at this site.

4.3.4 Case Study 2: Open channel with floodplain flow

Understand the hydraulics
The gauging station is a two-stage channel where both the main river channel and floodplain flows are gauged. The channel is 40m wide at the gauging section with a minimum bed elevation of 50.1m AOD. The gauge zero is at 52.0m AOD. The left and right banks are at 56.64m AOD and 56.56m AOD, indicating a bankfull stage of 4.56m. The floodplain rises to 61.5m AOD (a stage of 9.5m). The cross-section of the site is shown on Figure 4.5, which also shows the division lines used in the application of the divided channel method (Appendix C, Section C.6).

The existing rating extends to a flow of about 450 cumecs, whereas the extended rating would be required for flows of the order of 800 cumecs to cover the full range of flood flows.

Rating and gauging review
Eleven ratings have been used since 1972. The current rating is shown on Figure 4.6 together with flow gaugings at the site. There is a change in slope of the rating at a stage of 4.6m, corresponding to bank top level. The rating clearly appears to fit the gaugings well, including flow gaugings taken since the rating was introduced in 1993. Thus the rating is considered to be satisfactory for extrapolation.

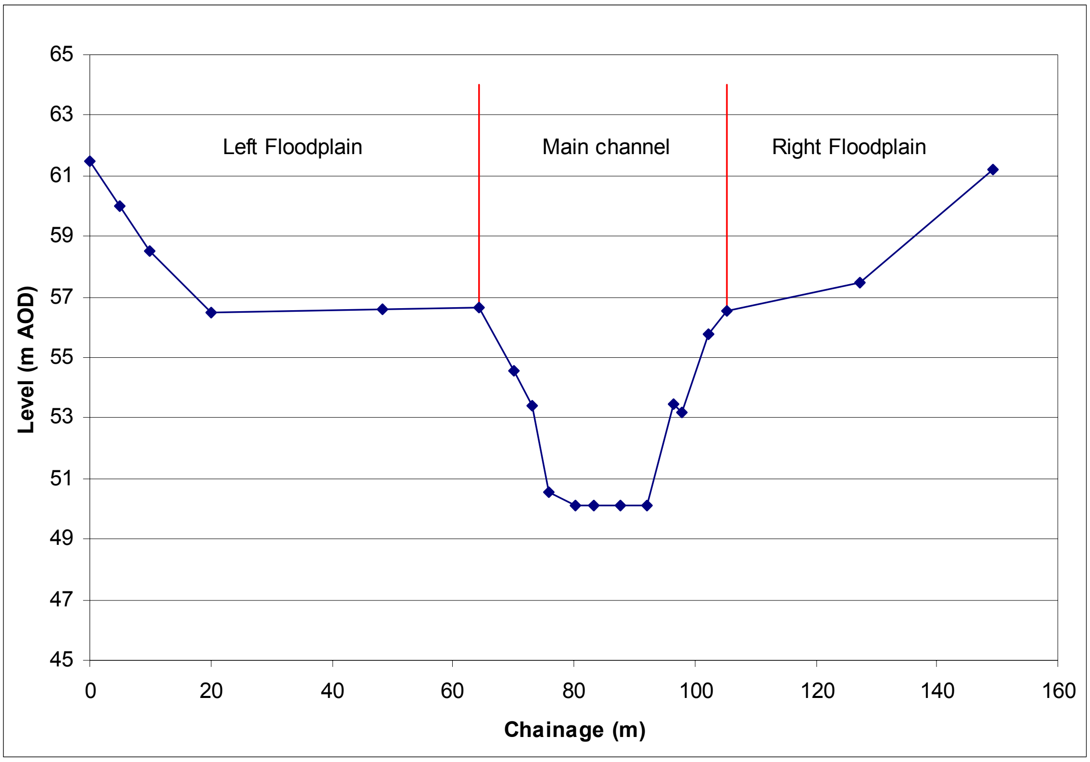

Figure 4.5: Case Study 2: Cross-section

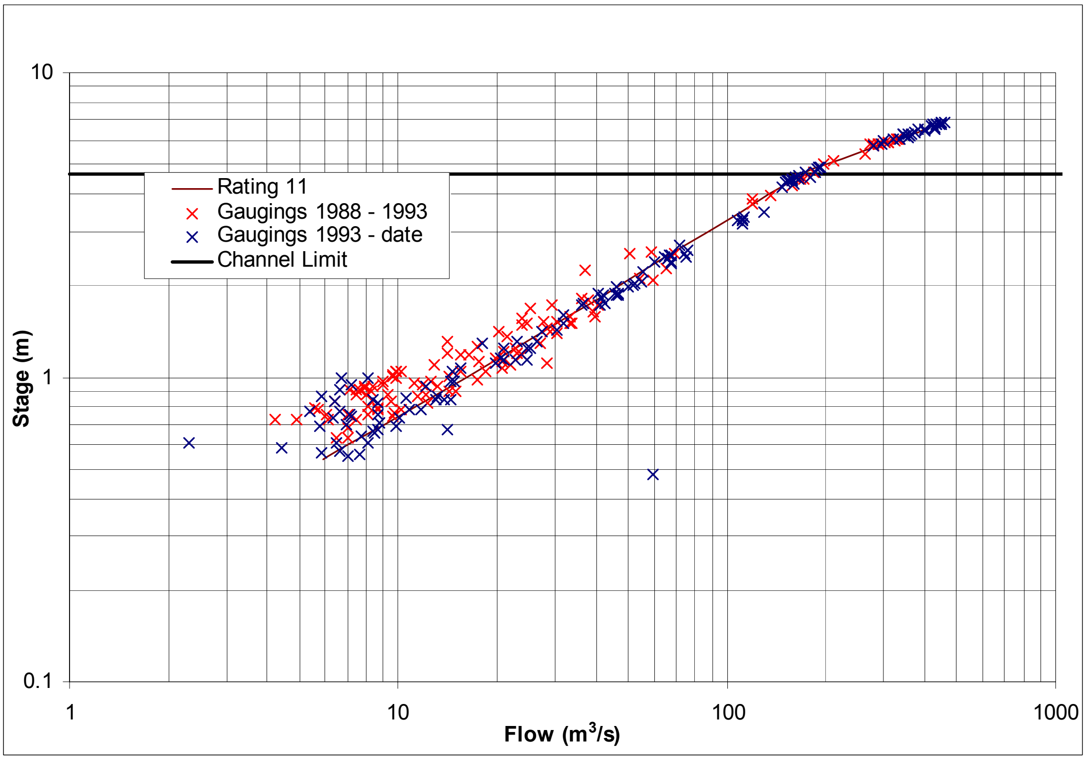

Figure 4.6: Case Study 2: Environment Agency rating and gaugings

Select method(s)
As the site includes overbank flow, the only simple method that would be theoretically applicable if the rating extrapolation were required from below bank top level would be the Divided Channel Method. However, as the existing rating includes a reliable overbank limb, other simple methods are theoretically applicable apart from of course the weir equation.

The velocity methods are not however considered to be suitable because of the large variations in velocity which occur between the channel and the floodplain and an average velocity would not vary in a linear way. Thus the simple extension and Slope-Area methods have been applied in addition to the Divided Channel Method. In addition, the velocity extrapolation methods have been applied at this site for illustrative purposes, although they are not generally suitable because of the large variation in velocity across the river and floodplain cross-section.

Collate data
The following data were collected in order to apply the simple methods to the site:

- The existing rating curves;
- Site cross-section including the floodplains;
- The gauge zero level; and
- Gauged flow data.

Apply method(s)
Application of the simple extension is covered in Section C.2 of Appendix C, and the results are shown on Figure 4.7.

The Divided Channel Method is covered in detail in Section C.6 of Appendix C, and the results are shown on Figure 4.8.

A summary of results for all the simple methods applied at then site is shown on Figure 4.9.

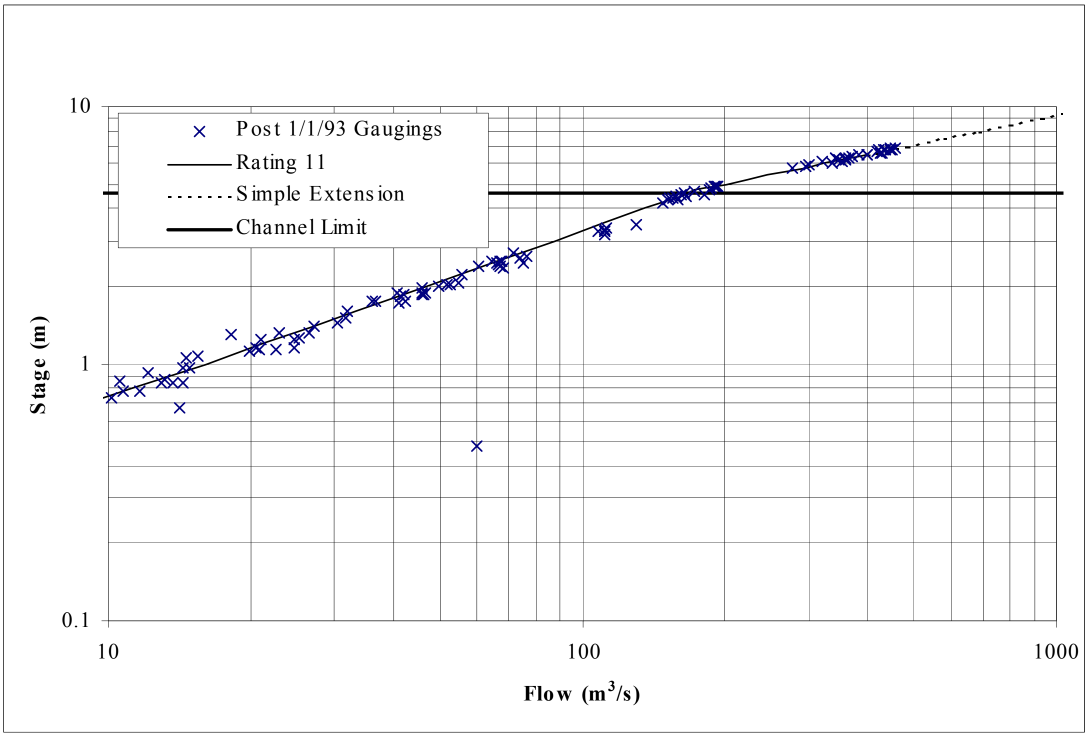

Figure 4.7: Case Study 2: Simple extension of Environment Agency rating

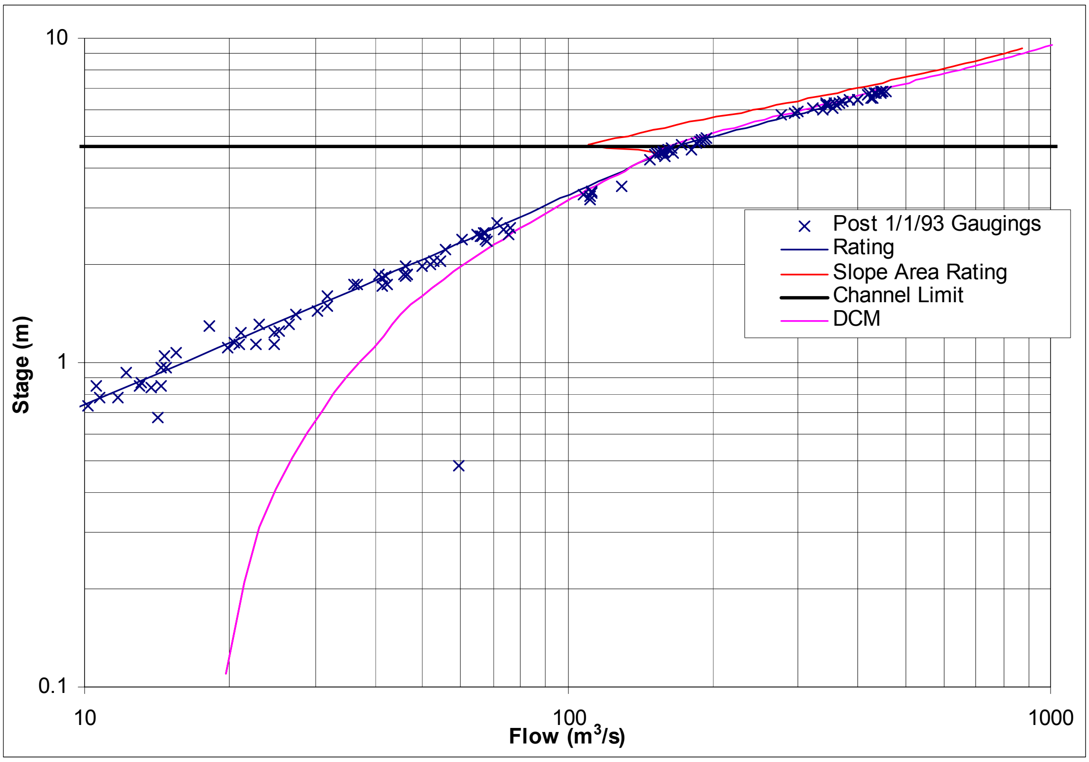

Figure 4.8: Case Study 2: Slope-Area and DCM ratings

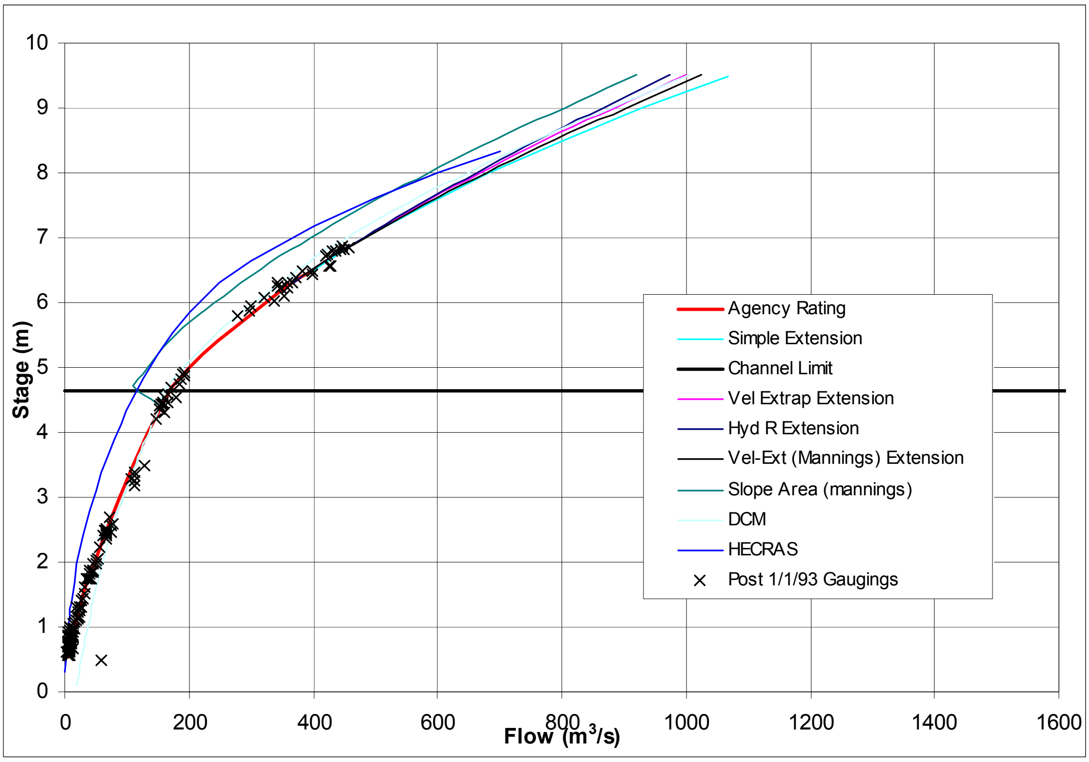

Figure 4.9: Case Study 2: Comparison of extension methods

Generate 'best estimate' rating curve

The Divided Channel Method and the simple extension both give similar results. It is however recommended that as a general rule the Divided Channel Method should be applied for compound channels. Thus the rating extrapolation derived using this method is considered to be the 'best estimate'.

Uncertainties in the rating

The existing rating shows very good agreement with gauged flows up to about 450 cumecs (stage about 6.7m). As the site does not have any breaks in channel and floodplain section between this stage and the maximum limit of the rating extrapolation, the uncertainty in the rating extrapolation will be relatively small. The maximum limit of the rating extrapolation is less than twice the flow at the upper limit of the existing rating. If it is assumed that the uncertainty in gauged flows (and hence the existing rating) is +/- 10%, it is suggested that the uncertainty will rise to +/-20% at a flow of 800 cumecs. Reasons for an increase in uncertainty with stage include possible changes in roughness values at higher stages, as values of Manning's *n* normally reduce with stage.

Applicability of rating

The rating is applicable for the full range of required flows up to about 800 cumecs and a stage of about 8.4m.

4.3.5 Case Study 3: Compound sluice

Understanding the hydraulics

This case study is an example of developing a rating curve at a complex non-standard structure by hand and spreadsheet calculation.

The gauging station is a compound flow measurement structure which includes four separate components: an adjustable broad crested weir, two vertical lifting gates, a navigation lock with sluices in the upstream gates. The dimensions and hydraulic parameters of these structures are described below:

- A variable level 6.05m wide weir which, at its lowest position is flush with the weir block (3.435m AOD) and can be raised to a maximum of 4.004m AOD. This weir is contained with vertical wing walls of 5.640m AOD;
- The two bays with vertical lifting gates each 6.10m wide and 1.94m high. The gates act as thin plate weirs when the structure is overtopped or undershot. The invert of the gates is at 2.388m AOD and when closed the top of the gates are at 4.328m AOD. These are contained with vertical wing walls of 5.640m AOD;
- A lock controlled by a vertical lifting gate at the downstream end and a pair of mitre gates at the upstream end. When the lock is not in use the downstream gates are left open hence the control is the upstream mitre gates and these can act as a broad crested weir once water levels exceed the top of the gates at 4.3m AOD; and
- The two upstream lock gates each contain a paddle sluice which, although not designed for river regulation purposes, can be used as a method of trimming upstream water levels. Each sluice is 0.66m high by 0.81m wide and controlled by vertical lifting gates mounted on the upstream face of the lock gates. The sill of each sluice is flush with the surface of the step downstream at 2.435m AOD.

The lock is separated from the remainder of the structure by a 2.44m wide pier whilst the remaining 3 bays (the adjustable weir and the two gate bays) are separated by 1.83m wide vertical piers at 5.64m AOD. The weir and gates discharge into a common stilling basin at 0.15m AOD and which is fitted with a faced bar or lip at 1.67m AOD.

A cross-section of the structure is shown on Figure 4.10. The structure can be bypassed at high flows, and the overall structure and floodplain cross-section is shown on Figure 4.11. The maximum flood flows at the structure can be of the order of 250 cumecs. However the top of the wing walls of the structure and dividing walls at 5.640m AOD forms the rating maximum that can be achieved using simple methods. The total flow at this stage is of the order of 100 cumecs.

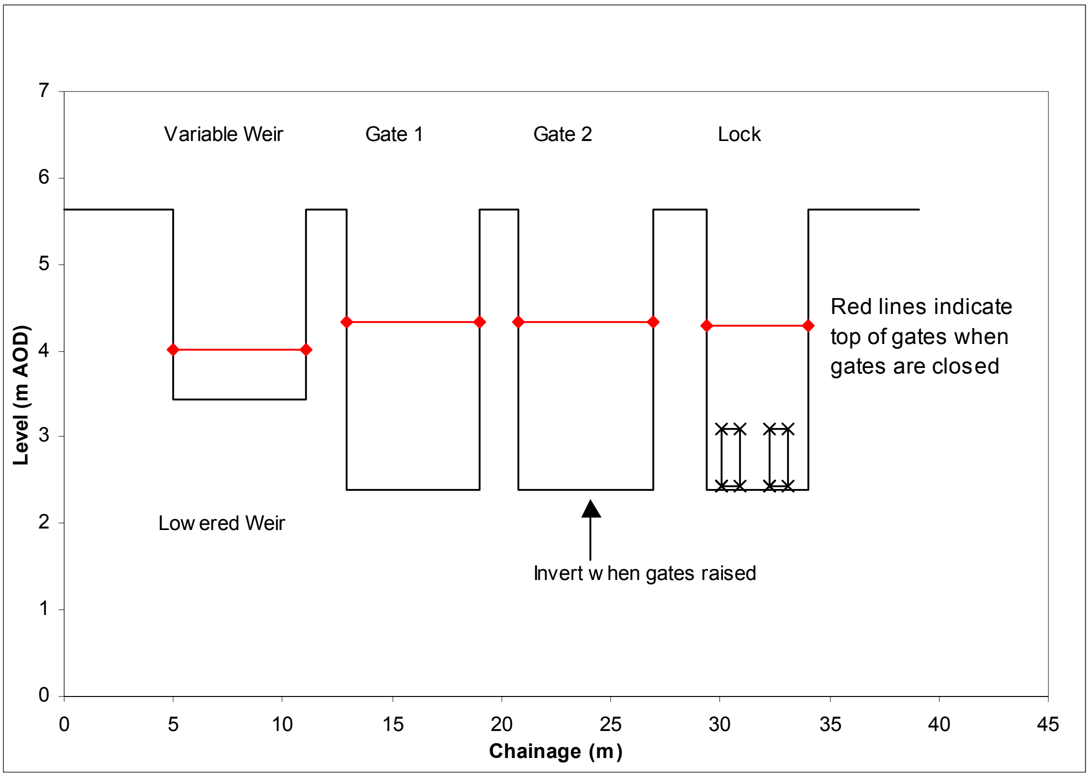

Figure 4.10: Case Study 3: Cross-section

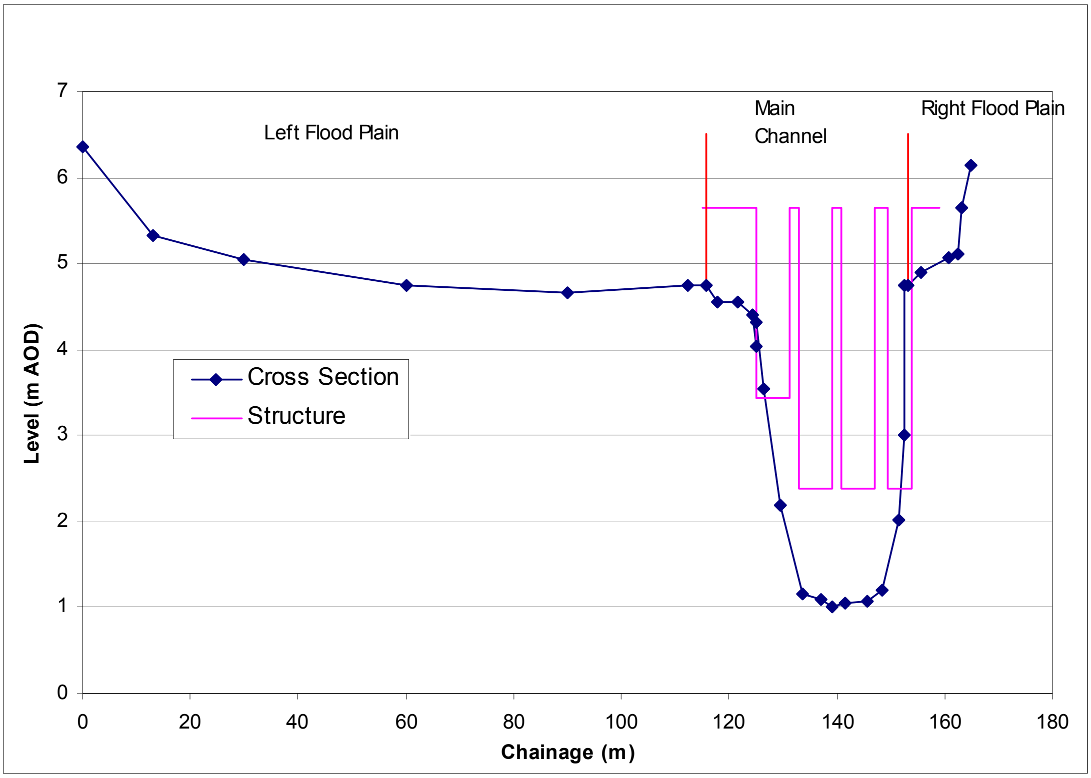

Figure 4.11: Case Study 3: Adopted cross-section upstream of site

Water levels are measured 27m upstream and 13m downstream of the sluice gates. Tail water levels are influenced by tidal movements and the structure can be operated in both drowned and modular flow conditions.

A particular complexity of this site is the large number of combinations of gate openings that could occur and the impact of variable downstream water levels. In order to derive a meaningful high flow rating the following assumptions were made:

- All gates will be open during high flood flows; and
- Downstream levels are determined by the flood flow (i.e. tidal effects are negligible under high flow conditions).

Rating and gauging review
There is no available rating in Agency format as the stage-flow relationship at this site is a complex combination of flow over or under each of the structures identified above. Flows are calculated using a Fortran programme based on upstream and downstream head and the settings of the various structures. Stage discharge relationships have been provided for one or more of the following elements of the structure from physical modelling or detailed analysis:

- the variable crest weir at different crest elevations;
- under the 2 vertical lifting gates when open;
- over the 2 vertical lifting gates when closed;
- over the lock mitre gates; and
- through the lock sluices.

The relationships are described more fully in Appendix D, Section D.4. There are no available gaugings to validate any derived rating.

Select method(s)
In view of the lack of an existing rating, the only appropriate method is to develop a rating for the combined structure components and combine this with the Divided Channel Method to account for bypassing of the site.

Collate data
The following data were collected in order to apply flow equations for structures together with the Divided Channel Method for this complex structure site:

- Equations for modular and drowned flow for each of the structure components, obtained from a number of sources;
- Site cross-section including floodplains;
- Dimensions and levels for all of the structure components;
- The gauge zero level; and
- A minimum of one cross-section is required downstream of the structure. This is needed to determine a rating curve downstream of the structure for calculating the modular limit and drowned flow part of the structure rating curve. In this case, a steady state hydraulic model was used to develop the downstream rating which used 8 sections covering about 2,000m of river.

Apply method(s)

Development of a theoretical rating for the structure is described in detail in Appendix D, Section D.4. Extending this method to include the floodplains using the Divided Channel Method is described in Section D.6. The results are shown in Figure 4.12.

Hydraulic modelling has also been applied for comparative purposes, and the results are shown in Figure 4.13.

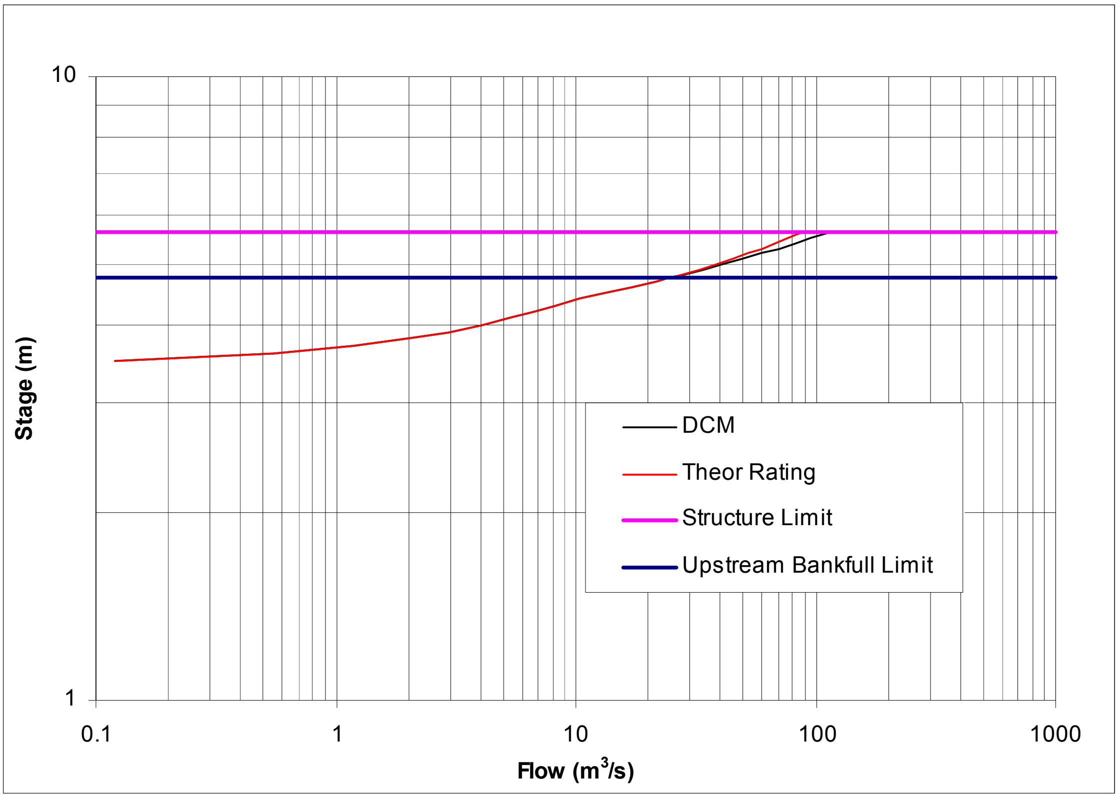

Figure 4.12: Case Study 3: DCM and theoretical rating (Case 2)

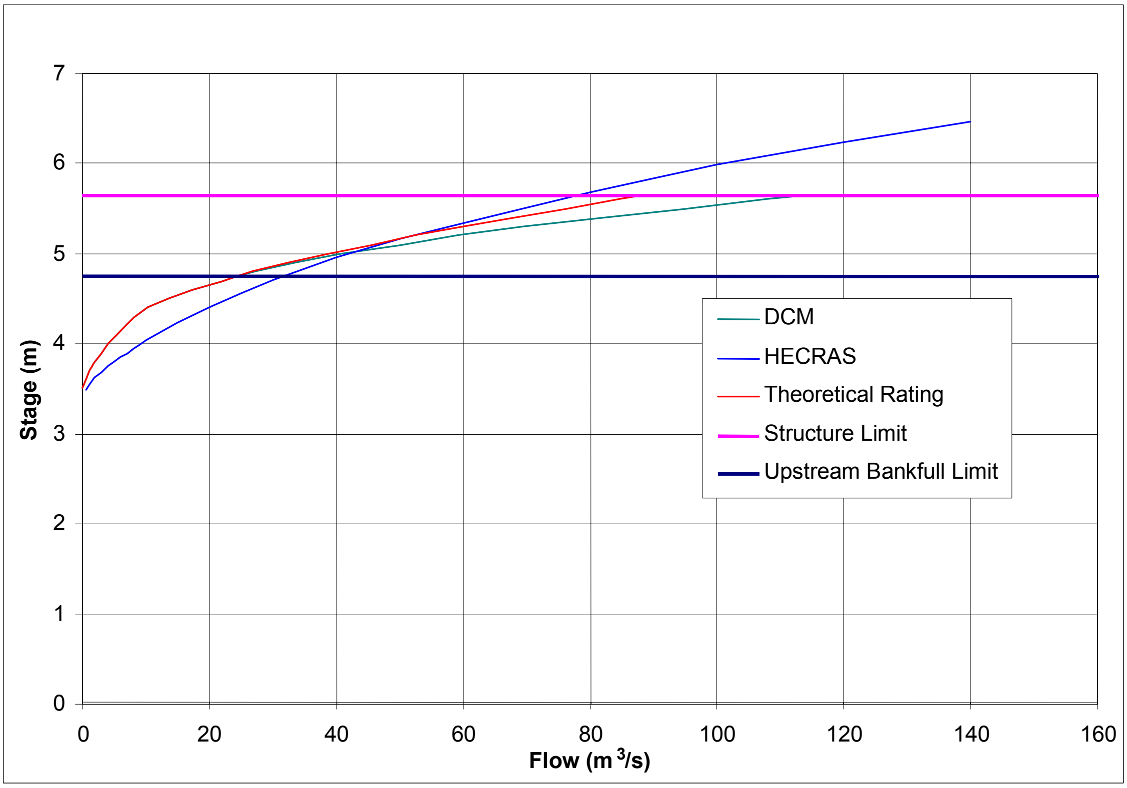

Figure 4.13: Case Study 3: Comparison of extension methods

Generate 'best estimate' rating curve

The rating curve shown on Figure 4.12 is the 'best estimate' rating curve as it is the only appropriate simple method.

The method is clearly complicated to apply because of the number of structure components and the need to calculate both the modular and drowned flow performance of each component. It is therefore more suited to the application of computational 1-D hydraulic modelling, particularly as this automatically includes prediction of downstream water levels and hence drowned flow performance of the structure. However, if computational modelling is used, it is vitally important to ensure that the structure components are correctly represented in the model including correct coefficient values and form of modular flow and submergence factor equations.

Uncertainties in the rating

It is difficult to assess the uncertainty in the rating because of the lack of gauged flow data. More work is also needed to assess the impacts of tidal conditions of the rating. This would require a detailed review of water level data downstream of the structure to identify whether significant tidal fluctuations occur during periods of high fluvial flow. If tidal effects are found to be important under high flow conditions, a family of rating curves would be needed based on both downstream and upstream water levels.

Assuming that tidal effects are not important, the uncertainty in the rating might be of the order of +/-20%. Although no gaugings are available to validate the rating, the performance of the individual structure components has been determined by modelling and should be accurate to within +/-10%.

Applicability of rating

The rating is applicable up to a flow of about 100 cumecs at a level of 5.64m AOD. Above this level, the structure will be overtopped. It is suggested that hydraulic modelling is used at higher stages in order to estimate flows over piers, wingwalls etc and take full account of drowning of the different structure elements.

5. EXTENSION OF RATING CURVES USING COMPUTATIONAL HYDRAULIC MODELLING

5.1 Overview

This section of the report is aimed primarily at staff involved in planning, commissioning, supervising and accepting rating curve extension work.

5.1.1 Background to the method

Standard hydraulic calculations for steady flows can be performed manually. Whilst this is relatively easy to do for simple geometries, it becomes much more difficult for natural channel shapes. With the development of computers, backwater programs were developed to undertake steady flow "backwater" calculations.

Many problems in river engineering, however, require unsteady flow, for example flood storage or tidal conditions. Development of unsteady flow 1-D computational models followed, and by the early 1970s major river studies in the United Kingdom (UK) were being undertaken using these techniques. Initially the models did not have a user interface and great care was needed when entering data. As the demand for models has increased, user interfaces have been developed to make the software easy to use.

1-D computational hydraulic models have now become a standard tool for river studies world-wide and the three leading software packages for 1-D computational river modelling in use in the UK have been applied in the development of this manual.

2-D and 3-D computational models have also been developed and, whilst these are more complex than 1-D models, they contain more detailed hydraulics and are a viable option for modelling complex gauging sites. A summary of the differences between 1-D, 2-D and 3-D models is given in Table 5.1.

Table 5.1: Comparison of 1-D, 2-D and 3-D models

Type of model	Features
1-D	Based on 1-D St Venant equations; Assumes flow perpendicular to model sections; Section locations based on assumed flow direction; Water level constant across model sections; No explicit allowance for losses in bends, etc; Structures treated as modules with specified coefficient values; and Steady (backwater) and unsteady flow. Generally suitable for: In-bank rating curves including downstream backwater effects; Standard structures including drowning; and Overbank flow where direction of floodplain flow known.

Table 5.1: Comparison of 1-D, 2-D and 3-D models (continued)

Type of model	Features
2-D	Based on 2-D shallow water equations; Plan view variation in velocity but average velocity applied in vertical; Predicts horizontal variation in water level and flow direction, which is particularly important for floodplain flow; Structures not explicitly modelled; Cannot model orifice flow; and Steady (backwater) and unsteady flow. Generally suitable for: Sites with significant overbank flow where the direction and magnitude of floodplain flow is not known.
3-D	Based on 3-D Navier-Stokes equations; Plan view and vertical variations in velocity; Predicts plan view variation in water level and flow direction; and Predicts secondary currents and therefore able to predict flows at structures and interaction between channel and floodplain flow. Generally suitable for: Non-standard structures; Structures with non-standard approach and other conditions; and Overbank flow where channel/floodplain interaction important.

Background to 2-D computational hydraulic models

Two-dimensional models of the Shallow Water Equations have been around in various forms since the late 1960s when JJ Leendertse of the Rand Corporation implemented a simple first order finite difference solution in Cartesian co-ordinates. The finite difference method involves discretising the equations onto a fixed rectangular grid or mesh of computational points, an example of which is shown in Figure 5.10.

Many researchers have built on the original ideas to produce research codes and some of these have been taken on and developed further by consultants and modelling specialists and used in modelling studies. The vast majority of these studies have been in the marine or estuarine environment.

Finite element models of the Shallow Water Equations have also been developed since the 1970s but despite many applications again largely in the marine or estuarine environment, as noted by Wright (2000), they can suffer from mass conservation problems. The finite element method approximates the solution of the equations at each computational point.

In recent years, finite volume codes have begun to appear which have the possibility of fully unstructured meshes and blur the difference between what is a finite element code and what is a finite difference code. The finite volume method solves the equations by integrating over a set of small volumes of fluid. These models can also be implemented using Riemann Solvers which are able to deal well with transcritical flows and the well known 'wetting and drying' problem (where grid cells become wet or dry during a simulation) in principle without violating mass conservation.

Commercial codes have been developed such as MIKE21, owned and marketed by the Danish Hydraulic Institute and TELEMAC which is owned and marketed by Electricite de France. Others are referenced in Section 2.9 or by Wright (2000)

Despite the existence of such codes, two-dimensional modelling has never become accepted practice in the fluvial environment, partly due to the computational effort required for 2-D model runs and lack of clear evidence of the benefits of using such models.

Despite this there are examples of use of two-dimensional modelling in the fluvial environment. HR Wallingford has applied TELEMAC previously to the simulation of flows around problematic flow measurement structures such as Ebley Mill on the Frome near Stroud in Midlands Region, where extensive bypassing of the gauging weir occurs, and floodplain flows such as the Severn at Gloucester, where there are a large number of obstructions to flow on the floodplains. EdenVale Modelling Services has used the TRIASSIC code to simulate dam break flows and researchers at Leeds University have used the same code to simulate out of bank flows. Peter Brett Associates have recently used the TUFLOW software to simulate floodplain flows on the Lower Arun and Cuckmere rivers in Southern Region.

Quasi two-dimensional modelling however has been used as part of one-dimensional modelling applications in the UK for many years – this facility was described in Cunge, Holly and Verwey (1980) reflecting modelling practice in the 1970s and has been implemented in all one-dimensional commercial codes known to the authors. It requires the definition of an unstructured grid of flood cells through which a flood wave may be propagated via pre-defined control sections. These methods, including so-called cell-automated methods do not involve the solution of the 2-D Shallow Water Equations and are implemented through the application of 1-D modelling software.

An example of a hybrid 1-D/2-D approach implemented in commercial software is TUFLOW referred to in Section 5.1.9. Hybrid approaches are not discussed in this report.

A discussion of available software packages is presented in Section 5.1.9.

Background to 3-D computational hydraulic models

Three dimensional models of the Navier Stokes equations have been around in various forms since the late 1970s and examples include the general purpose 3-D modelling codes FLUENT, CFX (formerly called Flow3D) Phoenics and FLOW-3D itself. Most of these codes originated from universities (FLUENT, PHOENICS) and government research institutions (CFX, FLOW-3D) and their primary focus was not on open channel flows.

The first attempts at using Computational Fluid Dynamics (CFD) considered simplified channels (e.g. Rastogi and Rodi, 1978), but did demonstrate the potential application.

Demuren (1993) applied CFD to an inbank, meandering flow with varying bed topography with some success. However, the techniques used were of low accuracy. Cokljat (Cokljat and Younis, 1995) carried out a detailed study for the case of a straight channel and limited extensions to different geometries (Basara and Younis, 1995)

Meselhe *et al* (1995) used the CFD standard k-ε turbulence model to predict flow patterns in a trapezoidal meandering channel. Hodkinson (1996) was one of the first to present results using a commercial CFD code. In this case the FLUENT (FLUENT Inc., www.fluent.com) software was used to predict the 3D-flow structure in a 90-degree bend on the River Dean in Cheshire. This work used a simplified representation of the river cross-section with a computational grid consisting of 18,000 cells which is low by current standards. The water surface was modelled both with a fixed lid assumption (i.e. where the water surface is represented by a fixed model boundary and there is no free surface) and with the correct, surveyed, position of the water surface which showed little effect on velocity predictions. Comparisons with measured velocities showed good qualitative agreement and the model accurately captured the main features of the flow. However, quantitative comparison of velocities revealed discrepancies. This was felt to be caused by the roughness representations which had necessitated a smoother value than that in the physical situation.

One of the first three-dimensional models to attempt to model natural river geometry in full was reported by Olsen and Stokseth (1995). A short reach (20 x 80m) of the Skona River in Norway was modelled. A very coarse mesh was used to discretise the solution domain (600 nodes). The discrepancy between velocities predicted by the model and those measured on site was found to be between 5 and 100%.

Sinha *et al* (1998) modelled a 4km stretch of the Columbia River downstream of the Wanapum Dam. This encompassed rapidly varying topography, contractions / expansions and multiple islands. The k-ε turbulence model was used and boundary roughness was simulated through wall functions, whereby an analytical profile is assumed to represent the effects of boundary shear layers. The position of the fixed lid used for the free surface was determined from field and laboratory measurements. This work is of particular interest for its calibration against site measured velocities at various discharges. Roughness values were amended in localised areas to improve agreement with the measured data. After this good agreement between predicted and measured values was seen.

Wu *et al* (2000) calculated the free surface position using the 2-D equations within a 3-D model. The velocities calculated from the 3-D solution were converted into depth averaged values which were then used in the 2-D equations to derive the free surface position. In addition to using the k-ε turbulence model the authors based the roughness value on the grain roughness k_s. The case considered was a 180-degree rectangular channel bend and a 12km stretch of the River Rhine. Up to 100,000 cells were used and reasonable agreement with measurements of velocity was obtained.

As for the 2-D modelling, the first modellers implemented a simple first order finite difference solution in Cartesian co-ordinates but modern codes now employ finite volume methods with unstructured meshes. A notable exception to this rule is FLOW-3D which still uses a cartesian mesh albeit with non-uniform grid sizes. To enable this, a sophisticated algorithm was developed to enable accurate representation of non-Cartesian geometries.

Despite the existence of various commercial codes, three dimensional modelling has never become accepted practice in the fluvial environment, partly due to problems resolving the different structures, topographic features and scales associated with channel flows, floodplain flows and indeed the wetting and drying fronts.

Despite this there are examples of use of three-dimensional modelling in the fluvial environment. HR Wallingford have applied both SSIIM and FLOW-3D to a range of structures including weirs and side weirs. EdenVale Modelling Services have used the SSIIM code to simulate river restoration measures and also CFX to simulate the influence of outfalls on the marine environment.

Layered two-dimensional models with hydrostatic pressure distribution within the layers has also been carried out but this is generally an extension of two-dimensional modelling techniques and are not discussed in this manual. They do not involve the solution of the full Navier Stokes equations, including vertical momentum.

A discussion of available software packages is presented in Section 5.1.9

5.1.2 Theory and assumptions behind computational hydraulic models

This section aims to provide a brief overview of the theory behind computational hydraulic models.

1-D computational hydraulic models are based upon numerical solution of the backwater equation for steady flow or the St. Venant equations for unsteady flow. In either case, the flow resistance is estimated using a standard resistance formula, such as the Manning formula, and hydraulic structures by structure equations, such as those in the British Standards. No attempt is made to model flow resistance effects, or flows at structures, in detail.

The equivalent set of equations in two-dimensional flow to the well known St. Venant equations used to describe fluid flow in one dimension, are the so-called Shallow Water Equations. These express the conservation of mass and momentum in nearly horizontal flows. As such the key assumptions are that vertical velocities and accelerations are negligible and the water depth is small compared to its width.

Various methods are available to solve these equations and the main ones are summarised below:

i. The finite difference method is the simplest method whereby the equations are discretised onto a fixed rectangular grid or mesh of computational points. Numerical analysis is then performed to determine equations to solve for flow and level at each point. While it is more usual to solve for both flow and level at each point (a collocated mesh), it is also possible to solve for flow and level at separate adjacent points – referred to as a staggered mesh or grid.

ii. A variant on this approach is to solve the equations on a body fitted or curvilinear mesh whereby the rectangular grid is transformed onto a non-rectangular grid which may more accurately reflect the position of embankments or other features on the floodplain or river channel.

iii. Another approach is the finite element method which approximates the solution of the equations at each computational point using linear or quadratic 'base functions'. The advantage or this approach is that the computational grid is not constrained to be rectangular or even curvilinear. It is not required to have any structure at all and indeed is usually referred to as using an unstructured grid, as used for the case study in this report.

iv. The final and possibly best approach is the finite volume method which solves the equations by integrating over a set of small volumes of fluid. Like the finite element method, it can be used on an unstructured mesh but it is recognised as being better at conserving mass than the finite element method. Conserving mass ensures that the same amount of fluid leaves the computational domain as that which enters, under steady state conditions.

2-D computational hydraulic models are based upon numerical solution of the Shallow Water Equations for steady and unsteady flow. The flow resistance term is estimated using a standard resistance formula, such as the Manning equation. Hydraulic structures are very rarely represented by any form of special treatment in the mesh and are thus treated as a specific linear feature of the topography that must be resolved closely.

3-D computational hydraulic models are based upon numerical solution of the full Navier Stokes Equations for steady and unsteady flow. The friction term is estimated using a surface roughness formula rather than a flow resistance formula, such as the Manning formula. As for the 2-D case, hydraulic structures are not represented by any form of special treatment in the mesh and are thus treated as a specific linear feature of the topography that must be resolved closely.

Steady and unsteady flow

As noted above, computational river models may be used with steady and unsteady flow. In steady flow, a single flow is used throughout the model and no account is taken of the transient effects of a flood hydrograph as it passes down the river. In steady flow it is assumed that all storage areas are full. Steady flow can be simulated in two ways: using a backwater calculation or running a steady flow using an unsteady flow solver. The backwater method can be applied in 1-D and 2-D models, but not in 3-D models. Steady flow can be simulated using any computational model by running the unsteady flow solver with steady flow boundary conditions.

Unsteady flow enables the model to simulate transient effects including flood storage, etc.

Representation of the river system

1-D models

The river is represented by a series of river cross-sections. 1-D models can be used to represent any configuration of river system including loops and branches.

Floodplains are generally represented in the following ways:

- As extensions to each river cross-section. This approach is normally used where there are no embankments or high ground between the river and the floodplain;
- As separate floodplain cross-sections, where the floodplain forms a separate flow path to the river. This approach is normally used where there are embankments or high ground between the river and the floodplain. In this case there is a flow path between the river and the floodplain which is normally represented by weirs in the model; and
- As static storage areas, where the floodplain does not have a flow path passing through it. In this case it is represented as an "offline" storage area with a single flow path connecting it to the adjacent river or floodplain.

2-D models
The river system is represented by a series of cells in plan view which are usually triangular in the case of unstructured finite element or finite volume meshes or Cartesian in the case of finite difference grids. 2-D models can be used to represent any configuration of river system including loops and branches.

Floodplains are not treated any differently to river cells except that they are normally dry for in-bank flow. In practice it is usually imperative for there to be a minimum depth of flow on the floodplain even for in-bank flow and it is this issue that can lead to problems with mass conservation. It is normal practice however to constrain the mesh to accurately resolve linear features such as embankments and hydraulic structures. Static storage areas are thus treated in the same way as dynamic floodplain cells. An example of a 2-D computational mesh is shown in Figure 5.10.

It is not possible to represent culverts or gates in two-dimensional models.

3-D models
The river system is represented by a grid of cells in both plan view and through the water column which may be cuboid for structured meshes but are usually tetrahedral in the unstructured case. 3-D models can be used to represent any configuration of river system including loops and branches but are usually restricted in their scope to areas surrounding particular features such as structures or confluences due to the considerable number of grid cells and thus computing resources necessary for their application. An example of a computational grid is shown in Figures 5.17 and 5.18.

As for the 2-D case, floodplains are not treated any differently to river cells except that they are normally dry for in-bank flow. In practice it is usually imperative for there to be a minimum depth of flow on the floodplain even for in-bank flow and it is this issue that can lead to problems with mass conservation. Again as for the 2-D case, it is normal practice however to constrain the mesh to accurately resolve linear features such as embankments and or course hydraulic structures. Static storage areas are thus treated in the same way as any other grid cell

Unlike the 2-D case however, it is possible to represent culverts and gates in three-dimensional models.

Representation of flow in the river system

1-D models

1-D models make the assumption that the flow direction is perpendicular to the model sections. It follows from this assumption that:

- Lateral flows and velocities are neglected;
- The water level is constant across the section; and
- Flow occurs across the whole width of the section. The modeller may restrict flow to part of a cross-section by the use of "conveyance pointers". This is normally done when part of the cross-section represents an area of flood storage with very little downstream flow.

Where there are significant lateral variations in velocity, for example at bends, 1-D models will underestimate head losses. In addition, there can be considerable variations in level across the floodplain and this is not represented by a 1-D model.

2-D models

2-D models do not make the assumption that the flow direction is perpendicular to the mesh as required for a 1-D model and thus in principle, the mesh orientation is independent of the flow direction. It follows from the basic Shallow Water Equations and the assumption therein that:

- Lateral flows and velocities are included;
- The water level can vary across a floodplain so that super-elevation around meanders can be simulated; and
- Out of bank flow can occur along flow paths which do not need to be defined 'a priori' by modellers.

Where there are significant lateral variations in velocity, for example at bends, 2-D models should be more accurate than 1-D models in the estimation of head losses. In addition, there can be considerable variations in level across the floodplain and a 2-D model with appropriate resolution of key topographic features can represent this.

3-D models

3-D models do not make the assumption that the flow direction is perpendicular to the mesh as required for a 1-D model and thus in principle, the mesh orientation is independent of the flow direction. It follows from the basic Navier Stokes Equations and the assumption therein that:

- Lateral flows and velocities are included;
- The water level can vary across a floodplain so that super-elevation around meanders can be simulated; and
- Out of bank flow can occur along flow paths which do not need to be defined beforehand by modellers.

Where there are significant lateral variations in velocity, for example at bends, 3-D models should be more accurate than either 2-D or 1-D models in the estimation of head losses due to the resolution of the vertical flow structure, including secondary flow circulations. In addition, there can be considerable variations in level across the floodplain and a 3-D model with appropriate resolution of key topographic features can represent this in the same way as a 2-D model.

Friction flow
Flow in river channels is represented in 1-D and 2-D models using friction flow, where the gravitational downstream movement of water is resisted by the friction of the river bed, banks and other features that resist flow, for example vegetation. There are a number of different formulae for representing friction flow in hydraulic models including the Chezy, Manning and Colebrook-White equations. The equation most frequently used is the Manning equation.

In 1-D models, friction flow is also used to represent flow on the floodplain where the floodplain cross-section is an extension of the river cross-section. Where the floodplain cross-sections are separate from the river cross-sections, the floodplain resistance is either represented using a friction flow formula or by a series of weirs at floodplain cell boundaries.

In 2-D models, floodplain flow and flow between the main channel and the floodplain is represented by the application of the full Shallow Water Equations.

Surface roughness is represented in 3-D models. For example, FLOW-3D uses a parameter called *ks* to represent surface roughness. The *ks* value is analogous to the Colebrook–White roughness length in that both have dimensions of length. The precise relationship between *ks* and Manning's n or indeed Colebrook-White k_s is not defined in the literature.

As opposed to the 1-D and 2-D cases, interaction between floodplain flow and the main channel and the floodplain <u>is</u> represented by the application of the full Navier Stokes Equations.

Use of roughness formulae
The Manning equation is a simple expression for flow resistance where the magnitude of the resistance is represented by a coefficient known as Manning's n. Whilst this equation has limitations, it is very widely used and guidelines for estimating the resistance of river channels and floodplains are normally expressed in terms of n.

Particular effects which are not taken automatically into account in the use of the Manning equation include:

- Additional flow resistance caused by bends;
- Variation of Manning's n with stage. Generally in-bank values of n decrease as the flow and water level increase; and
- Significant changes in the channel cross-section (transitions).

Manning's equation is the most commonly used roughness equation in 1-D and 2-D models. However this is not generally the case in 3-D models, which tend to use method specific roughness equations.

It should also be noted that the concept of roughness in a 1-D or 2-D model is not the same as that in a 3-D model due to the need to represent energy losses associated with processes not explicitly represented by lower dimensional models. For 1-D models this includes losses due to changes in channel geometry and in the 2-D case losses due to secondary circulations in the flow structure. For a 3-D model, all processes are represented by the Navier Stokes Equations.

Flow over embankments

Weir flow formulae are used to represent flow over embankments in 1-D models. These formulae have been developed for standard weir structures where the flow is perpendicular to the weir crest. In practice the approach angle of flow to river embankments is not perpendicular, and the modeller must make allowances for this when calibrating the model.

Weir flow formulae are also used in 1-D models to represent floodplain flow in some models in the case where the floodplain is separate from the river channel. In such cases the floodplain consists of "reservoirs" between the cross-sections where the water level is constant. The cross-sections are modelled as weirs with a discrete head loss. This approach is most appropriate where the main losses on the floodplains are caused by linear features such as hedges and raised tracks.

In 2-D models, the full Shallow Water equations are used to represent flow over embankments. In 3-D models, the full Navier Stokes equations are used to represent flow over embankments. It is imperative that the computational mesh in both 2-D and 3-D models resolves linear features on the floodplain such as embankments or raised tracks.

Modelling of floodplains

When modelling floodplains using 1-D models, it is very important to estimate the direction of flow on the floodplains before selecting cross-section locations. This is because 1-D models assume that the flow is perpendicular to the cross-section.

In straight rivers with parallel floodplains the selection of floodplain cross-section locations is relatively straightforward. However for sinuous channels the flow direction changes with stage and there is considerable inflow and outflow from the river channel. In such cases a 1-D model is at best an approximation and the modeller is advised to select cross-section locations which correspond to the flow direction at the flow of greatest interest.

Where embankments exist between the river and the floodplain, some 1-D steady flow (backwater) models assume a constant water level across the river and floodplain even though there may be no flow on the floodplain. Other steady flow (backwater) models only assume flow on the floodplain when the water level is higher than the bank level.

When modelling floodplains using 2-D models, it is less important than modelling in 1-D to estimate the direction of flow on the floodplains before calculating or determining a computational grid.

When modelling floodplains using 3-D models, it is not necessary to estimate the direction of flow on the floodplains before calculating or determining a computational grid. It is however necessary to resolve the surface position accurately with any fixed grid model such as FLOW-3D. Some models such as CFX have a capability to automatically adapt the grid according to the computed flow patterns and this can be an extremely useful, if computationally expensive, feature.

Two stage channels
There is considerable interaction between flows in the river channel and the floodplains, which causes a discontinuity between the in-channel and overbank rating curves. This effect is not included in the current 1-D and 2-D models since it is a three dimensional effect. It is however often important for accurate assessment of overbank flow, particularly at low depths of flow on the floodplains. This interaction is however included in current 3-D computational models.

Sediment movement and scour
Most model applications (with the notable exception of the 3-D software SSIIM) assume that the river bed is fixed and no account is taken of sediment movement. Many British rivers are relatively stable and the fixed bed assumption is reasonable. However in other cases significant sediment movement can occur during a flood. This will change the river cross-section and the flood water levels.

It is good practice to survey the gauging section regularly to identify geomorphological changes in the section. This practice may not however identify the river cross-section during flood events because sediment mobilised during the flood will settle after the event. In this case the cross-section at the peak of the flood will not be accurately known.

If sediment movement is considered to be important, some 1-D and 2-D modelling software have sediment modules which can be used to assess changes to the river cross-section. The difficulty of using such modules is the availability of calibration data. It is sometimes possible to estimate the river bed shape during a flood by a flood flow current meter gauging although this is subject to uncertainty because of drag on the cable.

If sediment movement is considered to be important, very few of the 3-D modelling software have sediment modules which can be used to assess changes to the river cross-section.

Structures
The empirical formulae used for structures in 1-D models are based on open channel flow theory but with flow coefficients obtained from experimental work. Whilst there is a reasonable degree of reliability in the coefficients used for standard structures which have been tested in the laboratory (for example, British Standard flow measurement weirs), the majority of river structures are 'non-standard'. In addition, many structures operate under 'drowned' flow conditions during floods, where the uncertainties in their performance are greater than under 'modular' flow conditions.

Empirical formulae are not used for 2-D modelling of structures. Since vertical accelerations are neglected in 2-D models, the modelling of structures in general may be less accurate than either 1-D or 3-D models.

Empirical formulae are not used for 3-D modelling of structures since the full physics of the flow are represented by the Navier Stokes Equations.

5.1.3 Hydraulic conditions where modelling can and cannot be applied

Whether or not hydraulic modelling can be applied to extend the rating curve at a specific site is to some extent a value judgement based upon experience and an assessment of whether the hydraulics of the model differ from those of the actual gauging site. However, some general guidelines as to when hydraulic models may be expected to give reasonable results are given in Table 5.2.

It would be desirable to give specific examples of where the use of models to extend rating curves has and has not been shown to work. However there is very little information on gauging sites that have been modelled to extend the rating curve, where the results are compared with a reliable rating curve extension from another source.

Table 5.2: Hydraulic conditions to which models are theoretically suited or not suited

Hydraulic condition (note: S indicates suited, N indicates not suited; where there is a choice, the option in bold is likely to be the most suitable method taking into account hydraulics, costs, etc.)	**Model type**		
	1-D	**2-D**	**3-D**
In-bank rating relationship where the rated section is stable, the rating is controlled by variations in stage and velocity along the channel, and lateral variations in stage are not significant. An example of this case is a stable velocity-area rated section.	**S**	N	S
Rating relationship controlled by modular flow at standard and non-standard structures.	**S**	N	S
Rating relationship controlled by drowning of standard and non-standard structures due to a non-variable backwater where downstream water levels are available to calibrate the drowned flow performance of the structure.	**S**	N	S
Structures where the approach conditions are far from ideal (for example, downstream of a bend or the structure is at a skew angle to the channel), and there are little or no calibration data. 3-D models do not depend on the structure coefficients based on specified approach conditions. Unlike 3-D models which, given appropriate grid resolution, account for lateral variations in channel flow structure, 1-D models are unable to accurately model the flow conditions that would occur when the approach conditions are not ideal.	N	N	S
Structures rated by current meter gaugings that cannot be adequately approximated by the Shallow Water Equations. This is because there are flow patterns and secondary currents that are not represented in 1-D or 2-D models.	N	N	S
Drowning of non-standard structures where the drowning effects can be represented by the Navier Stokes Equations.	N	N	S
Drowning of non-standard structures where the drowning effects cannot be represented by the Shallow Water Equations. Example could include cases where the hydraulic jump or roller downstream of a structure exhibits considerable energy loss – this being a three-dimensional feature.	N	N	S

Table 5.2: Hydraulic conditions to which models are theoretically suited or not suited (continued)

Hydraulic condition	Model type		
	1-D	**2-D**	**3-D**
Drowning of non-standard structures where there are no downstream water level data to produce a rating curve downstream of the structure. This is because models require downstream water levels to calibrate the modular limit and drowning function of the structure and ensure that drowning occurs at the correct flow. Models can provide default values but these will have a degree of uncertainty.	N	N	N
Complex non-standard structures with little or no calibration data. 3-D models can be used in this case because they are able to model the hydraulics of structures in detail. The only empirical calibration required is for surface roughness calculations for drowned flows.	N	N	S
By-passing of the gauging site where the flow paths are reasonably well defined and straight.	S	S	S
Rating relationship significantly affected by outflanking of the gauging site whether or not the flow paths are reasonably well defined.	N	S	S
Situations where rating equations are affected by flows over a wide area, for example extensive floodplains. Problems may arise with the grid size necessary to simulate extensive areas of floodplain.	S	S	N
Wide floodplains, where there are significant variations in velocity and level across the floodplain. Unlike 2-D models, 1-D models assume that the water level is constant across the floodplain and no allowance is automatically made for lateral flows.	N	S	S
Floodplains where there are embankments and other obstructions which affect the high flow rating, and there are no calibration data. 1-D models require flow coefficients for these structures which are not standard values.	N	S	S
Complex and/or wide floodplains where the floodplain flow has a significant impact on the rating, but little topographical data for the floodplain are available. This is because information on flow direction is needed to locate floodplain cross-sections and constrain the 2-D and 3-D model mesh to linear features that may affect the flows. Without this information, floodplain flows may be in error in the model and this in turn will lead to uncertainties in total flow.	N	N	N
The transition between in-bank and overbank flow, where interference occurs between the fast moving channel flow and the slow moving floodplain flow. This is because the three-dimensional hydraulic effects of the interference are included in 3-D models. These effects can be very significant for low depths of floodplain flow.	N	N	S
Situations where any form of orifice or closed channel flow is significant such as in culverts, sewers, bridges or gated structures. In 3-D models, these flows are modelled explicitly.	S	N	S

In practice an approximation to many of these conditions can be achieved but there will be an associated degree of uncertainty.

5.1.4 Potential accuracy of rating curves extended using computational hydraulic models

1-D models

Results from the case studies and other work referred to in the Stage 1 Report show that uncertainties in flow predictions from rating curves extended using appropriate 1-D computational models can be less than 10% compared with actual (measured) flows for the following cases:

- In-bank flows at rated sections;
- Modular flows at standard structures;
- Modular flows at non-standard structures where the discharge coefficient has been calibrated against flow gaugings; and
- Flows which are within the structure-full limits of gauging structures, the downstream rating is known, and the modular limit and drowning factor are calibrated.

Uncertainties increase for overbank flow and situations where the hydraulics are not well represented in the 1-D model. For example, where overbank flow occurs at a complex site, the uncertainty has been shown to rise to +/-40% for the section of the rating curve affected by a discontinuity at low depths of overbank flow. However the uncertainty reduces at higher flows to about +/- 30% for the highest gauged flow (Environment Agency 2002, Case Study 2).

It is not possible to generalise about the uncertainties for overbank flows as these are site specific. For example, Montford on the River Severn has a straight channel with parallel floodplains and gauged overbank flows. In this case the uncertainty associated with extension of the rating curve may be of the order of 10%. For more complex sites where overbank flow data are not available, the uncertainties will be of the order of 20 to 30% or even higher.

From the above discussion, suggested uncertainty ranges for the extension of rating curves at some of the sites categorised in Table 5.2 are as follows:

- Rating relationship controlled by along channel variations in stage and velocity and where the rated section is stable:
  +/-10% for in-channel flows

- Rating relationship controlled by modular flow at standard and non-standard structures:
  +/-10% for in-channel flows

- Rating relationship controlled by drowning of a British Standard structure due to a non-variable backwater and the downstream rating curve is known:
  +/-10% for in-structure flows but can be higher near the point where drowning occurs

- By-passing in reasonably well defined straight flow paths:
  +/-15-30% depending on the proportion of total flow on the floodplains and the stage. The uncertainty can be large at low

depths of overbank flow (greater than 30%) but this may reduce as the stage increases and the effects of channel/floodplain flow interactions reduce.

2-D models
The improvement in accuracy associated with 2-D modelling as opposed to 1-D modelling primarily arises from better representation of outflanking of flow gauging structures, in particular where the flow paths are not known prior to a study or simulation. In this case it is estimated that accuracy may increase from the order of +/-30% error in the case of significant outflanking to a figure of the order of +/-15% or better depending on the availability of appropriate topographic data. If topographic data to define the outflanking flow is sparse or not available, it is unlikely that 2-D modelling will increase the accuracy obtained from a 1-D or quasi-2D representation using control sections or simple weir formulae.

As for the 1-D case, it is not possible to generalise about the uncertainties for overbank flows as these are site specific.

3-D models
The improvement in accuracy associated with 3-D modelling as opposed to 1-D or 2-D modelling primarily arises from the explicit representation of complex or non-standard structures and the interaction between main channel and floodplain flows. For the former case, the estimated error reduction compared with 1-D and 2-D codes should be from 30% to about 10% depending on the availability of local survey data at and near the structure. For the latter case, the estimated error reduction could be from 25-30% down to 15%. These estimated reductions are based on the theoretical performance of 3-D models and the authors' own professional judgement as there are very few direct comparisons of model results that could be used to provide more definitive information.

There are also potential improvements in accuracy in cases of outflanking of flow gauging structures, in particular where the flow paths are not known in advance. Note that outflanking may or may no involve interaction between main channel and floodplain flows. 2-D models may however be better in these cases due to the number of grid points necessary to simulate large flow areas. In this case it is estimated that the error may reduce from the order of 30% error in the case of significant outflanking to a figure of the order of 15% depending on the availability if appropriate topographic data. If topographic data to define the outflanking flow are sparse or not available, it is unlikely that either 2-D or 3-D modelling will increase the accuracy obtained from a 1-D or quasi-2D representation using control sections or simple weir formulae.

As for the 1-D and 2-D cases, it is not possible to generalise about the uncertainties for overbank flows as these are site specific.

Factors affecting accuracy

The accuracy of a rating curve extended using a computational model is affected by a number of factors, the most important of which are:

- The appropriate representation of the hydraulics in the model. In particular this relates to the resolution of key linear features in the computational grid of 2-D and 3-D models;
- The accuracy of the hydrometric data against which the models are calibrated. This is less important for 3-D models;
- Accuracy of flow measurement using standard structures. A flow measurement accuracy of +/-5% is considered to be a very good result;
- Accuracy of water levels used to construct the rating. Theoretically this should be small if the water level recorder has been accurately levelled into position;
- Maximum flow and level for which the model is calibrated. The accuracy of model calibration is clearly a key element in the overall uncertainty associated with 1-D computational river models. A typical uncertainty for a well-calibrated model of a gauging site might be of the order of 0.10m but this will increase significantly for predicted flows that are much greater than the highest calibration flow. Accuracy of model calibration is less important for 3-D models where the physics are modelled explicitly; and
- Extension of the rating curve into conditions where the flow pattern changes, for example drowned flow or overbank flow. It is often difficult to assess accuracy for these flow states because of the lack of calibration data. However in principle 3-D models should out-perform 2-D and 1-D models at an equivalent grid resolution, and similarly 2-D models should out-perform 1-D models.

5.1.5 Staff experience required

The staff experience required to use computational models for the extension of rating curves should include:

- Good understanding of basic stage/discharge analysis and experience of deriving stage/discharge relationships;
- Good knowledge of open channel hydraulics and the methods used by the software;
- Knowledge and understanding of the site specific hydraulics and flow paths;
- Competence in the use of computational models;
- Good knowledge of Computational Fluid Dynamics (CFD) for 3-D modelling and the methods used by the software;
- Demonstrable competence in the use of higher dimensional computational models for 2-D and 3-D modelling;
- Demonstrable competence in the specific software used for modelling;
- An understanding of the limitations of modelling software and the likely accuracy;
- An appreciation of the behaviour of the river being studied in order to relate model results to reality;
- Being able to specify the site survey requirements and ensure that the as-built model satisfactorily represents the network;
- Being able to specify the site survey requirements and ensure that the as-built computational grid adequately represents the local hydraulics including significant linear features; and
- A questioning mind, so that information provided either for the construction or operation of the model is checked and not accepted at face value. This includes topographic, flow and water level data, and outputs from the model. This is

particularly relevant for 2-D and 3-D models where results may be more difficult to interpret.

In reality it may require a team of at least two people to undertake a rating curve extension using 2-D and 3-D modelling, a specialist modeller and someone with operational knowledge.

5.1.6 Time and cost of using 1-D models

Any model study will be unique and the associated costs specific to each case. However some guidance is given in this section on the time taken for undertaking a model study to extend a rating curve, and the associated cost. The cost of a model study will primarily comprise:

- Model specification;
- Field survey;
- Model construction;
- Model calibration;
- Prediction of the rating curve; and
- Reporting.

Indicative times and costs for three examples of rating curve extension are given in Table 5.3 below. The sites covered by each case study are outlined below:

- Example 1 is a rated section with a floodplain on the right bank, separated from the river by high ground. The length of the modelled reach is about 2km and required seventeen channel cross-sections and a floodplain survey (this is based on Case Study 4, see Figure 5.2);
- Example 2 is a weir in a very complex site with over 5km of main river channel, a considerable length of subsidiary channels and extensive floodplains; and
- Example 3 is a weir with little overbank flow. About 1.5km of river are modelled using 19 channel cross-sections survey (this is based on Case Study 5, see Figure 5.5).

The costs below are based on the assumption that ten river cross-sections are needed for Example 1 and eight river cross-sections for Example 3.

The costs in Table 5.3 are based on recent experience of constructing and calibrating small hydraulic models, but do not include the cost of collecting any hydrometric data. They also do not include the initial investment in software, which may range from:

- less than £1,000 for a 1-D steady flow (backwater) package;
- in excess of £5,000 for a 1-D unsteady flow package;
- less than £10,000 for an unsupported 2-D or 3-D package available through a University;
- in excess of £20,000 for a fully supported 2-D commercial package; and
- in excess of £30,000 for a fully supported 3-D commercial package.

These are based on the costs for a fixed-price ‘turnkey’ contract. The estimated costs below are similar for 2-D and 3-D models. The reason for this is that the costs for setting up 2-D and 3-D models are similar and the cost depends to some extent on the experience of the modeller and familiarity with the software.

Cost savings could be made by using a model which has already been constructed for another purpose, for example floodplain mapping.

Table 5.3: Indicative time and cost of rating extension using computational models (Year 2002)

Example	Model	Study time (elapsed-weeks)		Costs (£'000)			
		Elapsed weeks	Person weeks	Planning and data collection	Survey	Modelling	Total
1	1-D	5	6	2	5	5	12
2	1-D	10	15	5	10	15	30
3	1-D	3	4	2	3	3	8
1	2-D	8	9	2	5	10	17
2	2-D	14	19	5	10	20	35
3	2-D	N/A	N/A	N/A	N/A	N/A	N/A
1	3-D	8	9	2	5	12	19
2	3-D	N/A	N/A	N/A	N/A	N/A	N/A
3	3-D	4	5	2	3	10	15

5.1.7 Data requirements

Broadly speaking, two types of data are required for computational hydraulic modelling: topographic data, describing the physical layout of the river, floodplains and structures to be modelled, and hydrometric data, describing the flow of water. For the purposes of this report, surveys of extreme water levels, such as wrack marks, are classed as hydrometric data since they relate to the flow of water.

Topographic data requirements
Topographic data requirements will establish, and be established by, the model layout, and will include some or all of the following:

- River cross-section surveys;
- Floodplain cross-section surveys and/or floodplain contour maps;
- Embankment survey levels;
- Dimensions of structures in river channels and on floodplains; and

- Other features which affect the flow, such as embankments for railway lines and roads.
- Floodplain DTM data such as that gathered from LIDAR.

Hydrometric data requirements
Hydrometric data provide boundary conditions in terms of flows and water levels for the model, and calibration data. Hydrometric data required for a river model will include some or all of the following:

- The existing rating curve, which is to be extended;
- Rating curves for other points within the river reach being modelled;
- Flow and level hydrographs at the gauging station where the rating curve is to be extended;
- Flow and level hydrographs at other points within the river reach being modelled;
- Current meter gaugings at any points within the river reach being modelled;
- Peak water level measurements within the river reach being modelled;
- Floodplain velocity measurements within the river reach being modelled;
- Measurements of gate openings or variable crested weir levels during structure operation; and
- Flood outlines for the river reach being modelled.

Further details of data requirements can be found in Section 5.3.1.

5.1.8 Risks in using the method

When undertaking a modelling study to extend a rating curve, as with all modelling work there are risks that the quality of the extension will be below what is required and that the work takes longer, or costs more, than expected. There are however measures that can be taken to assess the risks and reduce them to acceptable levels. If appropriately applied, modelling is potentially an important approach to extending rating curves particularly where significant outflanking of the gauging site occurs.

Risks to quality
The principal unsatisfactory outcomes from a study will be that no extension can be found, that an extension can be found, but that it is unacceptably inaccurate, or that an extension is found that appears to be acceptably accurate, but actually is not.

The most important risks that need to be appreciated and minimised in order to avoid these unsatisfactory outcomes are:

- Simplifying assumptions in models may lead to errors particularly where calibration data are not available. Those assumptions which are of particular concern for flow gauging include variation of flow resistance with depth, the transition between in-bank and overbank flow, assumptions about the drowning characteristics of structures, and the treatment of complex structures. As the models become more complex, some of these simplifications become less important. For example, 3-D models are able to simulate vertical flows at structures whereas 2-D models are not;

- Use of unrealistic calibration parameter values to force the model to fit observations. It is likely that either the model cannot represent the particular situation, or there are errors in the data;
- Use of default discharge coefficients without appreciating the wide variation that can occur between different structures;
- Use of roughness values based on low flows for high flows, leading to over-prediction of levels;
- Use of old survey data, which do not take account of recent changes in channels and floodplains;
- Choosing a modelling software package that does not adequately approximate the real hydraulics. For 2-D and 3-D models this will relate to its ability to generate an appropriate computational mesh;
- Choosing to use 1-D modelling when it is not the appropriate tool; and
- Lack of good quality model calibration data, although 3-D model results are expected to be less sensitive to calibration data than lower dimensional models which use empirical parameters.

Risks leading to cost and time increases
The most important risks that need to be appreciated and minimised in order to avoid cost and time overruns are:

- Insufficient assessment of the hydraulics leading to an incorrect modelling strategy;
- Hydrometric data requires more quality control than expected;
- Survey data requires more quality control than expected;
- Insufficient calibration data available, requiring additional time to set up the model, or additional sensitivity analysis;
- Insufficient survey data collected, requiring an additional survey;
- Model does not give results of sufficient quality, requiring further investigation and modelling to improve the result; and
- Use of inexperienced or incompetent staff.

5.1.9 Software available to implement 1-D computational hydraulic models

There are many different software packages that can be used to implement 1-D computational hydraulic models, all based on essentially the same principles, but with different strengths and weaknesses. These strengths and weaknesses can relate to both the representation of the hydraulics, and the user-friendliness of the package.

The recommended software packages for implementing 1-D hydraulic modelling for rating curve extension are the Agency's three Best Interim Systems (BIS's) for 1-D hydraulic modelling:

- HEC-RAS;
- ISIS Flow; and
- MIKE 11.

This project makes no particular recommendation for particular software packages for implementing 2-D and 3-D hydraulic modelling for rating curve extension. That would require a benchmarking exercise analogous to that currently being undertaken for the Agency's Best Interim Solutions for 1-D hydraulic modelling.

It is recommended that the software chosen for 2-D modelling should have an unstructured grid capability together with the ability to constrain the grid according to the key topographic features near the location where the rating extension is required. A constrainable body fitted or curvilinear mesh may also be acceptable in many cases.

Examples of 2-D modelling software are given in Table 5.4.

Table 5.4: Examples of 2-D modelling software

Name	Commercial or Research	Source of Software	Website for Information
TELEMAC	Commercial	EDF, France	http://www.wallingfordsoftware.com/products/telemac.asp
Tuflow	Commercial	WBM Consultancy Brisbane, Australia	http://www.wbmpl.com.au/home.htm
HEMAT	Research	Cardiff University, Wales	N/A
DIVAST	Research	Cardiff University, Wales	N/A
MIKE21	Commercial	DHI Water and Environment, Denmark	http://www.dhisoftware.com/ mike21
DELFT3D	Commercial	Delft Hydraulics, Netherlands	http://www.wldelft.nl/soft/d3d
AMAZON	Research	Manchester Metropolitan University, UK	http://www.vows.ac.uk
POLPRED	Research	Proudman Oceanographic Laboratory, UK	http://www.pol.ac.uk/appl/polpred.html
CH3D	Research	Waterways Research Station, Vicksburg, USA	http://chl.wes.army.mil/software/ch3d
TRIASSIC	Research	Leeds University, UK	N/A

It is recommended that the software chosen for 3-D modelling should have an unstructured grid capability or at least a sophisticated means of representing key non-Cartesian features. The ability to constrain the grid according to the key topographic features near the location where the rating extension is required is essential. In many cases, a constrainable body with fitted, curvilinear (or even non-uniform Cartesian mesh) may also be acceptable.

Examples of 3-D modelling software are given in Table 5.5.

Table 5.5: Examples of 3-D modelling software

Software Name	Primary application (Commercial or Research)	Source of Software	Website for Information
FLOW-3D	Commercial	Flow Science, Los Alamos, USA	www.flow3d.com
CFX	Commercial	AEA Technology, UK	www.cfx.aeat.com
FLUENT	Commercial	Fluent Inc., USA	www.fluent.com
PHOENICS	Commercial	CHAM Ltd, UK	www.cham.co.uk
SSIIM	Strictly research only	Trondheim University, Norway	www.sintef.no/nhl/vass/ssiim.html

Other packages may also be used where they offer an improved representation of the hydraulics, have a proven track record of successful application to similar situations, and where there is sufficient expertise available in their operation.

5.2 Step by Step Procedure

5.2.1 Overview

The main steps in the application of computational hydraulic modelling to extend a rating curve are shown on the flow chart in Figure 5.1. Although it is primarily a linear process, there are many options for reviewing and revisiting earlier stages in the light of later stages. These should be done where considered necessary, and some of the most important feedback loops are shown on the Figure.

2-D and 3-D modelling should only be chosen where there is a clear advantage over other methods as the amount of effort required and hence the cost is greater than the use of simple hydraulic techniques or 1-D models. If there is any doubt regarding the application of this method at the outset, a simpler approach should be considered first. The knowledge gained through the application of simpler methods will help to determine whether or not a 2-D or 3-D model should be used.

These tasks are described in more detail below.

It is expected that all work described in Section 5.2 would be carried out by a competent modeller.

5.2.2 Understand the hydraulics

This is initially done at a high level in order to decide which general approach to rating curve extension is to be used. Having chosen to use computational modelling, the purpose of this task is to develop a more detailed understanding of the site hydraulics in order to design the model in detail. The steps needed to understand the hydraulics are described in Section 3.2, including the preparation of a brief planning document.

5.2.3 Obtain and collate relevant hydrometric data

All hydrometric data that might be required should be identified and reviewed.

Section 5.3.1 describes the possible hydrometric data requirements for using hydraulic models to extend rating curves. The specific data requirements for a given modelling project will need to be identified based upon what data are available, and how the model is to be set up.

The information should be collated with the assistance of the Agency's Area hydrometric team. Where data are not available the need for the data should be assessed. If the data are needed and collection is feasible, arrangements should be made for its collection.

A brief summary statement should be prepared of available hydrometric data including its location and quality. Copies of information required to extend the rating curve should be obtained.

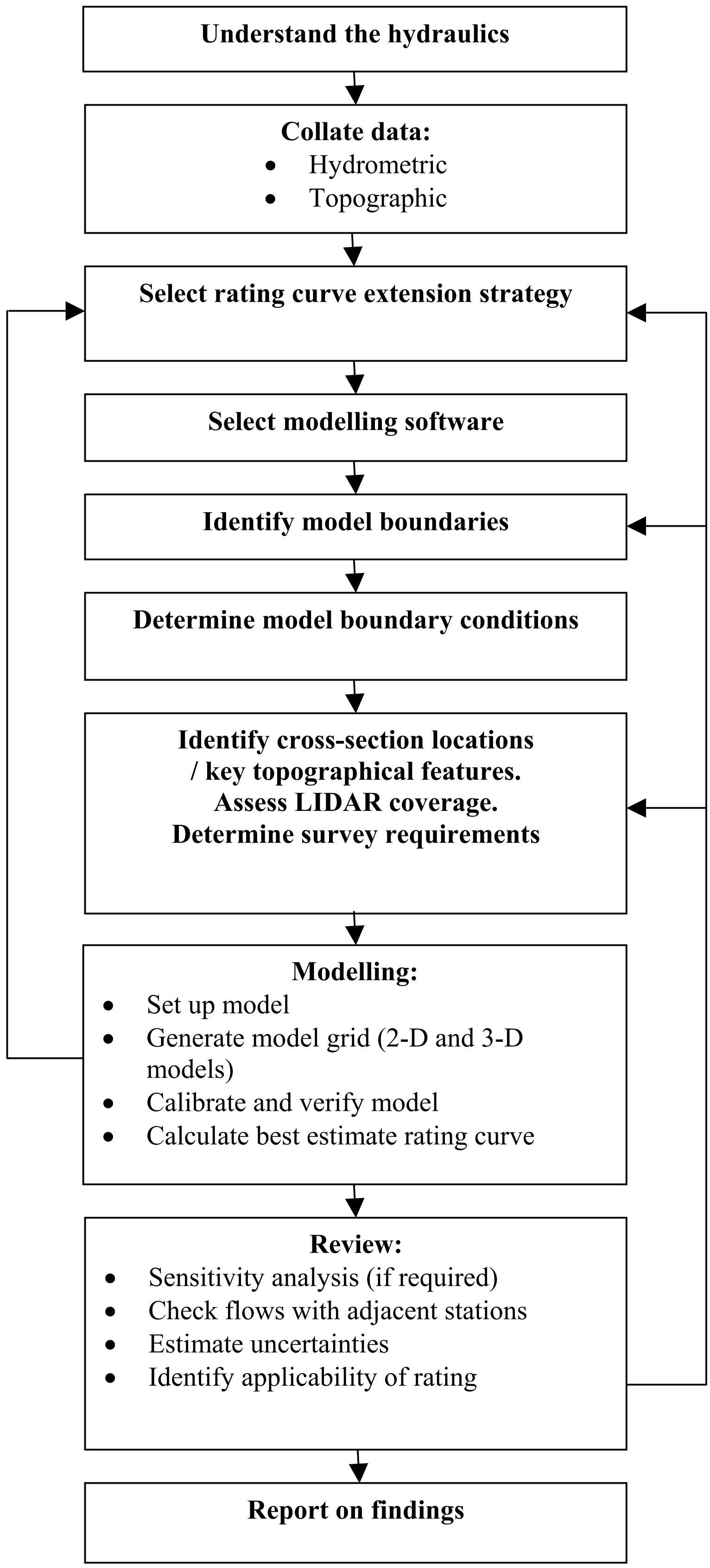

Figure 5.1: Use of computational hydraulic models: Step by step procedure

5.2.4 Obtain and collate relevant topographic data

Topographic data for the site should be identified in consultation with the Agency's Area hydrometric team. Topographic data may be available from several sources in the Agency including the hydrometric team, Section 105 flood mapping team and dedicated survey teams in Areas where they exist. In addition, any other modelling studies which cover the site should be identified.

Decisions regarding the need for new topographic data should be made after the model has been designed (see Section 5.2.10 below).

A brief summary statement of available topographic data should be prepared including the date of survey, where it is located and its quality. The available topographic data should be plotted on a copy of the base plan of the site.

5.2.5 Select rating curve extension strategy

There are a number of possible strategies for rating curve extension using 1-D computational hydraulic models. The most common approaches are:

- Series of steady flows using the steady flow solver if available (i.e. backwater calculation);
- Series of steady flows using the unsteady flow solver. The advantages of this approach compared with the steady flow (backwater) method is discussed in Section 5.3.2; and
- Series of realistic unsteady flows using the unsteady flow solver. This approach should be used where there are significant changes in peak flood flow along the reach.

Guidance on when to use steady and unsteady flow is given in Section 5.3.2.

In addition, the following strategies could be used in particular circumstances:

- Single realistic inflow hydrograph using the unsteady flow solver. This may be applicable where there are no transient effects in the reach and the rising limb rating curve is exactly the same as the falling limb rating curve. In this case the rating curve is derived directly from the predicted stage hydrograph using a single model run. This approach is not suitable where hysteresis occurs and the flow is different on the rising and falling limbs of a hydrograph for the same stage; and
- Single inflow hydrograph which rises in a series of steps. This may be applicable where flow conditions at each step are completely steady throughout the model and there are no transient effects resulting from the transition between steps. In this case the rating can be derived from a single model run.

5.2.6 Select appropriate modelling software

Having decided the modelling strategy, a modelling package is selected that provides the best representation of the identified hydraulics. In many cases there will be little to choose between the available 1-D packages, particularly as they are continually being improved. Particular features to consider include:

- Treatment of steady flow; and
- Whether or not the software includes a good representation of the structures in the modelled reach, particularly standard flow gauging structures.

Guidance on the capabilities of the three BIS packages is given in Section 5.3.3.

As noted previously, there is no BIS software used in the Agency for 2-D and 3-D modelling. Consequently the selection of software should depend on the following:

- Ability to model on an unstructured grid;
- Ability to constrain the computational grid to key topographic features such as embankments and other structures;
- Ability to simulate non-homogeneous roughness so that differences between surface roughness in different areas of the computational domain can be specified;
- Availability of benchmarking information for the chosen software for similar problems; and
- Demonstrability of mass conservation for the problem under investigation.

It should also be noted that the exact calculation procedures vary and this can result in different predictions of water level for exactly the same model input data.

2-D models do not have the ability to simulate structures apart from weirs which are specified as topographic features.

3-D models often do not permit roughness to be specified in a conventional way using Manning's n. Often the roughness is specified as a height or element size analogous to the Colebrook-White parameter k_s although the numeric values are different. Derivation of these values is discussed in Section 5.3.6.

5.2.7 Identify appropriate model boundaries

The extent of the model must now be defined and this is done by deciding where the upstream and downstream model boundaries should be. The extent of the model in 2-D and 3-D modelling is referred to as the computational domain.

The downstream boundary should be located at one of the following:

- A site where a reliable stage discharge curve is available;
- A control structure where a stage discharge curve can be derived; and
- A location which is sufficiently far downstream of the gauging site that any errors in the stage discharge curve at the downstream boundary will not affect the rating. As a guide, this distance should be $0.7D/s$, where D is the bankfull depth and s is the water surface slope. s can be considered to be the bed slope where no water surface data are available.

Ideally the downstream boundary should be at a location where the flow is contained and there is no floodplain flow. However this is often not possible and the downstream boundary must therefore include any adjacent floodplain.

The upstream boundary should be located at one of the following:

- Where flows are contained in the channel, the upstream boundary can be close to the gauging site; or
- Where floodplain flow occurs at the gauging site, the upstream boundary should ideally be located where the flow is contained and there is no floodplain flow. Where this is not possible, the upstream boundary should be located where the river and floodplain flow can be well defined. In Case Study 4, a location was chosen where the floodplain is parallel to the river and the split of flow between the river and floodplain could be defined.

When deciding the locations of model boundaries, the model should be as short as possible consistent with the need to achieve an accurate rating which is not affected by errors in model boundary flow and water level data. This is particularly true for 3-D modelling where considerable computer resources are required to represent variations in the vertical dimension as well as those in plan view.

5.2.8 Determine model boundary conditions

In order to run the model for the first time it is necessary to establish a compatible set of boundary conditions which will enable a model to initialise and run. This process is a matter for the experience and skill of the modeller. The boundary data required for the model will depend upon the flow regime (sub-critical or super-critical) at each boundary and possibly on the numerical methods used in the modelling software.

For the extension of rating curves, the downstream model boundary should consist of a stage-discharge relationship. This may either be a known stage discharge curve at a particular point, a derived stage discharge curve from a control structure, or a normal depth calculation for the river channel and floodplain.

The upstream model boundary requires inflows to be used for the rating extension. This may either be a series of steady flows or a number of separate unsteady flow hydrographs with a range of peak flows. The range of steady flows or unsteady hydrograph peak flows must cover the full range of flows required for extending the rating curve, and the maximum flow should be at least equal to the estimated 1 in 200 year flow at the site.

5.2.9 Identify required cross-section locations and key topographic features

Having decided the model limits, the locations of channel cross-sections, floodplain cross-sections, overflows and storage areas in 1-D models can be decided.

Having decided the extent of the computational domain in 2-D and 3-D models, the locations of key elements of the local topography should be identified in order to apply the appropriate constraints during grid generation. These will include any hydraulic structures, embankments, raised tracks or roads across the floodplain. It is not possible to model bridges or any orifice or gated structure in 2-D models without some sort of special treatment for certain grid cells.

The model layout will depend on the understanding of the hydraulics gained using the guidance in Sections 5.2.2 and 3.2 above, in particular the plan of the site showing estimated flow paths.

The proposed model network or grid should be drafted on the plan. This should include all model sections, structures and other key topographic features for which the survey data are required. The base plan forms an important reference document which clearly shows the model layout. It may also be used to check the model layout and the lengths of the survey sections.

Guidance on the selection of cross-section locations is given in Section 5.3.4.

5.2.10 Identify survey requirements

Existing survey information identified from Section 5.2.4 above should have already been plotted on a copy of the base plan for the site. By overlaying this with the model layout determined in Section 5.2.9, the differences between the required topographical data and available data can be identified. If LIDAR coverage is available, the LIDAR data needed for the study should be identified.

The survey work needed can then be specified. When deciding survey requirements, attention should be given to the age and quality of existing data. If, for example, the river cross-sections are old it is advisable to re-survey them. The new survey should then be overlaid on the old survey to identify changes and obtain an indication of the stability of the channel.

For 2-D and 3-D modelling it is possible to interpolate a grid or co-ordinates using surveyed cross-section data which may be available from a 1-D study of the watercourse. This should be done as a pre-processor prior to commencing the grid generation in the chosen modelling software. For example HEC-RAS has a facility to spatially interpolate geo-referenced cross-sections.

A specification for the required survey should be prepared based on the Environment Agency's standard specification and used for procurement purposes. It is strongly advised that the survey is either carried out by Agency staff or surveyors who are known by the Agency to be competent in river survey work. This is because river survey work is specialised, there is a need for accuracy, and the size of the survey is likely to be relatively small.

Guidance on topographic data and survey requirements is given in Section 5.3.5.

5.2.11 Set up model

The modeller, either a contractor or Agency staff member if the work is to be carried out internally, should carry out the tasks described below.

A 1-D model is constructed using the following components:

- River channel cross-sections;
- Floodplain cross-sections. These may either be extensions of existing sections or separate sections. The floodplains may also include embankments across the floodplains and other features;
- Embankments between the river and floodplains;
- Structure data; and

- Model boundary data.

For 2-D and 3-D modelling, the model grid is constructed using the following components:

- Co-ordinates interpolated from river channel cross-sections;
- Co-ordinates interpolated from floodplain cross-sections. These may either be extensions of existing sections or separate sections. The floodplains may also include embankments across the floodplains and other features;
- Embankments between the river and floodplains;
- Structure data; and
- Model boundary data.

Entry of the data into computer files should be a systematic process carried out with reference to the base map(s) of the model. Methods of checking the data include the following, depending on facilities available:

- Manual checks of data files against original drawings and survey data;
- Plotting of cross-sections;
- Plotting of longitudinal sections showing bed and bank profiles; and
- Plotting the model layout.

Data for all key structures and other control points should be carefully checked by the modeller. The plots can be used to check for abnormalities in levels, channel widths and depths, and location of banks.

In order to run the model it is necessary to select approximate parameter values for the following:

- River channel roughness;
- Floodplain roughness where friction flow is used;
- Embankment weir flow coefficients (1-D models);
- Flow coefficients for floodplain flow where it is represented by weir flow (1-D models);
- Structure discharge and drowning coefficients and modular limits (1-D models);
- Diffusion coefficients (2-D and 3-D models); and
- Parameters dependent on the numerical scheme employed by the model (2-D and 3-D models).

Guidance on the selection of roughness coefficients and structure discharge coefficients for 1-D models are given in Sections 5.3.6 and 5.3.7 respectively. Whilst the values should be similar, it should be noted that the roughness coefficients in 3-D models will not be the same as those used in 1-D or 2-D models due to the fact that internal energy losses caused by turbulent dissipation are represented separately and explicitly in a 3-D model. The effects of surface roughness are however conceptually identical in a 1-D and 3-D model so it could be argued that the Manning's *n* values in Chow for example could be used with more confidence than they could be in a 1-D modelling exercise. Further guidance on this matter is given in Section 5.3.6.

The model should be run for a range of flows to ensure it is consistent with the conceptual understanding of the hydraulics. In addition, a sensitivity analysis should be carried out to demonstrate that boundary conditions and other parameter choices do not affect the results at the gauge.

Deliverables from this step should include the operational model and a brief audit trail describing:

- How the understanding of the hydraulics has been implemented within the modelling software; and
- Choice of parameter values and their possible ranges.

5.2.12 Calibrate model

The purpose of model calibration is to adjust the model parameters to achieve a best-fit to the observed hydrometric data. This should be done for the highest reliable flows on the record of flow gaugings at the site. Once the model is calibrated, it provides an accurate representation of the gauging station for the highest reliable gauged flows. The calibrated model is assumed to provide a reliable basis for extending the rating curve to higher flows, as described in Section 5.2.14.

If the model is to be used to extend the rating curve downwards for low flows, it should be calibrated separately for low flows as there will be a different set of parameter values.

In general, roughness coefficient values should be calibrated whenever observed water level and corresponding flow information are available. When gauged data are not available, roughness coefficient values computed for similar stream conditions or values obtained from experimental data should be used as guides in selecting values of roughness coefficient.

Model calibration includes:

- Adjusting the roughness of the river channel to match observed conditions both at rated sections and also locations elsewhere in the model where water level data exist for known flows. For 3-D models, the effect of roughness should be evident primarily at drowned flows;
- Adjusting the roughness of the floodplains where gauged flow data are available for the floodplains;
- Adjusting the discharge coefficients of gauging structures to match gauged flow data for modular flows, bearing in mind the uncertainty associated with flow gaugings (in 1-D models);
- Adjusting the modular limit and drowning function for structures where reliable calibration data are available for drowned flow conditions (in 1-D models); and.
- Adjusting the diffusion parameter (in 2-D and 3-D models).

Roughness coefficient values are initially estimated in order to run the model before calibration can take place. Roughness coefficient values that are used to extend rating curves should be estimated as follows:

- For rated sections, observed gaugings should be used to calibrate the in-channel roughness coefficient. Where the value of coefficient varies with stage, values for the highest gauged flows should be used;
- The floodplain roughness is normally estimated (using the methods in Appendix A) except in the rare cases where gauged flood flows and levels exist;
- For other sites, for example structures, other level/flow data are needed to calibrate the roughness coefficients. These may include downstream levels at structures (where they are gauged) or other water levels in the vicinity of the gauging site; and
- Where no calibration data are available, the guidelines given in Appendix A should be applied.

Discharge coefficients for structures in 1-D models are initially estimated using standard values for modular flow. The discharge coefficient should then be calibrated for modular flows using actual check gaugings. This should include British Standard Structures, although the reasons for any significant deviations from standard values should be investigated.

In view of the uncertainty associated with gaugings, it is recommended that several gaugings are used. Each gauging should be shown on the stage discharge plot. The stage discharge curve for different discharge coefficients should be plotted to identify the coefficient which gives the best match to the observed data.

Drowned flow conditions are difficult to calibrate because they require water levels upstream and downstream of the structure when check flow gaugings are made. In addition, the headloss is often small and the uncertainty in water level differences relatively large because of turbulence and waves affecting the water level measurement. If good results can be obtained, the modular limit and drowning function in 1-D models should be calibrated using the structure discharge coefficient obtained from the modular flow calibration.

Out of bank flow presents a number of difficulties for model calibration. This is because there is often a lack of gauged flow data for floodplains or embankments. Without this information it is not possible to calibrate floodplain roughness coefficients (or embankment discharge coefficients, modular limit and drowning function in 1-D models).

The deliverables from this step include the calibrated operational model and a brief audit trail statement describing the selection of calibrated parameter values and the calibration performance.

5.2.13 Validate model

The purpose of model validation is to run one or more separate events that were not used for calibration, and demonstrate that the model adequately reproduces the observed data. If the model is to be used to extend the rating curve to high flows, the validation events should be high flow events. The events used for model validation should be gauged by current meter or other suitable flow measurement technique.

The deliverables from this step are an operational model that has been independently verified together with a brief audit trail statement of validation performance

5.2.14 Use model to generate 'best estimate' rating curve

The 'best estimate' rating curve is the best rating curve that can be derived from the modelling and can be expressed as rating equations in an appropriate format for the Agency's flow processing system. In particular:

- The number of segments must not exceed the number permitted by the flow processing system;
- Where segments intersect, the flow should increase with increasing stage; and
- Care is needed to ensure that the 'best estimate' rating extension is continuous with the existing reliable rating.

Once the model has been calibrated and validated, it may be used to produce the rating curve. This is done by running a series of steady flows or unsteady flow hydrographs depending on the modelling strategy chosen. The following data should be recorded:

- Water level at the location of the gauge recorder. This should be the peak water level in the case of unsteady flow; and
- Flow at the location of the gauge recorder, including any by-passing flow. This should be the peak flow in the case of unsteady flow. The flow in unsteady flow modelling will vary along the model.

The flow is plotted against level to produce the rating curve. Where there are seasonal effects caused by changes in vegetation and therefore roughness downstream, different values of downstream roughness should be used and rating curves plotted for different seasonal conditions (for example summer and winter).

The deliverables from this step will be the 'best estimate' rating curve both for the range where gaugings are available and for the extension. A brief audit trail statement should also be provided of how the stage discharge pairs were generated, and their values, and how the rating equation was fitted. The audit trail statement should also include information on the assumptions made during the process so that practitioners can subsequently understand the limitations of the rating.

Section 5.3.8 provides specific additional guidance on the following:

- Extension at a rated section;
- Extension at a standard structure rated by gauging;
- Extension at a non-standard structure rated by gauging;
- By-passing, not in contact with the main channel;
- By-passing, in contact with the main channel;
- Drowning with no downstream or crest tapping measured;
- Drowning due to tidal effects;
- Total drowning of structures;
- Weed growth;
- Low flow extension;
- Looped rating curves; and
- Combinations of these effects.

5.2.15 Carry out sensitivity analysis, if required

Where there are concerns about the accuracy of model parameters because of poor calibration data or any other reason, it is suggested that sensitivity tests are carried out to assess the impact of uncertainties in parameter values.

For example, if the only calibration data available for a rated section are for relatively low flows, the value of channel roughness may be inappropriate for high flows. In such cases sensitivity tests should be carried out using a range of parameter values that covers the likely range of flows that could occur. This will result in the generation of additional rating curve extensions for comparison with the 'best estimate'. This in turn will contribute to the estimate of uncertainty in the rating, discussed in Section 5.2.17 below.

A brief statement should be provided giving the results of sensitivity tests.

5.2.16 Check derived flows with adjacent stations

It is also advisable to check that the flows used for the extended rating curve at the gauging station are consistent with flows elsewhere in the catchment. The purpose of this check is to ensure that:

- The extended rating curve covers the required range of flows; and
- Flows predicted by the extended rating curve for the highest water levels recorded at the gauging site are consistent with flows elsewhere in the catchment.

It must of course be appreciated that extreme event flows at other gauging stations may not be accurate. In addition, there may not be any suitable gauges for comparative purposes, particularly for small catchments. If the derived flows using the extended rating curve are inconsistent with established 'correct' flows, the 'best estimate' rating curve and derivation of flow at other sites should be reviewed to find out why the differences exist. If it is decided that the 'best estimate' rating curve requires amendment, the results of the sensitivity tests referred to in Section 5.2.17 should be used to ensure that any adjustments are hydraulically reasonable.

A brief statement should be provided stating how consistent the derived flows are with flows at upstream and downstream gauging stations, and whether any adjustment was required to the extended rating curve.

5.2.17 Estimate uncertainties in the rating

The sensitivity tests results, consistency checks and any other relevant information should be used to estimate the uncertainty in the rating, and the impact this would have upon derived flows, particularly the flows in the annual maximum series.

A brief statement should be provided of uncertainty in the rating, and the uncertainty in the annual maximum series flows.

5.2.18 Identify applicability of the revised rating

The applicability of the rating should be stated in terms of:

- Maximum and minimum flows and stages; and
- The time period that the rating applies to.

A brief statement should be provided of the maximum and minimum flows and stages that the revised rating applies to, and the time period that the revised rating applies to.

5.2.19 Report on findings

A report should be produced on the work undertaken for the rating curve extension, based upon the statements generated in the previous tasks. The report should consider implementation of the revised rating, and provide enough detail to allow the work to be approved as complete and audited in the future.

The report should be addressed to two audiences: people who will have to implement the revised rating and people who will have to approve the work at its completion, or audit the work in the future.

It must include clear statements of:

- The revised rating, in a form suitable for incorporation in the flow processing system; and
- The uncertainties in flows derived from the extended rating.

In addition, the report should cover:

- The hydraulics of the site;
- The site plan including the model layout; and
- An audit trail including the statements required in Sections 5.2.11 to 5.2.18 above.

5.3 Detailed Guidance

5.3.1 Hydrometric data requirements

In order to use a computational model to extend rating curves, hydrometric data are required from both the gauging station itself and the modelled river reach.

In summary, the main hydrometric data requirements for using 1-D hydraulic models to extend rating curves are:

Essential

- An estimate of the maximum flow to which the rating curve is to be extended. This should normally be at least equal to the estimated 1 in 200 year flow at the site;
- The existing rating curve which is to be extended;
- Flow and stage hydrographs at the gauging station where the rating curve is to be extended, to be used for calibration and validation;
- Current meter gaugings at the gauging station where the rating curve is to be extended, to be used for calibration and validation (essential under some circumstances);
- Water level downstream of gauging structures if drowning is an issue. This is provided by crest tappings in some standard structures, for example the Crump Weir. Where crest tappings are not used, the distance downstream should be far enough away from the structure for the water surface to have fully recovered to the normal downstream river level (essential for reliable drowned flow rating); and
- Measurements of structure operation, for example manual and automatic gate openings or variable crested weir levels as they change with time (essential if structure operation affects the model);

Beneficial, but not essential

- Flow hydrographs or peak flows at the upstream model boundaries to give boundary conditions;
- A rating curve at the downstream model boundary to give a boundary condition;
- Flow and stage hydrographs at other points within the modelled river reach, to be used for calibration and validation;
- Stage hydrographs on the floodplains together with floodplain flow data if available;
- Current meter gaugings at other points within the modelled river reach, to be used for calibration and validation;
- Floodplain velocity estimates at points within the modelled river reach, to be used for calibration and validation;
- Peak water level measurements within the modelled river reach, to be used for calibration and validation, and possibly Manning's *n* estimation; and
- Flooded outlines for out-of-bank calibration.

The key requirement is that sufficient data are provided so that, together with the interpolation rules embedded in the software algorithms, the error between the observed and model hydrographs for all times is sufficiently small so as not to influence decisions made in subsequent stages of the modelling process. These requirements will normally

force a smaller time interval between observations on a tidal boundary than for a fluvial flow boundary.
The main "Calibration data" used for rating extrapolation are data at the gauging site including:

- Gaugings at rated sections and overspills;
- Gaugings at structures where available; and
- For standard structures where gaugings are not available, the rating curve upstream of the structure for modular flow based on the standard structure formula. For drowned flow conditions, water levels are needed upstream and downstream of the structure for calibration purposes.

Other calibration data may be needed to calibrate the model upstream and downstream of the gauging site, for example flows and levels at downstream controls. These consist of levels at specified locations and the corresponding flow, and are particularly important downstream of structures to calibrate the downstream reach. It should be noted that calibration conditions vary according to the condition of the channel, for example between summer and winter.

As overbank flow is one of the main reasons why flood flow ratings at gauging sites are often poor, any overbank flow calibration data would be of great value in improving model results. Unfortunately such data are difficult to obtain and only available for a very few sites.

Facilities to plot hydrometric information are available in the auxiliary software of some packages, and in these cases the data should be entered into data files after an initial inspection of the data has been carried out. The plotted output information may then be used to check the consistency of the data (for example, consistency of water levels along the reach for the same flow).

5.3.2 Steady and unsteady flow

1-D and 2-D computational models may either be run in steady ("backwater") or unsteady flow. 3-D computational models can be run in unsteady flow mode only. In order to simulate steady flow, the model is run with constant boundary conditions of inflow and water level. The main advantage of steady flow is that flow data preparation and running of the model is relatively quick and easy. However in steady flow no account is taken of flood attenuation and storage.

For the vast majority of cases the flow is entirely sub-critical apart from within gauging structures. In this situation a steady flow or discharge hydrograph (unsteady flow) is required at all inflow points including the upstream model limit and other tributary inflows. Outflow points are described by a single water level (steady flow) or by stage hydrographs or rating curves (unsteady flow). The interval between successive values on the hydrographs will normally be regular although some software packages may accept irregular data and interpolate values at fixed intervals.

Steady flow
Steady flow is applied using a single flow for each model run. A rating curve at the gauging site is therefore built up using a series of steady flows. A single water level is

needed at the downstream boundary for each run. Steady flow assumes that the flow does not vary along the reach as a result of flood storage and attenuation.
Steady flow can be modelled using a backwater calculation, or using the unsteady flow solver with constant inputs and outputs to the model. Technically the latter is more correct and generally provides a more accurate representation. The backwater calculation should be considered as a simplified modelling procedure within the software package. These two methods will not give the same results under certain circumstances. For example, when water passes from a river over embankments into a floodplain under steady flow conditions, the water levels can be different on either side of the embankment. This would be represented by steady flow simulated using an unsteady flow solver but not by a backwater calculation.

Steady flow can be modelled in certain 2-D software using a backwater calculation or using the unsteady flow solver with constant inputs and outputs to the model. An example of the backwater approach is the Aquadyn software (the web site for this software is:
http://www.scisoftware.com/products/aquadyn_details/aquadyn_details.html).

It should be noted that both of these two methods are dependent on the initial water levels for a given simulation.

Options for steady flow backwater modelling in HEC-RAS
In the HEC-RAS (without levees) steady flow (backwater) model and the ISIS Flow steady flow (backwater) model, the channel and floodplain at each cross-section in the model is represented as a single, continuous section. Representing a cross-section in this manner allows water across the full width of the section whether or not there are embankments between the river and floodplain. If the floodplain elevation is lower than the elevation of the river bank at a section, these models will assume that flow will occur on the floodplain before the water surface level exceeds the river bank level.

In the HEC-RAS (with levees) steady flow (backwater) option, no flow is assumed on the floodplain until the channel water surface level exceeds the bank level. This approach is a considerable improvement over the other steady flow options. However it does not accurately represent the case where the water level on the floodplain differs from the water level in the main channel.

In addition, in a natural river under flood conditions, flow will discharge from the river channel to the floodplain along the length of the river bank. Consequently, if the elevation of the river bank between surveyed river sections is lower or higher than the bank level at the adjacent surveyed sections, the representation of flow into the floodplains using the HEC-RAS (with levees) option will be inaccurate.

The unsteady flow models have levels along the full length of the embankments, and are therefore more accurate in this respect.

Unsteady flow
Unsteady flow is applied using a flow hydrograph (plot of flow against time) for a range of floods in order to build up the rating curve at the gauging station. A rating curve is needed at the downstream boundary to enable the downstream level to be computed under varying flow conditions.

Unsteady flow permits:

- Changes in flow along the reach caused by storage and attenuation; and
- Other time-varying effects including tides and gate operation.

When to use steady or unsteady flow
Generally unsteady flow should be used in the following cases:

- Where bypassing of the gauging site occurs and there are embankments or high ground between the channel and the floodplains (see Case Study 1). In this case, steady flow using the unsteady flow solver could be applied if the hydrograph peak flows does not change significantly along the reach;
- Where flood storage has a significant impact on the rating;
- Where the channel network is complex and may distort the flood hydrograph shape;
- Where the channel/floodplain interaction is complex and may distort the flood hydrograph shape;
- Where tidal influence is important; and
- Where the effects of gate operation are significant.

In all other cases, steady flow can be used.

5.3.3 How to choose 1-D modelling software

The three BIS software packages for 1-D modelling can all undertake both steady and unsteady flow simulations, and can all be used for the extension of rating curves. The HEC-RAS (with levees) steady flow (backwater) option differs from the other steady flow options, as described in Section 5.2.1 above. However, as noted in the same section, in some cases it is advisable to use steady flow with the unsteady flow solver.

There are a number of differences between the software which are summarised in Tables 5.6 to 5.11. As all three software packages are constantly being updated this situation may change. It should also be noted that the exact calculation procedures vary and this can result in different predictions of water level for exactly the same model input data.

Roughness formulae
The capabilities for incorporating roughness of the individual software are given in Table 5.6 below:

Table 5.6: Roughness formulae used by 1-D computational models

Software	Capability	Stage-Discharge Calculation Method
HEC-RAS	Manning's n	Lateral variation in n value. Vertical variation in n value based on either stage or discharge Seasonal variation in roughness
ISIS Flow	Manning's n Nikuradse k_s	Lateral variation in n or k_s value.
MIKE 11	Manning's n Manning's M ($M = 1/n$) Chezy C	Lateral variation in n, M or C. Vertical variation in n, M or C based on stage.

Standard structures
This section summarises the capabilities of the software to model common types of structure that are found within a river network and the method used by the software to analyse flow at the structure (Tables 5.7 to 5.10). The software generally uses values of discharge coefficients obtained from standard structures or standard hydraulic theory, which the user may change as part of the calibration process.

Table 5.7: Weir types used by 1-D computational models

Software	Capability	Stage-Discharge Calculation Method
HEC-RAS	Broad crested Ogee (see Note 1) Lateral weir	Standard theory
ISIS Flow	Round nosed broad crested Crump Sharp crested General purpose (see Note 2) Gated Syphon spillway	Standard theory
	Spill unit/Floodplain Section	User defined Q-H relationships
	Notional	Allows modular limit to be changed
MIKE 11	Broad crested Special (see Note 3) Weir formula (see Note 4)	Standard theory and User defined Q-H relationships

Notes:
1 Ogee weirs have a crest that is S-shaped in profile. The upper curve ordinarily conforms closely to the profile of the lower nappe of a ventilated sheet of water falling from a sharp crested weir.
2 The General purpose weir in ISIS models a broad crested weir with a rectangular throat. By amending the input coefficients, it is possible to model weirs with a parabolic or triangular control section.
3 The Special weir in MIKE11 uses User defined Q-H relationships to represent the weir equation.

4 The Weir formula in MIKE11 provides a choice of a Standard weir formula or the Honma weir formula. For the standard weir case, values are entered for the weir crest width, weir crest height, weir coefficient, exponential coefficient in the weir formula and a datum level to which the weir height refers. For the Honma weir values are entered for the multiplication coefficient in the Honma weir formula, weir crest width and the weir crest level. This type of weir is not used in the UK.

Table 5.8: Gated control structure types used by 1-D computational models

Software	Capability	Stage-Discharge Calculation Method
HEC-RAS	Vertical gates Radial gates	Standard theory
ISIS Flow	Vertical gates Radial gates	Standard theory
MIKE 11	Overflow gates Underflow gates Discharge (corresponds to a pump) Radial gates	Standard theory

Table 5.9: Culvert types used by 1-D computational models

Software	Capability	Stage-Discharge Calculation Method
HEC-RAS	Rectangular Circular Elliptical Arch Pipe arch Semi-circular Low profile arch High profile arch	Based on: Normann, 1985
ISIS Flow	Rectangular Circular Full arch Sprung arch Symmetrical arch	Based on: Ramsbottom et al, 1997.
MIKE 11	Rectangular Circular Irregular, defined by level-width table Irregular, defined by depth-width table Cross-section	Standard theory based on: Contraction, expansion, friction and bend loss coefficients and a critical flow correction coefficient.

Table 5.10: Bridge types used by 1-D computational models

Software	Capability	Stage-Discharge Calculation Method
HEC-RAS	FHWA/USBPR method for modern bridges	Based on: Bradley, 1978.
ISIS Flow	FHWA/USBPR method for modern bridges	Based on: Bradley, 1978
	HR Wallingford method for arch bridges	Brown, 1989
MIKE 11	FHWA/USBPR method for modern bridges	Based on: Bradley, 1978
	HR Wallingford method for arch bridges	Brown, 1989
	Biery and Delleur method for arch bridges	Biery and Delleur, 1962.

Non-standard structures

There are many non-standard structures in river systems including:

- Non-standard shapes (weirs, bridges, etc);
- Locations which affect the performance of structures, for example structures which are downstream of bends where the approach flows are uneven and affected by eddies; and
- Flood embankments and walls, which not only behave as non-standard weirs but also often behave as side weirs, where the head varies along the crest.

When modelling non-standard structures, the modeller has to assess appropriate discharge coefficients. In order to assist the user in the selection of coefficients, the theoretical background to the analytical approach adopted within each of the software is given in key references listed at the end of this section. It is advised that users consider carefully the values of coefficients that they use to ensure that they are realistic. To assist the user to model non-standard structures, the software provides the facility to vary discharge coefficients and modular limits, as indicated in Table 5.11.

Table 5.11: Formulae for modelling non-standard structures

Software	Capability	Stage-Discharge Calculation Method
HEC-RAS	Contraction/Expansion	Head Loss Coefficients
ISIS Flow	Bernoulli Loss	Bernoulli Equation
	General Head Loss	Head Loss Coefficients
MIKE 11	Energy Loss	Head Loss Coefficients

Key references
There is a separate section in the references that provide background on the hydraulics of structures and their discharge formulae under modular and drowned flow conditions, as referred to in the above tables.

5.3.4 Cross-sections locations (and computational mesh for 2-D and 3-D models)

Locations of river channel cross-sections
The issues that influence the selection of section location differ for river channels and floodplains. For the river channel, sections are normally needed at the following points:

- Upstream and downstream of all hydraulic structures, particularly gauging structures but also bridges, culverts, weirs and gated control structures where relevant;
- At sites where flow or level are monitored including rated sections at gauging sites;
- Model boundaries;
- Upstream and downstream of confluences and bifurcations;
- Locations where flood embankments begin and end;
- Any other locations of interest; and
- Convenient regular intervals along the channel using the spacing guidelines given below.

In practice the spacing of cross-sections for the relatively small models needed for gauging sites is often dictated by structures, measurement points, and other features where cross-sections are required.

In addition, it is good practice in grid generation for 2-D and 3-D models to ensure that variations of grid cell size between adjacent cells are limited to at most a factor 2 or 3. Large aspect ratio cells (where the ratios of cell width to cell length are very small or very large) are also to be avoided where possible. For example, the 3-D model in Case Study 7 used a grid size of 0.97m in the vertical, 1.1m in the lateral direction and 1.02m in the streamwise direction producing a total of 174,000 grid cells.
Were there are long lengths of channel between features where cross-sections are required, the following guidelines should be adopted in determining the longitudinal spacing of channel cross-sections:

- Cross-sections should generally not be more than $20B_t$ apart, where B_t is the top width of the river channel;
- Sections should generally not be more than $1/(2s)$ apart, where s is the mean slope of the river; and
- Sections should generally not be more than $0.2D/s$ apart where D is the bankfull depth.

Locations of floodplain cross-sections and grid cells
The discretisation of the floodplain will depend upon several factors including:

- The location of channel cross-sections;
- The estimated direction of flow on the floodplains for 1-D models;
- The presence or otherwise of flood defence embankments parallel to the river;

- The location of head-losses on the floodplain (e.g. roads, tracks, fences, hedges, cross-embankments, buildings etc); and
- The plan form of the river channel and the flood limits.

The situation is complicated by the fact that flow direction can change with stage, particularly in a meandering river. When locating floodplain boundaries and grid points, the full width and variation in floodplain width must be captured by the model to avoid errors caused by incorrect mass balance.

Additional guidelines for locating floodplain cross-sections in 1-D models are as follows:

- Estimated flow lines should be plotted on the base plan based on the known flood limit and site observations of where active floodplain flow is likely to occur, as described in Section 5.2.2. Cross-sections should be plotted which are perpendicular to the flow direction. They may be crooked or curved but one cross-section cannot intersect another; and
- Areas of floodplain where there is active flow and areas which only provide flood storage should be separated. This is done by the application of "conveyance pointers", which are points in the cross-section defined by the modeller that specify the limit of the cross-section where active flow occurs. The remainder of the cross-section is only used in the flood storage calculations. This can have a significant impact on predicted flood levels for a particular flow as it changes the cross-sectional area of flow in the hydraulic calculation. It is recommended that the base plan referred to above shows estimated boundaries between active flow areas and storage areas to guide the modeller on where to use conveyance pointers.

5.3.5 Topographic data requirements

Topographic data may be obtained from existing sources or from new surveys. Existing sources include river survey sections, contoured floodplain maps and drawings of structures. Whilst there are clearly cost and time savings in using existing information, great care must be taken to ensure that it is accurate. Particular problems include the following:

- Date of existing surveys. The shape and condition of river channels change with time, and check surveys should be carried out to identify if changes have occurred;
- Developments on the floodplains that have occurred since floodplain mapping was carried out;
- Changes in datum levels through time, rendering surveys incompatible; and
- Inaccurate floodplain levels based on aerial surveys.

It is quite acceptable in most cases to make use of existing cross-section data in conjunction with scattered data interpolation utilities to generate intermediate co-ordinates in 2-D and 3-D modelling.

Adequate definition of topographical and other features is essential if the model is to produce reliable results. Whilst not a full specification, the following sections give an indication of the level of survey detail required.

River channels

- Change in bed level between adjacent survey points in a cross-section not to exceed 0.50m; and
- Lateral spacing between survey points in a cross-section 2m to 3m for rivers up to 40m wide, 3m to 4m for rivers up to 60m wide and 4m for wider rivers. Effectively this is to make the number of survey points a function of the channel width in order to avoid requiring a very large number of points for wider rivers. Closer spacing is required to define local detail such as a river bank, with points taken at bank tops, the bottom of the banks and any other points where there is a significant change of slope.

Floodplains

- Change in ground level between survey points not to exceed 0.50m (0.25m on wide flat floodplains);
- Horizontal distance between survey points between 2m and 50m (up to 100m on wide flat floodplains);
- Points taken at changes of lateral slope; and
- Features affecting flood flows to be clearly marked on maps and dimensioned.

Embankments

- Change in level between survey points not to exceed 0.25m; and
- Horizontal distance between survey points not to exceed 50m.

Structures

- All leading dimensions and levels to be recorded including invert levels, soffit levels, widths, opening shapes, etc; and
- Sketches of the structure site, showing all features that might affect the flow.

When the data have been assembled a check for consistency of levels and dimensions should be made.

All surveying work should be carried out in accordance with the current version of the Agency's 'Best Practice Manual and Standard Specification for Surveying Services'. If this is not readily available then it can be obtained either from the Region's representative on the Agency's National Surveying User Group, or from the Thames Region Survey Group.

Where over-bank flow and floodplain storage are considered to influence the rating curve it is important that:

- A riverbank survey is undertaken, along the line of highest level between river channel and floodplain, to allow accurate simulation of flow from channel to floodplain; and
- The floodplain survey provides sufficient information to determine the floodplain stage-volume relationship.

5.3.6 Roughness coefficient selection

Estimates of flow resistance are required for both the river channel and floodplains. This involves selecting values of roughness coefficient for the particular resistance equation used by the model.

Selection of an appropriate value of roughness coefficient has a significant impact on the accuracy of computed water levels. The value of roughness coefficient is highly variable and depends on a number of factors including: surface roughness; vegetation; channel irregularities; channel alignment; scour and deposition; obstructions; size and shape of channel; stage and discharge; seasonal change; temperature; suspended material and bedload.

Manning's *n* is most commonly used in the UK, and this can be used directly in 1-D and 2-D models. Values of Manning's *n* roughness coefficient for a river channel or floodplain may be obtained from the following:

River channels
- Locally derived values from measurements;
- By comparison with national values, for example the Flood Discharge Assessment Engineering Guide which contains some values from the Midlands Region of the Agency;
- Chow Guidelines, 1959, reproduced in Appendix A, Table A.1;
- Chow photographs (Chow 1959, referenced in Section 4.3.4);
- The SCS method, reproduced in Appendix A, Table A.2 (SCS, 1963). In this method, a basic roughness coefficient, for a uniform, straight and regular channel, is adjusted to take account of the effects of surface irregularities, shape and size of channel cross-section, obstructions, vegetation, and degree of meandering of channel; and
- For steep streams with stable bed and bank materials in gravel, cobbles or boulders the relationship developed by Jarrett (1984) and given in Appendix A is recommended for estimating the Manning's 'n' coefficient.

Floodplains
- Locally derived values from measurements, although these are rarely available;
- Chow Guidelines, 1959, reproduced in Appendix A, Table A.1; and
- Roughness coefficients for grassed floodplains and floodplains with hedges for a range of flow depths and in the hedge case for clean (i.e. without debris) or dirty hedges (i.e. with debris), reproduced in Appendix A, Tables A.3 and A.4 (Klaassen and Van der Zwaard, 1974).

The following points should be noted:

- Locally derived values should always be used in preference to generalised estimates where available. Local data must of course be used to calibrate the model;
- Where no local data are available, estimates should be based upon several different generalised estimate methods, and best estimate and upper and lower bounds should be identified;

- If values of the roughness coefficient for a river channel is estimated using the approach given by Chow, and the channel is sinuous in nature, then the roughness coefficient will require adjustment based upon the degree of meandering of the channel; and
- Values of the Manning's *n* coefficient vary with stage, generally reducing as the stage increases.

In the case of 3-D models, roughness depends only on surface roughness, which may embody variations due to vegetation, scour, deposition, suspended material and bedload.

For example, FLOW-3D uses a parameter called *ks* to represent surface roughness. The following text is an extract from the FLOW-3D manual regarding *ks*

'*Wall shear stresses at obstacle surfaces can be modified by defining a wall roughness. The roughness has the dimensions of length. The length is, in some sense, proportional to the size of the roughness elements. It is incorporated into the usual shear stress calculations by adding to the molecular viscosity the product of fluid_density*roughness *relative_velocity, where relative_velocity means the difference between the local fluid velocity and the wall velocity.*

In this implementation the wall shear stress in laminar flow is equal to

rho(nu +k u) u/y,

where k is the roughness. When the roughness is large enough, the stress is proportional to

$rho*u^2 (k/y)$.'

It appears that the *ks* value is analogous to the Colebrook–White roughness height in that both have dimensions of length. The precise relationship between *ks* and Manning's *n* or indeed Colebrook-White k_s is not defined in the literature. Where possible, therefore, *ks* values should be obtained by calibration. However, values of Manning's *n* are used in order to make a first approximation of *ks*.

Once a Manning's *n* has been estimated, for most cases it will need to be converted to a roughness height for use in 3-D models. This can be estimated by using the conventional representation between Manning's *n* and Colebrook White k_s but more work is necessary to identify relationships for specific software packages. The relationship between Manning's *n* and Colebrook White k_s is as follows:

$$k_s = (n/0.038)^6$$

This relationship is applicable in the range of relative roughness (R/k_s) of:

$$7 \leq (R/k_s) \leq 130$$

Roughness values will be refined during model calibration, where model predictions are matched with observations.

5.3.7 Structure coefficients in 1-D models

The software contains default values for discharge coefficients, modular limits and drowning functions. These are based on standard theory and standard structures, particularly standard flow measurement weirs. Background on values for standard structures is given in Ackers *et al*, 1978.

These default values should be used initially for in-channel structures when the model is constructed unless the modeller is aware of a reason why these values do not apply in a particular case. In practice many gauging structures are non-standard or there are features of their installation which affect the discharge coefficients. In these cases the discharge coefficients will not be standard values.

As far as possible, values of discharge coefficient, modular limit and the drowning factor formula should be based on calibration using gauged flows. However this is particularly difficult for drowned flows which are common during large floods, because the downstream level is often not measured. The situation is further complicated where groups of structures occur, for example a combination of weirs and sluices which may be at various angles to the approach flow. Even the values for British Standard structures may change if conditions at the structure are not "ideal", for example poor approach conditions or sedimentation upstream of the structure.

Structure coefficient values are also needed for embankments and floodplain cross-sections represented by weirs. In the HEC-RAS and ISIS Flow unsteady models, the bank line of the river channel or embankments are represented as a sequence of weirs. The floodplain is divided into discrete areas which are linked to the weirs discharging flow from the river channel. They are also inter-linked to represent the discharge along the floodplain. The form of the links between the floodplain areas may either be by weirs, which try to represent natural features such as hedges and tracks, or by friction flow.

In representing the bank line and links between the floodplain areas as weirs, thought must to be given to the Coefficient of Discharge (C_d) applied to each weir. A section of bank of a river channel operating as a weir under flood flow conditions may be normal, skewed or parallel to the flow line and perform either in the manner of an in-line weir, an angled weir or a side weir. Floodplain sections linking floodplain areas, will perform as in-line weirs with a low profile.

Consequently, the bank of a river channel or a floodplain section will not be as efficient in discharging flow as a weir structure constructed according to British or International Standards and a lower coefficient of discharge than for a standard weir may be applicable. Experience suggests that discharge coefficient values will be lower for a weir simulating flow over embankments and natural ground, as compared to a man-made structure. Discharge coefficient values in the range 0.1 to 0.2 are suggested instead of the 0.5 to 0.7 which normally applies to man-made structures. With regard to drowning of embankments, the software normally applies default values which are based on model stability considerations.

Care is needed if weirs are used to represent flow between the floodplain areas. Coefficients should be selected which not only take account of discrete head losses at boundaries but also friction losses along the floodplain. Inappropriate selection of coefficients can result in too much flow on the floodplains, effectively bypassing the gauging site.

It should be remembered that the accuracy of coefficient values for long spills are not very critical. This is because a small increase in upstream water level will produce a large change in flow and therefore any error in coefficient values will only have a small impact on water levels.

5.3.8 Dealing with specific problems

Extension at a rated section
A rated section should be represented in a 1-D computational model by a channel cross-section, and in a 2-D or 3-D model by grid points across the section. The model should be calibrated to match the gaugings at the rated section. The calibrated model can then be used to extend the rating by applying a range of flows as described in Section 5.2.14.

Steady flow is normally suitable for simple rated sections unless any of the cases listed in Section 5.3.2 apply.

Extension at a standard structure rated by gauging
A standard structure should be represented in a 1-D computational model by an appropriate structure module, and in a 2-D or 3-D model by an accurate topographic specification.. The upstream limit of the structure should be at the location of the water level recorder used to provide flow data. The downstream limit varies for different structures, but it should be far enough downstream for the water surface to have fully recovered to the normal downstream river level.

In a 1-D model, a cross-section should be located at the point where the upstream water level is recorded. The model should initially be run using the default coefficients for the relevant standard structure. The structure discharge coefficient should then be calibrated for modular flows by matching water levels at the upstream water level recorder with the gauged flows. Where there are significant differences compared with standard values, the cause should be investigated.

In 2-D and 3-D models, grid points should be located at the point where the site is gauged. In almost all 2-D and 3-D models known to the authors, the only tool available to achieve calibration is to vary local roughness. The model should be calibrated for modular flow by varying roughness coefficients to match water levels at the upstream water level recorder.

The model should then be run for flows that cause drowning at the structure. The crucial factor in obtaining an accurate drowned flow rating curve is to obtain an accurate rating downstream of the structure. Ideally the downstream channel should be calibrated against water levels recorded downstream of the structure by adjusting channel roughness values. If these are not available there will be a high uncertainty in the rating.

If downstream water levels are available, these together with the upstream levels and gauged flows can be used to calibrate the modular limit and drowning function in 1-D models and local roughness in 2-D and 3-D models. If no downstream water levels are available, it is suggested that the standard modular limit and drowning function are used in 1-D modelling, and the downstream stage discharge curve is adjusted by changing values of the channel roughness coefficient until an acceptable calibration is achieved.

Once the model is calibrated, it may be run for higher flows to extend the rating curve as described in Section 5.2.14. Steady flow is normally suitable for structures unless any of the cases listed in Section 5.3.2 apply. At higher stages the control may change from the structure to the channel.

Extension at a non-standard structure rated by gauging
For 1-D modelling, the structure module in the software that most closely represents the non-standard structure should be used to extend the rating curve. 2-D and 3-D models treat all structures identically so the same approach applies to non-standard structures as for standard ones. The upstream limit of the structure should be at the location of the water level recorder used to provide flow data. The downstream limit should be far enough downstream for the water surface to have fully recovered to the normal downstream river level.

In a 1–D model, a cross-section should be located at the point where the upstream water level is recorded. The model should initially be run using default values of coefficients or values selected by the user based on experience. The structure discharge coefficient should then be calibrated for modular flows by matching water levels at the upstream water level recorder with the gauged flows.

In 2-D and 3-D models, grid points should be located at the point where the site is gauged. In almost all 2-D and 3-D models known to the authors, the only tool available to achieve calibration is to vary local roughness. The model should be calibrated for modular flow by varying roughness coefficients to match water levels at the upstream water level recorder.

For modular flow, where there is a discontinuity in water level, the Shallow Water Equations in 2-D models do not accurately represent the hydraulics as the assumptions involved in their derivation are violated in this case. In 3-D models, the Navier-Stokes Equations should accurately represent the hydraulics.

The model should then be run for flows that cause drowning at the structure. The crucial factor in obtaining an accurate drowned flow rating curve is to obtain an accurate rating downstream of the structure. Ideally the downstream channel should be calibrated against water levels recorded downstream of the structure. If these are not available there will be a high uncertainty in the rating.

If downstream water levels are available, these together with the upstream levels and gauged flows can be used to calibrate the modular limit and drowning function in 1-D models and local roughness in 2-D and 3-D models. If no downstream water levels are available, it is suggested that the modular limit and drowning function in 1-D models are initially calibrated using the downstream stage discharge curve predicted by the model. If the resulting modular limit and drowning function look unrealistic, there is scope to adjust the downstream channel roughness and re-calibrate the drowning parameters.

Once the model is calibrated, it may be run for higher flows to extend the rating curve as described in Section 5.2.14. Steady flow is normally suitable for structures unless any of the cases listed in Section 5.3.2 apply. At higher stages the control may change from the structure to the channel.

By-passing, not in contact with the main channel
By-passing which is not in contact with the main channel should be represented in 1-D models by separate floodplain cells which are parallel to the main channel. The initial hydraulic analysis will be important in deciding the areas of active flow on the floodplains and the flow direction. This information is needed to decide cross-section locations and alignments, and the limits of active flow paths in the model. The high ground between the river channel and the floodplain is represented by a series of weirs which allow flow between the channel and floodplain if the water level is high enough.

In 2-D and 3-D models, the high ground between the river channel and the floodplain is represented by a series of initially dry grid cells which could allow flow between the channel and floodplain if the water level is high enough.

Care is needed in the selection of model boundaries to ensure that the distribution of flow between the channel and floodplains at the site is realistic. Guidance is given in Section 5.2.7.

Calibration of the floodplain flow is often difficult because floodplain flow gaugings are seldom available. Where floodplain flow forms a significant part of the overall flow, it is particularly important to correlate the total flow with upstream and downstream stations as outlined in Section 5.2.16.

Once the model is calibrated, it may be run for higher flows to extend the rating curve as described in Section 5.2.14. Unsteady flow should generally be used in this case although steady flow using the unsteady flow solver could be applied if the hydrograph peak flow does not change significantly along the reach.

By-passing, in contact with the main channel
In 1-D models, bypassing which is in contact with the main channel should be represented by extended cross-sections covering the river channel and the floodplains. Different values of roughness coefficients will be needed for the main channel and floodplain.

The greatest difficulty with this case is the impact of interference between the fast moving channel flow and the slower moving floodplain flows. This is particularly important at low depths of flow on the floodplain, but becomes less significant at higher stages. This effect is not included in current 1-D and 2-D computational models but it is included in 3-D models. Alternative methods are available for this situation based on recent research, but a separate calculation would be needed to apply them (Ackers method for straight channels and James & Wark method for meandering channels, NRA 1994). The results of the separate calculation would then have to be fed back into the computational model by artificial adjustment of roughness values for river channel and floodplain segments.

This approach of undertaking separate calculations and feeding back the results is not recommended. Where the interference is considered to be important (generally where the floodplain flow depth is large and the river is narrow), either 3-D computational or physical modelling should be used.

Calibration of the floodplain flow is often difficult because floodplain flow gaugings are seldom available. Where floodplain flow forms a significant part of the overall flow, it is particularly important to correlate the total flow with upstream and downstream stations as outlined in Section 5.2.16.

Once the model is calibrated, it may be run for higher flows to extend the rating curve as described in Section 5.2.14. Steady flow is normally suitable unless any of the cases listed in Section 5.3.2 apply.

Drowning with no downstream water level or crest tapping

The structure should initially be calibrated for modular flows to determine the discharge coefficient in 1-D models or local roughness in 2-D and 3-D models. An accurate downstream rating is the most important factor in the determination of a structure rating curve under drowned flow conditions. Where these data do not exist, it is strongly recommended that downstream water levels are recorded for medium and, if possible, high flows to calibrate the downstream reach. The model should then be run for flows that cause drowning at the structure.

For 1-D models, and two approaches are suggested for calibrating the model:

- For standard structures, the standard modular limit and drowning function should be used, and the downstream stage discharge curve adjusted until an acceptable calibration is achieved with gauged flows in the drowned flow range. If there are no calibration data there will be high uncertainty in the rating; and
- For non-standard structures, the modular limit and drowning function should initially be calibrated using the downstream stage discharge curve predicted by the model. If the resulting modular limit and drowning function look unrealistic, there is scope to adjust the downstream channel roughness and re-calibrate the drowning parameters until a realistic balance is achieved between downstream channels roughness and structure drowning parameters.

In the case of 2-D and 3-D models, the local roughness varied to obtain a value which represents the drowned flow case in general. It should be noted that the main advantage of 2-D modelling in this case is the more accurate simulation of drowned flow.

Drowning due to tidal effects

Where drowning occurs due to tidal effects, the structure has a non-unique rating curve and it is essential to know the downstream water levels in these circumstances.

During the hydraulic analysis outlined in Section 5.2.2, the impact of tides on the rating curve should be assessed. For example, in some cases tidal effects may only affect low flows and there may be a unique high flow rating. In such cases the standard procedure could be followed to produce a single rating curve. Sensitivity tests could then be undertaken to determine the lower limit of the rating.

However in the general case it would be necessary to know both the upstream and downstream water level for model calibration. A family of rating curves would then be developed for a range of downstream water levels. When the rating curve is applied, the downstream water level must be measured to decide which of the family of rating curves should be used to estimate the discharge. The curve corresponding to the observed downstream water level would be used with the observed upstream water level to estimate the flow. As the downstream level in tidal situations is constantly changing, and instantaneous measurement of both upstream and downstream water level would be needed.

This procedure will also apply to other cases where there is a non-unique rating curve, for example a structure affected by flows at a confluence downstream.

Total drowning of structures

When total drowning of structures occurs, the structure no longer controls water levels in the river. The control will transfer to the river channel which will become a rated section. Calibration of the site will therefore be carried out in three stages for 1-D models, as follows:

- Modular flows, to determine the structure discharge coefficient;
- Drowned flows, to determine the structure drowning parameters; and.
- Totally drowned flows, to determine channel roughness coefficients.

In practice it may not be possible to obtain calibration data for all of these cases. If it is not possible to calibrate the totally drowned part of the rating curve, the drowning function of the structure should be such that there is no significant head loss caused by the structure at stages where total drowning is known to occur.

For 2-D and 3-D models, calibration of each stage should be achieved by varying local roughness. 3-D modelling is the hydraulically most correct method, and the roughness in the three above flow states should not vary significantly.

Weed growth

Weed growth can affect both rated sections and structures. Sites affected by weed growth will have a non-unique rating curve. During the hydraulic analysis outlined in Section 5.2.2, the significance of weed growth on the rating curve should be assessed. If the impact is small, it may be acceptable to have a single rating curve with a greater degree of uncertainty than would otherwise be the case.

Otherwise a family of rating curves will be needed, depending on the condition of the downstream channel. Ideally the downstream channel should be calibrated under a range of conditions to determine roughness parameters for different amounts of weed growth. In the absence of calibration data, roughness coefficients should be determined using recent research on the resistance of vegetation (Fisher 2002).

Each of the family of rating curves would then be extended using the model with the appropriate roughness coefficients. The timing and amount of vegetation growth varies from season to season, and it is suggested that each rating curve is linked to a photograph of the downstream vegetation rather than a specific time of year. This will of course require site visits to determine which rating curve should be in use at different times.

A more reliable approach would be to measure both upstream and downstream water levels, and develop a family curves for a range of values of downstream level in the same way as for tidally influenced gauging stations.

Low flow extension
In principle, 1-D computational models can be used for low flow extension, although the model will require re-calibration for low flows. In the case of rated sections, the channel roughness values should be calibrated for the lowest gaugings. The user should also be aware that the value of Manning's *n* will increase significantly at low depths of flow, and the k_s roughness length is more appropriate under these conditions.

With regard to structures, the discharge coefficient should not change at low flows but calibration against gauged flows is advisable. One problem is that the uncertainty on low flow gaugings (where the velocity is low) can be large and several gaugings should be used.

The user should be aware that models are often unstable at low depths of flow. The use of slots in the river bed is an example of an artificial method of improving the stability of models under these conditions.

In principle, 2-D and 3-D computational models can be used for low flow extension but this is not recommended as other methods are likely to be more cost-effective.

Looped rating curves
Under certain circumstances, rating curves for a particular flood event can be looped. This well known phenomena is caused by changes in the water surface slope as the flood wave passes downstream. Thus the flow for a particular stage on the rising limb can differ from the flow for the same stage on the falling limb.

Peak flood flows obtained from a rating curve are normally from the highest recorded water level during an event. Where a gauging site is affected by hysteresis, the rating curve should be based on the flow corresponding to the peak water level of the hydrograph. It is recognised that this will not always be the maximum flow and therefore a degree of uncertainty will be introduced. If flows are required at stages other than the peak water level in a hydrograph, supplementary curves for the rising and falling limbs should be prepared. If these are to be used, the rating curve algorithm must be able to recognise whether the stage is rising or falling when calculating the flow.

Combinations of these effects
Where combinations of the above effects occur, they should be identified in the hydraulic analysis required in Section 5.2.2 and Section 3.2. A strategy for preparing the rating curve extension can then be developed by combining the appropriate measures in the sub-sections above.

5.4 Case Studies

5.4.1 General approach

Case studies for four sites are included in order to demonstrate the application of the computational hydraulic modelling to the extension of rating curves, to provide worked examples, and build confidence in the modelling approach. The case study sites are listed in Sections 5.4.2 below. The general approach is to apply the step by step procedure to each of the case study sites. In the case of 1-D modelling, more than one of the BIS models have been used in order to provide some comparison of the models and modelling approaches. The approach also includes comparing the results with gauged flows, which leads to recommendations for where computational modelling should be used, and how the models should be applied. These recommendations are based primarily on hydraulic considerations. Other factors including data availability, ease of application and cost should also be taken into account in rating extension work.

5.4.2 Description of case studies

Details of the case studies are given in Table 5.12

Table 5.12: Case studies

Case Study	Site Characteristic	Flow Characteristic	Model approach	Modelling Software
4	Rated open channel of compound section	Bypass flow on floodplain which is separated from the main channel	1-D	HEC-RAS Steady HEC-RAS Unsteady ISIS Flow Steady ISIS Flow Unsteady
5	Non-standard compound broad crested weir in open channel of simple section	Rating discontinuity	1-D	HEC-RAS Steady ISIS Flow Steady MIKE 11 Steady
6	Compound broad crested weir	Bypass flow on floodplain which is not separated from the main channel	2-D	TELEMAC
7	Non-standard compound broad crested weir in open channel of simple section	High flows cause drowning of the weir. All flows are within the channel embankments.	3-D	FLOW-3D

5.4.3 Case Study 4: 1-D model of open channel rated section with bypass flow on floodplain

Understand the hydraulics
The open channel, located on a lowland river, has a main channel width ranging between approximately 30 to 60m. The layout of the site is shown on Figure 5.2. The floodplain flow patterns have been estimated from site inspection and from information provided by the Environment Agency, and these are also shown on Figure 5.2 together with the model layout.

The river channel, sinuous in plan, runs mainly along the left side of the floodplain with the floodplain width ranging between 200 to 400m. The elevation of the floodplain is, in general, below the elevation of the top of the right bank of the river channel. The rated section is sited approximately 50m downstream of a sharp right hand bend. During major flood events, flow discharges over the right bank of the river channel for a distance of approximately 600m upstream of the rated section, the flood flow bypassing the gauging station on the right floodplain.

The Mean Annual Flood flow is about 140 cumecs (NERC 1998).

Collate data
The Case Study was selected because it was a part of an existing model, and therefore all the required data were available. The data collated for the Case Study included:

- The current rating curve for the site (see Table 5.13 below);
- Current meter gaugings at the site, given in Appendix E. These were taken during 1990 and 1991 and used to rate the site;
- A rating curve at the downstream boundary, from existing model results;
- Flow and stage hydrographs at the site, from the existing model study; and
- River cross-sections, floodplain cross-sections and river bank levels, all taken from the existing model.

The site started operating as a rated section in 1956 and thirteen ratings have been used since. The rating presently used for the site is given in Table 5.13 below.

Table 5.13: Rating for Case Study 4

		Rating Reference 12R $Q = C\,(h + a)^{\beta}$		
Segment	**Maximum Stage**	C	a	β
Segment 1	0.779	20.6324	-0.002	1.82936
Segment 2	2.613	22.5992	-0.098	1.43813
Segment 3	2.844	0.6594	0	5.06078

The zero of the staff gauge reading is set at 3.803m AOD.

It is understood that segment 3 includes overbank flow and therefore this segment can be considered as a true overbank flow rating when comparing methods.
Flow starts to bypass the site when the stage at the gauge is about 6.45mAOD (i.e. 2.65m on the gauge). The floodplain and bank top elevations at the gauging site are 4.8m AOD and 6.8m AOD respectively (1.0m and 3.0m on the gauge respectively).

Rating curve strategy
The key features of the site which dictated the rating curve strategy are as follows:

- The floodplain is separated from the main channel by high ground;
- During high flows, water overtops the right bank and flows down the floodplain. The water level on the floodplain can differ from the adjacent level in the river channel; and
- Flood volume was thought to be significant for events which just overtopped the river bank, causing some flow and storage on the floodplain.

The modelling strategy was as follows:

- Model the river channel and floodplain as separate channels; and
- Use unsteady flow hydrographs.

Unsteady flow was selected because the water level in the floodplain was known to be different from the river channel, and the volume of water entering the floodplain was considered to be important.

Steady flow (backwater) models were also applied to see whether they could give meaningful results.

At the time the original model was constructed, the software modelled floodplain cell boundaries as weirs, and this was repeated in the Case Study. However, it would have been better to use friction flow except at the boundary with the river channel, where a weir is more appropriate to represent flow over the high ground between the river and the floodplain.

Modelling software
HEC-RAS and ISIS Flow software was used to construct Steady and Unsteady flow models for the Case 4 Study, see Table 5.14.

Table 5.14: Case Study 4 - Models

Software	Run Type	Model configuration
HEC-RAS	Unsteady	Channel, lateral weirs and floodplain storage
ISIS Flow		Channel, spill units and floodplain storage
HEC-RAS	Steady	Channel and floodplain combined (without levees)
ISIS Flow		Channel and floodplain combined
HEC-RAS		Channel and floodplain combined (with levees)

Model boundaries
The upstream model boundary included the river channel and the right bank floodplain. There is no left bank floodplain in the reach under consideration. The main criterion for selecting the upstream boundary was to ensure that it would not affect the distribution of flow between the channel and the floodplain at the rated section.

A location was chosen:

- Where the river channel and floodplain could be represented as a single section;
- Where the proportion of total flow on the floodplain could be estimated with reasonable accuracy; and
- Which was far enough upstream of the rated section for the distribution of flow between the river channel and floodplain at the rated section to be determined by the model.

The downstream boundary is at a distance approximately equal to the backwater length as measured along the river channel, which is estimated to be about 1000 metres using the formula $0.7D/s$ where D is the bankfull depth and s is the river slope. The bankfull depth is estimated to be 3m from an inspection of channel cross-sections. The river slope is estimated to be 0.002 based on the average bed levels at the upstream and downstream ends of the reach.

The downstream boundary was chosen where the river and floodplain could be represented by a single cross-section (i.e. where there was no significant high ground between the river and the floodplain). It was therefore only necessary to have a single rating curve at the downstream boundary.

Model boundary conditions
Upstream inflow hydrographs were determined from the existing model study of which the rated section model is a part. In the absence of these data it would have been necessary to generate hydrograph shapes using the rainfall-runoff approach in the Flood Estimation Handbook (FEH). The shapes should be compared with any available information on hydrograph shape at the rated section. For example, hydrograph lengths could be determined from in-channel flow hydrographs for historic events. Adjustments should be made to the FEH hydrograph shape in order to develop as accurate a hydrograph shape as possible.

The downstream boundary consisted of a stage discharge curve. This was also determined from the existing model study. In the absence of an existing model, the stage discharge curve would be generated by a conventional normal depth hydraulic calculation.

Cross-section locations and survey requirements
Figure 5.2 shows the estimated flow lines on the floodplains, the model extent and the locations of channel cross-sections. The average cross-section spacing is less than values that would be derived from the guidelines given in Section 5.3.4 because of the variable nature of the river channel. It was considered important to represent the variation in cross-section accurately because this will affect local water levels and therefore the amount of water that spills into the floodplains.

It should be noted that there is a cross-section at the rated section, and results at this section are used for the extended rating.

Floodplain cross-sections were located opposite each river cross-section. Embankment longitudinal sections were required between each river section to represent the high ground between the river and the floodplains.

No additional survey work was needed as all the data were available from an existing model.

Set up, calibrate and validate the model
The hydraulic parameters including channel and floodplain roughness coefficients, lateral weir/spill unit coefficients, and floodplain storage volumes were obtained from the existing model. If the model had not been available it would have been necessary to estimate values from the guidance given in Sections 5.3.6 (roughness) and 5.3.7 (structure/spills). Storage volumes, if required, would be estimated from the topographic data for the floodplains.

The model was calibrated using the in-bank gauged data for the rated section, as shown on Figure 5.3. It is understood that the rating is based on overbank flow gaugings (by comparison with data from Ramsbottom, 1989). The overbank part of the rating curve was not used for calibration in order to assess the accuracy of the rating curve extension predicted by the models.

Generate 'best estimate' rating curve
One of the key aspects of this case study is to predict the overbank flow rating. As segment 3 of the rating curve represents overbank flow, the predicted extensions to the rating curve can be compared directly with the rating.

The ‘best estimate’ rating curve was generated using unsteady flow by running a series of hydrographs with different peak flows. The ‘best estimate’ curve is that obtained using ISIS unsteady flow, shown on Figure 5.3.

HEC-RAS was also applied using unsteady flow with the same parameter values as ISIS (i.e. it was not calibrated independently). It can be seen from Figure 5.3 that it predicts higher flows at the same stage. This difference between the two models may be attributed to the fact that whilst the C_d used in both models is the same, the modular limit for the ISIS Flow weir has been reduced based on experience of the performance of natural banks as weirs. Reducing the modular limit represents the fact that a weir simulating flow over natural ground will, as for the C_d, be less efficient than a man made structure. Within HEC-RAS it is not possible to adjust the modular limit of the weir.

The steady flow backwater options in ISIS and HEC-RAS both over-predicted flow, as shown on Figure 5.3. This is because they assume that the water level on the floodplain is the same as in the river channel. This clearly demonstrates that the backwater approach should not be used for river channels with separate floodplains.

The HEC-RAS steady flow (backwater) run with the ‘with levee’ option achieves a good in-bank calibration (because it assumes no flow on the floodplain). However the extrapolation to overbank flow provides erratic results when compared with the unsteady flow approaches, as shown on Figure 5.3. One reason for this is that the model assumes no flow on the floodplain until the level exceeds the bank level at the rated section (approximately 6.8m AOD), and therefore under-predicts flow. Once this level is exceeded, the model assumes flow on the complete floodplain, with the result that the stage drops for an increase in flow.

An approach that was not tried for this site was to use steady flow with the unsteady flow solver.

Sensitivity tests and uncertainties
Sensitivity tests
Sensitivity tests were undertaken using the ISIS Flow model to assess the effect on the predicted rating of increasing and reducing the calibrated channel and floodplain roughness coefficients. A change of 10% was selected to give an indication of the impact of a change in roughness. 10% was considered reasonable for the channel because the coefficients are already calibrated and a larger uncertainty in roughness values would not be expected. Where there is greater uncertainty in the model calibration, a larger change should be used.

The impact of changes to the channel/floodplain roughness coefficients on the rating extension is relatively small, see Figure 5.4. The water level difference caused by a 10% change in Manning’s *n* is approximately +/- 0.05m and the discharge range relative to the calibrated discharge at a particular stage is approximately +/- 5%. The effects are bigger for in-bank conditions and are less sensitive for overbank conditions because of the controlling effect of the banks.

Tests were also undertaken to assess the effect of raising and lowering the water level at the downstream boundary. A value of 0.5m was selected to demonstrate the sensitivity of changes in this parameter. As the downstream boundary conditions were obtained from a calibrated model, a larger uncertainty in downstream water levels would not be expected. If the downstream boundary had been derived from an uncalibrated normal depth calculation, a larger change should be considered.

Raising and lowering the tailwater level had a relatively small impact on the rating curve, see Figure 5.4. This indicates that the downstream boundary is far enough downstream of the rated section to not significantly affect predicted water surface levels at the gauging site.

Uncertainties
The model calibration for in-channel flows is good and the uncertainty associated with this part of the rating should be within 10%. However the rating extension is primarily concerned with overbank flow where the proportion of flow on the floodplain is predicted to approach 50% at a flow of 250 cumecs (return period about 25 years). The point corresponding to the top limit of the Agency rating curve has a level of 6.647m AOD and a flow of 131 cumecs. For this stage ISIS predicts a flow about 20 cumecs less and HEC-RAS about 20 cumecs more. Thus the uncertainty is about +/-15%.

It is suggested that the overall uncertainty in the rating at a flow of 250 cumecs might be +/-20%. The rating curve has a flat slope and the flow increases by about 25 cumecs for every 0.1m increase in stage. Thus this estimate assumes a level range of +/- 0.2m at 250 cumecs.

Sensitivity tests should be carried out on the values of parameters used to represent the embankments and floodplains, as the greatest uncertainty is in these components.

Applicability of rating
The rating is considered to be applicable for the full range of high flows as the hydraulics are reasonably well represented and would not be expected to change for further increases in stage. However, effort should be made to reduce the uncertainty associated with the overbank section of the rating by:

- Flow gauging on the floodplain using peak water level gauges and peak velocity meters;
- Correlation with upstream gauges if appropriate (there are no downstream gauges in this case); and
- Comparison with results from the full model, which has been calibrated elsewhere.

-

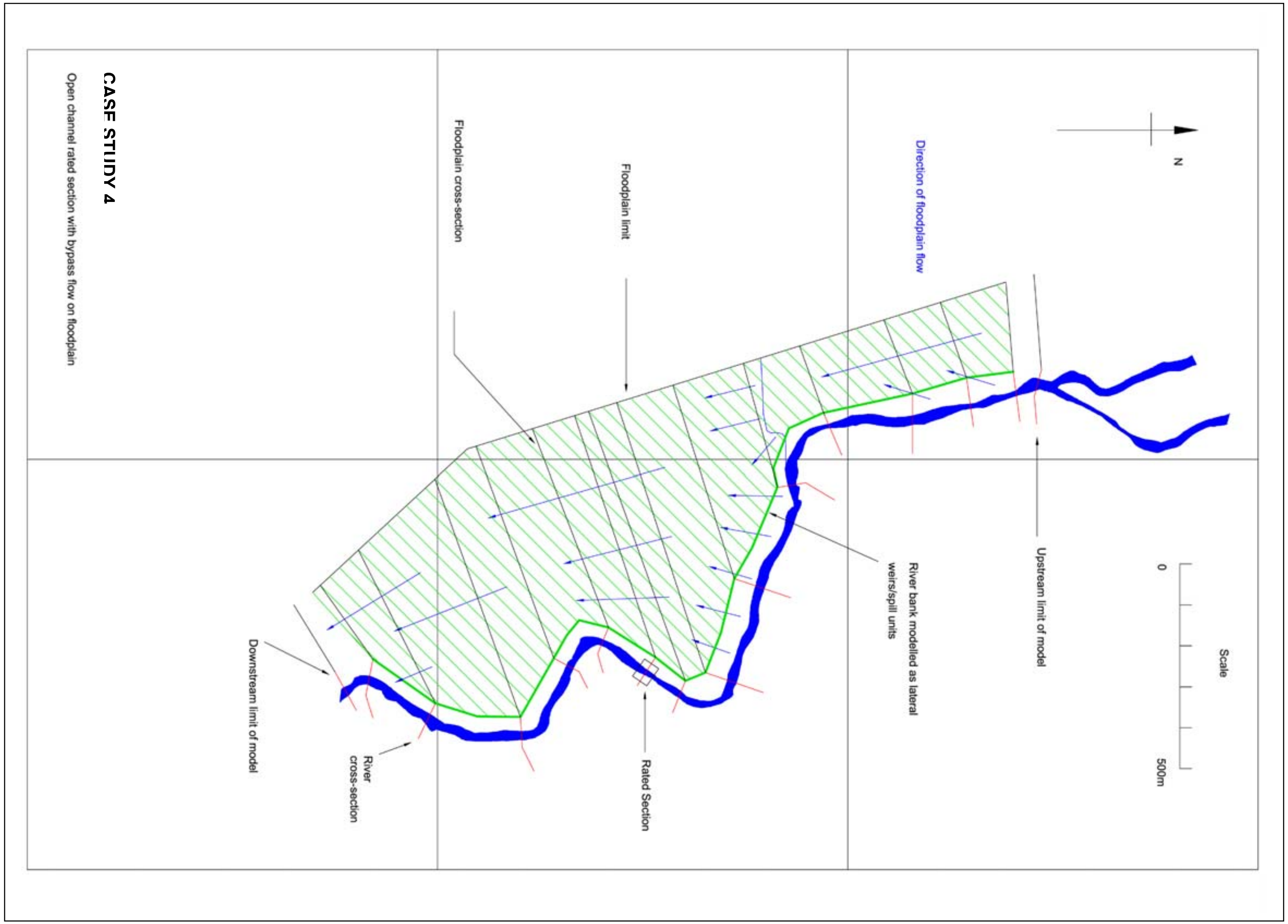

Figure 5.2: Case Study 4: Open channel rated section with bypass flow on floodplain

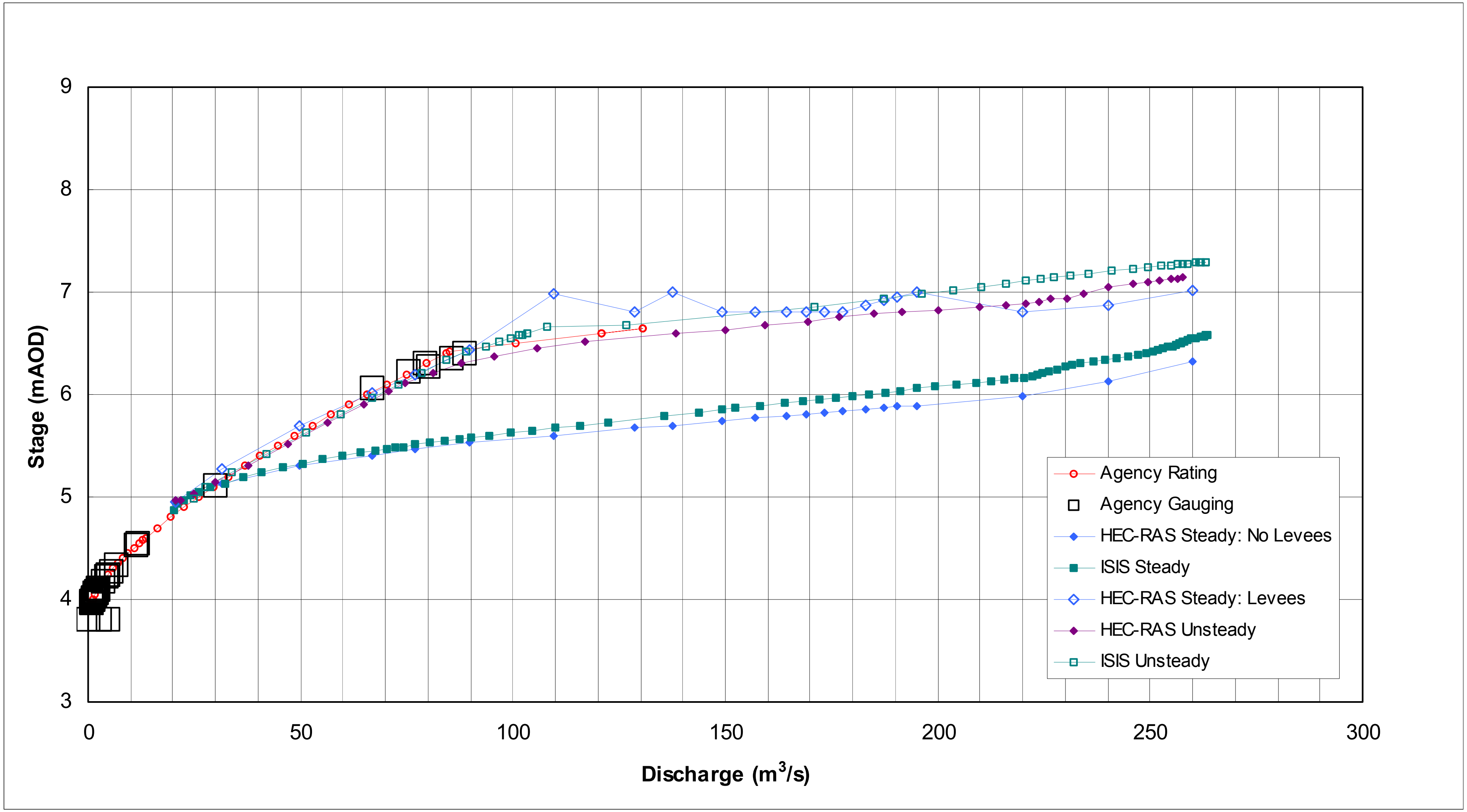

Figure 5.3: Case Study 4: Comparison of Environment Agency and computational model ratings

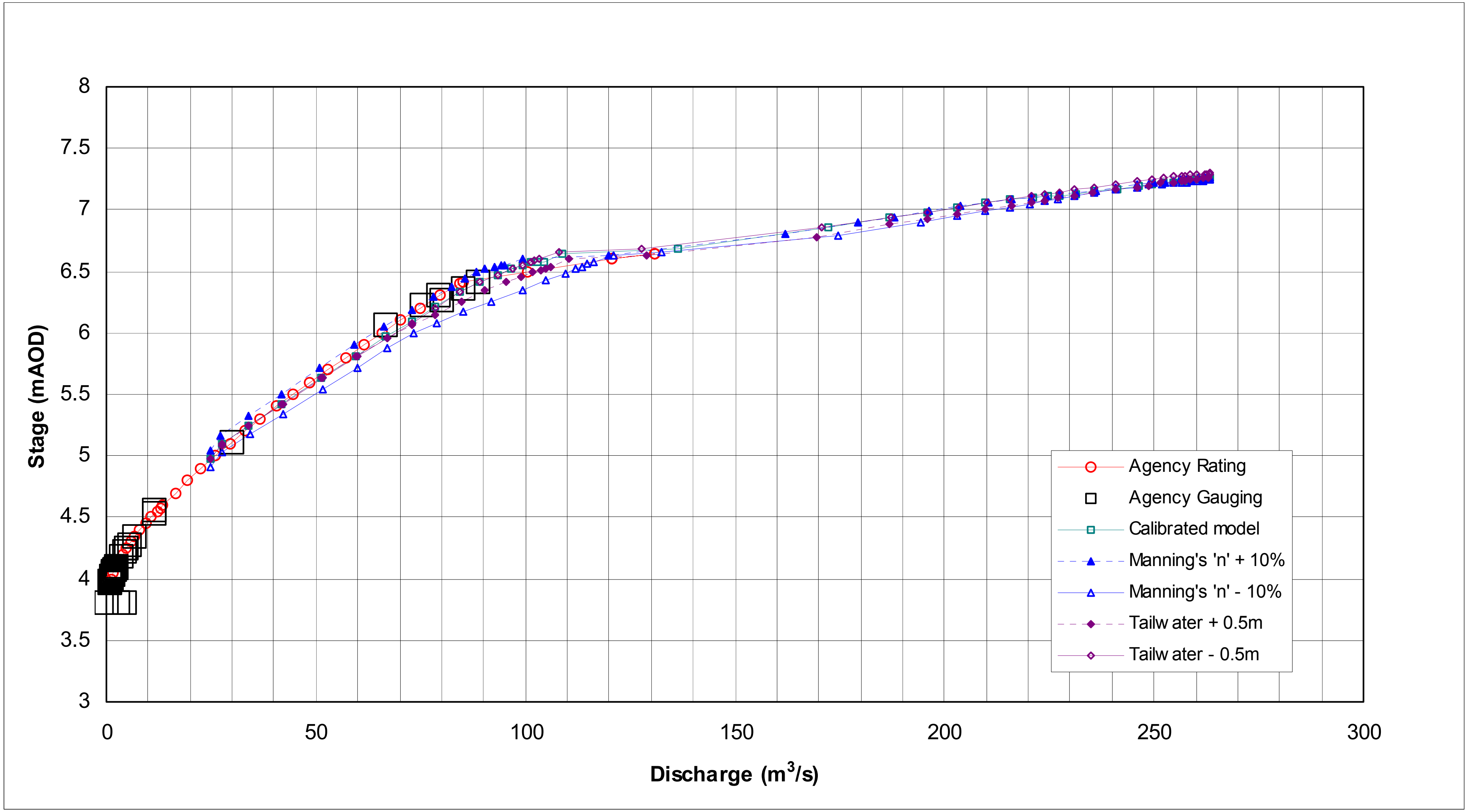

Figure 5.4: Case Study 4: Comparison of Environment Agency and computational model ratings (ISIS unsteady sensitivity tests)

5.4.4 Case Study 5: 1-D modelling of compound broad crested non-standard weir in open channel

Understand the hydraulics

The compound broad-crested weir has three crests with no divide piers; the low crest is 12.31m wide with a crest elevation of 14.76m AOD; the high crests are 11.62m wide in total with a crest elevation of 15.22m AOD. The weir is located in an incised river channel that has a width of 24m at structure full level

The channel opens out from a width of 24m and a level of 16.2m AOD, at the top of the weir wing walls, to a width of 37m at 18.3m AOD. A horizontal berm at 18.3m AOD opens the channel further to a width of 41m before the channel slopes upwards to give a top width of 50m at an elevation of 20m AOD. The layout of the site is shown on Figure 5.5 and the cross-section is shown on Figure 5.6 below.

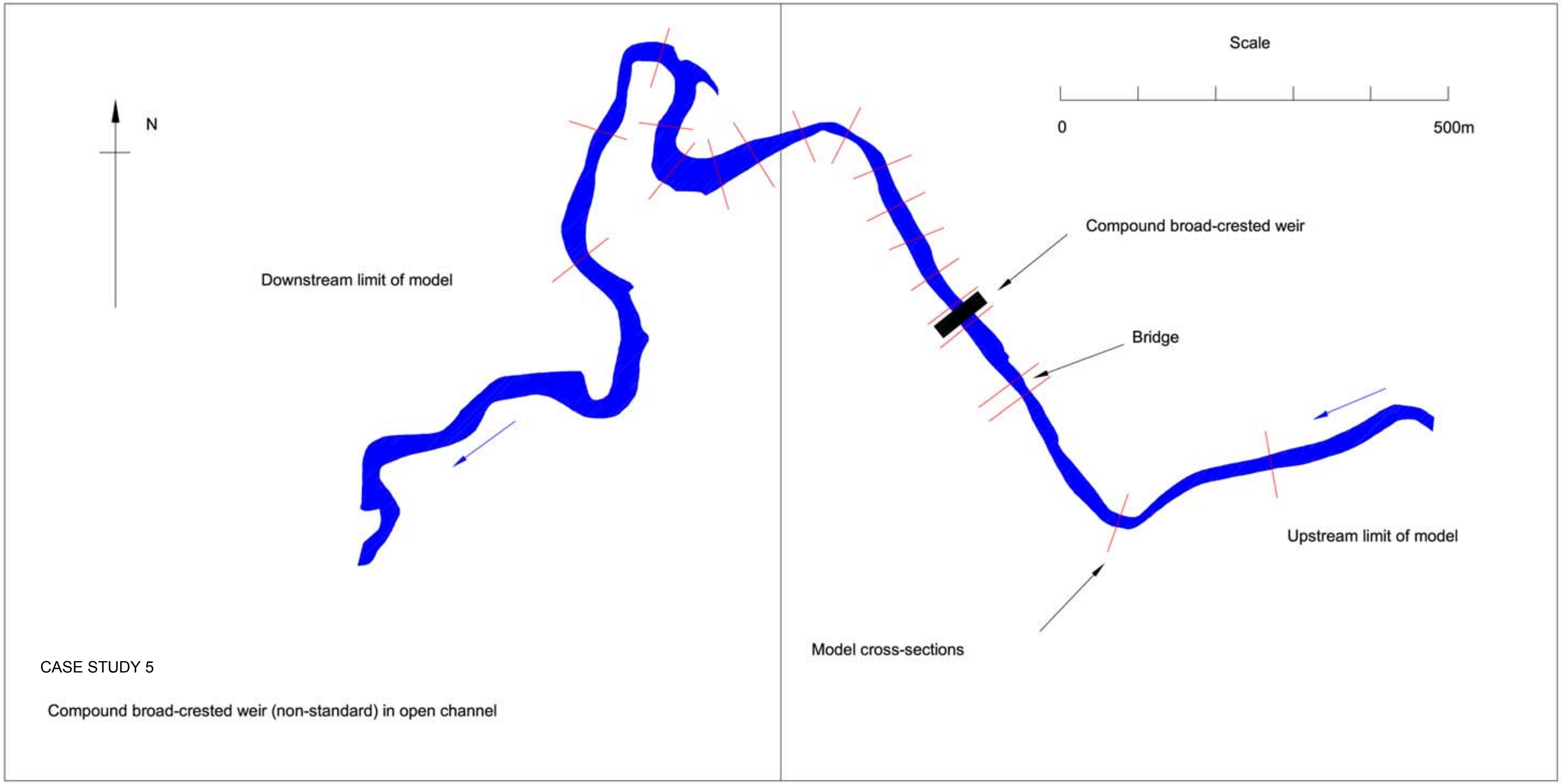

Figure 5.5: Case Study 5: Compound broad crested weir (non-standard) in open channel

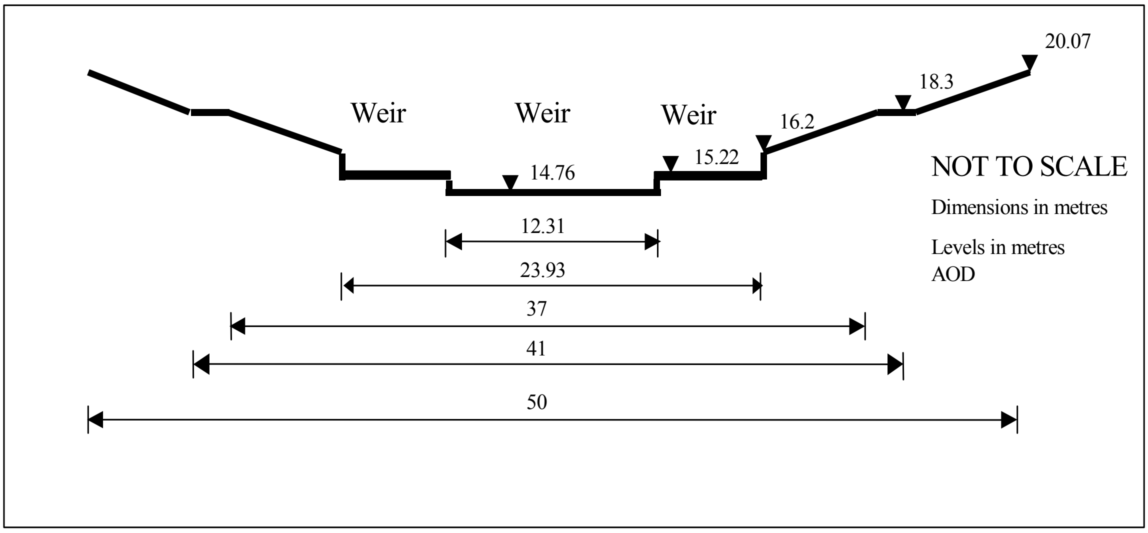

Figure 5.6: Cross-section at the Case Study 5 weir

The approach channel to the weir is straight with the channel showing a sinuous plan form downstream of the weir, see Figure 5.5. The bed slope of the river channel upstream of the weir is approximately 1 in 700. Downstream of the weir, for a distance of approximately 400m, the profile of the river bed is uneven with an average bed slope close to horizontal. Beyond 400m downstream of the weir the bed slope increases significantly to approximately 1 in 200.

The weir is modular to a stage and discharge of approximately 16.5m AOD and 100 m^3/s. Above a water surface level of 16.5m AOD weir starts to drown and becomes fully drowned at a water surface level of approximately 18m AOD.

The flow remains almost entirely in-bank up to the 100-year flow which is of the order of 400 cumecs.

The biggest difficulty associated with the hydraulics at the site is the fact that the flow gaugings show very little increase in flow between stages 16.5 and 18m AOD, see Figure 5.7. Reasons for this could be:

- Drowning of the structure. The pattern of drowning will be complex as the separate weir sections will drown at different flows. In addition, the pattern of gaugings suggests a variable downstream control which is causing drowning at different stages, for example vegetation growth or the operation of control gates. There is however no evidence of variable downstream control in the information presented for the study;
- Interference between slow moving flow on the sloping bank and faster moving flow in the main channel. The discontinuity in the gauged data begins when the stage is about 0.5m above the structure-full level, suggesting that it could be caused by this interaction. However the differences are greater than would be expected from this source; and
- The flow gaugings only use a single velocity measurement in the vertical. The river is over 5 metres deep and the single measurement may not fully identify the average velocity in the vertical.

The analysis suggests that drowning is the cause of the unusual rating curve shape.

Collate data
The Case Study was selected because it was a part of an existing model, and therefore all the required data were available. The data collated for the Case Study included:

- The current rating curve for the site (see Table 5.15 below);
- Current meter gaugings at the site. These were taken between 1993 and 2000 and used to rate the site, and are given in Appendix E;
- A rating curve at the downstream boundary, from existing model results;
- River cross-sections and structure details, all taken from the existing model.

The site started operating as a rated structure in 1992 and seven ratings have been used since. The rating presently used for the site is given in Table 5.15 below.

Table 5.15: Rating for Case Study 5

		Rating Reference 12R $Q = C_r\,(h + a)^{\beta}$		
Segment	**Maximum Stage**	C_r	a	β
Segment 1	0.477	24.1602	0.000	1.4568
Segment 2	1.485	32.234	0.000	1.8461
Segment 3	4.014	27.738	0.000	2.2258

The zero of the staff gauge reading is set at 14.87m AOD.

Rating curve strategy
The main factors affecting the rating curve strategy are as follows:

- Flows contained within the channel and no floodplain flow;
- No spills or storage areas that could distort the flood hydrograph; and
- No dynamic effects caused by the tide or gate operation.

A steady flow (backwater) approach was adopted in accordance with the guidance given in Section 5.3.2.

Modelling software
HEC-RAS, ISIS Flow and MIKE 11software packages were used to construct steady flow models for the Case 5 study, see Table 5.16. As the structure modules in the software were used for the weirs, it was necessary to add a parallel unit to model the channel above structure full conditions.

Table 5.16: Case Study 5 - Models

Software	Run Type	Model configuration
HEC-RAS	Steady	Weir and channel above structure full conditions modelled together as a single broad crested weir unit
ISIS Flow		Low and high level weir crests modelled independently using broad crested weir unit with channel above structure full modelled as a spill unit (using broad crested weir approach)
MIKE 11		The compound broad crested weir geometry, including the channel above structure full conditions, was modelled using a broad crested weir unit which allowed a level-width table to be specified.

The standard formula for calculating flow over a broad crested weir is

$$Q = C_w \, bH^{1.5}$$

where

C_w = Weir coefficient, equivalent to $(2/3)^{1.5}\, g^{0.5}\, C_d$
C_d = Coefficient of Discharge
b = crest width (across the channel)
H = Total head on the weir

In HEC-RAS and ISIS the coefficient C_w is referred to as the weir coefficient and discharge coefficient respectively. In MIKE 11, C_w is represented as a head loss factor applied to free overflow at the weir.

Model boundaries
The upstream model boundary was about 400m upstream of the weir. This could have been located much closer to the weir (say 100m upstream) under conditions of steady flow with no bypassing of the gauging site.

The downstream boundary was selected on the basis of backwater length L=0.7D/s where D is 5 metres and s is 1 in 200, extending downstream from the end of the 400m horizontal section of channel. Thus the downstream boundary was about 1,100m downstream of the weir.

Model boundary conditions
The upstream model boundary consisted of a series of steady flows to the maximum limit of the Agency rating.

The downstream boundary consisted of a stage discharge curve. This was determined from the existing model study. In the absence of an existing model, the stage discharge curve would be generated by a conventional normal depth hydraulic calculation.

Cross-section locations and survey requirements
Cross-sections were located upstream and downstream of structures. The spacing was about 100m in the open channel. This spacing was used because the data were available from the existing model. It is understood that the close spacing was considered necessary to identify variations in cross-section shape of the channel. However, if a new survey had been needed the number of cross-sections could be halved

No new survey work was needed because the data were taken from an existing model.

Set up, calibrate and validate the model
The hydraulic parameters including channel roughness coefficients and structure coefficients were obtained from the existing model. If the model had not been available it would have been necessary to estimate values from the guidance given in Sections 5.3.6 (roughness) and 5.3.7 (structures).

The model was calibrated using the in-bank gauged data, as shown on Figure 5.7. Calibration under drowned flow conditions was not possible because water level measurements were not available downstream of the weir.

Generate 'best estimate' rating curve
The rating curve was generated using a series of steady flows, and the results are shown in Figure 5.7. For flows and stages less than approximately 100 m^3/s and 16.5m AOD respectively the predicted rating accurately represents the head-discharge relationship associated with a broad-crested weir operating under modular flow conditions. In this range the discharge coefficient had been calibrated against flow gaugings, and therefore a good result would be expected.

Between a stage of 16.5 to 18m AOD there is a discontinuity in the gauged data. The gauged discharge varies between about 100 and 130 m^3/s throughout this range. It is understood that the gauge drowns in this range, and it is also noted that the channel becomes "two-stage" in this range. Thus, the cause of the discontinuity might be a combination of drowning and out-of-bank flow, as discussed above.

The shape of the discontinuity in flow gaugings observed at the site is similar to the discontinuities in stage discharge curves observed when overbank flow occurs in two-stage channels. This effect is not contained in 1-D model hydraulics. In contrast, the transition from modular to drowned flow is normally smooth and should be well represented by 1-D models, providing that the downstream section is well calibrated. In this case there were no calibration data for the downstream reach.

The trend of the gauged data above a stage and flow of approximately 18m AOD and 130 m^3/s reflects the drowned flow curve for the weir predicted by the models, see Figure 5.7.

It is evident from Figure 5.7 that:

- The Environment Agency rating does not take account of the discontinuity in the gauged flow data; and

- The models do not satisfactorily reproduce the rating curve based on flow gaugings. This is believed to be because the downstream rating varies depending on the downstream channel condition.

The transition from modular weir flow to drowned weir flow is represented by the HEC-RAS and ISIS simulations with the drowned flow curve from ISIS giving a better fit to the highest gauged flow data, but a worse fit to the medium flows. This is probably attributable to the ability in ISIS to change the modular limit for the weir. In ISIS this was set to 0.66, the recommended conservative value in the British Standard. In HEC-RAS the software automatically accounts for any submergence of the weir by the water surface level downstream of the weir. In MIKE 11 it was possible to simulate a good fit to the gauged data representing drowned weir flow by reducing the head loss factor to 0.66 but not possible to represent modular, transitional and drowned weir flow using a single head loss factor.

Uncertainties in the rating
Case study 5 shows that for modular flows a very good calibration has been achieved. In such cases the uncertainty in predicted flow is within +/- 10%.

The best flow rating curve obtained for Case Study 5 had deviations from gauged flow data of up to 40 cumecs, or about 40% of the gauged flow in the transition region. At higher stages, flow predictions provided by different models varied by up to 30%. It is therefore proposed that the rating curve extension produced by ISIS is the best estimate, but the uncertainty at high stages is 30%. The main reason for this is the discontinuity in gauged flows. Uncertainties could be reduced by calibrating the downstream reach and improving the modelled transition from modular to drowned flow, and then channel control.

Applicability of rating
The rating can be applied over the full range up to and including the estimated 100-year flow, which is in excess of 400 cumecs as there is no hydraulic reason why the trend in the rating should change. However, the rating is subject to the uncertainties discussed above. The primary difficulty with this rating is that the downstream rating is variable depending on channel conditions, and therefore there is not a unique rating for this site.

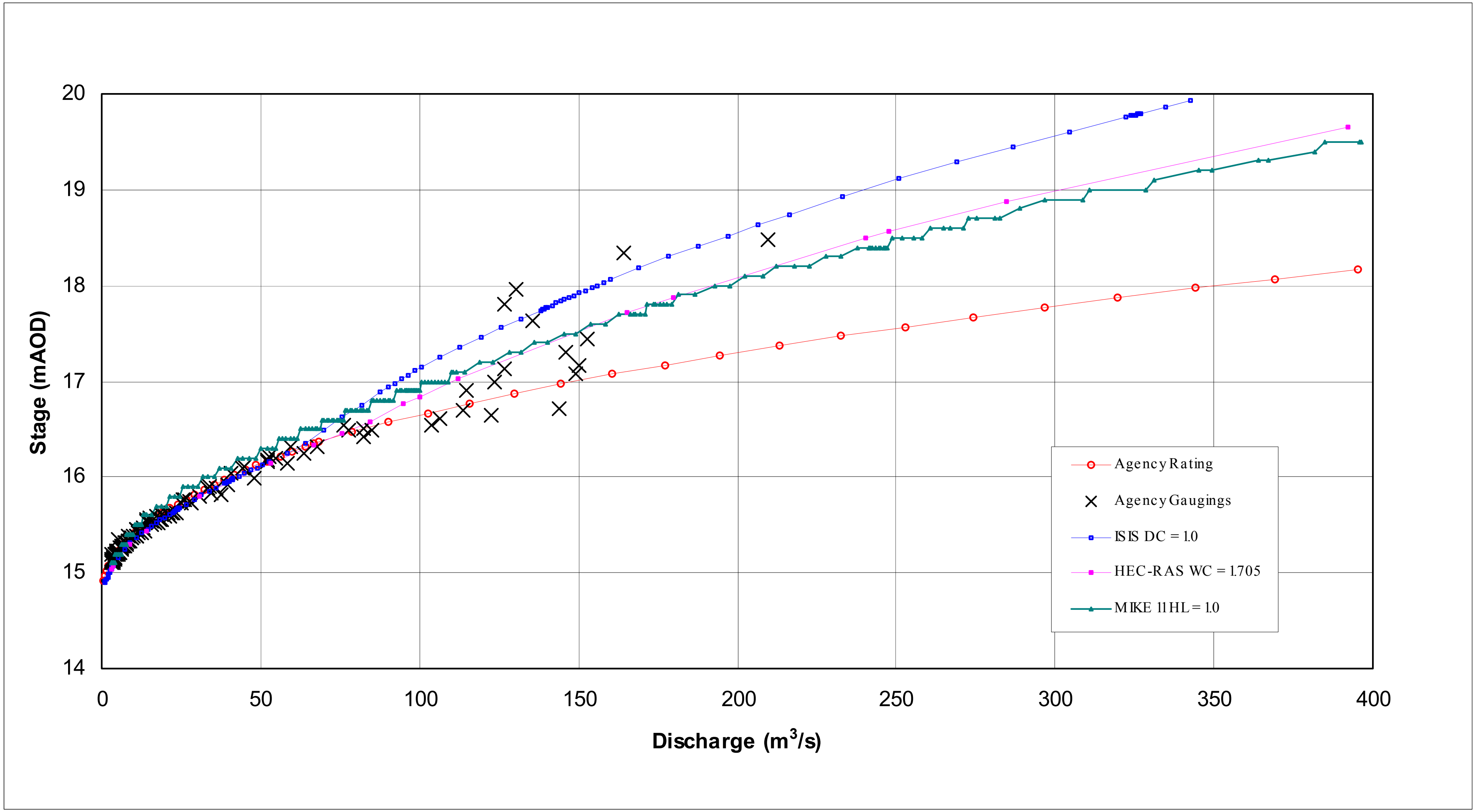

Figure 5.7: Case Study 5: Comparison of Environment Agency and computational model ratings

5.4.4 Case Study 6: 2-D modelling of a compound broad crested weir with bypass flow on the floodplain

Understand the hydraulics
The weir is located on a lowland river which has a main channel width ranging between approximately 10 to 20m. A smaller stream carrying water from a canal joins the river a short distance upstream of the gauging station. The layout of the site is shown on Figure 5.8. Bypass flows occur over the left bank floodplain during extreme events.

Collate data
The Case Study was selected because it had been used in previous comparative studies and various types of modelling had been done, including 2-D modelling. The data collated for the Case Study included:

- Current meter gaugings at the site, given in Appendix E;
- The existing rating curve for the site (see Table 5.17 below);
- Comparative rating curves derived in previous studies, including physical modelling as well as 1-D and 2-D modelling;
- A rating curve at the downstream boundary, from previous 1-D model results; and
- TELEMAC 2-D data files from previous studies, including bathymetry and boundary condition information.

The site started operating as a rated section in January 1969 and five ratings have been used since. The rating used at the time this report was produced is given in Table 5.17 below.

Table 5.17: Rating for Case Study 6

Segment	Rating Reference 5 $Q = C\,(h + a)^{\beta}$ Maximum Stage	*C*	*a*	*B*
Segment 1	0.334	7.7936	0.05721	1.28024
Segment 2	0.428	20.5842	0.074	2.42528
Segment 3	1.0	14.292	0.10796	2.09351

The zero of the staff gauge reading is set at 30.88m AOD.

Flow starts to bypass the site when the stage at the gauge is approximately 1m on the gauge. This would suggest that segment 3 may begin to reduce in accuracy with extrapolation above this level.

Rating curve strategy
The key features of the site which dictated the rating curve strategy are as follows:

- The floodplain is not separated from the main channel by high embankments;
- During high flows, water overtops the left bank and flows down the floodplain. The water level on the floodplain can differ from the adjacent level in the river channel; and

- Flood volume was thought to be significant for events which just overtopped the river bank, causing some flow and storage on the floodplain.

The modelling strategy was as follows:

- Model the river channel and floodplain as integral parts of the computational domain; and
- Use unsteady flow gradually increasing the flow to achieve steady state conditions for a range of flows.

Steady flow was selected because static storage on the floodplain was not viewed as significant.

Modelling software
The TELEMAC software was used to construct models for the Case Study.

Only the TELEMAC software was applied during this case study, however, comparative results were available from physical modelling and from 1-D modelling using SALMON-F.

Model boundaries
In this case study, the model boundaries were chosen at the same locations as those used in the physical modelling in order to provide a direct comparison of the results. The upstream model boundaries included the river channel and the inflow from the canal. The main criterion for selecting the upstream boundary was to ensure that it would not affect the distribution of flow between the channel and the floodplain at the gauging site.

A location was chosen:

- Where the river channel and floodplain at each location (river and canal) could each be represented as a single section;
- Where the proportion of total flow on the floodplain could be estimated with reasonable accuracy; and
- Which was far enough upstream of the rated section for the distribution of flow between the river channel and floodplain at the rated section to be determined by the model.

The downstream boundary was approximately 150 metres downstream of the weir. This is less than the full backwater length, but was chosen to coincide with the location used in the physical model.

The downstream boundary satisfied the criteria that the river and floodplain could be represented by a single cross-section (i.e. where there was no significant high ground between the river and the floodplain). It was therefore only necessary to have a single rating curve at the downstream boundary.

Model boundary conditions
Upstream inflows ramped from zero to the required flow for each scenario to reach a steady state solution. Event hydrographs were not simulated in this case study.

The downstream boundary consisted of a fixed water level, taken from the 1-D stage discharge curve at the specified flow. In the absence of an existing model, the stage discharge curve would be generated by a conventional normal depth hydraulic calculation.

Topography, survey requirements and grid generation
No additional survey work was needed as all the data were available from an existing model. The topographical data for the model was derived from the surveyed cross-sections used in the 1-D modelling. For 2-D modelling, the full channel and ground surface (model bathymetry) must be described using x,y,z co-ordinates. The cross-sections used in the 1-D modelling were spaced more widely than the width of the channel, and could not be used in isolation to produce the model bathymetry. Finely spaced cross-sections of the river channel therefore had to be created by a process of interpolation and spatial manipulation. The floodplain data was supplemented by digitised contours from aerial surveys as well as embankment profiles and additional spot height information available in x,y,z co-ordinates from the field surveys. Figure 5.9 shows the final model bathymetry used for the study (reproduced from Hollinrake & Millington, 1994).

Grid generation is extremely important in higher dimension modelling. The computational grid, defining points (or nodes) at which variables are calculated, is generated independently from the model bathymetry. The level at any point in the grid is then interpolated from the nearest adjacent points in the model bathymetry. In this respect an unstructured grid is very important since it allows higher density of computational nodes in critical modelling areas or where the bathymetry or water slope varies rapidly. The grid was therefore generated in a number of parts to allow differing resolution for the floodplain, river channel and weir. The final grid used for the study is shown in Figures 5.10 and 5.11 (reproduced from Hollinrake & Millington, 1994).

Set up, calibrate and validate the model
The hydraulic parameters such as roughness coefficients were tested in the sensitivity analyses discussed in subsequent sections of this report. The weir is defined in terms of its geometry rather than equations, and does not have a discharge coefficient. However, unlike 1-D models, the roughness coefficient can also influence the calculation of flow over the weir. Normally, estimates of roughness values would be based on the guidance given in Section 5.3.6. TELEMAC does not readily support different roughness values for the channel and floodplain, and considerable additional manipulation would be required to effect variable roughness using customised routines.

As a means of validation, the model was run at lower flows for comparison against in-bank spot gaugings and the existing rating curve. The results are presented in Figure 5.12. Additional illustrations from a 40 cumec run are also shown in Figures 5.13 to 5.15, including depth (inundation), water surface profile over the weir, and velocity lines.

Generate 'best estimate' rating curve
Although TELEMAC 2-D was the only model used during this case study, results were also available for previous studies at the same location using physical modelling and 1-D modelling. From the results presented in Figure 5.12 it would appear that the rating curve extension produced by a 1-D model for this particular case study is quite

adequate. The 1-D model constructed in this case had the advantage of the physical model results including floodplain flow patterns. It was therefore possible to align the cross-sections accordingly. The 1-D model results generally follows the physical model results more closely, suggesting that the 1-D model used for this case is more accurate than the 2-D model.

The 2-D curve is only applicable up to 50 cumecs, when the bridge downstream of the weir (see Figure 5.8) begins to surcharge (as found in the 1-D and physical modelling study). The 2-D model only represented the channel constriction from the bridge abutments, but not the bridge deck itself. TELEMAC does not have a long track record of use in river situations, and therefore does not have a wide variety of customised routines for representing structures. These high flows represent very extreme events with very high return periods, and are therefore not of major concern in this particular case study.

No calculations were performed to quantify the effects of the interaction between the main channel and floodplain beyond those modelled explicitly by the Shallow Water Equations and these 'weir type' effects were viewed as negligible in this case

There may well be cases in which the 1-D rating would not be sufficiently accurate, and higher dimension modelling may be required. The reader is referred to Section 5.1.3 for description of situations suited or not suited to 2-D modelling.

Sensitivity tests and uncertainties

Sensitivity tests

Sensitivity tests were undertaken using the TELEMAC model to assess the effect on the predicted rating of changing the roughness coefficients and various mathematical parameters used in the model. The scenarios are listed in Table 5.18 below, together with their respective results.

Table 5.18: Results for Case Study 6

Nr	Flow	Tail	Head	Description	Observations
01	40	31.44	32.12	Manning = 0.008	
02	40	31.44	32.18	01+Manning = 0.024	
03	40	31.44	32.07	01+BOTTOM SMOOTHING = 1	Low, no benefit in runtime
04	40	31.40	32.10	01+Tailwater = 31.40	Tailwater not very sensitive
05	40	31.44	32.10	01+VELOCITY DIFFUSIVITY = 0.0001	Velocity diffusivity not very sensitive
06	40	31.44	32.12	01+VELOCITY DIFFUSIVITY = 0.2	Velocity diffusivity not very sensitive
07	40	31.44	32.13	01+K-Epsilon model VELOCITY DIFFUSIVITY = 1.E-6	Turbulence model not very sensitive, slow
08	40	31.44	32.11	01+MASS-LUMPING ON H = 1	No benefit in runtime
09	40	31.44	32.12	01+MASS-LUMPING ON VELOCITY = 1	No benefit in runtime
10	40	31.44	32.13	01+Nikuradse = 0.001;	Similar to Manning 0.008 at this flow, slow
11	40	31.44	32.11	01+inflow 35 (river) + 5 (canal)	Inflow distribution not sensitive

Table 5.18: Results for Case Study 6 (continued)

Nr	Flow	Tail	Head	Description	Observations
12	40	31.44	32.11	01+inflow25 (river) +15 (canal)	Inflow distribution not sensitive
13	18.93	31.05	31.87	01+18.93 cumecs	Low relative to TELEMAC 94 results?
14	40	31.44	32.16	01+TIME STEP = 0.2;	Slow, more iterations required
15	12	30.88		12 cumecs, n=0.035	Water balance error (outflow 10 instead of 12 cumecs). Rerun higher accuracy as 21.
16	18.93	31.05	31.93	18.93 cumecs, n=0.035	see graph for final rating
17	30	31.29	32.09	30 cumecs, n=0.035	see graph for final rating
18	40	31.44	32.24	40 cumecs, n=0.035	see graph for final rating
19	50	31.56	32.34	50 cumecs, n=0.035	see graph for final rating
20	60	31.66	32.42	60 cumecs, n=0.035	see graph for final rating
21	12	30.88	31.81	12 cumecs (15 cont) at high accuracy	Slow, water level higher than gauged rating curve, possibly due to water surface curvature over weir

The results of each steady state simulation are presented in Figure 5.12 to show the range of values. The simulations were carried out on a Sun Solaris UNIX workstation at Wallingford and run-times varied between a typical 11-12 hours up to almost 33 hours for high accuracy settings.

The initial sensitivity analyses were carried out using a relatively low Manning's *n* value of 0.008 for comparison with previous TELEMAC analyses using Nikuradse roughness of 0.001. The impact of changes to the channel/floodplain roughness coefficient on the rating extension is relatively small. The water level difference caused by a 45% change in Manning's *n* (from 0.024 in scenario 02 to 0.035 in scenario 18) is approximately +/- 0.06m and the discharge range relative to the calibrated discharge at a particular stage is approximately +/- 8%.

Tests were also undertaken to assess the effect of changes to the water level at the downstream boundary. A change of 0.04m was selected to demonstrate the sensitivity of changes in this parameter, which produced a change in headwater level at the gauging station of 0.02m. As the downstream boundary conditions were obtained from a calibrated model, a larger uncertainty in downstream water levels would not be expected. If the downstream boundary had been derived from an uncalibrated normal depth calculation, a larger change should be considered. The low sensitivity to tailwater level indicates that the downstream boundary is far enough downstream of the rated section to not significantly affect predicted water surface levels at the gauging site.

Some of the mathematical parameters were also tested for sensitivity, such as the turbulence model type, velocity diffusivity and various smoothing options. The change in level at the gauging station location was small for all of these changes, and the smoothing options did not present significant reductions in run-time. One particular scenario (number 15, for 12 cumecs) converged on an incorrect solution. Repeating the run at higher accuracy managed to correct the problem (although the run duration was very long). This illustrated the importance of careful inspection of the results, rather than trusting a single model output at face value.

Uncertainties
The model calibration for in-channel flows is good and the uncertainty associated with this part of the rating should be within 10%.

However the rating extension is primarily concerned with overbank flow where the proportion of flow on the floodplain is predicted to approach 40% at a flow of 40 cumecs (return period in excess of 200 years). TELEMAC could not be used within the scope of this study to test the sensitivity of floodplain roughness for overbank flow. However, it is estimated that the uncertainty of the TELEMAC rating curve up to 40 cumecs will be within +/-15%.

Applicability of rating
The 2-D rating is considered to be applicable up to a maximum of about 50 cumecs, due to the surcharging of the bridge downstream of the weir. However, effort should be made to reduce the uncertainty associated with the overbank section of the rating by flow gauging on the floodplain using peak water level gauges and peak velocity meters during an out-of-bank event.

The comparison with the 1-D model and the physical model indicates that the rating curve produced by the 2-D model is no better than results from the 1-D model, and the maximum flow is limited by surcharging of the bridge downstream. It should be remembered that the 1-D model design had the benefit of the physical model results. However it would be hard to justify using a 2-D model in this case, particularly bearing in mind the additional modelling effort needed.

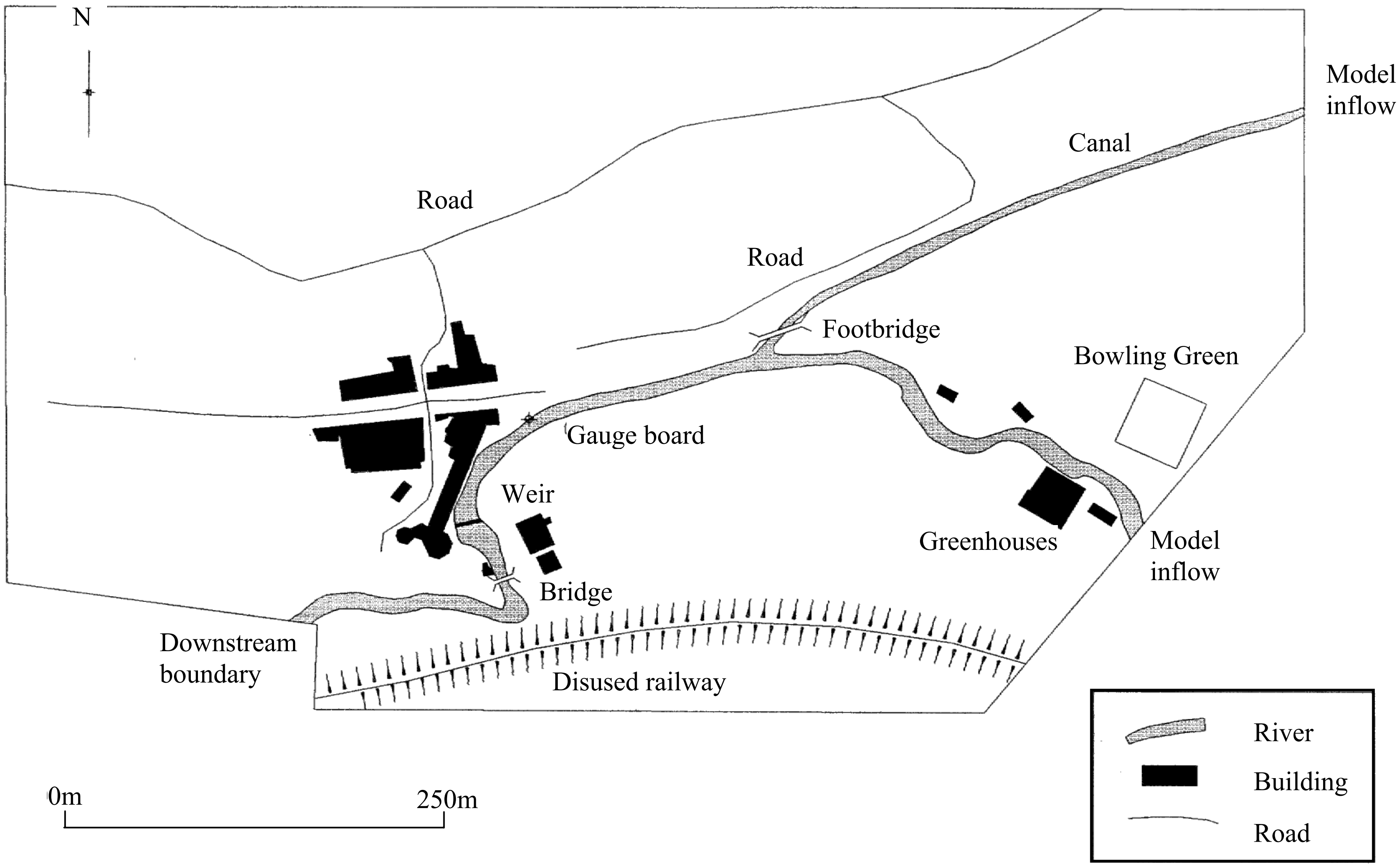

Figure 5.8: Case Study 6: Layout of weir with bypass flow on left bank floodplain

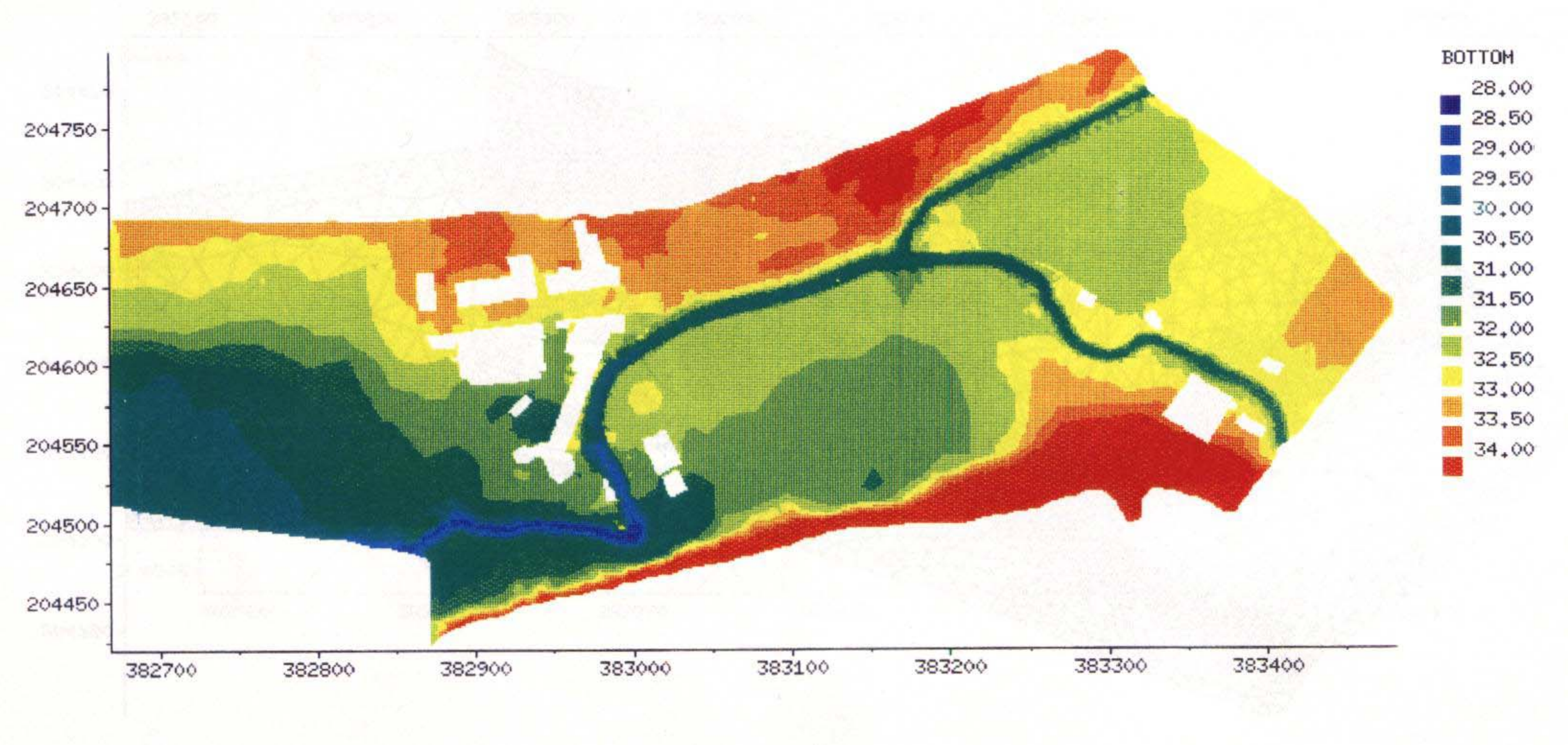

Figure 5.9: Case Study 6: Model bathymetry

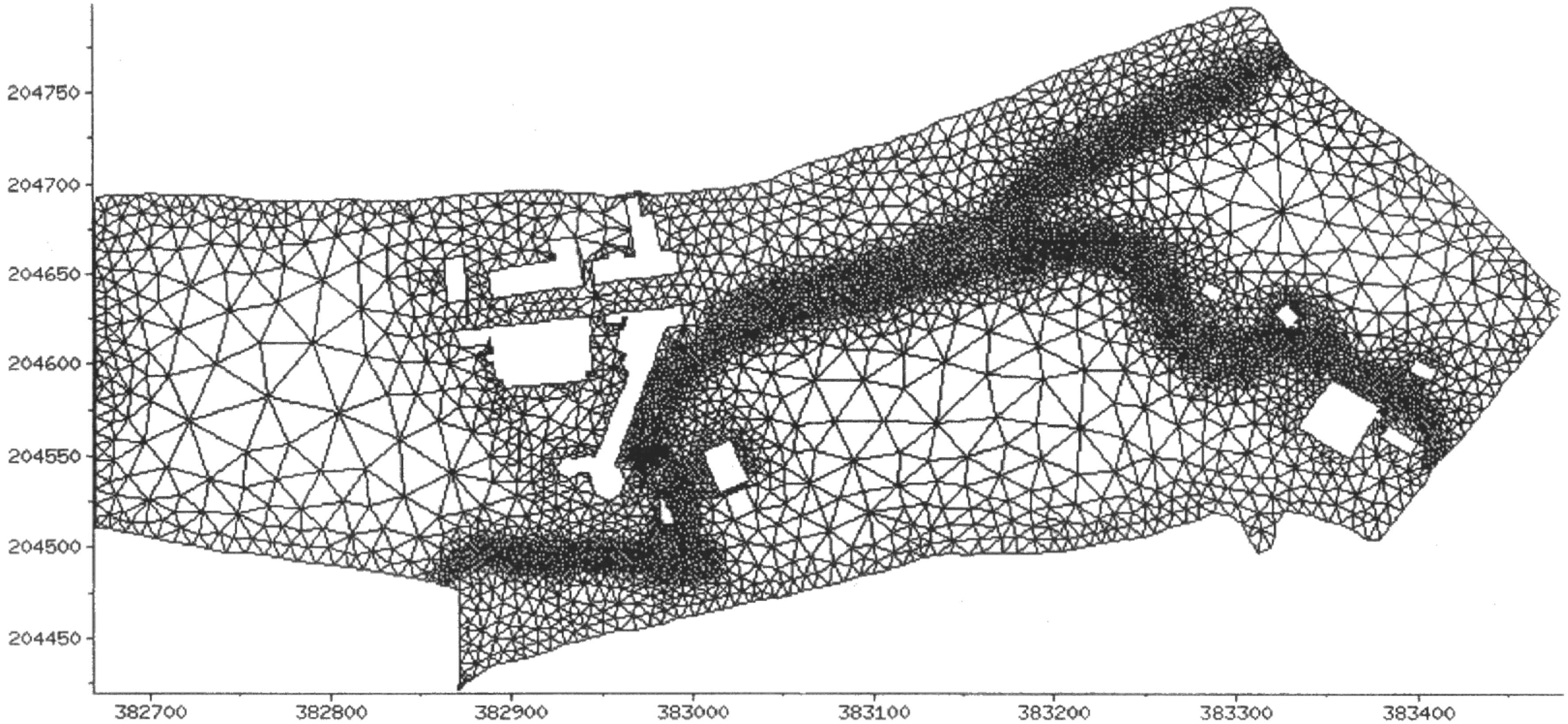

Figure 5.10: Case Study 6: Computational grid

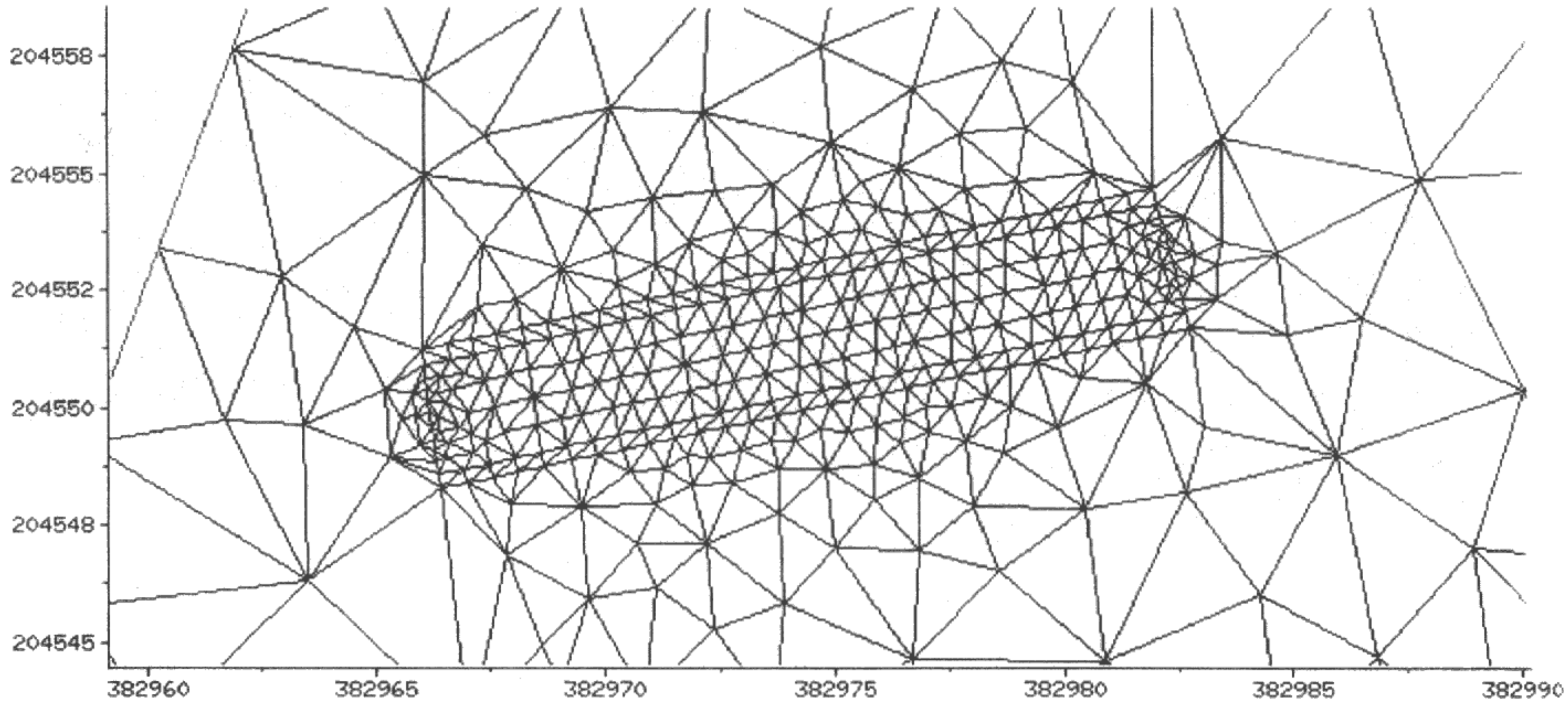

Figure 5.11: Case Study 6: Computational grid – weir detail

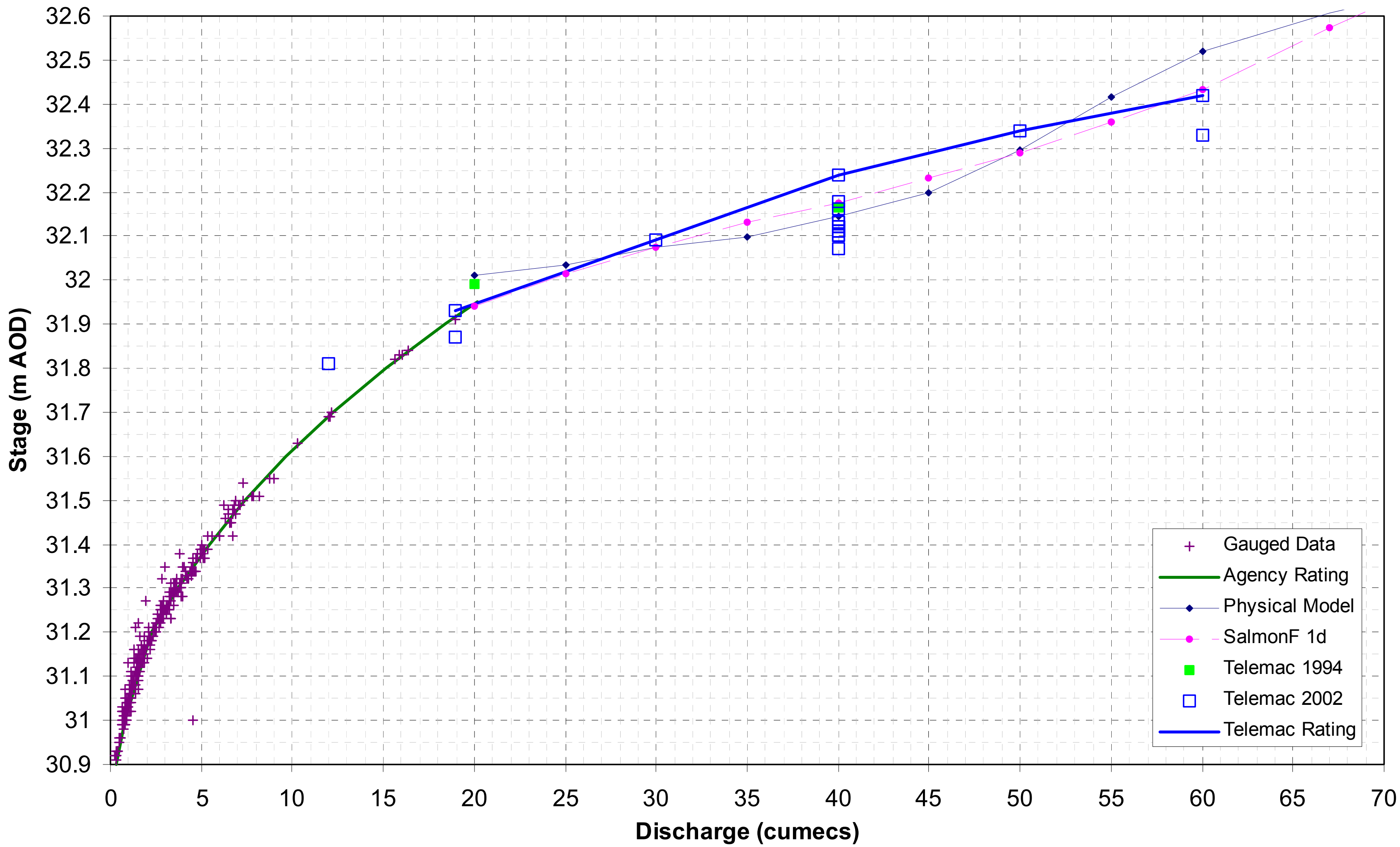

Figure 5.12: Case Study 6: Comparison of Environment Agency and computational model ratings

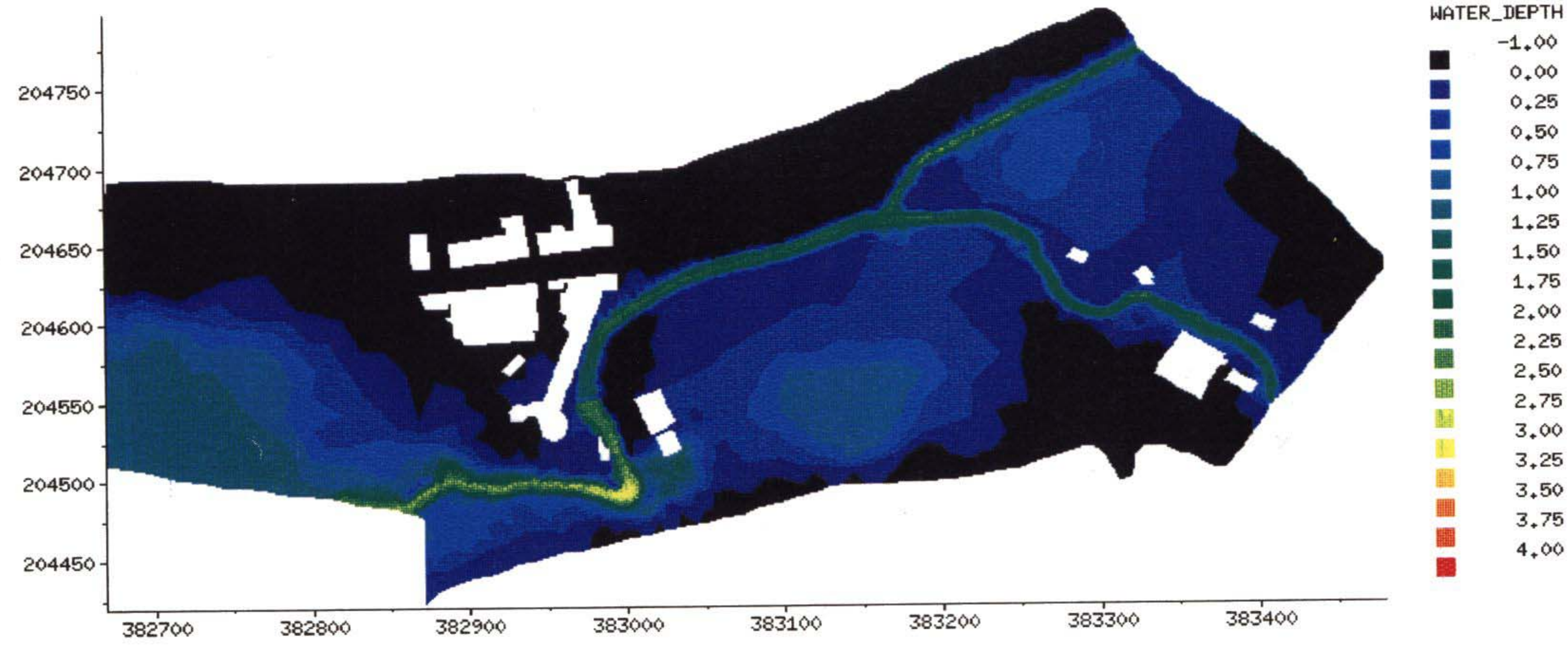

Figure 5.13: Case Study 6: Water depth for 40 cumec scenario

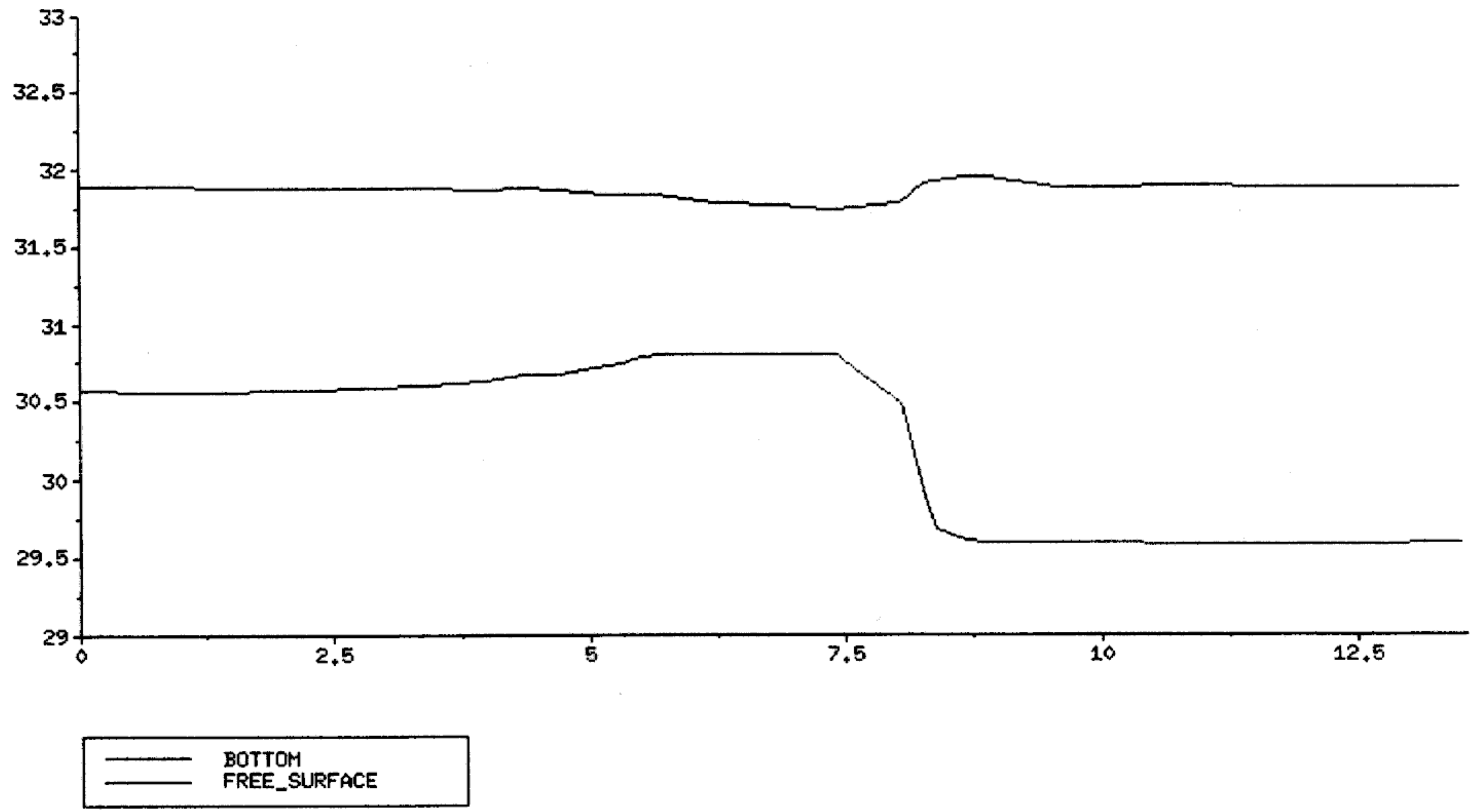

Figure 5.14: Case Study 6: Water surface profile for 40 cumec scenario

Figure 5.15: Case Study 6: Velocity lines for 40 cumec scenario

5.4.5 Case Study 7: 3-D modelling of a non-standard compound broad crested weir with drowning

Understand the hydraulics

The compound broad-crested weir has three crests with no divide piers; the low crest is 12.31m wide with a crest elevation of 14.76m AOD; the high crests are 11.62m wide in total with a crest elevation of 15.22m AOD. The weir is located in an incised river channel that has a width of 24m at structure full level

The channel opens out from a width of 24m and a level of 16.2m AOD, at the top of the weir wing walls, to a width of 37m at 18.3m AOD. A horizontal berm at 18.3m AOD opens the channel further to a width of 41m before the channel slopes upwards to give a top width of 50m at an elevation of 20m AOD. The weir is the same as that used in Case Study 5, and the cross-section is shown on Figure 5.6.

The approach channel to the weir is straight with the channel showing a sinuous plan form downstream of the weir. The bed slope of the river channel upstream of the weir is approximately 1 in 700. Downstream of the weir, for a distance of approximately 400m, the profile of the river bed is uneven with an average bed slope close to horizontal. Beyond 400m downstream of the weir the bed slope increases significantly to approximately 1 in 200.

The weir is modular to a stage and discharge of approximately 16.5m AOD and 100 m^3/s. Above a water surface level of 16.5m AOD the weir starts to drown and becomes fully drowned at a water surface level of approximately 18m AOD.

The flow remains almost entirely in-bank up to the 100-year flow which is of the order of 400 cumecs.

The biggest difficulty associated with the hydraulics at the site is the fact that the flow gaugings show very little increase in flow between stages 16.5 and 18m AOD, see Figure 5.18. Reasons for this could be:

- Drowning of the structure. The pattern of drowning will be complex as the separate weir sections will drown at different flows. In addition, the pattern of gaugings suggests a variable downstream control which is causing drowning at different stages, for example vegetation growth or the operation of control gates. From local knowledge of the reach downstream of the weir, there are no control structures down stream that could possibly exert an influence on the drowning characteristics of the weir. However from discussion with Agency staff in the region, it seems that the channel downstream of the weir is not maintained so a seasonal variation in vegetation and thus channel roughness is to be expected. The consequence of this is that the rating curve cannot be single valued in the drowned flow regime – a fact which is borne out by the scatter in the spot flow gaugings; and
- Interference between slow moving flow on the sloping bank and faster moving flow in the main channel. The discontinuity in the gauged data begins when the stage is about 0.5m above the structure-full level, suggesting that it could be caused by this interaction. However the differences are greater than would be expected from this source.

The analysis suggests that drowning and seasonal variation in channel roughness is the cause of the scatter in the spot flow gauging data.

Collate data
The Case Study was selected because it was a part of an existing model, and therefore all the required data were available. The data collated for the Case Study included:

- The current rating curve for the site (see Table 5.19 below);
- Current meter gaugings at the site. These were taken between 1993 and 2000 and used to rate the site, and are given in Appendix E;
- A rating curve at the downstream boundary, from existing model results; and
- River cross-sections and structure details, all taken from the existing model.

The site started operating as a rated structure in 1992 and seven ratings have been used since. The rating used at the time this report was produced is given in Table 5.19 below.

Table 5.19: Rating for Case Study 7

		Rating Reference 12R $Q = C\,(h + a)^{\beta}$		
Segment	**Maximum Stage**	***C***	***a***	***β***
Segment 1	0.477	24.1602	0.000	1.4568
Segment 2	1.485	32.234	0.000	1.8461
Segment 3	4.014	27.738	0.000	2.2258

The zero of the staff gauge reading is set at 14.87m AOD.

Rating curve strategy
The main factors affecting the rating curve strategy are as follows:

- Flows contained within the channel and no floodplain flow;
- No spills or storage areas that could distort the flood hydrograph; and
- No dynamic effects caused by the tide or gate operation.

A steady flow approach was adopted in accordance with the guidance given in Section 5.3.2.

Modelling software
The FLOW-3D software was used to construct the Case Study model. Only the FLOW-3D software was applied during this case study, however, comparative results were available from the modelling carried out in Case Study 5.

Model boundaries
The upstream model boundary was about 60m upstream of the weir.

This could have been located further away but this would have required a larger mesh and thus slowed down the model runs. Also the channel upstream of the weir is relatively uniform so there is little benefit in extending the model further upstream.

The downstream boundary was located approximately 100m downstream of the weir.

As for the location of the upstream boundary, this was selected for pragmatic reasons and coincided with one of the measured cross-sections on the river.

Model boundary conditions
Upstream inflows were increased from zero to the required flow for each scenario to reach a steady state solution.

The downstream boundary consisted of a fixed water level, taken from the 1-D stage discharge curve at the specified flow. In the absence of an existing model, the stage discharge curve would be generated by a conventional normal depth hydraulic calculation.

Topography, survey requirements and grid generation
Cross-sections were located upstream and downstream of structures. The spacing was about 10m in the open channel. This spacing was used because the data were available from the existing model. It is understood that the close spacing was considered necessary to identify variations in cross-section shape of the channel. However, if a new survey had been needed the number of cross-sections could be halved

No new survey work was needed because the data were taken from an existing model. The computational grid is shown on Figures 5.16 and 5.17.

Set up, calibrate and validate the model
A 3-D model requires no structure coefficients or modular limit values to facilitate the required calculations. The only empirical parameters required for the model were the surface roughness values required by the software. Initial values of *ks* are estimated as described in Section 5.3.6, but where possible *ks* values should be obtained by calibration.

Generate 'best estimate' rating curve
The rating curve was generated using a series of steady flows without any imposed surface roughness, and the results are shown in Figure 5.18. For flows and stages less than approximately 100 m^3/s and 16.5m AOD respectively the predicted rating accurately represents the spot flow gaugings without any attempt to calibrate the model.

Between stages of 16.5 to 18m AOD the gauged data do not correlate to a single curve and a range of levels have been measured at very similar flows. For example, for a flow range of just 8 cumecs between 122 and 130 cumecs, the gauged depth readings vary between 1.78m and 3.1m which is much larger that might be expected. It is also notable that the measurements were taken at different times of the year, adding further weight to the hypothesis that the variation may be due to seasonal variations in channel roughness.

Further runs were carried out using alternative values of channel roughness and downstream water level.
The blue triangles on the upper part of the flow regime in Figure 5.18 correspond to alternative downstream boundary conditions obtained from the one-dimensional model, at a *ks* value of 0.4. This appears to pass through approximately the trend line of the measured data, indicating that the 3-D model can represent the drowned flow regime, again without any specification of modular limit. Again it is expected that alternative curves would be expected at different values of surface roughness *ks*.

The '*ks*=0.4' curve in Figure 5.18 can be seen to correlate approximately to the HEC-RAS and MIKE11 curves. Indeed all of the 1-D models should produce similar curves and it is not clear why the ISIS curve should differ significantly from the other two packages. Once more it is worth emphasising that the one-dimensional models were calibrated in order to achieve these results whereas the 3-D model was not. Consequently the 3-D model could be used with much greater confidence than a 1-D model to extrapolate or indeed produce from scratch a stage discharge relationship at high flows.

It is evident from Figure 5.18 that:

- The Environment Agency rating does not take account that the stage discharge relationship is not single valued at flows higher than about 100 cumecs; and
- The models do not satisfactorily reproduce the rating curve based on flow gaugings unless variable roughness is considered.

Sensitivity tests and uncertainties

Sensitivity tests
Sensitivity tests were undertaken using the FLOW-3D model to assess the effect on the predicted rating of changing the turbulence model from laminar to a single turbulent energy equation and also to a k-epsilon turbulence model. The results from these simulations are shown in Table 5.20.

The initial runs (1-11) were carried with zero roughness which show very good agreement with the observed spot flow gaugings below 100 cumecs. This is encouraging as friction should exert a significant influence on the head upstream of the weir in these cases.

The second set of runs (12-31) were carried out to examine the sensitivity to channel surface roughness *ks*. These show that for values between 0.1 and 0.5 the results do not appear to differ greatly but the difference between the curves for each value of ks does increase for flows greater than 100 cumecs. There also appears to be a somewhat greater than expected difference (~150mm) between the zero roughness case and the *ks*=0.1 curve.

Runs 32-35 examined the sensitivity to downstream water level. In all previous runs, no downstream level was set and the software calculated a profile which conformed to a free discharge condition over the weir. Initially the normal depth downstream water level was calculated using a 1-D model and these values were imposed in runs 32-35 for flows greater than or equal to 120 cumecs and with a *ks* of 0.4. These results appeared to correlate well with the trend line of the spot flow gaugings in excess of 120 cumecs. It is further expected that the range of spot flow gaugings would be covered better by a set of runs which varied ks according to the seasonal range of channel roughness.

A more systematic variation of downstream level was carried out in runs 36-46. These showed that the calculated water level at the weir is sensitive to downstream water level at high flows. These results are to be expected from hydraulic theory.

The influence of the choice of turbulence was also investigated in the final set of runs (47-54) at a flow of 210 cumecs for both an un-set downstream water level and a fixed downstream water level at two values of *ks* (0.05 and 0.1). These tests showed that the calculated water levels upstream of the weir were not significantly influenced by the choice of turbulence model.

Figures 5.19 to 5.24 illustrate the model output for some of the model runs.

Table 5.20: Case Study 7 Results

Nr	Flow (Cumecs)	Tail (m AOD)	Head (m AOD)	Ks (m)	Turbulence Model	Observations
01	30	Not Set	15.822	0.0	Laminar	
02	60	Not Set	16.224	0.0	Laminar	
03	90	Not Set	16.626	0.0	Laminar	
04	120	Not Set	16.713	0.0	Laminar	
05	150	Not Set	16.958	0.0	Laminar	
06	180	Not Set	17.133	0.0	Laminar	
07	210	Not Set	17.327	0.0	Laminar	
08	240	Not Set	17.545	0.0	Laminar	
09	270	Not Set	17.683	0.0	Laminar	
10	300	Not Set	17.813	0.0	Laminar	
11	330	Not Set	18.072	0.0	Laminar	
12	30	Not Set	15.971	0.1	Laminar	
13	30	Not Set	15.971	0.2	Laminar	
14	30	Not Set	15.971	0.3	Laminar	
15	30	Not Set	15.971	0.4	Laminar	
16	30	Not Set	15.971	0.5	Laminar	
17	90	Not Set	16.627	0.1	Laminar	
18	90	Not Set	16.669	0.2	Laminar	
19	90	Not Set	16.692	0.3	Laminar	
20	90	Not Set	16.716	0.4	Laminar	
21	90	Not Set	16.723	0.5	Laminar	
22	150	Not Set	17.092	0.1	Laminar	
23	150	Not Set	17.144	0.2	Laminar	
24	150	Not Set	17.168	0.3	Laminar	
25	150	Not Set	17.183	0.4	Laminar	
26	150	Not Set	17.192	0.5	Laminar	
27	210	Not Set	17.577	0.1	Laminar	
28	210	Not Set	17.620	0.2	Laminar	
29	210	Not Set	17.606	0.3	Laminar	
30	210	Not Set	17.602	0.4	Laminar	
31	210	Not Set	17.596	0.5	Laminar	
32	120	17	17.294	0.4	Laminar	D/s head fixed at normal depth
33	150	17.4	17.671	0.4	Laminar	D/s head fixed at normal depth
34	180	17.8	18.021	0.4	Laminar	D/s head fixed at normal depth
35	210	18	18.232	0.4	Laminar	D/s head fixed at normal depth
36	210	14	17.553	0.05	Laminar	
37	210	16	17.572	0.05	Laminar	
38	210	18	18.250	0.05	Laminar	
39	210	14	17.547	0.1	Laminar	
40	210	16	17.546	0.1	Laminar	
41	210	18	18.199	0.1	Laminar	

Table 5.20: Case Study 7 results (continued)

Nr	Flow (Cumecs)	Tail (m AOD)	Head (m AOD)	Ks (m)	Turbulence Model	Observations
42	210	14	17.666	0.4	Laminar	
43	210	15	17.614	0.4	Laminar	
44	210	16	17.685	0.4	Laminar	
45	210	17	17.819	0.4	Laminar	
46	210	18	18.232	0.4	Laminar	
47	210	Not Set	17.578	0.05	Turbulent energy	
48	210	Not Set	17.589	0.05	k-epsilon	
49	210	16	17.598	0.05	Turbulent energy	
50	210	16	17.624	0.05	k-epsilon	
51	210	Not Set	17.607	0.1	Turbulent energy	
52	210	Not Set	17.607	0.1	k-epsilon	
53	210	16	17.600	0.1	Turbulent energy	
54	210	16	17.625	0.1	k-epsilon	

Uncertainties
The model calibration for in-channel flows is good and the uncertainty associated with this part of the rating should be within 10%.

While the current study did not address the issue of heterogeneous roughness, this could have a significant effect, particularly at high flows when the inundated area extends onto the sloping berms of the river.

Applicability of rating
The role of vegetative roughness should be further investigated in order to determine a seasonally dependent family of stage/discharge curves for the drowned flow regime for this particular weir.

This observation also applies to any flow gauging site where the hydraulic control is the channel roughness which may vary significantly due to a non-existent or ad hoc maintenance regime.

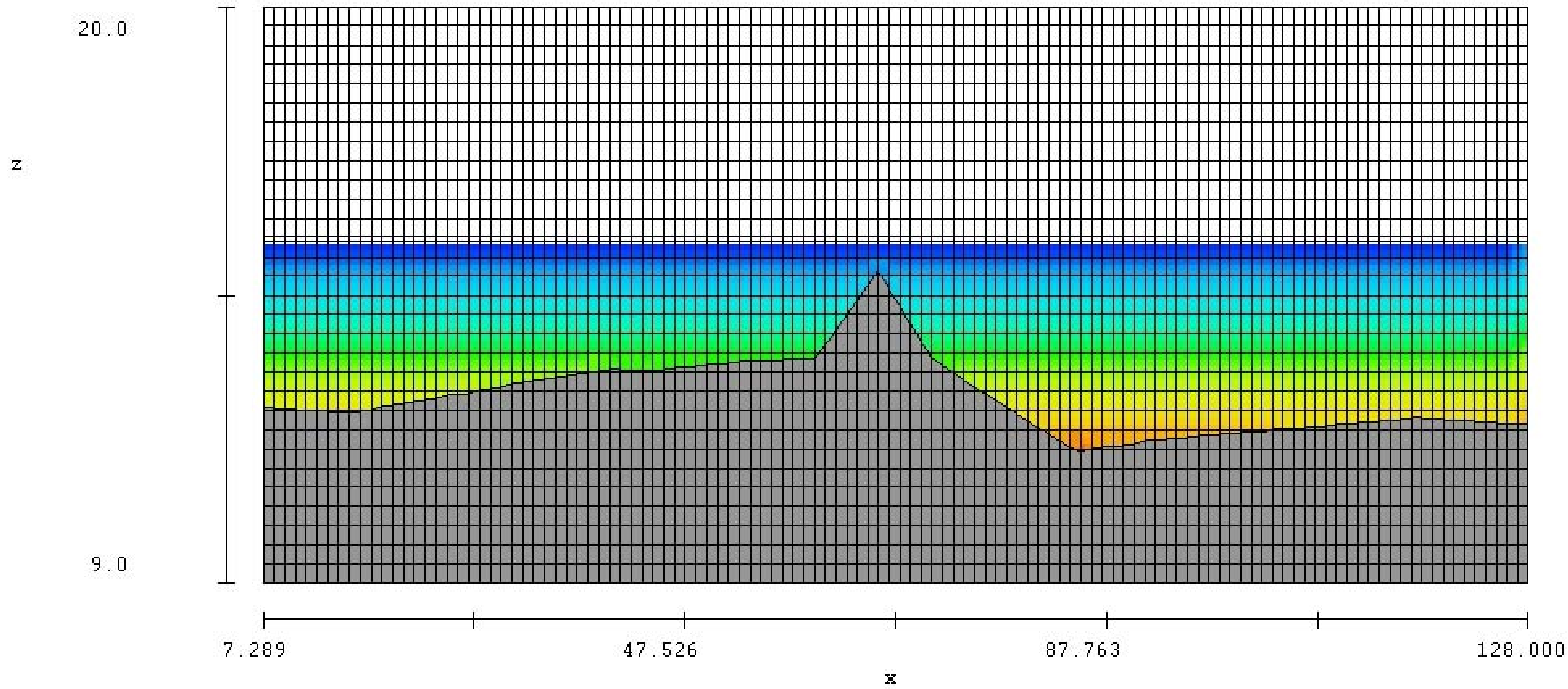

Figure 5.16: Case Study 7: Computational grid – Section through a longitudinal plane

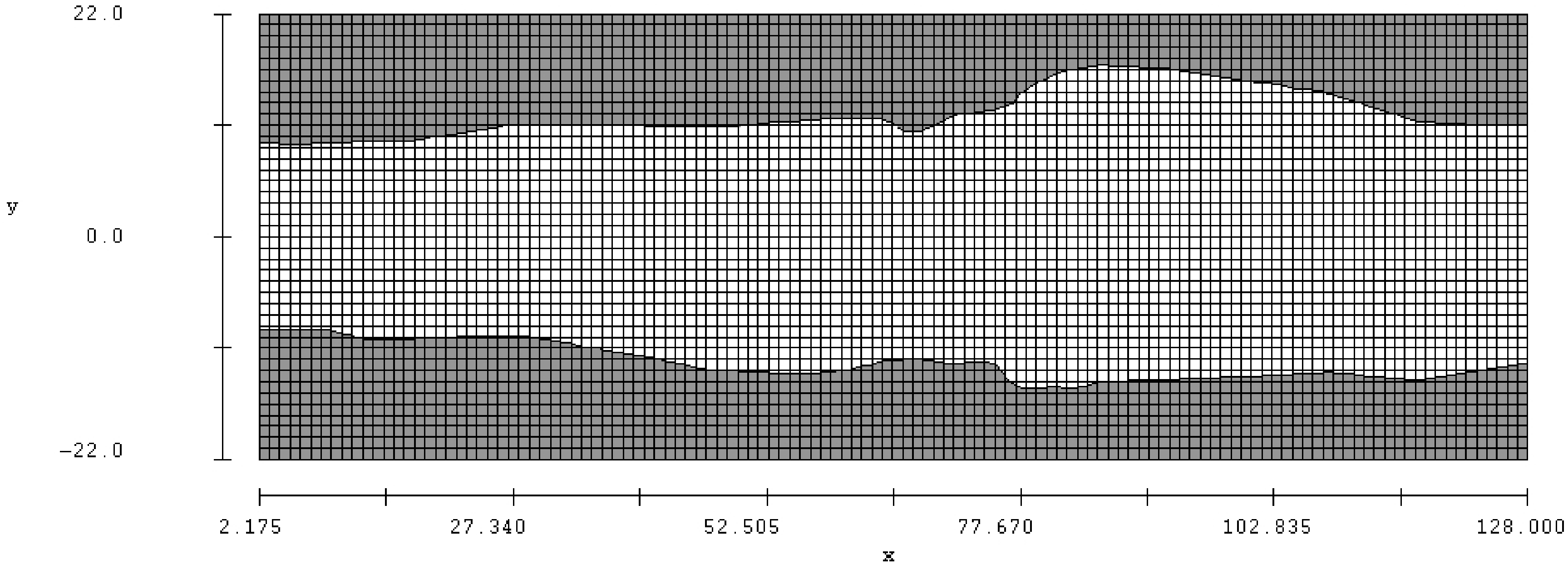

Figure 5.17: Case Study 7: Computational grid – Plan View

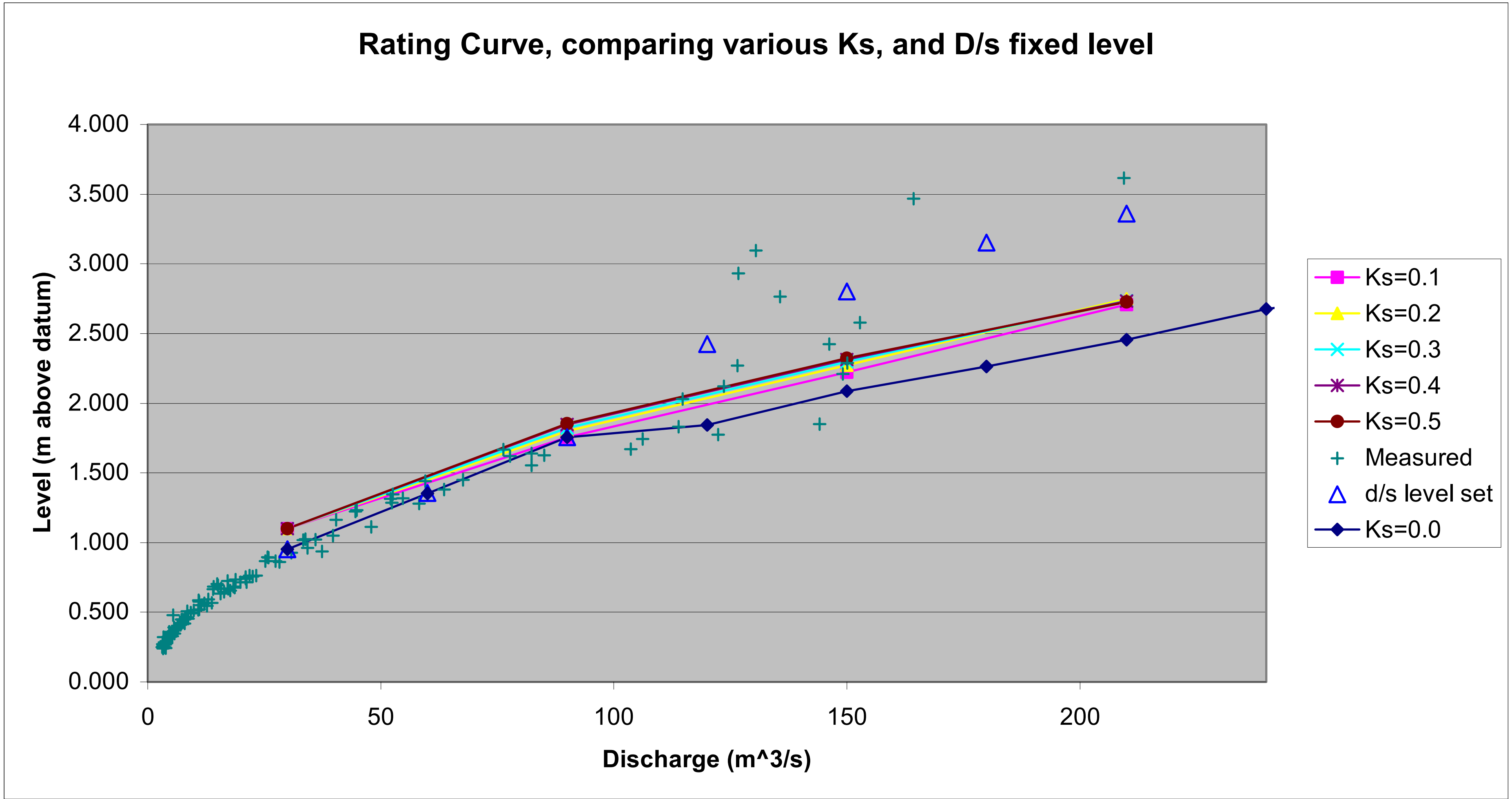

Figure 5.18: **Case Study 7: Comparison of Environment Agency and modelled rating curves**

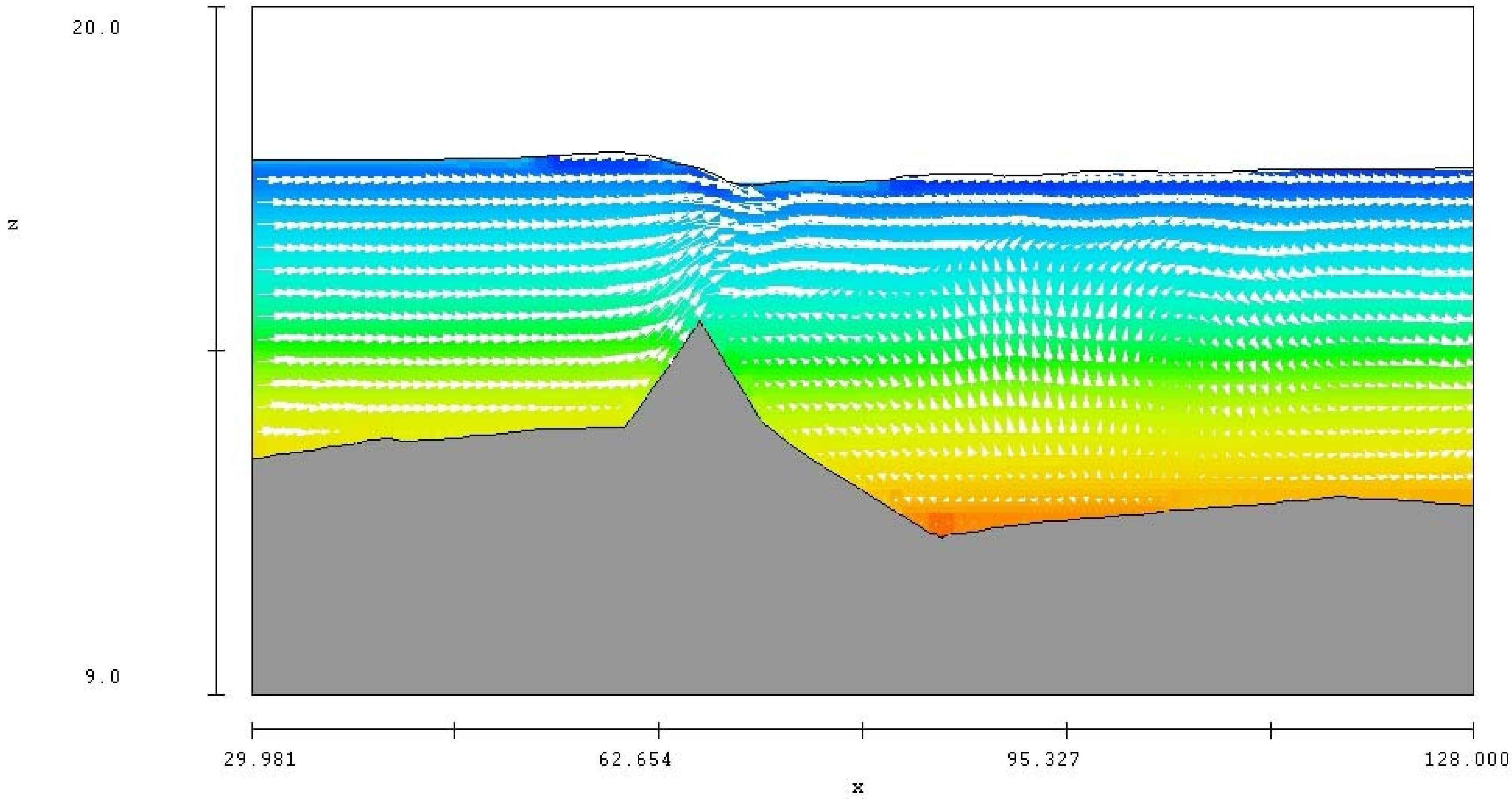

Figure 5.19: Case Study 7: Longitudinal water surface profile and velocity vectors from Run No 33

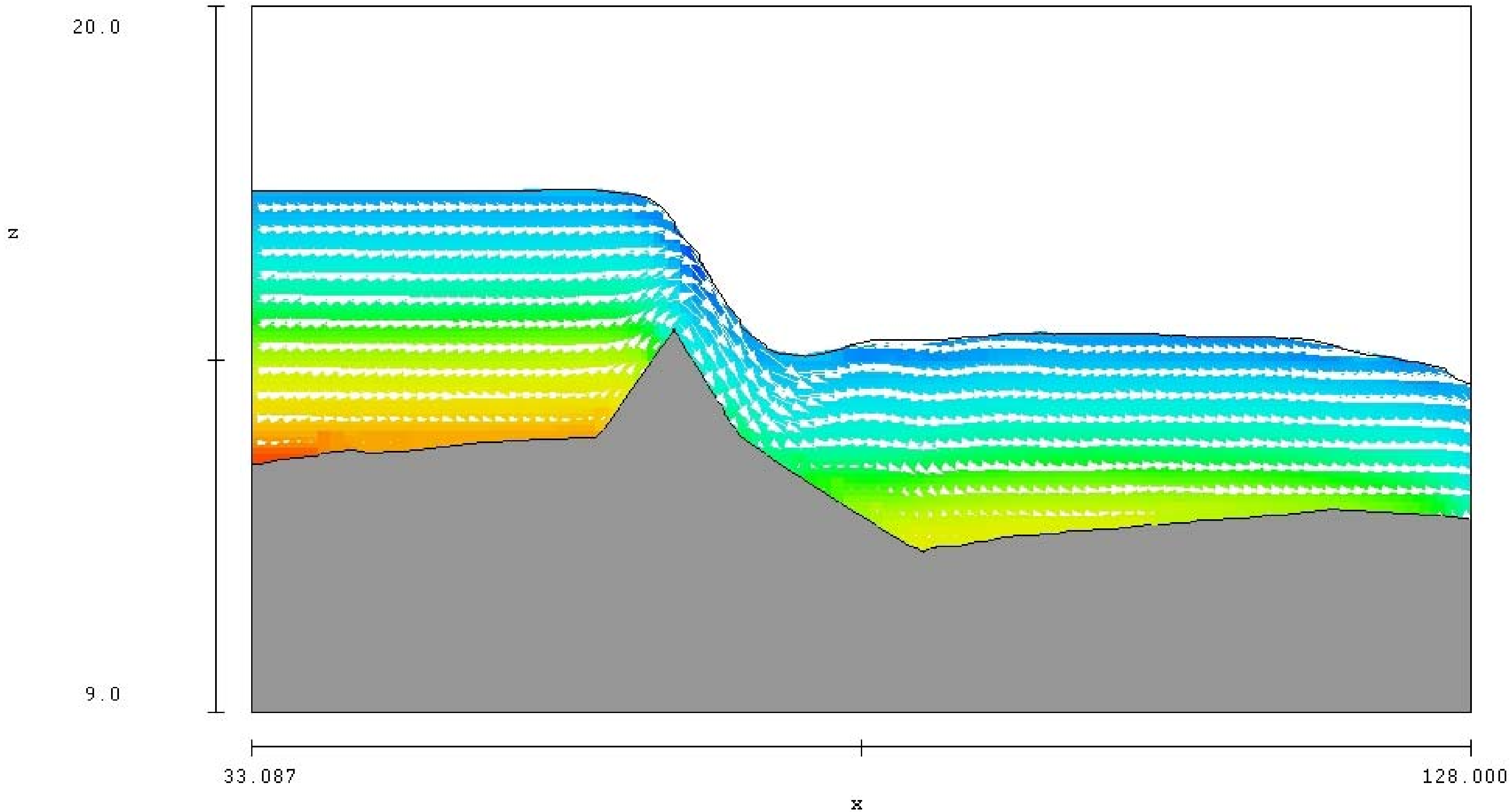

Figure 5.20: Case Study 7: Longitudinal water surface profile and velocity vectors from Run No 25

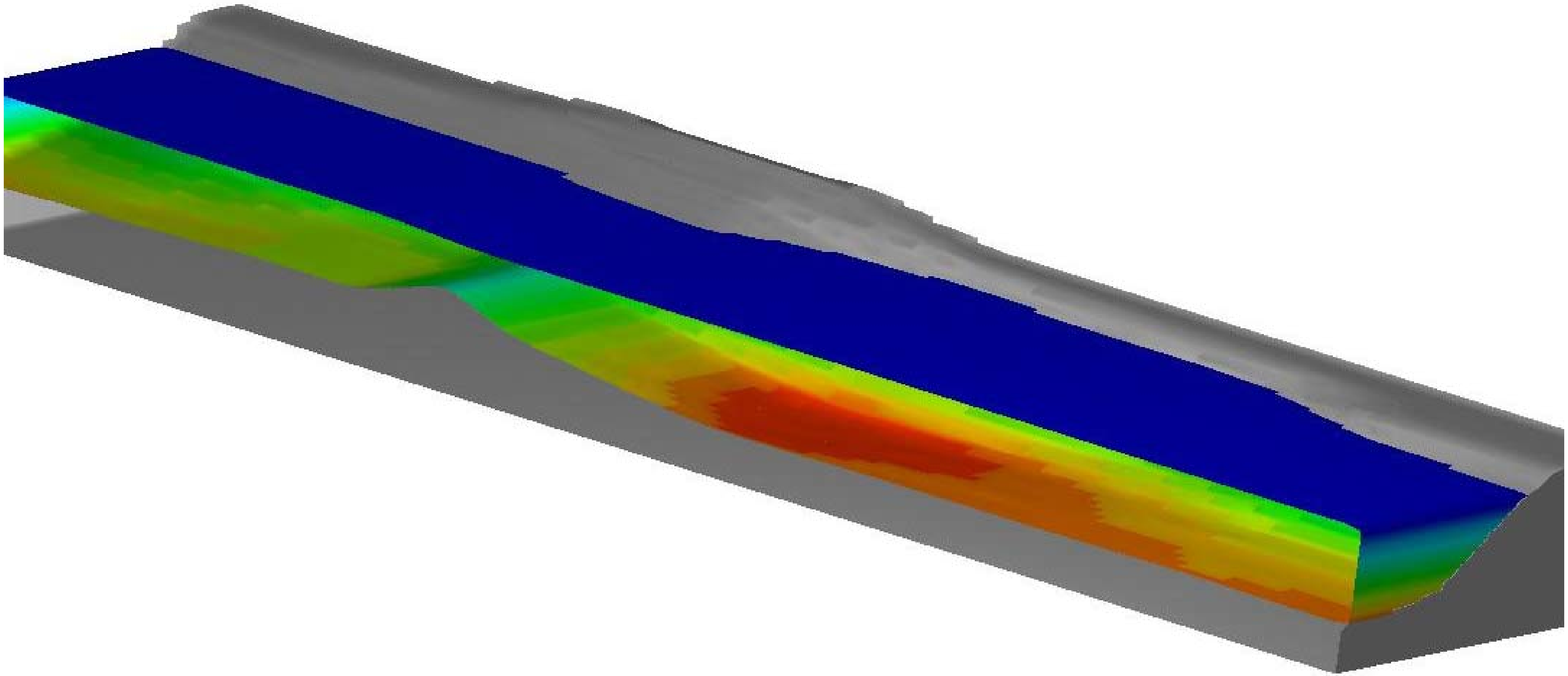

Figure 5.21: Case Study 7: 3-D Section from Run 33

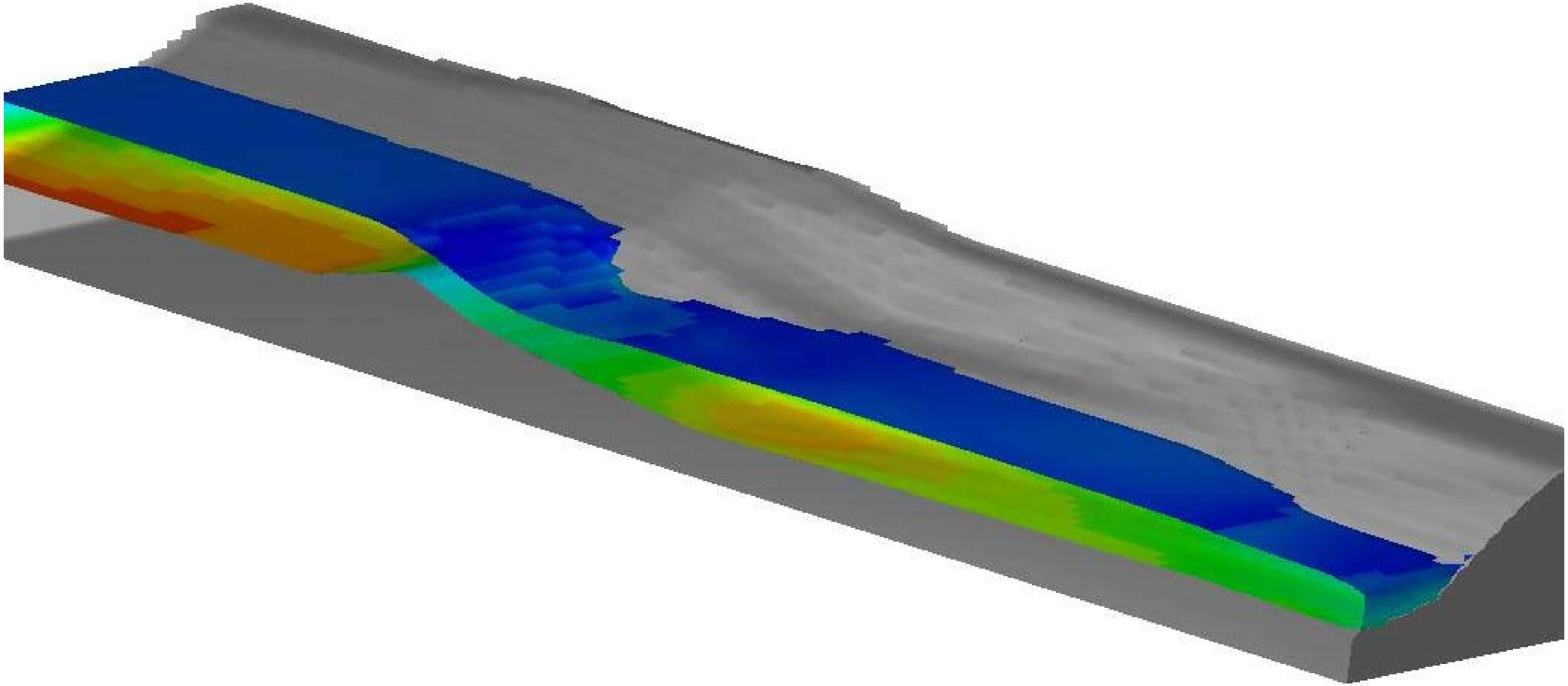

Figure 5.22: Case Study 7: 3-D Section from Run 25

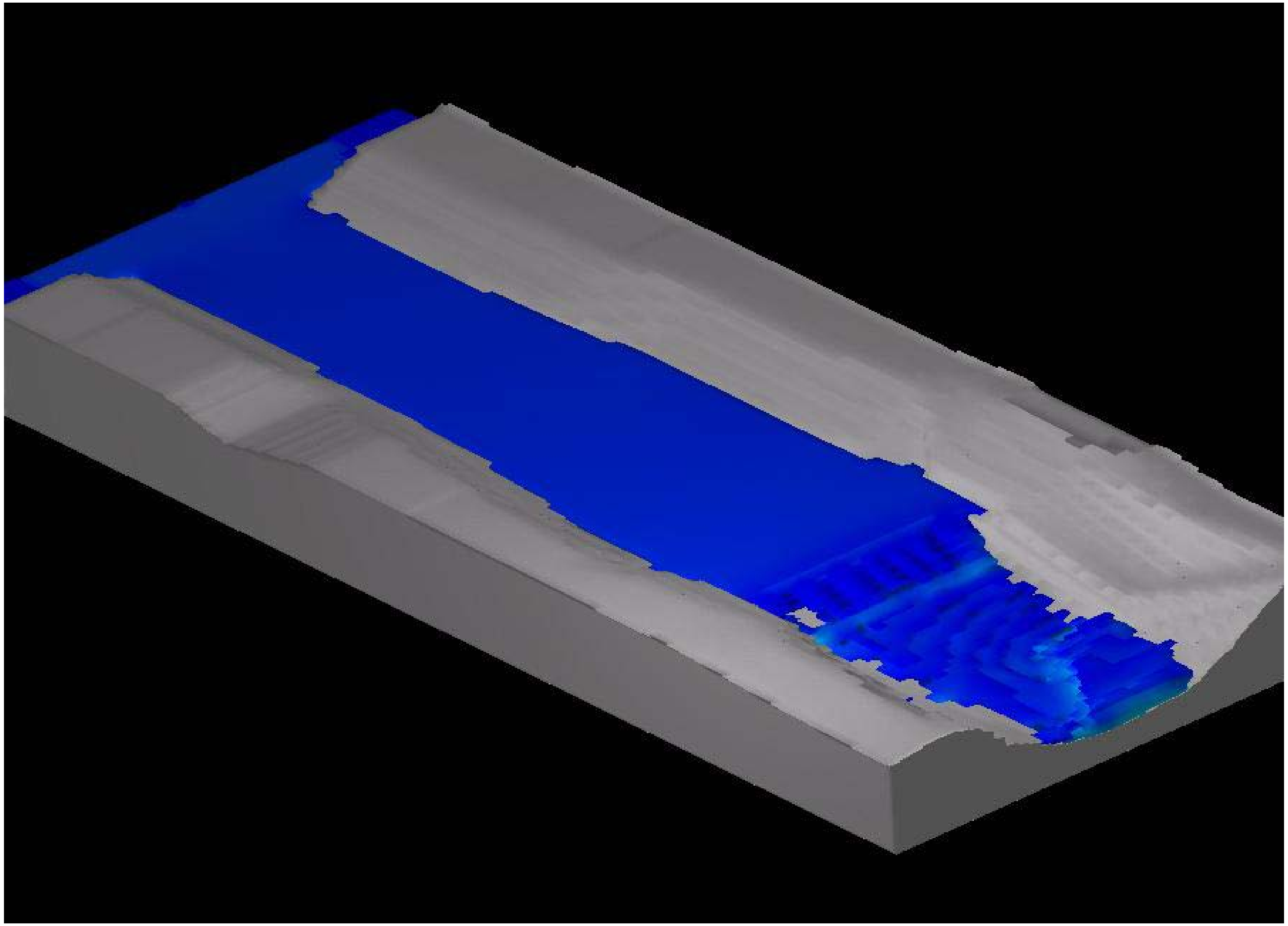

Figure 5.23: Case Study 7: 3-D Surface plot from Run 02 (60 Cumecs)

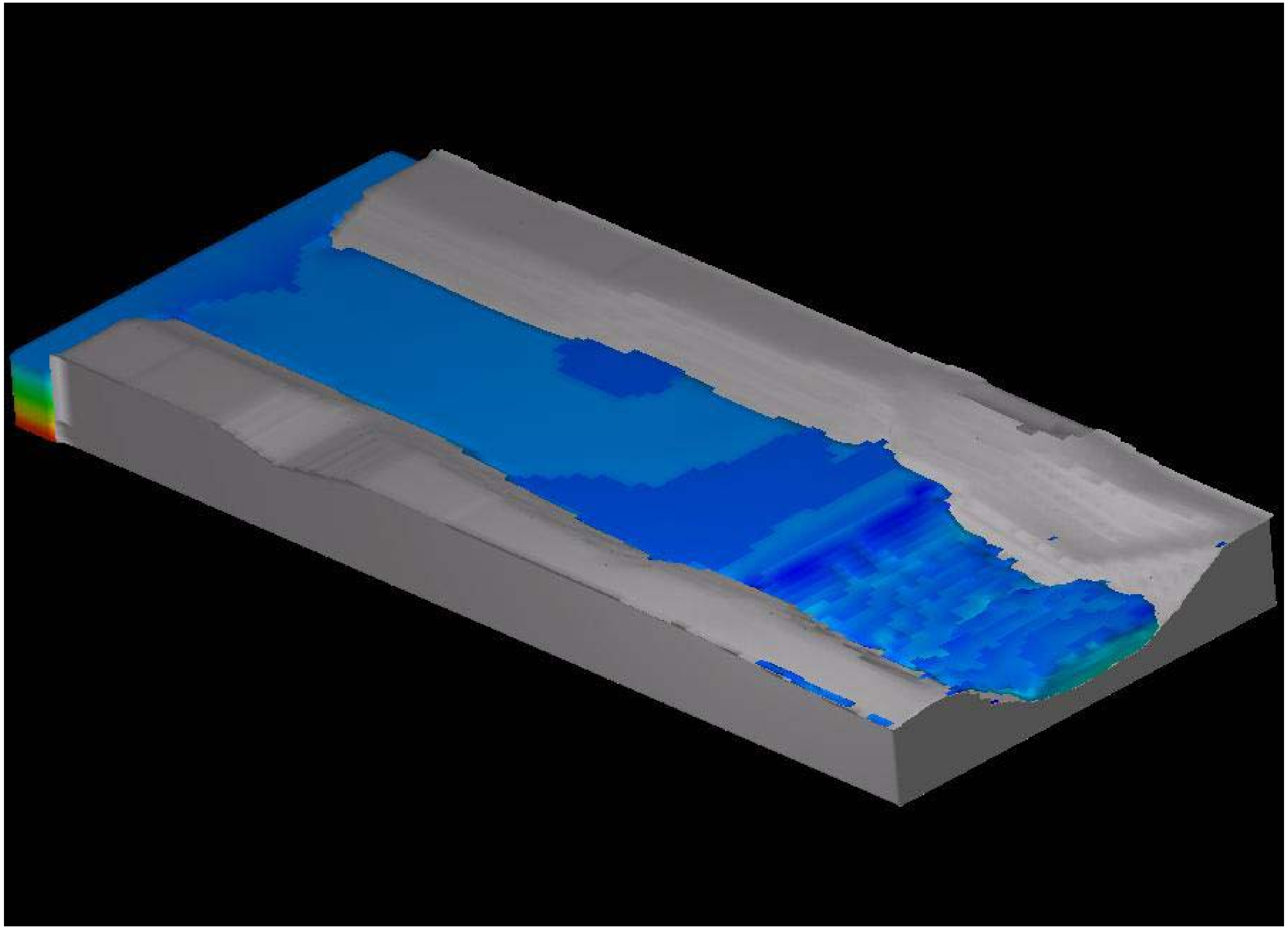

Figure 5.24: Case Study 7: 3-D Surface plot from Run 04 (120 Cumecs)

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Background on the hydraulics of structures

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APPENDICES

APPENDIX A

A.1 Methods of estimating Manning's *n* values

Table A.1: Manning's n values (Chow, 1959)

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 polls	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. Floodplains			
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

Table A.1: Manning's n values (Chow, 1959) (Continued)

Type of Channel and Description	Minimum	Normal	Maximum
B. Lined or Built-Up Channels			
1. Concrete			
a. Trowel finish	0.011	0.013	0.015
b. Float finish	0.013	0.015	0.016
c. Finished, with gravel bottom	0.015	0.017	0.020
d. Unfinished	0.014	0.017	0.020
e. Gunite, good section	0.016	0.019	0.023
f. Gunite, wavy section	0.018	0.022	0.025
g. On good excavated rock	0.017	0.020	
h. On irregular excavated rock	0.022	0.027	
2. Concrete bottom float finished with sides of:	0.015	0.017	0.020
a. Dressed stone in mortar	0.017	0.020	0.024
b. Random stone in mortar	0.016	0.020	0.024
c. Cement rubble masonry, plastered	0.020	0.025	0.030
d. Cement rubble masonry	0.020	0.030	0.035
e. Dry rubble or riprap			
3. Gravel bottom with sides of:			
a. Formed concrete	0.017	0.020	0.025
b. Random stone in mortar	0.020	0.023	0.026
c. Dry rubble or riprap	0.023	0.033	0.036
4. Brick			
a. Glazed	0.011	0.013	0.015
b. In cement mortar	0.012	0.015	0.018
5. Metal			
a. Smooth steel surfaces	0.011	0.012	0.014
b. Corrugated metal	0.021	0.025	0.030
6. Asphalt			
a. Smooth	0.013	0.013	
b. Rough	0.016	0.016	
7. Vegetal lining	0.030	Variable	0.500

Table A.1: Manning's n values (Chow, 1959) (Continued)

Type of Channel and Description	Minimum	Normal	Maximum
C. Excavated or Dredged Channels			
1. Earth, straight and uniform			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. Earth, winding and sluggish			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble side	0.028	0.030	0.035
e. Stony bottom and weedy banks	0.025	0.035	0.040
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline – excavated or dredged			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. Rock cuts			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. Channels not maintained, weeds and brush	0.040	0.050	0.080
a. Clean bottom, brush on sides	0.045	0.070	0.110
b. Same as above, high as flow depth	0.050	0.080	0.120
c. Dense weeds, high as flow depth	0.080	0.100	0.140
d. Dense brush, high stage			

A.2 The SCS Method

An alternative approach to estimating the roughness coefficient is provided by the Soil Conservation Service (SCS) method. This method involves the selection of a basic *n* value (***n***$_0$) for a uniform, straight, and regular channel in a native material and then modifying this value by adding a correction factors determined by consideration of:

- **Degree of channel irregularity (*n*$_1$).** The degree of irregularity is considered smooth for surfaces comparable to the best attainable for the materials involved; minor for good dredged channels, slightly eroded or scoured side slopes of canals or drainage channels; moderate for fair to poor dredged channels, moderately sloughed or eroded side slopes of canals or drainage channels; and severe for badly sloughed banks of natural streams, badly eroded or sloughed sides of canals or drainage channels, and unshaped, jagged, and irregular surfaces of channels excavated in rock.

- **Variations of channel cross-section (*n*$_2$).** The character of variations in size and shape of cross-section is considered gradual when the change in size or shape occurs gradually, alternating occasionally when large and small sections alternate occasionally or when shape changes cause occasional shifting of main flow from side to side, and alternating frequently when large and small sections alternate frequently or when shape changes cause frequent shifting of main flow from side to side.

- **Relative effect of obstructions (*n_3*).** This correction factor is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders, and fallen and lodged logs. The relative effect of obstructions may be judged by the extent to which the obstructions occupy or reduce the average water area, the character of obstructions (sharp-edged or angular objects induce greater turbulence than curved, smooth-surfaced objects), and the position and spacing of obstructions transversely and longitudinally in the reach under consideration.

- **Vegetation (*n_4*).** The retardance due to vegetation is primarily due to the flow of water around stems, trunks, limbs, and branches and only secondarily to the reduction of the flow area. In assessing the effect of vegetation on retardance, consideration must be given to the height of vegetation in relation to the flow depth, the capacity of the vegetation to resist bending, the degree to which the flow is obstructed, the transverse and longitudinal distribution of vegetation of various types, the densities and heights of vegetation in the reach being considered, and the season (i.e. is the vegetation dormant or growing).

  The degree of effect of vegetation is considered:

  low for dense growths of flexible grass or weeds where the average depth of flow is 2 to 3 times the height of vegetation and for tree saplings where the depth of flow is 3 to 4 times the height of the vegetation.

  medium for flexible grass where the average depth of flow is 1 to 2 times the height of vegetation; stemmy grasses, weeds, or tree seedlings with moderate cover where the average depth of flow is 2 to 3 times the height of the vegetation; brushy growths, moderately dense, during the dormant season, along side slopes of channel with no significant vegetation along the channel bottom where the hydraulic radius is greater than 0.6m.

  high during the dormant season for trees, inter-grown with weeds and brush, where the hydraulic radius is greater than 0.6m; and during the growing season for bushy growth in foliage along the side slopes of channels where the hydraulic radius is greater than 0.6m.

  very high for flexible grass where the average depth of flow is less than half the height of the vegetation; during the growing season dense brush and bush growth inter-grown with weeds along the side slopes of channels, dense vegetation growth along the channel bottom where the hydraulic radius is up to 3 to 4.6m; trees during the growing season inter-grown with weeds and brush where the hydraulic radius is up to 3 to 4.6m.

- **Degree of meandering (*m_5*).** The correction for the degree of meandering depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered minor for ratios of 1 to 1.2, appreciable for ratios of 1.2 to 1.5, and severe for ratios of 1.5 and greater.

Based upon the above the value of n for either channel or floodplain may be estimated by:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5$$

and the basic n value and n correction assessed by reference to the values given in the table below:

Table A.2: Values for the computation of Manning's n (SCS Method)

Channel conditions		Values	
Material involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross-section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010 – 0.015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010 – 0.015
	Appreciable		0.020 – 0.030
	Severe		0.040 – 0.060
Vegetation	Low	n_4	0.005 – 0.010
	Medium		0.010 – 0.025
	High		0.025 – 0.050
	Very high		0.050 – 0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

A.3 Additional Information on Floodplain Roughness

Additional information on roughness coefficients for floodplains with grass cover or grass cover separated by hedgerows as determined by Klaasen and Van der Zwaard, 1974 are given in Tables A.3 and A.4 below:

Table A.3: Roughness coefficients for floodplains with grass cover

Flow depth (m)	Manning's *n*
0.10	0.031
0.25	0.027
0.50	0.026
1.00	0.025
2.00	0.025

Table A.4: Roughness coefficients for floodplains with hedges

Hedge sep'n (m)	Manning's *n* roughness coefficient for clean or dirty hedges with varying flow depth									
	0.25m		0.50m		1.00m		1.50m		2.00m	
	Clean	Dirty	Clean	Dirty	Clean	Dirty	Clean	Dirty	Clean	Dirty
50	0.038	0.072	0.045	0.089	0.053	0.091	0.054	0.086	0.051	0.080
100	0.032	0.053	0.032	0.064	0.042	0.067	0.042	0.063	0.041	0.060
250	0.029	0.040	0.031	0.045	0.029	0.045	0.032	0.045	0.032	0.041
500	0.028	0.035	0.028	0.036	0.029	0.037	0.027	0.036	0.029	0.034
1000	0.027	0.031	0.027	0.032	0.027	0.031	0.025	0.029	0.027	0.027

A.4 Jarrett's Method for Steep Streams

Jarrett (1984) developed the following relationship for determining the Manning's *n* coefficient for steep streams with stable bed and bank materials in gravel, cobbles or boulders. The equation can be used for streams with slopes between 0.002 and 0.04.

$$n = 0.39\, S^{0.38}\, R^{0.16}$$

where

S = friction slope; the water surface slope may be used if the friction slope is unknown
R = hydraulic radius; relationship valid for hydraulic radii between 0.15m and 2.1m.

APPENDIX B

Application of Methods to Case Study 1: Non-Standard Weir

B.1 Site Description

The Non-Standard Weir is a compound structure of three broad crested weirs. The lower weir is 12.31m wide with a minimum crest elevation of 14.76m OD, whilst the total width of the other weirs is 11.62m at an elevation of 15.22m OD. The length of the weir parallel to the direction of flow is assumed to be 1.0m. The minimum left and right bank elevations upstream of the weir are 20.07m OD giving a maximum head above the weir crest of 5.31m before overbank flow occurs.

Three ratings have been used with the rating maximum increasing with each rating (Table B.1). The ratings and available gaugings are shown on Figure B.1.

Table B.1: Ratings

Rating	*C*	*a*	*β*	Min Stage	Max Stage	Start Date	End Date
SWR067101	24.604	0.072	2.1449	n/a	0.426	n/a	n/a
	34.411	-0.142	1.4564	0.427	0.914		
	85.045	-0.676	0.8957	0.915	1.036		
SWR047702	19.539	-0.003	1.4871	n/a	0.448	n/a	19/10/93
	28.463	-0.006	1.9348	0.449	1.500		
	32.963	-0.189	2.3297	1.501	2.000		
GM109303	24.1602	0.0	1.4568	n/a	0.477	19/10/93	Date
	32.234	0.0	1.8461	0.478	1.485		
	27.738	0.0	2.2258	1.486	4.014		

B.2 Simple Extension

Simple extension involves extending the rating equation above its stated upper limit. Figure B.2 shows the two historic and the existing rating extended from the relevant rating maximum to the bankfull limit of 5.31m. A key feature to note is the change in the slope of the gaugings and a departure from the rating at about 150 cumecs. As indicated on Figure B.2 the earlier rating SWR067101, which has a stated maximum of 1.036m, shows a departure from the other 2 ratings when extended to 5.310m because it has a significantly higher curvature when plotted on log-log paper (large '*a*' value of -0.676) compared to rating SWR047702, which has an '*a*' value for the upper segment of -0.189, and rating GM109303 which has an 'a' value of 0.0 and therefore less curvature in the segment and its extension.

B.3 Logarithmic Extrapolation

With Logarithmic Extrapolation the stage and flows are converted to logarithmic values and plotted as an extended line on arithmetic paper and an extended best-fit line fitted to all data points. Excel provides a convenient method of fitting best-fit lines using the linear trend line function in “charts”. If the extension is based on a best-fit line through all data points the coincidence of the rating and extended section and the junction between the two curves will depend on the degree of curvature of the rating. The straighter the rating (i.e. the lower the ‘*a*’ value) the better fit to a straight line is obtained and the more likely the rating and extended section would coincide. If 'a' equals 0 the log extrapolation will provide the same extension as the simple extension method. For this reason log extrapolation has not been applied in this case.

B.4 The Weir Equation

For a rectangular broad-crested weir a rating can be derived based on the relevant formula given in the appropriate British or International Standard. The current Standard is BS3680: Part 4E : 1990 / ISO 3846 : 1989. *Measurement of liquid flow in open channels. Part 4E. Rectangular broad-crested weirs*. More recent data has been published by W R White (2001) which includes some information on the drowned flow performance of the weir. The current Standard is being revised in the light of this new published information.

The Standard shows that the performance of the rectangular broad-crested weir is hydraulically complex and the coefficient of discharge varies with the dimensions of the weir and the instantaneous flow (stage value). Published experimental results do not provide performance data for all flow conditions and all height / length aspect ratios for this type of weir. In view of this the Standard limits the use of flow formulae to specific conditions. The most relevant limitation, as far as this case study is concerned, is the maximum stage / weir length ratio of 1.6.

The length of the test case weir is 1.0m. Hence the Standard performance data is strictly only applicable up to a stage of 1.6m. The in-situ current meter measurements go up to a stage of 3.5m and the channel limit is 5.0m. The use of the Standard performance data is thus only approximate at the upper end of the flow range, say from 100 m^3/s onwards.

Modular Flow

A stage discharge curve has been computed in accordance with the Standard assuming that the weir remains modular up to bankfull. In order to do this the value of weir height above upstream bed level needs to be known. In this case study we have assumed a weir height of 1.0m.

The modular stage discharge curve is given in Figure B.3 and shows a notable departure from the Agency ratings at high flows.

Non-modular flow

A second stage discharge curve has been computed taking into account the possibility of drowning at higher flows. To do this it has been necessary to speculate about the natural stage discharge curve of the channel since this controls water levels downstream of the weir and determines the flow at which drowning commences and the degree of drowning thereafter.

Ideally, a rigorous check on the tailwater rating curve would need to be made. This could either be done in the field, but would present practical difficulties at the high end of the flow range, or using detailed mathematical modelling based upon extensive survey information. The tailwater curve shown in Figure B.4 has been assumed purely for illustrative purposes in this case study.

The revised flow rating indicates that drowning commences, i.e. the modular limit is reached, at around 140 m^3/s. By the time the stage reaches 5.0m there is a 37 per cent reduction from the equivalent modular discharge. The drowned flow is 330 m^3/s against the corresponding modular figure of 530 m^3/s.

Full details of the "best estimate" rating are given in Table B.2. The relevant curves are given in Figure B.4.

Table B.2: Best estimate broad-crested weir rating

Stage (m)	Total flow (cumecs)	Low weir flow (cumecs)	High weir flow (cumecs)	Drowned flow reduction factor (*f*)
0.100	0.555	0.555	0.000	1.000
0.200	1.616	1.616	0.000	1.000
0.300	3.056	3.056	0.000	1.000
0.400	4.840	4.840	0.000	1.000
0.500	7.090	6.950	0.140	1.000
0.600	10.283	9.377	0.906	1.000
0.700	14.193	12.114	2.079	1.000
0.800	18.764	15.155	3.609	1.000
0.900	23.953	18.492	5.461	1.000
1.000	29.760	22.120	7.641	1.000
1.100	36.149	26.027	10.122	1.000
1.200	43.100	30.204	12.896	1.000
1.300	50.590	34.636	15.953	1.000
1.400	58.630	39.312	19.318	1.000
1.500	67.160	44.210	22.951	1.000
1.600	76.150	49.311	26.839	1.000
1.700	85.563	54.595	30.968	1.000
1.800	95.403	60.038	35.365	1.000
1.900	105.542	65.614	39.929	1.000
2.000	116.022	71.296	44.726	1.000
2.100	126.746	77.054	49.693	1.000
2.200	137.614	82.858	54.756	1.000
2.300	146.761	86.925	59.837	0.982
2.400	155.505	91.554	63.951	0.970
2.500	164.069	95.885	68.184	0.956
2.600	172.136	99.939	72.197	0.940
2.700	179.803	103.767	76.036	0.923
2.800	187.139	107.398	79.741	0.905
2.900	194.051	110.768	83.283	0.886

Table B.2: Best estimate broad-crested weir rating (continued)

Stage (m)	Total flow (cumecs)	Low weir flow (cumecs)	High weir flow (cumecs)	Drowned flow reduction factor (*f*)
3.000	200.633	113.622	87.011	0.865
3.100	206.417	115.295	91.122	0.837
3.200	211.955	116.751	95.204	0.810
3.300	217.645	118.640	99.005	0.787
3.400	223.715	121.892	101.823	0.773
3.500	229.631	124.980	104.651	0.760
3.600	236.014	128.150	107.864	0.747
3.700	241.442	131.076	110.366	0.734
3.800	248.030	134.419	113.611	0.724
3.900	254.234	137.749	116.485	0.713
4.000	260.872	141.104	119.768	0.704
4.100	267.120	144.448	122.672	0.695
4.200	273.803	147.810	125.993	0.686
4.300	279.999	151.075	128.924	0.677
4.400	286.960	154.676	132.284	0.670
4.500	292.394	157.542	134.852	0.660
4.600	300.155	161.511	138.644	0.655
4.700	307.236	165.171	142.065	0.649
4.800	313.722	168.650	145.072	0.642
4.900	320.723	172.191	148.532	0.636
5.000	327.298	175.730	151.568	0.630

B.5 Comparison of Methods

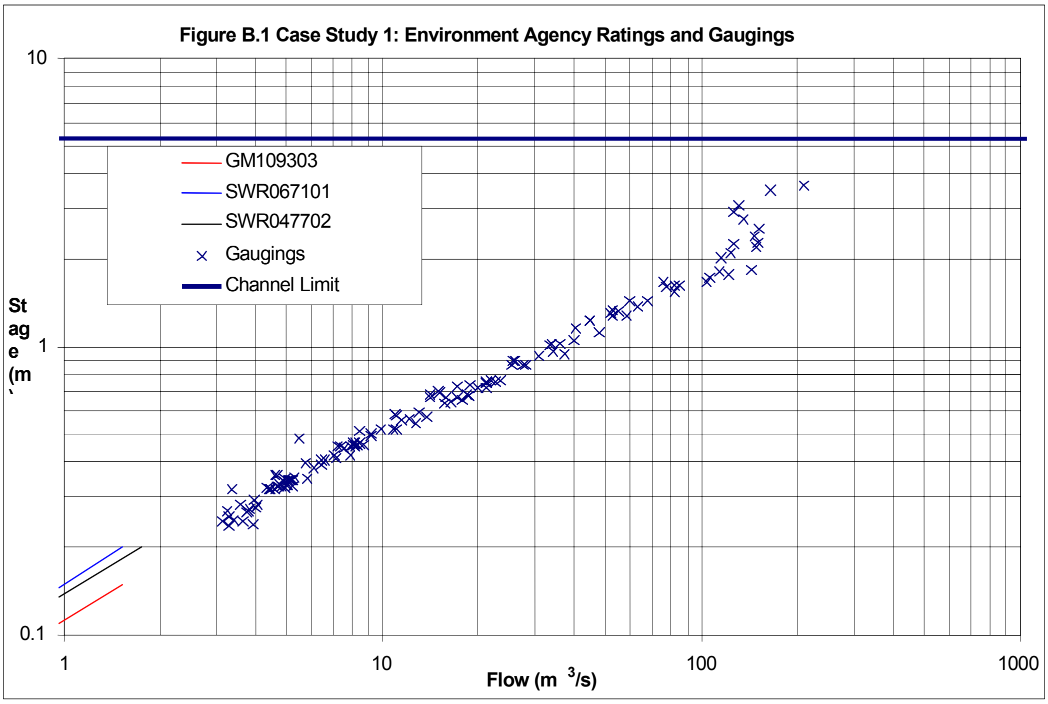
The simple extension methods match the rating to a reasonable degree, they all over predict gauged flows above 150 cumecs. This is because the existing rating does not match the gaugings above 2.2m (150 m^3/s) because the weir becomes drowned. Extension of a poor rating, using any of the above techniques, will compound these errors. This emphasises the need for a review of gaugings and ratings before an extrapolation is undertaken. Thus the simple methods are only suitable up to a flow of about 150 cumecs.

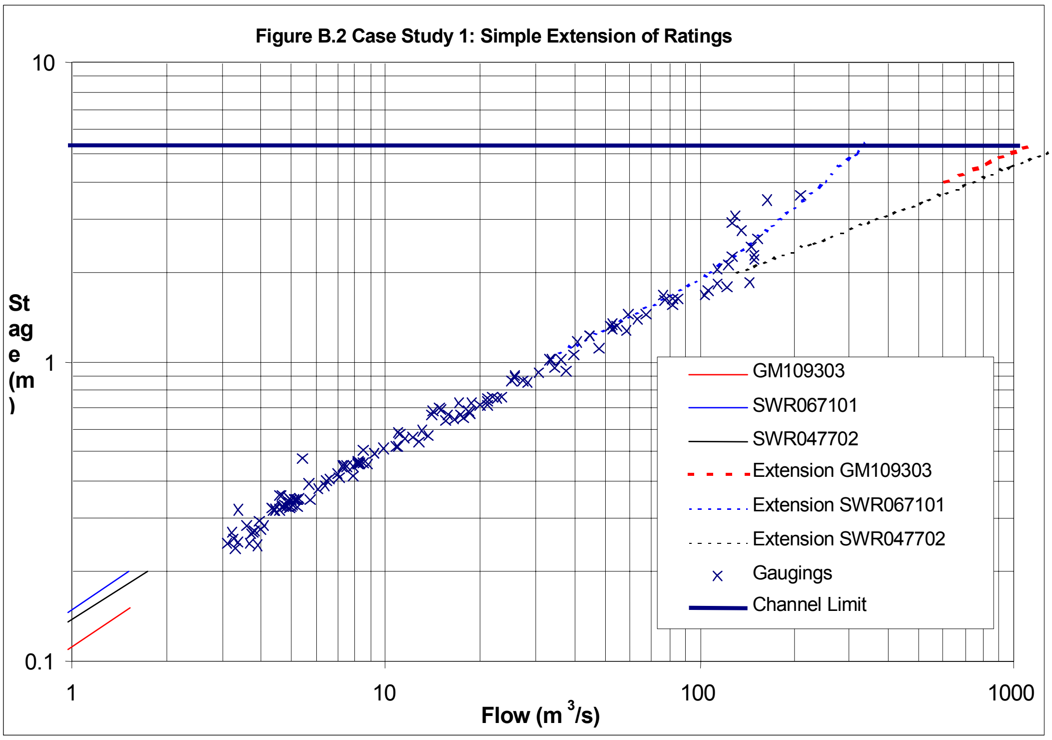
It appears that drowning is a problem at this weir as indicated by the departure of the gaugings from the rating at about 150 cumecs. The 'best estimate' approach at this site is therefore:

- The weir and the drowned weir equations are used since these are based on fundamental theoretical principles and include drowning. However, it is essential to have downstream rating curves before the drowned weir equation can be applied.

When the structure becomes heavily drowned and the headloss across the weir becomes very small, it is suggested that the Slope-Area (or DCM) method should be used to estimate the rating based on downstream channel conditions.

In conclusion it is recommended that the weir equations are used but calibrated against gauged flows, and the Slope-Area method is used for channel control, also calibrated against gauged flows above a stage of 3.1m.





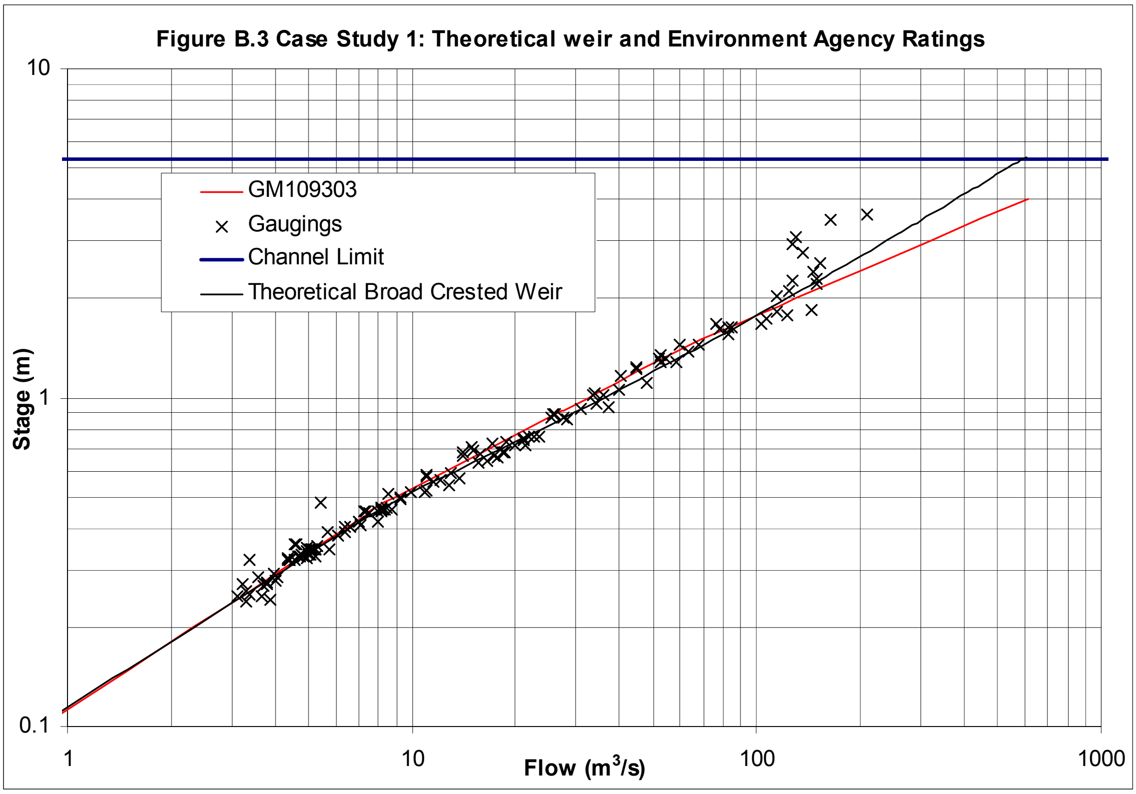


Figure B.3 Case Study 1: Theoretical weir and Environment Agency Ratings
10
1
0.1
Stage (m)
GM109303
Gaugings
Channel Limit
Theoretical Broad Crested Weir
1
10
100
1000
Flow (m3/s)

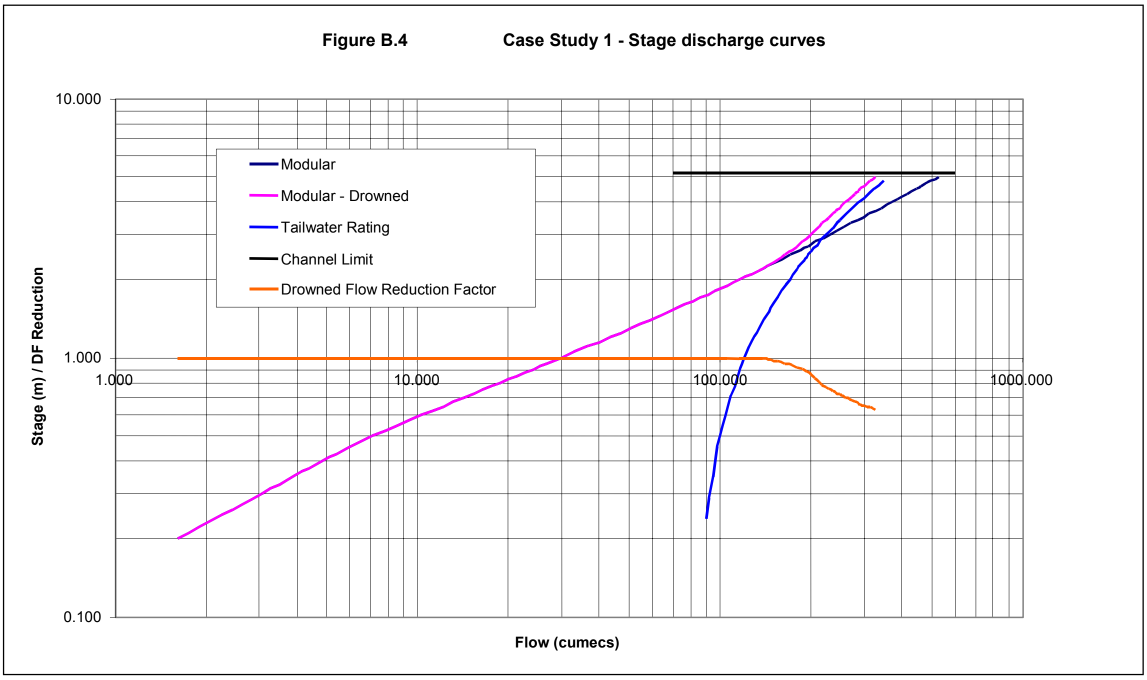


Figure B.4
Case Study 1 - Stage discharge curves
10.000
1.000
0.100
Stage (m) / DF Reduction
Modular
Modular - Drowned
Tailwater Rating
Channel Limit
Drowned Flow Reduction Factor
1.000
10.000
100.000
1000.000
Flow (cumecs)

APPENDIX C

Application of Methods to Case Study 2: Open Channel with Floodplain Flow

C.1 Site Description

The gauging station is a Velocity Area Station. The channel is 40m wide at the gauging section with a minimum bed elevation of 50.1m AOD and gauge board zero at 52.0m AOD. The left and right banks at 56.64m AOD and 56.56m AOD suggest a maximum stage of 4.56m before the banks are overtopped. The cross-section at the gauging site suggests that if over bank flow is accounted for the maximum of the rating could be extended to 9.5m, but the later ratings are extended to 6.5m only.

Eleven ratings have been provided since 1972 (Table C.1). The upper limb of ratings 10 and 11 are identical so that the same flood rating has been used from 01/01/88 to date.

Table C.1: Ratings

Rating	*C*	*a*	*β*	Min Stage	Max Stage	Start Date	End Date
C	2.121	-1.877	11.648	0	1.87	01/06/72	31/10/72
	1.8777	-1.49	9.8319	1.87	N/A		
F	1.272	-2.332	28.984	0	n/a	01/06/74	31/08/74
	1.6481	-2.109	20.809	n/a	2.5		
	1.878	-1.49	9.832	2.5	2.913		
G	1.44	-2.14	24.756	2.495	4.436	01/03/75	31/10/75
H	1.44	-2.14	24.756	2.495	3.559	01/11/75	31/12/75
	1.949	-1.244	7.9801	3.559	8		
4	1.44	-0.14	24.756	0.495	1.559	01/11/76	31/10/77
	0.7562	0.756	7.9801	1.559	6		
5	1.354	-0.185	25.921	0	0.49	01/11/77	30/06/79
	2.5032	0.184	3.755	.49	4.715	01/04/80	30/04/84
						01/10/84	28/02/86
6	1.914	0.058	15.817	0	0.6	01/07/79	31/03/80
	1.354	-0.185	25.921	0.6	1.586		
	2.503	0.184	3.755	1.586	4.715		
8	4.651	1.475	0.1947	0	n/a	01/05/84	30/09/84
	1.354	-0.185	25.921	n/a	1.84		
	2.503	0.184	3.755	1.84	4.715		
9	1.354	-0.265	25.921	n/a	n/a	01/03/86	31/12/87
	1.4728	-0.055	19.671	n/a	1.298		
	2.629	-0.002	2.808	1.298	5.317		
10	1.354	-0.265	25.921	0.54	1.346	01/01/88	31/12/92
	1.452	-0.028	19.292	1.346	4.927		
	2.629	0.078	2.808	4.927	6.5		
11	1.489	-0.056	17.485	0.54	4.713	01/01/93	date
	2.6295	0.078	2.8081	4.713	6.5		

The latest rating 11 is shown on Figure C.1 together with all available gaugings from 1988 to 1993 and from 1993 to date, which suggests the upper limb is supported by gaugings. However there is considerable variation of gaugings around the lower end of the rating possibly due to vegetation or weed growth. A rating and gauging review may identify the cause of this variation.

C.2 Simple Extension

Simple extension involves extending the Agency rating equation above the stated maximum limit. Figure C.2 shows the existing rating (Rating 11) extended from its maximum of 6.5m to 9.5m.

C.3 Logarithmic Extrapolation

At this site the shape of the control does not change significantly and the channel roughness remains fairly constant, and thus a straight-line extrapolation on a log-log plot is probably reasonable. However, the method would not provide any improvement over the simple extension and is not evaluated further,

C.4 The Weir Equation

As this is natural control the weir equation approach cannot be used.

C.5 The Velocity-Extrapolation Method

The three approaches to the velocity extrapolation method rely on calculating a best-fit line between velocity and either stage, hydraulic radius or $AR^{2/3}$. The application of these three techniques is described below.

Simple Approach

1. The Stage-Area relationship for the full range of conditions is calculated for one cross-section at the flow measurement section (Table C.2, columns 1 and 2)

2. The flow is calculated using the existing rating (Table C.2, column 3) up to the rating maximum and the velocity calculated from the flow and cross-sectional area using the continuity equation, $V = Q/A$ (Table C.2, column 4).

3. The Velocity-Stage relationship is plotted (Figure C.3) and, assuming little curvature occurs, a best-fit line calculated for the top part of the velocity-stage relationship. As with the log extrapolation technique described above, a best-fit line is calculated for all data on the upper limb and the last 2 data points on the rating. The best-fit lines are:

All Data	$V = 0.3918\ h - 0.1679$
Last 2 data pts	$V = 0.1135\ h + 1.6139$

where V = velocity
h = river stage

4. Velocities are then calculated for stage values above the rating maximum to the top of the cross-section (Table C.2, column 5) using a linear best-fit line which in this case is based on the last 2 data points.

5. The cross-sectional area between the rating maximum and top of the cross-section are also calculated (Table C.2, column 2) and on the basis of the derived velocity detailed above the flow is calculated using the continuity equation, $Q = V\,A$ (Table C.2, column 6).

6. This allows pairs of Q and h values to be calculated (Figure C.4), to which a power law equation can be fitted.

Table C.2: Velocity Extrapolation (Simple Approach)

1	2	3	4	5	6
Stage (m)	**Area (m^2)**	***Q*-Rating (m^3/s)**	***V=Q/A* (m/s)**	***V* (linear ext) (m/s)**	***Q = V * A* (col 5*col 2) (m^3/s)**
4.0	144	135.420	0.939		
4.5	164	161.690	0.987		
5.0	206	202.381	0.98		
5.9	305	310.582	1.017		
6.0	317	324.408	1.023		
6.5	377	399.238	1.058	1.058	399
7.0	440	Limit of current rating		1.074	473
7.5	505			1.098	555
8.0	573			1.122	642
8.5	641			1.146	735
9.0	711			1.170	832
9.5	781			1.194	932

Hydraulic Radius Approach

A logarithmic plot of velocity against the hydraulic mean radius (R) often shows a linear relationship and can be used to provide values of velocity above the rating maximum. Figure C.5 shows a plot of hydraulic radius against velocity (Table C.3, columns 4 and 5) and indicates the log-log plot is almost linear in the upper range. As with other approaches detailed above a best-fit line is calculated for all data and the last 2 data points only on the upper part of the rating. The best-fit lines are given by:

All Data $V = 0.9426\,R - 1.1815$
Last 2 data pts $V = 1.1039\,R - 1.7527$

Extension of these curves allows velocities to be calculated between the top of the rating and the limit of the cross-section (Table C.3, column 6) based on the hydraulic radius and hence the flows can be calculated from the continuity equation (Table C.3, column 7). In this case Table C.3 show calculations for the “last 2 data points” best-fit line.

Table C.3: Velocity Extrapolation (Hydraulic Radius)

1	2	3	4	5	6	7
Stage (m)	**Area (m^2)**	***Q*-Rating (m^3/s)**	***V=Q/A* (m/s)**	***R* (m)**	***V* (from best-fit) (m/s)**	***Q=V*A* (m^3/s)**
4.0	144.2	135.420	1.641	3.543		
4.5	163.8	161.690	1.663	3.079		
5.0	206.5	202.381	1.793	2.015		
5.5	259.4	258.953	1.995	2.227		
6.0	317.0	324.408	2.193	2.598		
6.1	328.9	338.608	2.232	2.672		
6.2	340.8	353.187	2.270	2.744	1.036	353.089
6.3	352.9	368.150	2.307	2.816	1.042	367.597
6.4	365.1	383.498	2.341	2.888	1.047	382.333
6.5	377.4	399.238	2.353	2.962	1.053	397.363
6.6	389.8	Limit of current rating		3.036	1.059	412.628
6.7	402.2			3.111	1.064	428.072
6.8	414.8			3.183	1.070	443.812
6.9	427.5			3.258	1.076	459.871
7.0	440.2			3.330	1.081	475.956
7.1	453.1			3.404	1.087	492.504
7.2	466.0			3.475	1.092	509.070
7.3	479.0			3.548	1.098	525.972
7.4	492.1			3.618	1.103	543.021
7.5	505.4			3.692	1.109	560.556
7.6	518.7			3.761	1.115	578.094
7.7	532.1			3.831	1.120	595.875
7.8	545.6			3.900	1.125	613.901
7.9	559.1			3.968	1.130	632.028
8.0	572.8			4.037	1.136	650.543
8.1	585.2			4.095	1.140	667.267
8.2	599.2			4.164	1.146	686.410
8.3	613.2			4.232	1.151	705.657
8.4	627.2			4.299	1.156	725.006
8.5	641.2			4.365	1.161	744.454
8.6	655.2			4.430	1.166	763.999
8.7	669.2			4.494	1.171	783.640
8.8	683.2			4.558	1.176	803.374
8.9	697.2			4.620	1.181	823.200
9.0	711.2			4.682	1.185	843.116
9.1	725.2			4.743	1.190	863.121
9.2	739.2			4.803	1.195	883.211
9.3	753.2			4.862	1.199	903.387
9.4	767.2			4.921	1.204	923.645
9.5	781.2			4.979	1.208	943.985

Again, the pairs of Q and h values can be used to calculate a power law equation. The rating extensions are shown on Figure C.6. Such extensions are not always linear as this depends on how the hydraulic radius and cross-sectional area vary with stage. The wetted perimeter at this section increases markedly as the bank full conditions are exceeded (Figure C.7) although the cross-sectional area increases at a steady rate. This occurs as the floodplain is first inundated and results in a drop on the hydraulic radius ($R = A/P$). Consequently any extension method based on the hydraulic radius will also show a drop in flow at just above the bankfull limit which is not supported by gaugings (Figure E.6). This method is only applicable where the change in the hydraulic radius is smooth throughout a cross-section and is generally not appropriate at this (or any) site above bankfull. This is because it assumes an average velocity whereas there is a large variation in velocity across an overbank flow section and the average velocity is less likely to rise in a linear way.

Manning's Equation

A third variation of the Velocity Extrapolation method is to use Manning's equation. Assuming $s^{1/2}/n$ remains constant a curve has been prepared for Q against $AR^{2/3}$ (Figure C.7). Again for illustration purposes the "all data" and "last 2 data pts" best-fit lines have been calculated (Figure C.8) and these are:

All Data $Q = 1.2101\ A\ R^{(2/3)} - 93.9273$
Last 2 Data Pts $Q = 1.2704\ A\ R^{(2/3)} - 114.9172$

Figure C.8 again illustrates the problem of the fall in hydraulic radius as the bankfull limit is exceeded resulting in lower $AR^{2/3}$ values and hence the top values in the series has been ignored in calculating the best-fit lines.

On the basis of A and R above the rating maximum this equation has been used to calculate flows (Figure C.9 and Table C.4 for the last 2 data points extension). This approach only works where the slope of the curve Q against $AR^{2/3}$ is constant i.e. where both s and n are the same throughout the range of flows and where R does not vary. Again a power law can be fitted to the pairs of Q and h values to give the rating in EA format.

Table C.4: Velocity Extrapolation (Manning's Approach)

1	2	3	4	5	6
Stage (m)	**Area (m^2)**	***Q*-Rating (m^3/s)**	***R* (m)**	***AR*$^{(2/3)}$**	***Q* (m^3/s)**
0.1	35.1	0.227	1.618	48.366	
0.5	43.5	5.397	1.900	66.720	
1.0	54.4	16.303	2.220	92.588	
1.5	66.3	30.533	2.351	117.226	
2.0	79.7	47.412	2.605	150.875	
2.5	94.2	66.561	2.863	189.942	
3.0	109.7	87.728	3.108	233.612	
4.0	144.2	135.420	3.543	335.129	
4.5	163.8	161.690	3.079	346.669	
5.0	206.5	202.381	2.015	329.395	
5.5	259.4	258.953	2.227	442.316	
6.0	317	324.408	2.598	599.136	

Table C.4: Velocity Extrapolation (Manning's Approach) (continued)

1	2	3	4	5	6
Stage (m)	**Area (m^2)**	***Q*-Rating (m^3/s)**	***R* (m)**	$AR^{(2/3)}$	***Q* (m^3/s)**
6.1	328.9	338.608	2.672	633.288	
6.2	340.8	353.187	2.744	667.962	
6.3	352.9	368.150	2.816	703.805	
6.4	365.1	383.498	2.888	740.494	386.290
6.5	377.4	Limit of current rating	2.962	778.437	402.890
7.0	440.2		3.330	981.588	491.769
7.5	505.4		3.692	1207.229	590.487
8.0	572.8		4.037	1452.168	697.647
8.5	641.2		4.365	1712.542	811.561
9.0	711.2		4.682	1990.434	933.139
9.5	781.2		4.979	2277.832	1058.876

Summary

As detailed in Section 2.5.3 the current Standard (BS ISO 1100-2) states that it is often difficult to estimate the velocity-stage and hydraulic radius-stage relationships accurately in the extended range and the Manning's method relies on a constant slope and Manning's *n* value throughout the flow range. These conditions may not be valid. For these reasons the velocity-area methods are regarded as inferior to the Slope-Area method described below. It does, nevertheless, form a series of approaches for the extension of rating curves if limited topographical survey (i.e. one cross-section) is available.

As with simple and logarithmic extensions this approach is only valid to bankfull stage or top of the topographic section. In this case they have been applied to an existing overbank rating.

C.6 Slope-Area Method

The Slope-Area Method is based upon estimating the roughness and friction slope of the channel at the gauging station. BS ISO 1100-2 states that these methods can be used to extrapolate the high end of rating curves under channel control and when properly applied, are the most hydraulically 'correct' of all the simple techniques, and as such are to be generally preferred. There are essentially two approaches to the Slope-Area method:

- The simple channel approach; and
- The Divided Channel Method (DCM).

Simple Channel Approach

The Simple Channel approach has been used as follows:

1. The cross-sectional area and hydraulic radius have been calculated for different stage values from a cross-sectional survey of a typical section of the control reach (Table C.5, Columns 2 and 3) as shown on Figure C.11.

2. Measurements of water surface slope are not available and the slope has been assumed to be the same as the bed slope.

3. Manning's *n* is estimated by calibration using the upper segment of the current rating.

4. The mean velocity and discharge have been calculated using Manning's formulae.

In general, both the channel roughness and the friction slope will vary with stage, so the method requires that both parameters are estimated for different values of stage. The variation of these parameters with stage is not known and hence constant values are assumed up to the top of the banks.

A channel slope of 0.001 was used based on topographical sections upstream and downstream of the gauging section. The Manning's *n* value was 0.035 based on calibration against the current rating. Table C.5 shows the calculations compared to the rated flow which are also shown on Figure C.10.

Table C.5: Slope-Area Method (Simple Channel Approach)

Stage (m)	**Area (m^2)**	***R* (m)**	***s***	***n***	***Q* Slope-Area (m^3/s)**		***Q* rating (m^3/s)**
0.1	35.1	1.618	0.0002	0.035	19.543		0.227
0.2	37.2	1.691	0.0002	0.035	21.334		1.080
0.3	39.2	1.758	0.0002	0.035	23.070		2.273
0.4	41.3	1.827	0.0002	0.035	24.944		3.726
0.5	43.5	1.900	0.0002	0.035	26.959		5.397
0.6	45.6	1.966	0.0002	0.035	28.911		7.258
0.7	47.8	2.034	0.0002	0.035	31.006		9.293
0.8	49.9	2.088	0.0002	0.035	32.937		11.485
0.9	52.1	2.153	0.0002	0.035	35.099		13.825
1.0	54.4	2.220	0.0002	0.035	37.411		16.303
1.2	58.9	2.319	0.0002	0.035	41.695		21.644
1.4	63.8	2.320	0.0002	0.035	45.178		27.459
1.6	68.9	2.401	0.0002	0.035	49.914		33.711
1.8	74.2	2.507	0.0002	0.035	55.326		40.370
2.0	79.7	2.605	0.0002	0.035	60.963		47.412
2.2	85.4	2.711	0.0002	0.035	67.092		54.815
2.4	91.2	2.815	0.0002	0.035	73.464		62.563
2.6	97.2	2.910	0.0002	0.035	80.056		70.639
2.8	103.3	3.003	0.0002	0.035	86.878		79.032
3.0	109.7	3.108	0.0002	0.035	94.393		87.728
3.2	116.2	3.201	0.0002	0.035	101.981		96.716
3.4	122.9	3.295	0.0002	0.035	109.958		105.988
3.6	129.8	3.389	0.0002	0.035	118.333		115.535

Table C.5: Slope-Area Method (Simple Channel Approach) (continued)

Stage (m)	Area (m^2)	*R* (m)	*s*	*n*	*Q* Slope-Area (m^3/s)		*Q* rating (m^3/s)
3.8	136.9	3.475	0.0002	0.035	126.898		125.348
4.0	144.2	3.543	0.0002	0.035	135.413		135.420
4.2	151.7	3.603	0.0002	0.035	144.068		145.744
4.4	159.6	3.677	0.0002	0.035	153.642		156.314
4.6	169.7	2.325	0.0002	0.035	120.328		167.125
4.8	187.4	1.938	0.0002	0.035	117.701		182.126
5.0	206.5	2.015	0.0002	0.035	133.096		202.381
5.2	226.8	2.096	0.0002	0.035	150.095		223.976
5.4	248.3	2.180	0.0002	0.035	168.678		246.944
5.6	270.7	2.302	0.0002	0.035	190.688		271.317
5.8	293.7	2.452	0.0002	0.035	215.766		297.128
6.0	317	2.598	0.0002	0.035	242.088		324.408
6.2	340.8	2.744	0.0002	0.035	269.897		353.187
6.4	365.1	2.888	0.0002	0.035	299.205		383.498
6.6	389.8	3.036	0.0002	0.035	330.223		
6.8	414.8	3.183	0.0002	0.035	362.701		
7.0	440.2	3.330	0.0002	0.035	396.621		
7.2	466	3.475	0.0002	0.035	431.987		
7.4	492.1	3.618	0.0002	0.035	468.644		
7.6	518.7	3.761	0.0002	0.035	506.910		
7.8	545.6	3.900	0.0002	0.035	546.209		
8.0	572.8	4.037	0.0002	0.035	586.764		
8.2	599.2	4.164	0.0002	0.035	626.652		
8.4	627.2	4.299	0.0002	0.035	670.019		
8.6	655.2	4.430	0.0002	0.035	714.098		
8.8	683.2	4.558	0.0002	0.035	758.855		
9.0	711.2	4.682	0.0002	0.035	804.257		
9.2	739.2	4.803	0.0002	0.035	850.272		
9.4	767.2	4.921	0.0002	0.035	896.872		
9.5	781.2	4.979	0.0002	0.035	920.383		

A rating equation could be fitted to the set of Q and h values if required.

Divided Channel Method

An extension of the simple channel approach to the Slope-Area method is the Divided Channel Method (DCM). In this method the flow along the left and right bank floodplains are assumed to be separate to the main channel and each is calculated according to the Slope-Area method. The flows from the three sections of channel (left, right and main channel) are summed to give the total flow. The advantage of this method is that different values of channel roughness can be applied to the upper part of a channel section or the floodplain.

The division of the channel into three sections is based on the cross-section provided in Figure C.11. This provides different values of *A*, *P* and hence *R* to which values of channel slope (*s*) and roughness (*n*) are applied to give the estimated flow using

Manning's equation. A summary of the calculations are given in Table C.6. The same slope is assumed for all three compartments.

Table C.6: Slope-Area (Divided Channel Method)

	Main Channel					Left Channel				Right Channel				
H (m)	*A* (m^2)	*R* (m)	*s*	*n*	*Q* (m^3/s)	*A* (m^2)	*R* (m)	*n*	*Q*-L (m^3/s)	*A* (m^2)	*R* (m)	*n*	*Q*-R (m^3/s)	*Q*total (m^3/s)
0.1	35.1	1.618	0.0002	0.035	19.543									19.543
0.2	37.2	1.691	0.0002	0.035	21.334									21.334
0.3	39.2	1.758	0.0002	0.035	23.070									23.070
0.4	41.3	1.827	0.0002	0.035	24.944									24.944
0.5	43.5	1.900	0.0002	0.035	26.959									26.959
0.6	45.6	1.966	0.0002	0.035	28.911									28.911
0.7	47.8	2.034	0.0002	0.035	31.006									31.006
0.8	49.9	2.088	0.0002	0.035	32.937									32.937
0.9	52.1	2.153	0.0002	0.035	35.099	Flow inbank								35.099
1.0	54.4	2.220	0.0002	0.035	37.411	No flow on floodplains								37.411
1.2	58.9	2.319	0.0002	0.035	41.695									41.695
1.4	63.8	2.320	0.0002	0.035	45.178									45.178
1.6	68.9	2.401	0.0002	0.035	49.914									49.914
1.8	74.2	2.507	0.0002	0.035	55.326									55.326
2.0	79.7	2.605	0.0002	0.035	60.963									60.963
2.2	85.4	2.711	0.0002	0.035	67.092									67.092
2.4	91.2	2.815	0.0002	0.035	73.464									73.464
2.6	97.2	2.910	0.0002	0.035	80.056									80.056
2.8	103.3	3.003	0.0002	0.035	86.878									86.878
3.0	109.7	3.108	0.0002	0.035	94.393									94.393
3.2	116.2	3.201	0.0002	0.035	101.981									101.981
3.4	122.9	3.295	0.0002	0.035	109.958									109.958
3.6	129.8	3.389	0.0002	0.035	118.333									118.333
3.8	136.9	3.475	0.0002	0.035	126.898									126.898
4.0	144.2	3.543	0.0002	0.035	135.413									135.413
4.2	151.7	3.603	0.0002	0.035	144.068									144.068
4.4	159.6	3.677	0.0002	0.035	153.642									153.642
4.6	162.9	3.703	0.0002	0.035	157.605	6.72	0.149	0.040	0.764	0.03	0.027	0.040	0.001	158.370
4.8	170.8	3.883	0.0002	0.035	170.511	15.84	0.344	0.040	3.144	0.73	0.124	0.040	0.073	173.728
5.0	178.9	4.067	0.0002	0.035	184.215	25.16	0.533	0.040	6.683	2.40	0.226	0.040	0.360	191.258
5.2	187.1	4.252	0.0002	0.035	198.427	34.70	0.720	0.040	11.262	5.00	0.325	0.040	0.954	210.644
5.4	195.3	4.440	0.0002	0.035	213.223	44.40	0.902	0.040	16.754	8.55	0.425	0.040	1.954	231.931
5.6	203.5	4.625	0.0002	0.035	228.254	54.30	1.082	0.040	23.119	12.90	0.556	0.040	3.525	254.898
5.8	211.7	4.811	0.0002	0.035	243.789	64.40	1.258	0.040	30.321	17.60	0.721	0.040	5.720	279.829
6.0	219.8	4.995	0.0002	0.035	259.532	74.70	1.431	0.040	38.330	22.50	0.879	0.040	8.342	306.204
6.2	228.0	5.182	0.0002	0.035	275.869	85.20	1.602	0.040	47.124	27.60	1.030	0.040	11.373	334.366
6.4	236.2	5.368	0.0002	0.035	292.603	95.90	1.769	0.040	56.686	33.00	1.179	0.040	14.878	364.167
6.6	239.2	5.323	0.0002	0.035	288.485	117.00	2.135	0.040	78.386	38.60	1.322	0.040	18.786	385.657
6.8	242.4	5.509	0.0002	0.035	305.515	128.00	2.306	0.040	90.282	44.40	1.461	0.040	23.094	418.892
7.0	250.5	5.693	0.0002	0.035	322.719	139.20	2.477	0.040	102.965	50.50	1.598	0.040	27.892	453.575
7.2	258.8	5.882	0.0002	0.035	340.737	150.40	2.643	0.040	116.177	56.80	1.732	0.040	33.096	490.010
7.4	266.9	6.066	0.0002	0.035	358.696	161.90	2.811	0.040	130.290	63.30	1.862	0.040	38.708	527.693
7.6	275.1	6.252	0.0002	0.035	377.250	173.40	2.974	0.040	144.905	70.20	1.994	0.040	44.941	567.097
7.8	283.0	6.432	0.0002	0.035	395.478	185.40	3.137	0.040	160.536	77.20	2.121	0.040	51.492	607.506
8.0	291.2	6.618	0.0002	0.035	414.761	197.40	3.295	0.040	176.634	84.20	2.239	0.040	58.234	649.628

Table C.6: Slope-Area (Divided Channel Method) (continued)

	Main Channel					Left Channel				Right Channel				
H (m)	*A* (m^2)	*R* (m)	*s*	*n*	*Q* (m^3/s)	*A* (m^2)	*R* (m)	*n*	*Q*-L (m^3/s)	*A* (m^2)	*R* (m)	*n*	*Q*-R (m^3/s)	*Q*total (m^3/s)
8.2	298.6	6.786	0.0002	0.035	432.475	209.40	3.450	0.040	193.174	91.20	2.351	0.040	65.145	690.795
8.4	307.6	6.991	0.0002	0.035	454.418	221.40	3.600	0.040	210.132	98.20	2.455	0.040	72.209	736.760
8.6	316.6	7.195	0.0002	0.035	476.793	233.40	3.746	0.040	227.487	105.20	2.553	0.040	79.410	783.690
8.8	325.6	7.400	0.0002	0.035	499.596	245.40	3.889	0.040	245.218	112.20	2.646	0.040	86.734	831.548
9.0	334.6	7.605	0.0002	0.035	522.823	257.40	4.028	0.040	263.306	119.20	2.734	0.040	94.171	880.300
9.2	343.6	7.809	0.0002	0.035	546.471	269.40	4.164	0.040	281.735	126.20	2.817	0.040	101.709	929.915
9.4	352.6	8.014	0.0002	0.035	570.535	281.40	4.296	0.040	300.487	133.20	2.896	0.040	109.341	980.363
9.5	357.1	8.116	0.0002	0.035	582.722	287.40	4.361	0.040	309.981	136.70	2.933	0.040	113.189	1005.892

The DCM rating (Figure C.10) provides a more reliable extension compared to the Slope-Area method because it takes account of variation in roughness and velocity between the channel and floodplains.

C.7 Hydraulic Modelling

A steady state HECRAS hydraulic model has been constructed based on the available topographical cross-sections and similar estimates of Manning's *n* as described above. The derived rating is shown on Table C.7 and on Figure C.12.

Table C.7: HECRAS Steady State Model Rating

Water Surface Elevation (m AOD)	Head above Gauge board zero (m)	*Q* Total (m^3/s)
52.32	0.32	1
52.53	0.53	2
52.68	0.68	3
52.82	0.82	4
52.93	0.93	5
53.03	1.03	6
53.12	1.12	7
53.21	1.21	8
53.29	1.29	9
53.37	1.37	10
53.7	1.70	15
53.97	1.97	20
54.42	2.42	30
54.79	2.79	40
55.11	3.11	50
55.39	3.39	60
55.65	3.65	70
55.90	3.90	80
56.12	4.12	90
56.34	4.34	100
56.81	4.81	125
57.21	5.21	150
57.54	5.54	175

Table C.7: HECRAS Steady State Model Rating (continued)

Water Surface Elevation (m AOD)	Head above Gauge board zero (m)	*Q* Total (m^3/s)
57.84	5.84	200
58.30	6.30	250
58.65	6.65	300
59.19	7.19	400
59.62	7.62	500
60.00	8.00	600
60.34	8.34	700

The HECRAS derived rating provides a poor fit to the existing rating but this is based on an uncalibrated model. Clearly calibration would be needed to provide a better result.

C.8 Comparison of Methods

A comparison of the methods is provided in Figure C.13 and on linear axes on Figure C.14. Generally most of the simple methods could be used to extend the rating using the overbank segment of the rating.

The only method that could predict the change in slope at bankfull is the Divided Channel Method, as this takes account of the changes in channel parameters as the flow goes out-of-bank.

A rating curve extension could be obtained using the simple extension methods based on extension of the overbank rating.

However the recommended 'best estimate' extension should be based on the Divided Channel Method as this takes account of differences in velocity in the main channel and floodplains.

The velocity extrapolation methods and the Slope-Area methods are not recommended. The velocity extrapolation methods assume an average velocity whereas there are significant differences between velocities in the main channel and the floodplains. The Slope-Area method assumes a single section and does not take account of the differences between the main channel and the floodplains.

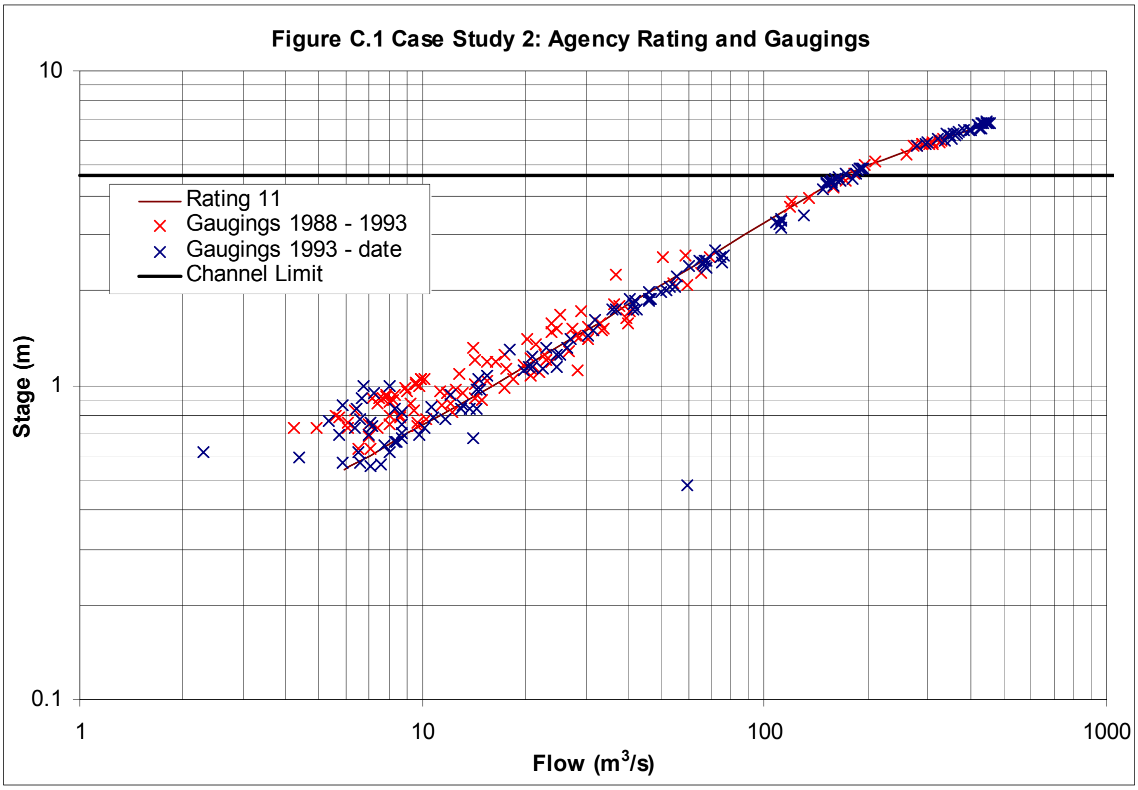


Figure C.1 Case Study 2: Agency Rating and Gaugings
10
1
0.1
Stage (m)
1
10
100
1000
Flow (m3/s)
Rating 11
Gaugings 1988 - 1993
Gaugings 1993 - date
Channel Limit

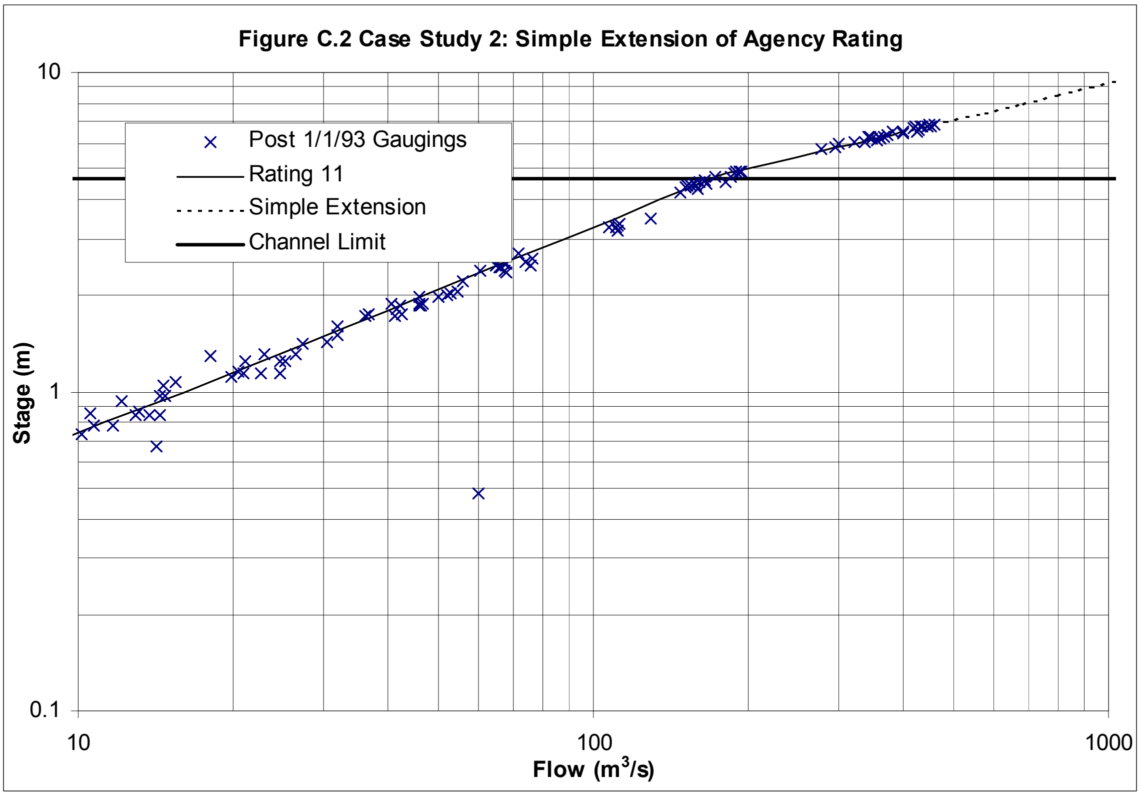


Figure C.2 Case Study 2: Simple Extension of Agency Rating
10
1
0.1
Stage (m)
10
100
1000
Flow (m3/s)
Post 1/1/93 Gaugings
Rating 11
Simple Extension
Channel Limit

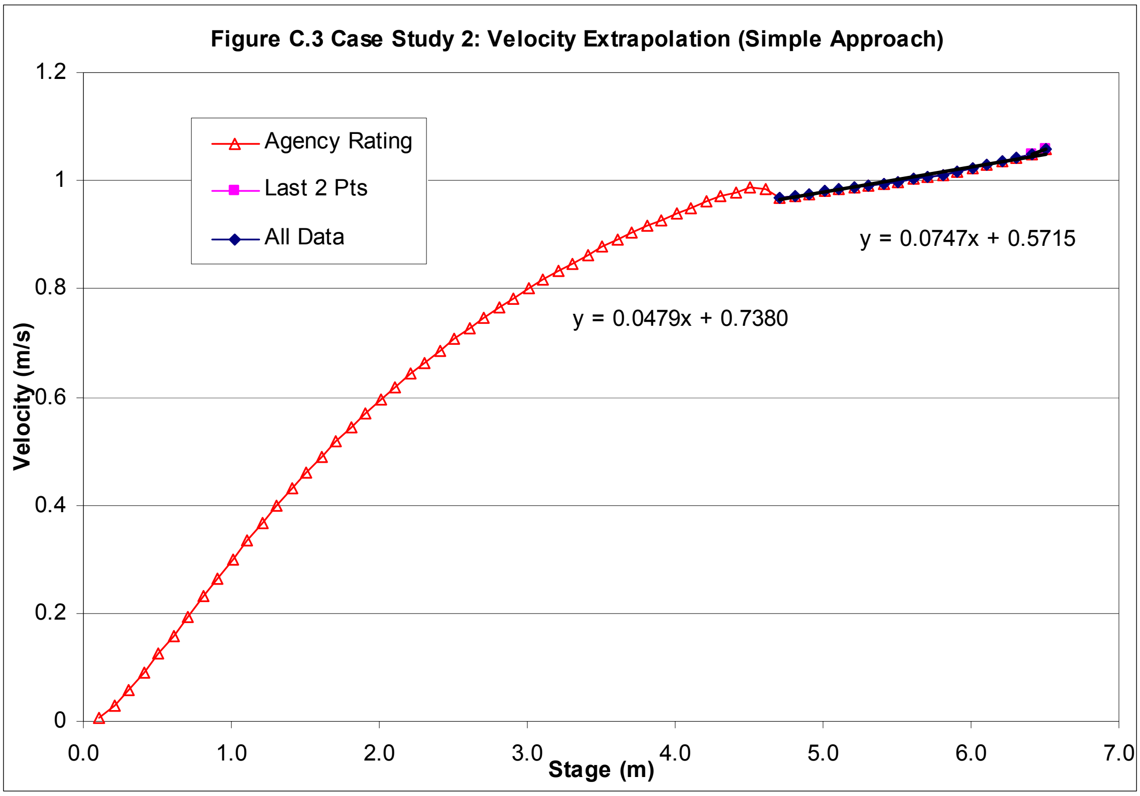


Figure C.3 Case Study 2: Velocity Extrapolation (Simple Approach)
Agency Rating
Last 2 Pts
All Data
y = 0.0747x + 0.5715
y = 0.0479x + 0.7380
Velocity (m/s)
Stage (m)
1.2
1
0.8
0.6
0.4
0.2
0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0

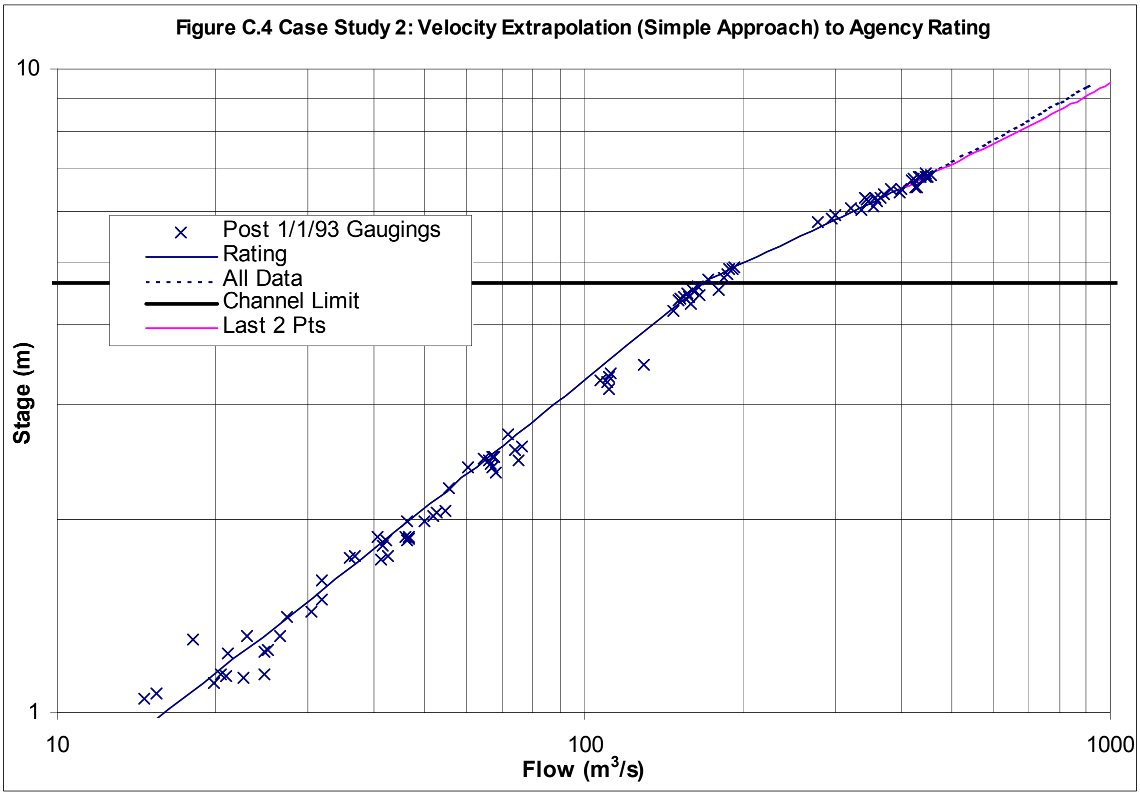


Figure C.4 Case Study 2: Velocity Extrapolation (Simple Approach) to Agency Rating
Post 1/1/93 Gaugings
Rating
All Data
Channel Limit
Last 2 Pts
Stage (m)
Flow (m³/s)
10
1
10
100
1000

Figure C.5 Case Study 2: Velocity Extrapolation (Hydraulic Radius)

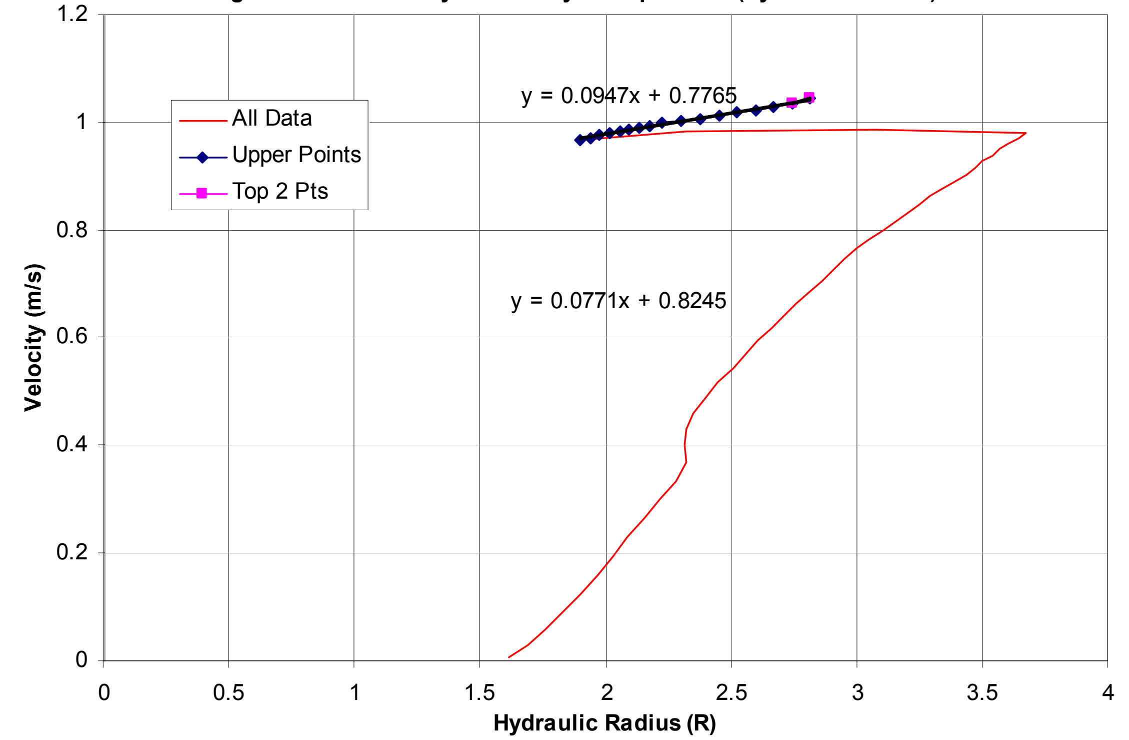


Figure C.6 Case Study 2: Velocity Extrapolation (Hydraulic Radius) to Agency Rating

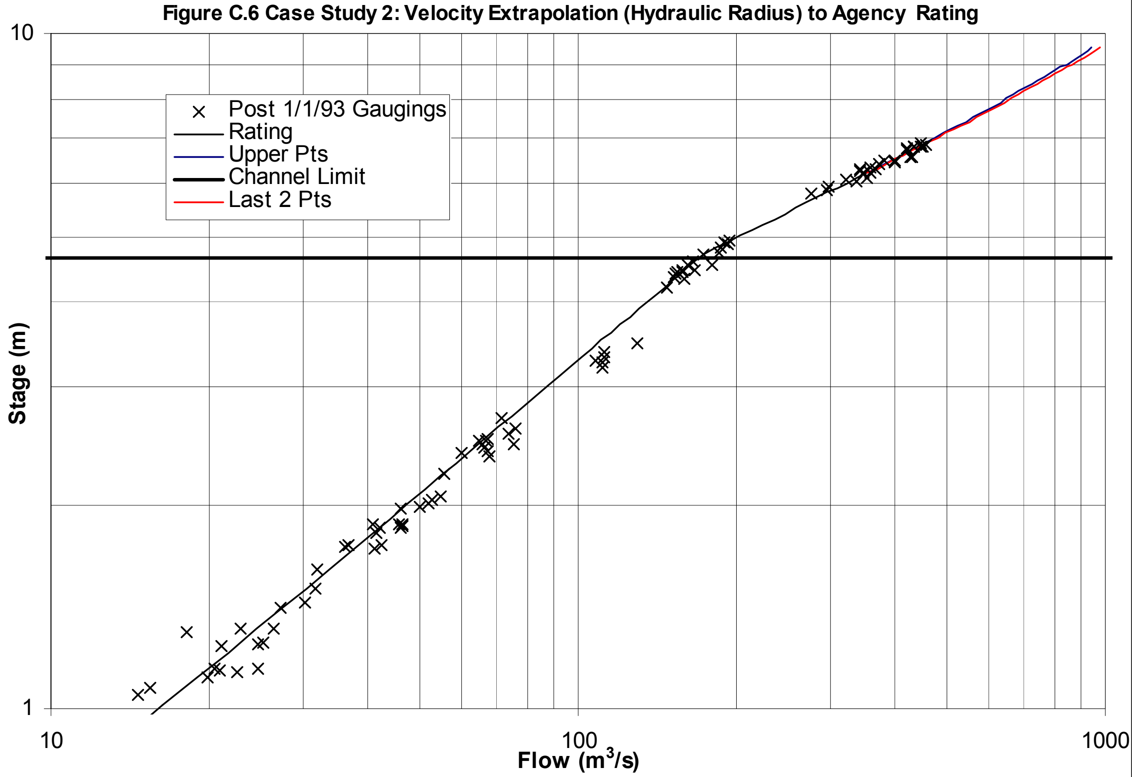


Figure C.7 Case Study 2: Variation in Channel Parameters

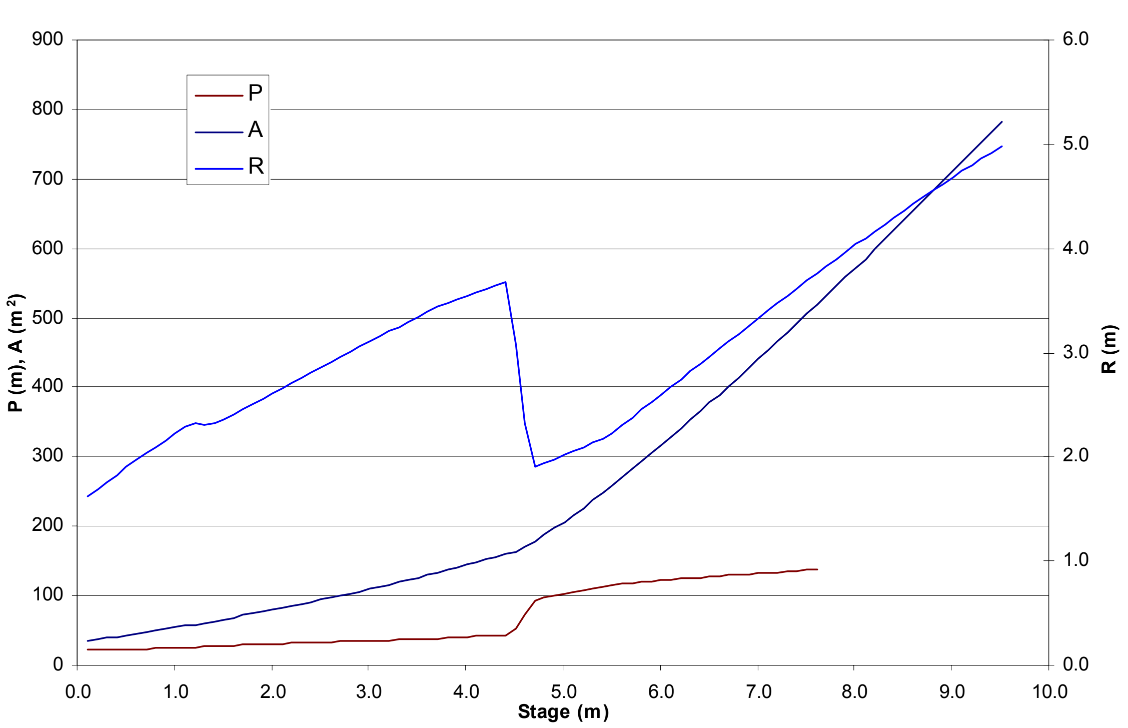
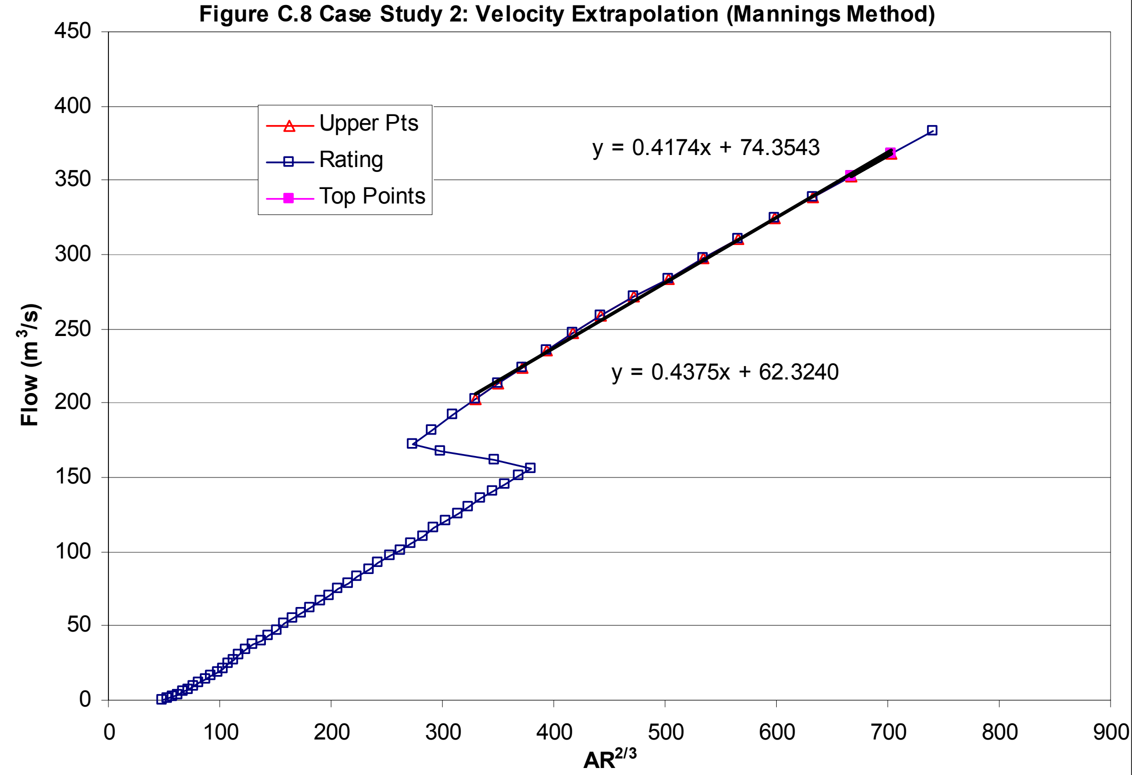


Figure C.8 Case Study 2: Velocity Extrapolation (Mannings Method)



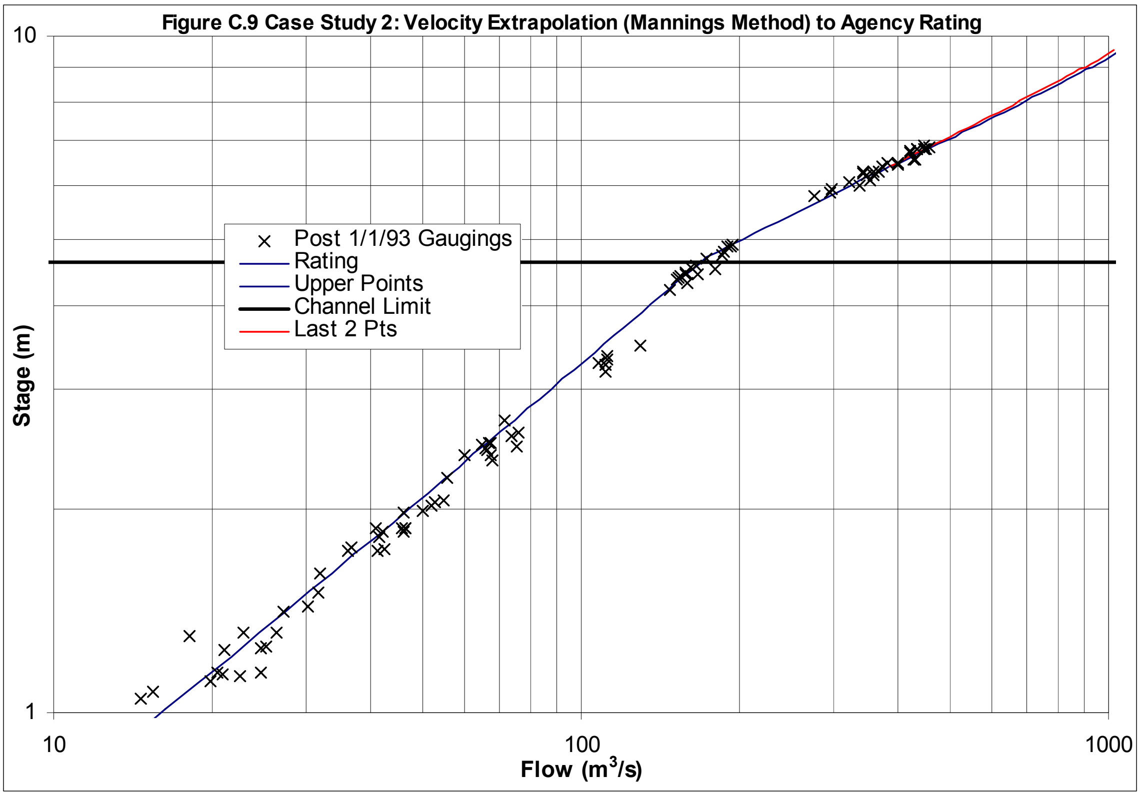


Figure C.9 Case Study 2: Velocity Extrapolation (Mannings Method) to Agency Rating
10
1
Stage (m)
10
100
1000
Flow (m³/s)
Post 1/1/93 Gaugings
Rating
Upper Points
Channel Limit
Last 2 Pts

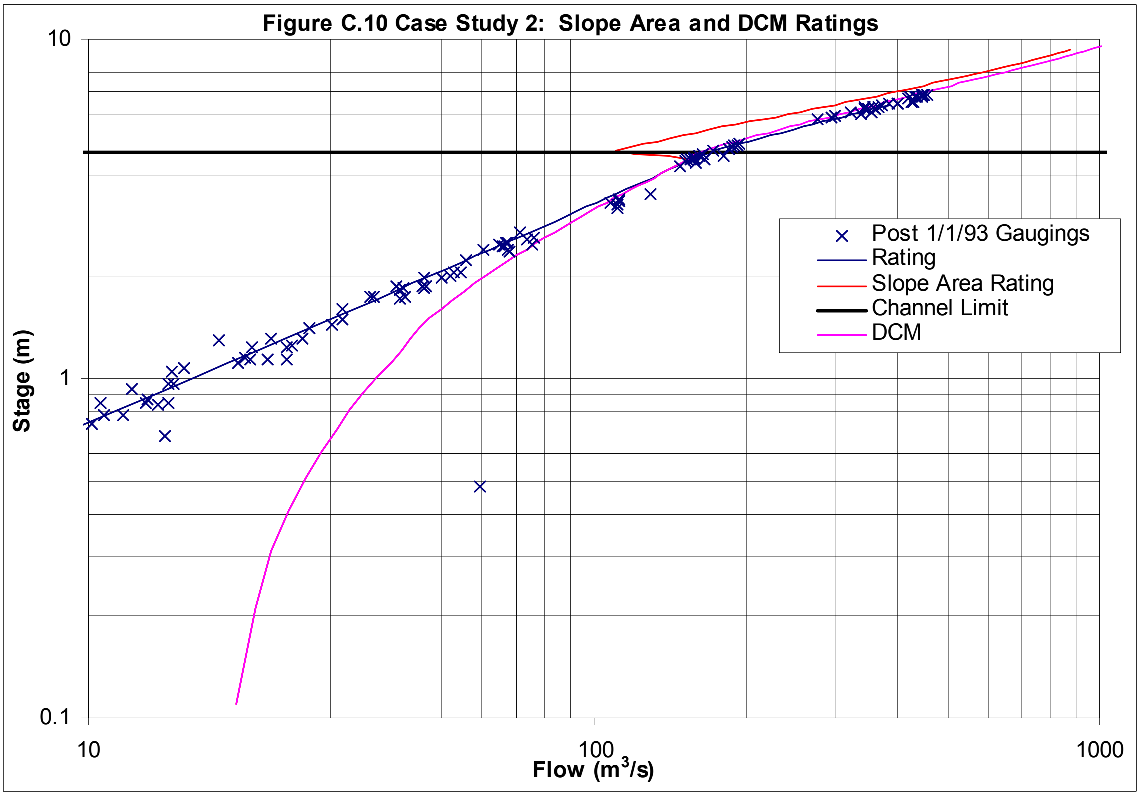


Figure C.10 Case Study 2: Slope Area and DCM Ratings
10
1
0.1
Stage (m)
10
100
1000
Flow (m³/s)
Post 1/1/93 Gaugings
Rating
Slope Area Rating
Channel Limit
DCM

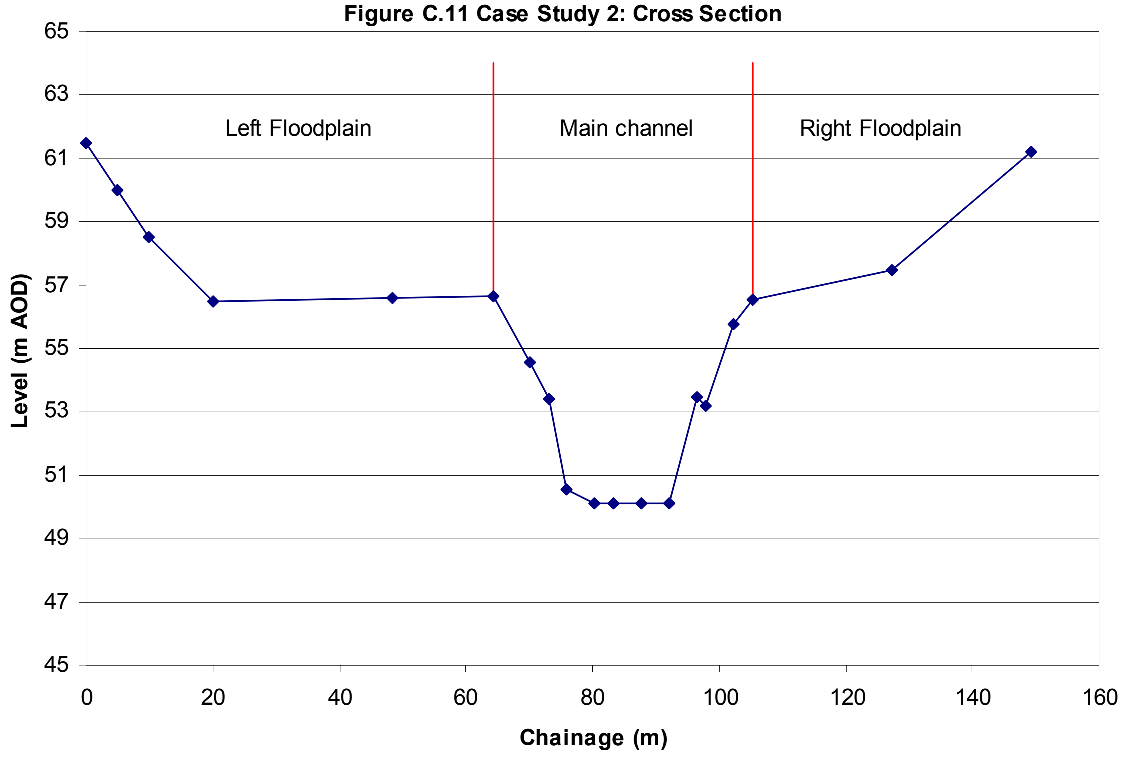


Figure C.11 Case Study 2: Cross Section

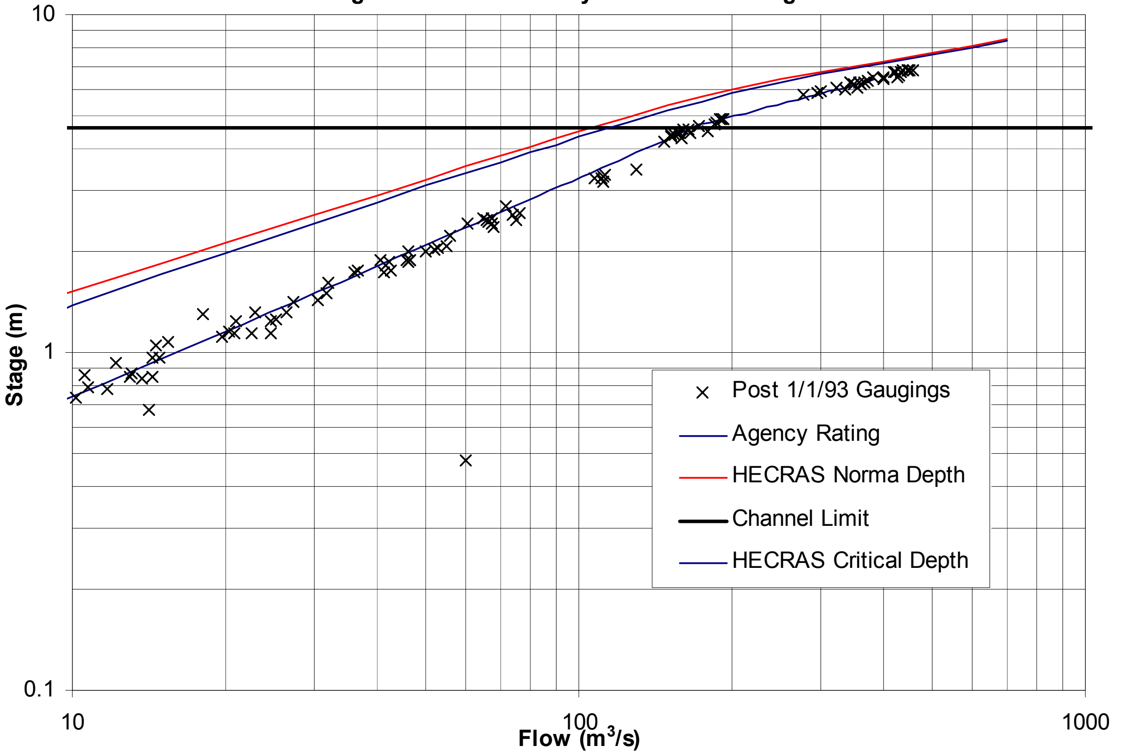


Figure C.12 Case Study 2: HECRAS Rating

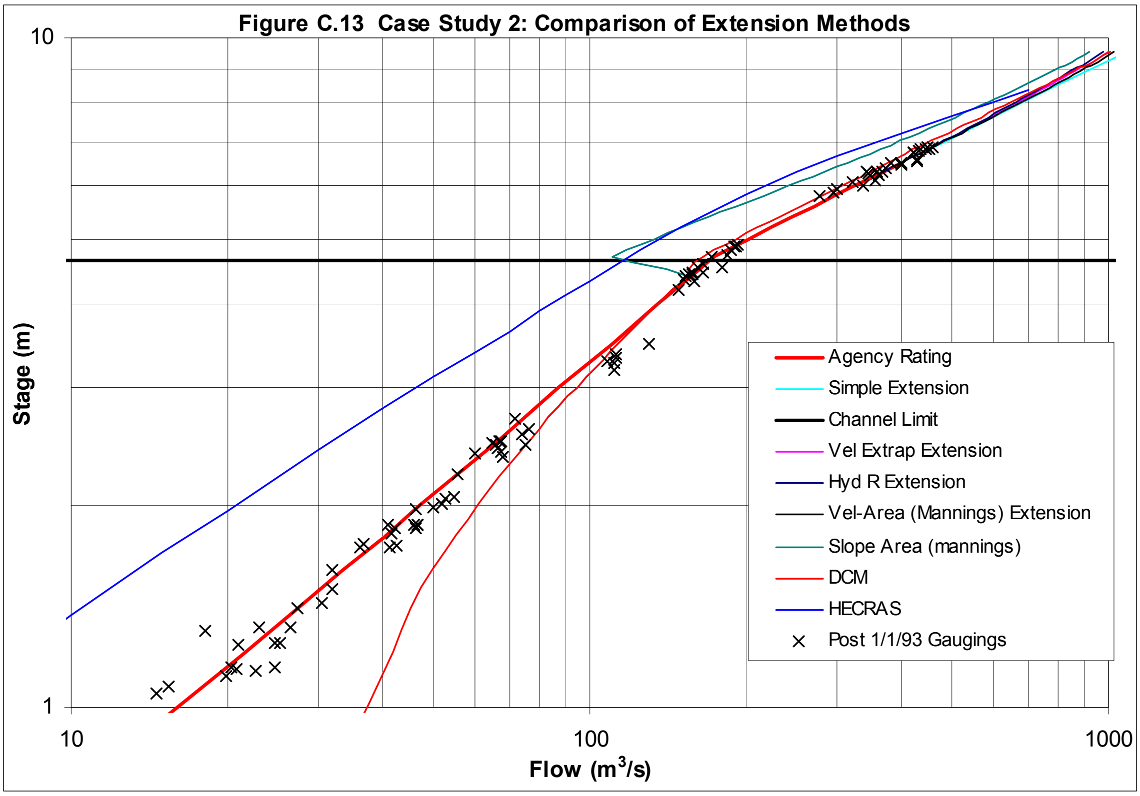


Figure C.13 Case Study 2: Comparison of Extension Methods
Stage (m)
Flow (m3/s)
10
1
10
100
1000
Agency Rating
Simple Extension
Channel Limit
Vel Extrap Extension
Hyd R Extension
Vel-Area (Mannings) Extension
Slope Area (mannings)
DCM
HECRAS
Post 1/1/93 Gaugings

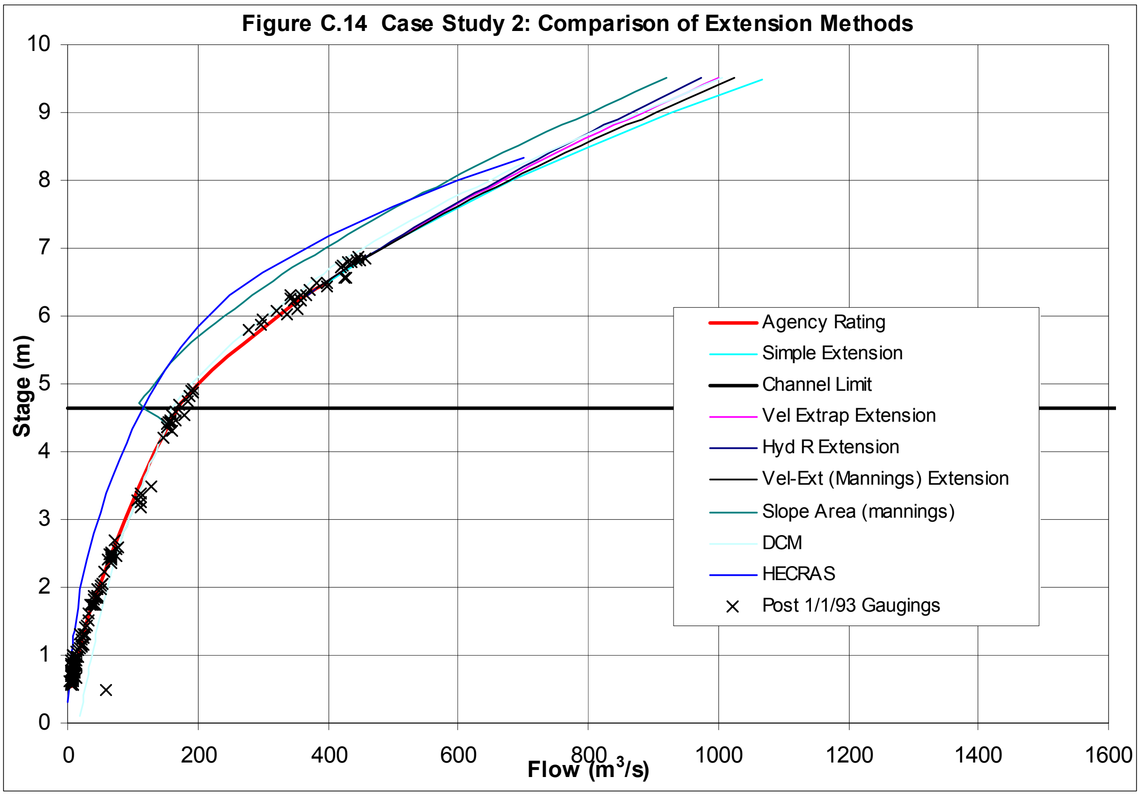


Figure C.14 Case Study 2: Comparison of Extension Methods
Stage (m)
Flow (m3/s)
10
9
8
7
6
5
4
3
2
1
0
0
200
400
600
800
1000
1200
1400
1600
Agency Rating
Simple Extension
Channel Limit
Vel Extrap Extension
Hyd R Extension
Vel-Ext (Mannings) Extension
Slope Area (mannings)
DCM
HECRAS
Post 1/1/93 Gaugings

APPENDIX D

Application of Methods to Case Study 3: Compound Sluice

D.1 Site Description

The gauging station is a compound flow measurement structure. It includes four separate components: two vertical lifting gates, an adjustable broad crested weir and a navigation lock with sluices. The dimensions and hydraulic parameters of these 4 structures are given in Table D.1 and described below. A cross-section of the compound structure is shown on Figure D.1

- The right hand bay contains a variable level 6.05m wide weir with a flat topped sharp edged timber crest (0.267m in the direction of flow) that moves vertically against the upstream face of the concrete weir block. At its lowest position it is flush with the downstream slope of the weir block (3.435m AOD) and can be raised to a maximum of 4.004m AOD. This weir is contained with vertical wing walls of 5.640m AOD

- The two centre bays each house a vertical lifting gate 6.10m wide and 1.94m high. The top and bottom of these two gates are.

- formed of 12mm plate and act as thin plate weirs when the structure is overtopped or undershot. The invert of the gates is at 2.388m AOD and when gates are closed the top of the gates is at 4.328m AOD. These are contained with vertical wing walls of 5.640m AOD.

- The left hand bay is a lock which is controlled by a vertical lifting gate at the downstream end and a pair of mitre gates at the upstream end. When the lock is not in use the downstream gates are left open hence the control is the upstream pointing gates and these can act as a weir once water levels exceed the top of the gates. The lock is 4.7m wide, 25.5m long and the top of the gates are at 4.300m AOD.

- The two upstream lock gates each contain a sluice which can also provide a flow of water. Although not designed for river regulation purposes, opening the paddles or sluices in the lock doors can be used as a convenient method of trimming upstream water levels. Each sluice is 0.66m high by 0.81m wide and controlled by vertical lifting gates mounted on the upstream face of the lock gates. The sill of each sluice is flush with the surface of the step downstream at 2.435m AOD.

Table D.1: Dimensions of Structures

	Structure			
Dimension	**Variable crest weir**	**Sluice Gates (as one)**	**Lock Gates**	**Lock Sluice**
Invert or Crest Level (m AOD)	3.435 (min) 4.004 (max)	2.388 (gates open) 4.328 (gates closed)	4.300	2.435
Weir Length (m)	0.267	0.500	0.500	0.500
Weir Width (m)	6.050	12.200	4.700	0.810
Gate Height (m)		1.940		0.660
Top of wing walls (m AOD)	5.640	5.640	5.640	5.640

The lock is separated from the remainder of the structure by a 2.44m wide pier and a wall 1.52m wide at water level which extends 28m upstream of the pier. The remaining 3 bays (the adjustable weir and the two gate bays) are separated by 1.83m wide vertical side wall concrete piers with semicircular noses which are at 5.64m OD.

The weir and gates discharge into a common stilling basin which is fitted with a faced bar or step on an apron in place of a conventional terminal lip. The base of the apron is at 0.15m AOD and the lip at 1.67m AOD.

Existing Rating

There is no available rating in Agency format as the stage-flow relationship at this site is a complex combination of flow over or under each of the structures identified above. Flows are calculated using a Fortran programme based on upstream and downstream head and the settings of the various structures. Stage discharge relationships have been provided for one or more of the following elements of the structure:

- the variable crest weir at different crest elevations;
- under the 2 vertical lifting gates when open;
- over the 2 vertical lifting gates when closed;
- over the lock mitre gates; and
- through the lock sluices.

The relationships are described more fully in Section D.2 below. All flow equations are based on a series of reports produced by Hydraulics Research and Loughborough Consultants based on hydraulic and physical modelling studies.

Water levels are measured 27m upstream and 13m downstream of the sluice gates. Tail water levels are influenced by tidal movements and the structure can be operated in both drowned and modular flow conditions.

There are no available gaugings to validate any derived rating.

Rating Maxima

As detailed above the top of the wing walls of the structure and dividing walls are at 5.640m AOD and this forms the rating maximum.

The upstream approach channel is 27m to 35m wide with a minimum bed elevation of 1.503m AOD. The minimum upstream bank elevation is 4.750m AOD and once water levels exceed this level the river will overtop the upstream banks and bypass the gauging structure. Thus whilst a rating could be extended to the structure maximum of 5.640m AOD this will not take account of over bank flow. The upstream minimum upstream bank level of 4.750m AOD therefore forms the rating maximum although extension is undertaken to 5.640m AOD on the basis that the upstream banks could be raised in the future.

There are no AMAX data in FEH data files to determine a suitable level or flow to which extension is required. The maximum flood flows at the site are of the order of 250 cumecs and ideally the rating should be extended for this flow.

D.2 Simple Extension

Simple extension involves extending an existing rating equation above the stated maximum limit. As indicated in Section D.1, due to complexities of the various weir, sluice and gate settings and combinations a rating in EA format does not exist for this site. Whilst it may be possible to calculate and extend the derived rating equations for the 6 combinations given in Table D.8, for a compound structure this approach is not recommended. This is because the theoretical calculations include extensions above the recommended upstream bank levels and there is no benefit in converting the derived hydraulic equations into best-fit lines and extending these rating equations to achieve extrapolation.

D.3 Logarithmic Extrapolation

Similarly, there is little benefit in converting theoretical equations into best-fit lines to undertake log extrapolation and hence this approach is not recommended at this site. This is because the hydraulic equations are applicable to the structure maximum level.

D.4 The Theoretical Weir Equations

The stage – flow relationship is formed by the combination of the variable broad crested weir, the sluice gates, the lock gates and lock sluices. The most appropriate method to develop a rating for such a compound structure is to use the theoretical approach to calculate a rating for each of the structures described above which are then combined to provide a rating for the complete structure.

As detailed above, extension is taken to the top of the structure at 5.64m AOD although it is recognised that this is probably unrealistic due to over bank flooding and bypassing of the structure above 4.75m AOD once the upstream banks are overtopped. However, the 5.64m level is taken as the required rating maximum for the purposes of the theoretical calculations.

Variable Crest Weir

For the rectangular and variable broad crested weir a rating has been derived based on the relevant formula given in BS 3680/4E (ISO 3846) supplemented by model studies:

$$Q = C\, b\, (H_1)^{1.5}$$

where	Q	= flow (m^3/s)
	C	= a weir coefficeint
	b	= weir crest width
	H_1	= head above weir crest (i.e. gauged head plus velocity head)

The weir coefficient is given by:

$$C = C_d\ C_v \left(\frac{2}{3}\right)^{1.5} \sqrt{g}$$

where	C_d	= coefficient of discharge
	C_v	= coefficient of velocity
	g	= acceleration due to gravity

The coefficient of velocity (C_v) is often taken as unity and hence the broad crested weir equation for the variable crest weir requires only the evaluation of the coefficient of discharge (C_d) for a given head (h_1) and weir width (b). Hydraulic model tests of this weir indicate this coefficient varies with weir geometry and is given as a function of head (h_1) and weir crest setting (H_c);

$$C_d = K_1\ \left[1.0368 + 0.2057\ Sin\{180(H_c - 3.482)\}\right] h_1^{\ 0.13}$$

Where;	K_1	= a constant (0.95 for $H_c = 3.482$ or 1.0 for $H_c > 3.482$m)
	H_c	= weir crest elevation above base of structure
	h_1	= head above weir crest

The evaluation of this coefficient for four weir crest elevations is shown on Figure D.2 and the derived ratings given on Figure D.3. A summary of the calculations is given in Table D.2 for the variable crest weir fully lowered and fully raised.

Table D.2: Calculated Flows for Variable Crest Weir Lowered and Raised

	Weir fully lowered				**Weir fully raised**		
Level (m AOD)	**Head (m)**	C_d	Q (m^3/s)		**Head (m)**	C_d	Q (m^3/s)
3.5	0.065	0.706	0.121				
3.6	0.165	0.796	0.551				
3.7	0.265	0.847	1.192				
3.8	0.365	0.883	2.008				
3.9	0.465	0.911	2.980				
4.0	0.565	0.935	4.094				
4.1	0.665	0.955	5.339		0.096	0.916	0.281
4.2	0.765	0.972	6.709		0.196	1.005	0.899
4.3	0.865	0.988	8.196		0.296	1.060	1.761
4.4	0.965	1.002	9.796		0.396	1.101	2.830
4.5	1.065	1.015	11.504		0.496	1.134	4.085

Table D.2: Calculated Flows for Variable Crest Weir Lowered and Raised (continued)

	Weir fully lowered				Weir fully raised		
Level (m AOD)	**Head (m)**	C_d	Q (m^3/s)		**Head (m)**	C_d	Q (m^3/s)
4.6	1.165	1.027	13.317		0.596	1.161	5.511
4.7	1.265	1.038	15.230		0.696	1.185	7.096
4.8	1.365	1.048	17.240		0.796	1.206	8.832
4.9	1.465	1.058	19.346		0.896	1.224	10.711
5.0	1.565	1.067	21.545		0.996	1.241	12.727
5.1	1.665	1.076	23.834		1.096	1.257	14.875
5.2	1.765	1.084	26.211		1.196	1.271	17.151
5.3	1.865	1.091	28.674		1.296	1.285	19.549
5.4	1.965	1.099	31.222		1.396	1.297	22.067
5.5	2.065	1.106	33.853		1.496	1.309	24.701
5.6	2.165	1.113	36.566		1.596	1.320	27.449
5.64	2.205	1.116	37.674		1.636	1.324	28.579

Flow over Sluice Gates when closed

Although the thickness of the sluice gates is greater than 2mm, these were considered as full width thin plate weirs and hence the Rehbock formula was considered appropriate for flow over the sluice gates when closed:

$$Q = \left(\frac{2}{3}\right) \sqrt{2g}\ C_d\ b\ (h_1)^{1.5}$$

where

$$C_d = 0.602 + 0.083\,(\frac{h_1}{p})$$

and

p = upstream height of ‘weir’ (i.e. gate)

Treating the sluice gates as thin plate weirs when the gates are closed gives a crest height for the invert of 2.388m AOD, plus the height of the gates (1.940m) i.e. 4.328m AOD. The sidewalls at 5.640m AOD provide the maximum level of this part of the structure. The flow calculations are given in Table D.3 and the rating shown on Figure D.4.

Table D.3: Rating for Flow over Sluice Gates

Level (m AOD)	Head (m)	C_d	Q (m^3/s)
4.3	0.000	0.602	0.000
4.4	0.072	0.605	0.421
4.5	0.172	0.609	1.566
4.6	0.272	0.614	3.136
4.7	0.372	0.618	5.051
4.8	0.472	0.622	7.269
4.9	0.572	0.626	9.764
5.0	0.672	0.631	12.518
5.1	0.772	0.635	15.518
5.2	0.872	0.639	18.754
5.3	0.972	0.644	22.219
5.4	1.072	0.648	25.906
5.5	1.172	0.652	29.809
5.6	1.272	0.656	33.926
5.64	1.312	0.658	35.631

Flow under Sluice Gates when open

The standard equation for flow under a sluice gate is:

$$Q = C_d \sqrt{2g} \; b\, w \sqrt{h_1}$$

where b = width of gate
w = height of gate opening

The above equation was revised to include both drowned and modular flow:

$$Q = C_d \sqrt{2g} \; b\, w \sqrt{(h_1 - 0.5w)}$$

Where; C_d = 0.571 for $Ø > 0.52$
C_d = 0.581 Sin(2.104 $Ø^{0.45}$) for $Ø < 0.52$
$Ø$ = $(w/h_2) - (w/h_1)$
h_2 = downstream head

The ratings for the opening sluice gates are shown on Figure D.5 for 4 levels of opening (0.5m, 1.0m, 1.5m and 1.94m) and the calculations are summarised in Table D.4. The gate structure is assumed to operate as a broad crested weir up to the level of the gate bottom when opened. There will clearly be a transition between these two flow states.

Table D.4: Calculated Flows under Sluice Gates

Level (m AOD)	Head (m)	C_d (Gates)	C_d (BCW)	Q (m^3/s) for Gate Opening (m)			
				0.5m	1m	1.5m	1.94m
2.388	0.001	0.581	0.730	0.000	0.000	0.000	0.000
2.39	0.002	0.581	0.750	0.001	0.001	0.001	0.001
2.4	0.012	0.581	0.760	0.021	0.021	0.021	0.021
2.5	0.112	0.581	0.973	0.759	0.759	0.759	0.759
2.6	0.212	0.581	0.986	2.001	2.001	2.001	2.001
2.7	0.312	0.581	0.990	3.589	3.589	3.589	3.589
2.8	0.412	0.581	0.992	5.459	5.459	5.459	5.459
2.9	0.512	0.581	0.994	8.035	7.573	7.573	7.573
3.0	0.612	0.581	0.995	9.445	9.906	9.906	9.906
3.1	0.712	0.581	0.995	10.670	12.439	12.439	12.439
3.2	0.812	0.581	0.996	11.769	15.158	15.158	15.158
3.3	0.912	0.581	0.996	12.773	18.050	18.050	18.050
3.4	1.012	0.581	0.997	13.704	22.466	21.106	21.106
3.5	1.112	0.581	0.997	14.575	24.562	24.316	24.316
3.6	1.212	0.581	0.997	15.397	26.493	27.675	27.675
3.7	1.312	0.581	0.997	16.178	28.292	31.176	31.176
3.8	1.412	0.581	0.998	16.922	29.984	34.813	34.813
3.9	1.512	0.581	0.998	17.635	31.585	41.111	38.582
4.0	1.612	0.581	0.998	18.321	33.108	43.725	42.477
4.1	1.712	0.581	0.998	18.981	34.565	46.192	46.495
4.2	1.812	0.581	0.998	19.620	35.963	48.533	50.633
4.3	1.912	0.581	0.998	20.238	37.308	50.767	54.886
4.4	2.012	0.581	0.998	20.838	38.607	52.906	62.176
4.5	2.112	0.581	0.998	21.421	39.863	54.962	65.091
4.6	2.212	0.581	0.998	21.989	41.081	56.944	67.881
4.7	2.312	0.581	0.998	22.542	42.263	58.860	70.561
4.8	2.412	0.581	0.998	23.083	43.414	60.714	73.143
4.9	2.512	0.581	0.998	23.610	44.535	62.514	75.636
5.0	2.612	0.581	0.999	24.127	45.628	64.264	78.050
5.1	2.712	0.581	0.999	24.632	46.696	65.967	80.392
5.2	2.812	0.581	0.999	25.127	47.740	67.627	82.667
5.3	2.912	0.581	0.999	25.613	48.761	69.248	84.881
5.4	3.012	0.581	0.999	26.090	49.762	70.831	87.039
5.5	3.112	0.581	0.999	26.558	50.743	72.380	89.145
5.6	3.212	0.581	0.999	27.018	51.705	73.896	91.202
5.64	3.252	0.581	0.999	27.199	52.085	74.494	92.012

Lock mitre gates

The lock mitre gates are assumed to act as a broad crested weir when fully closed and, as with the variable weir, gauged head is substituted for total head to simplify the equations:

$$Q = C_d \ C_v \left(\frac{2}{3}\right)^{1.5} \sqrt{g}\ b\ (h_1)^{1.5}$$

A relationship was provided between C_d and h_1 (Figure D.6) to which a polynomial has been fitted:

$C_d = -0.5143\ h_1^2 + 1.3606\ h_1 + 0.2900$

The rating for flow over the lock gates has therefore been calculated (Table D.5) and is shown on Figure D.7.

Table D.5: Calculated Flows over Lock Gates

Level (m AOD)	Head (m)	C_d	Q (m^3/s)
4.3	0.0	0.290	0.000
4.4	0.1	0.421	0.107
4.5	0.2	0.542	0.388
4.6	0.3	0.652	0.858
4.7	0.4	0.752	1.524
4.8	0.5	0.842	2.385
4.9	0.6	0.921	3.431
5.0	0.7	0.990	4.648
5.1	0.8	1.049	6.016
5.2	0.9	1.098	7.512
5.3	1.0	1.136	9.105
5.4	1.1	1.164	10.764
5.5	1.2	1.182	12.452
5.6	1.3	1.190	14.129
5.64	1.34	1.190	14.788

Lock Sluices

Flow through the lock gate sluices is based on the standard equation for flow through an undershot weir:

$$Q = C_d\ \sqrt{2g}\ \ b\,w\ \sqrt{(h_1 - 0.5w)}$$

where $C_d = 0.574$ for $Ø > 0.79$

$C_d = 0.574\ \mathrm{Sin}(1.74\ Ø^{\,0.43})$ for $Ø < 0.79$

$Ø = (w/h_2) - (w/h_1)$

The calculated rating for the lock gates as an undershot sluice is shown on Figure D.8 and on Table D.6.

Table D.6: Calculated Flow through Lock Gate Sluices

Level (m AOD)	Head (m)	C_d (Gates)	C_d (BCW)	Q (m^3/s)
2.4	0.000	0.574	0.952	0.000
2.5	0.065	0.574	0.952	0.044
2.6	0.165	0.574	0.979	0.181
2.7	0.265	0.574	0.986	0.372
2.8	0.365	0.574	0.989	0.603
2.9	0.465	0.574	0.991	0.868
3.0	0.565	0.574	0.992	1.164
3.1	0.665	0.574	0.993	1.487
3.2	0.765	0.574	0.994	1.836
3.3	0.865	0.574	0.994	2.263
3.4	0.965	0.574	0.994	2.497
3.5	1.065	0.574	0.995	2.710
3.6	1.165	0.574	0.995	2.908
3.7	1.265	0.574	0.995	3.094
3.8	1.365	0.574	0.995	3.269
3.9	1.465	0.574	0.995	3.435
4.0	1.565	0.574	0.996	3.593
4.1	1.665	0.574	0.996	3.745
4.2	1.765	0.574	0.996	3.891
4.3	1.865	0.574	0.996	4.031
4.4	1.965	0.574	0.996	4.167
4.5	2.065	0.574	0.996	4.298
4.6	2.165	0.574	0.996	4.426
4.7	2.265	0.574	0.996	4.550
4.8	2.365	0.574	0.996	4.671
4.9	2.465	0.574	0.996	4.788
5.0	2.565	0.574	0.996	4.903
5.1	2.665	0.574	0.996	5.016
5.2	2.765	0.574	0.996	5.125
5.3	2.865	0.574	0.996	5.233
5.4	2.965	0.574	0.997	5.338
5.5	3.065	0.574	0.997	5.441
5.6	3.165	0.574	0.997	5.543
5.64	3.205	0.574	0.997	5.583

Combined Rating

The rating is therefore a combination of flow over the four components detailed above, each of which has various settings and hence there are numerous possible combinations:

- The variable broad crested weir - fully lowered, fully raised or an intermediate position;
- The 2 sluice gates – fully closed or raised, or an intermediate position;
- The Lock gates – always fully closed and overtopped; and
- The Lock sluice - fully opened or closed or an intermediate position.

Because of the large number of combinations it is not possible to provide an all encompassing rating hence the Agency's use of a computer program to calculate flows based on upstream and downstream water levels, and the various gate, weir and sluice settings. The calculated flows are provided in Table D.7 for the variable broad crest weir fully lowered and fully raised, the sluice gates (open by 1m, 2m and closed) the lock gates (closed) and the lock sluice (fully opened). When the lock sluices are closed the flow is obviously 0.0 cumecs.

Table D.7: Calculated Flows for various elements

	Variable Weir		Sluice Gates			Lock Gates	Lock Sluice
Level (m AOD)	Fully Lowered	Fully Raised	Open by 1m	Open by 2m	Closed and over topped	Closed and over topped	Fully Open
2.388	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2.39	0.000	0.000	0.001	0.001	0.000	0.000	0.000
2.4	0.000	0.000	0.021	0.021	0.000	0.000	0.000
2.5	0.000	0.000	0.759	0.759	0.000	0.000	0.044
2.6	0.000	0.000	2.001	2.001	0.000	0.000	0.181
2.7	0.000	0.000	3.589	3.589	0.000	0.000	0.372
2.8	0.000	0.000	5.459	5.459	0.000	0.000	0.603
2.9	0.000	0.000	7.573	7.573	0.000	0.000	0.868
3.0	0.000	0.000	9.906	9.906	0.000	0.000	1.164
3.1	0.000	0.000	12.439	12.439	0.000	0.000	1.487
3.2	0.000	0.000	15.158	15.158	0.000	0.000	1.836
3.3	0.000	0.000	18.050	18.050	0.000	0.000	2.263
3.4	0.000	0.000	22.466	21.106	0.000	0.000	2.497
3.5	0.121	0.000	24.562	24.316	0.000	0.000	2.710
3.6	0.551	0.000	26.493	27.675	0.000	0.000	2.908
3.7	1.192	0.000	28.292	31.176	0.000	0.000	3.094
3.8	2.008	0.000	29.984	34.813	0.000	0.000	3.269
3.9	2.980	0.000	31.585	38.582	0.000	0.000	3.435
4.0	4.094	0.000	33.108	42.477	0.000	0.000	3.593
4.1	5.339	0.281	34.565	46.495	0.000	0.000	3.745
4.2	6.709	0.899	35.963	50.633	0.000	0.000	3.891
4.3	8.196	1.761	37.308	54.886	0.000	0.000	4.031
4.4	9.796	2.830	38.607	62.176	0.421	0.107	4.167
4.5	11.504	4.085	39.863	65.091	1.566	0.388	4.298
4.6	13.317	5.511	41.081	67.881	3.136	0.858	4.426
4.7	15.230	7.096	42.263	70.561	5.051	1.524	4.550
4.75	16.223	7.946	42.843	71.863	6.124	1.931	4.611
4.8	17.240	8.832	43.414	73.143	7.269	2.385	4.671
4.9	19.346	10.711	44.535	75.636	9.764	3.431	4.788
5.0	21.545	12.727	45.628	78.050	12.518	4.648	4.903
5.1	23.834	14.875	46.696	80.392	15.518	6.016	5.016
5.2	26.211	17.151	47.740	82.667	18.754	7.512	5.125
5.3	28.674	19.549	48.761	84.881	22.219	9.105	5.233
5.4	31.222	22.067	49.762	87.039	25.906	10.764	5.338
5.5	33.853	24.701	50.743	89.145	29.809	12.452	5.441
5.6	36.566	27.449	51.705	91.202	33.926	14.129	5.543
5.64	37.674	28.579	52.085	92.012	35.631	14.788	5.583

A rating has then been calculated for 6 combinations of gate settings as detailed in Table D.8 although there are of course numerous combinations based on intermediate gate, weir and sluice settings.

Table D.8: Calculated Flows for various elements

	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Variable Weir	Lowered	Lowered	Lowered	Raised	Raised	Raised
Sluice Gates	Open by 1m	Open by 2m	Closed and Overtopped	Open by 1m	Open by 2m	Closed and Overtopped
Lock Gates	Closed	Closed	Closed	Closed	Closed	Closed
Lock Sluices	Closed	Closed	Closed	Closed	Closed	Closed

The resulting combined ratings for these 6 cases are shown on Table D.9 and Figure D.9.

Table D.9: Calculated Flows over Lock Gates

Level	Flow (m^3/s)					
(m AOD)	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
2.388	0.000	0.000	0.000	0.000	0.000	0.000
2.39	0.001	0.001	0.000	0.001	0.001	0.000
2.4	0.021	0.021	0.000	0.021	0.021	0.000
2.5	0.759	0.759	0.000	0.759	0.759	0.000
2.6	2.001	2.001	0.000	2.001	2.001	0.000
2.7	3.589	3.589	0.000	3.589	3.589	0.000
2.8	5.459	5.459	0.000	5.459	5.459	0.000
2.9	7.573	7.573	0.000	7.573	7.573	0.000
3.0	9.906	9.906	0.000	9.906	9.906	0.000
3.1	12.439	12.439	0.000	12.439	12.439	0.000
3.2	15.158	15.158	0.000	15.158	15.158	0.000
3.3	18.050	18.050	0.000	18.050	18.050	0.000
3.4	22.466	21.106	0.000	22.466	21.106	0.000
3.5	24.682	24.437	0.121	24.562	24.316	0.000
3.6	27.043	28.226	0.551	26.493	27.675	0.000
3.7	29.484	32.368	1.192	28.292	31.176	0.000
3.8	31.992	36.821	2.008	29.984	34.813	0.000
3.9	34.565	41.562	2.980	31.585	38.582	0.000
4.0	37.202	46.571	4.094	33.108	42.477	0.000
4.1	39.904	51.835	5.339	34.846	46.776	0.281
4.2	42.672	57.342	6.709	36.862	51.532	0.899
4.3	45.504	63.083	8.196	39.069	56.648	1.761
4.4	48.510	72.079	10.324	41.543	65.113	3.358
4.5	51.755	76.983	13.458	44.336	69.564	6.039
4.6	55.256	82.056	17.311	47.450	74.250	9.505
4.7	59.017	87.315	21.805	50.884	79.181	13.671
4.75	60.996	90.017	24.277	52.719	81.740	16.000
4.8	63.039	92.768	26.894	54.631	84.359	18.485
4.9	67.312	98.413	32.541	58.677	89.778	23.906
5.0	71.821	104.243	38.711	63.003	95.425	29.893
5.1	76.546	110.242	45.368	67.588	101.284	36.410
5.2	81.462	116.389	52.477	72.402	107.329	43.417
5.3	86.540	122.661	59.998	77.415	113.535	50.873

Table D.9: Calculated Flows over Lock Gates (continued)

Level (m AOD)	Flow (m^3/s)					
	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
5.4	91.748	129.025	67.892	82.593	119.870	58.736
5.5	97.048	135.450	76.115	87.895	126.298	66.962
5.6	102.400	141.897	84.621	93.283	132.780	75.504
5.64	104.546	144.474	88.093	95.451	135.379	78.998

Extended Ratings

As detailed above the minimum upstream bank elevation of 4.750m AOD provides the rating maximum and once water levels exceed this level the river will overtop and bypass the gauging structure. Thus whilst the rating calculations have been extended to the structure maximum of 5.640m AOD this does not take account of over bank and flow bypassing. The upstream bank level of 4.750m AOD therefore forms the rating maximum although extension to 5.640m AOD may be possible on the basis that the upstream banks could be raised in the future.

Drowned Flow Conditions

Ideally the calculation of drowned flow should be undertaken for each component of the structure based on upstream and downstream water level measurements but this depends on the settings of adjacent structures and the modular limit for the entire structure is therefore variable. For example, details of the modular limit for the variable weir at its lowest position (3.482m AOD) were provided but this will change depending not only on the setting of the weir, but also the adjacent sluice gates since this will affect the upstream water levels. Drowned flow adjustments were included in the calculation of C_d.

According to the British Standards the modular limit for a broad crested weir will occur when the ratio of the downstream to upstream head is 0.66, although a value of 0.7 was calculated for the variable weir based on hydraulic model tests. The HECRAS model described in Section D.7 has been used to determine the point at which modular flow occurs for various gate settings based on the submergence ratio of 0.66. This allows the maximum limit of the theoretical equations to be determined rather than developing drowned flow equations for each combination of gate settings which may prove complex and not include interactions between each structure.

A HECRAS model has been set up and run for cases 2, 3, 4 and 5 detailed in Table D.8. The results of the model in terms of upstream and downstream water levels and head above the minimum structure level, and hence the submergence ratios are given in Tables D.10 to D.13.

Table D.10; Submergence Ratio for Case 2 – Sluice Gates Open and Variable Weir Lowered

Q (m^3/s)	Downstream Water Level (m AOD)	Upstream Water Level (m AOD)	Downstream Head (m)	Upstream Head (m)	Ratio
0.5	1.00	2.49	-1.388	0.102	
1	1.11	2.54	-1.278	0.152	
2	1.26	2.62	-1.128	0.232	
3	1.39	2.69	-0.998	0.302	
4	1.49	2.75	-0.898	0.362	
5	1.58	2.80	-0.808	0.412	
6	1.67	2.85	-0.718	0.462	
7	1.74	2.90	-0.648	0.512	
8	1.82	2.95	-0.568	0.562	
9	1.89	2.99	-0.498	0.602	
10	1.95	3.04	-0.438	0.652	
15	2.24	3.23	-0.148	0.842	
20	2.46	3.41	0.072	1.022	0.070
25	2.67	3.56	0.282	1.172	0.241
30	2.85	3.72	0.462	1.332	0.347
40	3.16	3.96	0.772	1.572	0.491
50	3.45	4.16	1.062	1.772	0.599
60	3.70	4.34	1.312	1.952	0.672
70	3.94	4.51	1.552	2.122	0.731
80	4.14	4.68	1.752	2.292	0.764
90	4.33	4.83	1.942	2.442	0.795
100	4.49	4.98	2.102	2.592	0.811
120	4.78	5.23	2.392	2.842	0.842
140	5.06	5.46	2.672	3.072	0.870
160	5.31	5.68	2.922	3.292	0.888
180	5.55	5.90	3.162	3.512	0.900
200	5.78	6.13	3.392	3.742	0.906
250	6.31	6.63	3.922	4.242	0.925
300	6.80	7.08	4.412	4.692	0.940
400	7.68	7.93	5.292	5.542	0.955

Table D.11: Submergence Ratio for Case 3 – Sluice Gates Closed and Variable Weir Lowered

Q (m^3/s)	Downstream Water Level (m AOD)	Upstream Water Level (m AOD)		Downstream Head (m)	Upstream Head (m)	Ratio
0.5	0.93	3.58		-2.505	0.145	
1	1.01	3.66		-2.425	0.225	
2	1.16	3.79		-2.275	0.355	
3	1.28	3.90		-2.155	0.465	
4	1.39	3.99		-2.045	0.555	
5	1.48	4.08		-1.955	0.645	
6	1.56	4.16		-1.875	0.725	
7	1.64	4.24		-1.795	0.805	
8	1.70	4.31		-1.735	0.875	
9	1.76	4.39		-1.675	0.955	
10	1.82	4.45		-1.615	1.015	
15	2.08	4.64		-1.355	1.205	
20	2.28	4.75		-1.155	1.315	
25	2.46	4.86		-0.975	1.425	
30	2.63	4.96		-0.805	1.525	
40	2.91	5.14		-0.525	1.705	
50	3.15	5.31		-0.285	1.875	
60	3.37	5.47		-0.065	2.035	
70	3.57	5.62		0.135	2.185	0.062
80	3.76	5.76		0.325	2.325	0.140
90	3.92	5.89		0.485	2.455	0.198
100	4.07	6.02		0.635	2.585	0.246
120	4.32	6.27		0.885	2.835	0.312
140	4.53	6.46		1.095	3.025	0.362
160	4.73	6.63		1.295	3.195	0.405
180	4.91	6.79		1.475	3.355	0.440
200	5.09	6.94		1.655	3.505	0.472
250	5.50	7.29		2.065	3.855	0.536
300	5.88	7.62		2.445	4.185	0.584
400	6.55	8.23		3.115	4.795	0.650

Table D.12: Submergence Ratio for Case 4 – Sluice Gates Open and Variable Weir Raised

Q (m^3/s)	Downstream Water Level (m AOD)	Upstream Water Level (m AOD)		Downstream Head (m)	Upstream Head (m)	Ratio
0.5	0.93	2.49		-1.458	0.102	
1	1.01	2.54		-1.378	0.152	
2	1.16	2.62		-1.228	0.232	
3	1.28	2.69		-1.108	0.302	
4	1.39	2.75		-0.998	0.362	
5	1.48	2.80		-0.908	0.412	
6	1.56	2.85		-0.828	0.462	
7	1.63	2.90		-0.758	0.512	
8	1.70	2.95		-0.688	0.562	
9	1.76	2.99		-0.628	0.602	
10	1.82	3.04		-0.568	0.652	
15	2.08	3.23		-0.308	0.842	
20	2.28	3.41		-0.108	1.022	
25	2.46	3.56		0.072	1.172	0.061
30	2.63	3.72		0.242	1.332	0.182
40	2.91	3.99		0.522	1.602	0.326
50	3.15	4.25		0.762	1.862	0.409
60	3.37	4.48		0.982	2.092	0.469
70	3.57	4.70		1.182	2.312	0.511
80	3.76	4.86		1.372	2.472	0.555
90	3.92	5.01		1.532	2.622	0.584
100	4.07	5.14		1.682	2.752	0.611
120	4.32	5.38		1.932	2.992	0.646
140	4.53	5.61		2.142	3.222	0.665
160	4.73	5.84		2.342	3.452	0.678
180	4.91	6.05		2.522	3.662	0.689
200	5.09	6.26		2.702	3.872	0.698
250	5.50	6.74		3.112	4.352	0.715
300	5.88	7.06		3.492	4.672	0.747
400	6.55	7.68		4.162	5.292	0.786

Table D.13: Submergence Ratio for Case 5 – Sluice Gates Closed and Variable Weir Raised

Q (m^3/s)	Downstream Water Level (m AOD)	Upstream Water Level (m AOD)		Downstream Head (m)	Upstream Head (m)	Ratio
0.5	0.93	4.15		-3.074	0.146	
1	1.01	4.23		-2.994	0.226	
2	1.16	4.36		-2.844	0.356	
3	1.28	4.44		-2.724	0.436	
4	1.39	4.47		-2.614	0.466	
5	1.48	4.51		-2.524	0.506	
6	1.56	4.54		-2.444	0.536	
7	1.63	4.57		-2.374	0.566	
8	1.70	4.60		-2.304	0.596	
9	1.76	4.63		-2.244	0.626	
10	1.82	4.66		-2.184	0.656	
15	2.08	4.79		-1.924	0.786	
20	2.28	4.90		-1.724	0.896	
25	2.46	5.01		-1.544	1.006	
30	2.63	5.11		-1.374	1.106	
40	2.91	5.29		-1.094	1.286	
50	3.15	5.46		-0.854	1.456	
60	3.37	5.62		-0.634	1.616	
70	3.57	5.76		-0.434	1.756	
80	3.76	5.91		-0.244	1.906	
90	3.92	6.04		-0.084	2.036	
100	4.07	6.17		0.066	2.166	0.030
120	4.32	6.38		0.316	2.376	0.133
140	4.53	6.56		0.526	2.556	0.206
160	4.73	6.72		0.726	2.716	0.267
180	4.91	6.88		0.906	2.876	0.315
200	5.09	7.03		1.086	3.026	0.359
250	5.50	7.39		1.496	3.386	0.442
300	5.88	7.71		1.876	3.706	0.506
400	6.55	8.32		2.546	4.316	0.590

Table D.14: Summary of Modular Limit Calculations

Case	Sluice Gates	Variable Weir	Modular Flow (m^3/s)	Upstream Head (m)	Upstream Water Level (m AOD)
2	Open	Low	58.3	1.922	4.310
3	Closed	Low	> 400	> 4.795	> 8.23
4	Open	Raised	134.9	3.164	5.552
5	Closed	Raised	> 400	>4.316	> 8.32

Table D.14 summaries the modular limit calculations for the 4 case studies presented in Tables D.10 to D.13 and suggests that when the sluice gates are lowered (Cases 3 and 5) the modular limit is greater than 400 m^3/s and upstream water levels will be in excess of 8.23m. This is well above the structure limit of 5.64m AOD and suggests that drowned flow will not occur. When the sluice gates are open (Cases 2 and 4) the control is at a lower level and drowned flow is predicted to occur at 58 m^3/s when the variable weir is in its lower position, or 135 m^3/s when the variable weir is raised. These flows equate to upstream water levels of 4.31m AOD and 5.552m AOD respectively. Again this suggests that drowned flow is unlikely except when the sluice gates are open and the variable weir is at its lowest position.

Although the HECRAS model is uncalibrated it does allow the applicability of the modular flow equation to be determined. At this structure the modular flow equations apply to almost all flow conditions but these limits should be considered to determine when drowned flow conditions occur. One practical problem at this site was the lack of suitable calibration data for the model.

D.5 The Velocity-Extrapolation Method

The three approaches to the velocity extrapolation method rely on calculating a best-fit line between velocity and stage, hydraulic radius and $AR^{2/3}$ using the existing rating to calculate flow and hence velocity. Again, this approach is not recommended at this site and the theoretical ratings should be used to calculate flows to the structure maximum level.

D.6 Slope-Area Method

The Slope-Area Method is based upon estimating the roughness and friction slope of the channel at the gauging station. BS ISO 1100-2 states that these methods can be used to extrapolate the high end of rating curves under channel control and, when properly applied, are the most hydraulically 'correct' of all the simple techniques and as such are to be generally preferred. There are essentially two approaches to the Slope-Area method:

- The Simple Channel Approach; and
- The Divided Channel Method (DCM).

Simple Channel Approach

As with other approaches detailed above it is considered there is little benefit in using the simple channel approach of the Slope-Area method based on Manning's equation. The theoretical hydraulic equations offer a rational basis for calculating the stage-flow relationship for in-bank flows over the structure particularly as the top of the structure is above the minimum bank elevation. If the structure was within or below the upstream bank levels there may be benefit in using the simple channel approach once the channel becomes the hydraulic control. For such situations the divided channel method should preferably be used to extend a rating since this allows for in bank flows (based on the hydraulic equations) and the floodplain or over structure flows based on different channel roughness and slope. The use of the DCM is described below.

Divided Channel Method

An extension of the simple channel approach to the Slope-Area method is the Divided Channel Method (DCM). In this method the flow along the left and right bank floodplains is assumed to be separate to the main channel and each is calculated according to the Slope-Area method. The advantage of the DCM over Slope-Area is that each compartment can be ascribed different values of channel slope and roughness, which may be more appropriate for floodplain flow. At this site the in bank flows are calculated based on the hydraulic equations described in Section D.1. The overbank flows (left and right bank) are calculated using the Slope-Area method and the flows from the three sections of channel (left, right and main channel) are combined to give the total flow.

The division of the channel into three sections is based on the cross-section provided in Figure D.10. This cross-section, based on a combination of the upstream cross-sections, provides values of A, P and hence R to which values of channel slope (s) and roughness (n) are applied to give the estimated flow using Manning's equation. A summary of the calculations is given in Table D.15. The same slope is assumed for all three compartments.

Table D.15: Slope-Area (Divided Channel Method)

	Main Channel	Left Channel					Right Channel					
H **(m)**	***Q*** **(m^3/s)**	***A*** **(m^2)**	***R*** **(m)**	***s***	***n***	***Q*-L** **(m^3/s)**	***A*** **(m^2)**	***R*** **(m)**	***s***	***n***	***Q*-R** **(m^3/s)**	***Q*total** **(m^3/s)**
2.388	0.000											0.000
2.39	0.000											0.000
2.4	0.000											0.000
2.5	0.000											0.000
2.6	0.000											0.000
2.7	0.000											0.000
2.8	0.000											0.000
2.9	0.000											0.000
3.0	0.000											0.000
3.1	0.000											0.000
3.2	0.000											0.000
3.3	0.000											0.000
3.4	0.000											0.000
3.5	0.121											0.121
3.6	0.551											0.551
3.7	1.192											1.192
3.8	2.008											2.008
3.9	2.980											2.980
4.0	4.094											4.094
4.1	5.339											5.339
4.2	6.709											6.709
4.3	8.196											8.196
4.4	10.324											10.324
4.5	13.458											13.458
4.6	17.311											17.311
4.7	21.805											21.805
4.75	24.277	2.088	0.040	0.00044	0.06	0.085						24.363
4.8	26.894	4.869	0.080	0.00044	0.06	0.316	0.019	0.025	0.00044	0.06	0.001	27.211
4.9	32.541	10.430	0.147	0.00044	0.06	1.014	0.172	0.075	0.00044	0.06	0.011	33.565
5.0	38.711	15.991	0.196	0.00044	0.06	1.888	0.549	0.105	0.00044	0.06	0.043	40.641
5.1	45.368	27.703	0.311	0.00044	0.06	4.444	1.823	0.223	0.00044	0.06	0.234	50.046
5.2	52.477	36.264	0.382	0.00044	0.06	6.672	2.049	0.221	0.00044	0.06	0.262	59.410
5.3	59.998	44.826	0.444	0.00044	0.06	9.127	3.007	0.321	0.00044	0.06	0.492	69.618
5.4	67.892	57.042	0.551	0.00044	0.06	13.405	3.975	0.420	0.00044	0.06	0.779	82.076
5.5	76.115	67.303	0.642	0.00044	0.06	17.519	4.952	0.518	0.00044	0.06	1.116	94.750
5.6	84.621	77.565	0.732	0.00044	0.06	22.017	5.938	0.615	0.00044	0.06	1.501	108.139
5.64	88.093	81.669	0.767	0.00044	0.06	23.917	6.335	0.653	0.00044	0.06	1.668	113.677

The DCM rating compared to the theoretical rating is shown on Figure D.11 although there are no flood gaugings to determine which of these two ratings is preferable or the most appropriate.

D.7 Hydraulic Modelling

A steady state HECRAS hydraulic model has been constructed based on the available topographical cross-sections and estimates of Manning's n given in the ISIS data files. A steady state model has been used. The derived ratings for the sluice gates open and closed is shown on Table D.16 and on Figure D.12 for a section immediately upstream of the control structure.

Table D.16: HECRAS Steady State Model Rating

	Gates Open		Gates Closed	
Q **(m^3/s)**	Water Surface Elevation **(m AOD)**	**Head (m)**	**Water Surface Elevation (M AOD)**	**Head (m)**
0.5	2.49	3.49	3.58	4.58
1	2.54	3.54	3.66	4.66
2	2.62	3.62	3.79	4.79
3	2.69	3.69	3.90	4.90
4	2.75	3.75	3.99	4.99
5	2.80	3.80	4.08	5.08
6	2.85	3.85	4.16	5.16
7	2.90	3.90	4.24	5.24
8	2.95	3.95	4.31	5.31
9	2.99	3.99	4.39	5.39
10	3.04	4.04	4.45	5.45
15	3.23	4.23	4.64	5.64
20	3.41	4.41	4.75	5.75
25	3.56	4.56	4.86	5.86
30	3.72	4.72	4.96	5.96
40	3.96	4.96	5.14	6.14
50	4.16	5.16	5.31	6.31
60	4.34	5.34	5.47	6.47
70	4.51	5.51	5.62	6.62
80	4.68	5.68	5.76	6.76
90	4.83	5.83	5.89	6.89
100	4.99	5.99	6.02	7.02
120	5.24	6.24	6.27	7.27
140	5.47	6.47	6.46	7.46
160	5.69	6.69	6.63	7.63
180	5.90	6.90	6.79	7.79
200	6.11	7.11	6.94	7.94
250	6.59	7.59	7.29	8.29
300	6.98	7.98	7.62	8.62
400	7.59	8.59	8.23	9.23

The results (Figure D.12) show no major difference between critical depth and normal depth downstream boundary conditions.

The HECRAS derived rating provides a reasonable fit to the theoretical rating, but this is based on an uncalibrated model and hence the accuracy may be questionable. For situations where the structure becomes drowned and the channel becomes the main control the HECRAS model has advantages providing that the cross-sectional extent includes all areas where floodplain flow occurs and areas of dead storage or no flow are identified.

D.8 Comparison of Methods

A comparison of the methods is provided in Figure D.13 and on linear axes on Figure D.14. As detailed above, for complex structures involving a combination of hydraulic gates or weirs, many of the available methods for rating extension may not be appropriate and the theoretical equations provide the best solution. Once these have been developed there is little to be gained in undertaking simple extension, log extension, velocity extrapolation or Slope-Area calculations. The theoretical equations can be combined with the Slope-Area method using the Divided Channel Method to provide a rating above the limits of the structure. Also the development of a simple hydraulic model using HECRAS, although uncalibrated, appears to match the theoretical rating and could be used as the basis of a flood rating subject to calibration.

Thus several of the methods are not applicable to complex weir structures and it is therefore recommended that the 'best estimate' rating extension at this site is adopted from one of the following:

- Theoretical equations for the structures; and
- The theoretical equation combined with the DCM method for floodplain flow.

The theoretical equations offer a sound basis for rating calculation an extension for compound flow control structures. It is recommended that these equations are used combined with the DCM method for floodplain flow, subject to confirmation of the drowned flow limits and derivation of drowned flow equations where appropriate.

Figure D.1 Case Study 3: Cross Section of Sluice

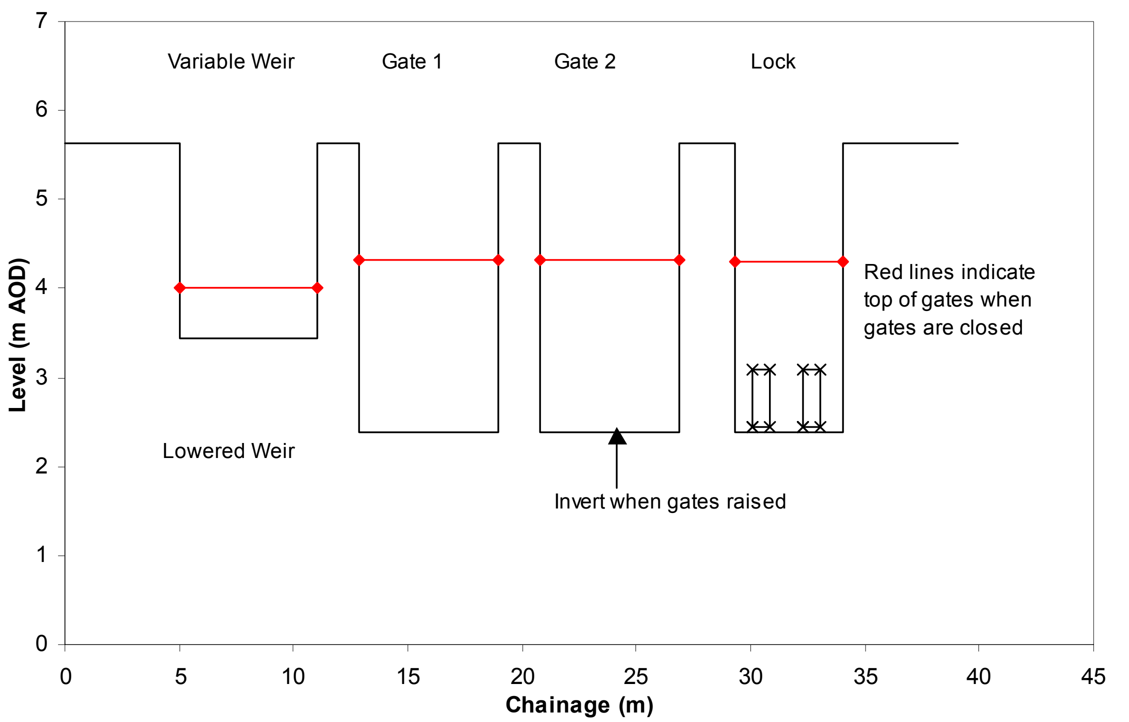
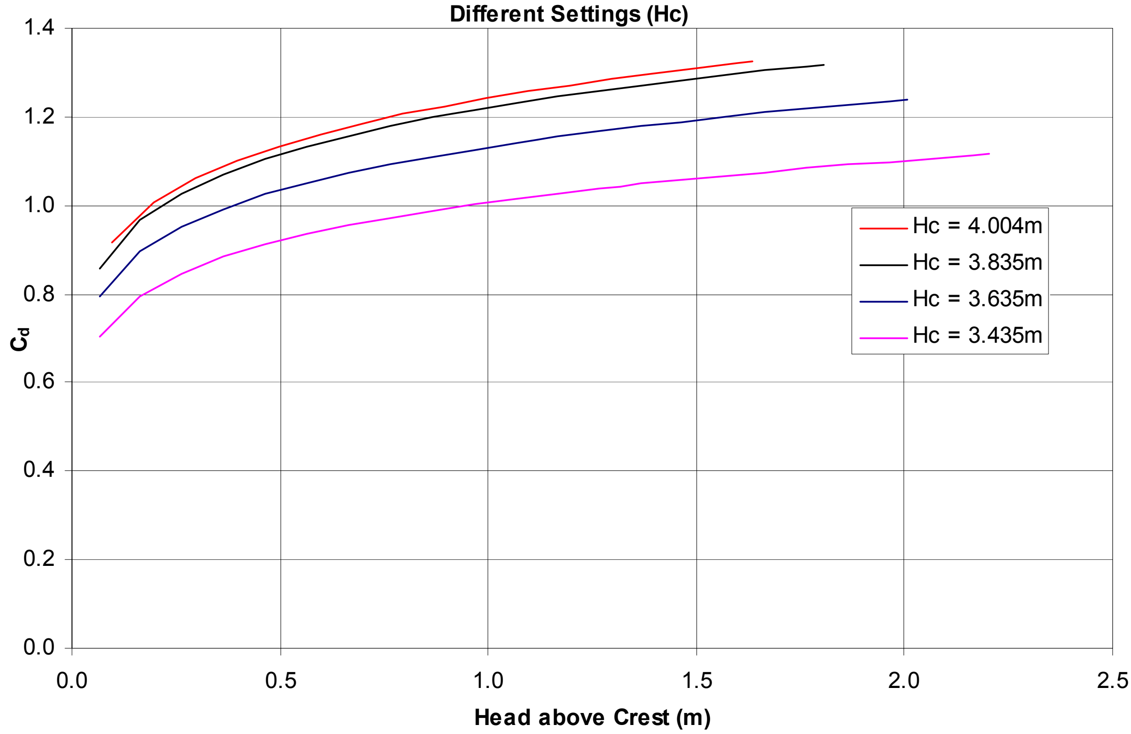


Figure D.2 Case Study 3: Coefficient of Discharge (C_d) for Variable Crest Weir at Different Settings (Hc)



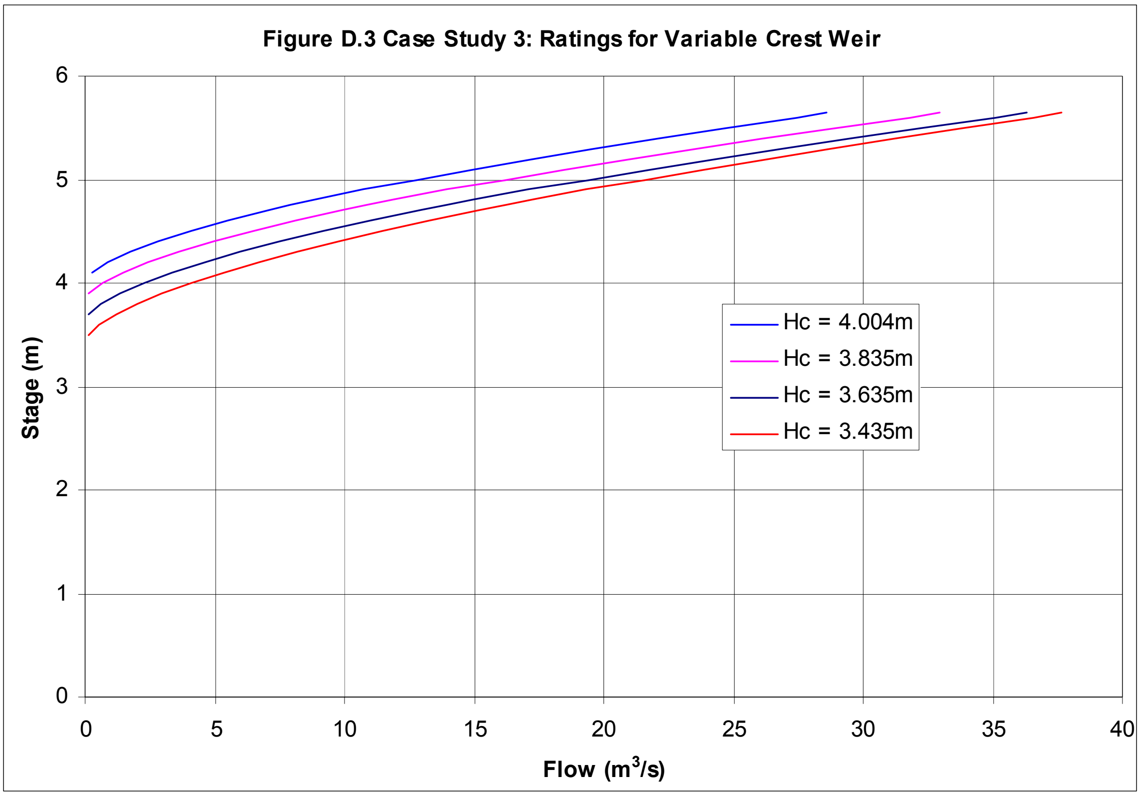


Figure D.3 Case Study 3: Ratings for Variable Crest Weir
Stage (m)
Flow (m³/s)
Hc = 4.004m
Hc = 3.835m
Hc = 3.635m
Hc = 3.435m
0
1
2
3
4
5
6
0
5
10
15
20
25
30
35
40

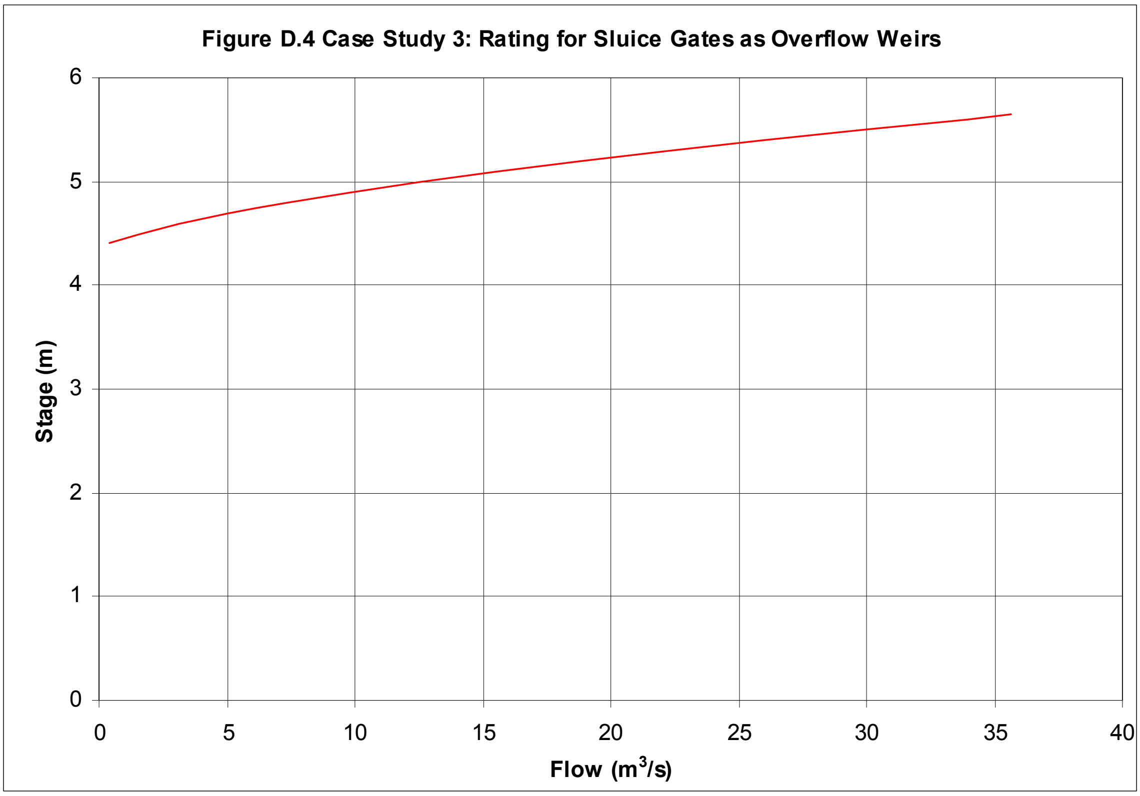


Figure D.4 Case Study 3: Rating for Sluice Gates as Overflow Weirs
Stage (m)
Flow (m³/s)
0
1
2
3
4
5
6
0
5
10
15
20
25
30
35
40

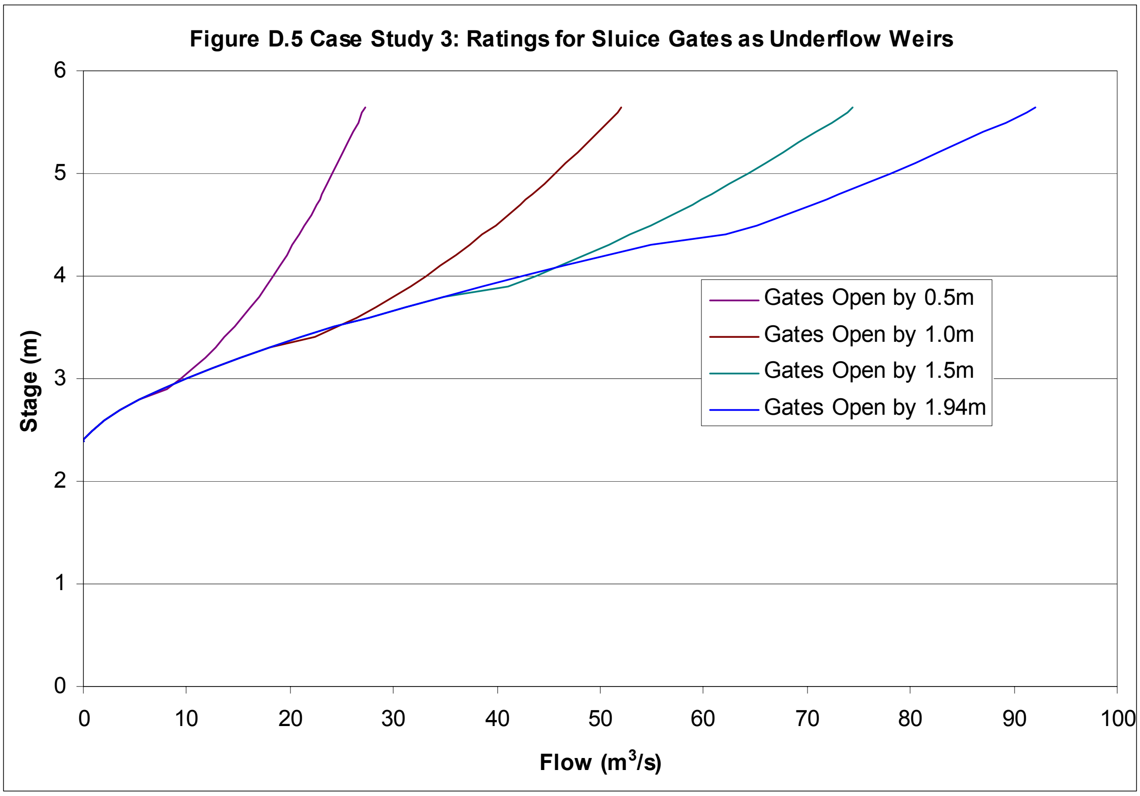


Figure D.5 Case Study 3: Ratings for Sluice Gates as Underflow Weirs
6
5
4
3
2
1
0
Stage (m)
0
10
20
30
40
50
60
70
80
90
100
Flow (m3/s)
Gates Open by 0.5m
Gates Open by 1.0m
Gates Open by 1.5m
Gates Open by 1.94m

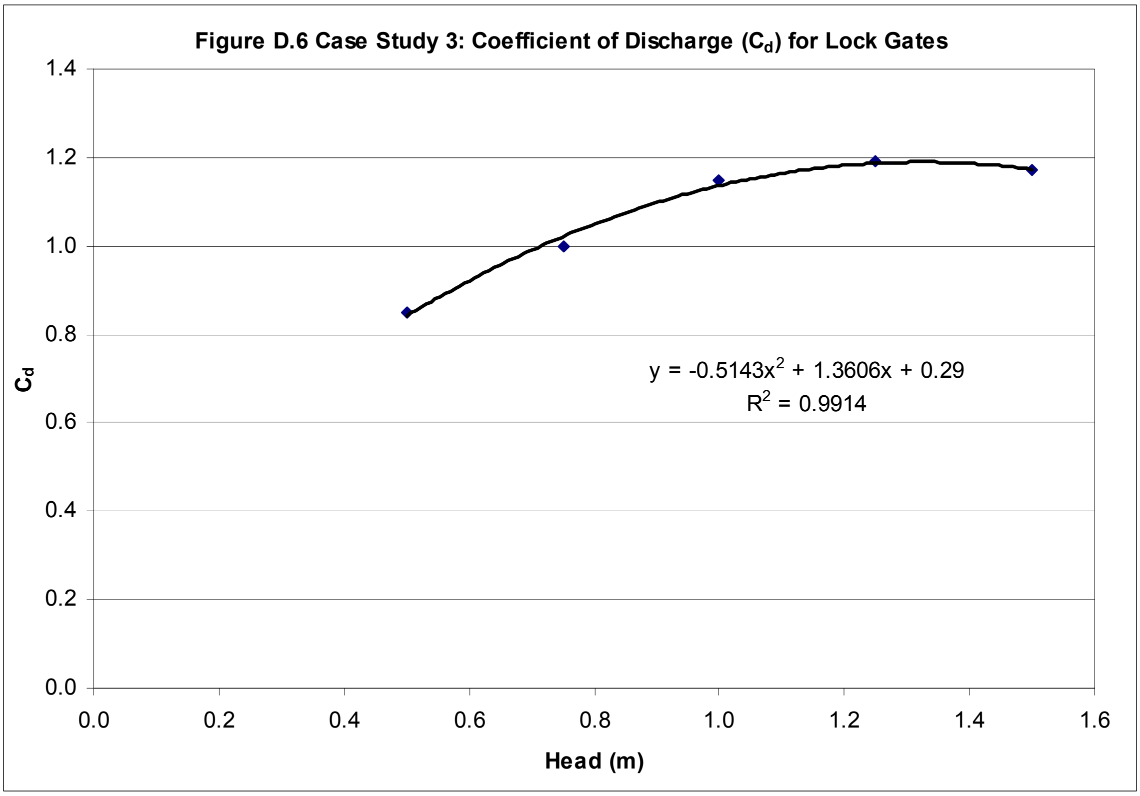


Figure D.6 Case Study 3: Coefficient of Discharge (Cd) for Lock Gates
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Cd
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
Head (m)
y = -0.5143x2 + 1.3606x + 0.29
R2 = 0.9914

Figure D.7 Case Study 3: Rating for Flow over Lock Gates

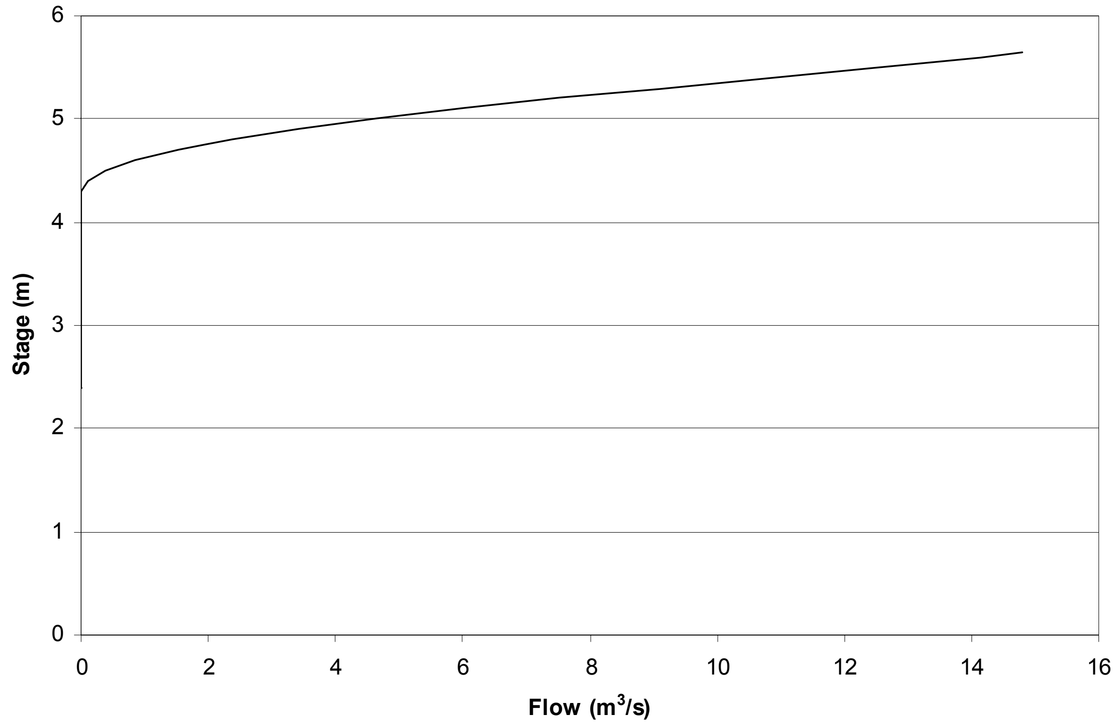
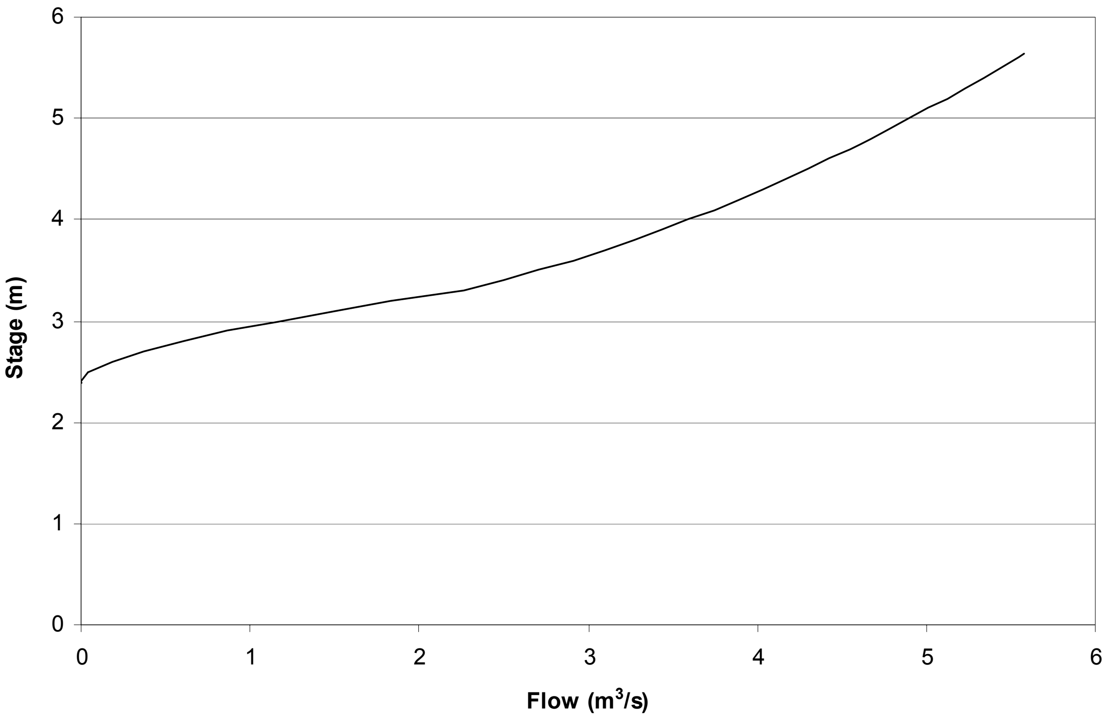


Figure D.8 Case Study 3: Rating for Flow through Lock Gate Sluices



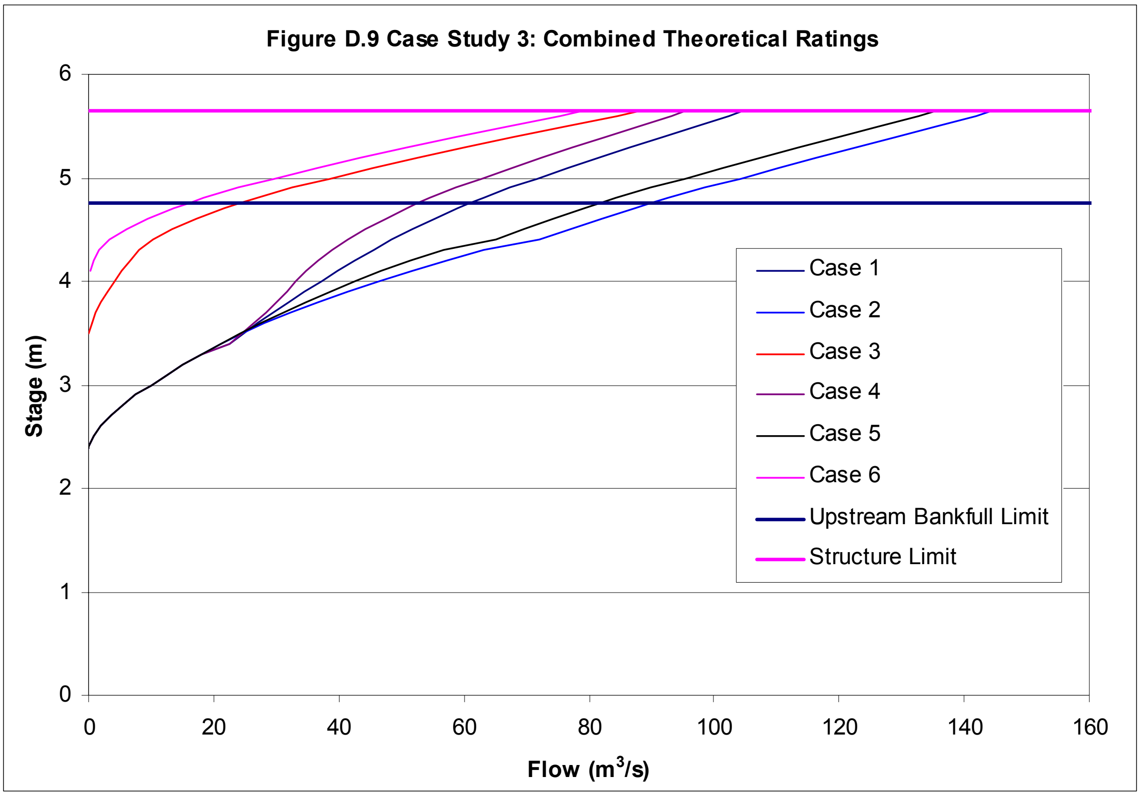


Figure D.9 Case Study 3: Combined Theoretical Ratings
Stage (m)
Flow (m3/s)
0
1
2
3
4
5
6
0
20
40
60
80
100
120
140
160
Case 1
Case 2
Case 3
Case 4
Case 5
Case 6
Upstream Bankfull Limit
Structure Limit

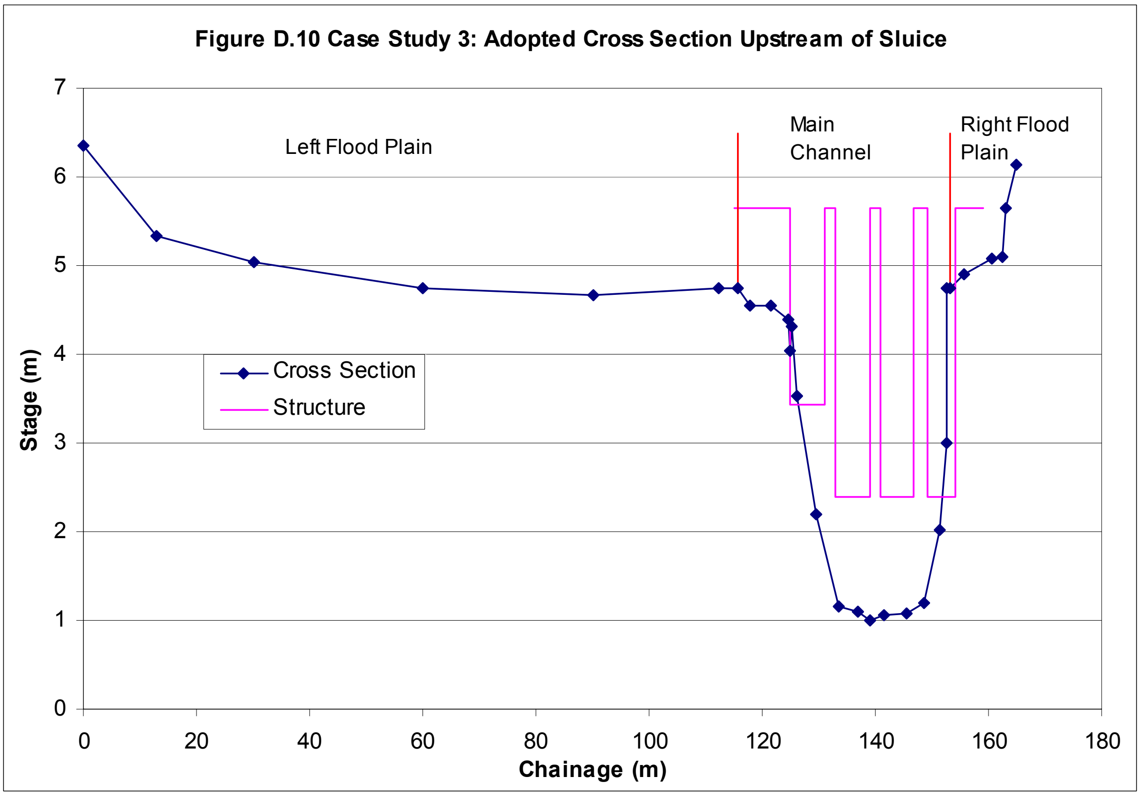


Figure D.10 Case Study 3: Adopted Cross Section Upstream of Sluice
Stage (m)
Chainage (m)
Left Flood Plain
Main Channel
Right Flood Plain
Cross Section
Structure
0
1
2
3
4
5
6
7
0
20
40
60
80
100
120
140
160
180

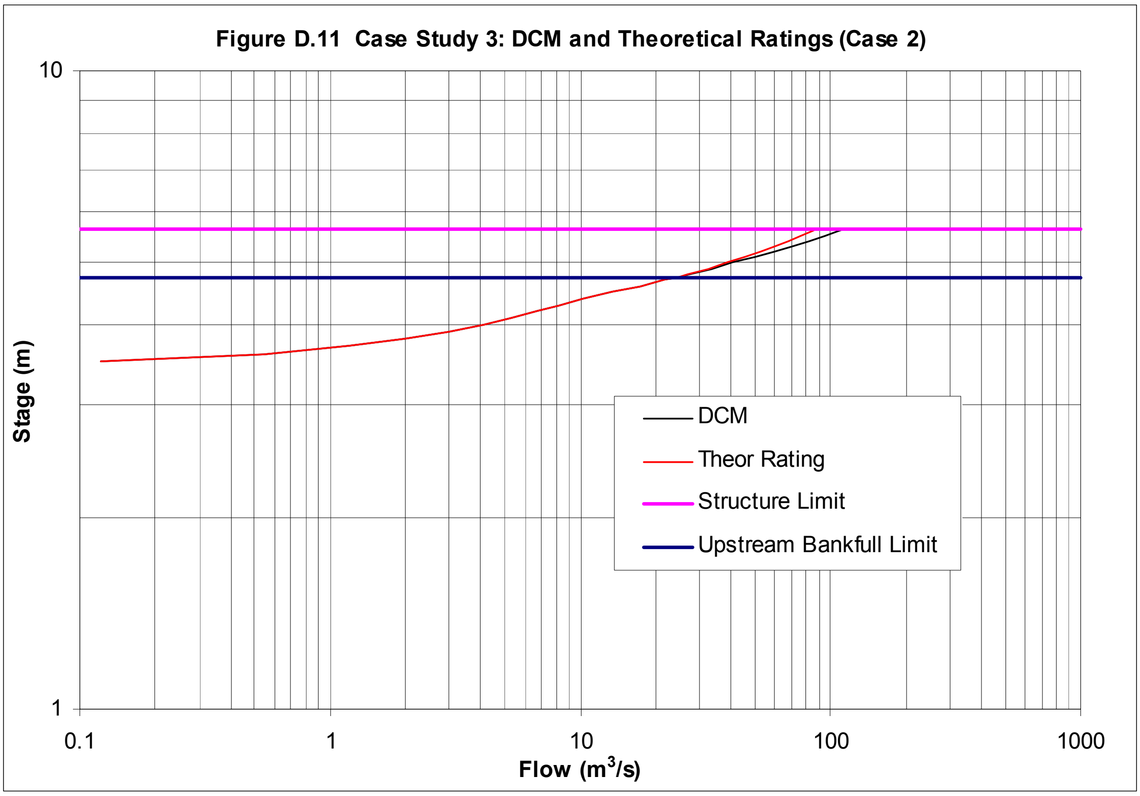


Figure D.11 Case Study 3: DCM and Theoretical Ratings (Case 2)
Stage (m)
10
1
0.1
1
10
100
1000
Flow (m3/s)
DCM
Theor Rating
Structure Limit
Upstream Bankfull Limit

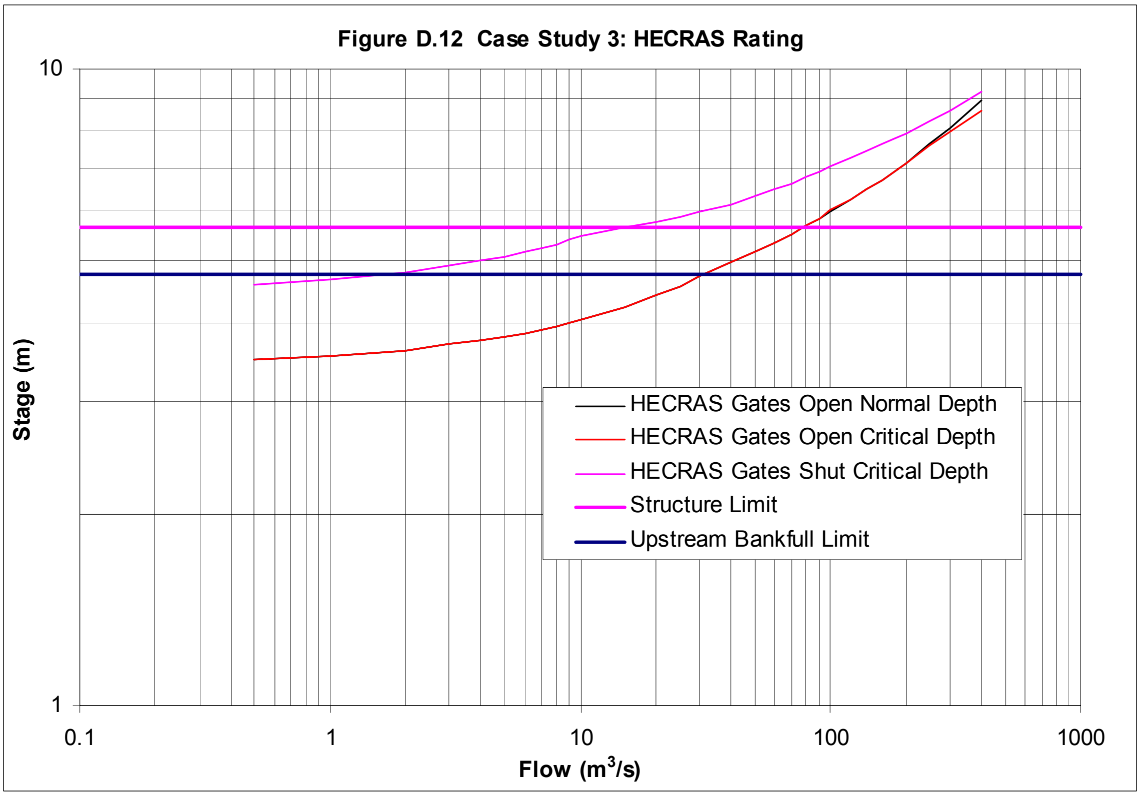


Figure D.12 Case Study 3: HECRAS Rating
Stage (m)
10
1
0.1
1
10
100
1000
Flow (m3/s)
HECRAS Gates Open Normal Depth
HECRAS Gates Open Critical Depth
HECRAS Gates Shut Critical Depth
Structure Limit
Upstream Bankfull Limit

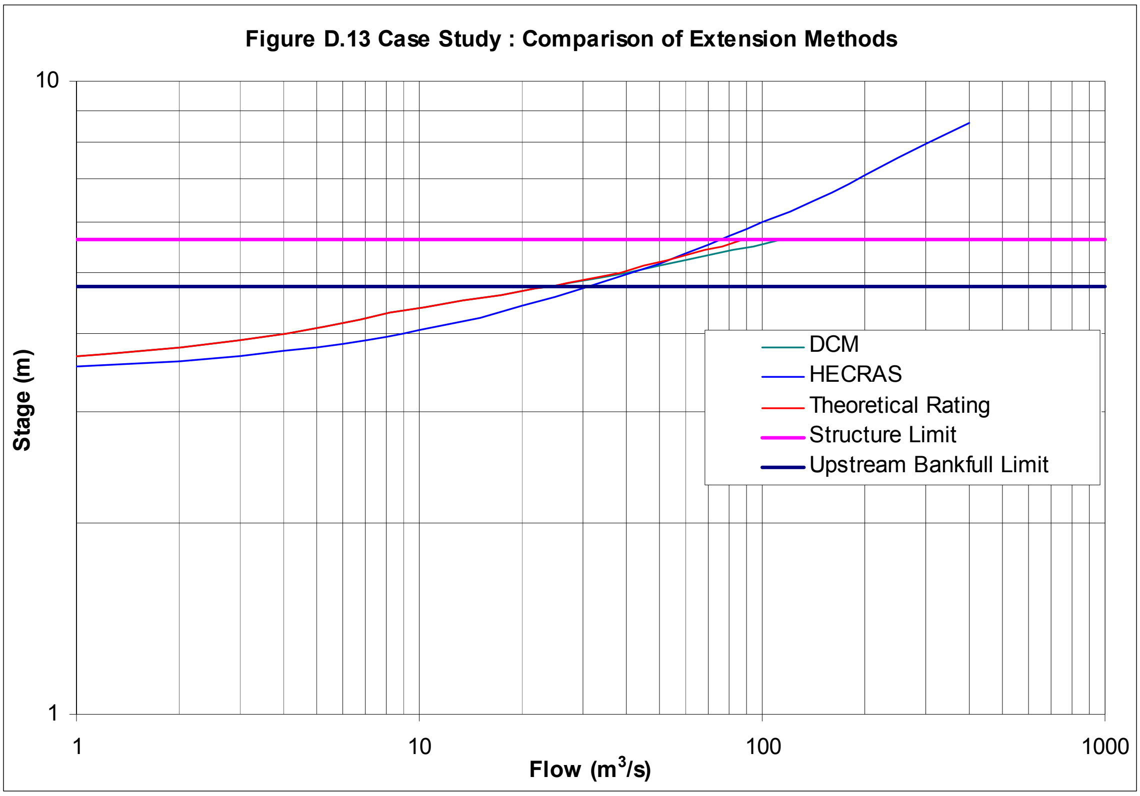


Figure D.13 Case Study : Comparison of Extension Methods
10
1
Stage (m)
1
10
100
1000
Flow (m3/s)
DCM
HECRAS
Theoretical Rating
Structure Limit
Upstream Bankfull Limit

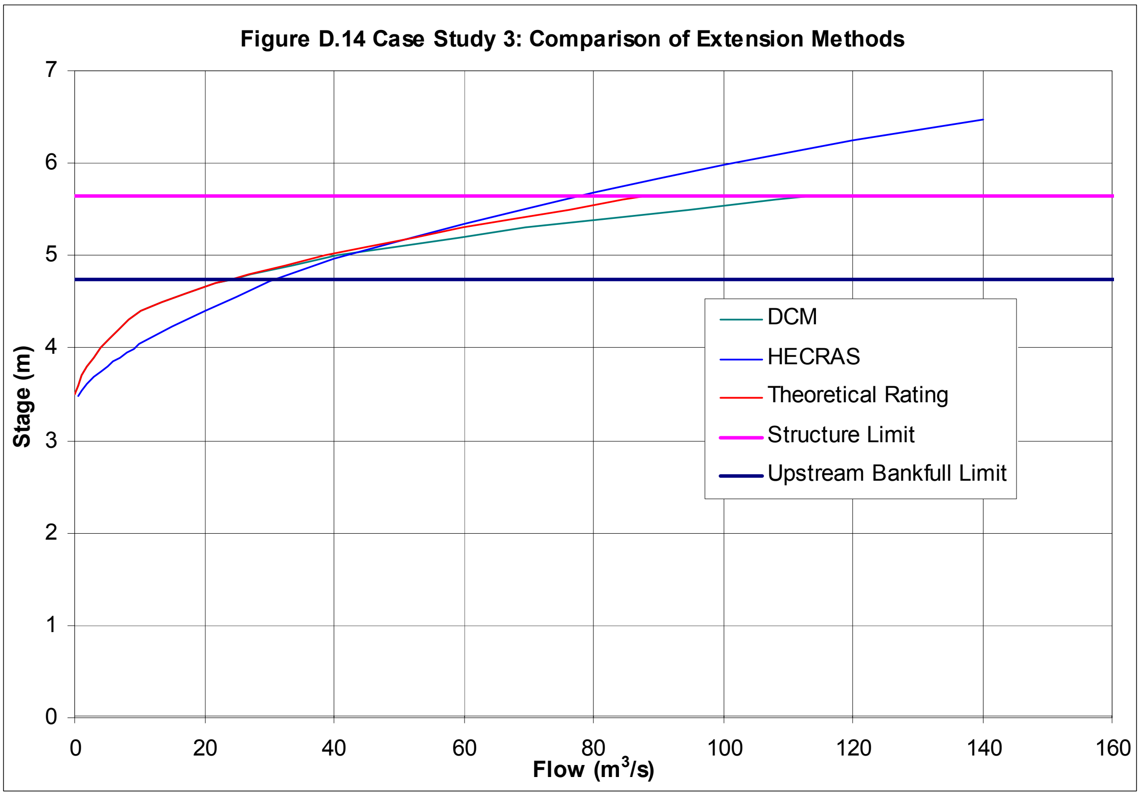


Figure D.14 Case Study 3: Comparison of Extension Methods
7
6
5
4
3
2
1
0
Stage (m)
0
20
40
60
80
100
120
140
160
Flow (m3/s)
DCM
HECRAS
Theoretical Rating
Structure Limit
Upstream Bankfull Limit

APPENDIX E

Gauged data for Case Studies

Table E.1: Gauged Data for Case Study 4

Stage (m)	Discharge (m^3/s)	Stage (m)	Discharge (m^3/s)
0.162	0.730	0.291	2.190
0.170	0.832	0.306	2.423
0.174	0.839	0.378	3.362
0.179	0.839	0.418	4.178
0.187	0.947	0.446	4.682
0.198	1.056	0.479	5.476
0.201	1.121	0.542	6.589
0.206	1.050	0.734	11.406
0.216	1.189	0.755	11.576
0.222	1.262	1.305	29.811
0.231	1.384	2.270	66.704
0.236	1.528	2.423	75.407
0.239	1.425	2.469	79.917
0.246	1.540	2.507	79.297
0.261	1.741	2.559	85.324
0.271	1.942	2.611	88.710
0.271	1.798	2.664	93.199
0.273	1.878		

Table E.2: Gauged data for Case Studies 1, 5 and 7

Stage (m)	Discharge (m^3/s)	Stage (m)	Discharge (m^3/s)	Stage (m)	Discharge (m^3/s)
0.239	3.301	0.389	6.411	0.692	15.065
0.243	3.892	0.392	5.692	0.704	14.868
0.248	3.132	0.402	6.421	0.716	19.883
0.249	3.664	0.405	6.586	0.717	21.221
0.252	3.395	0.411	7.113	0.726	17.167
0.258	3.281	0.419	7.921	0.733	18.862
0.268	3.724	0.421	7.028	0.741	21.019
0.27	3.231	0.448	7.535	0.752	20.988
0.271	3.777	0.449	7.347	0.757	22.516
0.273	3.806	0.449	7.231	0.763	23.319
0.278	4.006	0.449	7.923	0.764	21.901
0.284	3.578	0.453	8.177	0.861	28.292
0.284	4.066	0.454	8.296	0.867	25.290
0.293	3.953	0.455	8.705	0.868	27.422
0.321	4.585	0.456	8.135	0.891	25.875
0.321	3.381	0.459	8.253	0.895	25.737
0.322	4.424	0.463	8.160	0.926	30.852
0.322	4.414	0.463	8.467	0.937	37.386
0.322	4.369	0.465	8.122	0.961	34.339
0.324	4.926	0.478	5.443	1.018	33.389
0.325	4.357	0.493	9.217	1.020	36.004
0.328	5.246	0.499	9.211	1.025	33.842
0.329	4.719	0.509	8.498	1.051	39.729
0.329	4.833	0.513	9.894	1.112	47.964
0.329	4.658	0.518	10.858	1.163	40.457
0.331	4.984	0.519	10.998	1.224	44.524
0.332	5.058	0.544	12.740	1.232	44.872
0.333	4.774	0.553	11.483	1.279	58.213
0.340	4.949	0.563	12.151	1.285	52.409
0.344	5.014	0.568	13.734	1.315	52.132
0.345	5.242	0.578	11.040	1.318	54.753
0.345	5.113	0.585	10.971	1.347	52.620
0.345	5.158	0.593	13.008	1.382	63.578
0.345	5.110	0.633	15.596	1.441	59.547
0.346	5.081	0.645	16.434	1.451	67.664
0.346	5.222	0.655	17.700	1.553	82.318
0.346	5.144	0.664	14.018	1.620	77.714
0.347	5.766	0.668	17.221	1.626	85.134
0.349	4.939	0.670	15.775	1.639	82.327
0.352	5.284	0.679	18.618	1.667	76.353
0.358	4.621	0.679	18.618	1.671	103.684
0.359	4.572	0.684	14.113	1.743	106.195
0.379	6.066	0.685	18.418	1.775	122.387

Table E.2: Gauged data for Case Studies 1, 5 and 7 (continued)

Stage (m)	Discharge (m^3/s)	Stage (m)	Discharge (m^3/s)	Stage (m)	Discharge (m^3/s)
1.831	113.948	2.268	126.561	2.932	126.709
1.849	144.172	2.288	150.113	3.094	130.488
2.031	114.762	2.424	146.244	3.467	164.352
2.121	123.604	2.579	152.841	3.615	209.485
2.211	149.182	2.763	135.715		

Table E.3: Gauged data for Case Study 6

Spot flow measurements - Summary Report

Case study 6							30.88
Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
Highest flows (> 10 cumecs)							
30/05/79			18.930	1636	1.03		31.91
13/12/00	11:29	12:26	16.347	1412	0.96		31.84
13/12/00	12:29	13:31	16.038	1386	0.95		31.83
13/12/00	10:25	11:28	15.903	1374	0.95		31.83
13/12/00	09:21	10:21	15.662	1353	0.94		31.82
08/12/00	10:43	11:55	12.166	1051	0.82		31.70
08/12/00	11:57	13:10	12.076	1043	0.81		31.69
08/12/00	15:03	15:59	11.981	1035	0.81		31.69
31/10/00	14:50	16:18	10.298	890	0.75		31.63
Gaugings in date order (1976 to 2002)							
21/07/76			0.317	27	0.05		30.93
29/07/76			1.077	93	0.16		31.04
30/07/76			0.404	35	0.05		30.93
04/08/76			0.398	34	0.05		30.93
19/08/76			0.281	24	0.04		30.92
25/08/76			0.350	30	0.03		30.91
15/09/76			0.344	30	0.04		30.92
21/09/76			0.336	29	0.04		30.92
06/10/76			1.024	88	0.17		31.05
13/10/76			1.771	153	0.27		31.15
20/10/76			2.505	216	0.33		31.21
03/11/76			2.531	219	0.32		31.20
10/11/76			2.709	234	0.35		31.23
17/11/76			2.484	215	0.34		31.22
23/11/76			2.084	180	0.32		31.20
01/12/76			3.498	302	0.40		31.28
08/12/76			3.016	261	0.37		31.25
15/12/76			2.741	237	0.37		31.25
19/05/77			2.996	259	0.37		31.25
31/05/77			2.118	183	0.30		31.18
16/06/77			3.821	330	0.42		31.30
21/06/77			2.775	240	0.34		31.22
11/07/77			2.206	191	0.29		31.17
14/07/77			1.844	159	0.27		31.15
11/08/77			1.331	115	0.20		31.08
17/08/77			6.786	586	0.61		31.49
17/08/77			6.687	578	0.60		31.48
17/08/77			6.486	560	0.60		31.48
27/09/77			1.522	132	0.21		31.09
29/09/77			1.518	131	0.19		31.07
19/10/77			1.068	92	0.19		31.07
23/11/77			3.099	268	0.37		31.25
30/11/77			2.820	244	0.36		31.24
08/12/77			3.489	301	0.38		31.26
17/01/78			4.464	386	0.45		31.33
30/01/78			6.009	519	0.54		31.42
09/02/78			5.372	464	0.54		31.42
15/02/78			4.766	412	0.50		31.38
28/02/78			4.911	424	0.49		31.37
09/03/78			4.707	407	0.49		31.37
16/03/78			5.151	445	0.51		31.39
05/04/78			4.441	384	0.46		31.34
18/04/78			3.980	344	0.44		31.32
15/05/78			2.604	225	0.35		31.23
14/06/78			1.313	113	0.25		31.13
29/06/78			1.407	122	0.22		31.10
06/07/78			1.154	100	0.19		31.07
13/07/78			1.102	95	0.18		31.06

Table E.3: Gauged data for Case Study 6 (continued)

Spot flow measurements - Summary Report

Case study site							30.88
Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
10/08/78			2.737	236	0.36		31.24
14/08/78			1.644	142	0.23		31.11
17/08/78			1.403	121	0.23		31.11
14/09/78			1.229	106	0.20		31.08
21/09/78			1.043	90	0.18		31.06
28/09/78			1.098	95	0.19		31.07
11/10/78			0.808	70	0.15		31.03
26/10/78			0.745	64	0.12		31.00
14/12/78			1.655	143	0.27		31.15
09/02/79			5.150	445	0.49		31.37
22/02/79			4.574	395	0.46		31.34
26/03/79			6.690	578	0.59		31.47
28/03/79			6.852	592	0.62		31.50
12/04/79			3.792	328	0.50		31.38
12/04/79			4.537	392	0.49		31.37
24/05/79			3.877	335	0.44		31.32
24/05/79			3.573	309	0.43		31.31
30/05/79			18.930	1636	1.03		31.91
19/07/79			2.062	178	0.29		31.17
26/07/79			1.948	168	0.28		31.16
02/08/79			1.748	151	0.27		31.15
08/08/79			1.446	125	0.25		31.13
15/08/79			1.475	127	0.24		31.12
30/08/79			1.220	105	0.22		31.10
12/09/79			1.049	91	0.17		31.05
27/09/79			0.958	83	0.15		31.03
11/10/79			0.875	76	0.16		31.04
08/11/79			0.922	80	0.14		31.02
30/11/79			0.940	81	0.14		31.02
18/12/79			2.784	241	0.37		31.25
03/01/80			5.385	465	0.51		31.39
10/01/80			4.592	397	0.46		31.34
24/01/80			4.115	356	0.46		31.34
31/01/80			3.880	335	0.43		31.31
01/02/80			4.561	394	0.48		31.36
07/02/80			4.917	425	0.51		31.39
06/03/80			3.965	343	0.47		31.35
20/03/80			5.080	439	0.51		31.39
02/04/80			5.611	485	0.54		31.42
28/05/80			1.720	149	0.26		31.14
05/06/80			1.530	132	0.26		31.14
11/06/80			1.330	115	0.28		31.16
09/07/80			1.290	111	0.26		31.14
16/07/80			0.956	83	0.25		31.13
24/07/80			1.030	89	0.18		31.06
31/07/80			1.210	105	0.20		31.08
13/08/80			1.370	118	0.20		31.08
28/08/80			0.860	74	0.14		31.02
04/09/80			0.650	56	0.15		31.03
10/09/80			0.820	71	0.15		31.03
17/09/80			1.100	95	0.17		31.05
08/10/80			1.200	104	0.19		31.07
22/10/80			3.090	267	0.36		31.24
29/10/80			2.550	220	0.34		31.22
05/11/80			1.780	154	0.28		31.16
26/11/80			3.350	289	0.43		31.31
03/12/80			2.760	238	0.36		31.24
07/01/81			3.470	300	0.41		31.29
21/01/81			3.300	285	0.39		31.27
18/02/81			1.830	158	0.28		31.16
11/03/81			6.450	557	0.59		31.47
08/04/81			3.590	310	0.43		31.31
21/05/81			3.010	260	0.47		31.35

Table E.3: Gauged data for Case Study 6 (continued)

Spot flow measurements - Summary Report

Case study site **30.88**

Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
08/07/81			1.970	170	0.39		31.27
29/09/81			1.670	144	0.25		31.13
22/10/81			2.850	246	0.35		31.23
05/11/81			2.620	226	0.36		31.24
12/11/81			2.820	244	0.38		31.26
03/12/81			2.890	250	0.39		31.27
09/12/81			3.040	263	0.37		31.25
06/01/82			6.290	543	0.58		31.46
20/01/82			7.030	607	0.61		31.49
10/02/82			4.100	354	0.45		31.33
11/03/82			6.560	567	0.57		31.45
17/03/82			6.200	536	0.61		31.49
13/05/82			1.740	150	0.29		31.17
03/06/82			1.510	130	0.24		31.12
16/06/82			1.150	99	0.23		31.11
02/07/82			1.128	97	0.20		31.08
08/07/82			1.094	95	0.19		31.07
13/08/82			0.968	84	0.16		31.04
09/09/82			0.687	59	0.14		31.02
16/09/82			0.635	55	0.11		30.99
14/10/82			1.220	105	0.19		31.07
28/10/82			2.550	220	0.33		31.21
15/12/82			6.690	578	0.54		31.42
06/01/83			5.960	515	0.54		31.42
13/01/83			5.040	435	0.52		31.40
20/01/83			4.050	350	0.47		31.35
02/02/83			4.250	367	0.45		31.33
17/02/83			3.900	337	0.40		31.28
24/02/83			2.380	206	0.33		31.21
10/03/83			2.040	176	0.30		31.18
21/04/83			4.280	370	0.44		31.32
27/04/83			4.230	365	0.44		31.32
12/05/83			6.900	596	0.59		31.47
01/06/83			7.300	631	0.62		31.50
08/06/83			4.790	414	0.49		31.37
15/06/83			3.480	301	0.43		31.31
21/07/83			1.520	131	0.26		31.14
27/07/83			3.900	337	0.42		31.30
11/08/83			1.470	127	0.24		31.12
19/08/83			1.280	111	0.22		31.10
08/09/83			1.600	138	0.24		31.12
14/09/83			1.200	104	0.20		31.08
14/10/83			1.060	92	0.20		31.08
20/10/83			1.370	118	0.20		31.08
04/11/83			1.290	111	0.20		31.08
10/11/83			1.330	115	0.20		31.08
22/11/83			1.090	94	0.19		31.07
22/12/83			3.600	311	0.41		31.29
05/01/84			3.260	282	0.41		31.29
24/01/84			4.630	400	0.46		31.34
29/03/84			2.650	229	0.34		31.22
31/05/84			1.520	131	0.28		31.16
22/06/84			1.510	130	0.22		31.10
12/07/84			1.099	95	0.16		31.04
01/08/84			0.844	73	0.17		31.05
31/08/84			0.668	58	0.12		31.00
07/09/84			0.643	56	0.11		30.99
17/09/84			0.738	64	0.13		31.01
30/07/85			2.839	245	0.44		31.32
29/07/86			1.350	117	0.33		31.21
29/07/86			1.510	130	0.34		31.22
22/05/87			2.069	179	0.33		31.21
12/06/87			1.854	160	0.31		31.19

Table E.3: Gauged data for Case Study 6 (continued)

Spot flow measurements - Summary Report

Case study site							**30.88**
Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
07/07/87			1.600	138	0.31		31.19
14/08/87			1.129	98	0.22		31.10
28/09/87			0.889	77	0.16		31.04
18/12/87			3.139	271	0.37		31.25
16/05/88			1.708	148	0.28		31.16
06/07/89	09:50	10:40	1.145	99	0.18	WEEDY SECTION.	31.06
						WEIR NOW CLEARED	
						ALL FLOW WITHIN NOTCH	
27/07/89	14:30	15:05	0.932	80	0.17	GAUGED BY WADING	31.05
						20M D/S OF FOOTBRIDGE	
						BELOW WEIR	
						GOOD SECTION TO GAUGE	
08/08/89	15:00	15:45	0.850	73	0.15		31.03
12/09/89	08:50	09:15	0.716	62	0.14	D/S OF WEIR	31.02
28/09/89	11:15	11:45	0.703	61	0.13	D/S WEIR	31.01
11/10/89	11:45	12:30	0.615	53	0.14		31.02
05/12/89	09:15	09:40	1.455	126	0.26	200M D/S WEIR	31.14
01/03/90	11:00	11:30	5.102	441	0.51		31.39
08/03/90	10:15	11:00	4.484	387	0.46		31.34
15/03/90	09:20	10:00	3.771	326	0.43		31.31
30/04/90	12:30	13:00	1.885	163	0.28	250M D/S G180WEIR	31.16
14/06/90	11:50	12:15	1.007	87	0.15	STAGE FELL 0.015M .	31.03
						IN 2HRS.BEFORE GAUGING	
						NO APPARENT REASON	
						FOR FALL	
03/07/90	10:40	11:05	0.862	74	0.14	D/S WEIR	31.02
12/07/90	11:00	11:30	0.861	74	0.14		31.02
19/07/90	11:15	11:45	0.884	76	0.12		31.00
03/08/90	10:00	10:30	0.635	55	0.11		30.99
05/08/90	05:50	06:20	0.713	62	0.10	D/S WEIR	30.98
06/09/90	10:45	11:15	0.587	51	0.08	150M D/S WEIR	30.96
11/09/90	06:25	06:50	0.510	44	0.08	150M D/S WEIR	30.96
14/09/90	10:10	10:30	0.563	49	0.08	150M D/S WEIR	30.96
10/10/90	10:20	10:45	0.520	45	0.07	U/S CABLEWAY	30.95
25/04/91	10:30	11:00	2.213	191	0.31	WEED ON WEIR	31.19
						MAKING ACTUAL LEVEL	
						POSSIBLY LESS THAN	
						0.313M	
06/06/91	09:45	10:10	1.534	133	0.23	D/S WEIR	31.11
03/07/91	13:50	14:15	1.461	126	0.22	D/S WEIR	31.10
29/08/91	12:25	12:55	0.919	79	0.13	150M D/S WEIR	31.01
						LEVEL FELL ABOUT 30MM	
						AFTER WEED CLEARED	
						FROM WEIR	
17/09/91	06:30	07:00	0.769	66	0.11	LOW FLOW SURVEY	30.99
						200M D/S WEIR	
14/10/91	12:45	13:20	4.524	391	0.12	GAUGED D/S OF WEIR	31.00
15/11/91	11:35	12:10	1.879	162	0.25	EMG USING	31.13
						VALEPORT	
04/03/92	11:00	11:30	1.943	168	0.28	D/S WEIR	31.16
27/03/92	13:15	13:45	1.630	141	0.25	D/S WEIR	31.13
27/03/92	12:00	12:30	1.626	140	0.25	D.S.WEIR	31.13
						EM GAUGING	
09/06/92	10:30	11:30	1.369	118	0.19	GAUGED D/S OF WEIR	31.07
20/08/92	09:50	10:15	0.816	70	0.12	D/S WEIR	31.00
02/10/92	10:15	10:45	1.055	91	0.16	D/S WEIR	31.04
26/02/93	13:15	14:15	2.877	249	0.35	SG.0.360 & 0.350	31.23
02/03/93	12:30	13:30	2.804	242	0.35		31.23
26/03/93	13:40	14:25	2.049	177	0.29		31.17
15/04/93	10:45	11:20	2.409	208	0.32	SG. 0.325	31.20
						WEED ON CREST	
						TOO DEEP TO CLEAR	
						SAFELY	

Table E.3: Gauged data for Case Study 6 (continued)

Spot flow measurements - Summary Report

Case study site							30.88
Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
04/05/93	15:30	16:00	2.292	198	0.30		31.18
12/05/93	09:00	09:50	2.023	175	0.29	CREST OF WEIR VERY WEEDY. WINTER GROWTH NOT REMOVED YET	31.17
17/05/93	11:00	11:45	2.054	177	0.26	GAUGED D/S OF WEIR	31.14
09/06/93	10:15	10:45	1.850	160	0.26	D/S WEIR	31.14
22/06/93	13:30	14:00	1.686	146	0.29		31.17
30/07/93	11:55	12:20	1.563	135	0.22		31.10
26/08/93	09:45	10:10	1.116	96	0.17	150M D/S WEIR	31.05
13/01/94	13:25	15:20	8.966	775	0.67	GAUGED AT CABLEWAY	31.55
18/01/94	11:10	12:30	7.876	680	0.63		31.51
18/01/94	13:00	14:15	7.797	674	0.63		31.51
18/01/94	09:45	11:00	8.170	706	0.63		31.51
02/02/94	11:10	12:15	4.376	378	0.46	SG.0.460	31.34
02/02/94	10:05	11:05	4.500	389	0.46	SG.0.460	31.34
02/03/94	14:10	15:00	5.119	442	0.49	SG 0.495	31.37
19/05/94	12:15	13:00	2.282	197	0.30		31.18
23/05/94	11:45	12:30	2.346	203	0.31		31.19
24/06/94	10:30	10:55	2.173	188	0.28	GAUGED D/S OF WEIR	31.16
16/08/94	14:00	14:30	0.903	78	0.17		31.05
21/09/94	13:20	13:50	1.106	96	0.17		31.05
10/02/95	13:15	14:10	6.605	571	0.57		31.45
10/02/95	11:40	12:35	6.536	565	0.57		31.45
31/03/95	12:15	13:15	3.340	289	0.40	CALIBRATION GAUGING	31.28
31/03/95	13:15	14:05	3.322	287	0.40	CALIBRATION GAUGING	31.28
05/07/95	12:15	12:45	0.808	70	0.19		31.07
18/07/95	08:00	08:30	0.815	70	0.14		31.02
18/07/95	08:30	09:00	0.808	70	0.14		31.02
03/08/95	13:15	13:45	0.775	67	0.12	LOW FLOW SURVEY	31.00
11/08/95	12:15	12:45	0.771	67	0.11		30.99
14/09/95	09:15	09:45	0.676	58	0.11		30.99
23/02/96	12:50	14:00	3.107	268	0.38	SOME WEED GROWTH 0N BED	31.26
08/03/96	13:30	14:30	2.874	248	0.37	SG 0.37 WEED GROWTH 0N BED SOME TRAILING WEED OVER WEIR CREST	31.25
14/05/96	14:45	16:00	1.787	154	0.27		31.15
14/05/96	13:30	14:45	1.915	165	0.28		31.16
12/06/96	10:15	11:45	1.764	152	0.25		31.13
10/09/96	09:55	10:10	0.775	67	0.11		30.99
10/09/96	09:25	09:50	0.747	65	0.11		30.99
01/10/96	13:15	13:45	0.663	57	0.12		31.00
07/11/96	14:15	15:00	1.026	89	0.14		31.02
07/11/96	15:05	15:40	0.877	76	0.14		31.02
17/12/96	11:00	11:35	1.416	122	0.23		31.11
17/12/96	10:00	10:50	1.416	122	0.23		31.11
17/01/97	08:25	09:10	1.441	124	0.22	LOW FLOW SURVEY	31.10
13/03/97	14:30	15:45	3.298	285	0.35		31.23
13/03/97	14:30	15:45	3.299	285	0.35		31.23
17/06/97	13:10	14:00	1.070	92	0.17		31.05
30/06/97	14:40	15:40	1.189	103	0.19		31.07
14/08/97	12:40	13:30	0.854	74	0.12		31.00
16/09/97	10:35	11:15	0.896	77	0.14		31.02
19/11/97	14:40	15:45	1.571	136	0.27	RAPIDLY FALLING STAGE	31.15
09/01/98	14:15	16:05	8.717	753	0.67		31.55
29/01/98	15:00	16:05	4.736	409	0.46		31.34
05/02/98	13:30	14:50	3.994	345	0.40		31.28

Table E.3: Gauged data for Case Study 6 (continued)

Spot flow measurements - Summary Report

Case study site							30.88
Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
05/02/98	15:00	15:40	3.566	308	0.41		31.29
25/02/98	14:45	15:20	2.502	216	0.33		31.21
25/02/98	15:20	15:55	2.636	228	0.33		31.21
25/02/98	14:45	15:20	2.530	219	0.33		31.21
29/03/98	13:45	15:00	2.974	257	0.37		31.25
29/03/98	15:00	16:00	2.976	257	0.37		31.25
01/05/98	08:50	09:50	3.783	327	0.42		31.30
25/06/98	15:20	15:55	2.428	210	0.33		31.21
07/08/98	12:10	13:35	1.168	101	0.14		31.02
11/09/98	13:15	14:00	1.375	119	0.18	HEAVY RAINSTORM AT 13:35 FOR 5-10 MINUTES	31.06
25/09/98	14:00	14:50	0.827	71	0.13		31.01
08/10/98	14:00	14:20	0.917	79			30.88
01/12/98	12:15	13:30	3.281	283	0.37		31.25
24/02/99	14:30	15:15	3.734	323	0.41		31.29
24/02/99	15:20	16:00	3.657	316	0.41		31.29
05/03/99	15:00	16:10	4.437	383	0.47		31.35
23/03/99	13:50	14:30	3.449	298	0.41	WEEDS IN CHANNEL FROM 7.5 TO 8.5 METRES AFFECTING MOVEMENT OF IMPELLER	31.29
23/03/99	14:35	15:20	3.383	292	0.41	WEEDS IN CHANNEL FROM 7.5 TO 8.5 METRES AFFECTING MOVEMENT OF IMPELLER	31.29
29/03/99	13:15	14:30	3.308	286	0.41	WEEDS IN CHANNEL FROM 7.5 TO 8.5 METRES OBSTRUCTING MOVEMENT OF IMPELLER	31.29
02/06/99	11:15	12:30	3.630	314	0.44		31.32
16/06/99	14:00	14:45	3.170	274	0.38		31.26
16/06/99	15:00	15:55	3.210	277	0.38		31.26
20/06/99	08:55	10:00	2.820	244	0.36		31.24
13/12/99	16:10	17:15	3.629	314	0.43		31.31
17/02/00	13:35	14:25	2.899	250	0.37		31.25
21/02/00	14:30	15:20	2.856	247	0.37		31.25
17/03/00	10:43	11:49	3.274	283	0.40		31.28
23/03/00	13:32	16:17	2.950	255	0.38		31.26
23/03/00	11:43	16:21	2.750	238	0.38	WEED AFFECTED FLOW AT RIGHT BANK	31.26
19/04/00	14:14	15:21	6.805	588	0.60	WEED IN CHANNEL AT RIGHT BANK	31.48
21/06/00	08:26	09:22	2.677	231	0.34		31.22
14/07/00	09:10	10:37	1.959	169	0.28	CENTRE CHANNEL FLOW AFFECTED BY WEED	31.16
01/08/00	12:00	13:08	1.472	127	0.25	SOME WEED G353 REMOVED FROM CURRENT METER PATH	31.13
27/09/00	09:30	10:46	1.212	105	0.21		31.09
31/10/00	14:50	16:18	10.298	890	0.75		31.63
30/11/00	14:20	15:08	7.124	616	0.61		31.49
08/12/00	10:43	11:55	12.166	1051	0.82		31.70
08/12/00	15:03	15:59	11.981	1035	0.81		31.69
08/12/00	11:57	13:10	12.076	1043	0.81		31.69
13/12/00	12:29	13:31	16.038	1386	0.95	V.HIGH FALLING	31.83
13/12/00	11:29	12:26	16.347	1412	0.96	V.HIGH PEAKING	31.84
13/12/00	09:21	10:21	15.662	1353	0.94	V.HIGH RISING	31.82
13/12/00	10:25	11:28	15.903	1374	0.95	V.HIGH RISING	31.83
26/04/01	15:32	16:31	5.011	433	0.50	SOME WEED GROWTH NEAR RIGHT BANK	31.38

Table E.3: Gauged data for Case Study 6 (continued)

Spot flow measurements - Summary Report

Case study site							**30.88**
Date	T start	T end	cumecs	Ml/d	metres	Comment	mAOD
06/07/01	14:40	15:12	1.847	160	0.29		31.17
10/08/01	13:33	14:15	1.350	117	0.26		31.14
19/09/01	14:02	15:03	1.005	87	0.16		31.04
17/10/01	14:25	15:22	0.875	76	0.16		31.04
06/02/02	13:21	14:08	7.259	627	0.66		31.54